Earthquake Engineering, Tenth World Conference © 1992 Balkema, Rotterdam. ISBN 90 5410 060 5

Seismic performance evaluation of suspension bridges

A.M.Abdel-Ghaffar & S.F.Masri
Civil Engineering Department, University of Southern California, Los Angeles, Calif., USA
A.-S.M.Niazy
Structural Research and Analysis Corporation, Santa Monica, Calif., USA

ABSTRACT: A well-instrumented suspension bridge in Los Angeles was shaken by the October 1, 1987 Whittier earthquake ($M_L = 5.9$) which generated the most comprehensive set of data on the response and the multiple-support excitation of a bridge yet recorded from a damaging earthquake anywhere in the world. This study addresses the following objectives: (1) three-dimensional modeling; (2) temporal and spectral analyses of the records; (3) modal identifications, including estimation of damping values; (4) a comparative study of the modes of vibration from the devised 3-D analysis, previous analytical studies, the ambient vibration survey and the identified properties from the earthquake records; (5) envelopes of maximum earthquake-induced displacements and straining actions, and (6) recommendations for upgrading the present state of the bridge instrumentation and for damping augmentation devices at critical zones. Reasonable agreement was found between the calculated and measured dynamic characteristics and response quantities.

1 EARTHQUAKE AND SUSPENSION BRIDGES

The Whittier earthquake is considered to be the largest to strongly shake a modern suspension bridge in the last quarter century. The main shock generated the most comprehensive set of data (from the Vincent-Thomas Suspension Bridge) on the bridge structural response yet recorded from a damaging earthquake anywhere in the world. Because of the great wealth of information, the bridge's natural dynamic characteristics are identified from its recovered seismic records and are compared with those from previous studies (by Abdel-Ghaffar, et al., 1976 and 1978) on the bridge dynamics and its ambient vibration survey. Moreover, validation of the devised 3-D bridge model is discussed.

In this paper, the analysis procedure for suspension bridges experiencing asynchronous support motions is utilized. The accelerograms recovered from the bridge foundations during the earthquake are used to define a set of input motions including the rocking motion at the east tower (Niazy, et al., 1991).

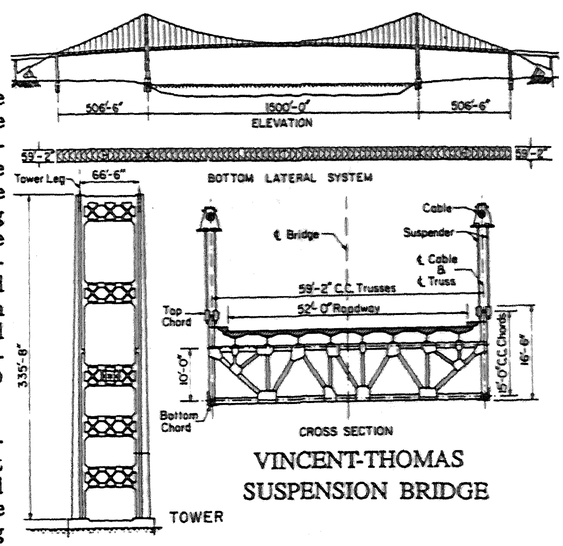

Figure 1 Structural details of the bridge.

2 THE VINCENT-THOMAS BRIDGE

The Vincent-Thomas Suspension Bridge (Fig. 1), which was constructed in the early 1960's, is located about 35 km southwest of the earthquake epicenter. The suspended structure consists of two stiffening trusses, floor trussed beams and a lower chord wind bracing of K-truss type; these elements form a box-system having relatively high torsional rigidity. A detailed description of the bridge can be found in the report by Abdel-Ghaffar, 1976.

3 STRONG-MOTION INSTRUMENTATION

Figure 2 shows the layout of the 26 sensors of the strong-motion instrumentation (installed and maintained by the California Division of Mines and Geology) on and near the bridge. The maximum recorded acceleration on the bridge sub- and super-structure were 0.07g and 0.30g, respectively, and the maximum displacement on the deck was equal to 8.71 cm, which is considered very small compared to the bridge dimensions.

