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Monitoring and performance assessment of a highway bridge via operational modal analysis

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Abstract. In this paper, through operational modal analysis and ambient vibration tests, the dynamic characteristics of a multi-span simply-supported reinforced concrete highway bridge deck was determined and the results were used to assess the quality of construction of the individual spans. Supporting finite element (FE) models were created and analyzed according to the design drawings. After carrying out the dynamic tests and extracting the modal properties of the deck, the quality of construction was relatively assessed by comparing the results obtained from all the tests from the individual spans and the FE results. A comparison of the test results among the different spans showed a maximum difference value of around 9.3 percent between the superstructure's natural frequencies. These minor differences besides the obtained values of modal damping ratios, in which the differences were not more than 5 percent, can be resulted from suitable performance, health, and acceptable construction quality of the bridge.

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.

Generally, classical boundary conditions have some well-known and predictable effects on the dynamic behavior of bridges and structures. However, the effects of geometric characteristics and material properties are slightly different. In other words, variations of these parameters have

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different effects on the dynamic behavior of bridges. To better understand this matter, the dynamic characteristics of a single-span beam and plate -that can be simply extended to bridge decks- are reviewed here.

According to the classical theory of free vibration, the radial/angular frequencies of a single span beam and plate of uniform cross-section are as follows (Warburton 1976, Leissa 1973)

$$\omega_n = \lambda_n \sqrt{\frac{EI}{\rho L^4}} \qquad n = 1,2,3,\ldots \quad \text{for beams} \tag{1}$$

$$\omega_n = \lambda_n \sqrt{\frac{D}{\rho a^4}} \qquad n = 1,2,3,\ldots \quad \text{for plates} \tag{2}$$

where λ represents a non-dimensional modal parameter related to the mode number n and support conditions. EI and D are respectively the bending rigidity of the beam and plate. E and I are the elastic modulus and moment of inertia. L and a represent the length of the beam and the length of the plate (free edge). ρ is the mass density and $\rho = \gamma A$ for materials of specific mass γ and cross-sectional area A. For a beam section of unit width and uniform height h, $I = h^3/12$. Also, for a plate strip of unit width and uniform thickness t and Poisson's ratio ν, $D = Et^3/12(1-\nu^2)$. By substituting the above quantities in Eqs. (1) and (2), the following relations obtain

$$\omega_n = A_n \frac{h}{L^2} \sqrt{\frac{E}{\gamma}} \qquad n = 1,2,3,\ldots \quad \text{for beams} \tag{3}$$

$$\omega_n = A_n \frac{t}{a^2} \sqrt{\frac{E}{\gamma(1-\nu^2)}} \qquad n = 1,2,3,\ldots \quad \text{for plates} \tag{4}$$

In these relations, $A_n = \frac{\lambda_n}{\sqrt{12}}$ and the two quantities of $\frac{h}{L^2}$ and $\frac{t}{a^2}$ are related to the geometry. The two quantities of $\sqrt{\frac{E}{\gamma}}$ and $\sqrt{\frac{E}{\gamma(1-\nu^2)}}$ are related to the material properties.

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 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.

According to Eqs. (3) and (4), both the geometry of the element and material properties affect the vibration behavior of the structure. On the other hand, the dynamic properties of two beam/plate elements of similar support conditions only depend on the geometry of the element and material properties. 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

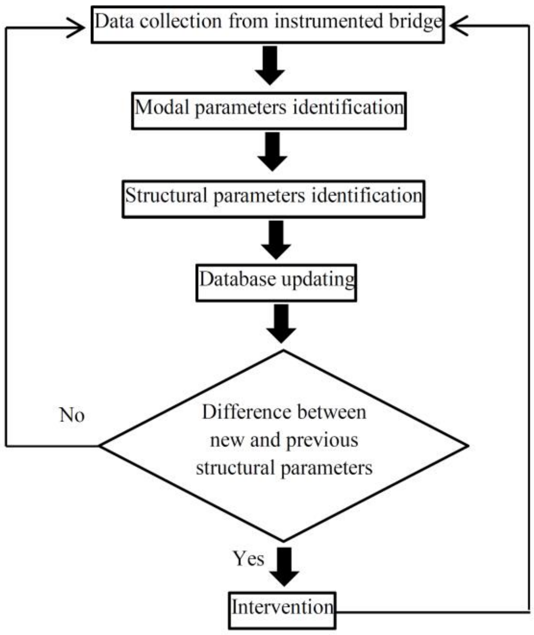


Fig. 1 General framework for VbSHM

(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 224m 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) vibration-based 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.

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 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.

