

RESPONSE OF THE NORTHWEST CONNECTOR IN THE  
LANDERS AND BIG BEAR EARTHQUAKES

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ABSTRACT

The strong motion records from the Northwest Connector (located in Colton, Calif.) in the 1992 Landers and Big Bear earthquakes provide valuable information about the seismic behavior of this common type of freeway bridge. Modeling of the bridge with reasonable estimates of the column stiffness and pile foundation properties and with gap elements for the intermediate hinges provides a good correlation between the computed and the recorded earthquake response of the bridge. Pounding of the hinges produces large acceleration spikes, causing sharp increases in the column shear forces. Since modeling the hinge opening-closing is relatively simple, this type of analysis is recommended for the design of multiple-frame bridges.

INTRODUCTION

Connectors are elevated, curved bridges that allow traffic to travel between freeways at an interchange. Interest in the seismic behavior of connectors began when the 1971 San Fernando earthquake caused severe damage to several bridges. As shown in an experimental study conducted nearly twenty years ago (Williams and Godden, 1976), the curved superstructure of a connector can be very effective in resisting seismic loads. The intermediate hinges between the frames, however, reduce the stiffness of the box girder and are susceptible to local damage.

The Landers and Big Bear earthquakes on June 18, 1992, triggered an extensive strong motion instrumentation network at the Northwest Connector, a bridge at the Interstate 10/215 interchange in Colton, Calif. (see Fig. 1). The Connector is a 2540 ft long, curved, concrete box girder bridge with sixteen spans supported by single column bents and diaphragm abutments.

This paper summarizes a study of the seismic response of the Northwest Connector in the two 1992 earthquakes. The objectives of the study are to use the strong motion data recorded at the Connector to improve the understanding of the behavior of this common type of freeway bridge and calibrate the modeling and analysis procedures used in bridge design.

DESCRIPTION OF SITE, BRIDGE AND INSTRUMENTATION

The elevation and plan of the Connector, including the strong motion instrumentation, are shown in Fig. 2. Although the design was completed in 1969, the need to identify the causes for the failure of connectors in the 1971 San Fernando earthquake, and incorporate the lessons into new bridge construction, delayed completion of the Colton interchange until 1973. While the interchange was under construction, the San Jacinto fault zone was mapped in close proximity to the structures. The location of the fault in relation to the interchange is indicated in Fig. 1.

The fault acts as a barrier against ground water flow, causing a drop in the water table on the southwest side of the fault compared with the northeast side. The foundation soils consist of

slightly compact to dense sand in the upper layer underlain by compact to dense sand, silty sands, and sand and gravel mixtures (Jackura, 1991). The sandy soils and high ground water table present a potential for liquefaction during an earthquake.

The structural system for the bridge consists of six frames, connected at five intermediate hinges. The hinges are designated by the spans in which they are located: H3, H7, H9, H11, and H13. The frames have a cast in-place box girder superstructure supported by two to four single column bents. The box girder is 8 ft deep, and the original columns have 8 ft by 5.5 ft octagonal sections. The box girders in two of the frames (H3 to H7, and H9 to H11) are post-tensioned in the longitudinal direction. The spans of the four conventionally reinforced frames range from 75 ft to 155 ft. The spans of the two post-tensioned frames range from 183 ft to 204 ft. The column height (from top of pile cap to the box girder soffit) varies from 24 ft for Bent 16 to 77 ft for Bent 5. Most of the bents are oriented without skew, with the exception of Bents 11 to 14.

The five intermediate hinges in the superstructure have a seat width of 32 in. or 36 in., of which 2 in. is expanded polystyrene joint seal. The hinge support for the girders is provided by elastomeric bearing pads. Although it was not possible to determine material specification for the pads, the plans show they are 4 to 5.5 in. thick. Relative transverse displacement at a hinge is prevented by a 1 ft shear key, with a specified 1/4-in. joint filler.

The original foundations for the single column bents consist of a pile cap (without top reinforcement) and reinforced concrete piles. The largest foundations (for Bents 4, 5, and 7) have a 24 ft by 23 ft pile cap, 6.75 ft thick, and 48 piles. At the diaphragm abutments, the box girder is integral with a 13 ft high backwall.

The decision to retrofit the Northwest Connector was based on the importance of the bridge, known deficiencies in the original design (Roberts, 1991), and the fact that it crosses a mapped fault. The features of the extensive retrofit are field-welded steel jackets on all the bent columns, strengthened pile caps and foundations for most of the bents, new cable restrainer units for the hinges, and supplemental support beams under the box girder near the abutments.

The Northwest Connector was instrumented with a network of strong motion accelerometers by the California Division of Mines and Geology, in conjunction with Caltrans. The instrumentation consists of 34 force-balance accelerometers, as shown in Fig. 2. In this paper the directions based on the chord of the bridge are called the global longitudinal and global transverse directions. The tangential and radial directions of the curved bridge are called the longitudinal and transverse directions, respectively. A sheltered free-field ground motion station is located approximately 1400 ft from Bent 8.

### EARTHQUAKES RECORDED AT THE CONNECTOR

Within a year of the seismic retrofit, the Connector withstood three earthquakes in the Landers sequence, as summarized in Table 1. The plots of the processed records are available in the DMG reports (Darragh, Cao et al., 1993). During the current study, the Connector was subjected to the Northridge earthquake. Table 1 also summarizes the maximum free-field ground motion at the bridge site.

After the two earthquakes on June 28, 1992, Caltrans personnel inspected the Connector. The left barrier (inside edge of deck) near Hinge 3 had spalled an area 8 in. by 6 in. with a depth of 6 in., exposing steel reinforcing bars. The seat of Hinge 3 had three hairline cracks radiating from the reentrant corner. Another inspection report described settlement at the hinges, possibly caused by earthquake damage to the elastomeric bearing pads (Yashinsky, Maulchin et al., 1992).