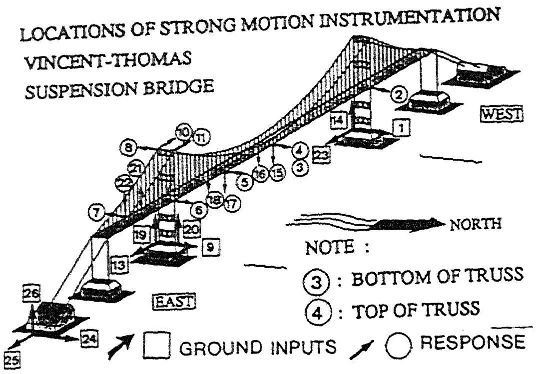

Figure 2 Locations and direction of sensors.

4 FINITE ELEMENT MODELING

To reduce the structural model to a manageable size and yet account for all structurally significant features of different members, and to make the model more tractable for the seismic analysis and the comparison with measured response quantities, a 3-D global model is developed. This bridge model, is idealized as assemblies of finite elements, including 3-D frame and truss elements, (Fig. 3).

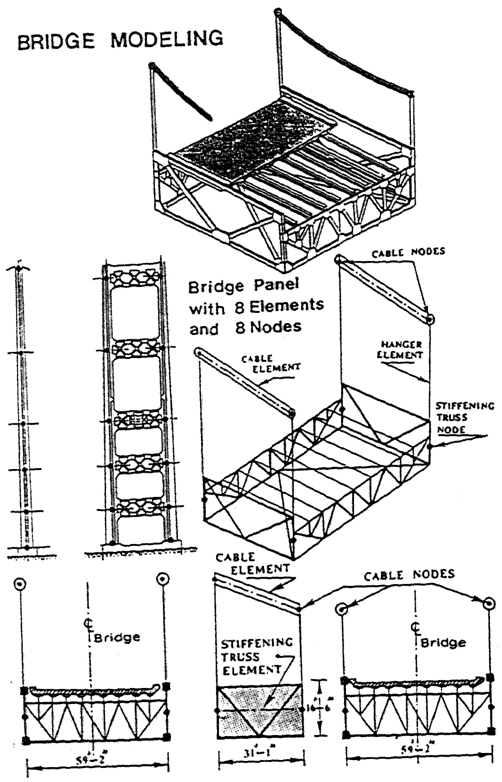

Figure 3

5 FREE VIBRATION CHARACTERISTICS

The tangent stiffness matrix of the bridge in its dead-load deformed configuration, obtained through an iterative nonlinear static analysis, is utilized in solving the eigenvalue problem. The results provided the 100 lowest closely spaced modes in the frequency range of 0.169 Hz to 5.904 Hz. By examining the computed modes of vibration, the following comments can be made:

1. Most modes can be categorized as vertical, torsional and lateral (Figs. 5 and 6). Within the 100 calculated modes there are 16 dominantly vertical, 32 dominantly lateral and 16 dominantly torsional; 28 modes are purely cable modes, and 8 are coupled tower-bridge modes.
2. Strong coupling in the three orthogonal directions occurs in a number of modes (Fig. 4). The dominant lateral motion of the bridge deck is strongly coupled with torsional deck and tower vibrations, moreover the primarily torsional vibrations are associated with lateral deck motion (cable vibrations are also present within these modes). Furthermore, the bridge towers' dominantly longitudinaly dominant motion is associated with vertical deck vibrations.
3. In the pure cable modes the two cables vibrate laterally, either in phase or 180°-out-of-phase with respect to the vertical plane of the single cable (not the bridge center line).

The above-mentioned complicated patterns cannot be captured by 2-D modeling.

