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VIBRATION MEASUREMENT-BASED SIMPLE TECHNIQUE FOR STRUCTURAL APPRAISAL OF STEEL BRIDGES

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Abstract. *The structures experience increasing traffic volume and weight, deteriorating of components and large number of stress cycles. Therefore, assessment of the current condition (i.e. Structural Health Monitoring) of steel railway bridges becomes necessary. Most of the commonly available approaches for Structural Health Monitoring are based 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 noticed that dynamic modal parameters or vibration measurements-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 methods and observations of visual inspection, material testing and in-situ load testing are discussed under separate sections. Then the development of validated FE model of the considered bridge is comprehensively discussed. Hence, variations of modal parameters versus position of the damage were 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 were clearly illustrated.*

1 INTRODUCTION

A most of railway bridges in the world (i.e. mainly in UK, parts of Europe and North America) 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 (e.g. it is estimated that there is more than 6000 in UK). As a result, in the past two decades, a significant amount of effort has been directed towards the development of structural health monitoring and non-destructive assessment methods to maintain these bridges more efficiently [2-12].

Structural appraisal has been received more attention from bridge engineers due to recent failures in bridges in both developed and developing countries such as collapse of the Interstate 35W bridge in Minneapolis, Minnesota in July 2007; the Hoan bridge failure in Milwaukee, 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 cracks and fractures, 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.

In health monitoring of bridges, visual inspections are widely used. However, uncertainties of skills of inspectors and above mentioned accessibility issues finally hindered the visual inspection based health monitoring of bridges for part of the structure. Even though the nondestructive testing-based damage detection approaches are more accurate, those approaches are highly expensive and time consuming. Recently researches noticed that dynamic model parameters or vibration measurements-based structural health monitoring techniques may provide more realistic predictions to damage state of steel structures [13,17]. This approach is mainly based on variation of model parameters (i.e. i.e. natural frequency, mode shapes and model damping) with structural integrity. Therefore, periodical model parameter measurements can be used to monitor structural condition. Since model parameter measurements can be cheaply acquired, the approach could provide an inexpensive structural assessment technique [17,18]. Even though large number of studies have been done on this area, vibration measurement-based detection of damage or deterioration due to more or equal 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 damaged 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, are more precisely locate by this proposed technique and it is the one of the main limitations of this approach. 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 bellow.

2 CONSIDERED BRIDGE

The selected bridge is one of the major railway bridges in Sri Lanka spanning 160 m (Figure 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 suffered from variable amplitude loading.

Figure 1: General views of the riveted railway bridge

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 (see Figure 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.

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.

5 STATIC AND DYNAMIC LOAD TESTING

Static and dynamic load tests were performed to study the real behavior of the bridge under various load combinations. The in-situ measurements were performed using two M8 engines, each weighting 1120 kN, which is the heaviest rail traffic in current operation. The bridge was instrumented with strain gauges placed at selected locations to measure normal strains. 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 Figure 2.

To obtain the different type of load combinations, which are critical to the bridge, the two test-engines were placed together as well as moved under different speeds. 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 Figure 3. The criteria, which were considered for dynamic load combinations, basically illustrate that 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 was subjected to a two days continuous field measurement program under present day actual traffic. When the bridge is affected by maximum load due to the present day heaviest train passage, the obtained sample measurements are shown in Figure 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.

