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Case study

Vibration measurement-based simple technique for damage detection of truss bridges: A case study

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ABSTRACT

The bridges experience increasing traffic volume and weight, deteriorating of components and large number of stress cycles. Therefore, assessment of the current condition of steel railway bridges becomes necessary. Most of the commonly available approaches for structural health monitoring are based on visual inspection and non-destructive testing methods. The visual inspection is unreliable as those depend on uncertainty behind inspectors and their experience. Also, the non-destructive testing methods are found to be expensive. Therefore, recent researches have noticed that dynamic modal parameters or vibration measurement-based structural health monitoring methods are economical and may also provide more realistic predictions to damage state of civil infrastructure. Therefore this paper proposes a simple technique to locate the damage region of railway truss bridges based on measured modal parameters. The technique is discussed with a case study. Initially paper describes the details of considered railway bridge. Then observations of visual inspection, material testing and in situ load testing are discussed under separate sections. Development of validated finite element model of the considered bridge is comprehensively discussed. Hence, variations of modal parameters versus position of the damage are plotted. These plots are considered as the main reference for locating the damage of the railway bridge in future periodical inspection by comparing the measured corresponding modal parameters. Finally the procedure of periodical vibration measurement and damage locating technique are clearly illustrated.

1. Introduction

Most of railway bridges in the world are near the end of their design lives and many of them exceeds 100 years of age [1,2]. Replacement of all these at once will be extremely expensive and practically impossible as there are large number of old bridges. As a result, in the past two decades, a significant amount of effort has been directed toward the development of structural health monitoring and non-destructive assessment methods to maintain these bridges more efficiently [2–12].

Structural appraisal has been receiving more attention from bridge engineers due to recent failures in bridges in both developed and developing countries such as collapse of the Inter-state 35 W bridge in Minneapolis, Minnesota in July 2007; the Hoan bridge failure in Milwau-kee, Wisconsin in 2000; partially collapse of Cosen bridge in Latchford, Canada in 2003 and etc. The detailed inspections of steel truss bridges in the word revealed damages such as cracks and fractures,

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severe deterioration due to corrosion of members, some of which already reached a complete loss of the cross section of the member [13–16]. Some of these damages are located in the regions, where it is difficult to access for visual inspections.

For damage detection of bridges, visual inspections are widely used. However, uncertainties of skills of inspectors and accessibility issues hinder damage detection based on the visual inspection of bridges for some part of the structure. This issues are often tackled from the probabilistic point of view, by the computation of the probability of failure or a reliability index at different stage of life to be compared to target reliability index [17–24]. Even though the nondestructive testing-based damage detection approaches are more accurate, those approaches are highly expensive and time consuming. Recently researches have noticed that dynamic model parameters or vibration measurements-based structural health monitoring techniques may provide more realistic predictions to damage state of steel structures [13,25]. This approach is mainly based on variation of model parameters (i.e. natural frequency, mode shapes and model damping) with structural integrity. Therefore, periodical model parameter measurements can be used to monitor structural condition or damage state. Since model parameter measurements can be inexpensively acquired, the approach could provide an inexpensive structural assessment technique [25,26]. Eventhough number of studies have been done on this area, vibration measurement-based detection of damage or deterioration due to the complete loss of cross section, has not been properly discussed for railway bridges.

To overcome the above problem to some extent, this paper discusses a vibration measurement based simple technique to locate the damage or deteriorated region for detailed inspection and quantification of damage. The scope of study is limited to the steel truss bridges. The concept of this damage locating technique is change of model parameters (i.e. natural frequency and mode shapes) due to presence of damage or deterioration. The damages or deterioration due to fully section loss of members, which are difficult to access for visual inspections, can be more precisely located by this proposed technique. This approach provides a warning of damage or deterioration before it is too late to attend for necessary detailed inspection or maintenance. The paper describes the proposed technique with a comprehensive case study as below.

2. Considered bridge

The selected bridge is a railway bridge spanning 160 m as shown in Fig. 1. It is a six span-riveted bridge with double lane rail tracks having warren type semi through trusses, supported on cylindrical piers. The bridge deck is made of wrought iron and the piers are made of cast iron casings with infilled concrete. The bridge was constructed in 1885. Details of trains carried by the bridge and their frequencies illustrate that the bridge is experienced variable amplitude fatigue loading.

