SEISMIC PERFORMANCE INVESTIGATION OF THE HAYWARD-BART ELEVATED SECTION INSTRUMENTED UNDER CSMIP

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ABSTRACT

This paper presents the results of a seismic performance investigation of the Hayward-BART elevated section, instrumented by the California Division of Mines and Geology under its Strong Motion Instrumentation Program (CSMIP), using the acceleration time-histories recorded during the October 17, 1989 Loma Prieta earthquake. The recorded structural responses are correlated with corresponding theoretically predicted responses. Adjustments of structural parameters and modelling concepts required to achieve satisfactory correlations are discussed, along with their implications to procedures of standard engineering practice. Recommendations are made toward improving the arrangement of CSMIP strong-motion instruments at the Hayward-BART site.

INTRODUCTION

The design of the present Bay Area Rapid Transit (BART) system started in 1963 and continued over a number of years. The state-of-the-art in the analysis and design of earthquake-resistant transportation structures has improved significantly since that time. Observing the performances of such structures during past earthquakes has been a major factor in bringing about this improvement. Most notably is the San Fernando earthquake of February 9, 1971, during which many elevated freeway structures collapsed. Following this event, major changes were made to the earthquake code provisions leading to improved structures form a seismic performance point of view (Ref. 1). As evidence of this fact, no freeway structures of post-San Fernando design suffered damages during the Loma Prieta earthquake while many of such structures of pre-San Fernando design were heavily damaged and/or collapsed.

While the BART aerial structures were undamaged during the Loma Prieta earthquake, that fact alone does not insure satisfactory performance under future moderate to maximum credible earthquake conditions. Considering that the CSMIP-instrumented section of the Hayward-BART aerial structure experienced deck-level peak horizontal accelerations as high as 0.60g during the Loma Prieta earthquake, even though the peak free-field horizontal ground acceleration at the site was only about 0.16g, its performance under free-field ground motions of two to four times this intensity of shaking is of considerable concern. Fortunately, the CSMIP recordings of structural response at this site have made it possible to develop realistic modelling of this structure, allowing not only an assessment of its performance during the Loma Prieta earthquake but an assessment of its expected performance during an earthquake of much higher intensity, say 0.70g peak ground acceleration (PGA).

DESCRIPTION OF STRUCTURE INVESTIGATED

The structure investigated under this research program is a three-span, nearly-straight section of the BART elevated system located immediately to the north of the BART Hayward Station. The structure consists of 3 simply-supported twin box-girders constructed of prestressed concrete, which are supported on four single-column piers designated as P132, P133, P134, and P135; see Figure 1.

The reinforced concrete single-column piers have a hexagon cross-section with a 5-foot dimension between opposite faces and they are reinforced with two rings of #18 Grade 60 reinforcing bars. Each column of piers P132, P133, and P135 has 28-#18 bars in its outer-ring which run the full height and

16-#18 bars in its inner ring which are cut at various heights; thus, for each column, a total of 44-#18 bars are present at its base. The column of pier P134 has 24 full-length #18 bars in its outer-ring and 12-#18 bars cut at various heights in its inner ring; thus, it has a total of 36-#18 bars present at its base. All columns are provided with #5 spiral bars running at 3-inch pitch covering almost the full height of column.

Each pier-column of P132, P133, and P135 is supported on an 18' x 18' square footing 5.5' deep, which is, in turn, supported by 18 one-foot diameter reinforced concrete piles, each having a capacity rating of 60 tons. Pier P134 is supported on a 16' x 16' square footing, 5.5' deep, and on 16 piles of the same capacity rating. Except for the vertical piles located along the horizontal axes of symmetry of the footing, all others are battered with a slope of 8:1 for the inner piles and 4:1 for the outer piles. All piles were driven into the soils to depths of 40 to 50 feet below the bottoms of the pier footings. The soil condition at the site, as indicated by the soil boring-logs for bore holes located near the site, consists of layers of sandy clay and silty sand. The water table at the site is located about 60 feet below ground surface.

Each prestressed-concrete box-girder is hinged at its north end to its corresponding pier-beam support through two vertical concrete-filled 5-in. diameter steel pipes and it rests on a bearing support at its south end allowing freedom of movement longitudinally relative to the support. Freedom of relative movement transversely is prevented however since the south end of each girder is hinged to the north end of the adjacent girder. All hinges of the girders have a 1-inch gap, tight fitted with a non-laminated elastomeric material and the girders themselves are supported on the tops of pier beams through two 15" x 12" x 1" elastomeric pads at each end of each girder. Thus, even for small relative displacements (<< 1 inch), the stiffnesses of the elastomeric pipe-hinge fillers and bearing pads play a role in controlling the girder vibration frequencies.

The BART train rails are fastened rigidly to the prestressed-concrete girders at 3-ft intervals longitudinally. Thus, even though the girders are simply supported between adjacent piers, stiffness coupling across the girder joints between spans exists due to the stiffnesses of the continuous rails which are rigidly fastened to the girders. Such coupling is very significant in the longitudinal direction due to the high axial stiffness of the rails but is not too significant in the transverse direction. As will be discussed later, the high stiffness coupling in the longitudinal direction did indeed play a major role in the seismic response behavior as recorded by the CSMIP instruments during the Loma Prieta earthquake.

