EFFECT OF RELATIVE DISPLACEMENTS BETWEEN ADJACENT BRIDGE SEGMENTS

Kazuhiko Kasai[1], Wen D. Liu[2], and Van Jeng[1]

ABSTRACT

This paper presents the results of a study on the responses of the San Juan Bautista 101/156 Separation Bridge instrumented by the California Division of Mines and Geology under its Strong Motion Instrumentation Program (CSMIP). The bridge experienced three significant earthquakes as follows: 1979 Coyote Lake earthquake (before seismic retrofit), 1984 Morgan Hill earthquake (after seismic retrofit), and 1989 Loma Prieta earthquake. The recorded seismic responses during the three earthquakes are analyzed and correlated with theoretically predicted responses. Adjustments of structural parameters and modeling concept to achieve satisfactory correlations are discussed. Important characteristics of relative motions between adjacent bridge segments are explained based on the records and analyses. Comments are also given regarding the performance of cable restrainers for seismic damage mitigation.

INTRODUCTION

During an earthquake, adjacent bridge segments often vibrate out-of-phase due to their different dynamic characteristics as well as their support conditions that allow the relative motions between the segments. The out-of-phase motion of the bridge segments leads to two types of relative displacement problems: First, when the distance between the segments increases and exceeds the range of support provided by either abutment, column, or hinge seat, a falling of bridge deck could occur. Many of the catastrophic loss-of-span type failures of bridges in the past earthquakes have been attributed to this effect. Second, when the distance between the segments decreases, pounding (i.e., collision) of the segments could occur and severe impact force could develop at the contact region. Unlike the case for buildings [2], this typically results in localized damage of the bridge segments.

To prevent the serious loss-of-span failure from occurring in either a new or an existing bridge, one could provide restrainers to limit the increase of distance between the segments. During the period from 1974 to 1985, such practice was performed on all existing California bridges by the California Department of Transportation (CALTRANS) as a part of its Phase 1 seismic retrofit project. The seismic behavior of such a retrofitted bridge appears to be extremely complex due to the participation of the cable restrainers in the relative displacement responses of adjacent bridge segments. Clearly, it is necessary to understand the relative motion between adjacent segments including the effects of cable restrainers and develop a rational relative motion analysis method in order to mitigate the catastrophic collapse of either a new or an existing bridge in a future severe earthquake. The study presented herein performs correlative seismic analyses for a bridge structure that has experienced moderate excitation in 1979 before cable restrainers installation, and in 1984 after cable restrainers installation as well as significant excitation in 1989. The objectives are: (1) to study the degree to which practical computer analysis models can accurately capture actual bridge seismic relative displacement time history response (with and without restrainers); (2) to discuss the complexities involved in bridge relative displacement problems that must be considered to evaluate the seismic performance of a bridge.

DESCRIPTION OF STUDY BRIDGE

Fig. 1 shows the San Juan Bautista 101/156 Separation bridge considered in this study. This multispan, steel plate girder structure is one of the first bridge instrumented (in 1977) by CSMIP. The

[1] Assistant Professor, Dept. of Civil Engineering, Illinois Institute of Technology, Chicago, IL 60616.
[2] Associate, Imbsen and Associates Inc., Sacramento, CA 95827.

bridge is a skewed bridge of total length 326 feet with five bents, and it consists of separated segments. The bent span varies from 33.5 to 68.5 feet. The bridge was constructed in 1958 and was retrofitted with cable restrainers in the early 1980's. The bridge is well instrumented having 12 acceleration recording channels at Bents 3 to 5 (see the layout in Fig. 1). These channels are oriented in two orthogonal directions (i.e., normal to the bent and tangential to the bent), respectively. The structure consists of six simple spans supported on seat-type abutments and five two-column bents (Fig. 1). The abutments and bents are skewed at 34.8 degrees with respect to the bridge centerline. The superstructure is composed of five steel girders composite with a reinforced concrete deck (Fig. 2). Each of the five girders is supported on fixed bearing at one end of the span which provide a pinned connection, and an expansion (rocker) bearing at the other end. The expansion (rocker) bearings are oriented to allow the movement in the longitudinal direction of the bridge. Across the girders, intermediate diaphragms normal to the bridge centerline provide lateral support to the girders. At both ends of each span, end diaphragms are provided along the direction parallel to the bent (i.e., 34.8 degrees skewed from the normal).

