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Dynamic properties of the Painter Street Overpass at different levels of vibration

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ABSTRACT: This paper describes the results from a series of ambient vibration studies conducted on the Painter Street Overpass in Rio Dell, California. Painter Street is a two-span, skewed reinforced concrete bridge with two single piers near the middle and monolithic abutments, typical of bridge overpasses in California. Strong motion instruments were installed on the bridge in 1977, and since then it has recorded the motions from more than ten significant earthquakes. Because of the valuable amount of strong motion data available, the aim of the ambient vibration tests was to determine the dynamic characteristics of the bridge at low levels of vibration and to compare these with those measured during the strong motion events. In this paper, a description of the recorded strong motion events is presented first, then the ambient vibration tests are described and the results are compared with those obtained from analyses of selected strong motion records. The magnitude of the events investigated ranges from M_L=4.4 to M_L=6.9, which produced accelerations of up to 0.54g at the free field site, 1.3g at the abutments, and 0.86g on the deck. The results of this study indicate that the overall dynamic properties of the bridge are very sensitive to the level of ground shaking and that soil-structure interaction is very important for this type structural system. Although the superstructure exhibited a nearly elastic response, the motions at the abutments and base of piers were significantly different for each event.

1 INTRODUCTION

The damage to bridges caused by recent earthquakes in the last decade has demonstrated the need to assess the seismic resistance of existing bridges built before the advent of modern seismic design codes. Because of this need, a great deal of effort has been placed on developing economical and effective seismic retrofit methods for bridges in order to minimize the potential damaging effects of earthquakes. Effective seismic retrofit studies for a specific bridge benefit greatly from knowledge about its actual dynamic characteristics, as these may be used as part of the seismic assessment study to calibrate computer models of the structure. The Painter Street Overpass in Rio Dell, in northern region of California offers a great opportunity to assess existing methods of analyses to determine the seismic behaviour of bridges. This bridge was instrumented with 20 accelerometers by the California Strong Motion Instrumentation Program (CSMIP) in collaboration with the California Department of Transportation (CALTRANS) in 1977. Because of the high level of seismic activity in the region, the motions of more than ten earthquakes have been well recorded by the instruments installed on the bridge and its vicinity. An ambient vibration study of the bridge was also performed by CALTRANS (Gates and Smith, 1982) as part of a comprehensive series of vibration tests on 57 seven bridges in California. Researchers of the University of British Columbia (UBC), conducted a series of ambient vibration tests on this bridge, and on another two bridges in the area, during in 1993. The results of these tests have been discussed previously by Ventura, Finn and Felber (1995). This paper includes the results of a more detailed analysis of this data making use of recent advances on analysis techniques of ambient vibration data.

2 BRIDGE DESCRIPTION AND STRONG MOTION INSTRUMENTATION

The Painter Street Overpass (PSO) is a two-span, prestressed concrete box-girder bridge that was constructed in 1973 to cross over the four-lane US Highway 101 in Rio Dell, California (see Fig. 1). Its construction is typical of the type of California bridges used to span two or four lane highways. The bridge is 15.85 m wide and 80.79 m long (Fig. 2a).

Figure 1. North side view of Painter Street Overpass.

The deck is a multi-cell box girder, 1.73 m thick and is supported on monolithic abutments at each end and a two-pier bent that divides the bridge into two spans of unequal length; one of the spans is 44.51 m long and the other is 36.28 m long. The abutments and piers are supported by concrete friction piles and

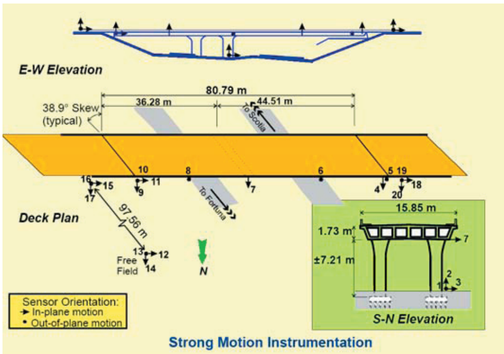

Figure 2. PSO dimensions and strong motion instrumentation.

are skewed at an angle of 38.9••. Longitudinal movement of the west abutment is allowed by means of a thermal expansion joint at the foundation level. The piers are about 7.32 m high and each is supported by 20 concrete friction piles. The east and west abutments are supported by 14 and 16 piles, respectively. The bridge was instrumented as part of a collaborative effort between CSMIP and CALTRANS to record and study strong motion records from bridges in California. Twenty strong motion accelerometers were installed at the site (see Fig. 2.)

