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3D FINITE ELEMENT MODEL OF THE BILL EMERSON CABLE-STAYED BRIDGE AND ITS CALIBRATION WITH FIELD MEASURED DATA

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ABSTRACT

The Bill Emerson Memorial cable-stayed bridge was instrumented with a seismic instrumentation system of 84-accelerometers. The instrumentation system provides an opportunity for real-time monitoring of the structural condition of the bridge. It generates a vast amount of data continuously. The ultimate goal of this study is to develop a structural health monitoring system using necessary data compression and system identification techniques. The focus of this paper is on the establishment of a three-dimensional finite element model (3-D FEM) of the Bill Emersion Memorial cable-stayed bridge and its calibration with field measured truck-induced vibration data. Frame elements were used to represent steel girders, floor beams and the main components of towers. Cable elements and shell elements were employed for steel cables and post-tensioned precast concrete panels/slabs, respectively. The natural frequencies and mode shapes of the bridge model were determined with modal analysis. Truck-induced vibration data were obtained from the 84-channel seismic instrumentation system and used to calibrate the 3-D FEM. In addition, the results from the 3-D FEM were compared with those from two simplified models. It was determined from the 3-D FEM that the fundamental frequency of the bridge is approximately 0.34 Hz, corresponding to the vertical vibration of the bridge deck. This frequency is in excellent agreement with the field measured data but is 10% higher than that of the simplified models.

Keywords: Cable-stayed Bridge, Finite Element Model, Field Measured Data, Data Analysis, Dynamic Characteristics

INTRODUCTION

The Bill Emerson Memorial Bridge was opened to traffic on December 13, 2003. The bridge is a long-span cable-stayed structure that crosses the Mississippi River near Cape Girardeau, Missouri. It is located approximately 80 km from New Madrid, Missouri, where three of the largest earthquakes in the U.S. continent occurred. During the winter of 1811-1812 alone, this seismic region was shaken by a total of more than 2000 events, over 200 of which were evaluated to have been moderate to large earthquakes. Each of the three most significant earthquakes has magnitude of above 8.0. Tremors occur in the New Madrid Seismic Zone on a daily basis. In the past two years, two earthquakes with magnitude of over 4.0 have already been recorded in the New Madrid Seismic Zone. Therefore, it is expected that the bridge will experience one or more significant earthquakes during its life span of 100 years.

Due to its criticality and proximity to the New Madrid Seismic Zone as well as lack of design ground motions, the cable-stayed bridge and its adjacent area were installed with an 84-channel seismic

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instrumentation system (Celebi et al., 2004). The so-called ASPEN system consists of a total of 84 Kinemetrics EpiSensor accelerometers, Q330 digitizers, and Baler units for data concentrator and mass storage. Antennas were installed at Piers 2 and 3, one free field site at each end of the bridge, and the central recording building near the bridge so that wireless communication of data can be initiated among various locations as well as from the bridge and free-field sites to the off-structure central recording building. The accelerometers installed throughout the bridge structure and adjacent free field sites allow the recording of structural vibration of the bridge and free-field motions at the surface and down-hole locations. They were deployed such that the acquired data can be used to understand the overall response and behavior of the cable-stayed bridge, including translational, torsional, rocking and translational soil-structure interactions at foundation levels.

This paper is mainly focused on the development of a 3-D FEM of the cable-stayed bridge and its calibration with field measured data. The characteristics of the bridge structure will be evaluated using the FEM. They will be compared with the results obtained from the measured data and those from other simplified computer models (Caicedo et al., 2000).

DESCRIPTION OF THE CABLE-STAYED BRIDGE

Jointly owned by the Missouri and Illinois Departments of Transportation, the cabled-stayed bridge structure was proportioned to withstand an M7.5 or stronger design earthquake (Woodward-Cycle Consultants, 1994). The 30% seismic load combination rules for earthquake component effects were used in accordance with the American Association of State Highway and Transportation Officials (AASHTO) Division I-A Specifications (AASHTO, 1996). They were then combined with the dead load applied to the bridge. As schematically shown in Fig. 1, the final design of the bridge includes two towers, 128 cables and 12 additional piers in the approach span on the Illinois side.