DESCRIPTION OF INSTRUMENTATION AND DATA COLLECTED DURING LOMA PRIETA EARTHQUAKE

The CSMIP instrumentation of the structure under investigation consists of 18 strong-motion acceleration sensors installed both on the structure and in the free-field. These sensors will be designated herein as Channel Nos. 1 through 8 and 10 through 19 (Channel No. 9 was not installed). The locations and directions of sensors are shown in Fig. 1.

During the October 17, 1989 Loma Prieta Earthquake, accelerograms were recorded by all 18 sensors. These accelerograms are shown as time-history plots in Fig. 2. The free-field recordings show that during the earthquake, the site region experienced ground-surface peak-accelerations of 0.16g horizontally and 0.08g vertically. The peak accelerations experienced at the girder deck level ranged from 0.39g to 0.60g in the transverse direction and from 0.21g to 0.26g in the longitudinal direction.

ANALYSIS OF RECORDED DATA AND OBSERVATIONS

The acceleration time-history data collected from the Loma Prieta earthquake shown in Fig. 1 have been analyzed extensively in this research program in an attempt to understand the seismic response behavior of this structure during the earthquake. In general, the data analyses performed consisted of: (1) computing and plotting acceleration response spectra (ARS) for a 2% damping ratio for the recorded acceleration time-history data; (2) computing and plotting Fourier spectra of acceleration time histories and the transfer functions (complex Fourier spectrum ratios) between the structural response motions and the free-field motions; (3) doubly integrating the acceleration time-histories to give displacement time histories from which selected relative displacement time-histories of interest were obtained; and (4) generating cross-correlation functions between pairs of selected recorded motions from which the apparent phase lags between these pairs of motions were determined. From the results of the data analyses described above, significant features of the seismic response of the structure during the earthquake were observed and deduced. These are summarized below.

Free-Field Motions - The 2%-damped ARS computed from the free-field recorded motions indicate that even though the recorded PGA values are the same for the NS and EW directions, the EW motion, which is in the transverse direction of the structure, contains significant components of motion in a narrow frequency range near 1 Hz; whereas these same components of motion are nearly absent in the NS motion. The significant content of motion near 1 Hz for the EW motion has a significant effect on the transverse response of the structure.

Longitudinal Structural Response at the Deck Level - The longitudinal structural response motions at the deck level recorded at sensor locations, 3, 4, 5, 6, 7, and 8 shown in Fig. 1 indicate that the longitudinal responses at all these sensor locations along the 3-span length are almost identical, indicating that, even though joints are present, the girders are strongly coupled longitudinally by the rails; thus, they behave essentially as a unit in this direction with almost no relative motions taking place across the joints. The relative displacement time-history obtained from recorded data of Sensors 4 and 5 indicate that the maximum relative displacement experienced at this joint was about 2 mm (0.08 inch) which is less than 10% of the joint gap of 1 inch.

Transverse Structural Response at the Deck Level - The transverse structural response motions at the deck level recorded at sensor locations 10, 11, and 12 shown in Fig. 1 indicate that, transversely, the girder and the pier-beam basically responded as a unit with very little relative motion across the elastomeric bearing pads. The maximum relative displacement between the girder and the pier beam obtained from the recorded data is 5.5 mm (0.216 inch). Using this amount of relative displacement and the transverse inertia force of the girder tributary to Pier 132, the apparent shear modulus of the elastomeric bearing pads, calculated taking into account the sitffnesses of elastomeric fillers around the hinges, is in the range of 500 to 600 psi which is about 4 to 5 times higher than the 120 to 155 psi given in the AASHTO code.

Structural Response Behavior at P132 - Pier 132 has been instrumented with the largest number of sensors as indicated in Fig. 1, namely, Sensors 2, 3, 4, and 6 measuring the longitudinal response motions and Sensors 11, 12, and 13 measuring the transverse response motions. Examining the 2%-damped ARS and the transfer function amplitudes obtained from analyses of recorded data shown in Fig. 3, one can observe that, longitudinally, the structural system at P132 has a major structural response peak at the frequency of 3.5 Hz and a minor peak at about 2.1 Hz; transversely, it has a major structural response peak at the frequency of 1.8 Hz and a minor peak at 3.6 Hz. Using the half-power bandwidth method, the modal damping values of the system associated with the major response modes at 3.5 Hz for the longitudinal response and 1.8 Hz for the transverse response are estimated to be 4% and 3.6%, respectively.

Structural Responses at the Bases of P132 and P135 - Two sets of triaxial sensors were installed at the bases of piers P132 and P135 which are separated by a distance of 231 feet (70.4 m). The recorded motions and their integrated displacement time-histories at these two locations indicate that these response motions are nearly identical with the response at P132 lagging behind P135 by a small amount, indicating that the seismic waves propagated in the general direction from South to North which is consistent with the relative location of the epicenter to the site. The time lags determined from the cross-correlation functions computed form the recorded motions are estimated to be 0.03 second for the longitudinal motions and 0.07 second for the transverse motions, giving the apparent wave propagation velocities of these motions at about 2.4 km/sec and 1.0 km/sec, respectively.

