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Structural health monitoring and resilient assessment by novel intelligent models

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Abstract. In this paper, to assess the performance of a multi-span simply supported RC bridge, the dynamic characteristics of the bridge were measured and determined by structural health monitoring and resilient assessment via operational modal analysis as well as FE modeling. Supporting finite element (FE) models were created and analyzed according to the design drawings. This study used 2D plane monitoring of locations of hole in the infill wall and used 3D health monitoring and resilient assessment. From the results of 3Dsymmetric frame, if the frame is unsymmetrical, the used model can lead to the reduction in the internal forces. The recommendations from this study is from some discrepancies observed between 2D and 3D models, if possible 3D model should be used in analyzing the real frames.

Keywords: acceleration sensor; bridge; modal analysis; structural health monitoring

1. Introduction

The dynamic behavior of bridge superstructures highly depends on the geometric characteristics, properties of materials, and boundary conditions (Akbari *et al.* 2018, Karimi *et al.* 2019). Besides the accuracy of structural design, both the robustness of construction and consistency of the constructed structure with the design drawings are two challenging points for supervisors and bridge owners. It is a fact that conventional testing devices and laboratory tools are only able for quick but non-continuous quality control of materials and construction. On the other hand, monitoring the overall response of structures and interaction of the geometry, the strength of materials, and boundary conditions as well as the consistency of the constructed structure with the design drawings is difficult with conventional testing methods.

Any change in the geometry as well as in the material properties produces different values for the vibration frequency of the structure. In between the above parameters, the span length has a higher effect because of the higher power of this parameter (Maadani *et al.* 2015, Akbari *et al.* 2018, Karimi *et al.* 2019). These assumptions and relations are only applicable for assessing the

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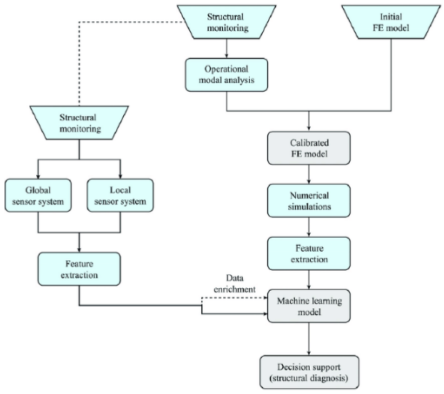


Fig. 1 General framework for VbSHM

natural frequencies of bending modes of vibrations. Similar to the span length parameter that has an influential effect in the bending natural frequencies, the parameter of element width (cross-section width of beam or plate) has higher effects in the torsional vibration frequencies.

It is a fact that in technical supervision of structures under construction only by using the conventional testing methods with simple laboratory apparatus, simultaneous control of support conditions, and both the geometry of elements and material properties of the constructed elements are necessary to provide a pre-expected overall performance of the structure. However, engineers and supervisors are conventionally able to perform a non-continuous assessment of elements and components. Dynamic testing of structures has been proven as a reliable solution to resolve the need for a continuous and overall assessment of structural behavior (Wang *et al.* 2005, Wenzel 2009, Maalek *et al.* 2010, Akbari *et al.* 2019, Cunha and Caetano 2006).