3. Visual inspection

The condition survey revealed that some places of the bridge have been subjected to mild corrosion due to the absence of anti-corrosive coating (refer Fig. 1). No visual cracks were observed in any component of the super structure. In situ measurements of member sizes, connections and support bearings verified the fact that the existing drawings were applicable and only few significant variations were observed. The visual inspection was done in year 2001.

4. Material testing

The sampling of materials, specimen preparation and testing were carried out according to the ASTM standards. The chemical analyses as well as microscopic examinations lead to the conclusion that the bridge super structure material is wrought iron. The obtained values for elastic modulus, yield strength, ultimate strength in tension, fatigue strength and density are 195 GPa, 240 MPa, 383 MPa, 155 MPa and 7600 kg/m^3 respectively. The material testing was also done in year 2001.

5. Static and dynamic load testing

Static and dynamic load tests were performed in year 2001 to study the real behavior of the bridge under various load combinations. The in situ measurements were performed using the two heaviest railway engines (i.e. Type M8), each weighting 1120 kN, which is the heaviest rail traffic in current operation. The bridge was instrumented with strain gauges

Fig. 1. General views of the riveted railway bridge.

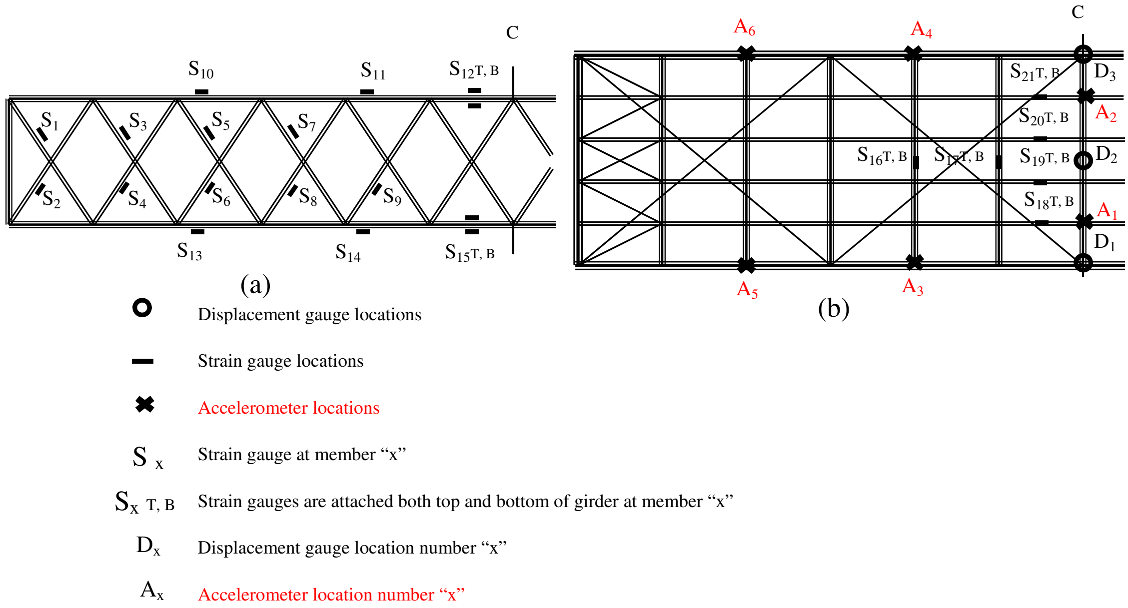

Fig. 2. Locations of the strain gauges and displacement gauges (a) main truss girder and (b) horizontal bridge deck.

placed at selected locations to measure normal strains [27–33]. In addition, the tri-axial vibrations were recorded at several locations using accelerometers. Displacement transducers were used to measures vertical deflection at three places around the mid-span area of the bridge. The measured locations are shown in Fig. 2.

To obtain the different type of load cases, which are critical to the bridge, the two test engines were placed together as well as moved under different speeds of 12, 19, 26, 29, 32, 34, 37, 39, 40, 44, 46, 47, 50 km/h respectively. The considered three static load combinations are defined as static load case (SLC) 1, 2 and 3 by considering criteria of maximum shear effect, maximum bending effect (maximum deflection) and maximum torsion effect to the bridge deck respectively. The loading positions corresponding to the mentioned three load cases are shown in Fig. 3. The criteria, which were considered for dynamic load combinations, basically illustrate the impact effect to the bridge with different levels of speed and traction force effect. Apart from the above mentioned formal field load testing, the bridge underwent a 2-day continuous field measurement program under present day actual traffic load. When the bridge is affected by maximum load due to the present day heaviest train passage, the obtained sample measurements are shown in Fig. 4. Finally the dynamic factors were obtained as 1.3, 1.4 and 1.4 for main truss girders, secondary cross girders and stringers respectively.

