Ambient vibration testing and seismic behavior of historical arch bridges under near and far fault ground motions

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Abstract This study investigates the effects of near and far fault ground motion on the seismic behavior of historical arch bridges through a combined numerical and experimental evaluation. The approach undertaken begins with finite element modeling of the arch bridge and identification of the most significant vibration modes of the bridge through ambient vibration testing. Uncertain parameters of the finite element model are then revised through systematic comparisons of the measured vibration models to those that are predicted by the model. The revised finite element model is used to predict the time history response for displacements and stresses through which the effect of the finite element model updating on model predictions are evaluated. Furthermore, displacements and stresses obtained considering both near and far fault ground motions are then compared. Results indicate that near fault ground motion imposes higher seismic demand on the arch bridge observed in both higher displacements and stresses.

Keywords Ambient vibration testing · Finite element model revision · Historical arch bridge · Near and far fault ground motion · Seismic behavior

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1 Introduction

Historic masonry bridges of various forms and dimension have been built worldwide for social, economic, and strategic purposes. Originally intended to carry only pedestrian and horse-drawn vehicles, many of these historical bridges currently serve as critical components of transportation systems and thus, must withstand significantly larger loads for which they were initially designed and hence, are degrading at a faster rate. Aside from being essential for the proper functioning of transportation networks, these historic masonry bridges also constitute an important part of cultural heritage, making their preservation for the next generations an important undertaking.

Given the inherent susceptibility of masonry structures to damage during seismic events, the preservation of historic masonry bridges must involve an assessment of their structural behavior during earthquakes. The seismic response of masonry arch bridges is closely related to its dynamic characteristics which include for instance natural frequencies, mode shapes and damping ratios. These can be modal parameters determined analytically using finite element analyses, such computations suffer from inevitable uncertainties in material properties, boundary conditions, internal composition of the structural components and existing damage. Hence, experimental methods are a more acceptable means of determining the dynamic characteristics of masonry structures (Atamturktur et al. 2009; Atamturktur and Sevim 2011; Sarhosis et al. 2014). Ambient vibration testing is one of the most widely used tools for estimating the structural dynamic properties of existing bridges (Bayraktar et al. 2009a; Atamturktur et al. 2010). In this method, the vibration response of a structure was measured under operational conditions through the use of a distributed sensor network. The disagreements between analytically and experimentally identified dynamic characteristics are typically used as a method for calibrating the poorly known parameters of finite element models (Bensalem et al. 1998; Fanning and Boothby 2001; Toker and Unay 2004; Ural 2005; Bayraktar et al. 2007; Brencich and Sabia 2008; Atamturktur and Laman 2012). These calibrated finite elements are then used to predict the seismic response of arch bridges (Gentile 2006; Ribeiro et al. 2012; Zordan et al. 2014).

Ground motions recorded within the near fault area of an earthquake at stations located toward the direction of the fault rupture tend to differ greatly from the usual far fault earthquake ground motion. This is because the ground motion recorded in the near fault region displays a long period pulse in the acceleration history that appears as a coherent pulse in both the velocity and displacement histories. Such a pronounced pulse does not exist in ground motions recorded at locations away from the near fault region, however (Chopra and Chintanapakdee 2001). Near fault ground motions and/or pulse type earthquakes are influenced by the rupture mechanism, rupture direction and location of faults (also known as directivity effects) (WSTC Report 2007). These ground motions are characterized by the long-period ($T_p$) pulses, which have been observed in recent near-fault earthquakes, i.e. 1989 Loma Prieta Earthquake, 1992 Erzincan Earthquake, 1994 Northridge Earthquake, 1995 Kobe and 1999 Chi–Chi Earthquakes, with a high peak ground velocity (Bayraktar et al. 2009b). The near fault effects can be classified into three types of pulses: acceleration, velocity, and displacement. The velocity pulse represents the cumulative effect of almost all of the seismic radiation from the fault and is more commonly available in earthquake recording compared to acceleration and displacement pulses (Somerville et al. 1997). The displacement pulse without the high velocity pulse, does not have a high damage potential, however, because the structure has time to react to the displacements.

