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Development of non-destructive method of detecting steel bars corrosion in bridge decks

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Abstract. One of the most common defects in reinforced concrete bridge decks is corrosion of steel reinforcing bars. This invisible defect reduces the deck stiffness and affects the bridge's serviceability. Regular monitoring of the bridge is required to detect and control this type of damage and in turn, minimize repair costs. Because the corrosion is hidden within the deck, this type of damage cannot be easily detected by visual inspection and therefore, an alternative damage detection technique is required. This research develops a non-destructive method for detecting reinforcing bar corrosion. Experimental modal analysis, as a non-destructive testing technique, and finite element (FE) model updating are used in this method. The location and size of corrosion in the reinforcing bars is predicted by creating a finite element model of bridge deck and updating the model characteristics to match the experimental results. The practicality and applicability of the proposed method were evaluated by applying the new technique to a two spans bridge for monitoring steel bar corrosion. It was shown that the proposed method can predict the location and size of reinforcing bars corrosion with reasonable accuracy.

Keywords: concrete bridge; defect detection; model updating; modal analysis; corrosion

1. Introduction

One of the most common invisible defects in reinforced concrete bridges is reinforcing bar corrosion due to chloride attack. Corrosion of the reinforcing bars reduces the bending and torsional stiffness of the bridge deck (Mahini *et al*. 2008, Ronagh and Dux 2003, Ronagh *et al*. 2000). It affects more than 80% of the bridges over 5 years old to different degrees (Sadeghi 2004). Due to a high probability of the occurrence of this defect in concrete bridges, regular monitoring of the state of reinforcing bar corrosion can help to minimize the local and global damage and reduce maintenance or retrofitting costs. In fact, early detection of structural defects minimizes the cost of repair and maximizes safety.

Even though the visual inspection has been the most common method used in detecting damages in various types of structures, the complexity of recent structures reduces its efficiency (Teughels and De Roeck 2004). Moreover, the conventional non-destructive damage detection

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techniques implemented to detect occurrence of damage require a prior knowledge of the location of the affected area and its accessibility. Recently, a dynamics-based damage identification technique has been developed by which a global way to assess the structural state is provided. Several dynamics-based damage identification methods have been introduced in the literature by several researchers, including Ho and Ewins (2000), Kim *et al*. (2003), Wu and Li (2004, 2006), Jaishi *et al*. (2007), Ren and Chen (2010), Shiradhonkar and Shrikhande (2011) and Ribeiro *et al*. (2012). They have used changes in eigenfrequencies and mode shapes as indicators to detect damages in structures. Catbas and Aktan (2002), Bernal (2000) proposed the use of the dynamically measured flexibility matrix for damage identification. Furthermore, Brownjohn *et al*. (2001) described a finite element model updating technique for structural conditions assessments. Although the applications of the proposed damage detection techniques have been shown in conditions assessments of some structures, their practicability and efficiencies in some particular structural damages are questionable (Sadeghi 2004). It indicates a need for improvement of the proposed methods or development of new techniques for complicated damages such as steel bar corrosion. In this research, a new detection method was developed. Applicability and reliability of the new method were tested by applying the new technique to a two spans bridge for monitoring its steel bar corrosion.

2. Proposed detection technique

The proposed detection technique is based on obtaining a reasonable correlation between experimental and numerical modal properties and as a result of doing such a procedure, the location of the bars corrosions and their quantity are determined. It relies on the fact that the occurrence of damage in a structural system leads to changes in its dynamic properties such as eigenfrequencies and mode shapes. Over the last two decades, the use of such method, namely model updating technique, for the prediction of damages in other structural systems has been investigated. A flow diagram of the proposed technique is presented in Fig. 1. The flow diagram consists of three parts. They include: (1) generation of a bridge theoretical model using FEM for the undamaged bridge; (2) calibration of the developed model and; (3) comparison of the experimental results (eigenfrequencies and mode shapes) with those obtained by the numerical analyses. The comparisons were made at predefined intervals in order to determine corrosion size and its location on the bridge deck. In other words, according to Fig. 1, first, a finite element model of the bridge is generated. This initial finite element model simulates the undamaged bridge, and contains all the physical stiffness and mass parameters. For the calibration of the model natural frequencies and mode shapes obtained from the model and the experiments (made at the beginning of the bridge's life) are compared; if the differences are notable, the stiffness of the model is modified such that the results obtained from the model match with those of the experiments. The calibrated model (CM) is considered a reference model and represents the dynamic behavior of the undamaged bridge.