In this paper, the dynamic response of an eight-span simply-supported reinforced concrete bridge with a total length of 224 m has been assessed. Among various monitoring methods for bridge structures (e.g., Min *et al.* 2016, Abdel-Jaber and Glisic 2019, Dang *et al.* 2022) vibrationbased monitoring has been selected here. The most important advantages of the method have been given in the next section. The main aim of this study is to assess the overall dynamic response and the quality of construction of the superstructure and to assess probable damages in the bridge deck. Dorji *et al*. (2009) investigated in-filled frame structure under seismic loads. The purposes of this study were to develop an appropriate technique for modeling the infill- frame interface and to study the seismic response of in-filled frame structures. The finite element method was adopted and the model structures were 10-story 3-span frames. For infill-frame interface, it was modeled by using gap elements and a single-story single-span infill-frame model was used to obtain the correlation between infill stiffness and gap stiffness by comparing with the test results. Two different materials for concrete and infill were assumed by varying the young modulus of each material. Time history analyses with three different seismic records scaled to the desired peak ground accelerations (PGA) were performed and the influences of infill strength, opening and soft-story phenomenon were inspected. The conclusions drawn from this study were that (1) increasing the Young’s modulus of infill and/or concrete led to decrease in the fundamental periods, the roof displacement and the story drift ratio, (2) the effect of opening size was to increase the fundamental periods, the roof displacement, the story drift ratio and the infill stresses, (3) although the inter-story drift ratio did not exceed the inter-story drift ratio limit for all input motions, infill stresses increased with increasing PGA, and (4) the performance of buildings was greatly influenced by the type of infill walls and the values of Young's Modulus.

Kose (2009) studied the effect of building height, number of bays, ratio of area of shear walls to area of floor, ratio of in-filled panels to total number of panels and type of frame on the fundamental period of RC building with infill walls. In this study 3D building models were adopted for finite element analyses. Three types of building model: bare- frame building, building without an open first floor and building with open first floor were constructed and infill wall were modeled as structural members with mass. Due to limitation of the software which did not have a feature of nonlinear analysis, this study used the iterative linear modal analysis. After conducting the analyses for 189 building models, a regression analysis with the parameters mentioned above was performed to propose an equation for evaluating the fundamental period of RC building which in author's opinion can give a better estimate than the code equations.

To minimize the factors involved, only the linear analysis and a single-story building structures in the forms of 2D and 3D frame were considered in this preliminary study. In the analysis, the beams and columns were modelled as linear beams members and the slabs and reinforced concrete infill walls modelled as plate elements. Various layouts of the reinforced concrete walls were analysed in a hope to illustrate the effect of reinforced concrete infill wall on the building responses in a comprehensive way.

2. Vibration-based monitoring of bridges

Condition monitoring of bridges is usually done via visual inspection or in-situ response measurement. Visual inspection is subjective and depends on the judgment and experience of the bridge inspector. In recent decades, engineers have used structural health monitoring (SHM) techniques to supplement visual inspections. In SHM, a variety of sensors are employed to collect data of bridge response (time history of the response), maybe remotely and/or automatically, and the data is used to assess the bridge condition. Practically, during the last decades, engineers or researchers began to widely use of vibration-based SHM of bridges (VbSHM). The method aim to evaluate bridge health through vibration response data, usually collected by accelerometers installed on the bridge. These sensors are widely used since obtaining the acceleration data is relatively easy and cost-effective and well-suited for many applications, and for various analyses and assessment goals. Generally speaking, VbSHM of bridges plays a significant role among various types of SHM implementation to support structural assessment and asset management.

Theoretically, in VbSHM, after collecting a time history of the bridge response via accelerometers, the collected data is converted from time domain into frequency domain using the well-known Fourier transform. Analysis of the frequency domain data is usually done via different techniques to extract modal parameters and to produce modal domain data. The suitability or advantages of modal data for damage monitoring of bridges is influenced by this fact that modal information is a reflection of the global system behavior while damage is a local phenomenon. A literature review revealed that no agreement exists between researchers about the suitability of modal data for damage detection and monitoring – a portion of opinions says that it is adequately sensitive while the other disagrees (Carden and Fanning 2004). To date, the opposing opinions have been demonstrated for specific test structures but have not been proven in a fundamental sense.