2.1 *Strong Motion Data*

The 10 most significant earthquakes recorded to date are summarized in Table 1 and the relative epicentral location and magnitude for each earthquake with respect to the bridge is shown in Fig. 3. The size of the circles in the figure is proportional to the magnitude of the event being represented. Most of the earthquakes occurred southwest of the bridge, in the vicinity of the San Andreas Fault. Table 1 also includes the peak horizontal accelerations recorded at the free-field and on the structure during each event. Recorded peak ground horizontal accelerations from these events range from .08g to 0.54g, while horizontal structural accelerations range from .10g to 1.09g. Although large structural accelerations have been recorded, no significant structural damage has been observed at the bridge. The extent of damage has been limited to settlement of the backfill and some cracking of the concrete. All but the records from the Petrolia event of 1991 (91ML6.0) have been digitized and processed by CSMIP.

Table 1 Significant Earthquakes Recorded at PSO

Event Code	Earthquake (Date)	Mag. (M_L)	Dist (km)	Acc. (g) FF	Acc. (g) Str.
80ML6.9	Trinidad Offshore (8/11/80)	6.9	88	0.15	.17
82ML4.4	Rio Dell (16/12/82)	4.4	15	--	.42
83ML5.5	Eureka (24/8/83)	5.5	61	--	.22
86_1ML5.1	Cape Mendocino1 (21/11/86)	5.1	32	.43	.40
86_2ML5.1	Cape Mendocino2 (21/11/86)	5.1	26	.14	.35
87ML5.5	Cape Mendocino (31/7/87)	5.5	28	.14	.34
91ML6.0	Petrolia (17/9/91)	6.0	37	.08	.10
92ML6.9	Cape Mendocino – Petrolia (25/4/92)	6.9	24	.54	1.09
92ML6.2	Cape Mendocino-Petrolia [AS1] (26/4/92)	6.2	42	.52	.76
92ML6.5	Cape Mendocino-Petrolia [AS2] (26/4/92)	6.5	41	.26	.31

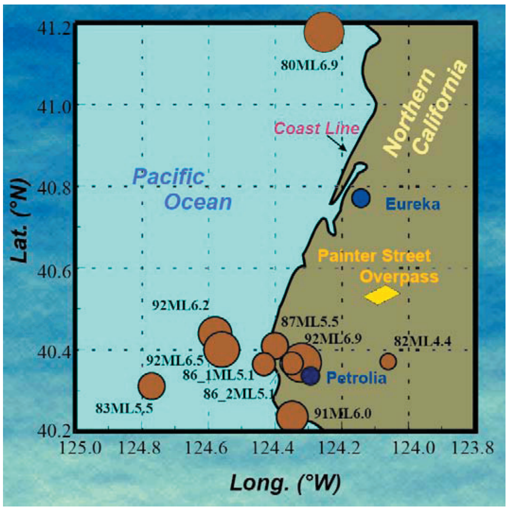

Figure 3. Epicentre and Magnitude of recorded earthquakes.

The strong motion records from the Painter Street Overpass have been studied in detail by several investigators. Maroney, Romstad and Chajes (1990) attempted to correlate the inferred natural periods from the recorded motions with those obtained by a finite element model of the bridge. The results were found to be very sensitive to the choice of the equivalent spring stiffness of the abutments. Goel and Chopra (1994) investigated the variation of abutment stiffness during strong shaking and the effect of torsional motions of the deck. Makris, et al (1994) used the 1992 Cape Mendocino earthquake records to calibrate a simple procedure to evaluate soil-foundation-superstructure interaction of pile-supported bridges. McCallen and Romstad (1994) used the same set of records to investigate two different approaches to modelling the response of simple bridge structures, a stick model and a detailed, large scale three-dimensional finite element model, including soil nonlinearities. Goel (1997) used this set of records to demonstrate that the lateral period and damping ratio more than double as the intensity of ground shaking increased, and that these changes are primarily due to abutment participation with increased intensity of shaking. As part of a series of studies of the ground motions recorded near the bridge, researchers at UBC have investigated the bridge behaviour at very low levels of excitation and compared this behaviour with that for strong motion.

