Finite element model calibration effects on the earthquake response of masonry arch bridges

Barış Sevim, Alemdar Bayraktar, Ahmet Can Altunişık, Sezer Atamtürktür, Fatma Birinci

Article history:
Received 18 November 2009
Received in revised form 24 November 2010
Accepted 10 December 2010
Available online 18 February 2011

Keywords:
Masonry arch bridges
Ambient vibration testing
Dynamic characteristics
Earthquake behavior
Historical construction
Finite element modeling
Model calibration

1. Introduction

Masonry arch bridges, the oldest railway infrastructure elements in existence, still serve as a major component of this system in many countries. For instance, Ellicks [1] noted that in the UK about 96% of the 35,000 in-service masonry arch bridges at the time were over 70 years old and 20% of these inventories were carrying principal roads.

Such bridges are an integral component of the heritage of cultures worldwide, and their preservation is of the utmost importance [2,3]. In Turkey, approximately 1300 of such historical bridges are still in-service. For example, the six-arch stone bridge 30 m in span built around 100 BC is still in existence in Alcantra, Spain [1]. Such bridges are an integral component of the heritage of cultures worldwide, and their preservation is of the utmost importance [2,3]. In Turkey, approximately 1300 of such historical bridges are still in-service.

These uncertainties limit the effectiveness of the FE method as a sole determinant of the dynamic characteristics of an existing structure. The FE method must be coupled with experimental methods while identifying natural frequencies, mode shapes and damping ratios of historical bridges. Two experimental methods available to determine dynamic characteristics of a structure, the Experimental Modal Analysis (EMA) and the Operational Modal Analysis (OMA). In EMA, typically the acceleration response of the structure, caused by a known excitation force generated by a shaker, hammer or other controlled excitation devices, is measured. However, the need for such a controlled excitation source may render this method more...
expensive as compared to OMA. When applied to large-scale historic masonry monuments, this method is also inadequate for evenly and uniformly exciting the structure with a localized high amplitude excitation source. The required force levels to excite global modes are typically high enough to induce nonlinear response, which violates the fundamental linearity assumption of EMA and tends to degrade the measurements. High excitation sources can also cause localized damage on the historical construction. However, with the OMA method, the structure is vibrated by environmental and operational loads as traffic, wind or earthquake, with only the output responses being measured. OMA is also cheaper and faster because it obviates the need for excitation equipment. Moreover, since the vibration response under environmental and operational loads is measured, OMA does not interfere with the functional operations of the system. In addition, the measured response in OMA is representative of the real operating conditions of the structure. Therefore, the authors believe OMA to be more suitable for identifying dynamic characteristics of historical structures than EMA.

FE models of the historical bridges are developed according to the existing engineering bridge drawings. Measurement tests, performed to validate the FE model predictions, typically yield a level of disagreement from the expected results of the analytical model. These discrepancies originate from the uncertainties and simplifying assumptions in structural geometry and materials, as well as from the inaccurate representation of boundary conditions. The problem of adjusting the analytical model such that it reproduces the experimental measurements with increased agreement is known as model calibration in structural dynamics [6]. The sole purpose of model calibration is to improve the uncertain model parameters or imprecise modeling assumptions such that the FE model predictions are closer representations of reality.