The calibrated model is updated based on the results obtained from experimental analysis of the bridge after occurrence of the corrosion. Comparing the results of natural frequencies and mode shapes obtained from field investigations with those obtained from the analyses of the calibrated model, the model is updated. That is, the element stiffness properties in the model are adjusted to correspond as closely as possible with the experimental results. For this purpose, an iterative sensitivity based FE model updating is applied for the elements located in zones related to the

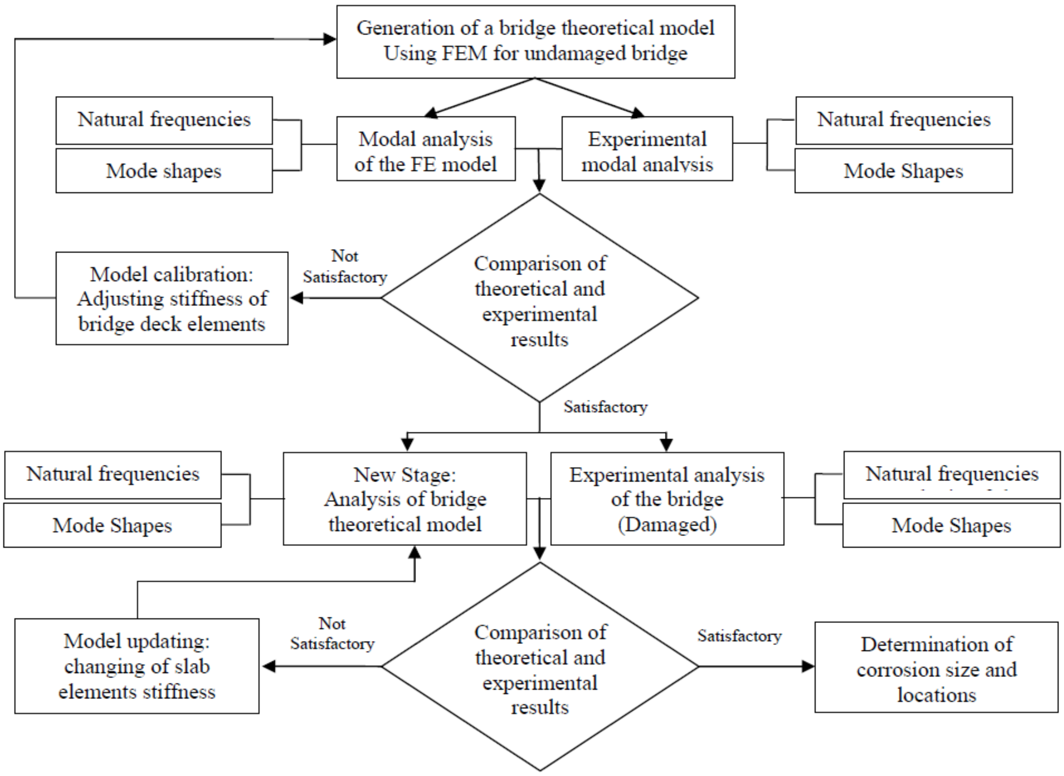

Fig. 1 Flow-diagram of bridge corrosion detection procedure adopted in this research

points where the modal displacements differ from those obtained in the field. In other words, the Young's modulus of the deck as updating parameter is revised in the zones where the experimental modal displacements differed from the FE model. This is made by changing the modules of elasticity of elements in the defected zone by a coefficient as defined under.

$$C.F. = -\frac{E^{zone} - E_{ref}^{zone}}{E_{ref}^{zone}} \tag{1}$$

Where *C.F.* is correction factor of Young's modulus of elements located in each zone; E^{zone} is Young's modulus of elements located in the zone for the final set of iterations at each stage; and E_{ref}^{zone} is Young's modulus of elements located in the zone at the reference state of the FE model (reference model). This process is repeated until the differences between the FE model and the experimental results are negligible.

As indicated above, corrosion in the bars is simulated in the model by a decrease in the bending and torsional stiffness of the individual deck elements, as represented by the Young's modulus. The location and the extent of the corrosions can be determined by comparing the differences between the reference model (CM) and the updated model (UM).

This technique has an important advantage with respect to the current damage assessment techniques, meaning that if identified correctly the model can be used to predict the structure's remaining safety and service life.

Fig. 2(a) Picture of concrete highway bridge comprised of two 23 m spans

Fig. 2(b) Schematic views of two-span concrete highway bridge (Dimensions in cm)

3. Evaluation of the proposed technique

To evaluate the proposed detection technique, corrosion in an existing concrete bridge was studied. A finite element model was created for the bridge. It was calibrated based on the results obtained from modal tests on the bridge in the first year of this investigation. The calibrated model was then updated using the experimental results obtained in the following years in five stages, identifying the location and extent of the corrosion in the bridge over a period of five years. At the end the fifth year, the bridge was perforated in order to examine the accuracy of the proposed model predictions.