DEVELOPMENT OF ANALYTICAL MODELS

Based on the dynamic response behaviors of the structure observed from the results of data analyses described previously, analytical models intended for capturing the gross dynamic response behaviors observed were developed. Since as described previously, the longitudinal and the transverse structural responses observed show essentially decoupled behaviors, separate longitudinal and transverse models could be used for the structure in capturing its overall behavior. Furthermore, since the structures of all three spans are essentially the same and their observed responses are quite similar, it is only necessary in developing analytical models to consider the structure and foundation system of a typical span. Since Pier 132 has been most extensively instrumented, a representative one-span structure tributary to it was used for developing the analytical models. Because the recorded data have indicated significant soil-structure interaction effects, the dynamic impedance characteristics of the pier foundation system were included in developing the analytical models.

Transverse Model - For response prediction in the transverse (EW) direction of the structure, a lumped-mass generalized-beam-stick model was used to represent the one-span structure tributary to pier P132 as indicated in Fig. 4. As shown in this figure, the model consists of: 2 lumped masses representing the twin box girders, which respond essentially as rigid bodies due to their very high fundamental horizontal frequency (10 Hz) relative to the critical system frequency (1.8 Hz); 3 lumped masses representing the pier-beam and column; and one lumped mass representing the pier footing (pile cap). For each lumped mass, its tributary rotary inertia is also included. The girder lumped masses are connected to the lumped mass representing the rigid pier beam through two shear springs (K_p) representing the apparent shear stiffnesses of the elastometric bearing pads. The lumped masses of the column are interconnected by elastic beam elements which have stiffness properties based on the gross uncracked concrete section of the column. The modal damping ratios for the fixed-base structure are assumed to be 2.5% for all modes.

The dynamic characteristics of the soil-pile foundation system are represented by a set of frequency-independent foundation impedances (i.e., soil springs and dampers). A set of translational soil spring and damper (K_{xx} and C_{xx}) and a set of rocking soil spring and damper ($K_{\theta\theta}$ and $C_{\theta\theta}$) are attached to the pier footing a distance H above its center of mass as shown in Fig. 4. This distance H is intended to simulate the effect of foundation embedment which results in increases in the foundation impedance values and creates a coupling impedance ($K_{x\theta}$ and $C_{x\theta}$) between the foundation translation and rocking rotation. The numerical values of the translation and rocking spring stiffnesses (K_{xx} and $K_{\theta\theta}$) were estimated using the results of a pile group test conducted recently by Caltrans (Ref. 3) and the axial stiffnesses of the battered piles. The stiffnesses as obtained were further adjusted considering the soil shear modulus degradation effect due to the free-field soil shear strains induced during the earthquake. The values of the translation and rocking damper coefficients (C_{xx} and $C_{\theta\theta}$) were derived by assuming a critical damping ratio of 20% for both the rigid body translation and rocking modes of the rigid structure on the flexible foundation. Distance H was left as a parameter to be adjusted in optimizing the correlation between the predicted and measured responses.

Longitudinal Model - For predicting the longitudinal (NS) response of the structure, the analytical model selected to represent a typical span of structure tributary to pier P132 is essentially the same as that of the transverse model described above; however, recognizing that the structure in the longitudinal direction is highly coupled to the stiffer and much more massive structure of the Hayward BART Station through the high axial stiffnesses of the girders and the rigidly-fastened rails across the girder joints, the longitudinal model for a representative span is coupled longitudinally through two axial links to a stiffer and more massive model representing the gross dynamic characteristics of the structures of the Hayward BART

Station immediately to the south. Since the recorded data indicate that the longitudinal structure responses throughout the 3-span structure have a dominant response frequency at about 3.5 Hz and a minor response frequency at about 2 Hz, it is postulated that the frequency at 3.5 Hz is dominated by the stiffer Hayward BART Station structure. Thus the model properties of the stiffer model representing the Hayward BART Station were adjusted to reflect a fundamental frequency in the longitudinal direction of about 3.5 Hz.

CORRELATION OF ANALYTICAL AND MEASURED RESPONSES

Based on the longitudinal and transverse analytical models developed as described previously, dynamic responses of the models subjected to the inputs of the free-field acceleration time-histories in the NS and EW directions as recorded by Sensors 17 and 19, respectively, were computed. Since model parameters, such as soil and elastomeric material properties are uncertain and since the recorded data are not sufficient to deduce the needed information, numerous parametric variations were considered in the analysis. Included in these parameter variations were the stiffnesses of the elastomeric bearing pads, the foundation soil modulus and damping values, and the distance H used in characterizing the foundation embedment effect. The final values of these parameters were selected as those which resulted in the best correlations between the analytical predicted responses and the corresponding measured responses. The responses obtained from analyses using the best-estimate parameter values are compared with the corresponding measured responses in the form of 2%-damped acceleration response spectra calculated from the acceleration response time histories. These comparisons and discussions of the results are summarized below.