Many analytical and experimental studies have been conducted on the historical bridges in the pertinent literature [7–10]. Beside these studies, Fanning and Boothby [11] reported the results of field testing and FE modeling of three masonry arch bridges. The bridges were tested using a reference frame constructed beneath the bridge structure, installing linear variable differential transformers (LVDT) on the reference frame to measure structural displacements, and loading the structure with a vehicle of known weight. Nonlinear FE models were analyzed using ANSYS software. It was found that 3D nonlinear finite element analysis using a reasonable set of material properties enable good prediction of the actual behavior of a masonry arch bridge. Frunzio et al. [12], in their three-dimensional FE analysis of a stone masonry arch bridge involving nonlinear material behavior, found that the results of their FE analysis were useful in generating a qualitative map of the intervention areas for restoration. They also found this analysis strongly dependent upon the exactness of mechanical parameters, which are often difficult to evaluate by experimental analyses, especially in cases of monuments and historical buildings. Toker and Unay [13] studied the
mathematical modeling techniques on a prototype model of a common arch bridge under different loading conditions. Ural [14] determined the dynamic characteristics of Coşandere Historical Arch Bridge using SAP200 software. They also performed earthquake analysis of the bridge using obtained Elcentro ground motion record and maximum principal stresses. Bayraktar et al. [15] determined the dynamic characteristics of Historical Sinik Bridge under ambient vibrations, and also updated the bridge FE model by adjusting the boundary condition definitions. Brencich and Sabia's [16] study of Tanaro Bridge involved investigation of the in-service conditions and the different stages of its demolition. In this study, the natural frequencies, mode shapes and damping ratios of this 18 span masonry construction were identified by dynamic tests. Diamanti et al. [17] used non-destructive ground-penetrating radar (GPR) on masonry arch bridges for monitoring of ring separation. In order to validate and update the analytical results, several laboratory experiments were conducted. Analytical modeling techniques were used to simulate GPR tests, and the analytical models were updated using laboratory experiments.

This paper details how the earthquake behaviors of two historical arch bridges (Osmanlı and Şenyuva Bridges) constructed in the Black Sea region of Turkey were determined using the authors’ initial and calibrated FE models. The characteristics and geometric properties of the case study bridges are first summarized, and the development of the initial FE models is then described along with the initial predictions of bridge characteristics. Finally, the planning, execution and outcome of ambient vibration tests of the two bridges is discussed. From the measurements, the modal parameters are identified by using both enhanced frequency domain decomposition (EFDD) and subspace structural identification (SSI) techniques.

The initial FE model predictions yield an on-average 10% disagreement for Osmanlı Arch Bridge and on-average 15% disagreement for Şenyuva Arch Bridge. Because the deviations are believed to be the consequence of an imprecise modeling decisions or inaccurate model parameters, the FE models of the bridges were calibrated based on experimental natural frequencies. The effect of this calibration exercise of the FE model solutions was investigated by comparing the earthquake behaviors of the bridges predicted by the initial and calibrated FE models.

2. Formulations

In the absence of input force measurements, ambient excitation does not lend itself to frequency response functions (FRFs) or impulse response functions (IRFs) calculations. Therefore, a modal identification procedure is based solely upon output data [18]. Several modal parameter identification techniques are available for extracting these modal parameters, all of which were developed due to improvements in computing capacity and signal processing procedures. These approaches are the Operating Vectors Method, the Complex Exponential Method, the Polyreference Time Domain Method, the enhanced frequency domain decomposition (EFDD) and various stochastic subspace identification (SSI) techniques. In this study, dynamic characteristics are extracted from experimental measurements using the last two in our list, the Enhanced frequency domain decomposition (EFDD) and stochastic subspace identification (SSI) techniques.

2.1. Enhanced frequency domain decomposition (EFDD) technique

In the EFDD technique, the relationship between the unknown input $x(t)$ and the measured responses $y(t)$ is expressed as [19–21]

$$[G_{yy}(j\omega)]^T[G_{xx}(j\omega)][H(j\omega)]=0$$

(1)

where $G_{yy}(j\omega)$ is the power spectral density (PSD) matrix of the input, $G_{xx}(j\omega)$ is the PSD matrix of the responses, $H(j\omega)$ is the frequency response function (FRF) matrix, and + and superscript $T$ denote complex conjugates and transpositions, respectively. The solution to Eq. (1) is given in detail in the literature [22].
2.2. Stochastic Subspace Identification (SSI) method

Stochastic Subspace Identification (SSI) is an output-only method that directly works with time data, circumventing the need to convert the time domain measurements to cross or auto-correlations or to frequency spectra. The method is especially suitable for operational modal parameter identification, and the reader is referred to the following references for detailed technical overview [23–25].