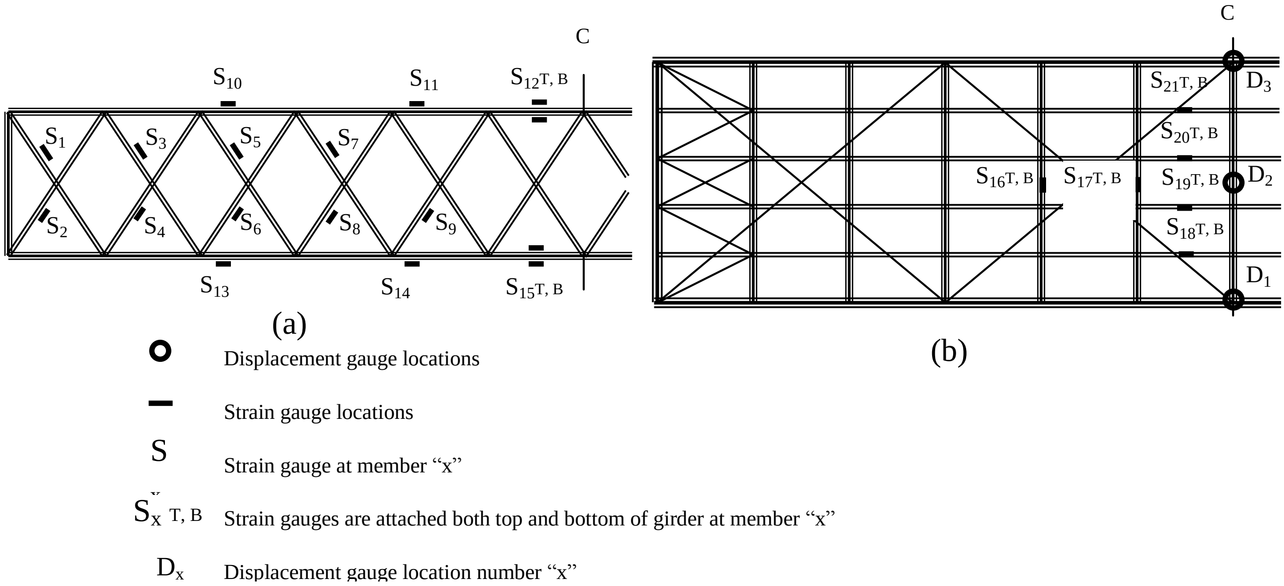


Figure 2. Locations of the strain gauges and displacement gauges (a) Main truss girder (b) Horizontal bridge deck