Table 1. Earthquakes and Free-Field Ground Motion Recorded at the Northwest Connector

Event	Date	Magnitude $M_s$	Epicentral Distance (miles)	Peak Free-Field Ground Acceleration (g)		Bracketed Duration <sup>b</sup> (sec)
				Horizontal <sup>a</sup>	Vertical	
Joshua Tree <sup>c</sup>	4/22/92	6.3	52			
Landers	6/28/92	7.6	50	0.089	0.062	26
Big Bear	6/28/92	6.6	28	0.112	0.073	14
Northridge <sup>d</sup>	1/17/94	6.8	72	0.10	0.04	

<sup>a</sup>Instantaneous peak horizontal acceleration.

<sup>c</sup>Joshua Tree record was not available from DMG.

<sup>b</sup>Time the ground acceleration peaks exceed 0.05 g.

<sup>d</sup>Based on unprocessed records.

### FREE-FIELD AND INPUT MOTION

The pseudo-acceleration response spectra, with 5% damping, for the horizontal free-field ground acceleration in the principal axes are shown in Fig. 3. The Big Bear earthquake has larger spectral ordinates than the Landers earthquake for vibration periods less than 1 sec. For periods greater than 3 sec, the spectral ordinates for Landers are about twice that for Big Bear. The spectra for both earthquakes have a peak near a period of 1.8 to 1.9 sec. This peak may be associated with the characteristic vibration period of the deep alluvial site.

The response spectra for ground motion in the principal axes can be compared with the smooth design response spectrum used by Caltrans, as shown in Fig. 4. The S.7GD51 ARS spectrum corresponds to a 0.7g peak ground acceleration and an alluvial site with soil depth greater than 150 ft (Caltrans, 1990). Also shown in Fig. 4 is the ARS spectrum reduced by a Z factor of four, a typical reduction factor to account for inelastic behavior and seismic risk of single column bents. In the short period range, less than 1 sec, the spectra for the earthquakes exceed the reduced ARS/Z spectrum. For longer periods the design spectrum envelopes the spectra for the two earthquakes, except for one peak for Landers near 1.9 sec. The important vibration modes of the Connector have periods between 1.0 and 1.7 sec. There is not a large difference between the reduced ARS/Z spectrum and the recorded spectra in this period range, so the forces developed in the columns during the two earthquakes may have approached the nominal design strength.

The free-field ground motion records were not time synchronized with the records from the Connector. The time lag between vertical Channel 23 relative to the vertical free-field channel is determined by computing the correlation coefficient between the processed displacement records. Based on the correlation coefficients, it is assumed that the Connector records started 1.96 sec after the free-field records for Landers and 0.64 sec for Big Bear.

The input motion for the Connector was recorded at four supports: Abutments 1 and 17, and Bents 3 and 8. Figures 5 and 6 show the displacement histories at the four supports in the global longitudinal direction and global transverse direction for the two earthquakes. The heavy line is the free-field displacement histories in the same directions, accounting for the aforementioned time lags. The displacement histories are similar at the four supports and the free-field location, indicating that the pseudo-static effects of multiple-support excitation are not large. In the long period range, the dynamic effects of multiple-support excitation are not large either.

## RECORDED DISPLACEMENT RESPONSE

The processed acceleration and integrated displacement records allow evaluation of the superstructure displacements relative to the supports and opening-closing displacements of the hinges by subtraction of two displacement records. The baseline correction and high-pass filtering obscures any residual displacement.

Bent 8 Response

The instruments at Bent 8, near the center of the Connector, provide detailed response of the bent. The displacement at the top of the column is computed considering the rotation of the box girder, the pile cap rotation, and deformation of the column, as shown in Fig. 7. The dotted line is the displacement at the top due to the pile cap rotation, and the solid line is the displacement at the top due to column deformation and pile cap rotation. The rotation of the pile cap is in-phase with the transverse displacement. The difference in the two histories is the deformation of the column, which is a maximum of 4.76 in. for Landers (drift=0.86%), and 2.98 in. for Big Bear (drift=0.54%).

The pile cap rotation produces displacements at the top of the column as a result of soil-structure interaction: 0.63 in. for Landers and 0.47 in. for Big Bear. For Landers, the ratio of the rotational soil stiffness to transverse stiffness of the column is 0.13, and the period is lengthened by a factor of 1.06 due to soil-structure interaction. For Big Bear, the stiffness ratio is 0.16 and the period ratio is 1.08. It is difficult to estimate the stiffness of the structure to within about 15%, but these soil-structure interaction effects are at about the limit for which they should be considered in the analysis of the bridge.

Hinge Response

The longitudinal acceleration records show several large spikes that are caused by pounding of adjacent frames at the hinges. The longitudinal opening-closing displacements at Hinge 7 and Hinge 11 (inside edge of the deck) are shown in Figs. 8 and 9 for the two earthquakes. Hinge 7 has the largest opening: 1.41 in. for Landers and 1.70 in. for Big Bear. The closing of the hinge (negative displacement) is harder to discern. Hinge 7 closes about 1/2-in. during the first 25 sec of the Landers earthquake. Later the maximum closing displacement is about 1 in., indicating that filler material in the hinge may have crushed during the strong motion response later in the earthquake. The opening at Hinge 11 is less than the opening of Hinge 7, and the maximum closing displacement is 0.86 in. Hinge 7 opens more in the Big Bear earthquake than in Landers. The maximum closing displacement is nearly 1.5 in., possibly indicating accumulated pounding damage at the hinge in the second earthquake. The response of Hinge 11 is similar for the two earthquakes. The opening displacements are well below the yield displacement of the restrainers (4.2 in.), even neglecting the initial slack in the cables.

As shown in Fig. 10, the maximum transverse displacement for Hinge 3 is 0.35 in. for Landers. The relative transverse displacement of Hinge 7 is substantially different than for Hinge 3. There is little displacement for the first 18 sec in Landers. However, there are then larger displacement excursions at a frequency of approximately 0.5 Hz. The maximum relative displacement is 0.47 in. The relative transverse displacement is slightly larger for Big Bear.

One concern about the seismic performance of hinges is the possibility that torsion of the box girder may cause vertical lift-off and pounding on the elastomeric bearing pads. The relative vertical displacement at the outside edge of Hinge 7 is less than 0.20 in. opening and 0.27 in. closing for both earthquakes, and it is most likely caused by deformation of the pads. In contrast, the relative vertical displacements at the inside edge are larger. The maximum relative opening is 0.34 in. and 0.39 in. for Landers and Big Bear, respectively. The maximum relative closing is

0.55 in. and 0.36 in. for Landers and Big Bear, respectively. It is possible that lift-off and pounding occurred on the inside edge of the Connector, with rotation about the outside edge. The drawings for the bridge do not show vertical tie-down restrainers.