The model of vibrating structures can be defined by a set of linear, constant coefficient and second-order differential equations [24]:

\[ M\ddot{U}(t) + C\dot{U}(t) + KU(t) = F(t) = Bu(t) \]  

(2)

where \( M, C, K \) are the mass, damping and stiffness matrices, \( F(t) \) is the excitation force, and \( U(t) \) is the displacement vector at continuous time \( t \). Observe that the force vector \( F(t) \) is factorized into a matrix \( B \) describing the inputs in space and a vector \( u(t) \). In the SSI method, the equation of dynamic equilibrium (Eq. (2)) is converted to the more suitable discrete-time stochastic state-space model [24]. Though this state-space model originates from control theory, it is also useful in mechanical/civil engineering for computing the modal parameters of a structure with a general viscous damping model [19].

3. Finite element modeling and analytical dynamic characteristics

Both the Osmanlı and Şenyuva arch bridges are located in the Fırtına Stream within the city limits of Rize, on the Black Sea Coast of north-eastern Turkey. The main structural elements of both bridges are the stone arches, side walls and timber block. The bridges have not been subject to a restoration study.

Commercially available ANSYS v. 12 [26] software was used all analytical modeling and bridge analyses. This software is preferable due to its capacity for linear, nonlinear, static and dynamic analyses and its myriad of element types.

3.1. Osmanlı Arch Bridge

Several views of the 19th century Osmanlı Arch Bridge are shown in Fig. 1. This two-spanned arch bridge has a total length of 51.7 m. The span of each arch is 25.2 and 6 m, and the radius of each is 13 and 3 m, respectively (see Fig. 2). The arches consist of inner and outer segments with thicknesses of 0.58 and 0.15 m, respectively. The thickness of the side walls is 0.5 m with a timber block between them, the width of which is 2.50 m. Geometrical properties of the bridge are provided in Fig. 2.

For the FE bridge analysis, three different material properties were considered: the stone arch, the side walls and the timber block. Identification of these material properties for historical structures was completed with respect to the related successful studies on other historical bridges [12,13,16]. The material property values compiled from the literature and used in this analysis are given below.

The solid model, the material types and the mesh of FE model of the bridge is shown in Fig. 3. A total 38,602 SOLID186 elements are used. Each SOLID186 element has 20 nodes and each node has three
degree of freedoms: \( x \), \( y \) and \( z \) translations. The bridge abutments and the side walls were constructed on the rock as shown in Fig. 1. Therefore, all degrees of freedom under the bridge abutments and at the side walls are assumed as fixed.

The first five natural frequencies have been estimated to range between 3 and 15 Hz. The bridge has three main types of vibration modes: bending modes in \( z \) direction, vertical modes in \( y \) direction and torsional modes. FE model predictions for the first five mode shapes of the bridge and their corresponding natural frequency predictions are shown in Fig. 4.

3.2. Şenyuva Arch Bridge

Fig. 5 shows the second subject bridge of our study, the 17th century Şenyuva Arch Bridge.

Built in 1696, the single-arch Şenyuva Bridge spans the Fırtına (Storm) River at a height of 12.4 m and a length of 24.8 m. The total length of the bridge is 52.4 m and the width of the deck is 2.5 m. The geometrical properties of the Şenyuva Bridge are shown in Fig. 6. The radius and thickness of this magnificent historical structure is 12.4 and 0.60 m, respectively. The thickness of the side walls is 0.50 m and there is a timber block between them, which is 1.5 m in width. The height of both side walls are 9.2 and 3.5 m, respectively. The bridge also has \( 0.30 \times 0.60 \text{ m}^2 \) dimensional parapets on both the sides of the bridge deck.

Fig. 7 shows a 3D FE model of the bridge, built with a total 26,408 SOLID186 elements. The materials properties used in the analyses are listed in Table 1. The boundary conditions are defined by fixing the translational and rotational degrees of freedom at all bridge abutments and both side walls.

The first five natural frequencies are predicted to fall within a range of 3–11 Hz. The first five mode shapes of the bridge, shown in Fig. 8, are bending modes in the \( z \) direction, vertical modes in the \( y \) direction and torsional modes.