Structures exposed to near fault ground motions typically exhibit higher levels of damage resulting in greater losses of both property and life. Consequently, it is imperative
to consider the effects of near fault ground motion when assessing structural systems, which have been the subject of several recent studies, specifically concerning highway and railway bridges, and girder and suspension bridges (Orozco and Ashford 2002; Choi 2007; Dreger et al. 2007; Jia and Ou 2008; Brown and Saiidi 2011; Adanur et al. 2012; Rodriguez 2012; Ling-kun et al. 2014). Clearly, these studies have established a precedent for considering near fault ground motion in the evaluation of the seismic response of the structures. There have been very few studies undertaken to evaluate the near fault ground motion effects on historic masonry arch bridges, however this paper is one such study. Here, the authors detail their use of validated finite element models to compare the effects of near and far fault ground motion on the seismic behavior of historical masonry arch bridges.

2 Description of Osmanli historical masonry arch bridge

In this study, the Osmanli arch bridge, also known as Timisvat historical masonry arch bridge was selected for the numerical and experimental studies. Spanning the Fırtına Creek in the city of Rize, in northeast Turkey, the construction of the bridge is believed date from the first half of the 19th century. There is no historical data regarding the original construction, and thus, the time of initial construction is anecdotal. The main structural elements of the bridge, as shown in Fig. 1, are composed of arches and parapets made from cut stone, side walls and timber block) and show no visual signs of repair or retrofit.

Fig. 1 Osmanlı (Timisvat) historical bridge with two arches (difficult site conditions combined with the accumulation of soil in front of the small arch prevented clear photos of this arch to be taken)
The bridge is a two-span arch construction with 51.7 m total length, with arch spans of 25.2 and 6 m, and arch heights of 13 and 3 m, respectively. The bridge arches consist of two inner segment with thicknesses of 0.58 m and outer segment with thicknesses of 0.15 m. The thickness of the sidewalls is 0.5 m and with a timber block between each, which is 2.50 m in width. The geometric dimensions of the bridge are given in Fig. 2 as is the dimensions of cross-section I–I which consist of the inner and outer arch, the side walls and the timber block. A schematic of the small and big arches is also illustrated in Fig. 2, which also details the nodal points underneath and on the front face of the arches, which are used to represent the results of the finite element analyses. Also shown in Fig. 2 is the material composition of the bridge. However as seen earlier in Fig. 1, the accumulation of both soil and boulders around the sidewalls of the smaller of the two arches of the bridge provide extra support to the structure.

3 Finite element modeling and ambient vibration tests

3.1 Finite element modeling

3D finite element model of the bridge was built in ANSYS v. 14 (2014) (see Fig. 3a) using the geometrical dimensions shown previously in Fig. 2. A mesh refinement study was also undertaken (see Fig. 3b; Table 1) and a model with 38737 SOLID186 elements was a deemed sufficient for yielding a series of numerically converged solutions. The SOLID186 element has 20 nodes, with each node having three translational degrees of freedom. The
model is built using a macro-modeling approach (Li and Atamturktur 2014) in that the individual stone units, the mortar joint, as well as the unit-mortar interface are smeared into a linear elastic homogeneous continuum, with the entire model discretized into finite elements. Hence, each finite element are connected to each other with nodal points without considering contact surfaces between individual stone units and the model constituted using properties. In defining the boundary conditions, all degrees of freedom under the bridge abutments and the side walls were assumed as fixed. Although non-linear modeling would be better suited for capturing the highly heterogeneous anisotropic material of the masonry assembly (see for instance Prabhu et al. 2014), the uncertainties associated with the nonlinear modeling of masonry system, especially due to the sheer number of poorly known input parameters required to represent the nonlinear constituent model of the arch bridge studied herein, make the nonlinear modeling approach infeasible.

