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Seismic performance investigation of a section of Bay Area Rapid Transit elevated structure during the 1989 Loma Prieta earthquake

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ABSTRACT: This paper presents the results of a seismic performance investigation of a section of the BART elevated structure, instrumented by the California Division of Mines and Geology (CDMG) 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.

1 INTRODUCTION

The design of the present San Francisco Bay Area Rapid Transit (BART) system in Northern California 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 from a seismic performance point of view (Penzien and Tseng 1973). As evidence of this fact, no freeway structures in California 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.

The BART aerial structures were undamaged during the Loma Prieta earthquake. However, considering the CSMIP-instrumented section of the 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 0.16g, its performance under the design-level 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 an assessment of its performance during the Loma Prieta earthquake and an assessment of its expected performance during a design maximum credible event of much higher intensity, namely, 0.70g peak ground acceleration (PGA).

2 DESCRIPTION OF STRUCTURE INVESTIGATED

The structure investigated is a three-span nearly-straight section of the BART elevated structure located immediately to the north of the Hayward BART Station. The structure consists of three 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 a total of 36 to 44 #18 Grade 40 reinforcing bars. All columns are provided with #5 spiral bars running at 3-inch pitch covering almost the full height of the column. Each pier-column is supported on a 15′ × 15′ or 16′ × 16′ square footing 5.5′ deep, which is, in turn, supported by 16 to 18 one-foot-diameter reinforced concrete piles, each having a 60-ton capacity. 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 consists of layers of sandy clay and silty sand. The water table at the site is located about 60 feet below ground surface.

The prestressed-concrete box-girder of each span is hinged at one end to its pier-beam support, and rests on elastomeric bearing supports at its other end allowing free longitudinal girder movement relative to the support. Relative transverse girder movement is, however, prevented through girder hinges. All hinges of the girders have a 1-inch gap, tightfitted with an elastomeric material. Thus, for small relative displacements ($<<1$ inch), the stiffnesses of the elastomeric

hinge fillers and bearing pads play a role in controlling the girder vibration frequencies.

The BART train rails are continuously welded and fastened to the prestressed-concrete girders through rail-fastener assemblies which are anchored to the girder at 3-ft. intervals longitudinally. Thus, stiffness coupling across the girder joints between spans exists, which 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. The high longitudinal stiffness coupling plays a major role in the longitudinal seismic response behavior of the structure.

3 INSTRUMENTATION AND RECORDED DATA

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 are designated 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 Figure 1. During the 1989 Loma Prieta Earthquake, accelerograms were recorded by all 18 sensors. 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 (Shakal, et al. 1989).

4 ANALYSIS OF RECORDED DATA AND OBSERVATIONS

The acceleration time-history data collected from the Loma Prieta Earthquake have been analyzed extensively in an attempt to understand the seismic response behavior of this structure during the earthquake. Significant features of the seismic response of the structure during the earthquake observed from the results of the data analyses are summarized as follows:

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

Transverse Response at the Deck Level - The transverse response motions at the deck level recorded at Sensors 10, 11, and 12 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 tributary transverse inertia force of the girder, the apparent shear modulus of the elastomeric bearing pads is estimated to be in the range of 500 to 600 psi which is about 4 to 5 times higher than the 120 to 155 psi given the AASHTO code.

Structural Response of P132 - Pier 132 has been instrumented with the largest number of sensors as indicated in Figure 1. Examining the 2%-damped ARS and the transfer function amplitudes obtained from analyses of recorded data, 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. Comparing the ARS computed from recorded motions at the foundation and in the free-field, significant soil-structure interaction effects are observed.

Responses at the Bases of P132 and P135 - The recorded motions and their integrated displacement time-histories at the bases of piers P132 and P135 which are separated by a distance of 70.4 m (231 feet) indicate that these response motions are nearly identical except differing by a small phase lag in the direction consistent with the direction of seismic wave propagations from the epicenter to the site. The time lags estimated from the recorded motions are 0.03 second and 0.07 second, respectively, for the longitudinal and transverse motions, giving the apparent seismic wave propagation velocities at the site of about 2.4 km/sec and 1.0 km/sec, respectively.