4. Operational modal analysis and identification of experimental dynamic characteristics

Uni-axial accelerometers (B&K 8340) along with a 17-channel data acquisition system (B&K 3560) were used during the ambient measurements of the bridges. Ambient vibration tests were conducted under existing environmental and operational loads as

---

Table 1
Material properties considered in the analytical solution.

<table>
<thead>
<tr>
<th>Material</th>
<th>Modulus of elasticity (N/m²)</th>
<th>Poisson ratio</th>
<th>Density (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stone arches</td>
<td>3.0E9</td>
<td>0.25</td>
<td>1600</td>
</tr>
<tr>
<td>Timber block</td>
<td>1.5E9</td>
<td>0.05</td>
<td>1300</td>
</tr>
<tr>
<td>Side walls</td>
<td>2.5E9</td>
<td>0.20</td>
<td>1400</td>
</tr>
</tbody>
</table>

---

Fig. 7. Finite element model of the Şenyuva Arch Bridge.

Fig. 8. Analytical mode shapes of the Şenyuva Arch Bridge.

Fig. 9. Equipment used during ambient vibration testing.
wind and human movement. To generate the detectable vibration levels for our study, between five and ten people were directed to walk across the bridge. This procedure, known as human movement and is used in the literature. The minimum frequency span and sensitivity of these accelerometers reign were assigned as 0.1 Hz and 10 V/g, respectively, and signals were transferred into the PULSE [27] Labshop software. The data acquisition system and accelerometers are pictured in Fig. 9. The operational modal analysis software (OMA) [28] was used to generate parameter estimations from the ambient vibration data. Because of various testing difficulties (e.g. inadequate accelerometers, very rigid structure, inadequacy vibration signals), the first five modes were only considered in our experimental measurements.

4.1. Osmanlı Arch Bridge

Fifteen uni-axial accelerometers were used in the ambient vibration tests of the bridge. The 3D schematic view of the accelerometer placement is given in Fig. 10.

![Fig. 10. Accelerometer locations on the Osmanlı Arch Bridge.](image)

![Fig. 11. Modal parameters of Osmanlı Bridge attained from EFDD and SSI techniques: (a) singular values of spectral density matrices and (b) stabilization diagram of estimated state space models.](image)
The OMA was conducted using the EFDD technique in the frequency domain and the SSI technique in the time domain. The EFDD technique was used to extract dynamic characteristics from singular values of each vibration signal, while SSI technique was used to compute the singular values of a collection of vibration signals. Fig. 11 shows the singular values of the spectral density matrices and the stabilization diagrams of estimated state space models obtained from vibration signals using these two techniques. This step consists of computation of singular values. Repeating the procedure with increasing polynomial orders yields a stabilization diagram, which plots the roots of the polynomial as the order of the polynomial is increased. The stable roots for increasing polynomial orders indicate a natural system frequency, and the user defines the maximum number of modes for the analysis, which corresponds to the maximum polynomial order. Typically, an over-specified mode order is defined, which later necessitates a judgment-based distinction between the physical and computational modes. The stabilization diagram aids this process. The premise is that as these physical (actual) poles stabilize for an increasing model order, the mathematical poles scatter. Though this pole stabilization is apparent in Fig. 11, the pole scattering is less clearly defined. The reason for this apparent discrepancy is the lack of appropriate signals collected for mode extraction.

The first five mode shapes obtained from ambient vibration tests are shown in Fig. 12. A visual comparison of Figs. 8 and 12 show good agreement between analytical and experimental mode shapes. A comparison of analytically and experimentally identified natural frequencies and damping ratios ($\zeta$) of Osmanlı Arch Bridge is provided in Table 2.

### 4.2. Şenyuva Arch Bridge

Twelve uni-axial accelerometers were used in the ambient vibration tests of the bridge. The 3D schematic view of location points of the accelerometers is shown in Fig. 13. The locations of the accelerometers are determined according to finite element results. Singular values of spectral density matrices and stabilization diagrams of estimated state space models attained from vibration signals using EFDD and SSI techniques are shown in Fig. 14.