Obtaining realistic approximations of material properties is necessary for building a model that can yield useful predictions. However, the cultural and historical importance of

![Three dimensional finite element model of the bridge.](image)

![Mesh refinement study for the ideal finite element model of the bridge.](image)

### Table 1  Element numbers and natural frequencies for the mesh refinement

<table>
<thead>
<tr>
<th>Finite element models</th>
<th>Element numbers</th>
<th>Mode 1 (Hz)</th>
<th>Mode 2 (Hz)</th>
<th>Mode 3 (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model 1</td>
<td>2298</td>
<td>3.86</td>
<td>7.57</td>
<td>9.49</td>
</tr>
<tr>
<td>Model 2</td>
<td>7455</td>
<td>3.85</td>
<td>7.55</td>
<td>9.39</td>
</tr>
<tr>
<td>Model 3</td>
<td>14,459</td>
<td>3.84</td>
<td>7.54</td>
<td>9.38</td>
</tr>
<tr>
<td>Model 4</td>
<td>25,953</td>
<td>3.84</td>
<td>7.53</td>
<td>9.38</td>
</tr>
<tr>
<td>Model 5</td>
<td>37,837</td>
<td>3.84</td>
<td>7.52</td>
<td>9.37</td>
</tr>
</tbody>
</table>
the arch bridge makes it a protected artifact prohibiting the removal of any core samples from the structure to conduct destructive tests in the laboratory and thus preventing the experimental determination of material properties. Instead, the authors undertook a thorough literature review (Frunzio et al. 2001; Toker and Unay 2004; Brencich and Sabia 2008; Sevim et al. 2011) to determine the initial estimates for these material properties, which are listed in Table 2. Non-destructive methods, such as ambient vibration testing conducted as part of this study (discussed in the next section), are then used to augment of the lack of knowledge regarding the parameter values.

3.2 Ambient vibration tests

The authors conducted ambient vibration experiments using a 17-channel data acquisition system (PULSE B&K 3560-C) and 15 B&K 8340 uni-axial piezoelectric annular shear design accelerometers. The locations of the accelerometers and testing equipment are shown in Figs. 4 and 5, respectively. The B&K 8340 accelerometers has a frequency range 0.1–1500 Hz and as sensitivity of 10,000 mV/g, and may be used at temperatures between −51 and +74 °C. The PULSE B&K 3560-C is a portable data acquisition unit with 17 input channels powered by a DC powered Type 2827 power supply unit.

In ambient vibration testing, the selection of testing time and sampling rates are of importance to ensure a proper signal quality. In this study, the authors conducted ambient vibration tests of the bridge in 20 min intervals with a selected sampling ratio of 200 Hz. This criteria was based upon earlier studies, specifically, those conducted by Bendat and Piersol (1986) in which the authors established a minimum of 17 min for the tests. Similarly, Ramos (2007) determined a minimum testing time of 10 min (1000 times the main period of the structure), and Caetano (2000) determined the time as 240–1280 times greater than the first period of the structure. The sampling rate defines the upper limit of the frequency band i.e., the number of the data samples acquired per unit of time.

The PULSE Lapshop (2006) and OMA (2006) software were used concurrently with the Enhanced Frequency Domain Decomposition (EFDD) technique to estimate modal parameters from the collected data. The EFDD technique, which entails the use EFDD uses the singular values of the spectral density matrices (Fig. 6), is used to decompose each of the estimated spectral density matrices. In these matrices, the singular values are the estimates of the auto spectral density of the single degree of freedom systems, and the singular vectors are the estimates of the mode shapes (Brincker et al. 2000). Equation (1) represents the relationship between the unknown input and measured response:

\[
G_{yy}(\omega) = [H(\omega)]^* [G_{xx}(\omega)] [H(\omega)]^T
\]

where \(G_{xx}\) is the \(r \times r\) power spectral density matrix of the input; \(r\) is the number of inputs; \(G_{yy}\) is the \(m \times m\) PSD matrix of the responses; \(m\) is the number of responses; \(H(\omega)\) is the

<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>

Table 2 Material properties considered in the analytical solution
Fig. 4 Uni-axial accelerometer locations on the bridge

Fig. 5 Data acquisition system and accelerometers used during tests

Fig. 6 Singular values of spectral density matrices obtained using EFDD technique
m × r FRF matrix; and superscripts * and T denote complex conjugate and transpose, respectively. A detailed solution of Eq. (1) can be found in Peeters (2000).

