Steel and Composite Structures, Vol. 10, No. 6 (2010) 541-561
DOI: http://dx.doi.org/10.12989/scs.2010.10.6.541

Finite element model updating of an arch type steel laboratory bridge model using semi-rigid connection

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(Received May 4, 2010, Accepted November 03, 2010)

Abstract. This paper presents finite element analyses, experimental measurements and finite element model updating of an arch type steel laboratory bridge model using semi-rigid connections. The laboratory bridge model is a single span and fixed base structure with a length of 6.1 m and width of 1.1m. The height of the bridge column is 0.85 m and the maximum arch height is 0.95 m. Firstly, a finite element model of the bridge is created in SAP2000 program and analytical dynamic characteristics such as natural frequencies and mode shapes are determined. Then, experimental measurements using ambient vibration tests are performed and dynamic characteristics (natural frequencies, mode shapes and damping ratios) are obtained. Ambient vibration tests are performed under natural excitations such as wind and small impact effects. The Enhanced Frequency Domain Decomposition method in the frequency domain and the Stochastic Subspace Identification method in the time domain are used to extract the dynamic characteristics. Then the finite element model of the bridge is updated using linear elastic rotational springs in the supports and structural element connections to minimize the differences between analytically and experimentally estimated dynamic characteristics. At the end of the study, maximum differences in the natural frequencies are reduced on average from 47% to 2.6%. It is seen that there is a good agreement between analytical and experimental results after finite element model updating. Also, connection percentages of the all structural elements to joints are determined depending on the rotational spring stiffness.

Keywords: ambient vibration test; dynamic characteristics; enhanced frequency domain decomposition; laboratory bridge model; rotational spring; semi-rigid connection; stochastic subspace identification.

1. Introduction

The knowledge of the dynamic characteristics (natural frequencies, mode shapes, and damping ratios) of engineering structures is important to understand dynamic performance of these structures during its operation. Finite element modelling is used as a powerful tool to estimate the dynamic behaviour of a structure, but it is not sufficient alone because finite element model is constructed on the basis of highly idealized engineering design and that may or may not truly represent all the physical aspects of an actual structure. So, dynamic characteristics can be identified experimentally either by ambient vibration tests or forced vibration tests. However, there is a discrepancy between both results due to the fact that there are a number of uncertain parameters in the finite element model, when the

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experimentally and analytically identified dynamic characteristics are compared with each other. Therefore, it is considered that finite element model should be updated by changing uncertain modelling parameters such as material and section properties or boundary conditions in order to eliminate differences as much as possible. This process is customarily termed as model updating (Modak *et al*. 2002, Lu *et al*. 2007, Altunýþýk *et al*. 2010, Wang *et al*. 2010).

The choice of the updating parameters is critical to improve the modelling of the structures. Material properties of the elements such as modulus of elasticity and mass density, geometric properties such as area and moment of inertia of the cross-section, nodal positions, or boundary conditions such as beam-to-beam or beam-to-column connection stiffness may be chosen as updating parameters (Zapico *et al*. 2003). Especially in laboratory models than constructed civil engineering structures, material and section properties can be considered as determined. Because the laboratory models generally involves factory fabricated elements. However, structural element connections of steel structures include indeterminacy depending on connection details and workmanship defects. Steel structural models including welded connections are studied by experimental modal analysis and revealed that this type of connections was semi-rigid (Kohoutek 2000). Therefore, these connections should be considered neither pinned nor rigid connections.

The updating process typically consists of manual tuning and then automatic model updating using some specialised software. The manual tuning involves manual changes of the model geometry and modelling parameters by trial and error, guided by engineering judgement. The aim of this is to bring the analytical model closer to the experimental one (Bayraktar *et al*. 2007, Zivanovic *et al*. 2007, Sevim *et al*. 2010). In this study, the manual tuning procedure is used for finite element model updating.

In the literature, many investigators built different type laboratory bridge models and used for different purposes such as analytical modelling, experimental measurements and finite element model updating. Sanayei and DiCarlo (2009) performed finite element model updating of a scaled bridge model using measured response data. Four impact hammers and eight accelerometer locations were pointed in the measurements. The measured excitations and responses were used to identify mode shapes for several natural frequencies. Zhu and Cheng (2008) carried out the test and analyses of a double arch steel gate under cyclic loading. Stress analysis, cyclic behaviour and bearing capacity of the gate model were determined. Yang (2007) modelled long span bridge model in the laboratory conditions to determine the seismic response including spatial variation of seismic waves in his doctorate thesis. Both shake table model tests for experimental identification and finite element analysis for analytical identification were carried out. Bilello *et al*. (2004) studied about experimental investigation of a small scale bridge model under a moving mass. The analysis was based on the continuous Euler-Bernoulli beam theory. A small scale model was designed to satisfy both static and dynamic similitude with a selected prototype bridge structure. Zapico *et al*. (2003) studied the finite element analysis and experimental measurements of the small scale bride model. Forced vibration tests using shaking table were conducted on the bridge deck and dynamic characteristics were attained.

The objective of this study is to obtain a comprehensive understanding of the dynamic characteristics and update the finite element model of an arch type steel laboratory bridge model. This is achieved by the combination of results identified from finite element analysis and ambient vibration tests. 3D finite element model of the bridge is analysed to determine analytical dynamic characteristics based on the existing drawings. Then ambient vibration tests are performed and experimental dynamic characteristics are extracted from the Enhanced Frequency Domain Decomposition (EFDD) and Stochastic Subspace Identification (SSI) methods. Finite element model of the bridge is updated by changing connection stiffness to reduce the differences between experimental and analytical results.

2. Formulation

2.1. Ambient vibration test

Ambient excitation does not lend itself to Frequency Response Function (FRFs) or Impulse Response Function (IRFs) calculations because the input force is not measured in an ambient vibration test. Therefore, a modal identification procedure will need to base itself on output-only data (Ren *et al.* 2004). There are several modal parameter identification methods available such as Enhanced Frequency Domain Decomposition (EFDD) and Stochastic Subspace Identification (SSI). These methods are developed by improvements in computing capacity and signal processing procedures. In this study, EFDD method in the frequency domain and SSI method in the time domain are used to extract dynamic characteristics.

2.1.1 Enhanced Frequency Domain Decomposition (EFDD) method

The idea of the EFDD method is to perform an approximate decomposition of the system response into a set of independent (Brincker *et al.* 2000). The decomposition occurs by simply decomposing each of the estimated spectral density matrices, which show the singular values are the estimates of the auto spectral density of the SDOF systems, and the singular vectors are the estimates of the mode shapes.

The EFDD method is often used in civil engineering practice for ambient vibration measurements due to its implementation simplicity and its speed. In this method, the relationship between the unknown input and the measured responses can be expressed as (Bendat and Piersol 2004)

$$[G_{\mathbf{yy}}(j\omega)] = [H(j\omega)]^*[G_{\mathbf{xx}}(j\omega)][H(j\omega)]^T \tag{1}$$

where $G_{\mathbf{xx}}$ is the Power Spectral Density (PSD) matrix of the input. $G_{\mathbf{xx}(jw)}$ is the rxr Power Spectral Density (PSD) matrix of the input, r is the number of inputs, $G_{\mathbf{yy}(jw)}$ is the mxm PSD matrix of the responses, m is the number of responses, $H(jw)$ is the mxr Frequency Response Function (FRF) matrix, and * and superscript T denote complex conjugate and transpose, respectively. Solution of the Eq. (1) is given detail in the literature (Felber 1993, Brincker *et al.* 2000, Peeters 2000, Rainieri *et al.* 2007).

2.1.2 Stochastic Subspace Identification (SSI) method

SSI is an output-only time domain method that directly works with time data, without the need to convert them to correlations or spectra. The method is especially suitable for operational modal parameter identification.