The first five mode shapes extracted from ambient vibration tests are given in Fig. 15. A comparison of analytically and experimentally identified dynamic characteristics, and the damping ratios ($\zeta$) of the Şenyuva Bridge is given in Table 3.

### 5. Finite element model calibration of the bridges

The natural frequencies obtained by the initial FE model of the two bridges show differences from the experimental results: an approximate 10% difference for the Osmanlı Arch Bridge (Table 2), and an approximate 15% difference for Şenyuva Arch Bridge (Table 3). Because we hypothesized that these deviations were the result of imprecision in the FE model, the FE models of these bridges were calibrated according to the experimental results.

The main purpose of the whole paper is to illustrate the importance of model calibration. Therefore, the material property values used in the FE model are determined according to pertinent literature for masonry structures obtained in the laboratory and in-situ tests. However, the actual material properties of the bridges must be obtained from laboratory and in-situ tests when considering earthquake performance for strengthening. Despite unavoidable uncertainties in the material properties, the material property definitions are considered to higher confidence. Therefore, the manual calibration exercise, described in the next section, will primarily focus on the boundary condition definitions. The goal of model calibration as implemented herein is to ‘improve’ the FE model, not to obtain a FE model that identically matches the experimental results.

#### 5.1. Osmanlı Arch Bridge

As shown in Fig. 16, some river materials such as soil, sand, rock and rubbish are collected at the side walls of Osmanlı Arch Bridge, which have constrained the displacement of the side walls over time. Therefore, the sidewall stiffness was increased in the calibrated model.

<table>
<thead>
<tr>
<th>Mode number</th>
<th>Analytical frequencies (Hz)</th>
<th>Experimental frequencies (Hz)</th>
<th>Damping (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>EFDD</td>
<td>SSI</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>3.843</td>
<td>4.640</td>
<td>4.642</td>
</tr>
<tr>
<td>2</td>
<td>7.527</td>
<td>8.094</td>
<td>8.325</td>
</tr>
<tr>
<td>4</td>
<td>10.638</td>
<td>12.340</td>
<td>11.910</td>
</tr>
<tr>
<td>5</td>
<td>14.563</td>
<td>15.840</td>
<td>15.420</td>
</tr>
</tbody>
</table>

Table 2: Comparison of analytically and experimentally identified dynamic characteristics of the Osmanlı Arch Bridge.
Table 4 shows a comparison of experimental and analytical natural frequencies after adjusting the boundary conditions in the FE model. It can be seen that the differences between natural frequencies are reduced from on average 15% to about on average 5% after model calibration. Considering the two different techniques used in the identification of modal parameters, EFDD and SSI, yielded up to 3% difference, on average 5% variability between measured and calculated natural frequencies is deemed acceptable.

5.2. Şenyua Arch Bridge

Similar to the Osmanli bridge, the displacement ability of the walls near the both sides of the Şenyua Bridge are decreased due to piling up the earth, stone, rubbish (Fig. 17). As a result, the stiffness of the side walls was increased in the calibrated model.

Table 5 shows a comparison of the experimental and analytical natural frequencies after FE model calibration. Note that the differences between natural frequencies are approximately 2% after model calibration, which is in the same order of magnitude of difference between the frequencies identified by EFDD and SSI methods.

6. Earthquake behaviors of Arch Bridges

The actual seismic performance of existing masonry arch bridges must be evaluated by using nonlinear materials and considering the interface between the masonry and the fill. Because the purpose is to illustrate the importance of model calibration and ambient vibration testing on the earthquake performance of masonry arch bridges, linear earthquake behaviors of Osmanlı and Şenyua Arch Bridges were considered to investigate the effect of FE model calibration. However, the study presented herein can be followed up with a nonlinear study.