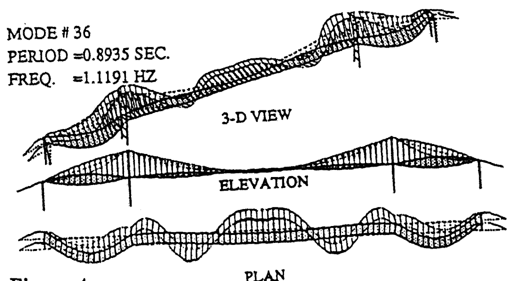

Figure 4

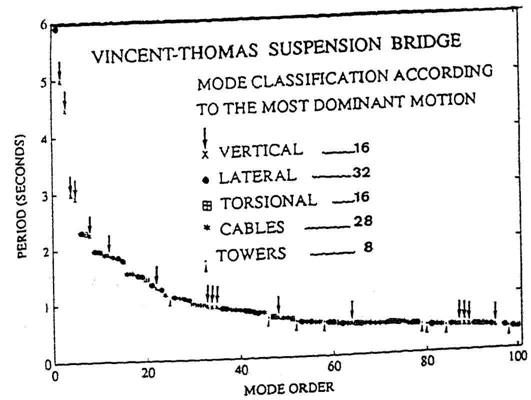

Figure 5

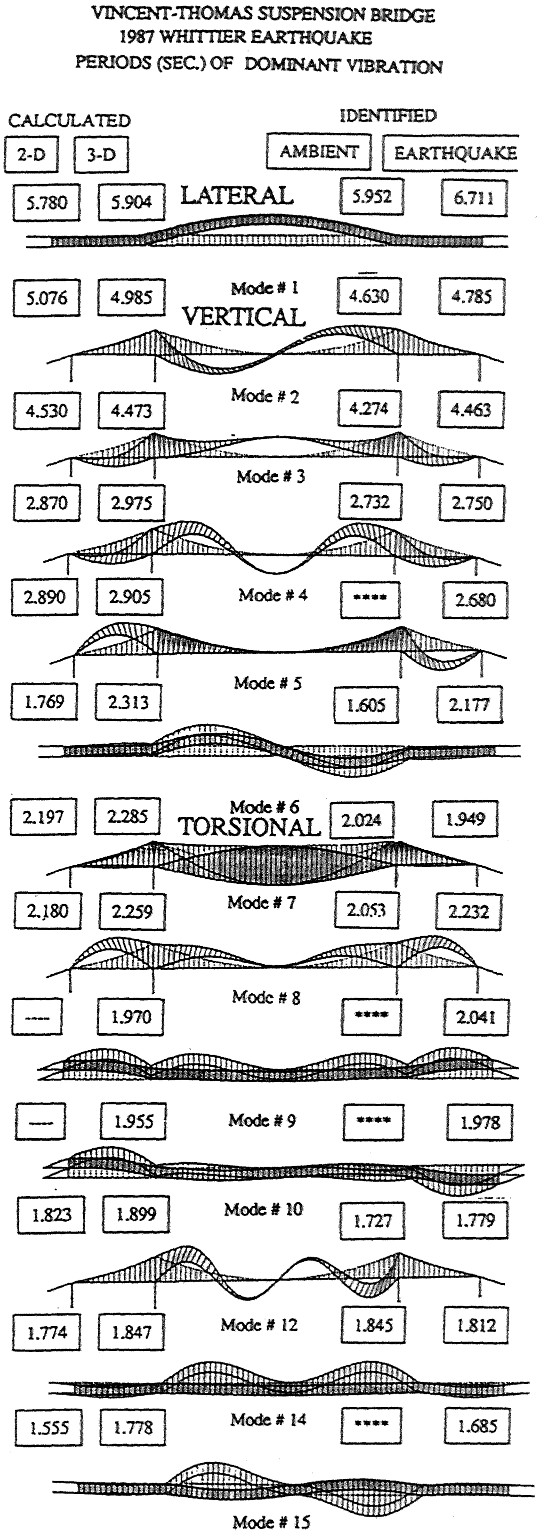

Figure 6

6 TEMPORAL AND SPECTRAL ANALYSES

The addition and subtraction of the records recovered from the accelerographs at different locations of the bridge significantly enhanced the identified directional mode shapes and their associated frequencies. The records were analyzed to obtained the Fourier amplitude- and auto-spectra of individual records, and the cross correlation, coherence and phase spectra of records from two different locations (Fig. 7,8).