### VIBRATION PROPERTIES OF CONNECTOR

The transmissibility functions are computed from an input acceleration in one direction relative to the output acceleration at various locations in the superstructure using power and cross-power spectral density functions. Since the predominant input motion is in the transverse direction, two transverse input motions are used to compute transmissibility functions: the support acceleration for Bent 8 (Channel 24), and the free-field ground acceleration. Because of space limitations, Figs. 11 and 12 show some of the transmissibility functions for transverse channels using the Bent 8 motion for the two earthquakes.

Based on the spectral analysis and additional identification using an auto-regressive analysis (Ljung, 1987), the vibration periods and damping ratios for the lower vibration modes of the Connector are listed in Table 2. The most significant finding is the increase in the fundamental mode period of the Connector from 1.56 sec in the Landers earthquake to 1.75 sec in the Big Bear earthquake. The period lengthening and the increase in damping may indicate that the structure or foundation softened in the first earthquake.

Table 2. Vibration Periods and Damping Ratio Identified from Recorded Motion

Mode	Landers Earthquake		Big Bear Earthquake	
	Period (sec)	Damping Ratio	Period (sec)	Damping Ratio
1	1.56	0.03	1.75	0.08
2	1.30	0.11	1.29	0.02
3	0.98	0.05	1.09	0.15
4	0.83	0.07	0.97	0.07

### MODELING OF THE CONNECTOR

Structural modeling and earthquake analysis are a central step in the design of new bridges and evaluation of existing bridges. There are always questions about how well the models predict the response of an actual bridge. An objective of the current study is to develop a model of the Northwest Connector and use the model to compute the response of the bridge to the earthquakes, and compare the computed response with the recorded response. The approach is to use methods of modeling and dynamic analysis typically available for bridge design. The details of the modeling will be available in the final report.

Since the Connector did not experience inelastic deformation in the earthquakes, it is appropriate to use linear, elastic models for the structural components. The foundations are modeled as linear, elastic springs. The opening and closing of the intermediate hinges, however, is nonlinear and that behavior is represented in the model of the Connector. The dynamic analyses

are performed using the computer program SADSAP (Wilson, 1992). The program includes standard linear frame elements, non-linear one-dimensional elements for gaps, tension-only elements, and compression-only elements. Models of the intermediate hinges are constructed using the nonlinear elements.

One of the parameters in the model is the factor by which the gross moments of inertia for the columns are reduced to represent the level of deformation. The reduction factor is selected as the primary parameter to match the computed response with the recorded response in the two earthquakes. Based on interpretation of the test data for steel jacketed columns (Priestly, Seible et al., 1992), the moments of inertia for a column are increased 20% or 10%, depending on the full or partial jacket retrofit, to account for the stiffening effect of the jacket.

The stiffness of the pile foundations are represented by translational and rotational springs at the pile cap. The translational spring stiffness is based on a lateral stiffness of 65 kip/in per pile, which is 30% of the stiffness based on the gross properties of the piles using the small strain estimate of shear wave velocity for the soil. The reduction accounts for pile cracking, larger soil strains, soil gapping, and group effect. The rotational stiffness of the pile cap was determined from the displacements of Bent 8 described previously. The rotational stiffness of  $4 \times 10^7$  k-ft/rad is also about 30% of the rotational stiffness of the pile group using the axial stiffness of the piles.

The three-dimensional model of the Connector consists of approximately 300 frame elements and 40 nonlinear elements with 1500 degrees-of-freedom. The model is illustrated in Fig. 13 with the first four vibration modes of the bridge. The properties are calibrated for the Landers response, in which the column stiffness is based on the gross section properties. The model is modified for the Big Bear earthquake by multiplying the stiffness of the columns by a factor of 0.65 and the springs for the pile foundations by a factor of 0.50. The first four vibration periods for the modified model are: 1.74 sec, 1.44 sec, 1.36 sec, and 1.12 sec.

Comparison of the calculated periods with the identified periods in Table 2 shows a good match for the first mode, but differences in the higher modes. More detailed study is required to determine the cause of the discrepancy. As discussed in the next section, the correlation of computed and recorded response is fairly good.

### RESPONSE COMPARISON

The models are analyzed using the recorded Bent 8 support motion as uniform input motion. The use of the recorded free-field motion as uniform input gives similar results. For Landers, the model is assumed to have a damping ratio of 3% in all modes, and for Big Bear the damping is assumed to be 5% in all modes.

The comparison of computed and recorded total displacements at selected channels is shown in Figs. 14 and 15 for the Landers and Big Bear earthquakes, respectively. In general, the models represent the overall dynamic response of the Connector recorded in the earthquakes. Although there are some differences in the peaks, the correlation is generally good for both earthquakes, particularly in regard to the phasing of the displacements.

The ability of the gap elements to model the hinge opening and closing is of particular interest. Figure 16 shows the longitudinal displacement of Hinge 7 and Hinge 11 for the two earthquakes. The simple hinge modeling represents the hinge displacements fairly well.

Since bridge design is currently based on linear models for earthquake analysis, the effects of intermediate hinges are bounded by a "tension model" and a "compression model." The former

includes linear elements representing the restrainers with no longitudinal constraint at the hinges. The latter model connects the frames with moment releases at the hinges. Figure 17 shows the maximum longitudinal shear force in the columns computed from the tension model, compression model, and the nonlinear model with hinge opening-closing. The two linear models underestimate the shear force at several of the columns, mostly the short ones. The larger shear forces develop because of pounding at nearby hinges, which is not captured in the linear models.

### CONCLUSIONS

The strong motion response of the Northwest Connector in the 1992 Landers and Big Bear earthquakes provides valuable information about the dynamic response of this common type of bridge. The intermediate hinges opened as much as 1.7 in., which is well below the yield displacement of the cable restrainers. Typical modeling of the bridge with estimates of the column stiffness and pile foundation properties provide a reasonable correlation between the computed and the recorded responses, although there are some errors in the periods of the higher modes. Pounding of the hinges produces large acceleration spikes, which causes increased shear forces in the columns. Since modeling the hinge opening-closing is relatively simple using gap elements, this type of analysis is recommended for the design of multiple-frame bridges.