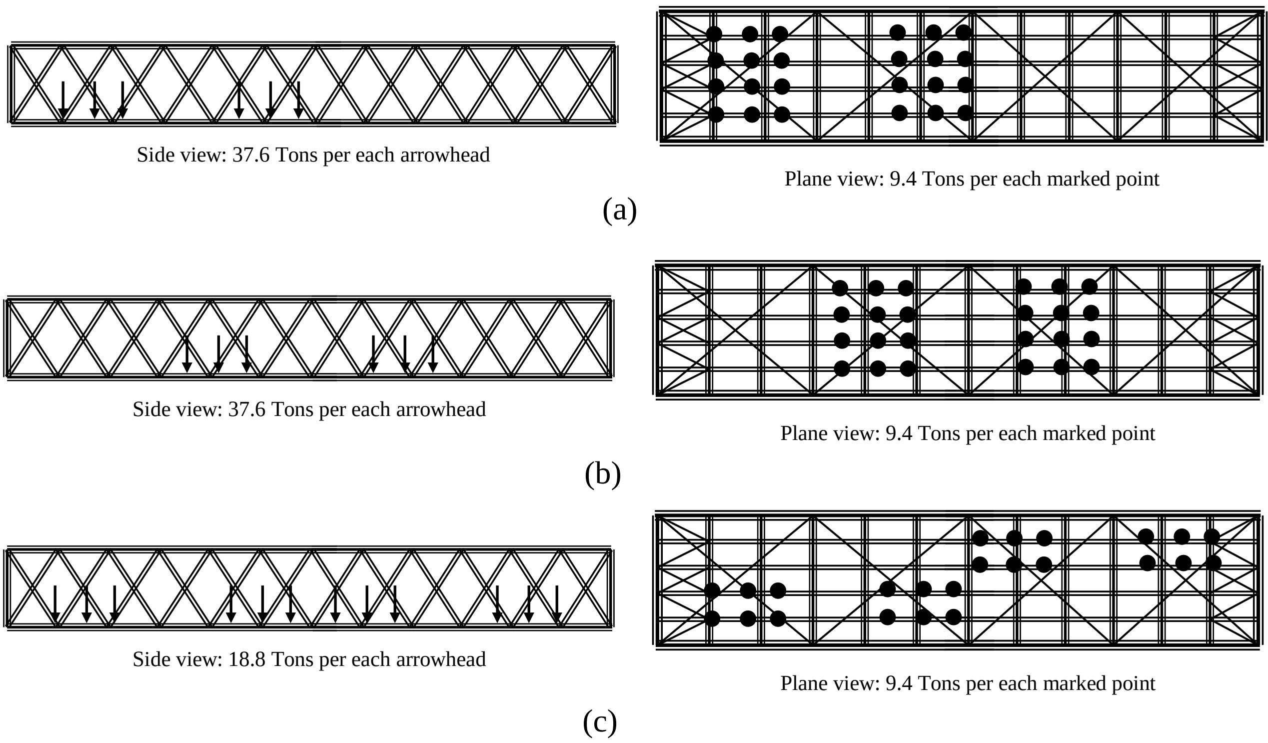


Figure 3. Loading positions corresponding to three static load cases (a) SLC 1 (b) SLC 2 (c) SLC 3

6 DEVELOPMENT OF VALIDATED FINITE ELEMENT MODEL

Then the bridge deck was analyzed using the finite element (FE) method employed general- purpose package SAP 2000. A three-dimensional (3D) model (Figure 5) of one complete middle span of the bridge was analysed 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 section properties calculated above were utilized for this FE analysis. The bridge deck was modelled with 3D frame elements and the riveted connections were assumed to be fully-fixed [4]. Moving load employed time history dynamic analysis was con-