The bridge has a total length of 1206 m. It consists of one 350.6 m long main span, two 142.7 m long side spans, and one 570 m long approach span on the Illinois side. The main span of the bridge provides more than 18.3 m of vertical clearance over the navigation channel. The 12 piers on the approach span have 11 equal spacing of 51.8 m. Carrying two-way traffic, the bridge has four 3.66 m wide vehicular lanes plus two narrower bicycle lanes. The total width of the bridge deck is 29.3 m as shown in Fig. 2 for a typical cross section. The deck is composed of two longitudinal built-up steel girders, a longitudinal center strut, transverse floor, and a precast concrete slab. A concrete barrier is located in the center of the bridge, and two railings and additional concrete barriers are located along the edges of the deck.

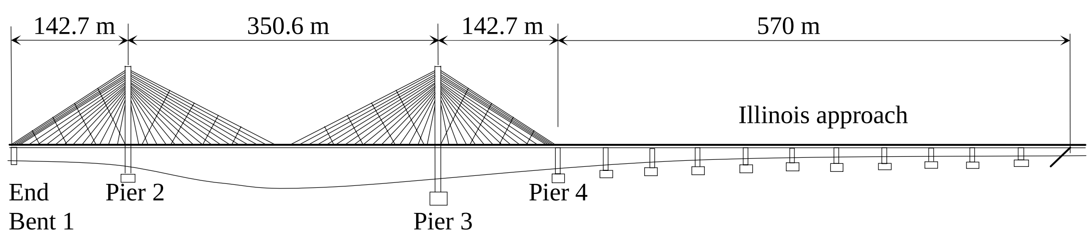


Figure 1. Schematic view of the Bill Emersion Memorial cable-stayed bridge.

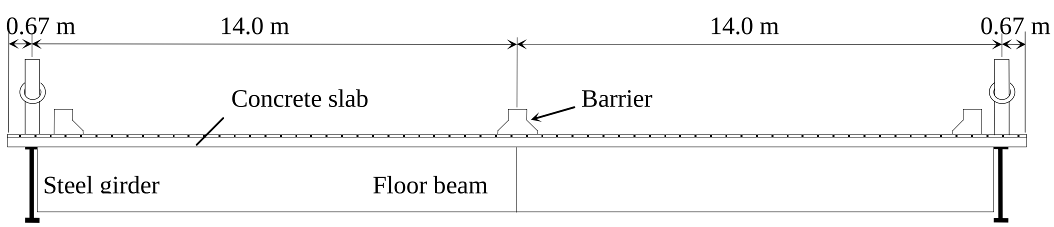


Figure 2. Typical cross section of the bridge deck.

SEISMIC INSTRUMENTATION NETWORK

All the data recorded from the Bill Emerson Memorial cable-stayed bridge since March, 2005 are stored in a single network called "NP" in the Incorporated Research Institutions for Seismology system (IRIS: http://www.iris.edu). A total of 43 stations and 84 channels are listed in Table 1. Here, HN2 represents the transverse/lateral component perpendicular to the traffic direction, HN3 means the traffic direction of the bridge or longitudinal component, and HNZ is the vertical component. The stations and channels are distributed on the bridge as illustrated in Fig. 3. Each arrow in Fig. 3 indicates one channel of acceleration data. The seismic instrumentation system on the Bill Emerson Memorial cable-stayed bridge continuously provides the structural vibration and soil responses at free field sites. As such, it can be used for real-time monitoring of structural condition.

Table 1. Designation of Station and Channels

Station	Channels	Station	Channels	Station	Channels
7405.B1	HN2 HN3 HNZ	7405.L1	HNZ	7405.P6	HN2 HN3 HNZ
7405.C1	HNZ	7405.L2	HN2 HNZ	7405.P7	HNZ
7405.C2	HN2 HNZ	7405.L3	HNZ	7405.P8	HN2 HNZ
7405.D1	HN2 HN3 HNZ	7405.L4	HN2 HNZ	7405.R1	HNZ
7405.D2	HN2 HN3 HNZ	7405.L5	HNZ	7405.R2	HN2 HNZ
7405.D3	HN2 HN3 HNZ	7405.L6	HN2 HNZ	7405.R3	HNZ
7405.E1	HNZ	7405.M1	HN3	7405.R4	HN2 HNZ
7405.E2	HN2 HNZ	7405.M2	HN2 HN3	7405.R5	HNZ
7405.E3	HN2 HN3 HNZ	7405.M3	HN3	7405.R6	HN2 HNZ
7405.F1	HN2 HN3 HNZ	7405.M4	HN2 HN3	7405.T1	HN2 HN3
7405.F2	HN2 HN3 HNZ	7405.P1	HN3	7405.T2	HN2 HN3
7405.F3	HN2 HN3 HNZ	7405.P2	HN2 HN3	7405.T3	HN2 HN3
7405.F4	HN2 HN3 HNZ	7405.P3	HN3	7405.T4	HN2 HN3
7405.F5	HN2 HN3 HNZ	7405.P4	HN2 HN3		
7405.F6	HN2 HN3 HNZ	7405.P5	HNZ		