### ACKNOWLEDGMENTS

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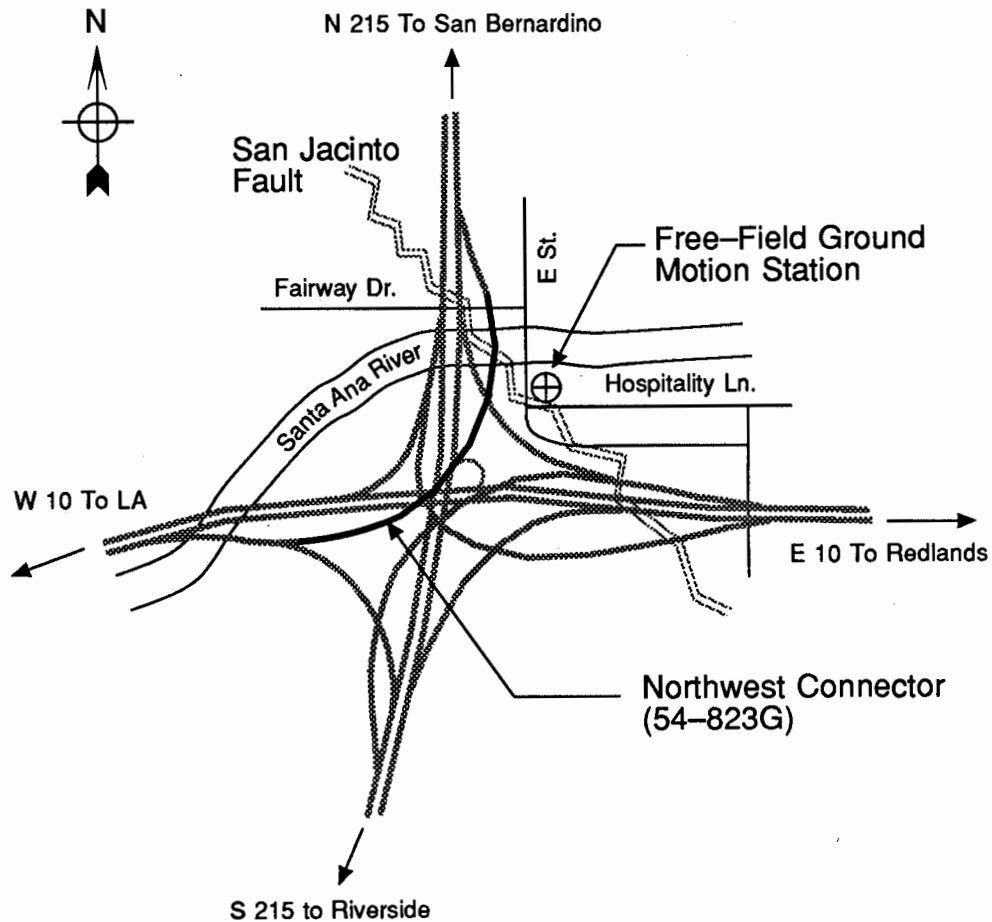


Fig. 1 Interstate 10/215 Interchange in Colton, CA.



San Bernardino - I10/215 Interchange  
 (CSMP Station No. 23631)  
**SENSOR LOCATIONS**