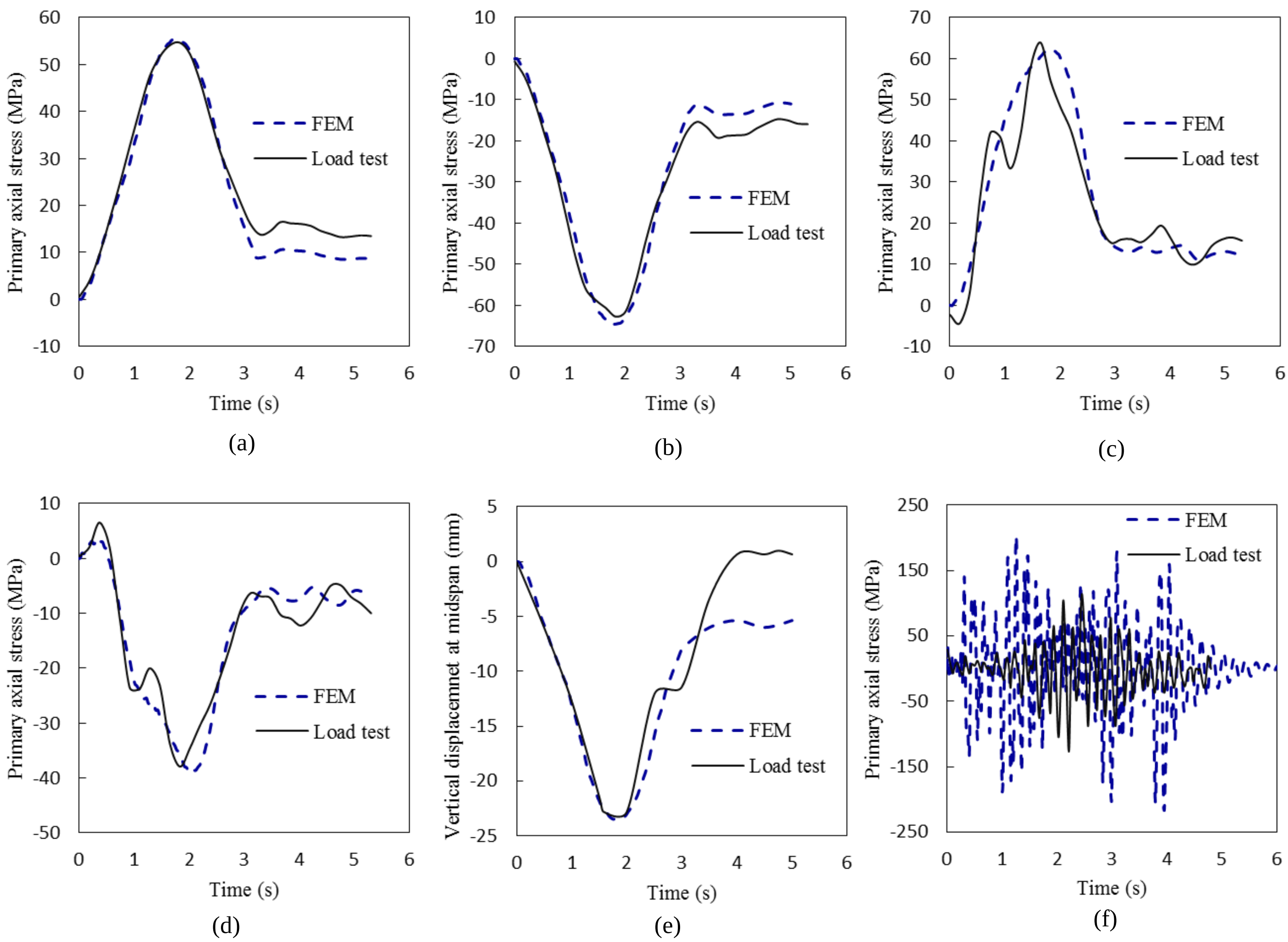


Figure 4. Comparison of FEM time history analysis results with field measurements: (a) Stresses at MT3,2 bottom chord of the main girder, (b) Stresses at MC3,2 top chord of the main girder, (c) Stresses at DT3 diagonal tension members, (d) Stresses at DC3 diagonal compression members, (e) Vertical displacemnt at midspan, (f) Vertical acceleration at midspan

ducted for each different load cases discussed in section 5. After appearing the general corrosion sign of each member of each location, the input values of sectional parameters were changed by calculated values of corroded section to obtain the load histories of the members.

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 Figure 4. These figures show that there is a good agreement 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 2001.

7 MODEL PARAMETER VARIATION WITH RESPECT TO DAMAGE

In this study, it is assumed that damage of the member represent 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