3.1 Bridge description

The bridge investigated in this research consisted of two spans 23m long and 9.6 m wide with an expansion joint between the spans. A pedestrian walkway 80 cm wide exists on each side. The girders were 23 m long, 80 cm wide and 96 cm high under the 25cm thick slab. Due to the similarity of the spans, this study focused on only one span. A picture and a schematic diagram of the bridge are presented in Fig. 2.

3.2 Bridge in-situ tests

In-situ modal tests were conducted using an impact hammer to excite the bridge (Fig. 3). The response vibration in the bridge was picked up by a piezoelectric accelerometer. The spectra from both the accelerometer and the hammer were amplified by two charge amplifiers and transferred to a Fast Fourier Transform Analyzer (FFT). Cross spectra were created based on the frequency domain in the FFT. The spectra in time and frequency domains and the cross- spectra were sent to a computer (PC) which was linked to the FFT through a computer package called IP (Instrument

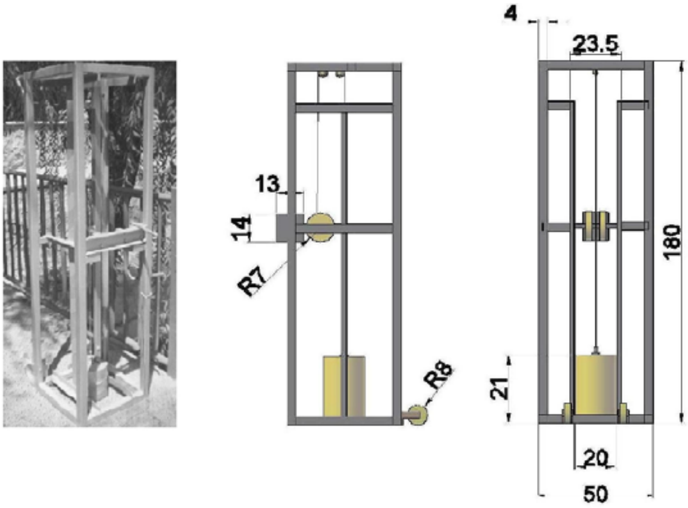

Fig. 3 Newly designed and constructed bridge exciter

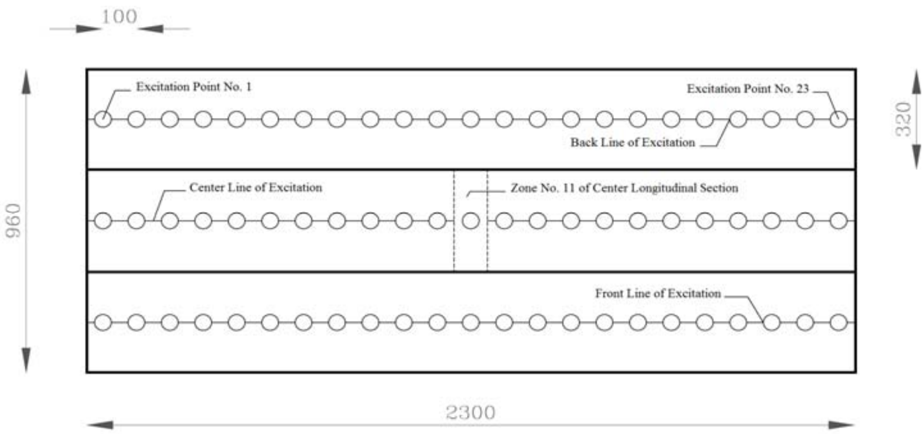

Fig. 4 The grid formed on the deck (Dimensions in cm)

Program). When the IP was excited by the PC, all of the analyzer's functions were downloaded into the analyzer's RAM. The spectra in time and frequency domains were saved by another computer package linked to the IP called STAR. The spectra saved by the IP were used to calculate the natural frequencies of the structure and the cross- spectra saved by STAR were used to investigate the mode shapes.

Modal analysis was performed on the right hand span. A grid of 23 transverse rows 1m wide and 3 longitudinal rows 3.2 m wide divided the deck into 96 sections. The sections were numbered from 0 to 23 for the back, center and front rows (Fig. 4). The excitation points were at the center of the sections.

Using a force transducer and two accelerometers, the excitation signal and vibration response of the bridge deck were measured simultaneously using two channels of the FFT. Frequency analysis of the deck was performed within the analyzer in conjunction with a desktop computer.

Responses of the test specimen were measured by the accelerometers. Measurements were made at various points on the deck and pictures of the responses of the deck were created.