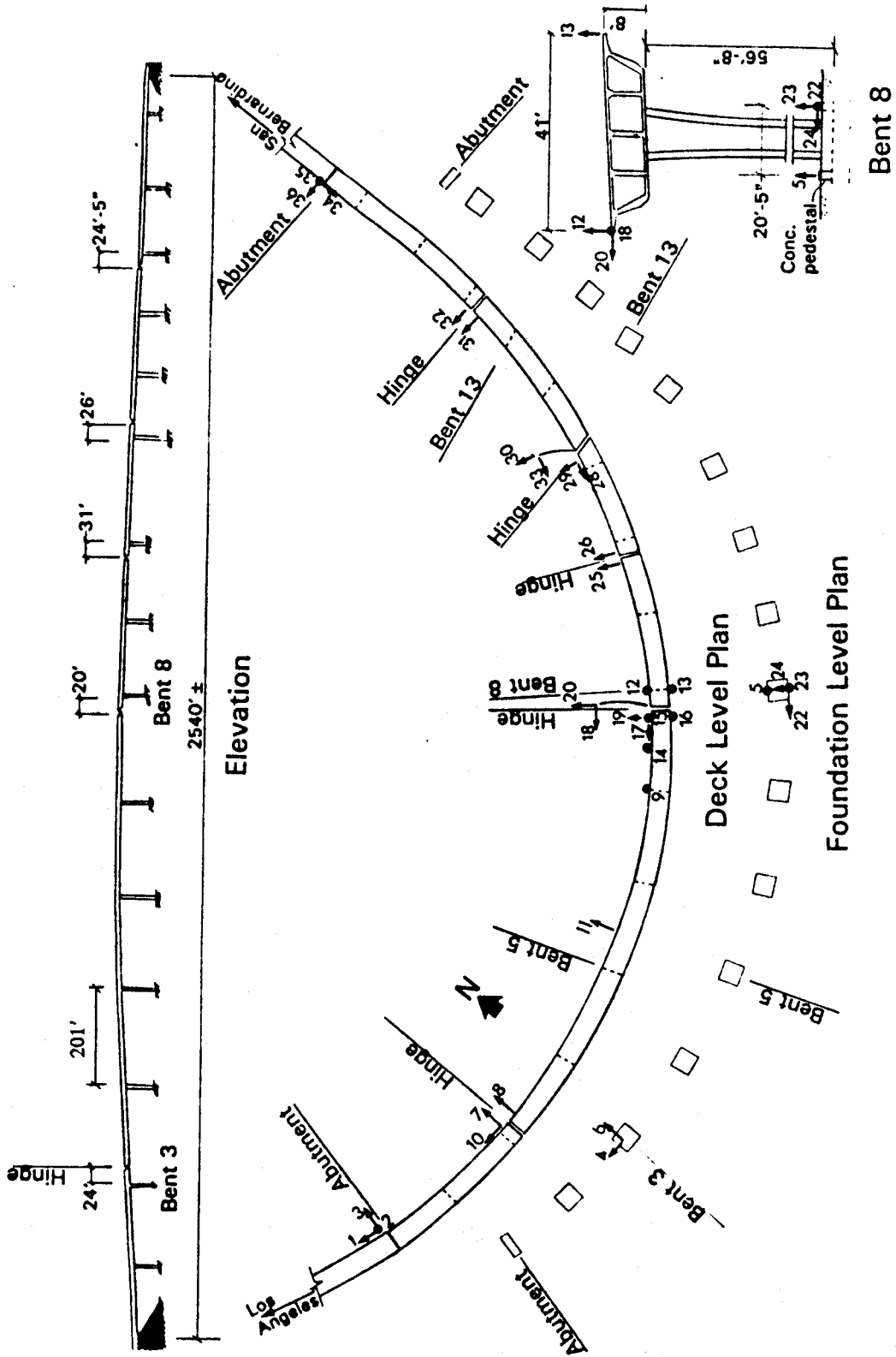


Fig. 2 Strong-Motion Instrumentation Plan for the Northwest Connector (Darragh, Cao et al., 1993).

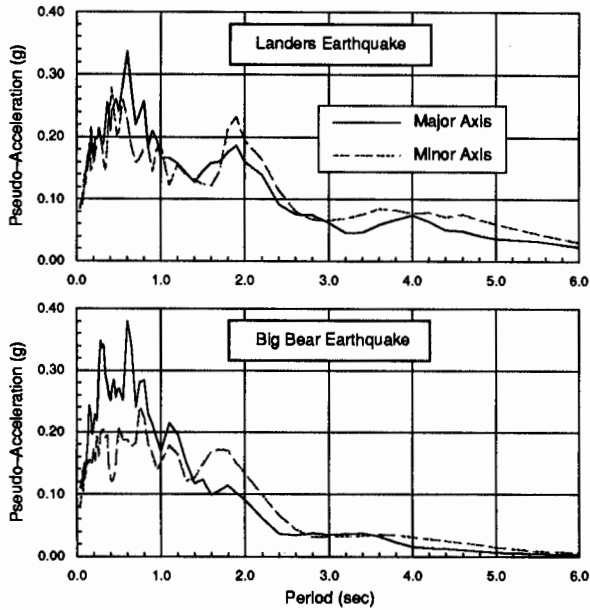


Fig. 3 Response Spectra for Free-Field Ground Motion (5% Damping).

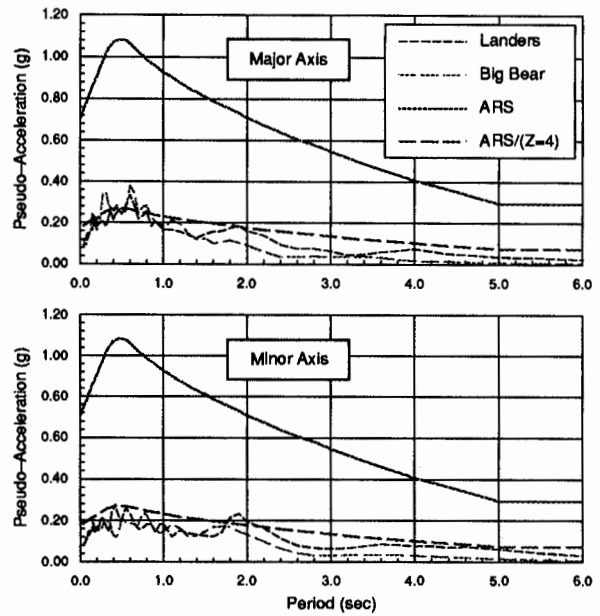


Fig. 4 Response Spectra for Free-Field Ground Motion Compared with ARS Design Spectra (5% Damping).

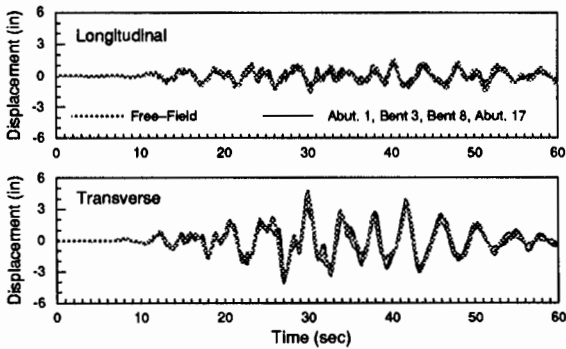


Fig. 5 Input Motion in Global Directions for Landers Earthquake.

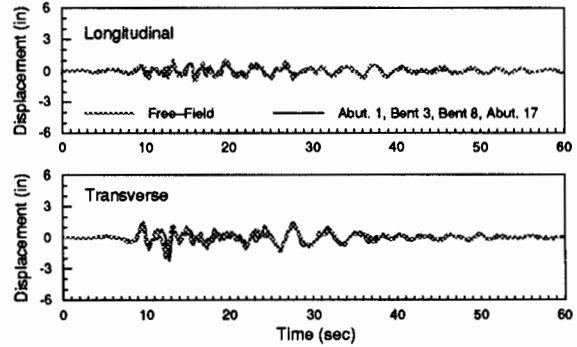


Fig. 6 Input Motion in Global Directions for Big Bear Earthquake.

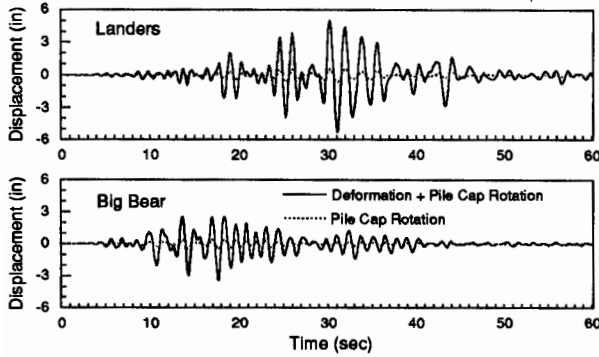


Fig. 7 Transverse Displacements of Bent 8 Column.