The model of vibration structures can be defined by a set of linear, constant coefficient and second-order differential equations (VanOverschee and DeMoor 1996, Peeters 2000, Juang and Phan 2001)

$$M\ddot{U}(t) + C_2\dot{U}(t) + KU(t) = F(t) = B_2u(t) \tag{2}$$

where M, C_2, K are the mass, damping and stiffness matrices, $F(t)$ is the excitation force, and $U(t)$ is the displacement vector depending on time t. Observe that the force vector $F(t)$ is factorised into a matrix B_2 describing the inputs in space and a vector $u(t)$. Although Eq. (2) represents quite closely the true behaviour of a vibrating structure, it is not directly used in SSI methods. So, the equation of dynamic equilibrium (2) will be converted to a more suitable form: the discrete-time stochastic state-space model. Solution of the Eq. (2) is given detail in the literature (Juang 1994, Yu and Ren 2005).

2.2 Semi-rigid connections

Structural elements and joints are modelled considering some idealizations. The joints of idealized frame elements are assumed to be constituted by ideally rigid connections. However, another assumption is that structural members of truss systems have ideally pinned connection at joints. Actually, structural connections should be named according to their moment-rotation curves. These curves are usually derived by fitting suitable curves to the experimental data. Various types of M-θ_r models have been developed as described by Chen and Lui (1991). As seen from M-θ_r curves given in Figure. 1 the moment (M) is depended on a function of relative rotation between structural members connected to the same joint. The finite element analyses are mostly performed assuming semi-rigid connections as rigid or pinned connections for simply calculation.

Connection flexibility is defined by various methods. To obtain an initial opinion on stiffness of rotational springs, use the modulus of elasticity (E), moment of inertia (I) and length (L) of related beam with constant cross-section is very effective and understandable approach. Stiffness matrix of a beam in local coordinates can be written using these attributes of this beam as follows (McGuire *et al*. 1999).

$$[k] = \begin{bmatrix} \frac{12EI}{L^3}\theta_1 & \frac{6EI}{L^2}\theta_2 & -\frac{12EI}{L^3}\theta_1 & \frac{6EI}{L^2}\theta_3 \\ \frac{6EI}{L^2}\theta_2 & \frac{4EI}{L}\theta_4 & -\frac{6EI}{L^2}\theta_2 & \frac{2EI}{L}\theta_5 \\ -\frac{12EI}{L^3}\theta_1 & -\frac{6EI}{L^2}\theta_2 & \frac{12EI}{L^3}\theta_1 & -\frac{6EI}{L^2}\theta_3 \\ \frac{6EI}{L^2}\theta_3 & \frac{2EI}{L}\theta_5 & -\frac{6EI}{L^2}\theta_3 & \frac{4EI}{L}\theta_6 \end{bmatrix} \tag{3}$$

where $\theta_{1\text{-}6}$ are the coefficients and given as follows,

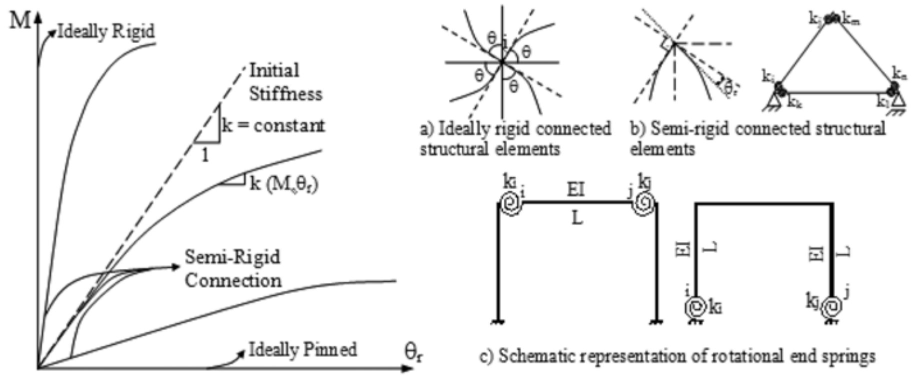


Fig. 1 Structural connections

$$\theta_1 = \frac{\alpha_i + \alpha_j + \alpha_i\alpha_j}{4(3+\alpha_j) + \alpha_i(4+\alpha_j)} \tag{4a}$$

$$\theta_2 = \frac{\alpha_i(2+\alpha_j)}{4(3+\alpha_j) + \alpha_i(4+\alpha_j)} \tag{4b}$$

$$\theta_3 = \frac{\alpha_j(2+\alpha_i)}{4(3+\alpha_j) + \alpha_i(4+\alpha_j)} \tag{4c}$$

$$\theta_4 = \frac{\alpha_i(3+\alpha_j)}{4(3+\alpha_i) + \alpha_j(4+\alpha_i)} \tag{4d}$$

$$\theta_5 = \frac{\alpha_i\alpha_j}{4(3+\alpha_i) + \alpha_j(4+\alpha_i)} \tag{4e}$$

$$\theta_6 = \frac{\alpha_j(3+\alpha_i)}{4(3+\alpha_i) + \alpha_j(4+\alpha_i)} \tag{4f}$$

Here, α_i and α_j are the stiffness indexes and can be used to obtain rotational spring stiffness as follows,

$$k_i = \alpha_i \frac{EI}{L} \tag{5a}$$

$$k_j = \alpha_j \frac{EI}{L} \tag{5b}$$

where, k_i and k_j are the rotational spring stiffness at i and j ends of the beam, respectively, and those change in 0-∞ range.

Semi-rigid connection may also be identified by connection percentage. Then, the parameters of θ_i can be written as follows (Chen and Lui 1991, Kartal 2004, Filho *et al.* 2004).

$$\theta_1 = \frac{r_i + r_j + r_{ij}}{3} \tag{6a}$$

$$\theta_2 = \frac{2r_i + r_{ij}}{3} \tag{6b}$$

$$\theta_3 = \frac{2r_j + r_{ij}}{3} \tag{6c}$$

$$\theta_4 = r_i \tag{6d}$$

$$\theta_5 = r_{ij} \tag{6e}$$

$$\theta_6 = r_j \tag{6f}$$

where, r_i, r_j and r_{ij} are the correction factors and obtained as follows,

$$r_i = \frac{3\nu_i}{4-\nu_i\nu_j} \tag{7a}$$

$$r_j = \frac{3\nu_j}{4-\nu_i\nu_j} \tag{7b}$$

$$r_{ij} = \frac{3\nu_i\nu_j}{4-\nu_i\nu_j} \tag{7c}$$

Here, ν_i and ν_j are the fixity factors and represent the semi-rigid connection as percentage. If the Eqs. (4) and (6) are equalized, a set of equations, which provides a direct relation with initial spring stiffness and connection percentage, is achieved as presented in Eqn 8 (Sekulovic *et al*. 2002),

$$k_{i,j} = \frac{3EI\nu_{i,j}}{(1-\nu_{i,j})L} \tag{8}$$

Then fixity factors can be given as follows (Monforton and Wu 1963),

$$\nu_{i,j} = \frac{k_{i,j}L}{3EI + k_{i,j}L} \tag{9}$$

3. Finite element modelling

In this paper, arch type steel laboratory bridge model is selected as an example. The main members of the bridge include main arches, columns, web members, beams and bracings. The arch is in the shape of a parabola. The bridge model is a single span, simply supported structure with a 6m length and 1.1 m width. Height of the bridge column is 0.85 m and the maximum arch height is 0.95 m. This model has 6 web members in the vertical direction to transfer the loads to the beams. Also, there are two beams with length of 6 m in the longitudinal direction to transfer the loads to the supports. Lateral stability is provided by transverse bracing at 1.5m intervals including both ends. In addition, the structure is doubly symmetric. The geometrical properties of the bridge are shown in Fig. 2.