While various approaches to bridge vibration monitoring exist, a general framework for the

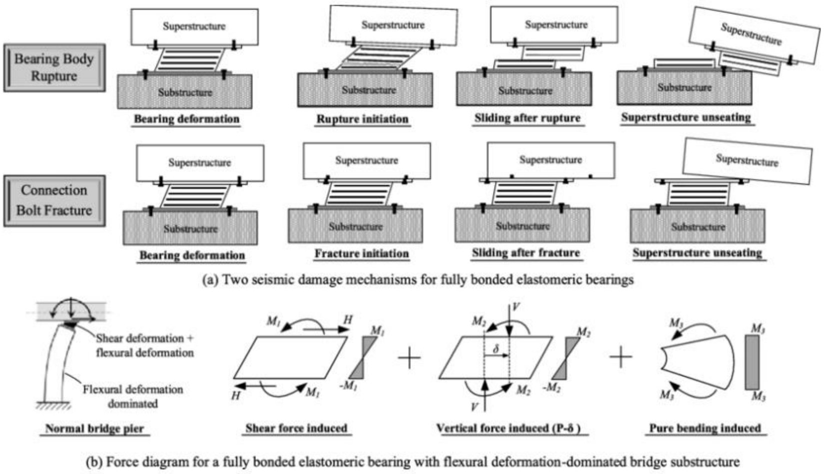


Fig. 2 Details of the bridge bearings

practical use of the method is shown in Fig. 1.

Firstly, a sensor network is installed on the bridge. The collected records are analyzed to identify the modal parameters of the bridge (i.e., natural frequencies, mode shapes, and damping ratios). To identify the first few lower-frequency global vibration modes, it is important to observe that the recorded acceleration data target the global response of the bridge. The vibration characteristics depend on the mechanical properties of the structure and the modal parameters identified are usually used to estimate the structural parameters of selected elements within the bridge (e.g., the stiffness). Results are stored in a database and compared with previous obtained values, and values that differ more than a predefined threshold may signal an abnormal structural behavior, damage, or other conditions, depending on the specific aim of monitoring. If necessary, corrective measures are performed, and the monitoring process restarts. In VbSHM, determinations are based on detecting changes in its vibration characteristics.

3. The construction monitoring

As shown in Fig. 2, on one side of all the girders, two side restrainers of steel angle profile of L250*250*20*180mm have been used and bolted to the cap beams and girders. According to the construction documents and design drawings, the compression strength of concrete is specified for the girders, the deck slab, and the substructure elements as 45, 30, and 25 MPa respectively.

As a result of similar support conditions, material properties, and geometric characteristics for all the spans, the bridge was selected for dynamic testing is all the spans separately. Because of similar support conditions and isolated bearings, it is predicted that no major effect exists from the substructure on the dynamic properties of the deck spans. In the case of similar dynamic properties among the spans and good agreement with finite element (FE) analysis results, it can be concluded that all the spans are in good condition as designed. In the case of major differences in the dynamic properties between some of the spans, or in comparison with the FE results, the existence of possible damages is probable.

4. Details of the dynamic tests

To obtain the largest interval for the fuzzy number by setting α=0. $\hat{\tilde{F}}_t^1 = \left(\hat{F}_t^{1,L}, \hat{F}_t^{1,R}\right)$Also based on Theorem 1 approximate fuzzy number$\hat{\tilde{F}}_{t+1}^0 = \left(\hat{F}_{t+1}^{0,L}, \hat{F}_{t+1}^{0,R}\right)$ with bottom bracket$\hat{F}_{t+1}^{0,L}$ and upper limit$\hat{F}_{t+1}^{0,R}$ for period t+1 you get as

$$\hat{F}_t^{0,L} = \hat{F}_t^{1,L} - \hat{F}_{t-1}^{1,L}, \quad \forall t \geq 2,$$
$$\hat{F}_t^{0,R} = \hat{F}_t^{1,R} - \hat{F}_{t-1}^{1,R}, \quad \forall t \geq 2 \tag{1}$$