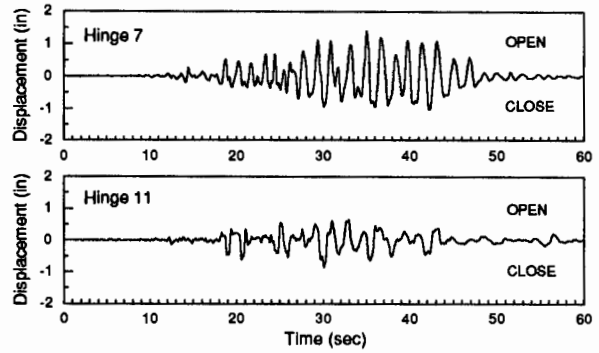


Fig. 8 Longitudinal Displacement of Hinges in Landers Earthquake.

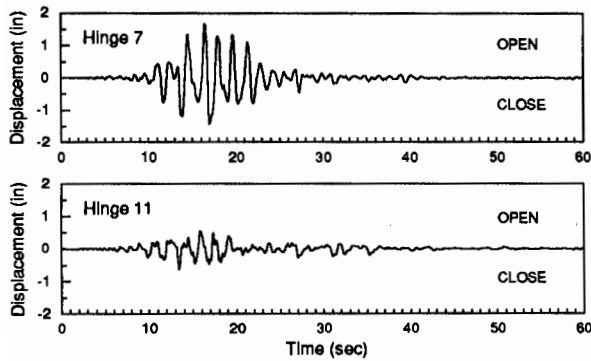


Fig. 9 Longitudinal Displacement of Hinges in Big Bear Earthquake.

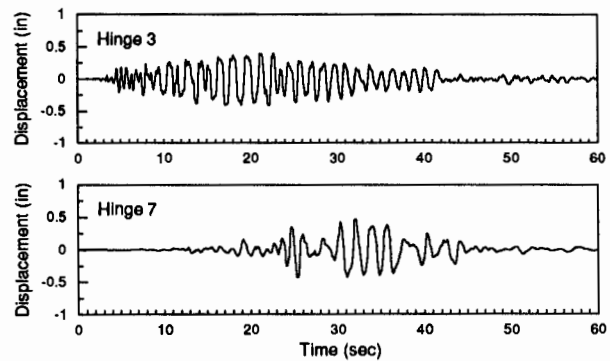


Fig. 10 Transverse Displacement of Hinges in Landers Earthquake.

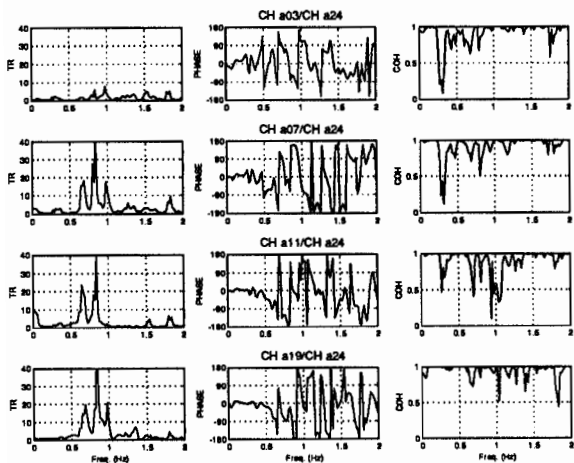


Fig. 11 Spectral Response Using Bent 8 Input Motion in Landers Earthquake.

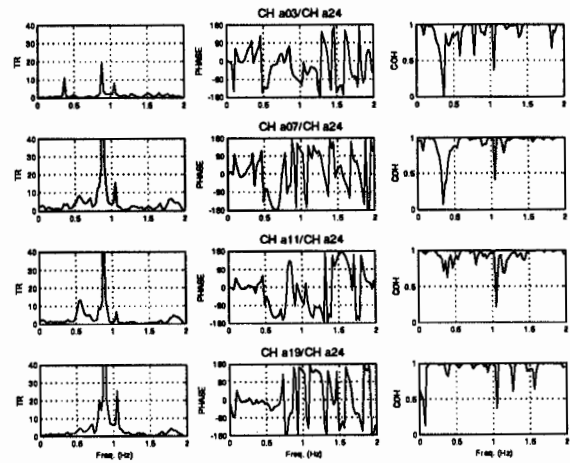


Fig. 12 Spectral Response Using Bent 8 Input Motion in Big Bear Earthquake.