The test equipment consisted of a newly designed and constructed hammer type excitation device, piezoelectric accelerometers magnetically attached to a metal washer which was glued to the bridge at a point close to its mid span, a piezoelectric accelerometer mounted on the weight of the hammer, a spectrum analyzer, two amplifiers which were set up to measure the displacements and accelerations and a PC with the IP package and cables. The excitation device consisted of a 55kg weight that is raised by spinning the handle. Releasing the handle allows the weight to drop along the guide rails and create the excitation impact. The overall height of the excitation device is 180 cm and the maximum possible height for the weight is 150 cm. The weight consists of three parts: two 20 kg weights on the top and a 15 kg weight at the bottom, joined by bolts (Fig. 3). A hook is placed on the weight and a cable is connected to the hook and the handle. An accelerometer was fixed to the weight to measure the acceleration of the hammer.

3.3 FE model

A three dimensional finite element model of the bridge deck was created using Opensees software. To build the numerical model "ShellMITC4" material and "Elastic-Membrane-Plate-Section" were used. Each bridge deck zone, indicated in Fig. 4, was comprised of 1280 shell elements. The material behavior was assumed to be linear elastic, isotropic and homogeneous. The concrete modulus of elasticity was estimated at 24000 MPa, based on concrete sample compression tests. The Poisson's Ratio of the concrete was assumed to be 0.2 and the mass density 2400 kg/m3. An additional mass of 200 kg/m was included to account for the bridge's walkway and handrail. To determine element properties such as moment of inertia, radius of gyration and the material specifications of the zone elements such as modulus of elasticity, all calculations were made for a cracked section with an equivalent area of the bars.

3.4 FE Model updating

In order to build up a reference model (CM), the FE model was calibrated against the modal properties of the bridge deck obtained from experimental analysis of the undamaged bridge. The model was updated by comparing the natural frequencies and mode shapes of the CM with those from the experimental analysis of the bridge in five stages over a period of five years.

The natural frequencies obtained from the modal analysis of the model and those obtained from the undamaged state of the bridge deck are presented in Table 1. This table indicates natural

Table 1 Natural frequencies and MAC values for undamaged bridge (FE-model and experiments)

Mode No.	Natural frequencies(Hz)			Relative error (%)		MAC values (%)	
	Exp.	FE model		FE model		FE model	
		Initial	Calibrated	Initial	Calibrated	Initial	Calibrated
1	2.89	2.67	2.88	7.61%	0.35%	99.96	99.96
2	4.53	4.16	4.52	8.17%	0.22%	99.76	99.81
3	7.46	8.23	7.56	-10.32%	-1.34%	95.82	98.97
4	9.78	9.04	9.71	7.57%	0.72%	97.85	99.54
5	11.62	10.18	11.53	12.39%	0.77%	96.18	97.87

Table 2 Natural frequencies and MAC values obtained from reference model (Ref.), updated models (Upd.), and experiments (Exp.)

undamaged		Damaged									
		First stage		Second stage		Third stage		Fourth stage		Fifth stage	
Natural frequencies (Hz)											
Mode no.	Ref.	Exp.	Upd.	Exp.	Upd.	Exp.	Upd.	Exp.	Upd.	Exp.	Upd.
1	2.88	2.74	2.72	2.68	2.67	2.64	2.66	2.55	2.49	2.47	2.41
2	4.52	4.35	4.32	4.29	4.34	4.20	4.16	4.03	4.11	3.85	3.72
3	7.56	7.08	7.15	7.02	6.89	6.82	6.71	6.61	6.65	6.33	6.16
4	9.71	9.15	9.09	8.92	9.03	8.83	8.78	8.72	8.79	8.54	8.43
5	11.53	10.78	10.89	10.66	10.84	10.38	10.49	10.12	9.98	9.63	9.93
MAC values (%)											
1		99.94	99.95	99.91	99.93	98.92	99.65	98.21	99.49	97.63	98.58
2		99.95	99.95	99.94	99.95	98.76	99.43	96.57	98.76	95.45	98.12
3		95.64	99.54	95.32	99.42	93.56	97.86	92.47	97.35	92.02	96.87
4		96.82	99.59	96.09	98.97	92.77	97.09	92.89	97.91	91.77	95.48
5		96.18	99.41	95.88	99.54	96.09	98.78	91.94	95.56	90.38	95.35