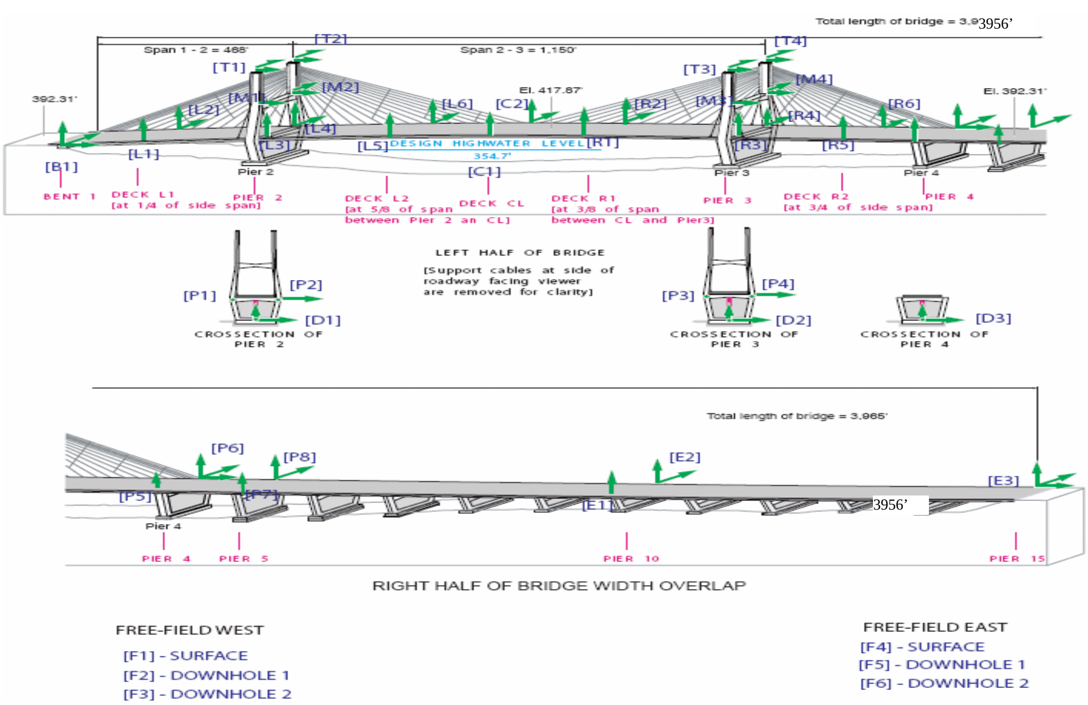


Figure 3. Location of station and channels.

FINITE ELEMENT MODEL

A 3-D FEM of the Bill Emersion Memorial cable-stayed bridge was developed using SAP2000 structural analysis software, as shown in Fig. 4. The bridge was modeled based on the geometries and material data from as-built drawings with necessary updated modifications in consultation with the Missouri Department of Transportation. The FEM employed frame elements for steel girders, floor beams and the center strut connecting any two adjacent floor beams. The main components of the bridge towers and pile caps were also represented by frame elements. The 280 mm thick precast concrete panel/slab supported by the steel girders was modeled with shell elements. In the model, cables were modeled using cable elements. The complete FEM has a total of 2012 joints, 2120 frame elements, 244 shell elements, and 274 rigid link elements for a total of 10326 degrees of freedom.

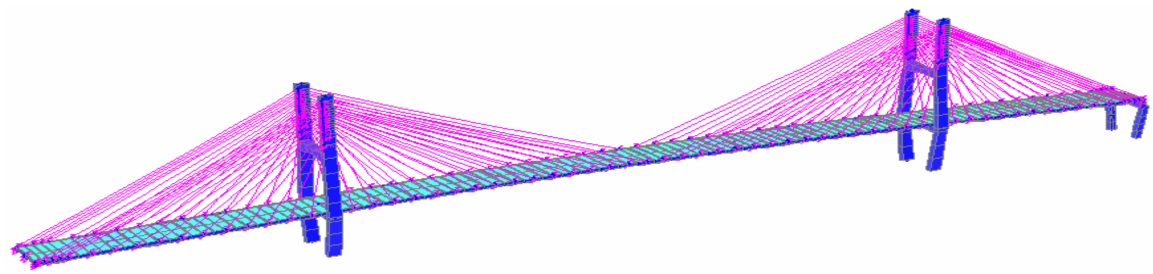


Figure 4. Computer model of the cable-stayed bridge.