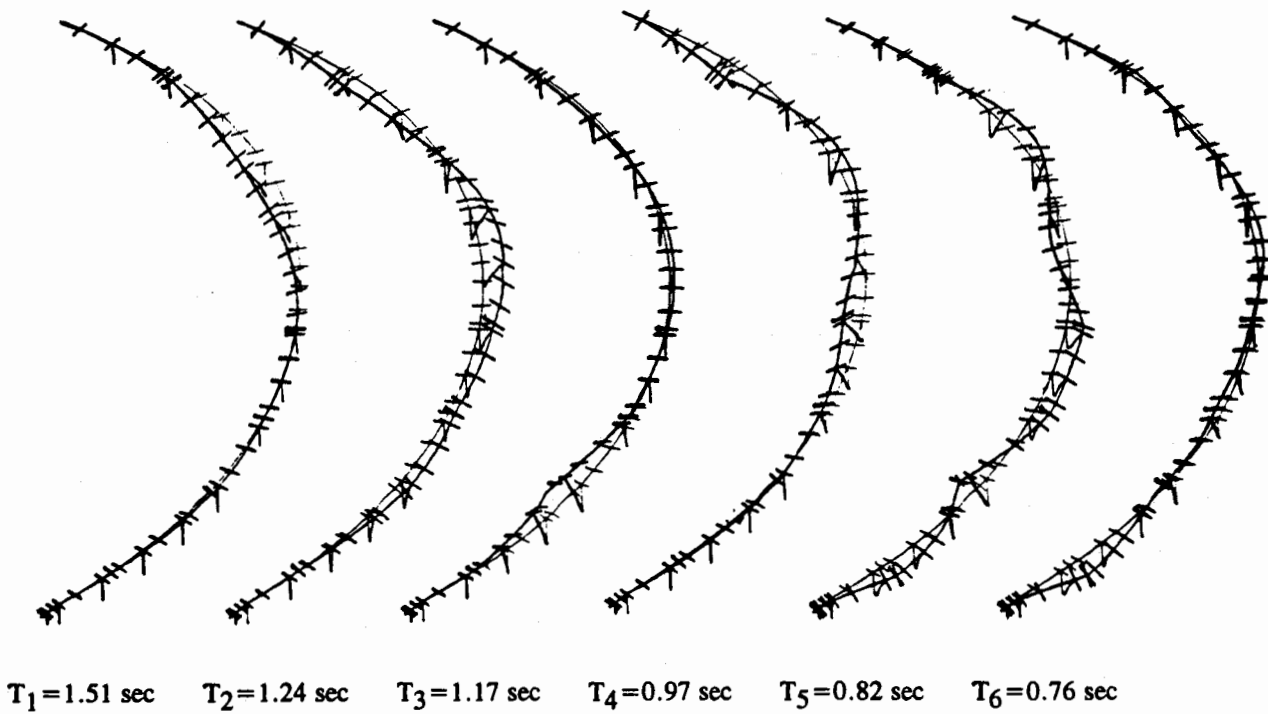


Fig. 13 First Four Vibration Modes of the Model for the Northwest Connector.

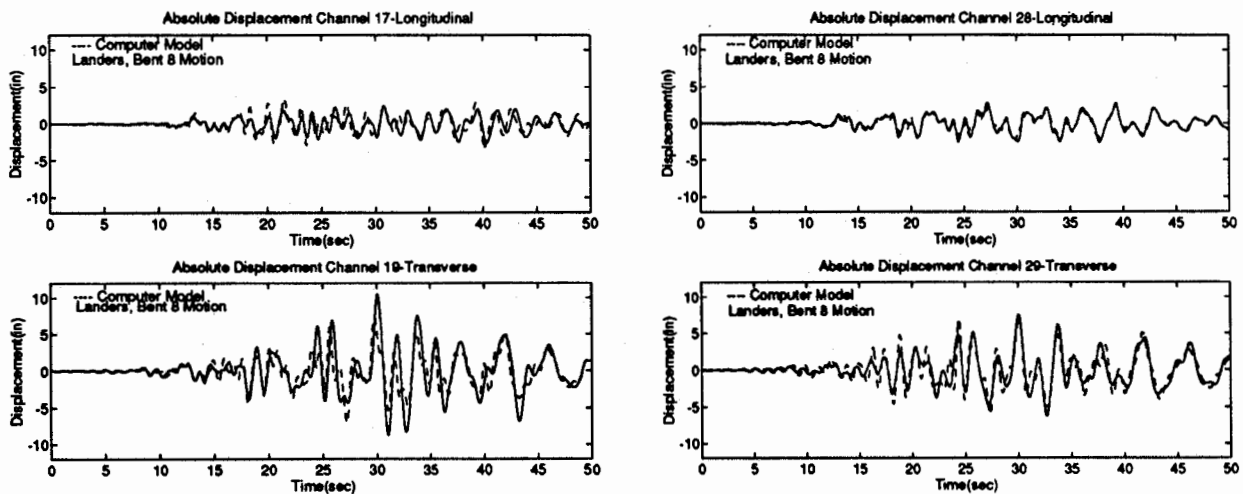


Fig. 14 Comparison of Recorded and Computed Response in Landers Earthquake.

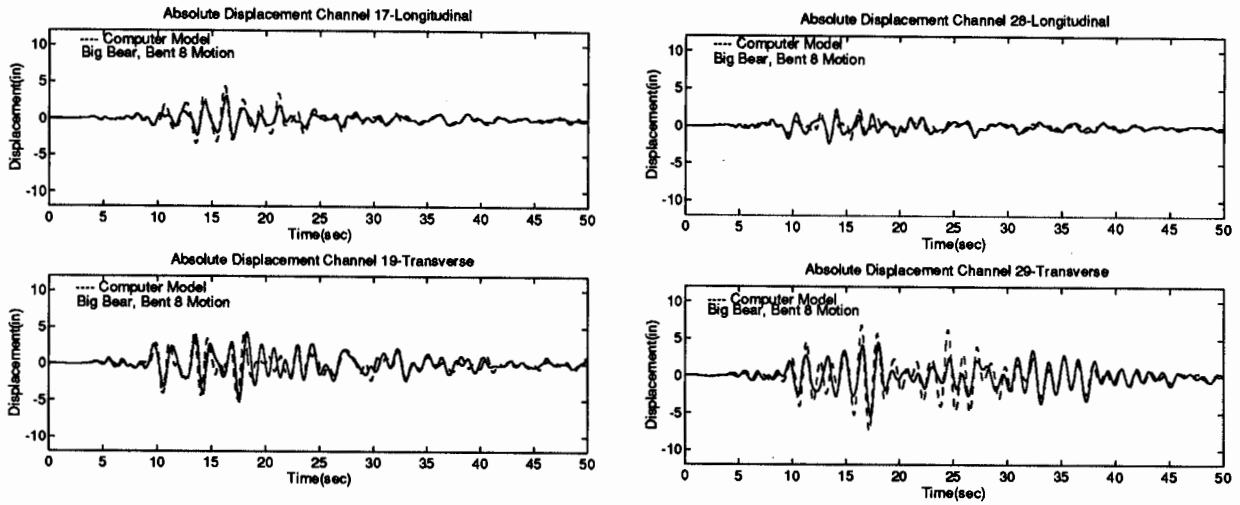


Fig. 15 Comparison of Recorded and Computed Response in Big Bear Earthquake.