You can get $\tilde{F}_1^{0*}, \tilde{F}_2^{0*}, \ldots, \tilde{F}_n^{0*}$explained fuzzy values $\hat{\tilde{F}}_{n+1}^0, \hat{\tilde{F}}_{n+2}^0, \ldots, \hat{\tilde{F}}_{n+k}^0, \forall t \geq 1$through the input data . Theorem 1 shows that a fuzzy number $\hat{\tilde{F}}_t^1, \forall t \geq 2$is not a symmetric fuzzy number, so $\hat{\tilde{F}}_t^0$it is also not a symmetric fuzzy number. Conversely, $\hat{\tilde{F}}_t^0$to estimate the forecast error, the mean is defined as $\hat{F}_t^{0,Aver}$, given by

$$\hat{F}_t^{0,Aver} = \frac{F_t^{0,L} + F_t^{0,R}}{2}, \qquad \forall t \geq 2.$$

Therefore, the gray time series model GM(1,1) is used.$F_2^0, \ldots, F_n^0$ is specified to extrapolate the values for . $\hat{F}_{t+1}^0, \hat{F}_{t+2}^0, \ldots, \hat{F}_{t+k}^0, \ \forall k \geq 1$. The main features of the Takagi-Sugeno multilayer fuzzy model are the expression of each rule by a linear equation of state, and the model is as follows (Chen 2014, Chen *et al.* 2019, Chen *et al*. 2020)

$$\text{Rule i : If present } x_{1j}(t) \text{ is } M_{i1j} \text{ and } x_{g\,j}(t) \text{ is } M_{ig\,j} \tag{2}$$

AS $\dot{x}_j(t) = A_{ij}x_j(t) + \sum_{k=1}^{N_j} A_{ikj}x(t-\tau_{kj}) + B_{ij}u_j(t)$

where $x_j^T(t) = [x_{1j}(t), x_{2j}(t), \cdots, x_{g\,j}(t)]$ _ $u_j{}^T(t) = [u_{1j}(t), u_{2j}(t), \cdots, u_{mj}(t)]$

r_j is the IF-THEN rule number for this A_{ij} j. Subsystem, A_{ikj} and B_{ij} are paired system matrices, state $x_j(t)$, input $u_j(t)$, τ_{kj} delay fuzzy set M_{ipj}($p = 1, 2, \cdots, g$) and assumption are used $x_{1j}(t)$~ $x_{g\,j}(t)$to derive the dynamic fuzzy model:

$$\dot{x}_j(t) = \frac{\sum_{i=1}^{r_j} w_{ij}(t)\left\{A_{ij}x_j(t) + \sum_{k=1}^{N_j} A_{ikj}x(t-\tau_{kj}) + B_{ij}u_j(t)\right\}}{\sum_{i=1}^{r_j} w_{ij}(t)}$$
$$= \sum_{i=1}^{r_j} h_{ij}(t)\left\{A_{ij}x_j(t) + \sum_{k=1}^{N_j} A_{ikj}x(t-\tau_{kj}) + B_{ij}u_j(t)\right\}$$

with

$$w_{ij}(t) = \prod_{p=1}^{g} M_{ipj}(x_{pj}(t)), \quad h_{ij}(t) = \frac{w_{ij}(t)}{\sum_{i=1}^{r_j} w_{ij}(t)} \tag{3}$$

where $h_{ij}(t) \geq 0$, is $M_{ipj}(x_{pj}(t))$ the M_{ipj} position of relatives if, $x_{pj}(t)$

$w_{ij}(t)\geq 0,\ i=1,2,\cdots,r_j$ and $\sum_{i=1}^{r_j} w_{ij}(t)>0 \quad i=1,2,\cdots,r_j\,,\quad \sum_{i=1}^{r_j} h_{ij}(t)=1\,.$

According to the above analysis, cf i could be $\_\ F_j$

$$\dot{x}_j(t)=\sum_{i=1}^{r_j} h_{ij}(t)\left\{A_{ij}x_j(t)+\sum_{k=1}^{N_j} A_{ikj}x(t-\tau_{kj})+B_{ij}u_j(t)\right\}+\sum_{\substack{n=1\\ n\neq j}}^{J} C_{nj}x_n(t) \tag{4}$$

Figs. 3 and 4 show the comparisons of natural frequencies of infill wall with door opening and with window opening of 2D single-story double-span frames, respectively. The trends observed are similar to those of 2D single-span case in that (1) the opening will decrease the natural frequencies, as compared with the full wall, (2) the including the infill wall in the model will lead to a significant increase in natural frequencies, as compared with pure frame models.