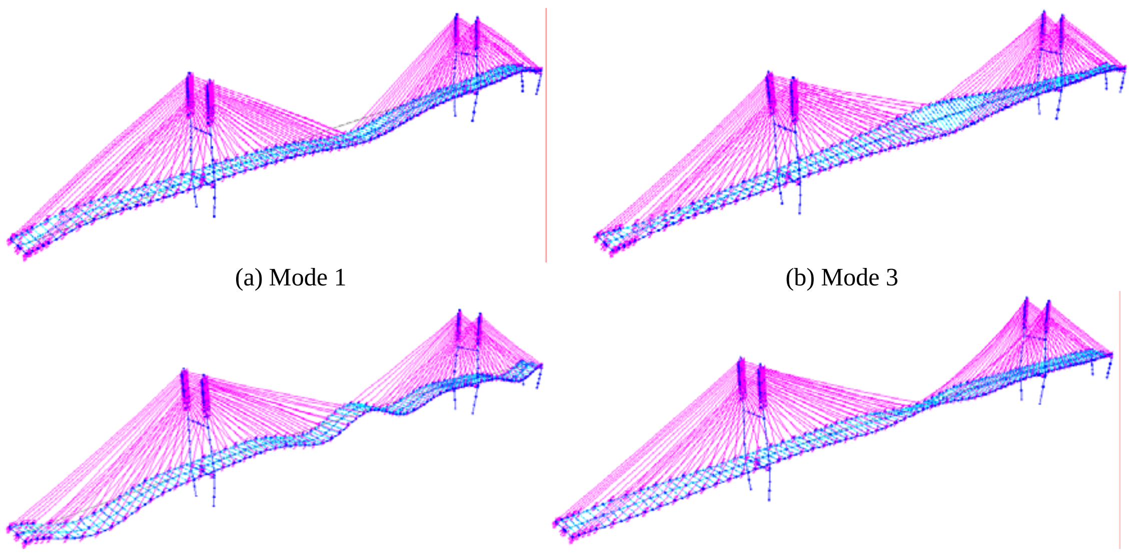
Each cable element was restrained in compression to prevent any compression deformation and simulate its practical condition on the actual bridge. Since each cable is attached at one end to the top flange of one composite steel girder and at the other end to the work point of the tower, both attachment points are away from the neutral axes of their respective supporting structural elements (deck and tower). Therefore, two rigid links were introduced to connect the cable to the neutral axis of the deck and the tower, respectively. The use of rigid links ensures that the theoretical lengths and horizontal angles of the cables are exactly the same as designed. In order to model the bridge accurately, non-prismatic members such as pier cap beams were represented by elements of varying section properties in the FEM. In addition, the elevation difference both in transverse direction and in longitudinal direction due to the designed slope and vertical curve was taken into account as well. The pot bearings used between steel girders and pier cap beams at Piers 2 and 3 were modeled to allow for the longitudinal translation and free rotation about any axis. The earthquake lateral restrainers at the center of the floor beam at Piers 1 to 4 were modeled to provide lateral restraints between the floor beam and cap beam at Piers 1 to 4. Two earthquake shock transmission devices were installed beside each pot bearing, which will limit the longitudinal movement in the event of a strong earthquake but nearly free to move under slowly-varying conditions such as thermal effects. As such, they were modeled as a hinge in the longitudinal direction for seismic analysis in this study.