In the construction of the system, approximately, 171 kgf steel and 150 bolts have been used. Actually, steel bridge model is about 149 kgf but the weight of the flanges and bolts are considered as 22 kgf. Three workers constructed the bridge model by working ten days. Some pictures from construction process are shown in Fig. 3. Hollow sections are selected for the most desirable in terms of modal frequencies, deflections, rotations, stresses and strains that are representative for typical short to medium span footbridges and highway bridges. As a boundary condition, bridge columns are fixed to

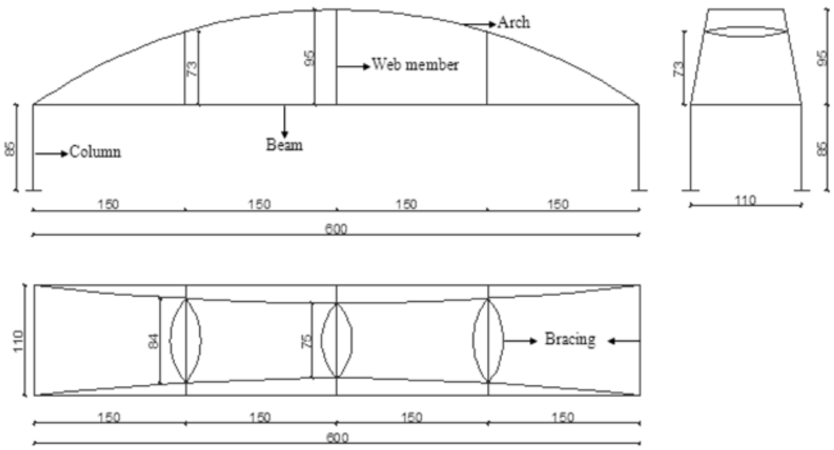


Fig. 2 Geometrical properties of the laboratory bridge model (unit: cm)

Fig. 3 Some pictures from construction process

the ground. Section and material properties of the bridge model are summarized in Table 1.

Three dimensional finite element model of the bridge model is constructed using the SAP2000 software (Fig. 4) (SAP2000 1998). This program can be used for linear and non-linear, static and dynamic analyses of the 3D model of the structure. In this paper, the program is used to determine the dynamic characteristics based on its physical and mechanical properties.

The arch type steel bridge is modelled as a space frame structure with 3D prismatic beam elements

Table 1 Section and material properties of the laboratory bridge model

Structural Element	Section Properties			Material Properties	
	Type	Diameter (mm)	Thickness (mm)	Modulus of Elasticity (N/m^2)	Mass per Unit Volume (kg/m^3)
Arch	Hollow	76	3.0	2.06E11	7850
Column	Hollow	76	3.0	2.06E11	7850
Web member	Hollow	48	2.5	2.06E11	7850
Beam	Hollow	48	2.5	2.06E11	7850
Bracing	Hollow	27	2.0	2.06E11	7850

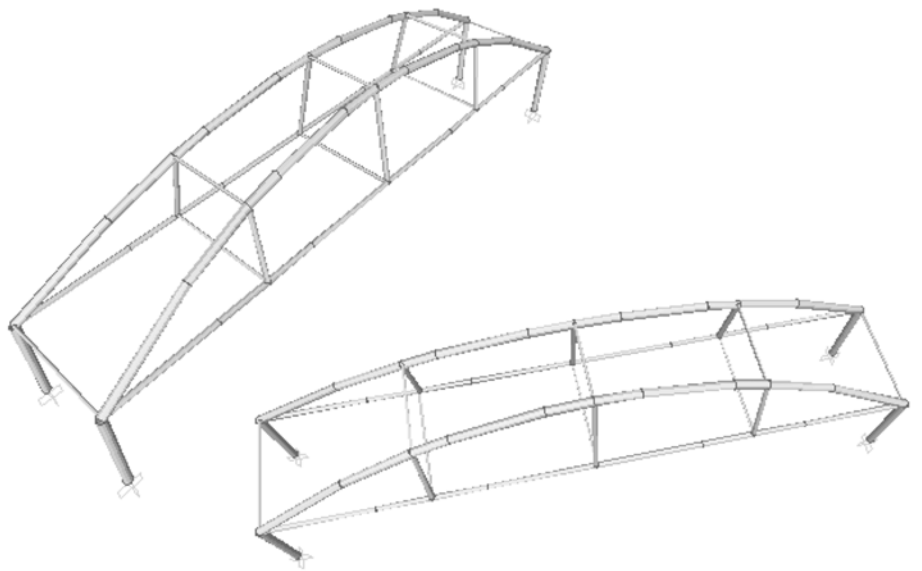


Fig. 4 3D finite element model of the steel bridge model

which have two nodes and each end node has six degrees of freedom: three translations along the global axes and three rotations about its local axes. The aim of the construction a detailed model is to be able to simulate the dynamic behaviour of the structure as well as possible. The key modelling assumptions are as follows: the main arch, columns and its transverse stiffeners are modelled using 3D beam elements, the supports of the bridge column are modelled as fully fixed, and the structural member connections to joints are modelled as rigid.

Natural frequencies and corresponding vibration modes are very important dynamic properties and have significant effect on the dynamic performance of structures. The first four natural frequencies of the bridge model are attained which range between 7.31 and 24.61Hz. The first four vibration mode shapes of the bridge model are shown in Fig. 5.

4. Ambient vibration tests

Ambient vibration tests are performed to determine the natural frequencies, mode shapes and damping ratios of the bridge model. Because of the fact that there are some differences between dynamic characteristics of the laboratory models and large scale models, the selection of the measurement equipments, properties and setups are very important.

In the ambient vibration tests, B&K 3560 data acquisition system with 17 channels, B&K 8206-002 type small impact hammer, B&K 4507-B005 type uni-axial and B&K 4506-B003 type tri-axial accelerometers are used. The uni-axial accelerometers have 1,000 mV/g sensitivity and 0.4-6000 Hz frequency range. The tri-axial accelerometers have 500 mV/g sensitivity and 0.3-2000 Hz frequency range. The measurements are performed on three test setups and excitations are provided from wind and small impact effects. Since the intended number of measurements is larger than the number of channels and sensors available, measurements are performed in some steps and the signals are

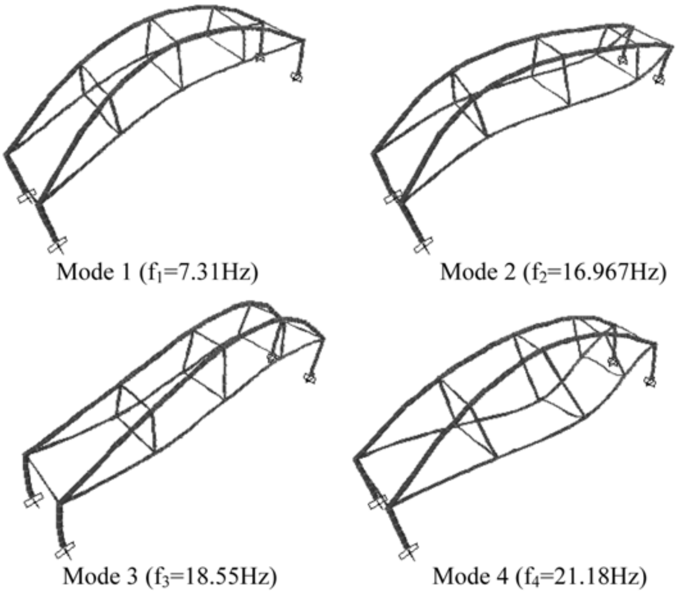
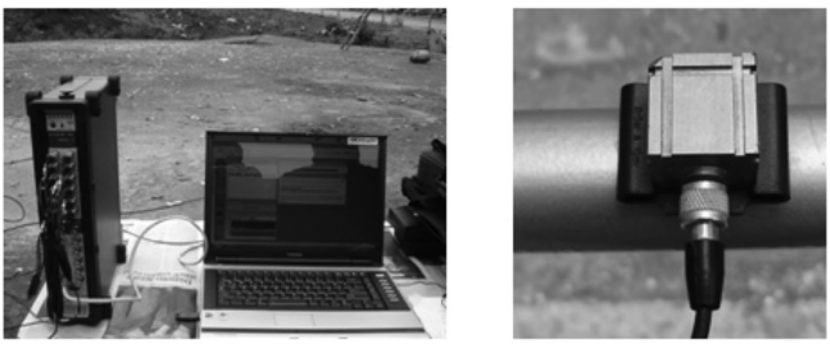


Fig. 5 Analytically determined first four mode shapes of the bridge model

incorporated using a uni-axial reference accelerometer. Signals obtained from the vibration tests are transferred into the PULSE (PULSE 2006) Lapshop software. For parameter estimation from the Ambient Vibration System data, the Operational Modal Analysis (OMA) software is used (OMA 2006).