For the models with door opening, it is interesting to find that for the first mode, second mode and the third mode, the natural frequencies are almost the same while the differences are observed for the third and fifth modes. For the model DFW-d3, its third mode natural frequency is smaller than that DFW-d4 model, while the trend is reversed for the fifth model.

For the models with single window opening, the natural frequencies of first, second and third modes are almost the same and this trend is the same as that of single door opening as discussed above. A point should be mad that for the DFW-w3 model, its third mode has almost the same natural frequencies as DFW-w5, while for other modes DFW-w3 model has higher natural frequencies than DFW-w4 models. On the other hand, with the increase in the opening area, the natural frequencies will decrease as expected. However, with the opening area occupying the 25% of the infill wall, we still can observe the high natural frequencies, as compared with pure frame models.

From the previous discussions, it be seen that although the locations of door opening affects the natural frequencies of 2D single-span models significantly, such an effect are not observed for at least the first two modes.

It is the comparison of shear forces of single-story double-span frames. At end-i, it can be seen that the variation of locations of opening does not affect the shear force significantly for Element Type A, Element Type B and Element Type C. This is also true for the shear forces at end j except the values are very small, close to zero. A point should be made if that for single-span

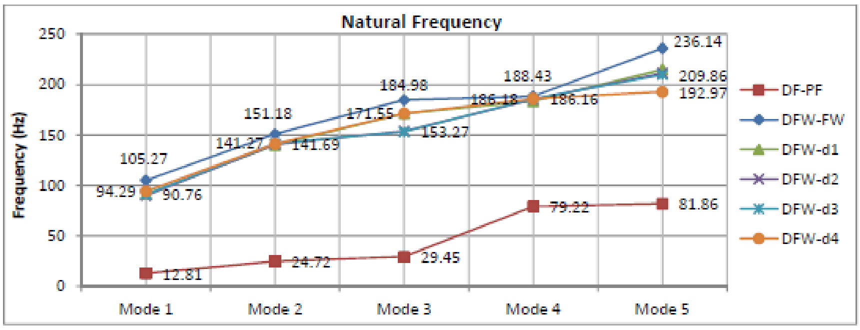


Fig. 3 The monitoring of 2D Single-Story Double-Span Frames

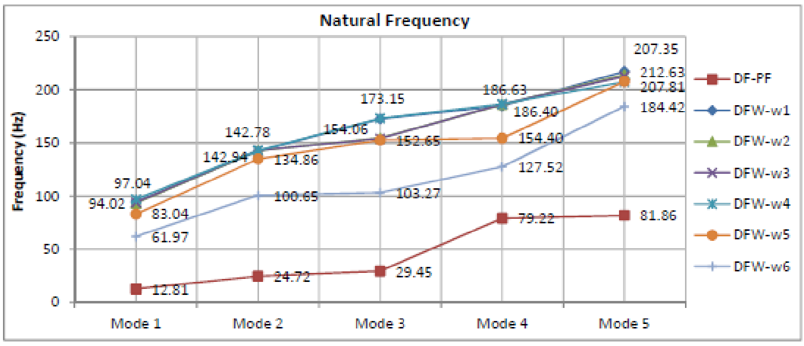
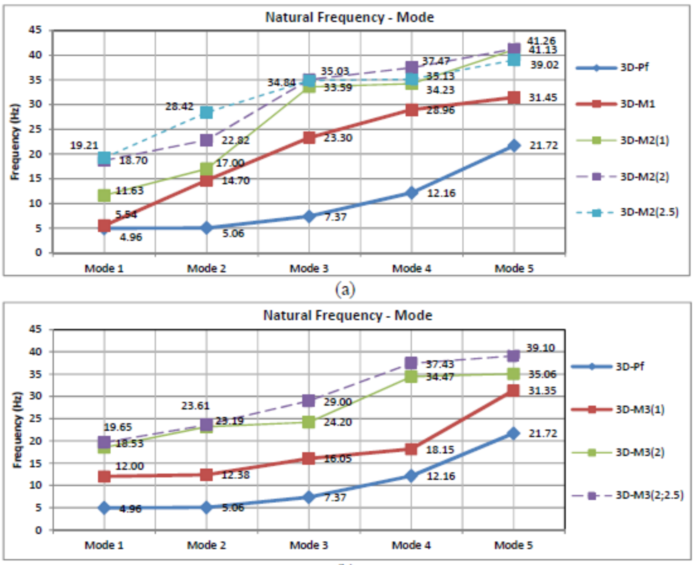


Fig. 4 Comparisons of Natural Frequencies of Infill Wall with Window Opening Effect of 2D Single-Story Double-Span Frames