6. Development of validated finite element model

Then the bridge deck was analyzed using the finite element method (FEM) employed general-purpose package SAP 2000. A three-dimensional (3D) model (Fig. 5) of one complete middle span of the bridge was analyzed under actual loading to determine stresses in members and deflections, as well as variations of stresses under moving loads. The material properties recorded in Section 4 and calculated section properties were utilized for this analysis. The bridge deck was modeled with 3D frame elements and the riveted connections were assumed to be fully-fixed [4]. A dynamic analysis was conducted for each different load case discussed in Section 5, determining load vs. time histories.

The validation of FE model was done by comparing the results from analysis with those from field-tests as shown in Table 1. In this paper, further validation of the FE model is done by comparing the results of time history dynamic analysis with those from measured time histories during appraisal in year 2001 as shown in Fig. 4. These figures show that there are good agreements among analytical results of the FE model and the measurement of the actual bridge. Therefore, the considered 3D frame element model was defined as “validated FE model” which can reasonably represent the actual static and dynamic behavior of the bridge at year 2001.

7. Modal parameter variation with respect to damage

In this study, it is assumed that damage of the member represents fully section loss or through thickness crack of inaccessible members (i.e. difficult to access for visual inspections) such as main girder bottom chord and cross girders. It was also assumed that probability is very low to damage or deteriorate two or more members until complete loss of cross section in an interval of two periodical inspections. Therefore, one inaccessible member (i.e. main girder bottom chord or cross girder member) was removed from the FE model, which was considered as undamaged model in Section 6, to simulate

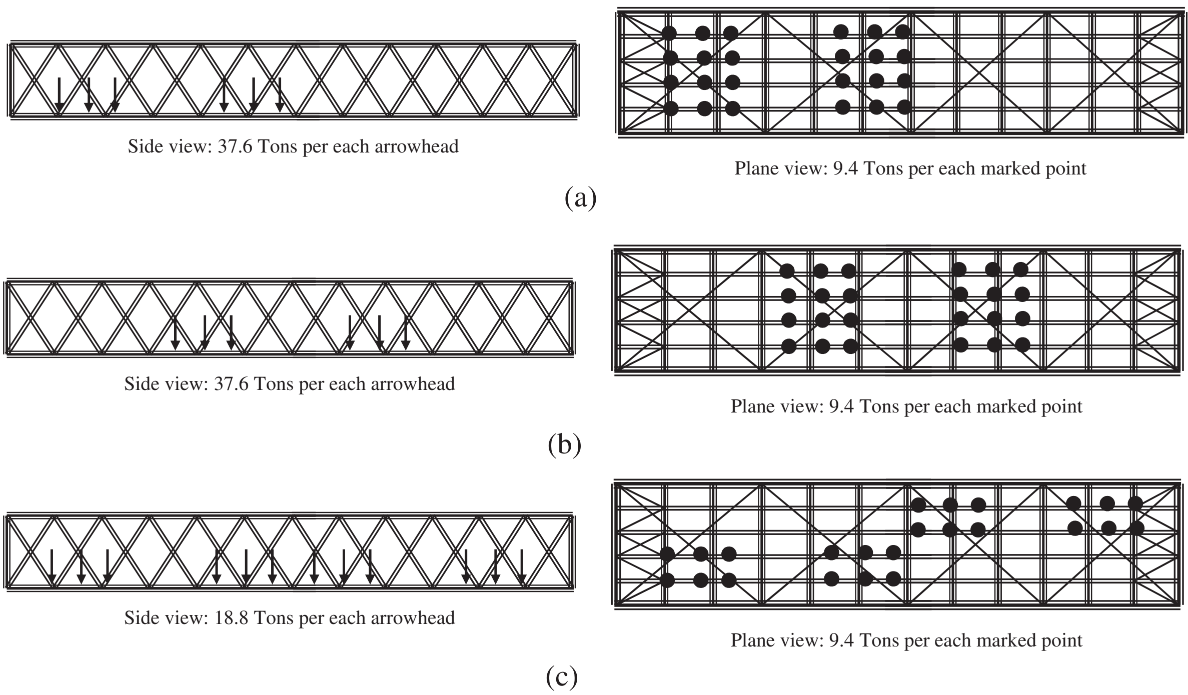

Fig. 3. Loading positions corresponding to three static load cases (a) SLC 1, (b) SLC 2 and (c) SLC 3.