Longitudinal Responses - The 2%-damped acceleration response spectra for the analytically computed longitudinal response motions at sensor locations 6, (girder), 3 (pier-beam), and 2 (pier-base) are compared with the corresponding spectra for the measured response motions in Fig. 5. As shown by these comparisons, the analytical results capture the gross response behavior in the longitudinal direction reasonably well; however, as indicated from the spectra shown, the longitudinal response are dominated by the structural amplification peak at the frequency of 3.6 Hz which is attributable to the major structural system frequency of the stiffer Hayward BART Station structure. A future confirmation of this response characteristic is desirable.

Transverse Responses - The 2%-damped acceleration response spectra for the analytically predicted transverse response motions at sensor locations 11 (girder), 12 (pier-beam), and 13 (pier-base) are compared with the corresponding results obtained from the measured response motions also in Fig. 5. As indicated by these comparisons, the transverse analytical model captured the fundamental mode response at the frequency of 1.8 Hz very well; however, it is somewhat deficient in predicting the second mode response at the frequency of 3.6 Hz, which is basically due to the foundation rocking. Because of the lack of recorded data that could be used in separating the rocking component and translation component of the pier base motions, further refinements of the foundation model, which significantly controls the transverse structural response behavior, could not be achieved rationally.

ASSESSMENT OF STRUCTURAL PERFORMANCE AND DESIGN IMPLICATIONS

The earthquake response data recorded at the three-span section of the BART elevated structure offer a unique opportunity to assess the seismic response behavior of this structure during the Loma Prieta earthquake. From the results of analyses presented previously, valuable insights into the seismic performance of this section of the BART elevated structure have been obtained and their implications on design have been assessed as follows:

(1) The apparent structural damping value of the BART structure as indicated from the recorded data and as found to give reasonable correlations, is about 2.5% for the fixed-base structure and about 4% for the structure-foundation system, both of which are lower than the value of 5% normally used in design. This lower apparent damping value leads to a structural response amplification factor at the deck level of about 4 which is higher than peak elastic spectral amplification factor of 3 normally used for design. However, considering that the peak horizontal acceleration of the free-field motions during the earthquake was only 0.16g, the damping value of 5% and the peak elastic spectral amplification factor of 3 at the design level of 0.35g to 0.70g can be judged to be reasonable and conservative for design purposes.

(2) As indicated by the recorded data, as well as by the parametric correlation studies, the soil-structure interaction effect on seismic response of the structure is significant. This effect tends to lower the structure system frequencies appreciably. For example, the analytical model developed for transverse response prediction shows the fundamental fixed-based structure frequency to be 2.5 Hz which is considerably above the system frequency of 1.8 Hz obtained when soil-structure interaction is considered. In the design of the BART structure, a fixed-base structural model is normally used which tends to over-estimate the frequencies and under-estimate the response.

(3) The recorded data indicate that the BART elevated structures are highly coupled in the longitudinal direction due to the presence of the continuous rails which are rigidly fastened to the girders, even though the structure of each span is designed to be simply-supported and free to move at one end. This implies that the single-pier model used for design in this direction may not be appropriate, especially for those elevated sections which have large variations in the pier column heights. When the system is strongly coupled longitudinally the shorter columns tend to experience higher seismically induced internal forces; whereas, the single-pier model without this coupling may not predict such a result. Thus in such situations, a model consisting of structures of several spans and piers may be necessary. Furthermore, due to the apparent strong coupling of the rails, the axial forces induced in the rails across the girder joints should be assessed in such situations.

(4) As discussed previously, the apparent shear stiffnesses of the elastomeric bearing pads for the BART girders have been found to be higher than the code values, indicating potential degradation of the material due to aging or other environmental effects. It would be very useful, if and when these pads are replaced, to perform tests of the existing pads to determine their properties. Furthermore, since their properties in the current condition are uncertain, design or assessment of the structure should consider a wide variation of these properties.

(5) Since the soil-structure interaction effect is shown to be important, design procedures for estimating the pile foundation impedances and capacities such as those published in Ref. 4 should be evaluated using actual earthquake response data. However, to make this possible, more instruments should be placed on the foundation base such that they can produce sufficient data for evaluating separate modes of foundation response. The current CSMIP instrumentations are not sufficient for such an evaluation.

CONCLUSIONS AND RECOMMENDATIONS

Based on the analysis and structural performance assessment results obtained in this study, the following conclusions and recommendations can be made:

(1) The data recorded during the Loma Prieta earthquake by the CSMIP instruments on the Hayward-BART elevated structure provide valuable information for understanding the seismic response of this structure.

(2) Due to the high axial stiffness of the continuous rails, the seismic response behavior of this structure in the longitudinal direction was found to be quite different from that in the transverse direction. The former behavior is controlled by the response of the entire coupled system; whereas, the later is more or less controlled locally from span to span. The responses in both directions are significantly influenced by soil-structure interaction effects. In the more-critical transverse direction, these effects actually result in higher responses than those obtained using the fixed-base design analysis procedure by a factor of 1.3 to 1.5.

(3) During the Loma Prieta earthquake, the maximum seismically-induced column base moment was approximately 1/3 of the column's ultimate moment capacity. However, using response spectrum compatible accelerograms normalized to the Maximum Credible Earthquake PGA level of 0.7g, as currently specified in the BART Extension Program, the maximum induced seismic base moment predicted by the models calibrated in this study was found to exceed the design moment capacity by a factor of about 3.4.