Table 2. Natural Frequencies, Modal Mass Ratios and Description of Mode Shapes

	3-D FEM			Spine Model		C-Shape Model	
Mode	Frequency (Hz)	Modal Mass (%)	Mode Description	Frequency (Hz)	Mode Description	Frequency (Hz)	Mode Description
1	0.3385	Z=6.3	Vertical	0.2978	Vertical	0.3034	Vertical
3	0.4847	Y=0.13	Torsion	0.5264	Torsion	0.4711	Torsion
5	0.5981	Z=0.8	Vertical	0.6306	Torsion	0.6019	Vertical
7	0.6986	Z=18.4	Vertical	0.6772	Vertical	0.6791	Vertical
9	0.7522	Y=23.4	Lateral	0.8600	Vertical	0.7323	Vertical

The natural frequencies of the first 5 significant vibration modes obtained from the FEM are listed in Table 2, along with the description of dominant motions and their respective mass participating ratios. Each of the five modes corresponds to a significant mass participating ratio in the excitation direction consistent with the particular vibration mode. For example, Y is for lateral or torsion motions, and Z is for vertical vibration. The natural frequencies from the Spine model and C-shape model (Caicedo et al., 2000) are also listed in Table 2 for comparison.

The shapes of four significant vibration modes (No. 1, 3, 7, and 9) are presented in Fig. 5. Corresponding to the fundamental frequency of 0.3385 Hz, the first vibration mode mainly involves the vertical vibration of the entire bridge deck. The third mode corresponds to the torsion of the bridge deck that is weakly coupled with longitudinal motion of the towers. The seventh mode is another vertical vibration mode engaged with a larger portion of the bridge structure. The ninth mode is dominated by the lateral motion of the towers and deck that is coupled with torsion of the deck.



(a) Mode 1 (b) Mode 3

(c) Mode 7 (d) Mode 9

Figure 5. First four mode shapes of the cable-stayed bridge.

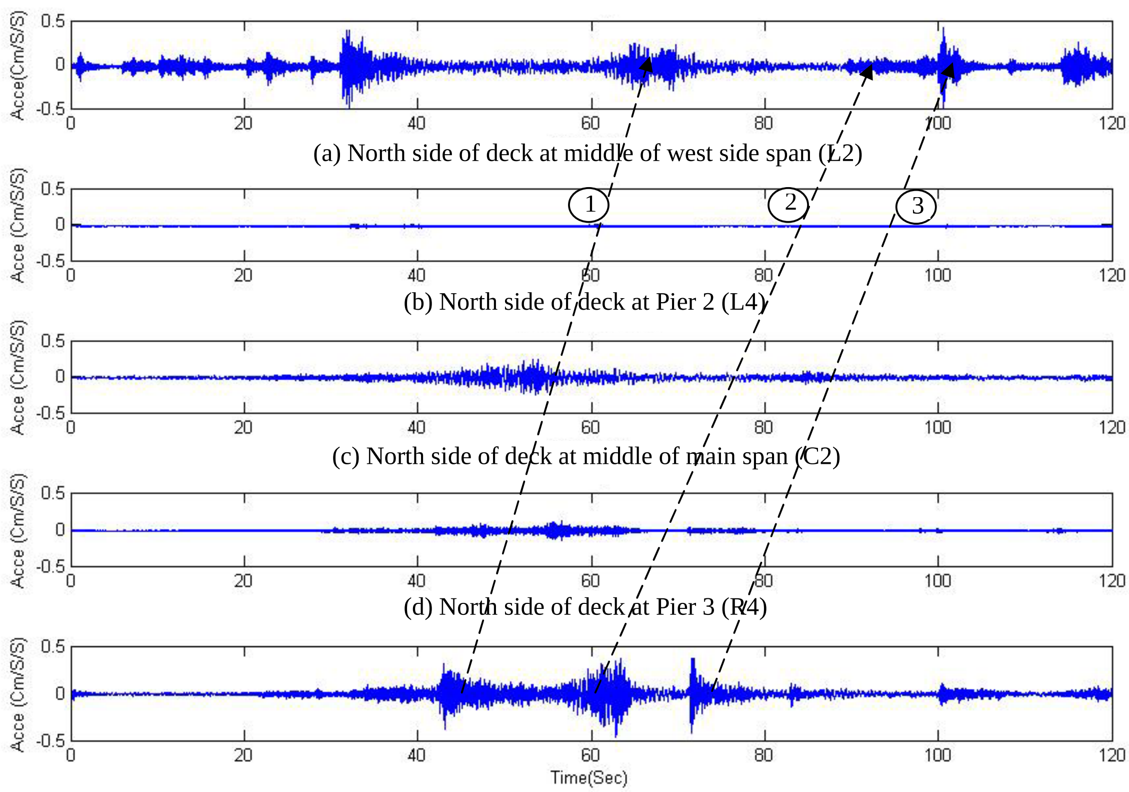
In comparison with the natural frequencies of the C-shape model (Caicedo et al., 2000), the fundamental frequency of the FEM is 10% higher. In good agreement with the measured data as will be discussed later, the higher frequency from the FEM indicate that the simplified model underestimated the stiffness of the bridge structure due likely to the neglect of diaphragm actions of the bridge deck between any two floor beams. The shapes of the four dominant modes with significant mass participations seem comparable between the FEM and the C-shape model. As pointed out in the study (Caicedo et al., 2000), the Spine model is even less accurate. Indeed, the natural frequencies determined by the C-shape model are all closer to those of the FEM in this study.