It is obvious that, the benefits of VbSHM are maximized through long-term monitoring. This prepares an opportunity to study the full history of the bridge. Some examples of such applications and advantages of VbSHM are as follows:

- After occurrence a potential damage, such as an earthquake, a long-term VbSHM enables engineers to determine damage condition such as location of the elements affected and the extent of the damage.
- Quantification of structural aging becomes possible via tracking changes of modal parameters throughout the bridge's service life. Aging produces small but progressive shifts in vibration characteristics associated with gradual decrements of the stiffness over time.
- The residual bridge's capacity is estimated via vibration data collected during its' service life. This allows making better decisions on subject matters such rehabilitation, estimating the remaining service life, verifying the load carrying capacity and deciding whether the bridge should be posted or removed from serviceability. In addition, for a monitored network of bridges, this enables rational decisions for prioritization of remedial works and more effective budgeting.
- Enabling timely implementation of corrective measures before deterioration expands further. On the other hand, VbSHM may help transition from a reactive maintenance strategy to a preventative one.

While long-term VbSHM is more supportive, short-term VbSHM have some advantages as follows:

- Bridges are instrumented with temporary sensors (i.e. accelerometers) to identify modal parameters that can be compared against the design values to conclude whether the actual behavior of the in service bridge complies with the design expectations (e.g. the current study).

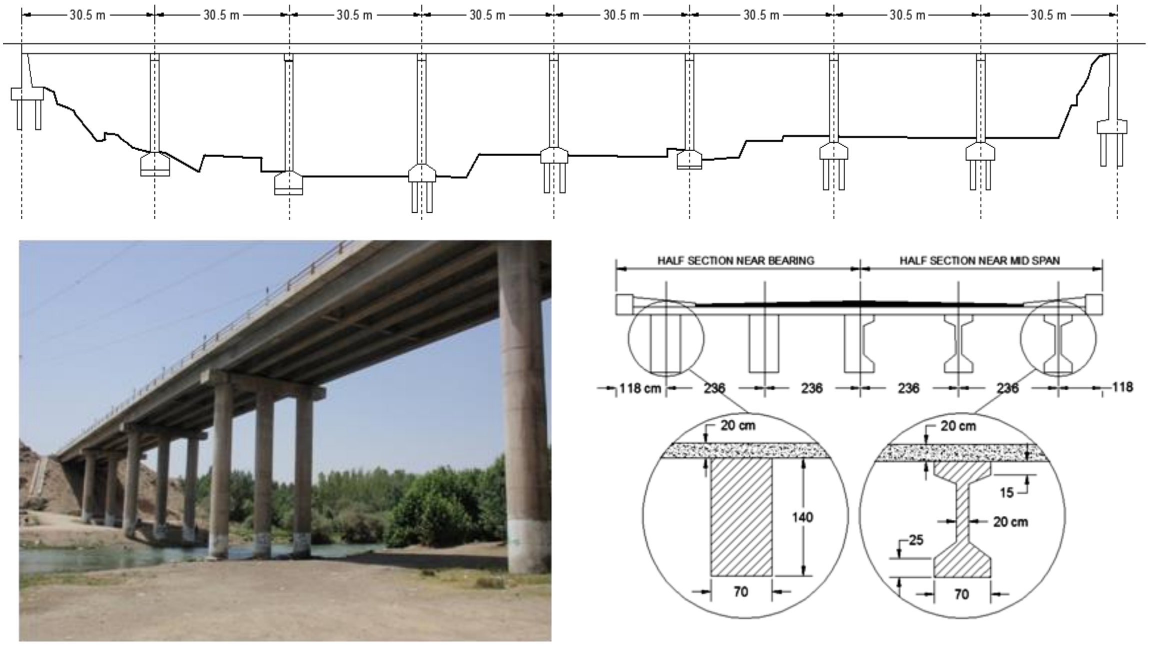


Fig. 2 The bridge under study

- Temporary accelerometers are used when bridge repair, retrofitting, widening, and so on, is performed to measure the difference between the modal characteristics identified before and after the intervention (e.g. Maadani *et al.* 2015). This information provides a measure of the effects of the intervention and allows verification of the design objectives.

3. The bridge under study

The bridge under study, constructed in 1985, is an eight-span reinforced concrete (RC) slab-on-girder bridge with a total length of 244 m with span lengths of 30.5 m located on the Isfahan-Shahrekord highway. The deck width is 11.8 m. All the spans are simply supported and separated by elastomeric-type expansion joints. In each span, five RC pre-stressed I-shaped girders rested on reinforced elastomeric bearings and were restrained with longitudinal restrainers (steel angles and bars) on one side and RC shear keys on both sides. The substructure includes seven middle piers each of which is composed of three circular RC columns of 1.6m diameter framed with an RC cap beam of a rectangular cross-section of 1×1.68 m. The distance between the columns (center-to-center) is 4.1 m. In accordance with the cross-section of the river bed, the elevation levels of the deck from the foundations are different from 13 to 19 m. A schematic longitudinal view as well as the deck cross-section dimensions together with a photo of the bridge is shown in Fig. 2.