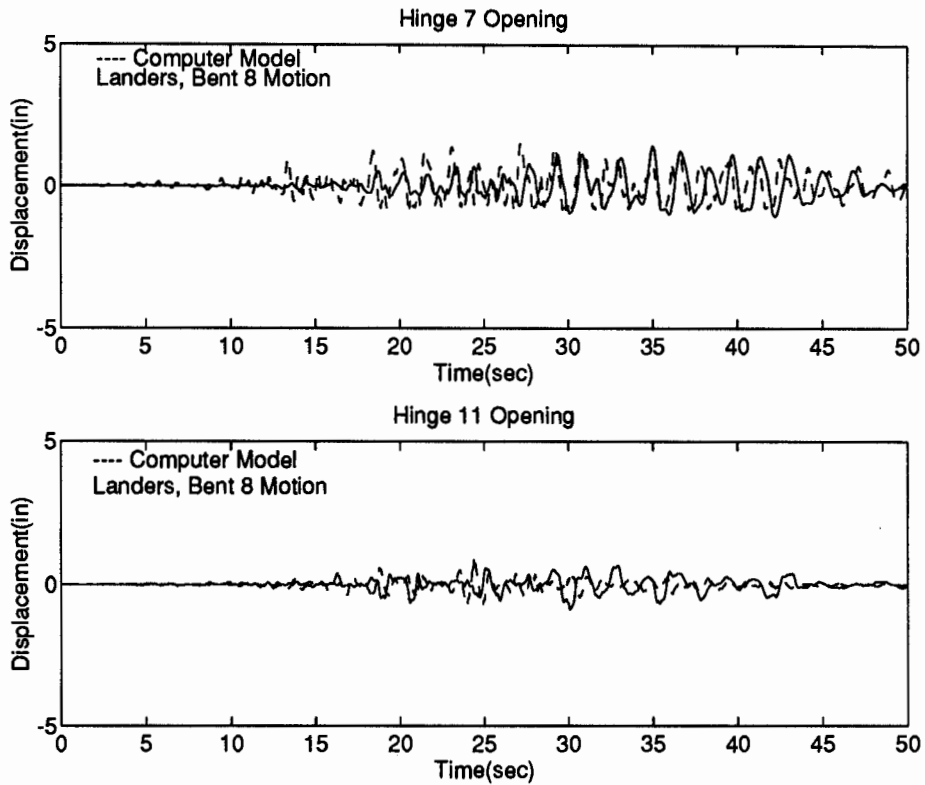


Fig. 16 Comparison of Recorded and Computed Longitudinal Hinge Displacement in Landers Earthquake.

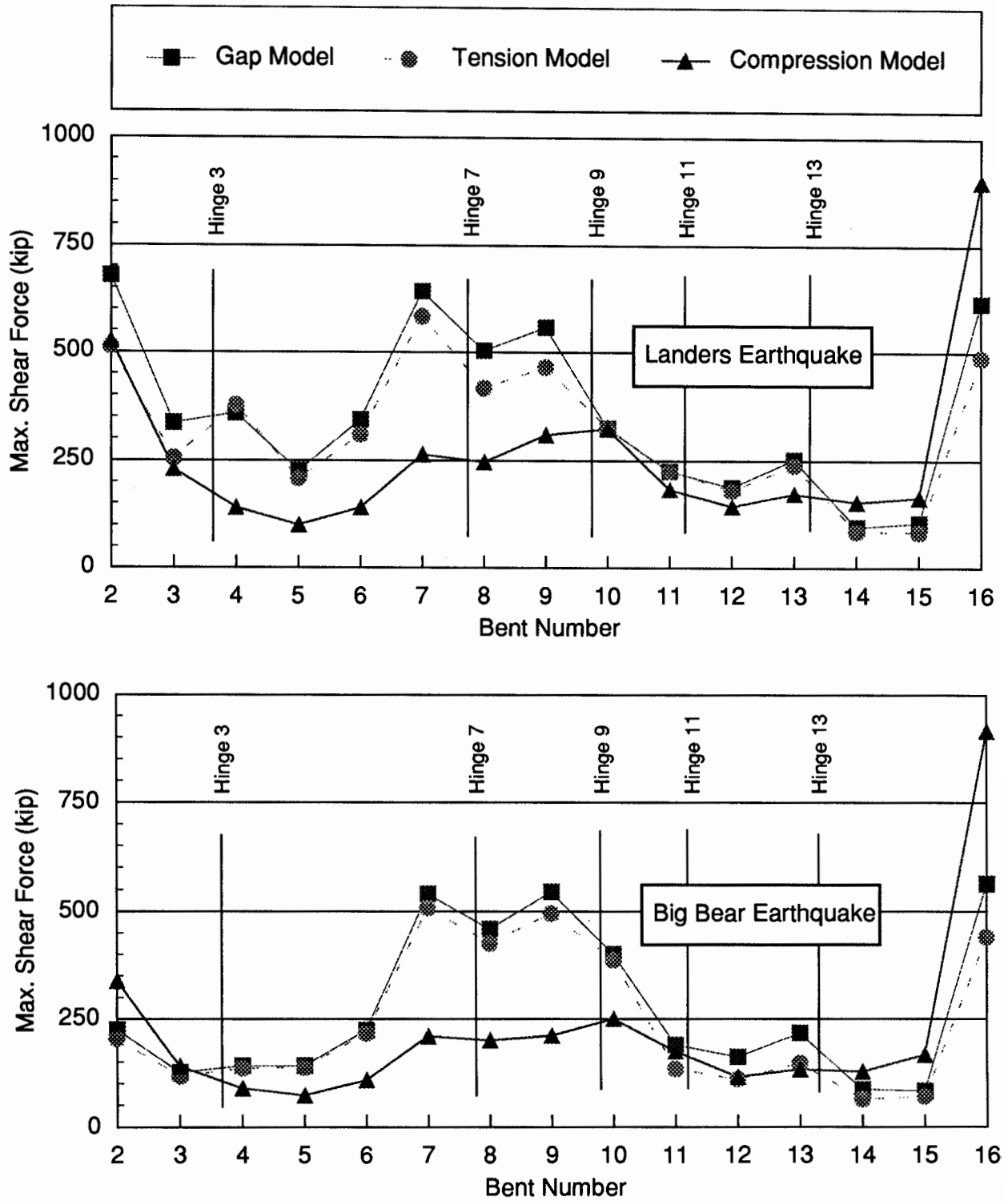


Fig. 17 Maximum Longitudinal Shear Force in Columns for Three Models.