The simple supports at the abutments and bents are made of metal bearings. The fixed bearing assembly is used at the northern end of each girder to provide a pinned connection. An expansion bearing assembly provides a roller connection at the southern end of each girder. Fig. 3 shows how these bearings were oriented on the concrete pedestals at abutments and bents to provide for thermal and shrinkage movements along the longitudinal axis of the bridge. The connections of these bearings are vulnerable to seismic damage. Once bearing connections failed, the deck may be unseated from the bent support. During the early 1980's, Caltrans retrofitted the bridge with longitudinal, transverse and vertical restrainers at various bearing locations and also extended the length of concrete pedestal at abutments. Fig. 4 shows the detail of longitudinal restrainers connecting the girder to the bent cap.

ANALYSIS OF RECORDED SUPPORT MOTIONS

The study bridge experienced three significant earthquakes as follows: (1) 1979 Coyote Lake earthquake (before cable restrainers addition); (2) 1984 Morgan Hill earthquake (after cable restrainers addition); and (3) 1989 Loma Prieta earthquake (after cable restrainers addition). The two earlier earthquakes originated from Calaveras fault, while the recent Loma Prieta earthquake originated from the San Andreas fault. The bridge is almost directly on top of the San Andreas fault.

Support Accelerations. - The recorded peak accelerations during the past three earthquakes are summarized in Table 1. The bridge shaking was the most severe during Loma Prieta earthquake, and was the least severe during Morgan Hill earthquake. The frequency contents of the two-directional support accelerations are obtained by calculating the smoothed Fourier amplitude spectra (FAS), and are shown in Fig. 5(a) for the three earthquakes. Loma Prieta earthquake caused extremely low frequency accelerations in both normal and tangential directions. Figs. 5(b) and (c) show the resulting response spectra with damping ratio = 5%. Note the extremely large displacement response of a low frequency system (Fig. 5(c)) subjected to the Loma Prieta motion.

Support Displacements. - Fig. 6(a) compares the displacement histories (integrated by CSMIP) of Bent 5 support due to the three earthquakes. By performing frequency analyses (Fig. 6(b)), it was found that the support motions have about 2.5 to 3.5 sec. period in the two orthogonal directions for both Coyote Lake (see also Ref. 4) and Morgan Hill earthquakes. However, a longer period of about 6 sec. was found (see Figs. 6(a) and (b)) for the support displacement due to Loma Prieta earthquake, which is consistent with the trend discussed earlier for the support acceleration (Fig. 5(a)). Further, the magnitude of the displacement is extremely large as compared with those from the other earthquakes. This striking difference might be attributed to the fact that the bridge is located on and parallel to the San Andreas fault and it is only about 6 miles away from the south end of the rapture of Loma Prieta earthquake. Transforming the recorded motion shown in Fig. 6(a), it was observed that the maximum ground displacement in the direction perpendicular to the earthquake wave moving direction is about 3 times the maximum ground displacement in the other direction. Similar trends were observed in many of the accelerometers located elsewhere during the Loma Prieta event (see Ref. 3).