The picture of the data acquisition system and accelerometers are given in Fig. 6. To identify the mode shapes, natural frequencies and damping ratios, structural responses at sufficient locations in the vertical, horizontal and lateral directions have been measured. Accelerometer locations are given in Fig. 7 and summarized in Table 2.

The Operational Modal Analysis is carried out by using the EFDD method in the frequency domain and SSI method in the time domain. In the EFDD method, dynamic characteristics are obtained from each vibration signals as a singular values. Its mean that number of measurement test setup does not affect the peaks in the singular values of spectral density matrices. But in the SSI method, dynamic



a) Data acquisition system b) Accelerometer

Fig. 6 The pictures from the ambient vibration test

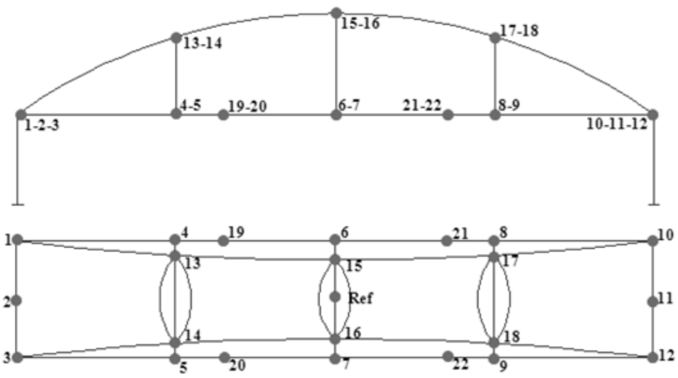


Fig. 7 Accelerometer locations on the bridge

Table 2 Measurement test setup and accelerometer locations

Test Setup	Accelerometer Points	Measurement Steps	Ref. Point	Frequency Span	Total Duration
1st	1,19,21 and 10,11,12 22,20,3	Three Step	Ref	0-50 Hz	9 min
2nd	1,6,10 and 3,7,12	Two Step	Ref	0-50 Hz	6 min
3rd	4,6,8 and 13,15,17 5,7,9 and 14,16,18	Four Step	Ref	0-50 Hz	12 min

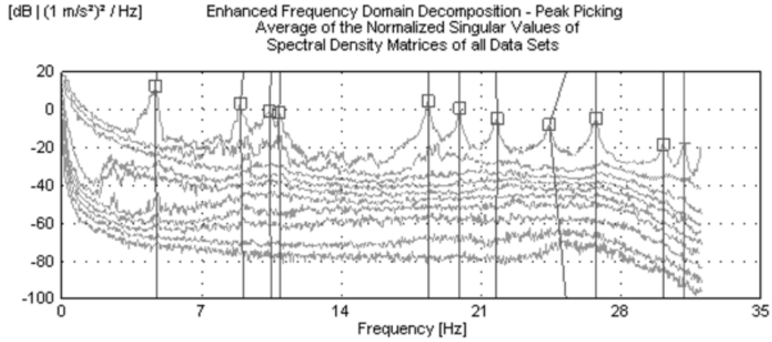
characteristics are obtained from collection of all vibration signals as a singular values. Its mean that number of measurement steps in each measurement test setup using references accelerometers affects the stabilization behaviour. Singular values of spectral density matrices, average of auto spectral densities, stabilization diagrams of estimated state space models and select-link modes across data sets of measurement test setups attained from vibration signal using EFDD and SSI techniques are shown in Figs. 8-10.

The first four mode shapes obtained from experimental measurement test setups are given in Fig. 11. Natural frequencies obtained from the OMA for all test setups are given in Table 3.

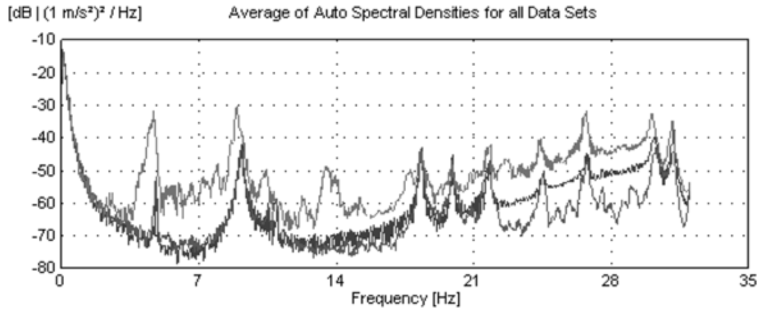
5. Finite element model updating

When the analytically and experimentally determined natural frequencies and mode shapes are compared with each other, it is seen that there is inconsistency between both results. Analytical natural frequencies are higher than experimental ones. Mode shapes in the same mode number are different. It is considered that these differences are based on some uncertainties in the structural geometry, material properties and boundary conditions. Because of these reasons, the finite element model of the bridge model must be updated. Material and section properties of all structural elements of the bridge model are the same, because they are fabricated sections. But, the bridge model is constructed with some pieces to easily install again and again and flanges are used in the connections of the structural elements. In the initial finite element analysis, rigid connections to joints are considered for bridge frame elements. So, finite element model of the bridge is updated considering boundary conditions for element connection rigidity.

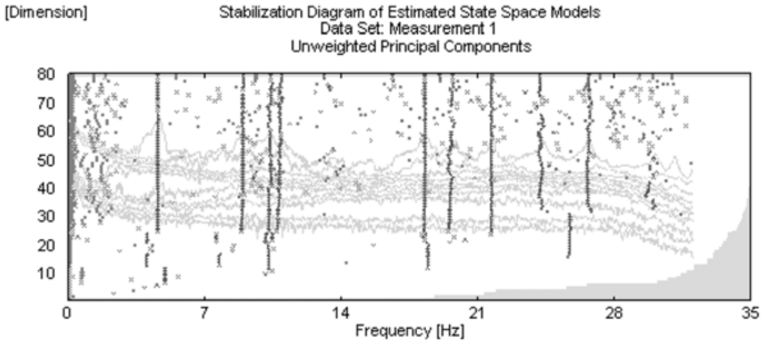
Bridge model has various connection types. The details of the structural element connections to joints are given in Fig. 12. The joints modelled in the finite element model of the bridge are given in Figure 13.



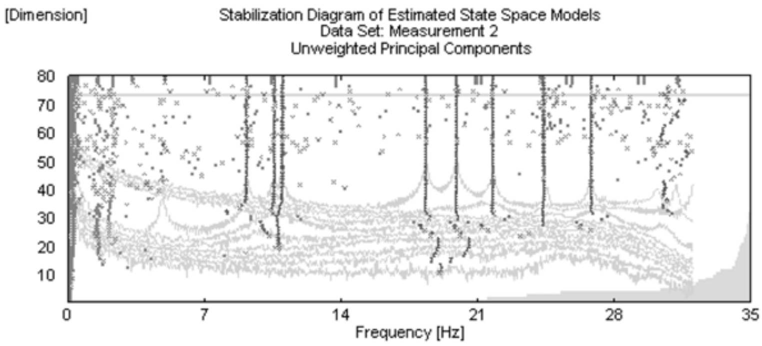
a) Singular values of spectral density matrices



b) Average of auto spectral densities

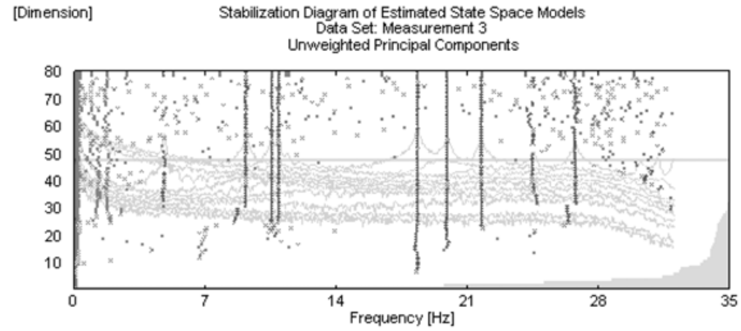


c) Stabilization diagrams of estimated state space models for the first step

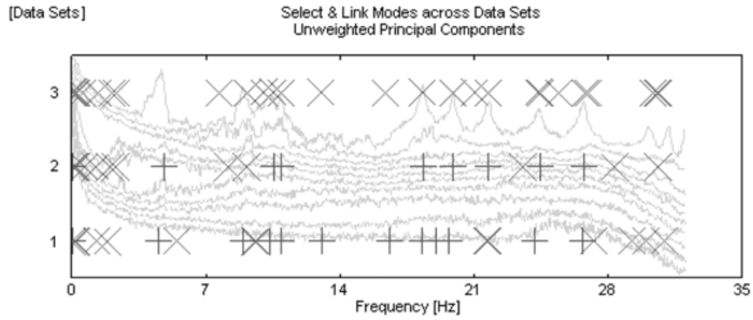


d) Stabilization diagrams of estimated state space models for the second step

Fig. 8 Dynamic characteristics obtained from first test setup using EFDD and SSI methods