Time-history analyses of the two bridges of interest were performed using ERZIKAN/ERZ-NS component of 1992 Erzincan earthquake (Fig. 18) [29]. The earthquake occurred in the North...
Anatolian Fault, which is the nearest fault to the bridges that were the subject of this work. The accelerations are applied to bridges on bending direction (in $z$ direction) where the first mode shapes are obtained.

In the time-history analyses, element mass and stiffness matrices are computed using the Gauss numerical integration technique [30]. The Newmark method is used in the solution of the equation. Because of the computational demand of this method, only the first 6.5 s of the earthquake were used during calculations. Such an abbreviated duration of time was not expected to adversely affect results as the first few seconds of the Erzincan Earthquake were the most effective (Fig. 18).

Because the damping ratios are unknown in the initial (uncalibrated) FE model analysis, the authors estimated the Rayleigh damping coefficients for an assumed 5% damping ratio. However, during the calibrated FE model analysis, Rayleigh damping coefficients were calculated according to the experimentally obtained damping ratios.

### Table 3
Comparison of analytically and experimentally identified dynamic characteristics of the Şenyuva Arch Bridge.

<table>
<thead>
<tr>
<th>Mode number</th>
<th>Analytical frequencies (Hz)</th>
<th>Experimental frequencies (Hz)</th>
<th>Damping (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>EFDD</td>
<td>SSI</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>3.347</td>
<td>4.045</td>
<td>4.066</td>
</tr>
<tr>
<td>2</td>
<td>5.772</td>
<td>7.750</td>
<td>7.960</td>
</tr>
<tr>
<td>3</td>
<td>7.554</td>
<td>8.020</td>
<td>8.044</td>
</tr>
<tr>
<td>4</td>
<td>9.055</td>
<td>10.000</td>
<td>10.100</td>
</tr>
<tr>
<td>5</td>
<td>10.044</td>
<td>12.160</td>
<td>11.750</td>
</tr>
</tbody>
</table>

### Table 4
Analytical and experimental natural frequencies of the Osmanlı Arch Bridge after model calibration.

<table>
<thead>
<tr>
<th>Mode number</th>
<th>Analytical frequencies (Hz)</th>
<th>Calibrated analytical frequencies (Hz)</th>
<th>Experimental frequencies (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>EFDD</td>
<td>SSI</td>
<td>EFDD</td>
</tr>
<tr>
<td>1</td>
<td>3.843</td>
<td>4.640</td>
<td>4.642</td>
</tr>
<tr>
<td>2</td>
<td>5.772</td>
<td>7.750</td>
<td>7.960</td>
</tr>
<tr>
<td>3</td>
<td>7.554</td>
<td>8.020</td>
<td>8.044</td>
</tr>
<tr>
<td>4</td>
<td>9.055</td>
<td>10.000</td>
<td>10.100</td>
</tr>
<tr>
<td>5</td>
<td>10.044</td>
<td>12.160</td>
<td>11.750</td>
</tr>
</tbody>
</table>

Anatolian Fault, which is the nearest fault to the bridges that were the subject of this work. The accelerations are applied to bridges on bending direction (in $z$ direction) where the first mode shapes are obtained.

In the time-history analyses, element mass and stiffness matrices are computed using the Gauss numerical integration technique [30]. The Newmark method is used in the solution of the equation. Because of the computational demand of this method, only the first 6.5 s of the earthquake were used during calculations. Such an abbreviated duration of time was not expected to adversely affect results as the first few seconds of the Erzincan Earthquake were the most effective (Fig. 18).

Because the damping ratios are unknown in the initial (uncalibrated) FE model analysis, the authors estimated the Rayleigh damping coefficients for an assumed 5% damping ratio. However, during the calibrated FE model analysis, Rayleigh damping coefficients were calculated according to the experimentally obtained damping ratios.

### 6.1. Osmanlı Arch Bridge

The maximum displacement contours of the Osmanlı Arch Bridge before and after model calibration are shown in Fig. 19(a) and (b), respectively. These contours represent the distribution of the peak values reached by the maximum displacements at any time during simulation at each point within the section. Note that the maximum displacements occur at the middle region of the bridge, top of the timber block and side walls (Fig. 18). Experimental damping ratios are used and the stiffness of the side walls is increased in the updated model.