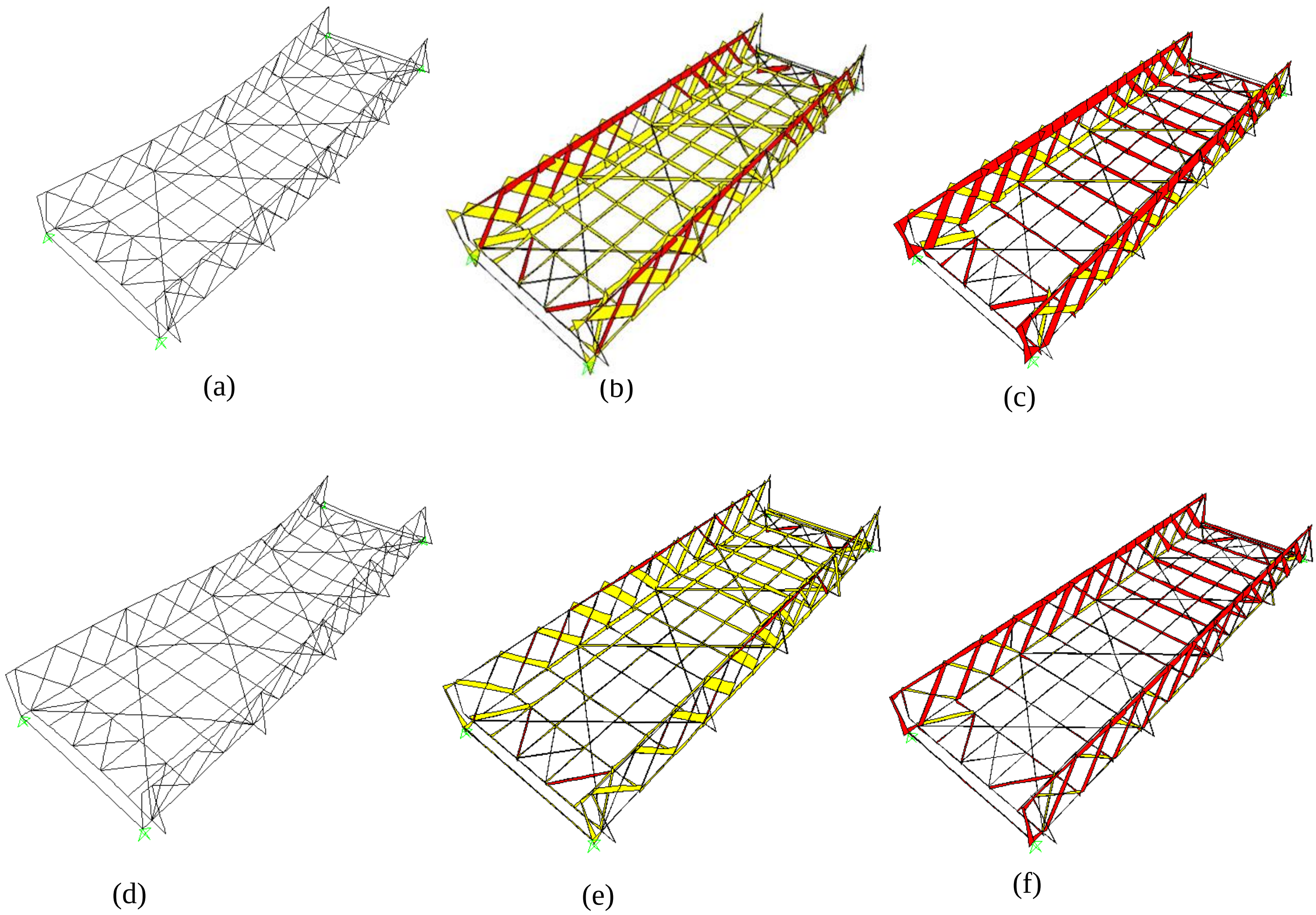


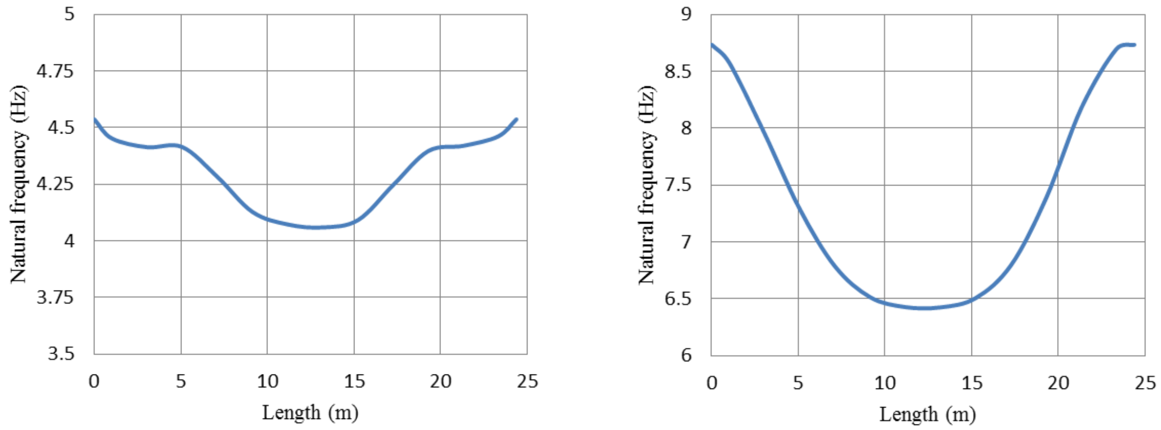
Figure 5. The FE analysis results for moving train load: (a) Vertical displacement when the train is in the middle of the bridge (b) Maximum stress taken over all stress points at each cross sections when train is in the middle of the bridge (c) Minimum stress taken over all stress points at each cross 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 (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
				$S_{15,}$T,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
				$S_{15,}$T,B	53.5	53.9
SLC 3	D_1	-	19.1	S_6	-39.5	-39.9
				S_5	35.2	41.5
				$S_{15,}$T,B	39.0	44.7