MEASURED TRAFFIC-INDUCED VIBRATION AND PRELIMINARY ANALYSIS

To understand the general characteristics of the low-amplitude vibration and calibrate the FEM of the bridge, three minutes of traffic-induced vibration data in two time periods 19:20’40” to 19:22’40” and 19:39’40” to 19:40’40” of July 25, 2006 were taken. It is noted that, although there was a $M_{cd}2.2$ earthquake occurred at 19:35’39” (Universal Time) on July 25, 2006 in southeastern Missouri (36.76N and 89.49W), the response at the bridge site is negligible. The vibration data used in this study were induced by traffic vehicles at slightly different time windows. Following is a presentation of the raw data and preliminary analysis of the vertical, transverse, and longitudinal responses at the bridge deck and towers.

Vertical Vibration of the Bridge Deck

The vertical accelerations along the length of the bridge deck are illustrated in Fig. 6 for Channels L2, L4, C2, R4, and R6 over a period of two minutes. It is clearly shown that the responses of the deck at the towers (Channels L4 and R4) are much smaller than those at other locations due to the support condition by the towers. Although it is difficult without video images of the traffic condition to identify the vehicles that resulted in the deck vibration, it is likely that there were three distinct events as marked by numbered dashed lines in Fig. 6. The north side of the bridge deck carries the westbound traffic on the state highway 74 as directed by the dashed lines. If a car or truck was driven at 48~97 km/h, the time required to move the vehicle from R6 to L2 is approximately 18~36 Sec, which is consistent with the slope of the three dashed lines in Fig. 6. It is speculated that, along the first path, a group of cars drove through the middle of east side span at approximately 43 Sec and arrived at the middle of west side span at 65 Sec. Along the third path, a heavy truck may drive through the bridge at a slightly slower speed. Another group of cars may drive though the bridge at a continuously reduced speed along the second path. The acceleration on the north side of the deck may also be somewhat affected by the eastbound traffic along the south side of the bridge deck.



(a) North side of deck at middle of west side span (L2)

(b) North side of deck at Pier 2 (L4)

(c) North side of deck at middle of main span (C2)

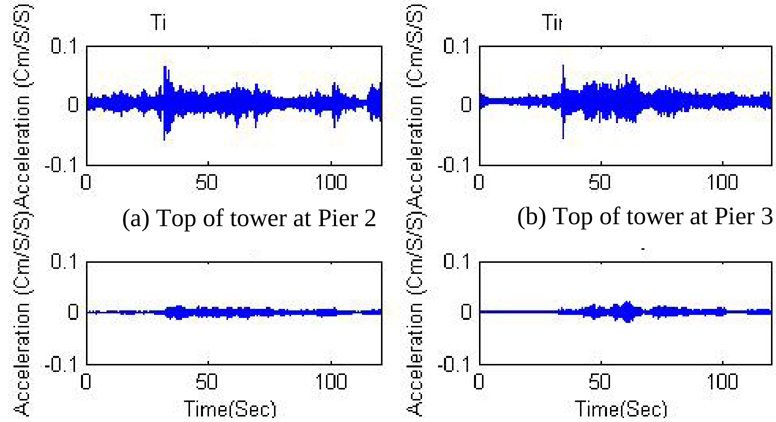
(d) North side of deck at Pier 3 (R4)

(e) North side of deck at middle of east side span (R6)

Figure 6. Vertical accelerations at deck during the period 19:20’40” to 19:22’40”.

Transverse/Lateral Vibration

Traffic-induced vibration is weak, particularly in the longitudinal and transverse directions. It also has limited bandwidth. Therefore, it is expected that some of the vibration modes may not be triggered by traffic loading. To see the general variation of the lateral vibration, Fig. 7 shows the lateral accelerations at the top and middle of the towers at Piers 2 and 3. It is seen that the vibration at the top is significantly stronger than that at the middle of tower. Both are much weaker than the vertical vibration at the bridge deck presented in Fig. 6.