e) Stabilization diagrams of estimated state space models for the third step



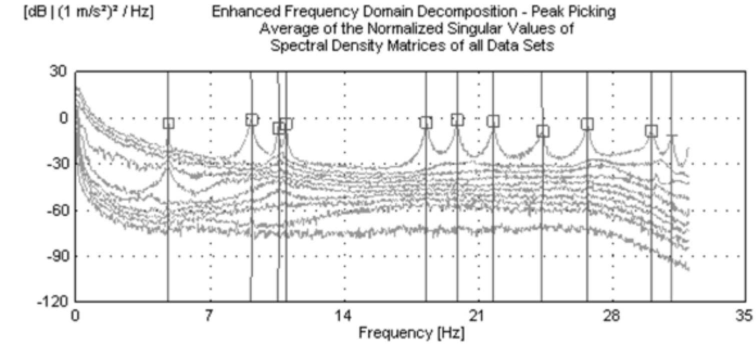
f) Select-link modes across data sets

Fig. 8 Continued

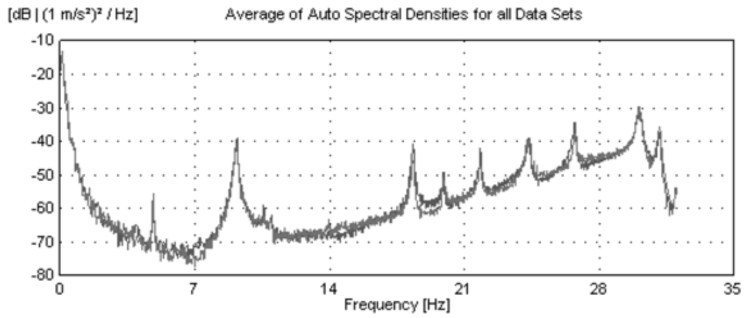
The corresponding structural element connections to joints are considered as semi-rigid. The values of linear rotational spring stiffness defined at the end of the hollow bars are determined using the parameters given in Table 4. Each type of the connections at joints of the bridge has different abilities. The arch and beam element connections are analogous and they have the same rotational spring stiffness about global X and Y axes, because the arch and beam element connections are constituted with bolts in local x axis. However bracings and web member connections have different connection properties in local y and z axes. Therefore, it is considered in the finite element model that bracings and web member connections have different spring coefficients in local y and z axes unlike arch and beam elements.

Natural frequencies and mode shapes obtained from analytical and experimental modal analyses after finite element model updating are given in Table 5 and Fig. 14, respectively. When Table 5 and Fig. 14 are examined, it is clearly seen that there is a good agreement between analytical and experimental results after finite element model updating.

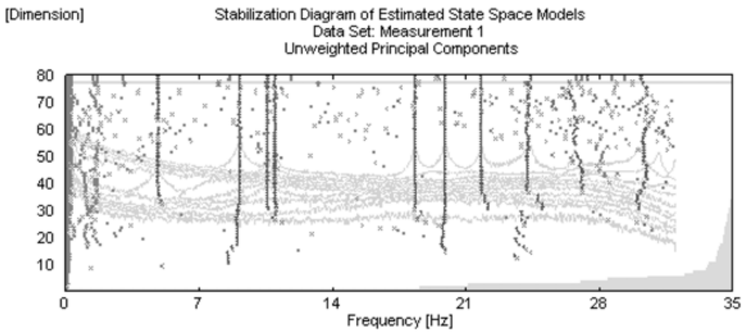
The bridge model was evaluated according to aesthetical, load bearing capacity, quick installation and light weight categories in the Design and Construct Steel Bridge Competition in Turkey. The competition was organized by Boğaziçi University in 22-25 April 2009. Fourty different university teams were attended to this competition. First day, the bridges were evaluated in the aesthetical appearance. Second day, quick installation and light weight categories were assessed. The bridge model, which forms the basis of this study, was installed in the 21 minute and its total weight was 171 kgf. Third day, load bearing capacities of the bridges were determined. 1,000 kgf and 250 kgf were implemented on the middle span and left side span of the bridges and also vertical displacements of the middle points of the bridges were measured. The vertical displacement (U_{yc}) in the middle span of the bridge model used in



a) Singular values of spectral density matrices



b) Average of auto spectral densities

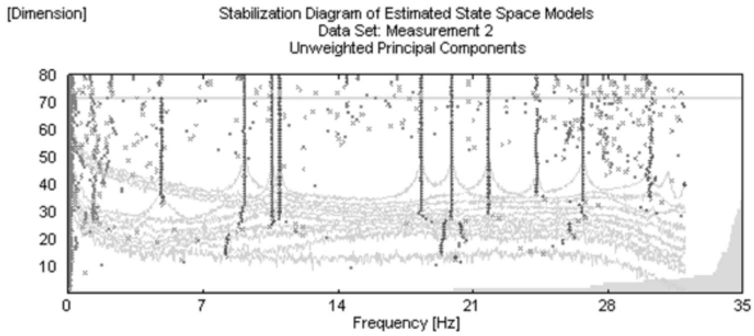


c) Stabilization diagrams of estimated state space models for the first step

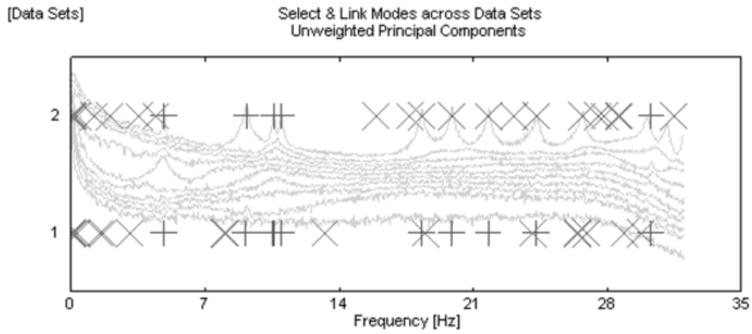
Fig. 9 Dynamic characteristics obtained from second test setup using EFDD and SSI method

this study was measured as 6.6 mm. This model was selected as winner between fourty universities considering all categories. Some pictures from the competition are given in Fig. 15.

Static analysis is carried out considering 1,250 kgf load (1,000 kgf on the main span and 250 kgf on the left side span) which is applied to the bridge model to determine the finite element model updating effect on the bridge response. The maximum vertical displacement (U_{yb}) obtained from the middle point of the bridge is determined as 4.0 mm in the initial finite element analysis. However, the maximum vertical displacement (U_{ya}) is obtained 6.0 mm after model updating. Therefore it can be stated that the finite element modelling including semi-rigid connections is successfully applied to this bridge.



d) Stabilization diagrams of estimated state space models for the second step



e) Select-link modes across data sets

Fig. 9 Continued

6. Conclusions

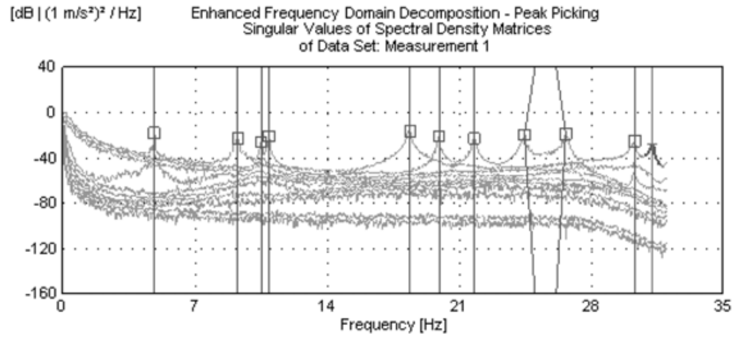
This paper presents finite element analyses, experimental measurements and finite element model updating of an arch type steel laboratory bridge model using semi-rigid connections. The finite element model of the bridge is constituted by SAP2000 program. Experimental measurements are performed using ambient vibration tests under natural excitations such as wind and small impact effects. Finite element model of the bridge is updated using linear elastic rotational springs in the supports and structural element connections. The following observations can be deduced from the study:

· The first four mode shapes, which range between 7-22 Hz, are attained analytically from the initial finite element model of the bridge. If the first four mode shapes are taken into consideration, these modes can be classified into lateral and longitudinal modes.