As shown in Fig. 3, 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.

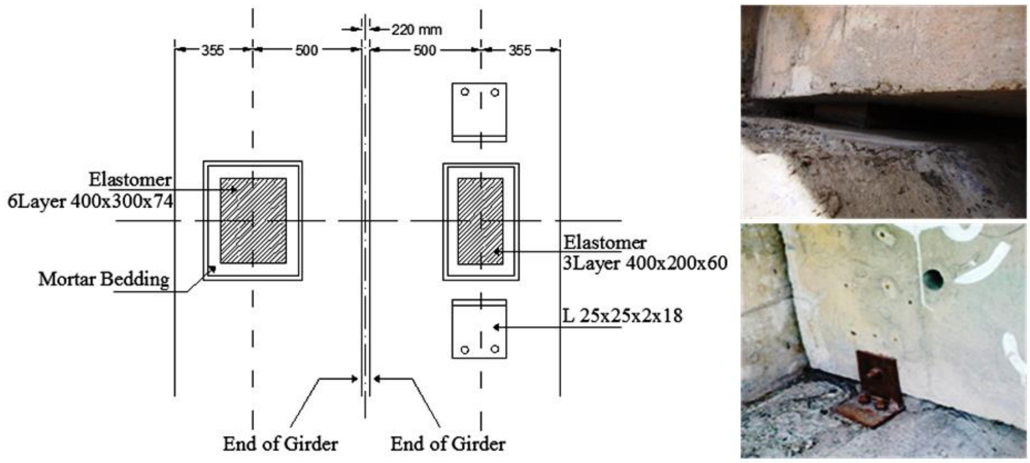
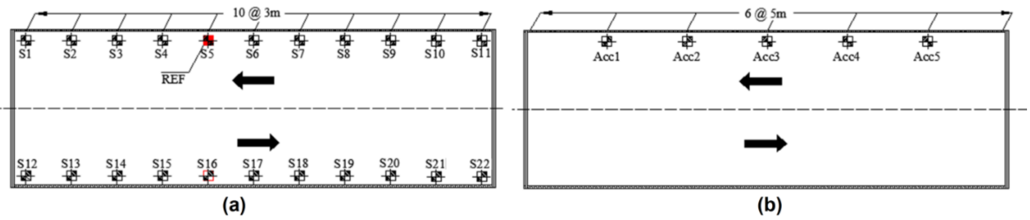


Fig. 3 Details of the bridge bearings

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

Because of the importance of the tests from the point of view of the required accuracy of the results for comparison between the spans, it was important to carry out the tests with sufficient care. The bridge deck has been tested span-by-span subsequently each one as a single and independent dynamic test. The bridge was under normal daily traffic during the tests (operational modal analysis). For the tests, the data have been recorded via a 36-channel dynamic data logger with a sampling rate of 200 Hz together with 12 triaxial accelerometers with an accuracy of ±0.001g. The duration of recording time of each measurement was 25 min, around 3000 times the first probable natural period of the superstructure. The accelerometers were suitable for measurements in the range of 0 to 50 Hz and were virtually insensitive to high-frequency vibrations. Therefore, 300000 data have been recorded by each sensor in each station. Because of the accessibility of the research group to both sides of the deck for two side spans, and the possibility of passing the measurement cables from under the deck, both sides of the deck for the two side spans have been used for measurement and the test has been done in two subsequent and continuous arrangement (see Fig. 4(a)). However, for the middle spans, because of the limitation in the length of the measurement cables and difficulties related to the accessibility of the research group to both sides of the deck, only one side of the deck has been used and the test has been done



(a) arrangement of the sensors for two side spans and (b) arrangement of the sensors for middle spans

Fig. 4 The measurement grid for the tests

Fig. 5 Some photos from the tests

in one arrangement (see Fig. 4(b)). It should be noted here that, adequate care has been performed to obtain both the vertical and torsional mode shapes of the deck as clearly as possible. The measurement arrays are shown in Fig. 4 and Table 1. On the other hand, a symmetric arrangement on both sides of the deck has been used for the two side spans and one side of the deck has been considered for the middle spans. In addition, one reference sensor has been used in a fixed position for the measurements of the two side spans (Point S5 in Fig. 4). Three photos from the tests are shown in Fig. 5.