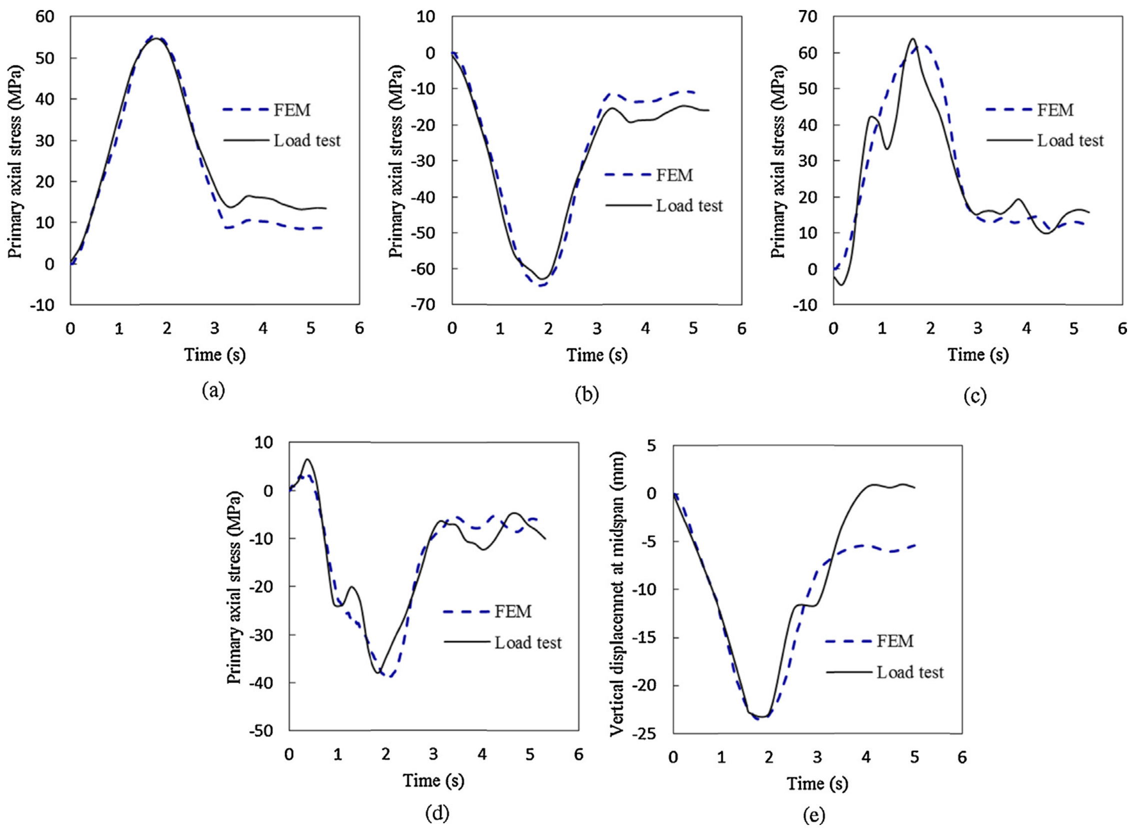

Fig. 4. Comparison of FEM time history analysis results with measured values by in situ load tests: (a) stresses at location $S_{15T,B}$ bottom chord of the main girder, (b) stresses at $S_{12T,B}$ top chord of the main girder, (c) stresses at S_5 diagonal tension members, (d) stresses at S_6 diagonal compression members and (e) vertical displacement at mid-span.