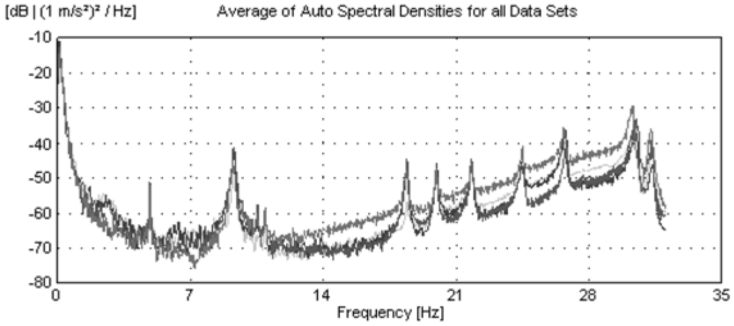
· In the ambient vibration test, three different measurements are performed on the bridge model. The first four natural frequencies are attained experimentally which range between 4-11 Hz. Observed mode shapes can be basically arranged as lateral and longitudinal modes. It is seen that there is a good agreement between all three test results obtained from EFDD and SSI methods.

· When comparing the analytical and experimental results, it is clearly seen that there are some differences in the natural frequencies in which analytical frequencies are higher than experimental frequencies. Also, mode shapes in the same mode number are different.

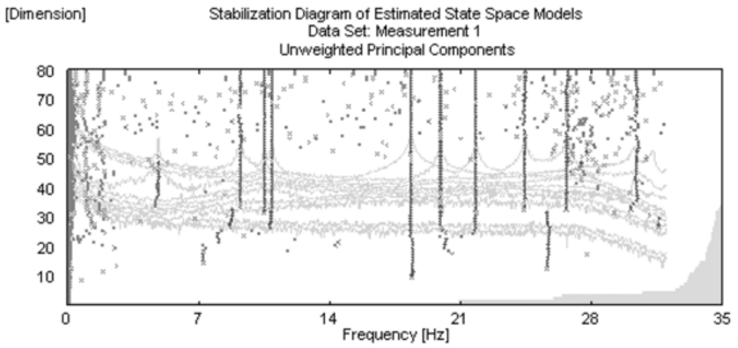
· To reduce differences between both results, finite element model of the bridge is updated by changing boundary conditions such as rigidity of the element connections. After the model updating, maximum differences in the natural frequencies are averagely reduced from 47% to 2.6%.



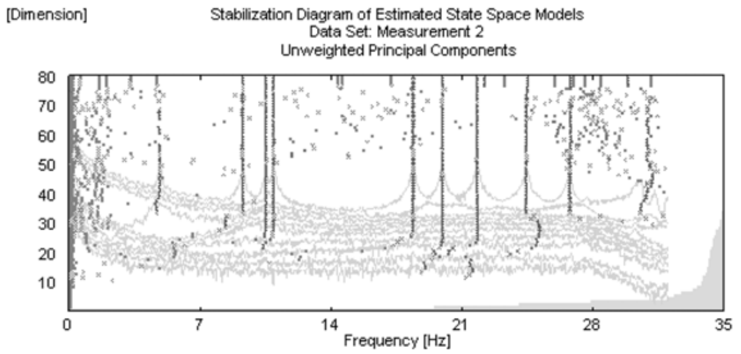
a) Singular values of spectral density matrices



b) Average of auto spectral densities

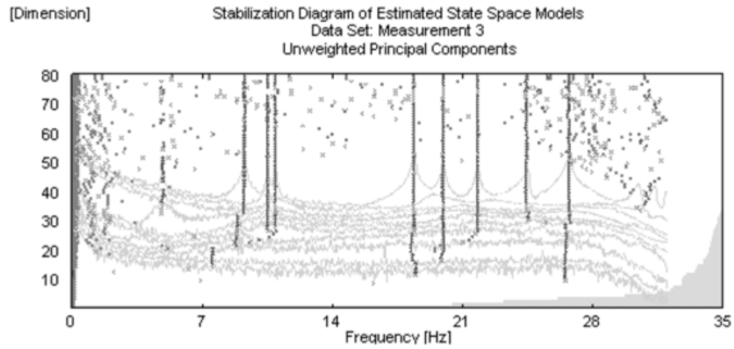


c) Stabilization diagrams of estimated state space models for the first step

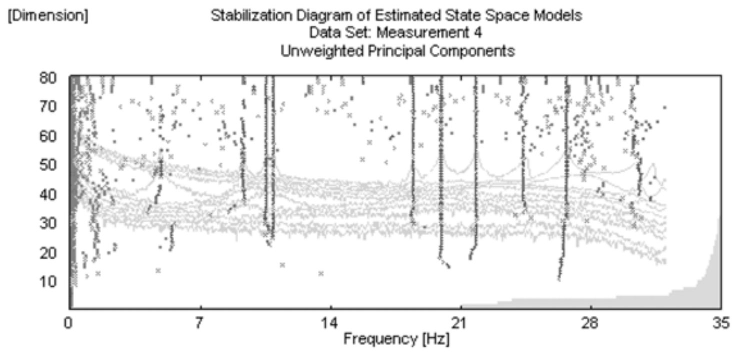


d) Stabilization diagrams of estimated state space models for the second step

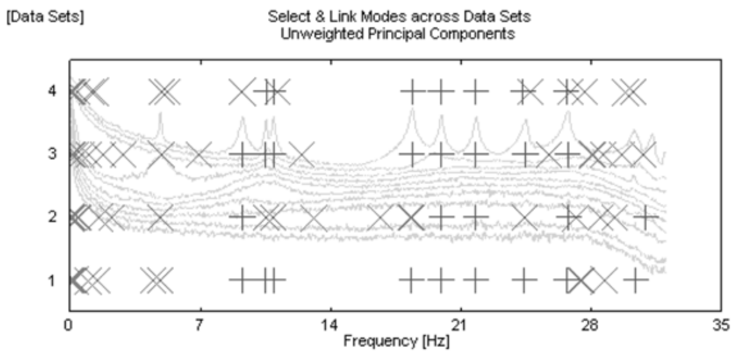
Fig. 10 Dynamic characteristics obtained from third test setup using EFDD and SSI methods



e) Stabilization diagrams of estimated state space models for the third step



f) Stabilization diagrams of estimated state space models for the fourth step



g) Select-link modes across data sets

Fig. 10 Continued

· In the Design and Construct competition, the maximum displacement for 1,250 kg-f load (1,000 kg-f on the main span and 250 kg-f on the left side span) is measured as 6.6 mm. The maximum vertical displacements of the middle point of the bridge model before and after model updating are obtained analytically as 4.0 mm and 6.0 mm, respectively. So, it can be stated that the model updating procedure using semi-rigid connections is successfully applied to this bridge model.