Relative Support Displacement. - The difference between the displacements of two distant supports (i.e., relative support displacement) has been long interested by engineers due to its importance in designing expansion joint and support length of long span structures such as bridges and pipelines. The recorded acceleration data at Bent 5 (Channels 1 to 3) and Bent 3 (Channels 10 to 12), 107 feet apart from each other, are used to study this. The relative support displacement histories between the two supports plotted in Fig. 6(c) as well as frequency analyses indicate that, due to the effect of travelling seismic wave, their periods are similar to the ones observed above for the single support displacement. Also, the relative support displacements tend to be large when the support displacements are large (compare Figs. 6(a) and (c)). Hall and Newmark [1], as well as Wilson [4] attempted to estimate the maximum relative support displacement by using the concept of simple wave propagation theory. The data obtained from the present study would be useful to verify the accuracy of their methods. Hall and Newmark's method obtains the ground strain averaged over the distance between the supports by dividing the maximum ground velocity due to earthquake divided by the typical ground wave velocity [1]. The maximum relative support displacement is obtained as a product of the strain and the support distance. On the other hand, Wilson's method obtains the displacement by taking the difference of sinusoidal ground motions at the two supports having the vibration phase that is calculated as the distance between the supports divided by the typical ground wave velocity. This method requires the knowledge on the maximum displacement of the support. These two methods are applied to estimate the maximum relative displacement between the respective supports of Bents 3 and 5, assuming the ground wave velocity of 1000 ft/sec. It is found that Wilson's method gives significantly unconservative results, and that Hall and Newmark's method provides very close results. As will be discussed below, the magnitudes of the relative support displacements of the study bridge during the 3 earthquakes are comparable to the maximum displacement of the deck measured with respect to the support. This is mainly due to the very short period and corresponding small deformation of the bridge. Thus, in similar cases, maximum relative support displacement becomes as important as the deformation of the bridge for the relative motion problem of the bridge adjacent segments.

ANALYSIS OF RECORDED SUPERSTRUCTURE MOTIONS

Basic Structural Dynamic Characteristics. - In order to study the vibration frequency contents of the study bridge, transfer functions as well as Fourier amplitude spectra are generated from the records of the channels located at the superstructure. Fig. 7 contrasts the frequency contents of the response of Channels 4 and 5 located at the top of Bent 5 subjected to Coyote Lake earthquake and Loma Prieta earthquake (Fourier amplitude spectrum normalized to its maximum value was used for each earthquake). Fig. 7 essentially indicates the significantly higher fundamental vibration frequencies of the bridge in both normal and tangential directions during Loma Prieta earthquake as compared to the Coyote Lake event. Additional analyses using the Morgan Hill earthquake records were also conducted, and same trend was observed. The dynamic characteristics of the bridge apparently changed prior to Morgan Hill earthquake. The dominant frequency in the direction normal to bent changed from 3.1 Hz to about 4.5 Hz (during Loma Prieta earthquake), and tangential to bent from 5.5 Hz to either 6 or 7 Hz. According to the record, a retrofit work of the bridge with cable restrainers was completed before the Morgan Hill earthquake, and it might be the reason for this significant frequency change. It also should be noted that the bridge response after the retrofit seems to contain a variety of significant frequency contents, suggesting a complex nonlinear response resulting from frequent change of the structure stiffness during the earthquakes.

Impulsive Accelerations. - The significant change in frequency contents observed above for the entire duration of earthquake is further investigated by closely examining the acceleration time history of Channel 8 at the bottom of Span 4 (Fig. 1). The location is sensitive to pounding between the adjacent decks or pulling of the cable restrainers. As seen from Fig. 8(a), during Coyote Lake event the the acceleration had essentially the same frequency contents before (i.e., from 3 to 4 sec.) and during (i.e., from 4 to 5 sec.) it reached the peak, indicating that there was no significant impact in the vicinity of the channel (compare the normalized FAS taken from 3 to 4 sec. and from 4 to 5 sec., respectively). In contrast, Figs. 8(b) and (c) show that significant changes in the frequency contents before and during the occurrence of the acceleration peak resulted. For example, see the acceleration history in Fig. 8(c) showing significant increase in the frequency of the acceleration during 3 to 4 sec., as well as the corresponding normalized FAS. The above analyses conducted for Channel 8 were repeated for other neighboring channels, and the evidence of impact (somewhat less significant than recorded by Channel 8) was also found. These impulsive accelerations indicate the occurrence of pounding or pulling during Morgan Hill and Loma Prieta earthquakes, as will be illustrated analytically.

Structural Displacement.- It should be noted that the relative displacement between the two structures is the sum of the difference of their support displacements (i.e., relative support displacement) and the difference of their structural displacement measured with respect to the ground (i.e., relative structure displacement). In order to see the contribution of the relative support displacement to the relative displacements between adjacent segments of the bridge, the maximum magnitudes of these responses were compared using Channels 4 and 8 (i.e., relative displacement between the Span 4 and Bent 5). It is found that the relative support displacements are of significant magnitudes for estimating the relative displacements, mainly due to extremely large ground motions developed as well as small deformation of the bridge during the Loma Prieta event. This, however, will not be the case if the structure has larger structural period, and its displacement is large.