While high values of temperature variations affect the dynamic characteristics of prestressed concrete girders, all the tests reported here have been done in two subsequent working days in the condition of nearly identical temperatures (around 10 a.m. to 2 p.m. with environmental temperature around 20°C). Huynh *et al.* conducted several dynamic tests on a lab-scaled prestressed concrete girder (Huynh *et al.* 2015). The temperatures were controlled in the range between 5.4 to 20.4°C. Vibration responses were measured to determine the modal parameters of the first two modes. They concluded that changes in the natural frequencies were small as temperatures varied up to 15°C. Here, as the temperature variations were negligible, the effects of temperature variations have not been considered.

5. The test results

To extract the modal parameters, the test data have been analyzed using the ARTeMIS Extractor version 3.43. The software of ARTeMIS is an open analyzer and post-processing platform for the

Table 1 Arrangement of the sensors for each test

Spans under test	Arrangement No.	Sensor numbers in Fig. 3	Reference sensor
Two side spans	#1	S1,S2,S3,S4,S5,S6,S7,S8,S9,S10,S11	S5
Two side spans	#2	S5,S12,S13,S14,S15,S16,S17,S18,S19,S20,S21,S22	S5
Middle spans	#1	Acc1, Acc2, Acc3, Acc4, Acc5	---

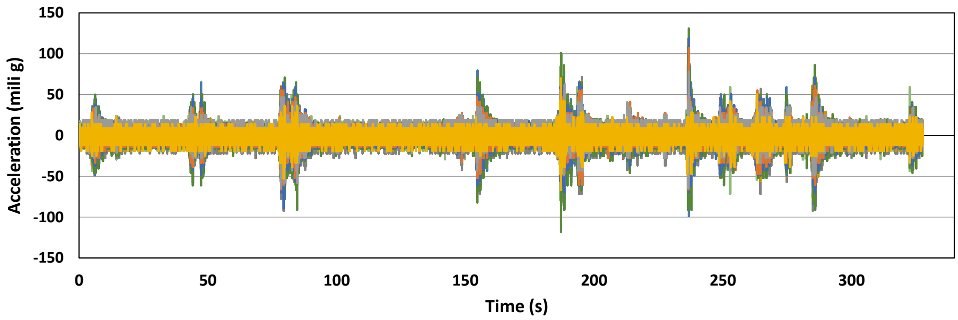


Fig. 6 Acceleration time history for 11 sensors in the first span in vertical direction

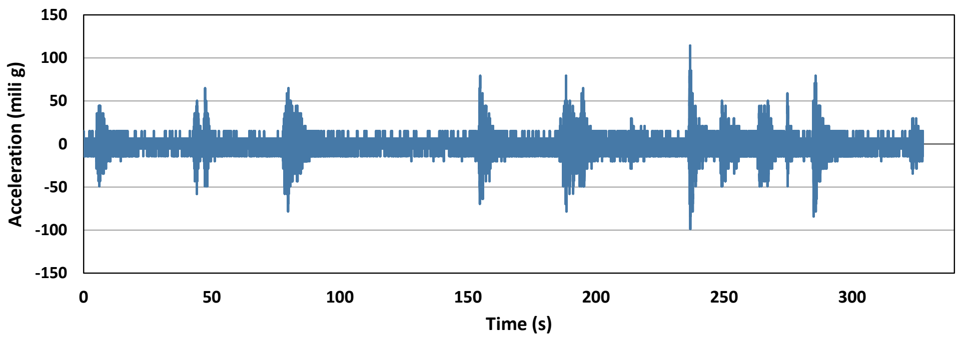
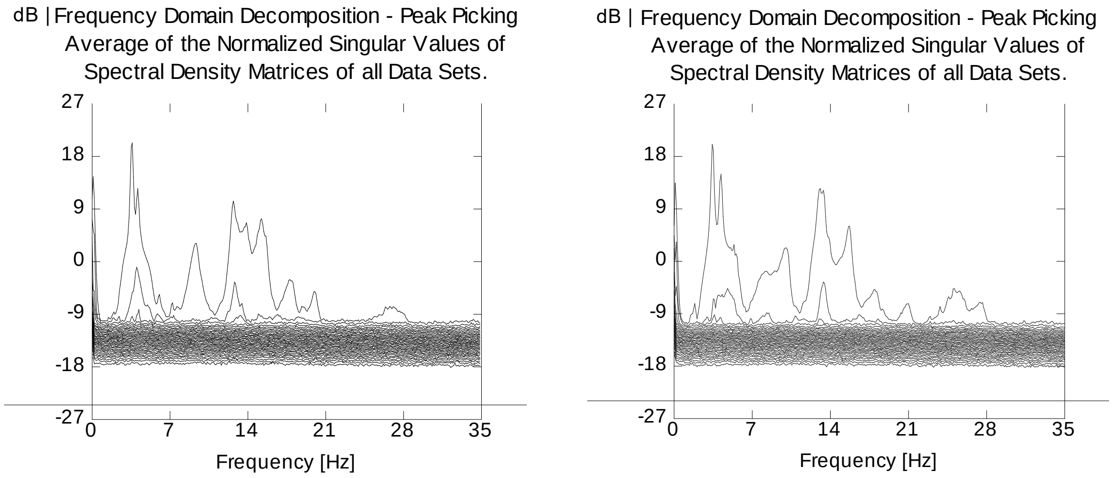