6.1 *Substructure Analysis*

Examination of the temporal and spectral correlation analyses of the records recovered from the two tower bases (piers) and the east anchor indicates that the characteristics of the recorded motions at the west tower-pier were significantly different from those of the east tower-per and east anchor, particularly in the lateral (or north) direction (Fig. 7). The nonuniform motion at these supporting locations considerably influences the nature of the bridge response. Differences in soil conditions at these sites, the soil-structure interaction and the nature of the propagating seismic waves can all account for these nonuniform motions. Another point of interest is the rocking motion of the east tower-pier; the vertically recorded motions showed some phase and amplitude differences (Fig. 7). Such rocking motion contributes

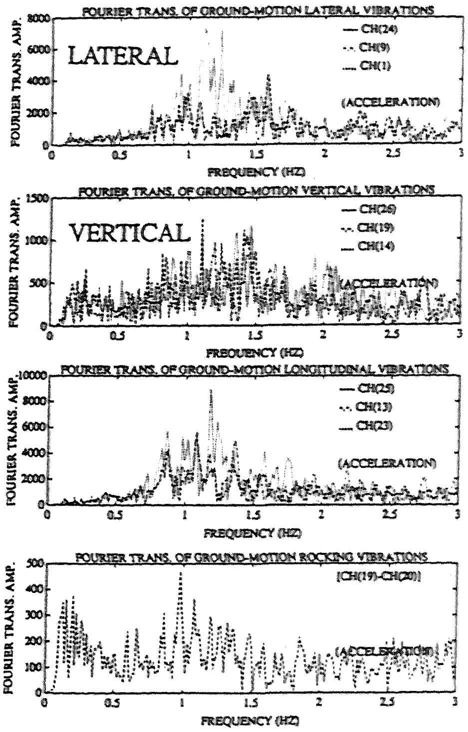

Figure 7

to the torsional deck motion (see Fig. 2). Information regarding the variation in ground motion and the rocking vibration at the west anchorage and the cable bents is not available due to the lack of instrumentation at these locations.

6.2 *Superstructure Analysis*

The addition of the recorded vertical motions of the bridge deck significantly enhanced the identified vertical bending mode shapes by eliminating torsional motion (Fig. 8). The torsional modes of the bridge vibration are obtained by subtracting the vertical (or lateral) recorded motions at the trusses of the same cross section (Figs. 2 and 8). Modes of lateral (or swinging) vibration were determined by adding the transverse recorded motions of the two accelographs at the mid-point section of the center span as well as by using the individual records (Figs. 2 and 8). The dominantly longitudinal tower modes are enhanced by adding the recorded longitudinal motions at the top of the tower legs; torsional modes are obtained by subtracting these records. The coherence-function spectral amplitude is almost equal to one at the identified modal frequencies (Fig. 8). Figure 6 shows a comparative study of modes and their natural frequencies from the analytical investigations (by Abdel-Ghaffar, 1976), the ambient vibration survey (by Abdel-Ghaffar and Housner, 1978) and the identified properties from earthquake records; this study reveals a reasonably good agreement between the measured and calculated dynamic properties. As in the results of the (small amplitude) ambient vibration survey, several closely spaced modes of vibration were evident in the results from the earthquake records. Furthermore, the excitation of the antisymmetric modes (Fig. 6), confirms the effect of spatial variation of nonuniform input motions at the substructure.