The time histories of displacements at the nodal point of stone arch in which the maximum displacements occurred are plotted before and after model calibration in Fig. 20(a) and (b), respectively. The maximum displacements, obtained from the initial FE model (prior to model calibration), were at an acceptable level [31].

The maximum principal stress contours of Osmanlı Arch Bridge before and after model calibration are shown in Fig. 21(a) and (b),
respectively. These stress contours represent the distribution of the peak values reached by the maximum principal stress at each point within the section. It can be seen in Fig. 21 that the maximum principal stresses are occurred displacements at any time during simulation at the region around the base of stone arch. The maximum and minimum principal stress values in the arch are lower than the acceptable strength of the stone [11,32].

The time histories of the maximum and minimum principal stresses are plotted before and after model calibration in Fig. 22(a)–(d), respectively. These stresses occur at the heel point of the bridge (Fig. 21). As seen in Fig. 22, the principal stresses obtained from initial model (before model calibration) are significantly larger than those of the calibrated model.

### 6.2. Şenyuva Arch Bridge

The maximum displacement contours of the Şenyuva Arch Bridge before and after model calibration are shown in Fig. 23(a) and (b), respectively. These contours represent the distribution of the peak values reached by the maximum displacements at each point within the section. It can be seen from Fig. 23 that the maximum displacements occurred at the middle region of the bridge and the top of the bridge.

#### Table 5
Analytical and experimental natural frequencies of the Şenyuva Arch Bridge after model calibration.

<table>
<thead>
<tr>
<th>Mode number</th>
<th>Analytical frequencies (Hz)</th>
<th>Calibrated analytical frequencies (Hz)</th>
<th>Experimental frequencies (Hz)</th>
<th>EFDD</th>
<th>SSI</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.347</td>
<td>4.070</td>
<td>4.045</td>
<td>4.066</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>5.772</td>
<td>7.780</td>
<td>7.750</td>
<td>7.960</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>7.554</td>
<td>8.020</td>
<td>8.020</td>
<td>8.044</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>9.055</td>
<td>10.670</td>
<td>10.000</td>
<td>10.100</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>10.044</td>
<td>12.090</td>
<td>12.160</td>
<td>11.750</td>
<td></td>
</tr>
</tbody>
</table>

![Image](image1.png)

**Fig. 18.** The time-history of ground motion acceleration of the 1992 Erzincan Earthquake.

![Image](image2.png)

**Fig. 19.** The maximum displacements contours of the Osmanlı Arch Bridge obtained before/after model calibration.

![Image](image3.png)

**Fig. 20.** The time histories of displacements at stone arch of the Osmanli Arch Bridge.

![Image](image4.png)

**Fig. 21.** The maximum stress contours of the Osmanlı Arch Bridge.
timber block and side walls. It can be generally stated that the
displacements are at an acceptable level [31].

The time histories of displacements at the nodal point of stone
arch where the maximum displacements occurred are plotted
before and after model calibration in Fig. 24(a) and (b), respectively.
The maximum values in time histories of displacements are obtained
for initial FE model (before the model calibration).

The maximum principal stress contours of Şenyuva Arch Bridge
before and after model calibration are shown in Fig. 25(a) and (b),
respectively. These stress contours represent the distribution of the
peak values reached by the maximum principal stress at each point
within the section. Similar to the Osmanli bridge, maximum
principal stresses are observed at the base of stone arch before
the model calibration (Fig. 21). However, after calibration, the
predicted maximum stress region shifted to the left upside of the
side wall. The maximum and minimum principal stress values in
the arch are lower than the stone strength [11,32].

The time histories of the maximum and minimum principal stresses
are plotted before and after model calibration in Fig. 26(a)–(d).
These stresses occur at the heel point of the bridge
(Fig. 25). As seen in Fig. 26, the principal stresses obtained from initial
model (before model calibration) are significantly larger than those of
the calibrated model.