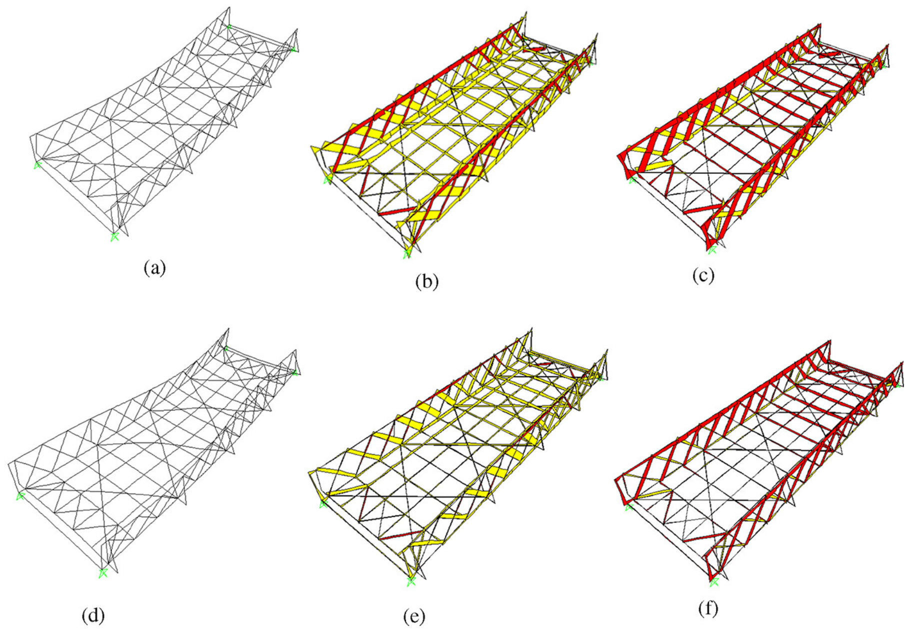

Fig. 5. The FE analysis results for moving train load: (a) vertical displacement when the train is in the middle of the bridge, (b) maximum stress taken over all stress points at each cross sections when train is in the middle of the bridge, (c) minimum stress taken over all stress points at each cross sections when train is in the middle of the bridge, (d) vertical displacement when the train just before leave the bridge, (e) maximum stress taken over all stress points at each cross sections when the train just before leave the bridge and (f) minimum stress taken over all stress points at each cross sections when the train just before leave the bridge. Yellow color: tensile stress. Red color: compressive stress

Table 1
Comparison of FE analytical results with load test results.

Static load case	Displacement (mm)			Stress (MPa)		
	Location of measurement	Load test	FEM	Location of measurement	Load test	FEM
SLC 1	D_1	19.4	21.0	S_6	−40.2	−40.6
				S_5	51.4	57.3
				$S_{15T,B}$	47.3	48.2
SLC 2	D_1	21.3	22.5	S_6	−37.8	−37.7
				S_5	44.5	43.6
				$S_{15T,B}$	53.5	53.9
SLC 3	D_1	–	19.1	S_6	−39.5	−39.9
				S_5	35.2	41.5
				$S_{15T,B}$	39.0	44.7

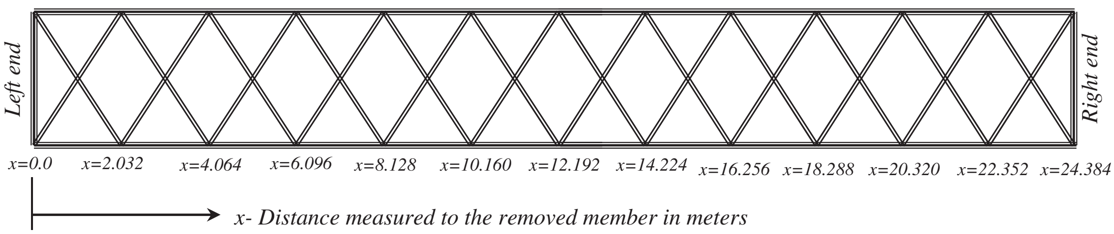

Fig. 6. Position of the removed inaccessible members.

the damage state of the bridge. Then the damaged FE model was analyzed to get the modal parameters of each mode of free vibration. In the second step, another inaccessible member was removed from the undamaged FE model to simulate next damage state of the bridge and the current damaged FE model was re-analyzed to get the modal parameters of each mode of free vibration. This procedure was repeated for all inaccessible members. Hence, type of the member, the location of the removed member, distance to the removed member from the left end of the bridge (refer Fig. 6) and observed values of modal parameters are recorded. Then variations of modal parameters versus position of the damage (i.e. distance to the