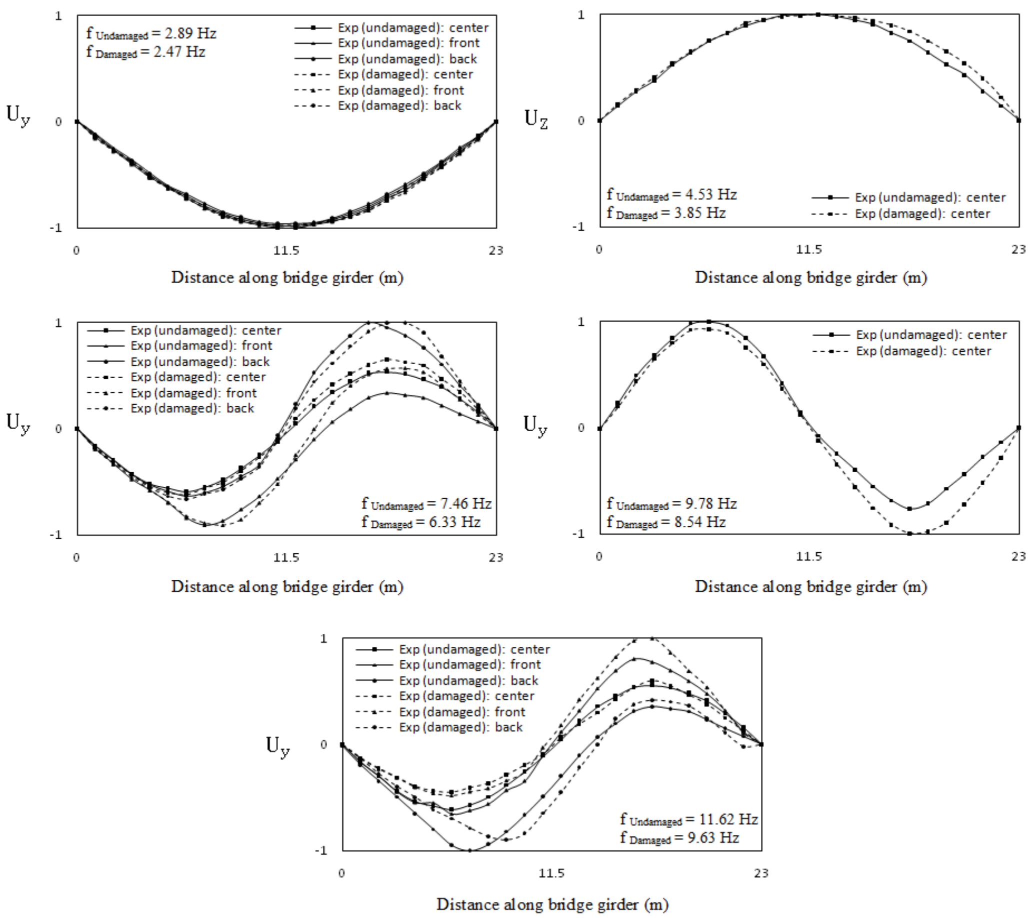

Fig. 5 Experimental frequencies and mode shapes before and after damage (at the 1st and 5th stages)