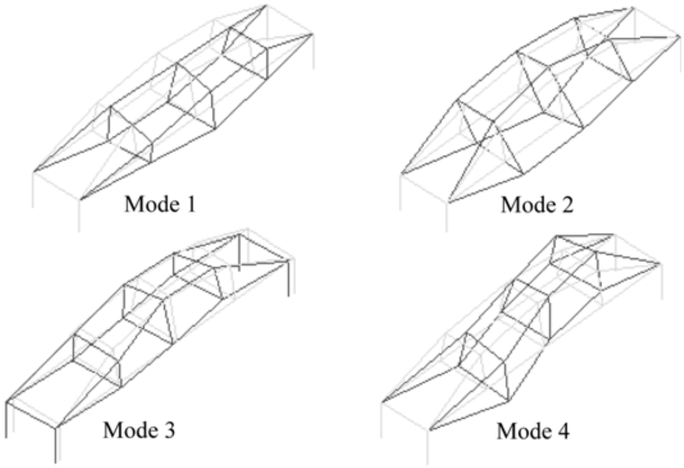


Fig. 11 Experimentally determined first four mode shapes of the bridge model

Table 3 Natural frequencies and damping ratios obtained from ambient vibration tests

Mode number	Dynamic characteristics											
	Measurement 1				Measurement 2				Measurement 3			
	EFDD		SSI		EFDD		SSI		EFDD		SSI	
	Freq* (Hz)	D R* (%)	Freq* (Hz)	D R* (%)	Freq* (Hz)	D R* (%)	Freq* (Hz)	D R* (%)	Freq* (Hz)	D R* (%)	Freq* (Hz)	D R* (%)
1	4.73	0.90	4.70	0.97	4.84	0.95	4.83	0.90	4.86	0.96	4.87	0.91
2	9.09	0.81	9.07	0.75	9.16	0.70	9.13	0.65	9.28	0.75	9.25	0.83
3	10.46	0.68	10.46	0.58	10.57	0.35	10.54	0.80	10.53	0.37	10.52	0.35
4	10.92	0.31	10.92	0.43	10.98	0.23	10.95	0.24	10.93	0.38	10.92	0.31

Freq* : Frequency, D R* : Damping Ratio

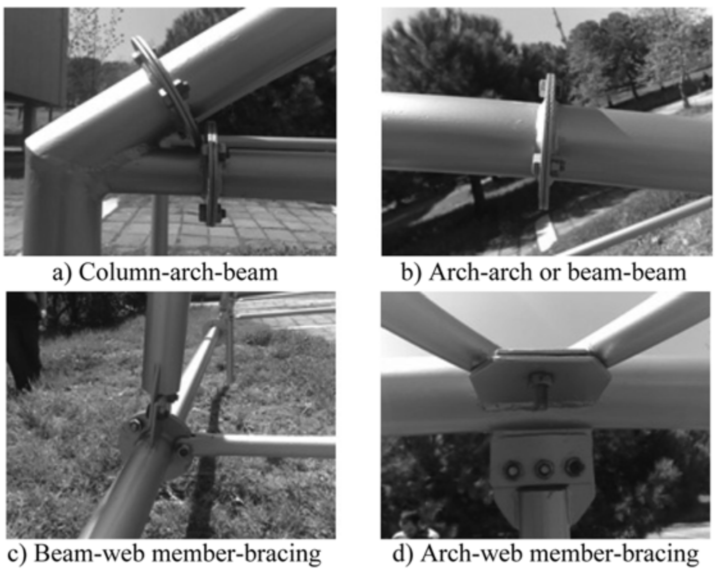


Fig. 12 The view of the connection details of the structural elements

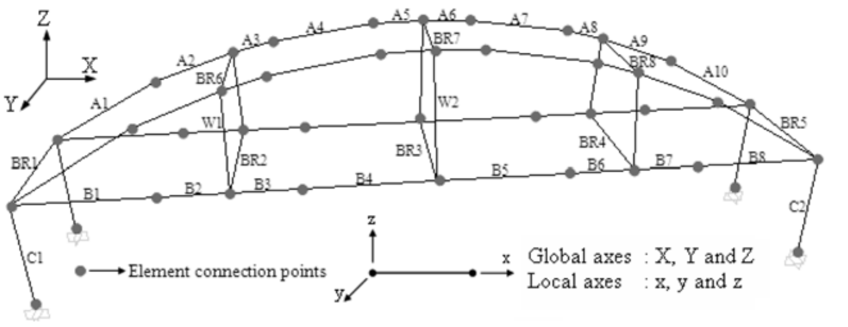


Fig. 13 Connection points on the finite element model of the bridge

Table 4 Steel bar parameters and corresponding rotational spring stiffness values

Structural element		Modules of elasticity (N/m^2)	Length (m)	Moment of inertia (cm^4)	K_y (Nm/rad)	K_z (Nm/rad)
Columns	*C1 and C2*	2.06E11	0.85	45.91	1.43E + 05	1.43E + 05
Arches	*A1 and A10*	2.06E11	1.26	45.91	2.32E + 05	2.32E + 05
	$A2_{left}$ *and A9 right*	2.06E11	0.68	45.91	3.43E + 05	3.43E + 05
	$A3_{right}$ *and* $A8_{left}$	2.06E11	0.33	45.91	7.14E + 05	7.14E + 05
	A4 and A7	2.06E11	0.80	45.91	2.90E + 05	2.90E + 05
	$A5_{left}$ *and* $A6_{right}$	2.06E11	0.40	45.91	5.85E + 05	5.85E + 05
Beams	*B1, B4, B5 and B8*	2.06E11	1.00	9.276	4.69E + 04	4.69E + 04
	B2, B3, B6 and B7	2.06E11	0.50	9.276	9.38E + 04	9.38E + 04
Web member	*W1*	2.06E11	0.74	9.276	5.82E + 03	5.82E + 03
	W2	2.06E11	0.97	9.276	4.47E + 03	4.47E + 03
Bracing(Bottom)	*BR1-5*	2.06E11	1.10	1.235	3.65E + 02	3.65E + 02
Bracing (Top)	*BR6 and BR8*	2.06E11	0.83	1.969	7.70E + 02	7.70E + 02
	BR7	2.06E11	0.75	1.969	8.54E + 02	8.54E + 02

Table 5 Comparison of the analytical and experimental frequencies after model updating

Mode number	Initial finite element model		Experimental tests		Updated finite element model	
	Frequencies (Hz)	Max dif. (%)	EFDD (Hz)	SSI (Hz)	Frequencies (Hz)	Max dif. (%)
1	7.31	33.24	4.86	4.87	4.88	0.40
2	16.97	45.31	9.28	9.25	9.41	1.38
3	18.55	43.23	10.53	10.52	10.81	2.59
4	21.19	46.79	10.93	10.92	10.88	0.45

Acknowledgements

This research was supported by the TUBITAK and Karadeniz Technical University under Research Grant No. 106M038, 2005.112.001.1 and 2006.112.001.1, respectively. Also, many thanks to 4BlackSea

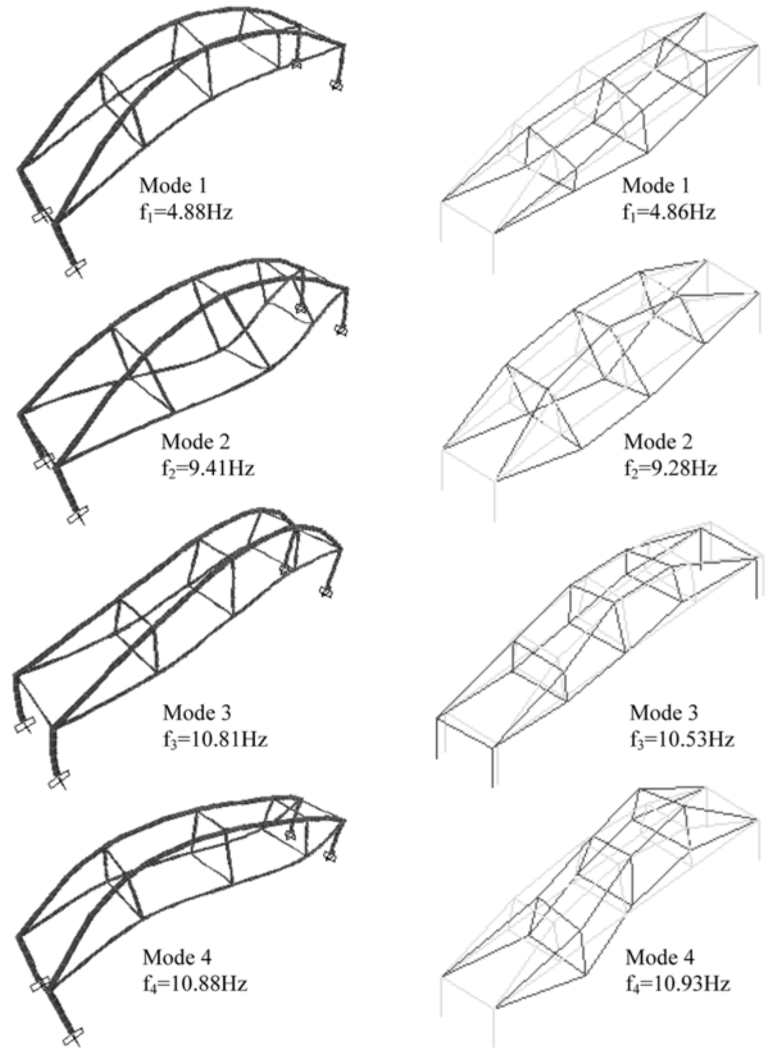


Fig. 14 Comparison of analytical and experimental mode shapes after model updating