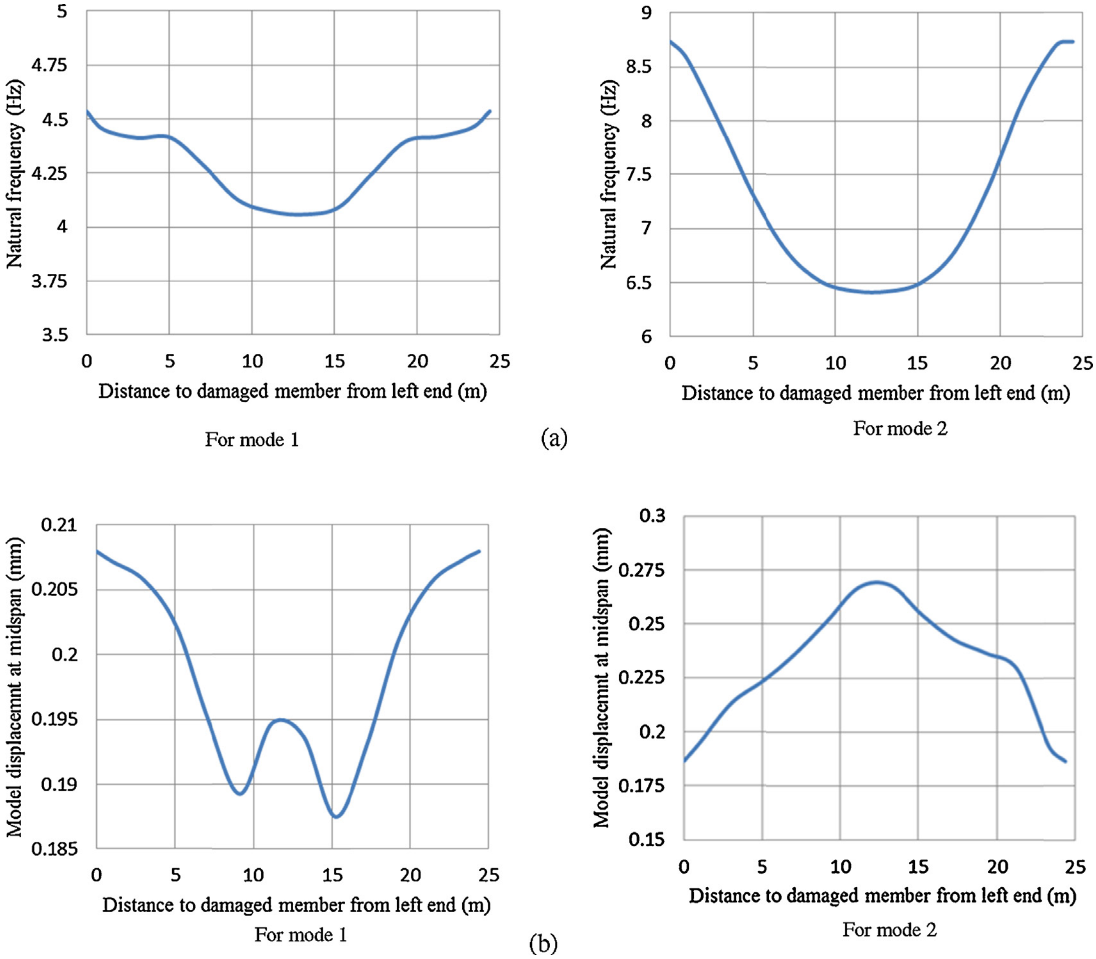

Fig. 7. (a) Variation of natural frequency versus distance to the damaged main girder bottom chord: (b) variation of mid span modal displacement versus distance to the damaged main girder bottom chord.

removed member from the left end of the bridge) were plotted as shown in Figs. 7 and 8 respectively for main girder bottom chords and cross girders.

At modal nodes (points of zero modal displacements), the stress is minimum for the particular mode of vibration. Hence, it can be seen that less change in a particular modal parameter when damage is located in members which are close to the modal node. However, the parameters in other modes of vibration can still be used to locate this damage of member [16]. Therefore, it is important to plot the variation of model parameters for few modes of vibrations. Finally, these plots were used as main reference for locating the damage of the railway bridge in future periodical inspection by comparing the measured corresponding model parameters.