Fig. 7 Acceleration time history for sensor #S6 in the first span in vertical direction

data extracted from modal testing and VbSHM. After collecting the vibration data, the dynamic characteristics of the structure are determined via ARTeMIS in terms of natural frequencies, mode shapes, and damping ratios. Different identification techniques are employed by ARTeMIS from deterministic (e.g., frequency domain methods) or stochastic methods. ARTeMIS software has been used by many researchers for operational modal analysis (e.g., Brincker *et al.* 2001).

After carrying out the tests and processing the data, plots of frequency domain decomposition of power spectral density functions have been drawn. Here, data have been analyzed for estimating the spectral densities with 1,024 frequency lines and a frequency line spacing of 0.09766 Hz that has been achieved using an overlapping of 66.7% (Akbari *et al.* 2019).



(a) the north span (b) the south span

Fig. 8 Average of the singular values of spectral density matrices for the tests of two side spans

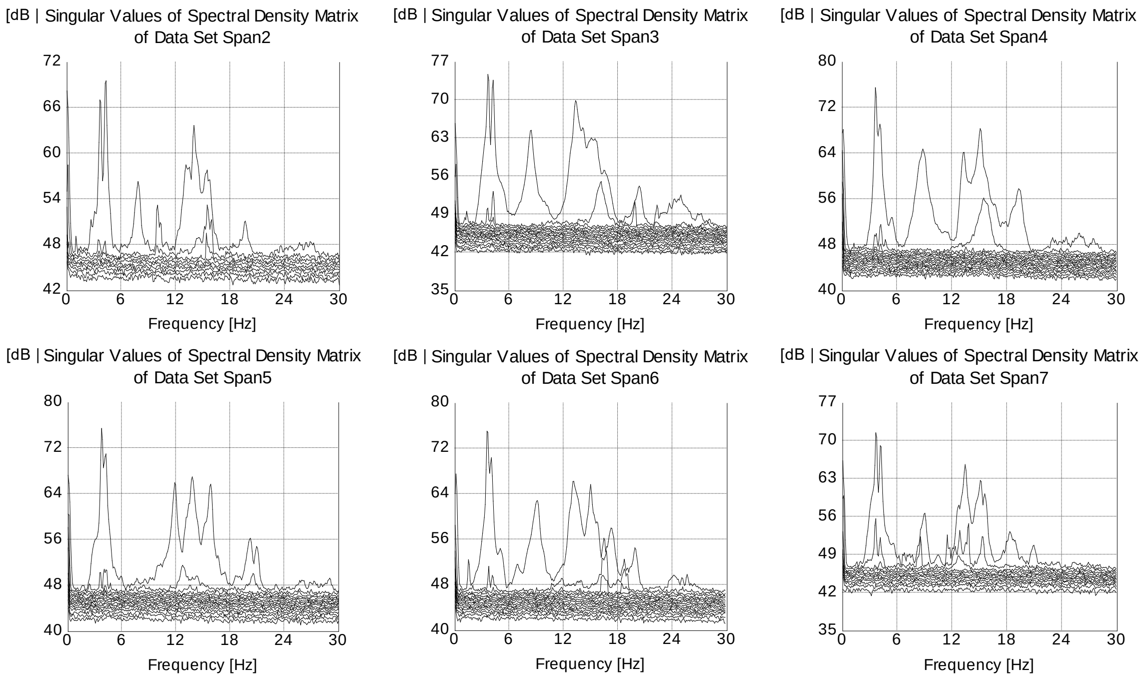


Fig. 9 Average of the singular values of spectral density matrices for the tests of middle spans

It should be noted here that all the measured signals collected from all the sensors have been used for the analyses. For each span of the two side spans, 22 roving sensors and one reference sensor have been used. For middle spans, 5 sensors have been used. The data from three main directions (i.e. longitudinal, transversal, and vertical directions) have been measured, however, the