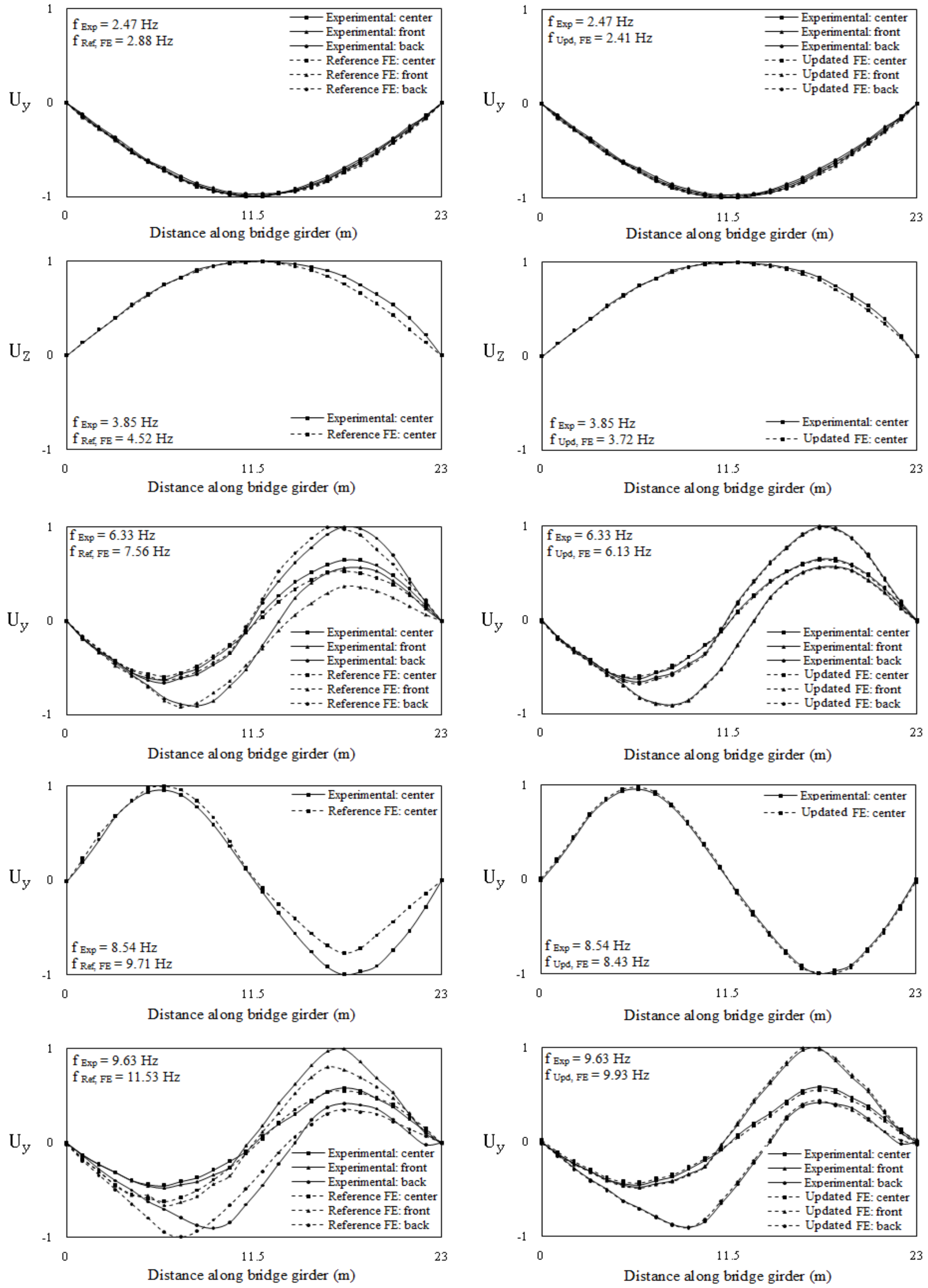

Fig. 6 Initial and updated numerical mode shapes of the damaged bridge in comparison with the experiment (at the 5th stage)

frequencies and MAC values for the undamaged bridge for the first five modes. The relative errors between the FE model and the experimental results vary between 7.57% and 12.39% for the first five vibration modes, but after revising the modulus of elasticity, the differences decreased to an acceptable range.

3.5 Defect detection

The results from the first five years of experiments and analyses of the CM are presented in Table 2. Experimental frequencies and mode shapes before and after the bridge damage (corrosion) are graphed in Fig. 5. Initial and updated numerical mode shapes of the damaged bridge in comparison with the experiment (at the 5th stage) are presented in Fig. 6. The relative errors between results obtained from five years of experiments on the bridge and those of the reference model (CM) for the first five modes are demonstrated in Fig. 7. Similarly, the relative errors for the updated model are presented in Fig. 8. Figs. 7 and 8 indicate that the differences between the first five vibration modes from the experiments and those of the calibrated model vary from 3.91% and 19.73% (from the first to the fifth stages), but are reduced to -3.38% to 3.12% after updating the model. Fig. 8 indicates that the updated numerical mode shapes are well matched with the experimental mode shapes.

Figs. 9 to 11 indicate bending stiffness of the bridge deck in the center, front and back rows, respectively. These were obtained from the initial, reference and updated models at the fifth stage. As illustrated in Fig. 9, the bending stiffness of the bridge deck at the center row was 1.795×10^5 N/m^2 in the CM, but became 1.546×10^5 N/m^2 after the last revision at the fifth stage. These changes are 1.815×10^5 N/m^2 to 1.398×10^5 N/m^2 and 1.804×10^5 N/m^2 to 1.530×10^5 N/m^2 for the front and back rows, respectively.