3.3 Determining the analytical and experimental dynamic characteristics

The first three natural frequencies determined from both finite element analysis and ambient vibration tests are between 3 and 10 Hz. The first two bending modes, primarily in the lateral (i.e. transverse) direction, and the third mode, primarily in the vertical direction are illustrated in Fig. 7. Here, the mode shapes are visually compared to ensure successful mode-pairing between analytical and experimental mode shapes. This comparison of the paired analytical and experimental natural frequencies yielded an average 10% deviation between the first three modes.

Fig. 7 Analytically and experimentally identified mode shapes and natural frequencies. a Analytical mode shapes and natural frequencies. b Experimental mode shapes and natural frequencies

Fig. 8 Restrictions on the boundary conditions of the bridge
<table>
<thead>
<tr>
<th>Ground motion</th>
<th>Station</th>
<th>Record</th>
<th>PGA (m/s²)</th>
<th>PGV (cm/s)</th>
<th>PGV/PGA (s)</th>
<th>Distance to fault (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Near fault</td>
<td>57476 Gilroy</td>
<td>LOMAP/GOF180</td>
<td>0.241 g</td>
<td>24.0</td>
<td>0.101</td>
<td>12.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>LOMAP/GOF090</td>
<td>0.284 g</td>
<td>42.0</td>
<td>0.151</td>
<td>12.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>LOMAP/GOF-UP</td>
<td>0.149 g</td>
<td>11.1</td>
<td>0.076</td>
<td>12.3</td>
</tr>
<tr>
<td>Far fault</td>
<td>57425 Gilroy</td>
<td>LOMAP/GMR000</td>
<td>0.226 g</td>
<td>16.4</td>
<td>0.074</td>
<td>24.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>LOMAP/GMR090</td>
<td>0.323 g</td>
<td>16.6</td>
<td>0.052</td>
<td>24.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>LOMAP/GMR-UP</td>
<td>0.115 g</td>
<td>5.6</td>
<td>0.053</td>
<td>24.3</td>
</tr>
</tbody>
</table>

Fig. 9 The time-histories of near fault ground motion acceleration components subjected to 1989 Loma Prieta earthquake. a LOMAP/GOF180 component, b LOMAP/GOF090 component, c LOMAP/GOF_UP component
3.4 Finite element model revision

The Osmanlı arch bridge is believed to have been built in the first half of the 19th century. Hence, the structure in its current state has accumulated material deterioration and geometric imperfections from natural events such as earthquakes, wind, snow, rain and floods. Surveys, observations and inspections of the bridge also show a change in condition, as seen in Fig. 8, specifically in the accumulation of soil debris around the sidewalls of the smaller of the two arches of the bridge. This debris aggregation in turn is believed to increase the constraints of the lateral stiffness of the bridge, ultimately leading to an increase in the sidewall stiffness.

Finite model revision entails using available experimental information to refine the model inputs, such as material properties of the structure and/or boundary conditions to obtain improved agreement with the observed reality (i.e., modal parameters measured

![Graphs](https://via.placeholder.com/150)

Fig. 10 The time-histories of far fault ground motion acceleration components subjected to 1989 Loma Prieta earthquake. a LOMAP/GMR000 component. b LOMAP/GMR090 component, c LOMAP/GMR_UP component
through ambient vibration testing). The authors adjusted the boundary conditions in the FE model. After the model refinement, there was an average reduction of 10% in the differences between the natural frequencies from 15 to 5%. The first three validated natural frequencies obtained were 4.64, 8.76 and 9.88 Hz, respectively. The results of the analytical modeling analysis derived the first two modes as lateral bending with the 67 and 24% modal participating mass ratio, respectively (Totally 91%). The third mode was a vertical bending with the 83% modal participating mass ratio.