ANALYTICAL MODELING OF STUDY BRIDGE

Basic Structural Model and Parameters. - Three-dimensional models of the bridge were developed which accounted for the skewed supports at abutments and bents, expansion joint discontinuities, and foundation flexibility effects. Across the deck-to-deck and deck-to-bent connections, the following features need to be considered: stretching of cable restrainers in the longitudinal direction which tie the deck to the bent cap; and impacting between adjacent spans. The analyses were carried out using the nonlinear time-history dynamic analysis program, IAI-NEABS. Bent columns and cap members are modeled using beam elements. For concrete column members, an effective moment of inertia equal to 40% of the gross-section moment of inertia was used to account for the nonlinear behavior. The composite deck (reinforced concrete deck supported on five steel girders) was modeled using a series of beam elements in a line. At the end of each span, a series of zero-length hinge subelements were used to model the multiple bearing supports and the effect of transverse diaphragm. Each girder bearing is idealized individually as a zero-length hinge element with proper translational stiffness coefficients. At the end of the span, the upper ends of the five subelements are connected to the centroid of the deck superstructure through the master-slave transformation. (This is an approximation typically used in modeling the two-dimensional deck geometry with a single line element and to preserve the primary end restraint, i.e., moment release along the direction of the bent.). If the transverse diaphragms and the composite deck form a rigid constraint, the entire deck will act as a unit and tend to rotate about an axis parallel to the bent. For this case, the above approximation is correct. At the other extreme, assuming no effect of diaphragms, individual girders will tend to bend about an axis normal to the bridge centerline and the concrete deck will experience two-way bending. In reality, the actual behavior is probably between these two extreme cases. For simplicity in practical modeling needs, the rigid-link assumption is usually adopted. Raleigh damping magnitude was used to achieve 5% damping ratio for the principal vibration modes of the bridge.

In seismic response analysis, the important structural responses are typically in the column. The adequacy of the above modeling assumption can be verified by the response measurement at the bent cap. Based on soil boring data, the following foundation stiffness coefficients for bent footing and abutments were calculated by the soil engineer assisting the present study: For footing, the coefficients are 1.84×10^5 k/ft, 1.40×10^5 k/ft, 1.46×10^5 k/ft, 4.95×10^6 k/ft/rad, 2.29×10^6 k/ft/rad, in the order of vertical, tangential to bent, normal to bent, strong axis rotational, and weak axis rotational directions, respectively. Similarly, for abutment stiffness, they are 1.70×10^6 k/ft, 1.26×10^5 k/ft, 1.4×10^5 k/ft, 5.01×10^7 k/ft/rad, and 1.92×10^6 k/ft/rad. In a previous study [4], it was concluded that in order to match the measured response due to the 1979 Coyote earthquake, the abutment longitudinal stiffness has to be reduced to 3000 kip/ft. We will reexamine this assumption. In the nonlinear model, cable restrainers are modeled as tension-only member (with or without initial cable slack) and the impact between adjacent deck are monitored at the two edges of the deck across the bent cap using gapped and compression-only stiffness properties. A seat gap of 0.3 inch was used.

CORRELATIVE STUDY (COYOTE LAKE EARTHQUAKE)

Linear dynamic analyses by the direct integration method were conducted for this event. The three-component records obtained at the base of Bent 5 (Channels 1, 2, & 3) were used as the uniform support excitation. The spatial variation of ground motion was shown to be small.

Model I. - Initially, rocker bearings at each bent are modeled as roller supports. Horizontal and vertical responses are decoupled. The resulting response correlations were not very good. The FAS of the calculated and measured total acceleration response at the top of Bent 5 is shown in Fig. 9 for the normal direction. As shown in the figure, it overestimated the normal response at 2 Hz, but did not capture the measured frequency at 3 Hz. In addition, the tangential response of the bent at 5 to 6 Hz were not predicted well by this model.