7. Conclusions

In this paper, FE modeling, modal testing, FE model calibration
of Osmanli and Şenyuva Historical Arch Bridges located in Rize,
Turkey are presented. The earthquake behavior of the two bridges
was investigated using both the initial (not calibrated) and
calibrated FE models. The following phenomena were observed:

- Initial analytical natural frequencies of Osmanli Arch Bridge were
attained at ranges between 3.8 and 14.6 Hz for the first five modes.
These can be classified into bending modes in the z direction,
vertical modes in the y direction and torsional modes. The first five
experimental modes were estimated within ranges between
4.6 and 15.8 Hz. The experimental mode shapes were observed
to be in close agreement with the analytical mode shapes.

- For Osmanli Arch Bridge, there is an approximate 10% difference
between the natural frequencies predicted by the initial FE
model and obtained through OMA. The FE model of Osmanli
Arch Bridge is calibrated by adjusting the FE model boundary
conditions. After model calibration, the differences in calculated
and measured natural frequencies were reduced to about 5%.

- The obtained initial analytical natural frequencies of Şenyuva
Arch Bridge ranged between 3.4 and 10 Hz for the first five
modes. These modes can be classified into bending, vertical and
torsional modes. The first five experimental modes were within
a range between 4.1 and 12.1 Hz. Experimental mode shapes
were visually comparable to analytical mode shapes.

- There is an approximate 15% difference between the initial FE
model and the experimental natural frequencies of the Şenyuva
Arch Bridge. The FE model of the Şenyuva Arch Bridge was also
calibrated by modifying boundary conditions around the side
walls where there was an accumulation of debris. After model
calibration, the discrepancy between the FE model and mea-
surements was observed as low as 2%.
The maximum displacements, obtained from the earthquake analyses of initial and calibrated models of Osmanlı Arch Bridge, were 0.022 and 0.016 m, respectively. The maximum displacements from the earthquake analyses of initial and calibrated models of Şenyuva Arch Bridge were as 0.039 and 0.025 m, respectively. The maximum displacements occurred at the middle region of the bridges and at the top of the timber block and side walls.

- It can be generally stated that the displacements are at an acceptable level.
- The maximum principal stresses of 2.6 and 1.2 MPa were obtained, respectively, from earthquake analyses of initial and calibrated models of the Osmanlı Arch Bridge. The maximum principal stresses of 2.7 and 1.6 MPa were obtained, respectively, from earthquake analyses of initial and calibrated models of the Şenyuva Arch Bridge. These maximum principal stresses generally occurred at regions around the base of stone arch for both bridges.
- The minimum principal stresses of 2.4 and 1.09 MPa were obtained, respectively, from earthquake analyses of the initial and calibrated models of the Osmanlı Arch Bridge. The minimum principal stresses of 2.7 and 1.6 MPa were obtained,

Fig. 22. Time histories of (a), (b) maximum and (c), (d) minimum principal stresses of the Osmanlı Arch Bridge.

Fig. 23. The maximum displacement contours of the Şenyuva Arch Bridge obtained before/after model calibration.

Fig. 24. The time histories of the stone arch displacements of the Şenyuva Arch Bridge.
respectively, from earthquake analyses of initial and calibrated models of the Şenyuva Arch Bridge.

- The maximum and minimum principal stress values in the arch are lower than the stone strength.
- The maximum and minimum results obtained from the initial models are bigger than those of the calibrated models for both the Osmanlı and Şenyuva Arch Bridges.
- These findings illustrate a severe overestimation of the uncalibrated FE model constructed and informed with the best-engineering practices. Therefore, an ambient testing and model calibration scheme must be an integral part of routine structural analyses when applied to historic masonry arch bridges.

Acknowledgements

This research was supported by the TUBITAK and Karadeniz Technical University under Research Grant nos. 106M038, 2005.112.001.1, and 2006.112.001.1, respectively. Editorial help of Godfrey Kimball of Clemson University is greatly appreciated.

References