4 Seismic behavior of the bridge

4.1 Near and far fault ground motions

In this section, the authors detail the near and far fault ground motions used in the time history analyses selection, which for our purposes was the 1989 Loma Prieta Earthquake with a magnitude of 7.1. The LOMAP/GOF and LOMAP/GFR ground motion acceleration records were used to represent the near and far faults, respectively, the properties of which are provided in Table 3. As clearly seen there, the peak ground acceleration (PGA) values are 0.284 and 0.323 g with the distances to the epicenter 12.3 km and 24.3 km, respectively (PEER 2014). The obtained peak ground velocity/peak ground

Fig. 11 Displacement contour through lateral direction for the near and far fault ground motions. a Displacement contour through lateral direction for near fault ground motion. b Displacement contour through lateral direction for far fault ground motion
acceleration (PGV/PGA) values were 0.151 and 0.052 s, respectively for the near and far faults (PEER 2014). To consider a record as near fault, the distance of record to epicenter should be shorter than 10–15 km, and PGV/PGA value should be larger than 0.1 s (Bayraktar et al. 2009b).

**Fig. 12** Displacement contour through vertical direction for near and far fault ground motions.  
(a) Displacement contour through lateral direction for near fault ground motion.  
(b) Displacement contour through vertical direction for far fault ground motion

**Fig. 13** Lateral displacements changing along to plan view III–III of the bridge
In this study, the components of the records in both the lateral vertical directions that are available in PEER Strong Motion Database (PEER 2014) were used. However, the main modes of vibration in the Osmanlı arch bridge were in the lateral direction, as indicated in Fig. 7. Although three components of both near and far fault are regularly used in conventional time history analyses, the authors used the LOMAP/GOF090 and the LOMAP/GMR090 to determine the lateral direction, also known as the crown-cross section of the bridge. Both the LOMAP/GOF090 and LOMAP/GMR090 have a larger PGV than the other components where the first natural mode, which has a greater mass percentage, was obtained. Therefore, in this analysis, the LOMAP/GOF090 component was used to simulate the near fault location and the LOMAP/GMR090 was used to simulate the far fault location. The time history acceleration records are provided for the near ground fault (Fig. 9a–c) and in far fault ground motions (Fig. 10a–c). The Newmark algorithm was used to perform the time history analyses for the both near and far fault ground motions considering the first 15 s of the time history records. Element matrices are computed using the Gauss numerical integration technique (Bathe 1996) and the Rayleigh damping was used to determine a damping ratio of 5 %. 

![Fig. 14](image-url) Vertical displacements changing along with plan view III–III of the bridge.

![Fig. 15](image-url) The time histories of lateral displacements for near and far fault ground motion. a Near fault displacements. b Far fault displacements.
4.2 Displacements

The lateral displacement contours (at the incident when the maximum displacement occurs) are provided in Fig. 11a, b for near and far ground motions, respectively. A comparison of these figures reveals larger displacements for the near fault ground motion than for the far fault ground motion. The vertical displacement contours, which are given for both near and far fault ground motion in Fig. 12a, b, respectively, yield a similar observation. Although the displacement of the represented figures shows a movement in opposite directions, with a maximum positive absolute the near fault, and a maximum negative absolute for the far fault. Consequently, the actual displacement must be an absolute value, which is accurately represented here.

The lateral and vertical displacements of the bridge through plan view III–III (recall Fig. 2) is given in Figs. 13 and 14, considering both the near and far fault ground motions. The displacements obtained from near fault ground motion model are 10 % higher than the far fault ground motion model.

The time histories of lateral displacements at the crown-point of the arch bridge are presented in Fig. 15a, b for the near and far fault ground motion models. Although far fault ground motion has higher acceleration value in lateral direction (recall Figs. 9, 10), the maximum displacements for near fault ground motions (13 mm) is higher than that of the far fault ground motions (12.3 mm).