8. Periodical measurement of vibration and locating damages

After 10 years later of previous inspection (i.e. 2011), bridge was instrumented with accelerometers placed at the locations shown in Fig. 2 to measure all three directions accelerations. In order to measure free vibration, accelerations were recorded after the trains had crossed the bridge. The measurements were taken for period of 12 h. Hence, natural periods of each mode were determined for each train passed the bridge. The modal analyses were conducted to obtain natural periods for each mode. Hence, the average values of corresponding natural frequencies were predicted as 4.72 Hz and 9.1 Hz respectively for mode 1 and 2. The mode displacements were obtained by fast Fourier transformation of measured accelerations in respective mode direction. The predicted average mode displacements at mid-span are 0.212 mm and 0.191 mm respectively for mode 1 and 2.

First the above obtained values of modal parameters (i.e. which were obtained during the field investigation on 2011) were compared with modal parameters of undamaged bridge (i.e. values 4.536 Hz, 8.73 Hz, 0.208 mm and 0.186 mm, which were obtained during the field investigation on 2001). As these values are different, it can be concluded that there may be a damage in the bridge. To trace approximate position of the damage, the above obtained values were compared with plots in

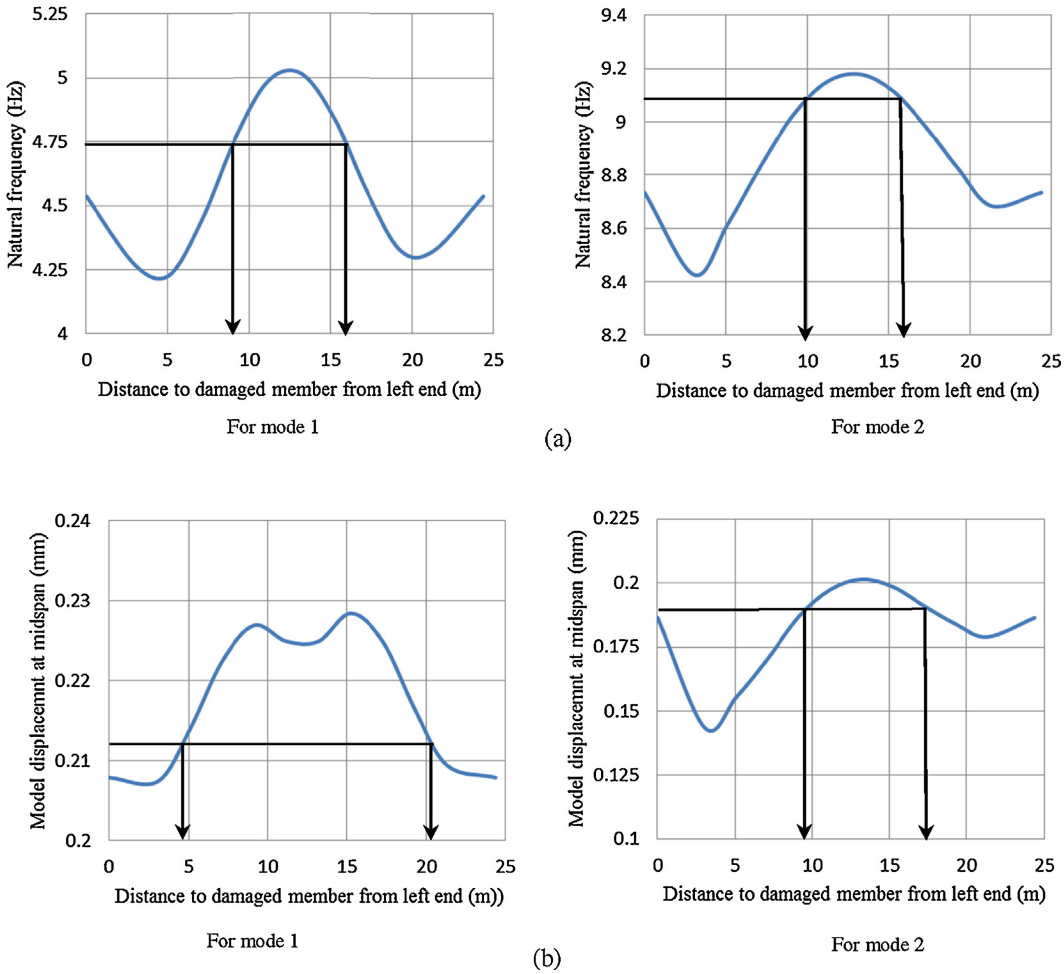

Fig. 8. (a) Variation of natural frequency versus distance to the damaged cross girder: (b) variation of mid span modal displacement versus distance to the damaged cross girder.