3 AMBIENT VIBRATION STUDIES

The purpose of this study was to determine key dynamic characteristics of bridge during low amplitude vibrations produced by wind, traffic and micro-tremors. It was aimed at providing an improved insight into how the different components of bridges interact dynamically to aid in the selection of seismic retrofit alternatives. The series of tests at the bridge included vibration measurements of the superstructure, abutments, backfill, pile caps and the free field. Details of these tests are given in Ventura, Finn and Felber (1995).

The selected measurement locations and orientations of the accelerometers are shown in Fig. 4. An eight-channel, accelerometer-based, data acquisition system was used to collect the data. Three of the accelerometers were installed permanently at location 0 in Fig. 4, while the other five were continuously moved from location to location until all the desired sites were measured. Each setup included installing the sensors, recording their signals for about 15 minutes and relocating them to the next locations.

The authors recently re-analysed the recorded accelerations using computer program ARTeMIS Extractor (SVS, 2004), which was developed by the second and third authors. This program was specifically developed for the analysis of ambient vibration data and implements in a very efficient way frequency-domain and time-domain signal processing techniques. In the frequency domain approach the identification is based on the singular value decomposition of the spectral density matrix.

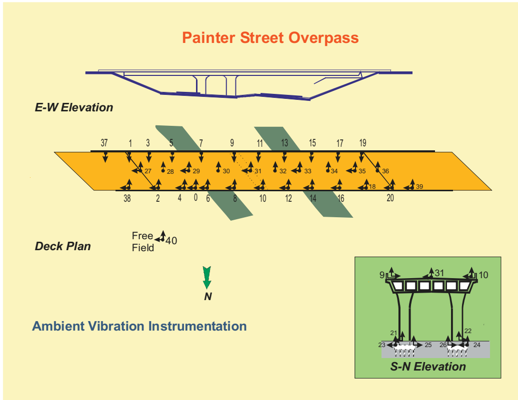

Figure 4. Sensor locations during ambient vibration tests.

The singular values are interpreted as a combination of auto spectral densities for a set of single degree of freedom (SDOF) systems. The user of the program identifies the modal parameters by visual evaluation of the plot identifying the SDOF bell densities. This technique is denoted Frequency Domain Decomposition (FDD) and is recognized today as the most time efficient and accurate of the frequency domain techniques. The technique deals effectively with closely spaced modes and with noise. It also gives a clear indication of the presence of harmonic components in the records due to equipment or machinery operating on the structure when the vibration response was measured. In the time domain approach the user can perform a very accurate modal identification using up to three different types of data driven Stochastic Subspace Identification (SSI) techniques. The techniques use all response data available estimating a full model in discrete time, and the user has the option to specify model stabilization criteria based on damping, natural frequency, mode shape and initial amplitude. The modal identification can be validated by comparing modal results from different identification techniques used for the same project.

The data analysis was limited to the evaluation of vertical and transverse modes of vibration in the frequency range 0 to 10 Hz. The vertical modes of vibration of the bridge were excited rather well by the highway traffic underneath the bridge and could be identified quite readily. This was a particular advantage since secondary effects associated with vertical modes contributed significantly to the observed lateral motions. As a first step in the data analysis, the average of the normalized PSDs, called ANPSD, was computed separately for all the vertical and horizontal signals recorded on the deck, and the result is shown in Fig. 5. For the frequency range shown, four significant peaks clearly identify the first four vertical modes of vibration of the bridge. For the transverse direction, the frequency of the fundamental mode is very distinct.

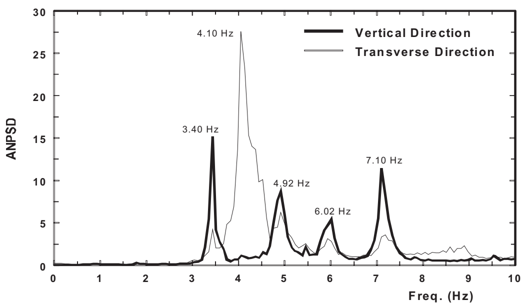

Figure 5. ANPSD of ambient vibration records

The results of the FDD analysis are shown in Fig. 6, in which the average of the first four normalized singular values of the spectral density matrices of all the records is presented. Both Figs. 5 and 6 convey similar information about the dominant frequencies present if the records, but Fig. 6 has the added benefit of helping to separate operating frequencies from natural frequencies of vibration (the peaks below 3 Hz all correspond to operating frequencies). The FDD method estimates modal damping values. The resulting damping ratios were 1%, 2%, 2%, 1.4% and 2.2% for each of the five modes, respectively.