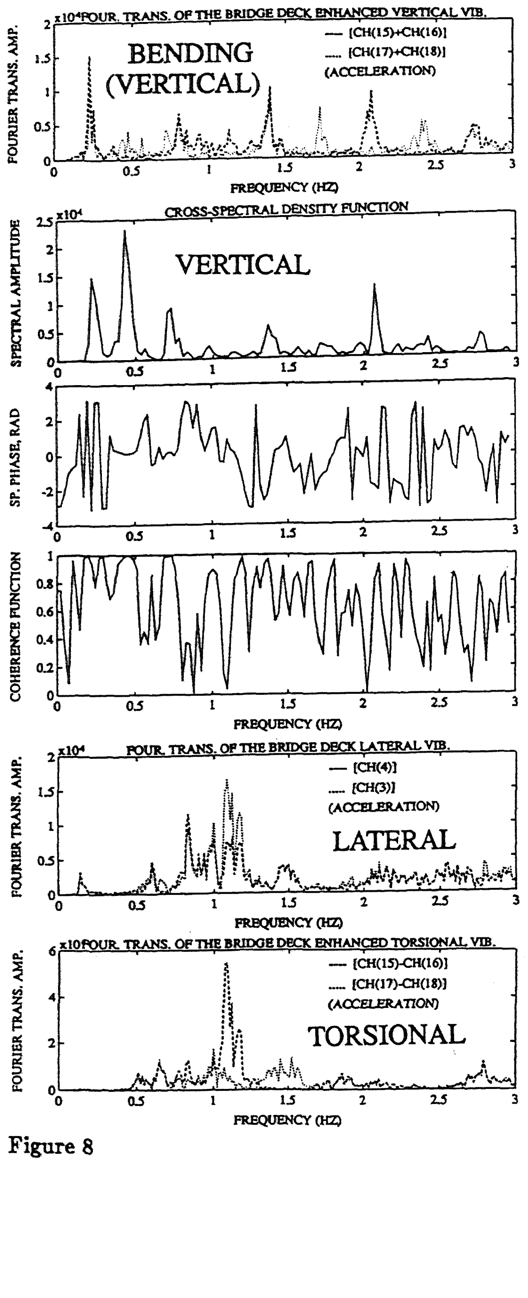

Figure 8

7 IDENTIFICATION OF DAMPING VALUES

In general, realistic estimates of the energy dissipation occurring in an inherently low-damping vibrating bridge, where a large number of closely spaced modes contributes to the response, can not yet be made with confidence. In an effort to minimize the difference between the calculated and the measured responses, and in examining the applicability of different modal damping estimations, a trial-and-error modification is made. Despite the disadvantages of uniqueness, and the large number of parameters, this trial-and-error method proved to be superior in attaining a *better solution.*

8 EARTHQUAKE RESPONSE ANALYSIS

Nonlinear geometric effects due to cable tension and tower axial-compression are taken into account by using geometric stiffness matrices. The dynamic analysis is then performed using the tangent stiffness of the dead-load deformed state. The general solution of the total modal displacements may be decomposed into quasi-static displacements (the cross product of the support motions and the corresponding quasi-static functions; Fig. 9) and relative (or vibrational) displacements (resulting from the fixed-base solutions). The Whittier earthquake records recovered from the bridge foundations are utilized (both the acceleration and the integrated displacement) as direct inputs at the bridge supporting points. Since there were no available records at the west anchorage and the west cable bent foundations their motions are assumed to be similar to those of the west tower pier. Similarly, the motions at the base of the east cable bent are taken to be similar to those recovered from the east anchorage location.

9 SEISMIC RESPONSE CHARACTERISTICS

In general, reasonable agreement is obtained between the measured and the computed responses at the sensors locations. Envelopes of induced maximum displacements and straining actions along the stiffening girder are illustrated in Fig. 10; based upon these values stresses are computed. It is evident that the stress level within the main and side spans was very low (compared with the steel yield stress of 36 ksi). Also the asymmetric pattern of the induced response quantities within the main span and the side spans can be ascribed to the excitations of antisymmetric modes due to the nonuniform support excitations.