(a)

(b)

Fig. 5 monitoring results from resilience, (a). Case-1 and (b). Case-2

frame the pure frame has significant higher shear force than the frame with infill wall, while for the double-span case, the difference in the shear force of pure frame and infill wall is not as significant as that of frame with infill wall for Element Type A and Element Type B.

Fig. 5(a) suggested as seen before, the inclusion of infill wall will increase the natural frequencies of the frame. An interesting point is that when the frame is a 2D one, it has very high natural frequencies, while for the 3D models; the natural frequencies are significantly reduced. For mode 1, surprisingly 3D-PF and 3D-M1 have the same value, indicating that this mode is mainly along the Line A and Line B directions. When infill walls are added along the Line A and Line B directions, the natural frequencies of first mode increase significantly and 3D-M2(2) and 3D-M2(2.5) have almost the same first-mode frequency; however, for the higher modes of these two models we can see the differences especially for second mode.

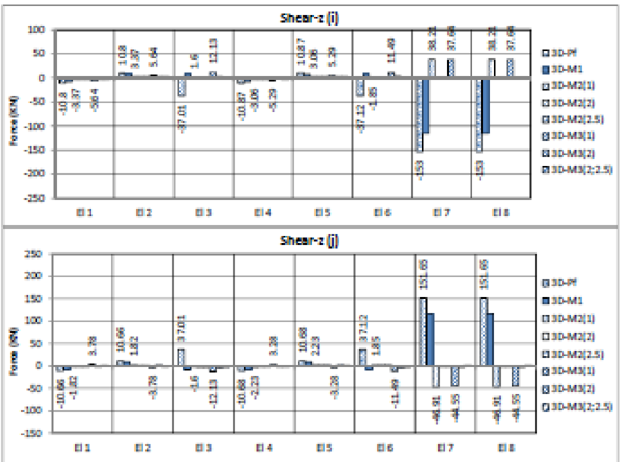


Fig. 6 Comparison of monitoring resilence

Shown in Fig. 5(b) are the comparisons. The trend is very similar to the one observed. An point should be noted that for 3D-M3(2) and 3D-m2(2) they have almost the same frequencies except for third mode, while for 3D- M2(2.5) and 3D-M3(2;2.5) models they have almost natural frequencies, indication that the opening of 500 mm along Line 1 and Line 2 does not affect the natural frequencies significantly.

It compares the natural frequencies of first 5 modes for all models with rigid floor assumptions. Since in this study the slab is modeled as plate elements, it can be seen that the use of rigid diaphragm or not will not affect the results which is not the case for model with pure frame only.