(4) The studies conducted in this research program point out an urgent need of instrumentation that allows independent recordings of the rocking rotation responses at the bases of pier-columns. A need also exists to obtain longitudinal response data at locations closer to the Hayward BART Station. Thus, the current instrumentation layout on this section of structure can be improved by shifting some of the redundant sensors for recording the longitudinal motions of girders to pier bases for measuring base rocking motions and to locations closer to the BART Hayward Station for measuring the longitudinal motions.

(5) The findings of this study suggest the need for an assessment of current design procedures, including modelling for seismic response predictions and criteria for setting limits on ductility demands.

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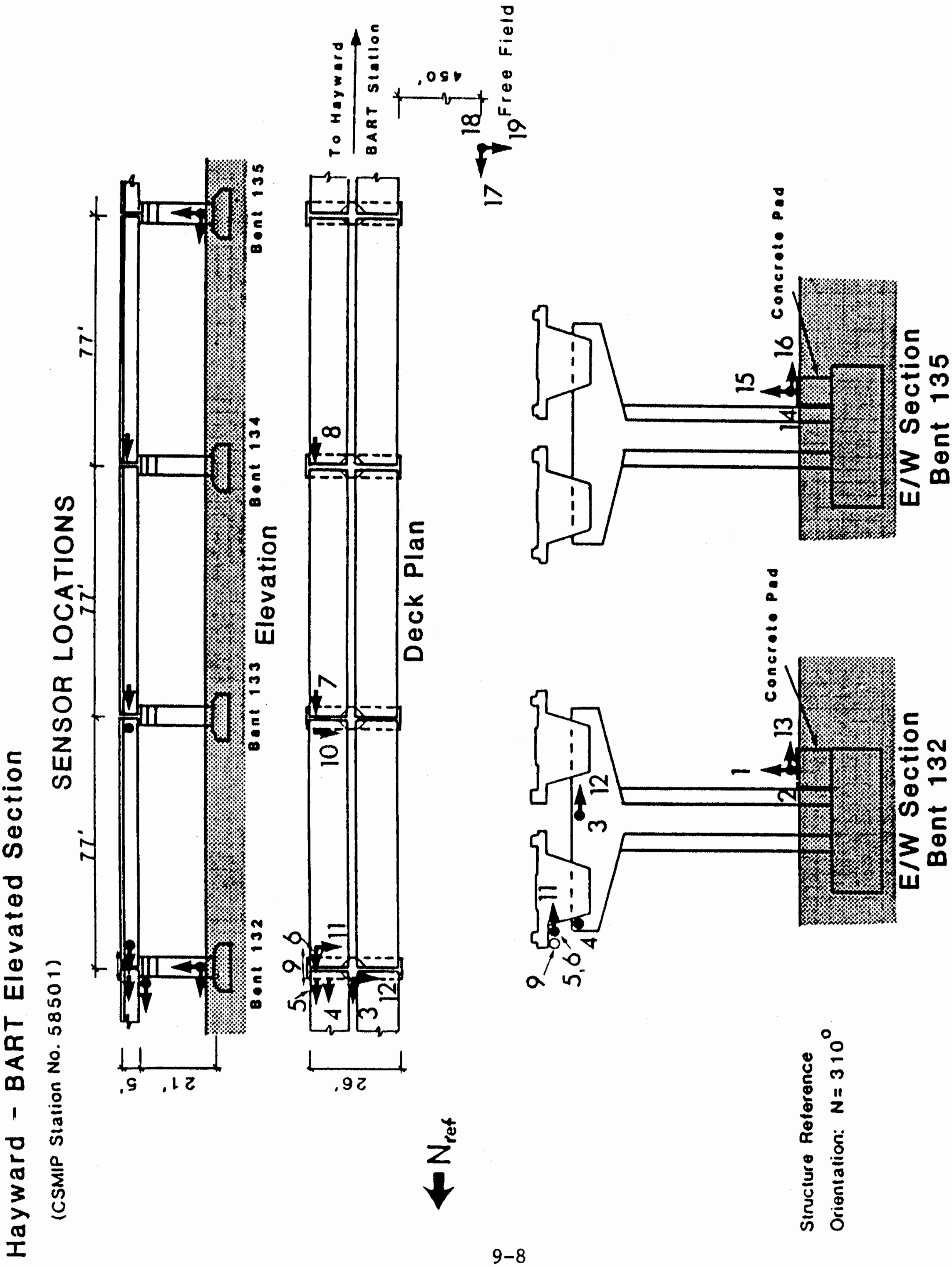

Figure 1 Structure Configuration of Hayward-BART Elevated Section and Sensor Locations