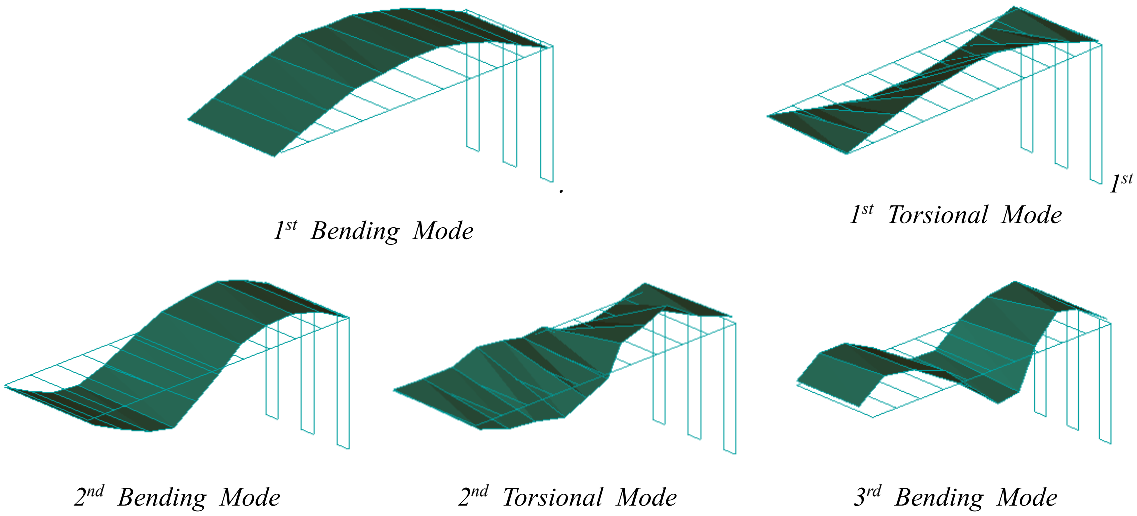


Fig. 10 Mode shapes identified from the tests carried out on each span of the bridge

data associated with the vertical direction has only been considered here. For each sensor, in the vertical direction, 65532 signals have been measured. For instance, the acceleration time history for eleven sensors of the first span in the vertical direction is shown in Fig. 6 and for sensor #S6 is shown in Fig. 7.

In Fig. 8, the average of the singular values of spectral density matrices for both arrangements of the tests of two side spans are shown. The results for the other six middle spans are shown in Fig. 9. Good agreements and clearness for the frequency peaks have been obtained.

The results of the tests for the first five natural frequencies and damping ratios extracted via the enhanced frequency domain decomposition (EFDD) technique are tabulated in Table 2. The modes shapes identified for the first north span are shown in Fig. 10. Similar mode shapes have been identified for other spans.

6. Finite Element (FE) modeling

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

Linear springs have been modeled for the elastomeric bearings with stiffness values calculated according to the bearing geometric and material characteristics (Akbari and Maalek 2009, Maalek *et al.* 2010, Akbari 2008). In general, the transverse and vertical stiffness of elastomeric bearings are calculated by $K_h = GA/h$ and $K_v = 6GS^2AK/(h(6GS^2 + K)$. G is the shear modulus of elastomer material, A is the cross-sectional area, h is the bearing's total thickness, K is the Bulk modulus of elastomer material, S is the shape factor defined as $S = A/(2h(L + W))$ in which L and W are length and width for rectangular bearings. 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.