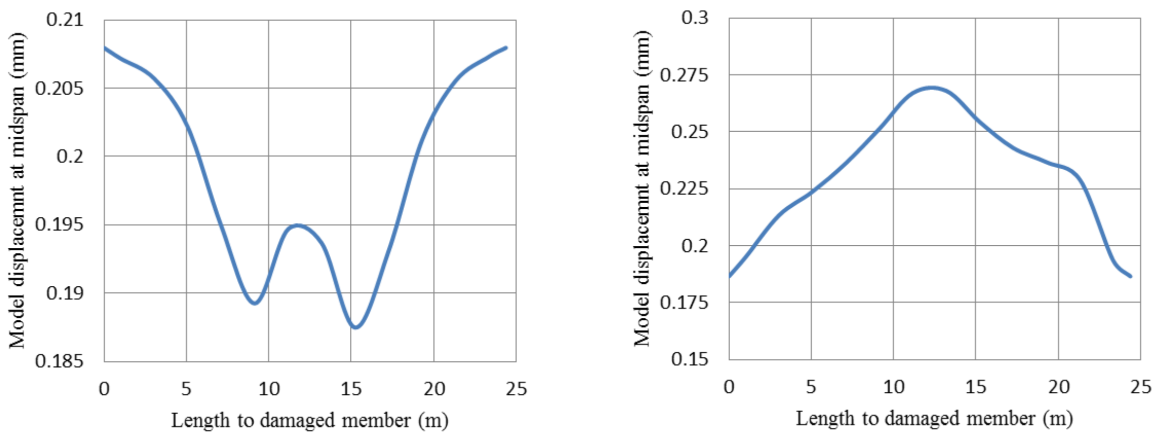
very low to damage or deteriorate two or more members until complete loss of cross section in an interval of two periodical inspections. Thus validated FE model obtained in section 6 was used to simply remove one member at a time and re-analyze to get the modal parameters of each mode of free vibration. Hence, the location, type of member and observed values of modal parameters are recorded and plot variations of modal parameters versus position of the damage as shown in Figure 6 -9.

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 parameter.



(a) (b)

Figure 6: Variation of natural frequency versus length to damaged member of main girder bottom chord: (a) mode 1, (b) mode 2

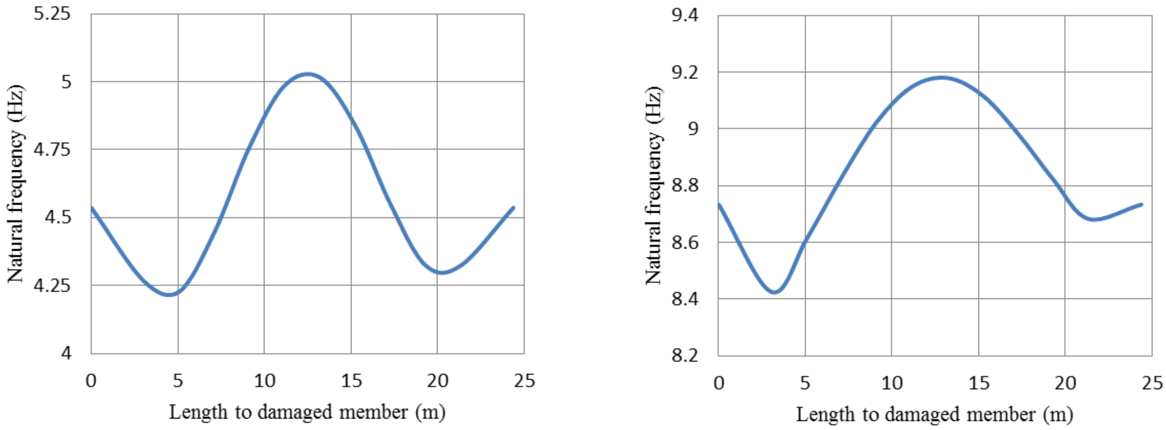


(a) (b)