The changes in the Young's modulus were quantified by a correction factor (Eq. (1)). Fig. 12 indicates the correction factor for the deck stiffness at the end of the fifth year. According to Fig. 12, the decrease in the deck stiffness 18m from the left support is considerable, indicating a severe

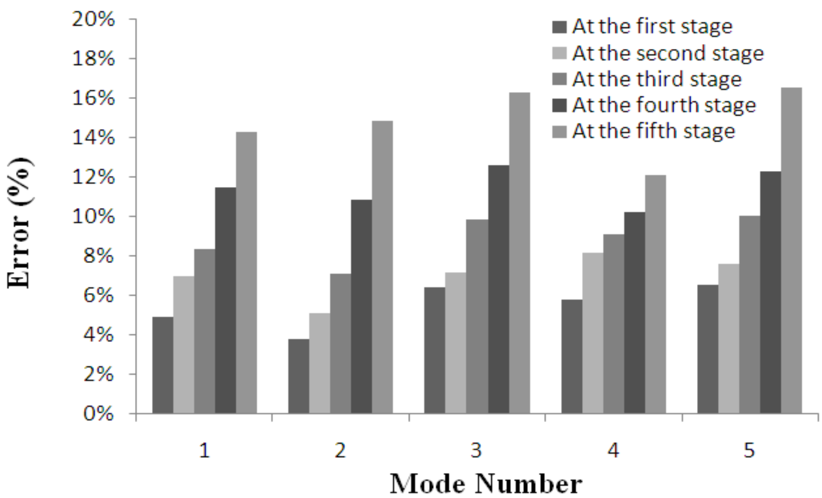

Fig. 7 Relative errors between damaged and FE (reference state) for first five modes

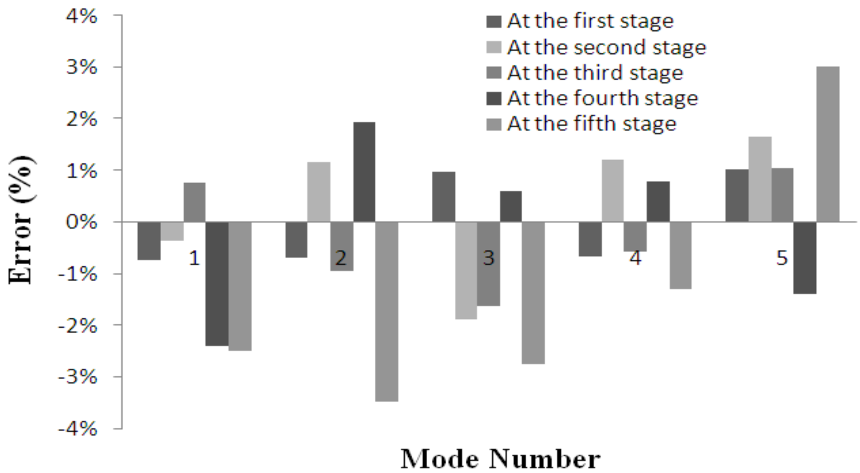

Fig. 8 Relative errors between damaged and FE (updated state) for first five modes