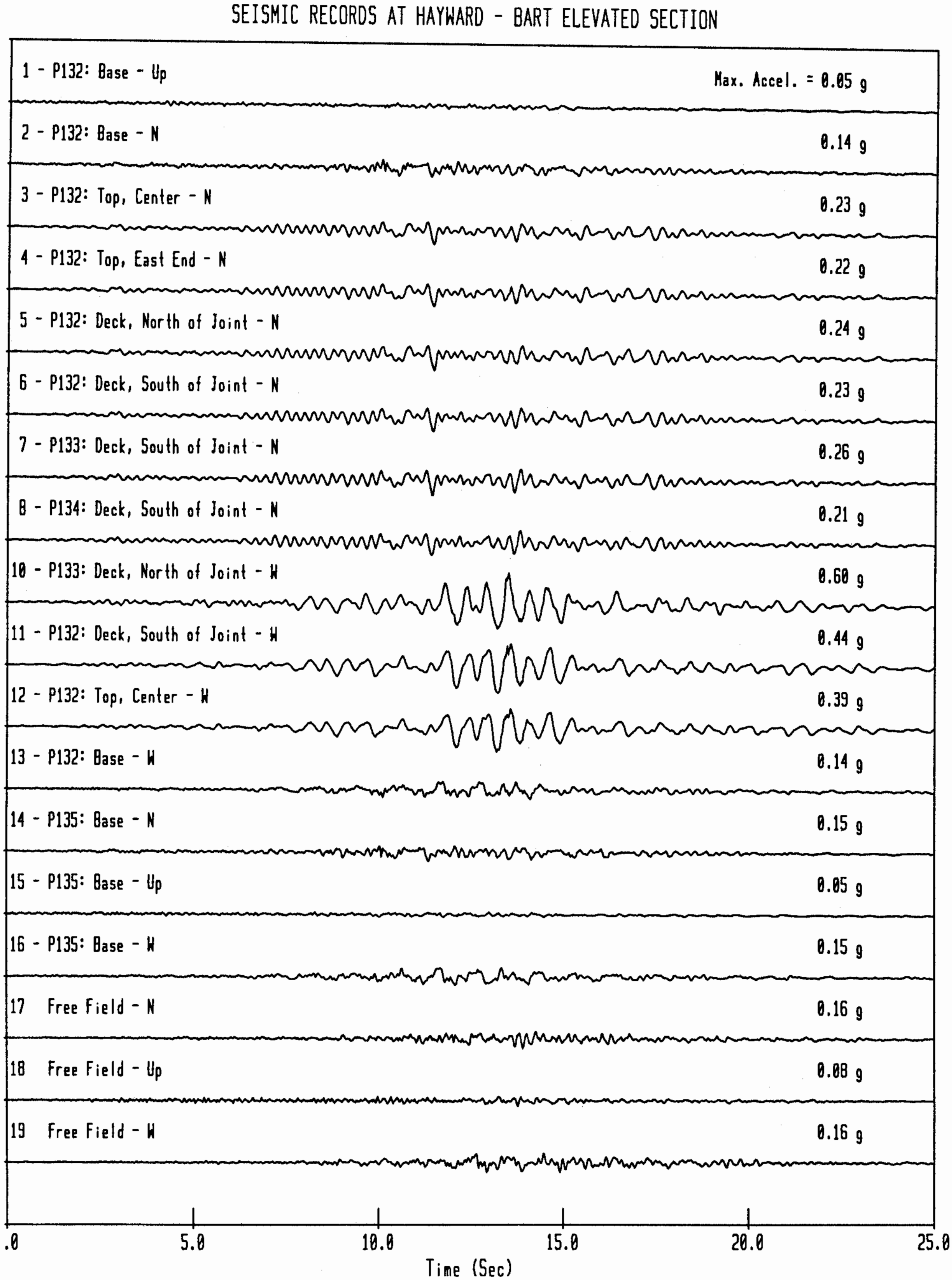

Figure 2 Accelerograms Recorded During the Loma Prieta Earthquake of 1989