5 ANALYTICAL MODELS

Based on the dynamic response behaviors of the structure observed from the results of data analyses, analytical models intended for capturing the gross dynamic response behaviors observed were developed. Since the longitudinal and the transverse structural responses are observed to be essentially decoupled, separate longitudinal and transverse models were developed for the structure. 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. 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 Figure 2. As shown in this figure, the model consists of: 2 lumped masses representing the twin box girders; 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 pier-beam lumped mass through two shear springs (K_p) representing the apparent shear stiffnesses of the elastomeric bearing pads. The stiffness properties of the column are 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 translational soil spring and damper (K_{xx} and C_{xx}) and a set of frequency-independent rocking soil spring and damper ($K_{\theta\theta}$ and $C_{\theta\theta}$) attached to the pier footing at a distance H above its center of mass. 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 both the results of a pile group test conducted by Caltrans (Abcarius 1991) and the axial stiffnesses of the 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% and 15%, respectively, for the translation and rocking modes of response of the rigid structure on the flexible foundation. The 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 developed 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 immediately to the south 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 to a stiffer model representing the structures of the Hayward BART Station immediately to the south. 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, as observed from the recorded data.

6 CORRELATION OF ANALYTICAL AND MEASURED RESPONSES

Based on the 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 achieved the best correlations between the analytically predicted and the corresponding measured responses. The predicted responses obtained using the best-estimate parameter values are compared with the corresponding measured responses in the form of 2%-damped ARS as shown in Figure 3.

Transverse Responses - The comparisons shown in Figure 3 indicate that 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.

Longitudinal Responses - As shown by the comparisons in Figure 3, the analytical results capture reasonably well the dominant structural response peak at the frequency of 3.6 Hz which is attributable to the major structural system frequency of the adjacent stiffer Hayward BART Station structure.

7 ASSESSMENT OF 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. As indicated by the recorded data, as well as by the parametric correlation studies, the soil-structure interaction effect on the seismic response of the structure is significant. This effect tends to lower the structure system frequencies appreciably. For exam-

ple, 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 was found to over-estimate the frequencies and under-estimate the response by a factor of about 1.3 to 1.5.

2. During the Loma Prieta earthquake, the maximum seismically-induced column base moment was approximately 1/2 of the column's ultimate moment capacity. However, using response spectrum compatible accelerograms normalized to the design Maximum Credible Earthquake PGA level of 0.7g, 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 4. Such a factor of exceedance should be evaluated on the basis of the ductility capacity of the columns and the service performance goal of the system.

3. The recorded data indicated that the BART elevated structure with simply-supported girder spans are highly coupled longitudinally due to the presence of continuous rails which are rigidly fastened to the girders. This implies that the single-pier model normally used for design may not be appropriate for this direction, especially for those elevated sections with large variations in the pier column heights. Furthermore, due to the apparent strong coupling of rails, the axial forces induced in the rails across the girder joints may be large and should be assessed in such situations.

4. 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. However, considering that the PGA of the free-field motions during the earthquake was only 0.16g, the damping value of 5% at the design level of 0.70g can be judged to be reasonable and conservative for design purposes.

5. Since the soil-structure interaction effect is shown to be important, current design procedures for estimating the pile foundation impedances and capacities (e.g., Lam and Martin 1986) 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 translation and rocking modes of foundation response. The CSMIP instrumentation on this site was not sufficient for this purpose.

8 CONCLUDING REMARKS

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. These data allowed not only a realistic assessment of the seismic performance of the structure during the Loma Prieta earthquake, but also an assessment of its expected performance during a design-level earthquake of much higher ground shaking intensity. The findings of this study point out the need of modelling soil-structure interaction for seismic response prediction for this type of structure and suggest the need for an assessment of current design procedures and criteria for setting limits on ductility demands.