Displacement and stress envelopes along the elevation of the tower shafts are shown in Figs. 7 and 8. There is a significant increase in the y-shear and z-bending moments from above to below-roadway levels in the two shafts. This increase could be a direct result of the transfer of the lateral inertia forces from the main and side spans to the towers through the hinged connections of the lateral wind bracing systems. Also, the increase in the y-bending moments at the bottom of the shafts can be ascribed to the longitudinal inertia forces transferred from the side spans through the hinged joints at the towers locations.

It is worth mentioning that the additional (absolute) seismic force, in any of the cables or hangers, does not exceed 11% of the initial tension due to dead-load effect; none of these members experience any slackness or buckling during the shaking.

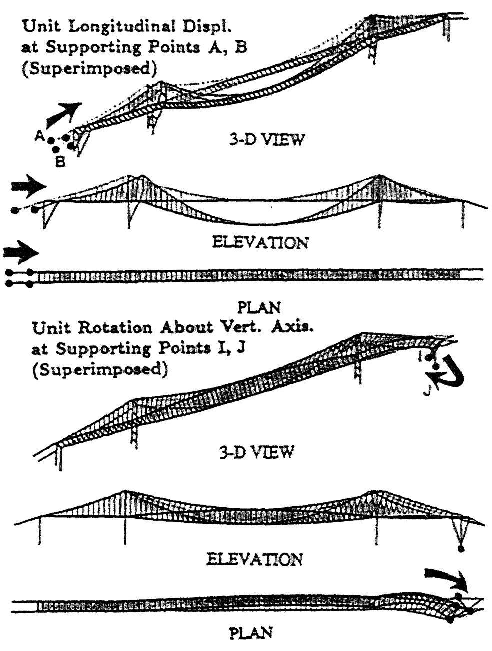

Figure 9 Examples of the quasi-static functions.

10 CONCLUSIONS

Analysis of the seismic records of a suspension bridge, in conjunction with the results from a 3-D modeling and the previous results of both analytical models and ambient vibration studies are used to evaluate its performance during the 1987 Whittier earthquake. Based on this study, the following conclusions can be made:

1. Spatial variations of recorded motions at the

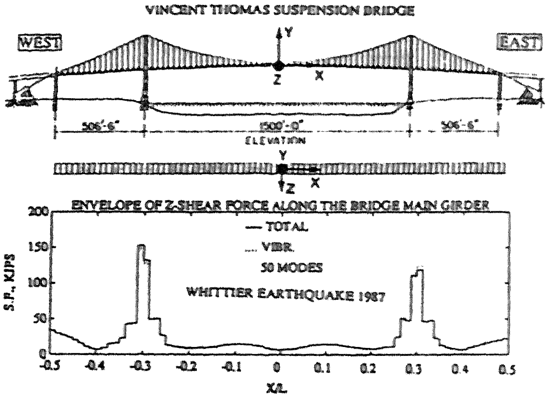

Figure 10 Envelope of maximum lateral shear.

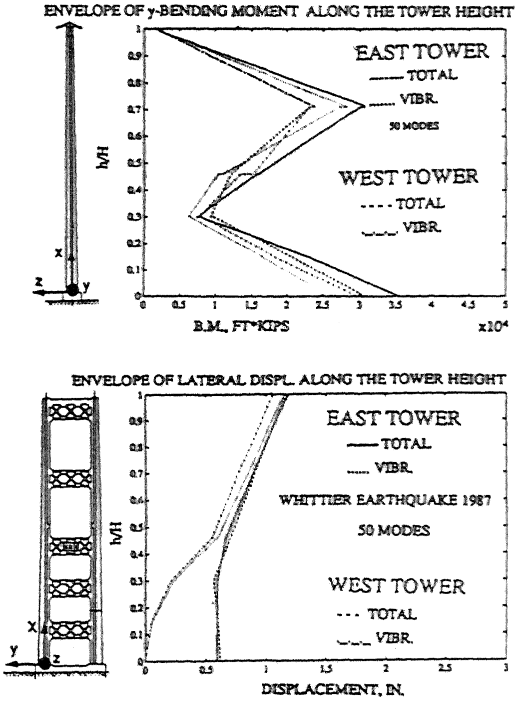

Figure 11 Envelopes of maximum moment and displacements.