Figure 7: Variation of mid span modal displacement versus length to damaged member of main girder bottom chord: (a) mode 1, (b) mode 2

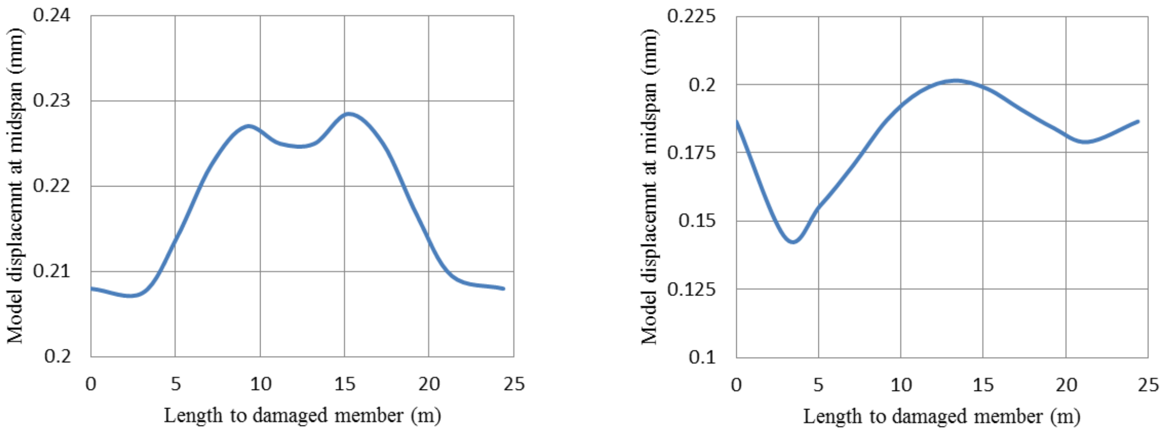
8 PERIODICAL VIBRATION MEASUREMENT AND LOCATING DAMAGES

After ten years later of previous inspection (i.e. 2011), bridge was instrumented with accelerometers placed at selected locations to measure all three directions accelerations. In order to



(a) (b)

Figure 8: Variation of natural frequency versus length to damaged member of cross girder: (a) mode 1, (b) mode 2



(a) (b)

Figure 9: Variation of mid span modal displacement versus length to damaged member of cross girder: (a) mode 1, (b) mode 2

measure free vibration, accelerations were recorded after the train had crossed the bridge. Hence natural periods of each mode were determined and corresponding natural frequencies were predicted as 4.72 Hz and 9.1Hz respectively for mode 1 and 2. The mode displacements were obtained by FFT transformation of measured accelerations. The predicted mode displacements at mid-span are 0.212mm and 0.191mm respectively for mode 1 and 2. Then these values compared with plots obtained in section 7. Hence, the suspected region of damage has been located in cross girders as circled in Figure 2(b).Then bridge inspectors stops the train operation for one night and detailed inspection was conducted. Finally its revealed that loosening of many of rivets finally hindered the contribution of cross girder number 16^{th} to the stiffness of the bridge. Suspected reason for these loosing might be the fatigue.

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 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 validated, was obtained by comparing the measured responses with FE model given responses. The periodical model parameter measurement has to be conducted and obtained measurement should be compared with above plots to locate the region of damage. As this periodical model 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 structural appraisal technique. Influence of partially loss of member cross section for change of model parameters are recommended for future studies.

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