Fig. 16 Maximum principal stress contour for near and far fault ground motion model. a Maximum principal stress contour for near fault ground motion. b Maximum principal stress contour for far fault ground motion
4.3 Maximum and minimum principal stresses

In this study, the plan views of II–II and III–III are represented respectively in Figs. 1 and 2, including both the small and large arches of the bridge. Because these arches represent...
the main structural system of the bridge that transfers the static and dynamic loads to the soil below, determining the effects of ground motion on these arches is important for assessing the integrity of the overall structure. Generally speaking, the main load carrying mechanism of a masonry arch is to withstand compressive stresses, and hence masonry arches tend to have low capacities for carrying tensile stresses. Such tensile stresses can occur on the structure under cyclic forces which are characteristic of earthquakes, making it important to ensure their resiliency against such maximum stresses. The maximum principal stresses that contour through the lateral direction at the instant at which the maximum stresses occur are shown in Fig. 16 for both the near and far fault ground motion. Once again, it can be seen that the maximum stresses occur at the middle of the plan view III–III of the bridge. The maximum principal (tensile) stresses obtained for near fault ground motion were nearly 50 % larger than those of the far fault ground motion model. It is possible that the limited tensile stress capacity of this type of masonry construction may result in damage to the main arch of the Osmanlı bridge, which is caused by the near fault ground motion.

The minimum principal stress contours (compressive), shown in Fig. 17a, b for both the near and far fault ground motion do not indicate stress levels that are significantly lower than the typical compressive capacity of stone masonry. The maximum principal values obtained as 2.38 and 1.60 MPa for both the near and far faults, respectively at the contour diagrams. However, determining the origin of the stress was most difficult; the closest approximation of that stress through arch (plan view III–III) elucidated stress values of 1.88 and 1.8 MPa for the near and far faults, respectively.

The maximum (tensile) and minimum (compressive) principal stresses received from the plan view II–II (Fig. 7) are shown in Fig. 18a, b. Similarly, the maximum and minimum principal stresses that occur on plan view III–III (Fig. 7) are given in Fig. 19a, b. Figures 18 and 19 support the previous observation that near fault ground motion results more dynamic response amplitude in the Osmanlı bridge even far fault ground motion has higher peak acceleration value. As clearly indicated in Figs. 18 and 19, the maximum values of the stress placed upon the bottom of the stone arch is clearly greater than stress placed upon the upper sides of the arch.

![Fig. 19 The maximum and minimum principal stresses at plan view III–III. a Maximum principal stresses. b Minimum principal stresses](image-url)
5 Conclusions

This paper presents the findings of a research campaign, in which ambient vibration testing, finite element model updating, and near and far fault ground motion analysis, were used to analyze a historical masonry arch bridge. The following observations were determined from this analysis:

1. The first three natural frequencies were estimated to range between 3 and 10 Hz for both finite element analysis and ambient vibration testing. The first two modes are bending modes in lateral direction, and the third mode is bending modes in vertical direction. While analytically and experimentally identified mode shapes demonstrate visual agreement with each other, an average 10% deviation was observed between the natural frequencies.

2. It was deemed necessary to adjust the boundary conditions of the FE model after an additional site survey and inspection. This model revision resulted in an average reduction of differences between natural frequencies from 15 to 5%.

3. The maximum displacements were observed to occur at the highest point of the bridge with displacements decreasing away from the top point of the bridge. The displacements obtained for the near fault ground motion are significantly higher (sometimes twice as high) than those of far fault ground motion model.

4. Although the far fault ground motion had a higher peak acceleration in the lateral direction, the resulting maximum displacements was higher for the near fault ground motion.

5. The maximum stresses occurred at the midpoint of the plan view III–III of the bridge. The maximum principal (tensile) stresses obtained for the near fault ground motion model were approximately 50% higher than those of the far fault ground motion.

6. Regarding the arch stone of the bridge, the near fault ground motion had a slightly greater affect on the dynamic behavior than the far fault ground motion, although the far fault ground motion exhibited a greater peak acceleration value. The maximum values of stresses towards bottom of stone arch were also higher than the upper sides.

The results obtained in this study demonstrate the effectiveness of using near fault ground motion to induce high dynamic responses in historical masonry arch bridges, and hence should be considered an integral part of the seismic assessment of these structures.

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