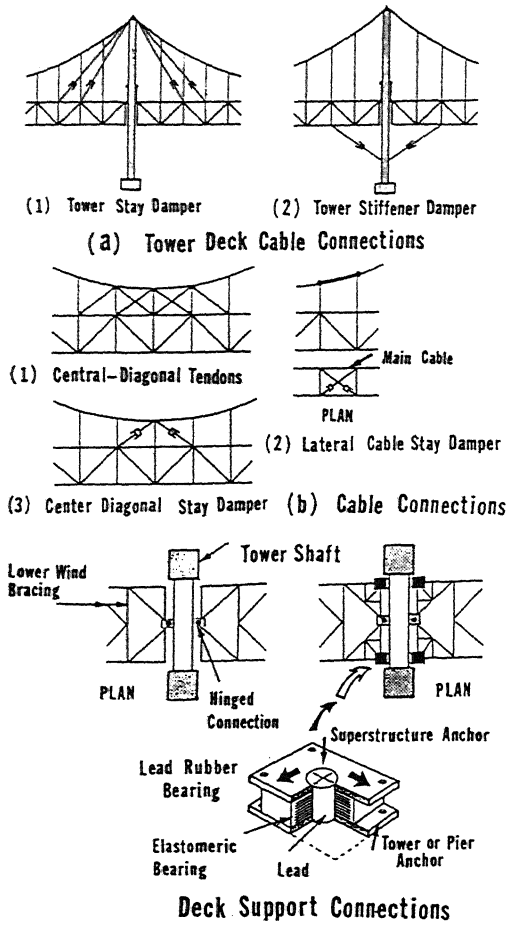

Figure 12 Damping augmentation measures to reduce the seismically induced forces and deformations.

bridge foundations were evident; such variations have significant effects on the response.

2. The addition and subtraction of the recorded motions and their cross-spectral analyses significantly enhanced the identified modes of vibration.

3. Strong coupling in the three orthogonal directions occurs in several modes of vibration; 100 modes are used in the analysis.

4. Computed dynamic characteristics compare very favorably with those identified from the earthquake records.

5. The level of nonlinearity experienced by the bridge during the earthquake is negligible.

6. There is an urgent need to *upgrade* the current state of the bridge instrumentation.

7. The bridge damping characteristics are very complex; damping values depends on the mode type, the modal order, the structural member and the vibration amplitude.

8. There is no evidence of any slackness of the bridge cables and hangers.

9. The bridge may be strengthened to resist stronger future earthquakes. Figure 12 shows the additional braces, viscous dampers and the energy dissipation devices of the lead-rubber bearings which can be placed at the hinged connections between the tower and the wind bracing in order to provide better transmission and absorption of the induced longitudinal or lateral forces.

ACKNOWLEDGMENTS

The research is supported by grants from the National Science Foundation and the California Department of Transportation; this support is greatly appreciated. The support and data processing provided by the California Division of Mines and Geology are also appreciated.

REFERENCES

Abdel-Ghaffar, A. M. and Housner, G. W. 1978. Ambient vibration tests of suspension bridge. Journal of the Eng. Mech. Div., ASCE, Vol. 104, (5): 983-999.

Niazy, A-S. M. 1991. Seismic Performance Evaluation of Suspension Bridges. Ph.D. Thesis, Department of Civil Engineering, University of Southern California, Los Angeles, CA.

Abdel-Ghaffar, A.M. 1976. Dynamic Analyses of Suspension Bridge Structures. Report No. EERL 76-01, Civil Eng. Dept., Calif. Inst. of Tech., Pasadena, CA.

Rubin, L.I., Abdel-Ghaffar, A.M. and Scanlan, R.H. 1983. Earthquake Response of Long-Span Suspension Bridges. Report No. 83-SM-13, Dept. of Civil Eng., Princeton Univ., NJ.