9 ACKNOWLEDGEMENT

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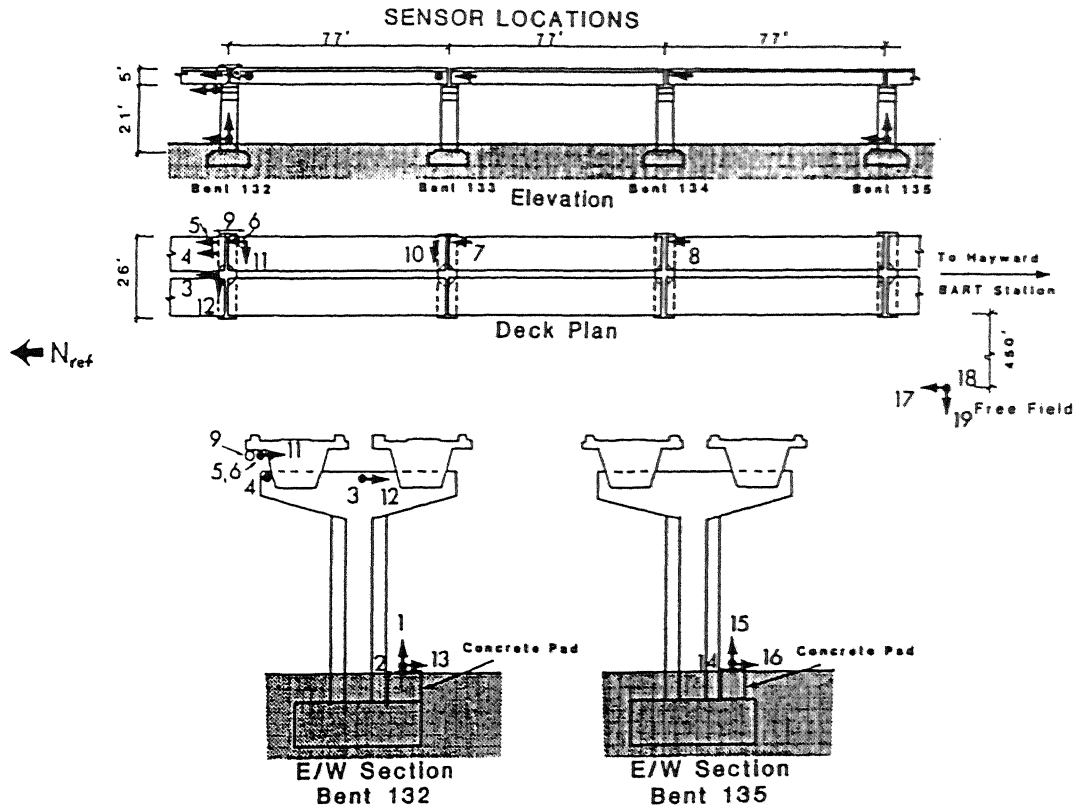

Figure 1 Structure Configuration of Hayward-BART Elevated Section and Sensor Locations