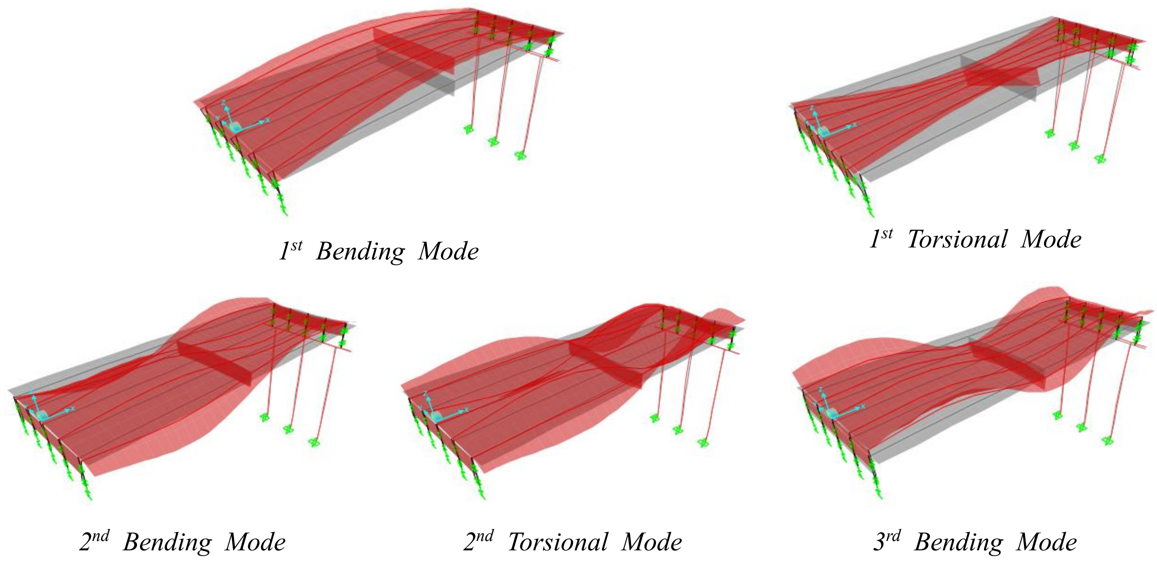


Fig. 11 Mode shapes obtained from the FE analyses

Table 2 The dynamic characteristics obtained from the tests for all the spans

Mode No.	Parameters	Span No. 1	2	3	4	5	6	7	8	Δf_{max}(%)*	CV (%)**
Mode-1 (1st Bending)	f (Hz)	3.51	3.61	3.61	3.61	3.71	3.51	3.61	3.41	+8.7	2.56
	Damping (%)	1.93	1.72	1.38	1.57	1.53	1.56	1.82	1.92	-	-
Mode-2 (1st Torsional)	f (Hz)	4.10	4.19	4.19	4.10	4.19	4.00	4.10	4.19	+4.7	1.68
	Damping (%)	2.15	2.15	1.75	1.58	1.53	1.57	1.57	2.53	-	-
Mode-3 (2nd Bending)	f (Hz)	15.23	15.04	15.43	15.04	15.82	14.94	15.04	15.63	+5.8	2.07
	Damping (%)	1.63	1.51	2.53	1.16	2.42	1.74	1.32	2.20	-	-
Mode-4 (2nd Torsional)	f (Hz)	20.02	19.34	20.31	19.24	20.21	19.82	20.9	20.9	+8.6	3.11
	Damping (%)	1.43	1.62	1.94	2.67	1.09	1.71	1.31	1.47	-	-
Mode-5 (3rd Bending)	f (Hz)	26.37	25.2	24.9	25.78	25.98	24.12	25.29	25.10	+9.3	2.76
	Damping (%)	1.76	1.42	3.57	1.86	1.91	1.35	1.28	1.69	-	-

* $\Delta f_{max} = (f_{max} - f_{min})/f_{min}$
** CV= Coefficient of Variation;

In the bridge under study, two types of elastomeric bearings have been used and modeled, as defined in Table 3. The bearings used in the position of side restrainers are named here as "fixed" and the others are named as "free". For the fixed bearings, simple hinge support has been assumed in the longitudinal and transversal directions. Finally, link elements named "Rubber Isolator" has been defined in the SAP2000 model for the linear springs with the stiffness values calculated according to the above relations and the assumptions given in Table 3.

Side restrainers have been modeled via restraining the associated degree of freedom for rotation

Table 3 Dimensions and characteristics of the elastomeric bearings

Type/Name	Length (mm)	Width (mm)	Thichness (mm)	G (MPa)	K (MPa)
fixed	200	400	50	1	2000
free	300	400	74	1	2000

Table 4 The natural frequencies obtained from the FE analyses for all the spans

Mode No.	Span No.								$\Delta f_{max}(\%)^*$	CV (%)**
	1	2	3	4	5	6	7	8		
Mode-1 (1st Bending)	3.60	3.47	3.46	3.47	3.48	3.48	3.48	3.49	+4.2	1.27
Mode-2 (1st Torsional)	4.01	3.82	3.81	3.83	3.83	3.82	3.82	3.84	+5.2	1.72
Mode-3 (2nd Bending)	15.09	15.01	14.90	14.74	14.49	14.56	14.56	14.90	+2.3	1.53
Mode-4 (2nd Torsional)	19.19	19.17	19.17	19.18	19.18	19.14	19.14	19.14	+0.2	0.10
Mode-5 (3rd Bending)	22.73	21.99	21.86	22.07	22.26	22.34	22.34	22.87	+4.6	1.61

* $\Delta f_{max} = (f_{max} - f_{min})/f_{min}$
** CV= Coefficient of Variation;

or displacement. The FE results for the first five natural frequencies and mode shapes are shown in Fig. 11 and Table 4.