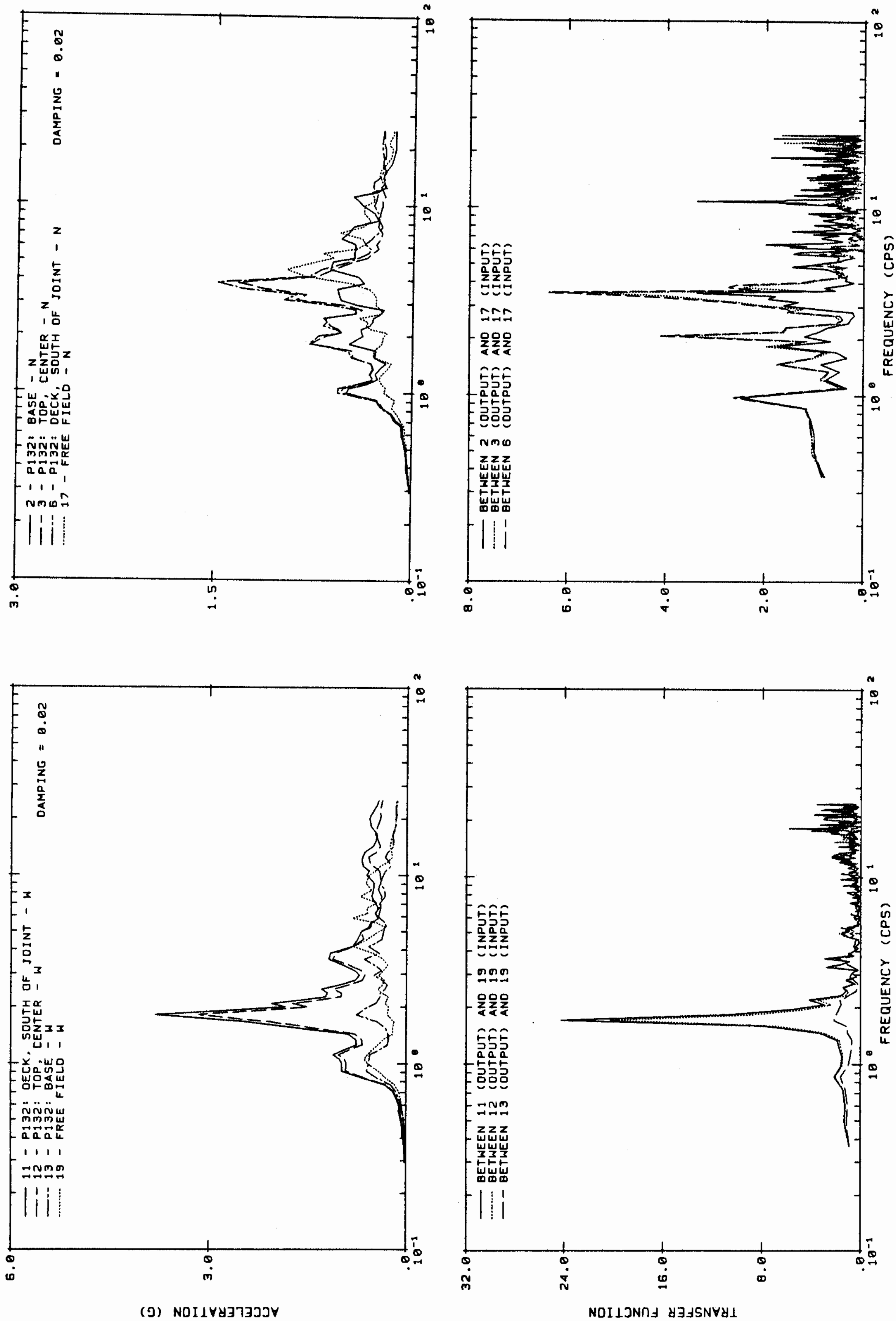

Figure 3 2%-Damped Acceleration Response Spectra and Transfer Function Amplitudes of Recorded Motions