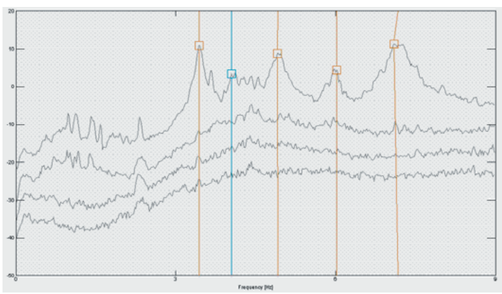

Figure 6. Singular values of spectral density matrices.

The corresponding mode shapes to the identified frequencies are shown in Fig. 7. The mode at 3.40 Hz is a very well defined vertical mode with very small transverse components. The modes at 4.92, 6.02 and 7.10 Hz show significant torsional and translational components. This is also apparent in Fig. 5 where the peaks for the transverse direction are significant at these frequencies. The transverse mode at 4.10 Hz does not exhibit significant vertical or torsional components. It is clear that the fundamental vertical and transverse modes are well defined and have unique directions; the higher modes have significant components in three directions.

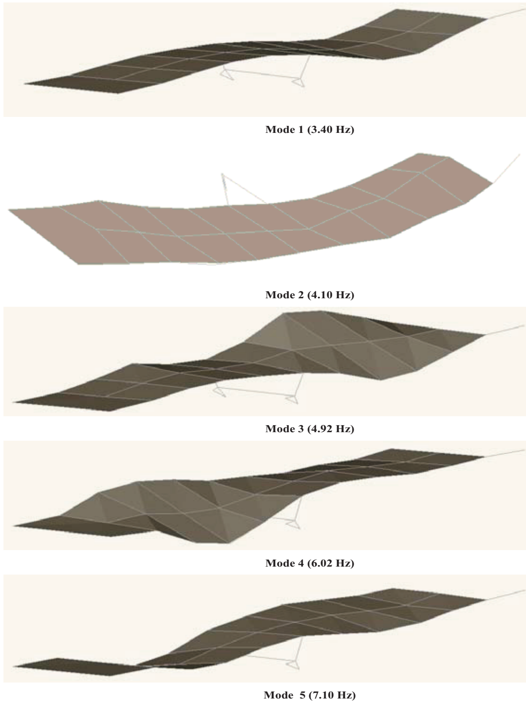

Figure 7. Mode shapes of PSO below 10Hz

The ambient vibration study by Gates and Smith in 1982 reported four frequencies within the same range: a) 3.61 Hz and 7.28 Hz for the first and second vertical modes, respectively, and b) 4.49 Hz and 7.42 Hz for the first and second transverse modes, respectively. The difference between identified values for the fundamental frequencies is less than 10%. The largest difference occurs for the second transverse mode, the difference is in this case about 16%. Gates and Smith, however, did not identify two vertical modes between 3.6 and 7.28 Hz.

4 COMPARATIVE ANALYSES

A frequency domain analysis of strong motion records and ambient vibration signals from selected locations of the bridge was conducted and the results are reported here. Because of space limitations the discussion is limited to characteristics of the PSDs for the ambient vibration data and selected strong motion records from Table 1. The events considered here are 80ML6.9, 82ML4.4, 83ML5.5, 86_1ML5.1, 86_2ML5.1, 87ML5.5, and 92ML6.9.

4.1 *Vertical Motion*

In order to identify and compare the frequency ranges in which the response of the bridge was prominent, the PSD of each record considered was normalized with respect to its peak value. The normalized PSDs for the vertical component of accelerations recorded near the centre of the east-side span of the bridge (location 6 for strong motion instrumentation and vertical component of location 16 for the ambient vibration instrumentation) are plotted in Fig. 8.

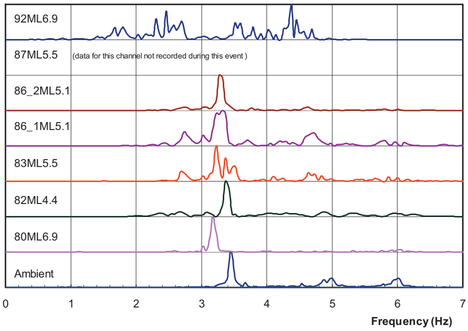

Figure 8. Normalized PSDs of vertical motion of PSO at location 6 of strong motion instrumentation.