7. Analysis of the results

According to the aims of the study, two main questions exist related to the test results. The first one is, how much the geometry and material properties are consistent between the spans. On the other hand, how much the bridge deck construction is homogeneous from span to span? The second question is how much the geometry and material properties are consistent with the design drawings or construction documents. To investigate these questions, the results of Table 2 have been analyzed.

The results of damping ratios in Table 2 show that the values are lesser than 5% in all the identified modes. This point in parallel with considering the fact of continuity of the identified mode shapes reveals that, after 35 years of operation, there are no signs of high energy dissipation in the bridge deck. This point is a criterion for bridge health, as visual inspections have confirmed this matter.

The results of Table 2 for the natural frequencies of different spans show that there are minor differences between the values of the natural frequencies among all the spans. The maximum value is around 9.3% in the third bending mode between the first and the sixth spans.

Kim *et al.* compared the results of numerical analyses of a damaged beam in different damage scenarios using mode-shape-based and frequency-based damage detection methods (Kim *et al.* 2003). They concluded that structural damage causes relatively small changes in modal parameters. Although the presented 9.3% change in the modal frequencies could not be minor, it was appeared once for two spans only (the first and sixth spans) for the 3rd bending mode. This may due to environmental effects, calculation error, accuracy of the tests, numbers of signals used for these two spans, and may be due to minor damages. Visual inspections have been done to support the results.

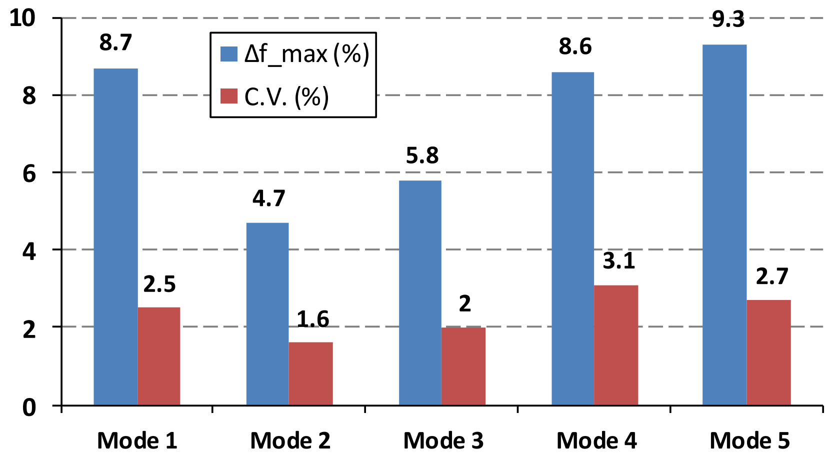


Fig. 12 Comparison of maximum coefficient of variation and maximum difference of the natural frequencies

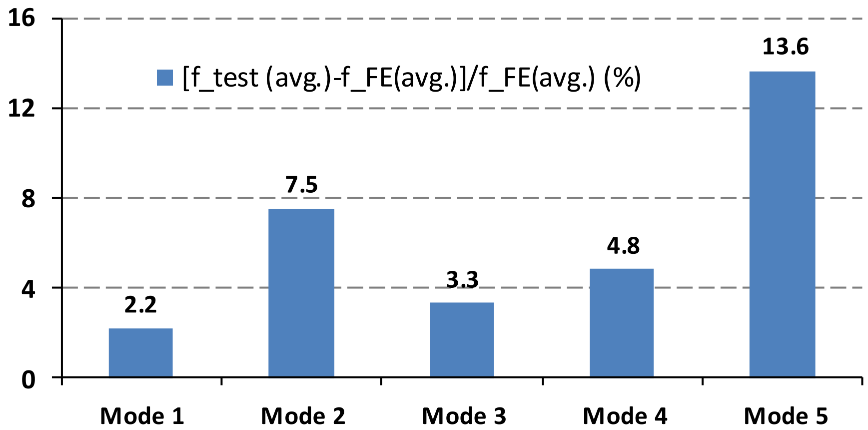


Fig. 13 Comparison of the differences between the average values of the natural frequencies from the tests and the FE analyses

The maximum coefficient of variation is around 3.1%. This point in parallel with considering the values of the damping ratios shows that the geometry and material properties of all the spans are similar and the deck for all eight spans is homogeneous. In addition, no significant event has happened regarding the stiffness degradation during the service life of the bridge. In Fig. 12, the maximum coefficient of variation and maximum difference on the natural frequencies are compared.

A comparison of the differences between the average values of the natural frequencies of all the spans obtained from the tests and the associated ones of the FE analyses shows that there are minor differences between the construction/as-built drawings and the design drawings. This point is shown in Fig. 13. 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 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.

8. 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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