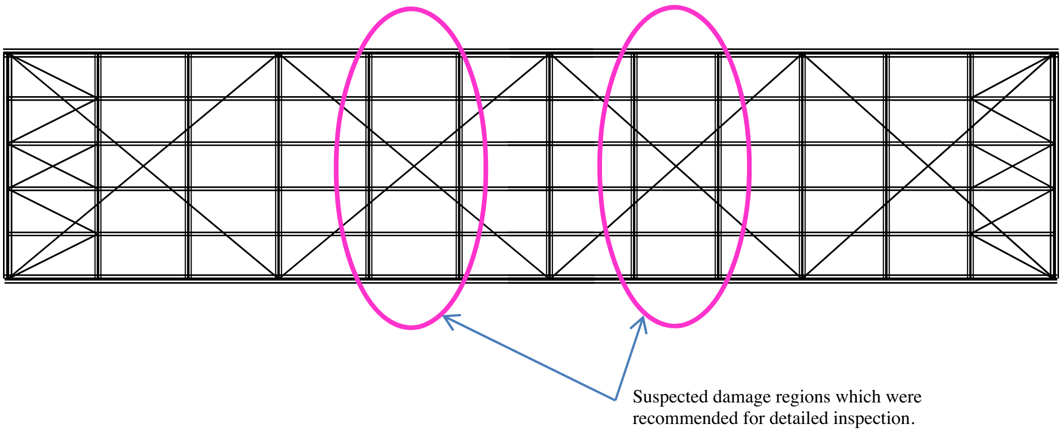

Fig. 9. Suspected regions of damage.

Figs. 7 and 8. When natural frequencies (i.e. 4.72 Hz and 9.1 Hz) are compared with Fig. 7(a), it can be seen that these values do not coincide with any of curves. When mid-span modal displacements (i.e. 0.212 mm and 0.191 mm) are compared with Fig. 7(b), it can be seen that only mode 2 value has just coincided. But distance to damage is close to zero and it means that there is no damage. Based on above comparison, it can be concluded that possibility of having damage in main girder bottom chord is very low.

Then the obtained values of modal parameters (i.e. 4.72 Hz, 9.1 Hz, 0.212 mm and 0.191 mm) were compared with Fig. 8 and it can be seen that every value coincides on respective curve as marked in Fig. 8. When the two graphs in Fig. 8(a) and mode 2 graphs in Fig. 8(b) are considered, it can be concluded that approximate distances to damage member from the left end of the bridge are 10 m and 16 m respectively. When looking at mode 1 graph in Fig. 8(b), it can be seen that the distances to damage members deviate a bit from above values. The uncertainty behind in situ measurements may affect for this deviation. Finally, it can be concluded that damage might locate in cross girders and somewhere around 10 m and 16 m from the left end of the bridge. Hence, the suspected regions of damage have been located closer to 4th and 5th cross girders from each end of the bridge as shown in Fig. 9.

Then detailed inspection was conducted. Finally, it revealed that loosening of rivets of 4th cross girder from downside support of the considered span of the bridge. Suspected reason for these loosing might be crevice corrosion and the fatigue. The loosening of the rivets causes to reduce the fixity at the cross girder to main truss girder joint and finally it hindered to contribute to overall stiffness of the bridge. The inspection was later expanded to investigate similar cross girders in other spans of the bridge as well.

9. Conclusions

A simple technique was proposed to locate the damage region of railway bridges based on measured model parameters. The case study shows the applicability of the introduced technique. The technique mainly depends on plots of variation of model parameter with respect to the position of damage which was obtained by validated FE model. The validated FE model, which represents more reasonably the actual static and dynamic behavior of the railway bridge at the time of validation, was obtained by comparing the measured responses with FE model given responses. The periodical modal parameter measurement has to be conducted and obtained measurement should be compared with above plots to locate the region of damage. As this periodical modal parameter measurement is based only on acceleration under usual moving train loads, it can be concluded that this measurement can be cheaply acquired and the proposed method provides an inexpensive damage locating technique. Influence of partially loss of member cross section for change of model parameters are recommended for future studies.

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