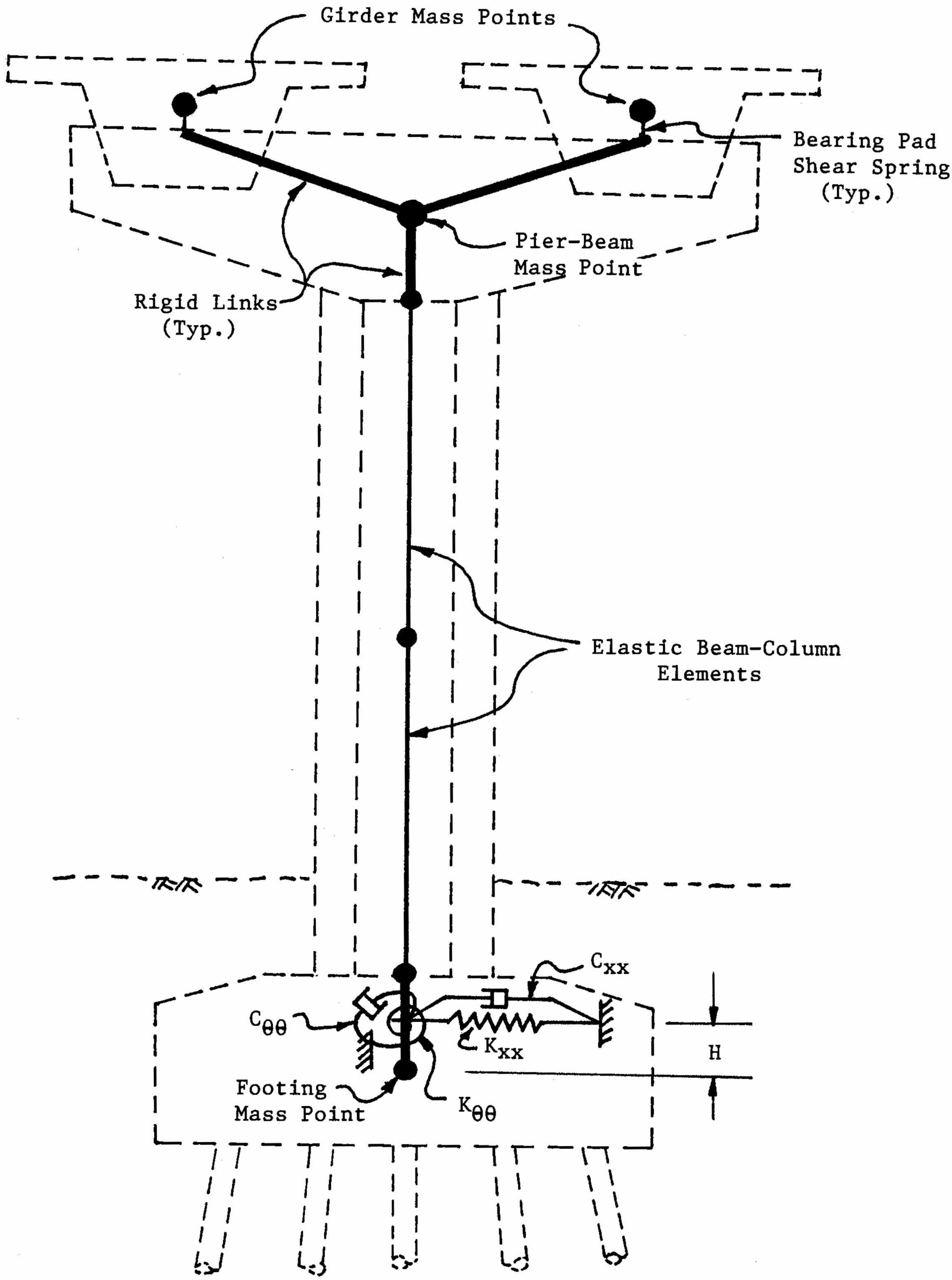

Figure 4 Analytical Model for the Transverse Response Analysis

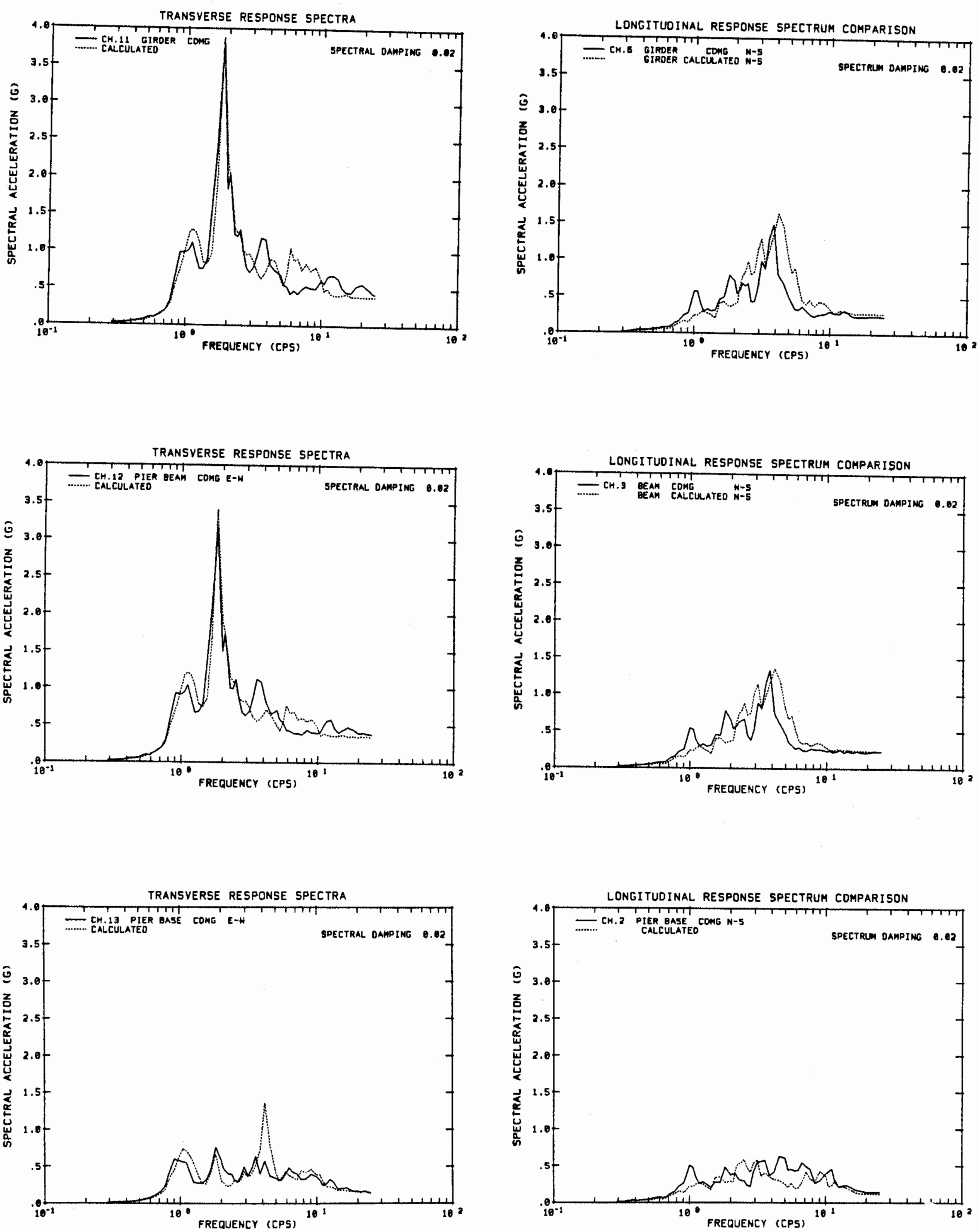

Figure 5 Comparisons of Analytically-Predicted and Measured Responses