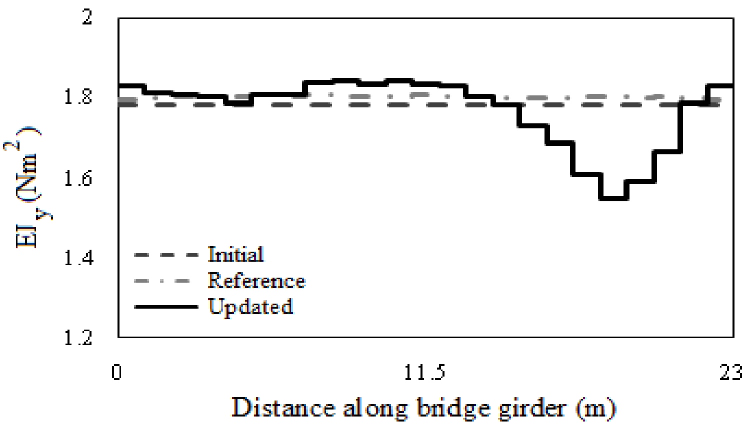

Fig. 9 Bending stiffness distribution for center line at the fifth stage

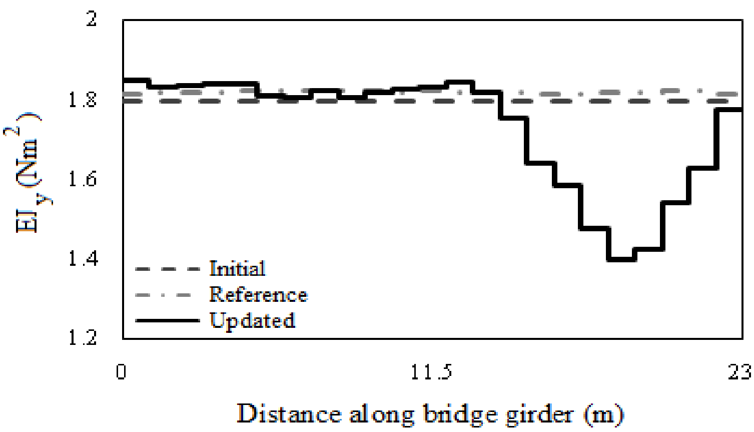

Fig. 10 Bending stiffness distribution for front line at the fifth stage

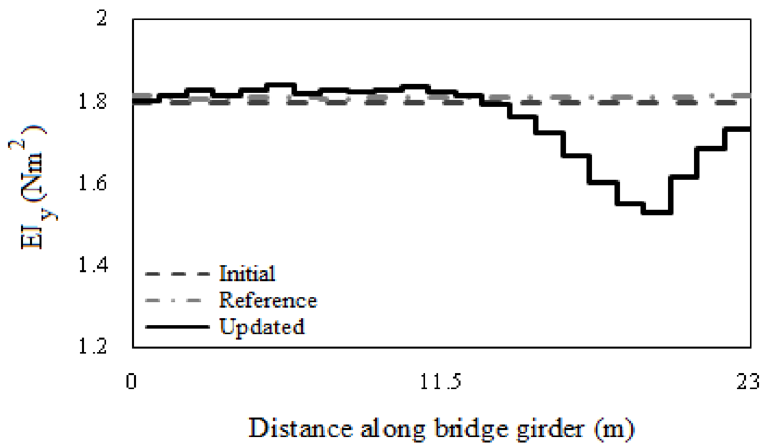

Fig. 11 Bending stiffness distribution for back line at the fifth stage

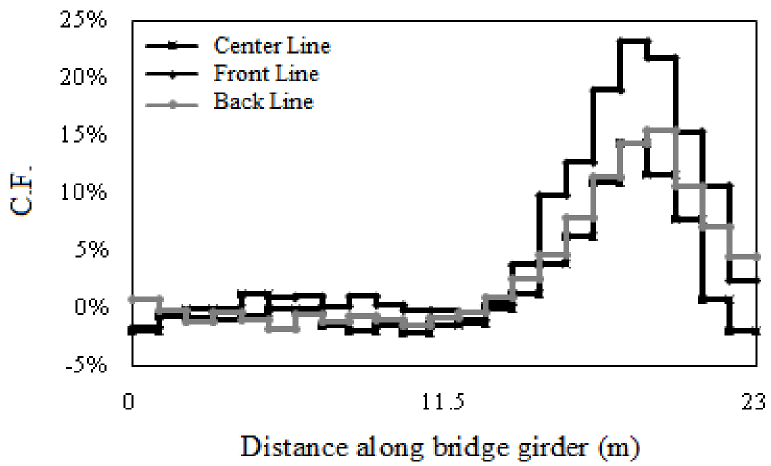

Fig. 12 Correction factor of Young's modulus at the fifth stage

defect in this area. The maximum correction factor is 23.21% for the front row, 14.35% for the centre row and 15.42% for the back row. Since the correction factor for the front row is the largest, this is the most critically damaged area.

3.6 Verification

As discussed above, the corrosion of the steel reinforcing bars leads to decrease their cross sectional area which directly affects the stiffness of the bridge. To evaluate the effectiveness of proposed method of corrosion prediction, the bridge deck was perforated 18m from the left pier where the FE model predicted the most severe corrosion. Considerable corrosion of the reinforcing bars was observed in the area. The damage to the bars (bar corrosion) was a 12% reduction in the bar cross sectional area for longitudinal bars and 14% for shear bars, causing progressive cracks in the right side of the deck.

4. Conclusions

This research develops a non-destructive method for detecting bridge reinforcing bars corrosion. Experimental modal analysis, as a non-destructive testing technique, and finite element model updating were used in this method. Based on the procedure proposed in this research, a finite element model of the bridge is generated and it is calibrated by comparing analysis results from the model with experimental results from the beginning of the bridge's life. This calibrated model is then updated based on the results obtained from experimental analysis of the bridge after occurrence of the corrosion. The location and the extent of the corrosions are determined by comparing the differences between the models updated before and after accordance of the corrosion.

The practicality and applicability of the proposed method were evaluated by applying the new technique to a two spans bridge for monitoring steel bar corrosion. A bridge FE model was created and the bridge vibration parameters were measured over five years. Following the procedure proposed in this research, the location and the extent of the corrosions were determined and the zones where corrosion had occurred were pinpointed. The bridge deck was then perforated in these zones and the reinforcing bars were examined. It was shown that the proposed method had correctly predicted the location and extent of damage to the bridge reinforcing bars.

Results obtained in this research indicate that bridge damages can be monitored using the new proposed technique over its lifespan. This technique has an important advantage with respect to the current damage assessment techniques, meaning that if identified correctly the technique can be used as an efficient tool in bridge maintenance management and prediction of the structure's remaining service life.

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