a) Quick installation b) Load bearing capacity

Fig. 15 Some views from the competition

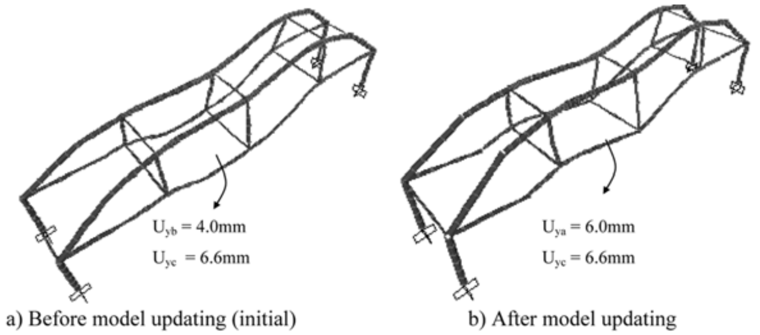


Fig. 16 Comparison of the maximum vertical displacement before and after model updating

team members (Abdullah Demir, Mehmet Akif Hamarat, Mustafa Sakallı, Mahmut Halil Küçükalioğlu, Mehmet Gökhan Karabacak and Nuh Uğur Gümüş) for giving much effort in the construction and competition processes.

References

Altunışık, A.C., Bayraktar, A. and Sevim, B. (2010). "Output-only system identification of post tensioned segmental concrete highway bridges", *J. Bridge. Eng.*, DOI: 10.1061/(ASCE)BE.1943-5592.0000150.

Bayraktar, A., Altunışık, A.C., Sevim, B. and Türker, T. (2007), "Modal testing and finite element model calibration of an arch type steel footbridge", *Steel. Compos. Struct.*, 7(6), 487-502.

Bendat, J.S. and Piersol, A.G. (2004), *Random data: analysis and measurement procedures*, John Wiley and Sons, USA.

Bilello, C., Bergman, L.A. and Kuchma, D. (2004), "Experimental investigation of a small-scale bridge model under a moving mass", *J. Struct. Eng-ASCE*, **130**, 799-804.

Brincker, R. Zhang, L. and Andersen, P. (2000), "Modal identification from ambient responses using frequency domain decomposition", *18th International Modal Analysis Conference*, San Antonio, USA.

Chen, W.F. and Lui, E.M. (1991), *Stability design of steel frames*, CRC Press Inc.

Felber, A.J. (1993), *Development of hybrid bridge evaluation system*, Ph.D thesis, University of British Columbia, Vancouver, Canada, 1993.

Filho, M.S., Guimarães, M.J.R., Sahlit, C.L. and Brito, J.L.V. (2004), "Wind pressures in framed structures with semi-rigid connections", *J. Braz. Soc. Mech. Sci. Eng.,* **26**, 180-189.

Juang, J.N. (1994), *Applied system identification,* Englewood Cliffs (NJ): Prentice-Hall Inc.

Juang, J.N. and Phan, M.Q. (2001), *Identification and control of mechanical systems*, Cambridge University Press, Cambridge, U.K.

Kartal, M.E. (2004), *The effect of partial fixity at nodal points on the behavior of the truss and prefabricated structures*, MS Thesis, Zonguldak Karaelmas University.

Kohoutek, R. (2000), "Non-destructive and ultimate testing of semi-rigid connections", *Fourth International Workshop on Connections in Steel Structures*, Roanoke, VA, October.

Lu, P., Zhao, R. and Zhang, J. (2010), "Experimental and finite element studies of special-shape arch bridge for self-balance", *Struct. Eng. Mech.*, **35**(1), 37-52.

McGuire, W., Gallagher, R.H. and Ziemian, R.D. (1999), *Matrix Structural Analysis*, 2nd ed., John Wiley & Sons, Inc., USA.

Modak, S.V., Kundra, T.K. and Nakra, B.C. (2002), "Comparative study of model updating methods using experimental data", *Compos. Struct.*, **80**, 437-447.

Monforton, G.R. and Wu, T.S. (1963), "Matrix analysis of semi-rigidly connected frames", *J. Struct. Div-ASCE*, **89**, 13-42.

OMA (2006), *Operational modal analysis*, Release 4.0. Structural Vibration Solutions A/S, Denmark.
Peeters, B. (2000), *System identification and damage detection in civil engineering*, Ph.D Thesis, K.U, Leuven, Belgium.
PULSE (2006), *Analyzers and solutions*, Release 11.2. Bruel and Kjaer, Sound and Vibration Measurement A/S, Denmark.
Rainieri, C., Fabbrocino, G., Cosenza, E. and Manfredi, G. (2007), "Implementation of OMA procedures using labview: theory and application", *2nd International Operational Modal Analysis Conference*, Copenhagen, Denmark, May.
Ren, W.X., Zhao, T. and Harik, I.E. (2004), "Experimental and analytical modal analysis of steel arch bridge", *J. Struct. Eng-ASCE*, 130, 1022-1031.
Sanayei, M. and DiCarlo, C. (2009), "Finite element model updating of scale bridge model using measured modal response data", S*tructures Congress*, Austin, Texas.
SAP2000 (1998), *Integrated finite element analysis and design of structures*, Computers and Structures Inc, Berkeley, California, USA.
Sekulovic, M. Salatic, R. and Nefovska, M. (2002), "Dynamic analysis of steel frames with flexible connections", *Compos. Struct.,* **80**, 935-955.
Sevim, B., Bayraktar, A. and Altunışık, A.C. (2010). "Finite element model calibration of berke arch dam using operational modal testing", *J. Vib. Control.*, DOI: 10.1177/1077546310377912.
VanOverschee, P. and DeMoor, B. (1996), *Subspace identification for linear systems: theory-implementation-applications*, Kluwer Academic Publishers, Dordrecht, NL.
Wang, Y.F., Han, B., Du, J.S. and Liu, K.W. (2007), "Creep analysis of concrete filled steel tube arch bridges", *Struct. Eng. Mech.*, **27**(6), 639-650.
Yang, C. (2007), *Seismic analysis of long span bridges including the effects of spatial variation of seismic waves on bridges*, Ph.D Thesis, University of Hong Kong.
Yu, D.J. and Ren, W.X. (2005), "EMD-based stochastic subspace identification of structures from operational vibration measurements", *Eng. Struct.*, **27**, 1741-1751.
Zapico, J.L., Gonzalez, M.P., Friswell, M.I., Taylor, A.J. and Crewe, A.J. (2003), "Finite element model updating of a small scale bridge", *J. Sound. Vib.*, 268, 993-1012.
Zhu, S. and Cheng, X. (2008), "Test and analyses of a new double-arch steel gate under cyclic loading", *J. Constr. Steel. Res.*, **64**, 454-464.
Zivanovic, S., Pavic, A. and Reynolds, P. (2007), "Finite element modelling and updating of a lively footbridge: the complete process", *J. Sound. Vib.*, **301**, 126-145.

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