Peaks in the vicinity of the first three vertical modes identified in the ambient vibration study (3.40, 4.92 and 6.02 Hz) are also apparent in most of the strong motion records. As expected, the shift of the fundamental frequency toward lower values during strong shaking is noticeable, especially for the larger events in which the shift is largest (up to 10% of variation). A shift of the peaks toward lower frequency values for the higher modes can also be noticed. The various peaks in the PSD of 92ML6.9 indicate that the fundamental period of the bridge may have changed during the event. The smallest event (82ML4.4) was the closest to the PSO (15 km) does not seem to have affected significantly the frequencies of the bridge in the vertical direction. In contrast, the most distant event, and with a large magnitude (80ML6.9) resulted in a reduction of the fundamental frequency of about 0.20 Hz.

4.2 *Horizontal Motion*

The normalized PSDs for the horizontal component of accelerations recorded on the deck, just above the top of the north-side pier (location 7 for strong motion instrumentation and transverse component of location 10 for the ambient vibration instrumentation) are plotted in Fig. 9. The shift towards lower values of the first transverse mode identified in the ambient vibration study (4.10 Hz) is very noticeable in this case. The most significant peak for event 92ML6.9 occurs at about 2.2 Hz, which is almost 50% of the frequency of the fundamental mode. The presence of various peaks in the vicinity of 2.2 Hz for this event indicates that the lateral frequency of the bridge may have changed during the event. A system identification study by Goel (1997) utilizing time-frequency domain techniques showed that the frequency of the bridge varied from 1.78 Hz to 2.86 Hz during this event. Goel's study was concentrated on how the frequency of this mode changed only.

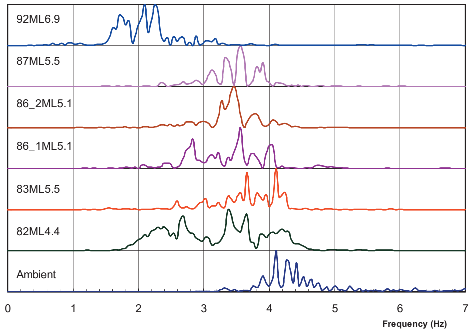

Figure 9. Normalized PSDs of horizontal motion of PSO at location of sensor 7 of strong motion instrumentation.

The variation of frequency observed in the 92ML6.9 is about 1 Hz, which is also observed in the 83ML5.5, 86_2ML5.1, and 87ML5.5 events. The variation is larger for the 86_2ML5.1 event (about 1.5 Hz), and much larger for the 82ML4.4 (about 2 Hz). The latter event happens to be the one with the smallest magnitude, but closer epicentral distance.

Since no significant structural damage has been observed at the bridge during all the events, it can be argued that the variations of natural frequencies are closely related to the location and level of shaking, and how these affect the site of the bridge. Results of research have indicated that the bridge superstructure, abutments and approach embankment soil constitute a strongly coupled system. The dynamical behaviour of the foundation and embankment soil has a first order influence on the dynamic response of the bridge superstructure. Analysis of measured strong motion response data has also indicated that localized non-linear behaviour of the embankment soil can result in significant non-linear global behaviour of the entire system, even when the bridge superstructure remains linear.

5 CONCLUSIONS

An extensive ambient vibration study of the Painter Street Overpass has been conducted, and some of the most significant findings from this study have been presented here. The frequencies of the fundamental modes of vibration in the vertical and transverse directions of the bridge have been identified at 3.40 Hz and 4.10 Hz, respectively. The salient features of recorded strong motions at the Painter Street Overpass bridge have been presented and compared with the recorded ambient vibration motions. The comparative analyses showed that the events investigated excited the vertical modes of vibration of the bridge more than its transverse modes of vibration. The results of indicate that the superstructure exhibited a nearly elastic response for all the events and that the fundamental frequencies tended to lower values as the level of shaking increased.

The set of records from the PSO help understand better the importance of including abutment-soil system in the structural idealization of this type of bridges. Ambient vibration and strong motion records provide valuable information on the range of stiffness values of abutment-soil springs that can be used for more realistic analyses, rather than having to rely on some simplified rules and trial-and-error process.

ACKNOWLEDGMENTS

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