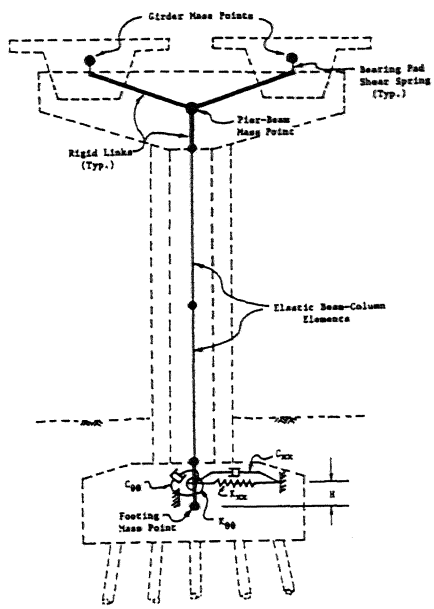

Figure 2 Analytical Model for the Transverse Response Analysis

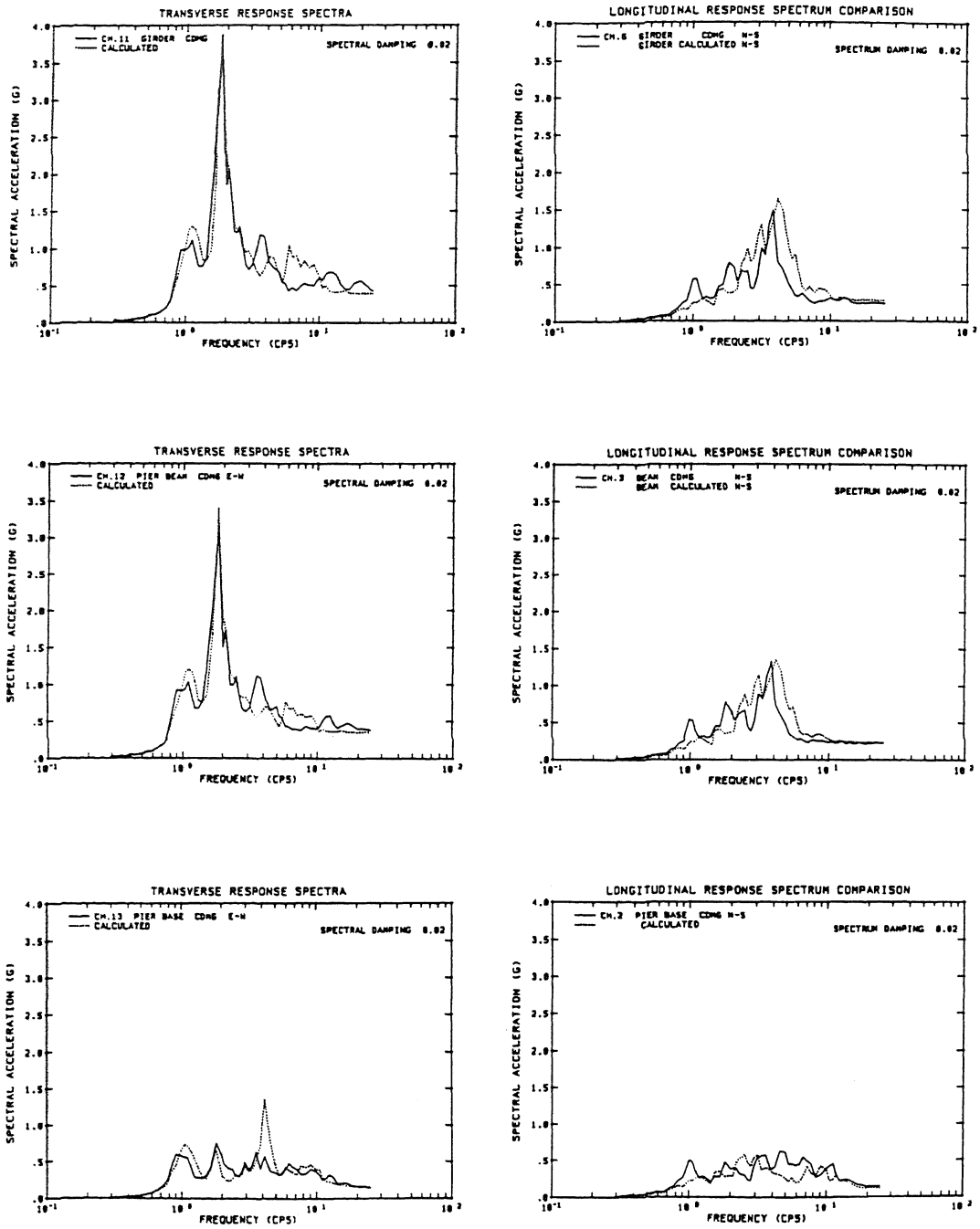

Figure 3 Comparisons of Analytically-Predicted and Measured Responses