It also shows the shear-y force of all models for X-direction and Y-direction inputs, respectively. Fig. 6 showed the shear-z force of all models for X-direction and Y-direction inputs, respectively. For both ends the Y-direction input induces much larger shear-y force than the X-direction input for the columns of all models especially for 3D-PF and 3D-M1 models. Also, with increase in the infill wall length the shear-y force will decrease slightly. Another point is that for the 3D-PF model, X-direction input gives higher value of shear-y force for Element 7 and Element 8 than Y- direction input, while the trend is reversed for Element 3 and Element 6. The shear-z forces of both ends of columns and Element 3 induced by X-direction input are much larger than those induced by Y-direction input. How for Element 7 and Element 8 the trend is reversed. This indicates the use of wind wall can reduce the shear force in the frame members significantly.

5. Modeling and assessment

A Finite Element (FE) model of the bridge has been created in the SAP2000 environment according to the design drawings and material specifications shown in Fig. 7. Frame elements have been used for the bridge girders and frame bents. Shell elements have been used for the deck slab.

The values of damping for these types of bearings are around 5% (for normal bearings) to 15% (for highly damped types). Here, a value of 5% has been considered.

The maximum difference is around 13.6% for the third bending mode. In addition, in all the identified modes, the values of the natural frequencies obtained from the tests are higher than the

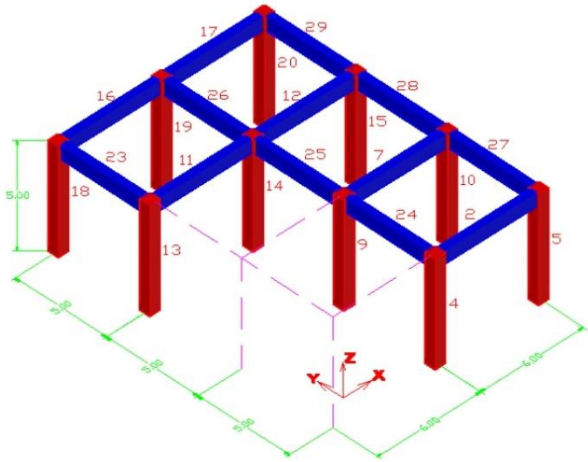


Fig. 7 Mode shapes obtained from the FE analyses

values of the FE analyses which can be certainly due to the fact of higher stiffness of the FE models than the real structure.

Generally, via the above-mentioned tests and investigations, the contractor's performance in the construction of a homogeneous deck and consistency of the design requirements with the as-built drawings has been assessed. Also, it can be concluded that the bridge is in healthy condition after 35 years of operation and no signs of degradation of stiffness or material properties exist and the damping ratios are at a normal level and less than 5%. From the substructure, additional investigations are required that are not considered here.

6. Conclusions

To assess the performance of a multi-span simply supported RC bridge, after 35 years of operation, the dynamic characteristics of the bridge were measured and determined via operational modal analysis as well as FE modeling. Because of the support conditions of the spans and the separation of the spans via elastomeric expansion joints, all the spans were tested separately.
Supporting FE models were created and analyzed. The results can be summarized as follows:

- Comparison of the natural frequencies obtained from the tests in between the different spans has shown that minor differences exist between the results. The maximum value was around 9.3%. This difference in parallel with considering the values obtained from damping ratios of less than 5% have shown that the deck was in a homogeneous condition from the point of view of boundary conditions, geometry, and material properties.
- Comparison of the natural frequencies and mode shapes obtained from the tests and the FE analyses has revealed that minor differences between the results exist. This can be regarded as a result of the consistency of the design drawings with the as-built construction. This point in parallel with considering the results of the values of damping ratios has shown that the superstructure is in good and healthy performance after 35 years of operation.
- These results have confirmed the contractor's acceptable performance from the point of view of the construction quality.

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