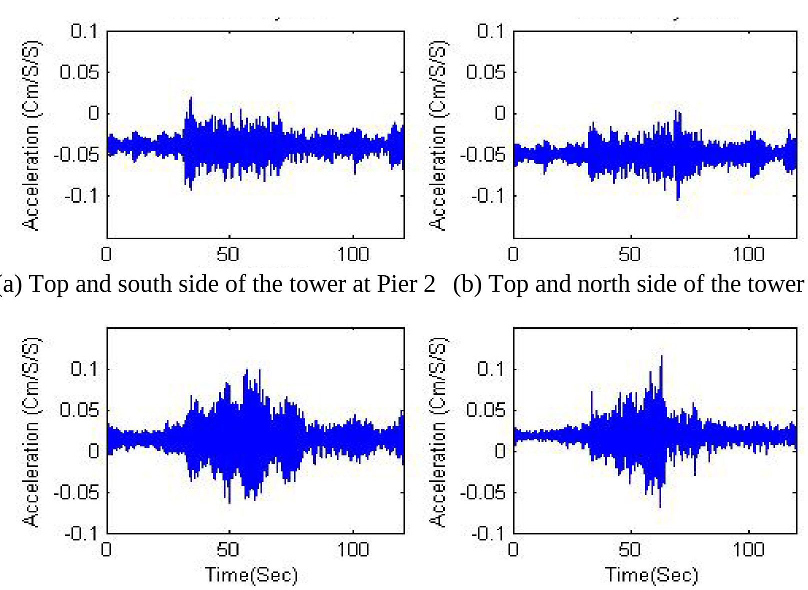
(a) Top of tower at Pier 2 (b) Top of tower at Pier 3

(c) Middle of tower at Pier 2 (d) Middle of tower at Pier 3

Figure 7. Lateral accelerations at bridge towers during the period 19:20’40” to 19:22’40”.

LONGITUDINAL VIBRATION OF THE BRIDGE TOWER

The longitudinal accelerations at the top of two towers are shown in Fig. 8. It is clearly seen that the vibration levels are similar at two sides of each tower, but quite different at two towers due to the passage effect of vehicular traffic. Overall longitudinal vibration is small in comparison with the vertical vibration in the bridge deck as shown in Fig. 6.



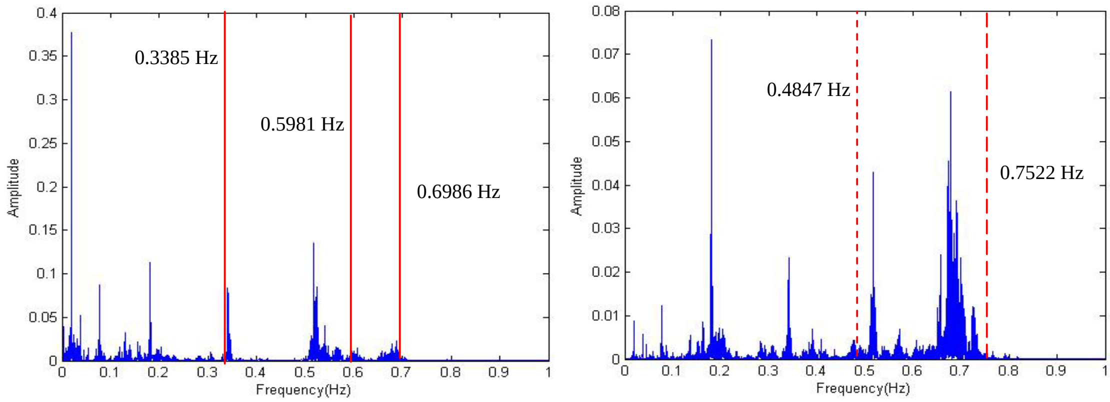
(a) Top and south side of the tower at Pier 2 (b) Top and north side of the tower at Pier 2

(c) Top and south side of the tower at Pier 3 (d) Top and north side of the tower at Pier 3

Figure 8. Longitudinal accelerations at towers during the period 19:20’40” to 19:22’40”.

CORRELATION BETWEEN MODEL PROPERTIES AND MEASURED DATA

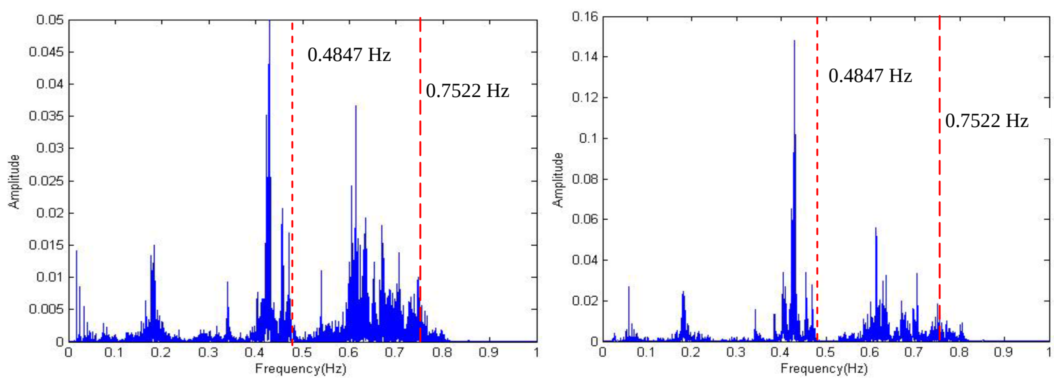
The Fourier spectrum of the vertical acceleration time history on the north side of deck at the middle of east side span was evaluated and presented in Fig. 9(a) up to 1 Hz for clarity. Also marked in the figure are the three calculated natural frequencies from Table 2. It is clearly observed that the fundamental frequency agrees well with the measured frequency at 0.34 Hz. This value is similar to the natural frequency identified from the earthquake-induced data (Celebi, 2006). The other two frequencies shown in Fig. 9(a) also correspond to the two frequencies but with much less vibration energy concentration. Note that raw measured data were processed in all Fourier spectra presented in this paper and low frequency noises are present as seen in Fig. 9.



(a) Vertical acceleration (b) Lateral acceleration

Figure 9. Fourier spectrum of the acceleration time histories on the north side of deck at middle of east side span (Channel R6) during the period 19:20’40” to 19:22’40”.

To understand the effects of vehicle dynamics on the identification of natural frequencies, the Fourier spectra of the lateral acceleration time histories in two time windows are plotted in Fig. 10. As one can see, the vibration energy in the two different time windows is concentrated around the same frequency ranges. This comparison suggests that the frequencies corresponding to the peak Fourier transform amplitudes are nearly independent of the dynamics of vehicles. These frequencies are mainly related to the natural frequencies of the bridge structure.



(a) From 19:20’40” to 19:22’40” (b) From19:39’40” to 19:40’40”

Figure 10. Fourier spectra of lateral accelerations at middle of bridge deck (Channel C2).

Fig. 10 also indicates that the calculated natural frequency of the lateral vibration mode (0.7522 Hz) corresponds to the measured frequency range of locally concentrated energy. In comparison with Fig. 9(b), Fig. 10(a) indicates that east side span is less involved in the lateral vibration mode. This observation is consistent with the mode shape of the lateral vibration mode. As indicated in Fig. 10, the calculated torsion mode frequency (0.4847 Hz) seems different from its corresponding measured frequency.

To understand the coupling effect between the lateral and vertical motions, Fig. 9(a, b) compares the Fourier spectra of the lateral and vertical acceleration time histories at the middle of west side span. It can be seen that the fundamental vibration mode seems weakly coupled with the lateral vibration at a frequency of 0.3385 Hz.

CONCLUSIONS AND RECOMMENDATIONS

Based on the finite element analysis of the Bill Emerson Memorial cable-stayed bridge and the preliminary analysis of traffic-induced bridge vibration data, some conclusions can be drawn from this study. The fundamental frequency of the Bill Emerson Memorial cable-stayed bridge is approximately 0.34 Hz, corresponding to the vertical vibration of the entire bridge deck. The finite element model of the bridge can accurately predict the fundamental frequency as well as a few frequencies of higher vibration modes. Detailed modeling on the various main components of a complex cable-stayed structural system is critical in accurately simulating the structural response and behavior. The diaphragm effect of the bridge deck can not be neglected unless special considerations are made. The simplified models without this effect give the fundamental frequency more than 10% lower than the measured value. The first ten vibration modes include approximately 25% of the total mass in lateral and vertical directions each. The traffic-induced vibration amplitude in lateral and longitudinal directions is less than 25% of the vertical motion. A refined analysis of the traffic-induced or earthquake-induced data is needed to further calibrate the FEM for higher natural frequencies and corresponding mode shapes.

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