Model II. - Based on measured responses at deck and top of Bent 5, there was no significant difference (Fig. 10). This is an indication that the rocker bearings were "locked." The model was modified to reflect this observation, i.e., each span now is pinned at both ends to the supporting bents. To soften now this continuous system, the abutment stiffness in the direction normal to the bent is reduced to 14,000 kip/ft (from 140,000 kip/ft) which is still much higher than the value used by Wilson [4]. The modal frequencies for modes involving bent 5 are 3 Hz for the normal response and 6 Hz for the tangential response. Comparisons for time history and FAS of total accelerations are shown in Fig. 11 for channels 4, 8, 5, 6 and 9. Very close correlation was obtained at top of the bent for both normal and tangential responses. Both the frequency content and the amplitude are predicted very well. This is a strong indication that the modeling assumptions adopted for the deck superstructure and the skewed support are sufficiently accurate. Furthermore, during this earthquake, there was no evidence of impacting between adjacent decks. Based on the longitudinal shear and vertical reaction calculated at each girder rocker bearing, an equivalent friction of 0.4 can be established which is the resistance required to prevent rocker "rocking".

CORRELATIVE STUDY (LOMA PRIETA EARTHQUAKE)

As discussed earlier, during the Loma Prieta earthquake, there was high acceleration (0.94g) recorded on the deck (uncorrected data) which suggested impacting between adjacent spans might have occurred. Also, after the seismic retrofit, the effect of cable restrainers should be included in the model. To accommodate these aspects, the "locked" rockers on top of each bent were released and modeled with nonlinear elements which include Coulomb friction, cable restrainer (tension only with no slack), and impact. Based on measured responses at the bent, there is an apparent shift in the frequency content of the bent vibration in the normal direction (see Fig. 10). The frequency is now increased to 4 to 5 Hz. Similar observation was made for the Morgan Hill Earthquake. The tangential response of the bent remains essentially the same. Several factors could contribute to this: (1) compaction of bent foundation material; (2) effect of cable restrainers; and (3) impacting of adjacent spans. To assess the effects, nonlinear analyses were carried out.

The impacting between adjacent spans produces very high frequency oscillation of acceleration (up to 30 to 60 Hz) at the point of direct contact. Because of the skewed geometry, impacting occurred at one edge of the deck. The structural elements cannot transmit such high frequency motion. At the point of direct impact, the acceleration response has a very high amplitude and high-frequency oscillation. During earlier part of the time history, impacting also occurred. However, it occurred during the subdivision of the regular time step but bounced back at the end of the regular step. The effect of such impact was accounted for in the analysis. Fig. 12 showed the forces in the cable restrainers. At about 3 seconds, cable restrainers began engaged. However, the forces are generally low which is consistent with the excitation level. At Bents 5 and 6, there is slight rotation of the deck which may be due to the edge-impact. The measured and calculated acceleration response at top of Bent 5 are shown in Fig. 13 for both normal and tangential directions. The essential features of the response were captured in the analysis. The correlation of calculated responses with measured responses during the Loma Prieta Earthquake was not as good as that obtained for the Coyote Lake Earthquake. This is due to the complex phenomenon of friction, cable stretching, and impacting.

CONCLUSIONS

Seismic response records of the study bridge subjected to the past three earthquakes were analyzed with an emphasis on the relative displacement characteristics of the bridge. The analyses indicate complex nature of the problem. Theoretical models were also developed and the responses of the analytical models were correlated with the recorded responses. Adjustments of structural parameters and modeling concept to achieve satisfactory correlations were discussed. The results indicate the need for further study on the modeling of bridge with cable restrainers subjected to pounding and pulling.

ACKNOWLEDGEMENTS

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Table 1. Peak Accelerations Recorded at the Bent Supports of San Juan Bautista 101/156 Separation Bridge.

		Prior to Seismic Retrofit	After Seismic Retrofit with Cable Restrainers	
Channel		Coyote Lake 1979	Morgran Hill 1984	Loma Prieta 1989
1 N 2 Up 3 E	Bent 5, Base of Column	.12g .05g .08g	.07g .03g .04g	.15g .10g .14g
10 N 11 Up 12 E	Bent 3, Base of Column	.12g .06g .11g	-- .03g .04g	.12g .08g .14g

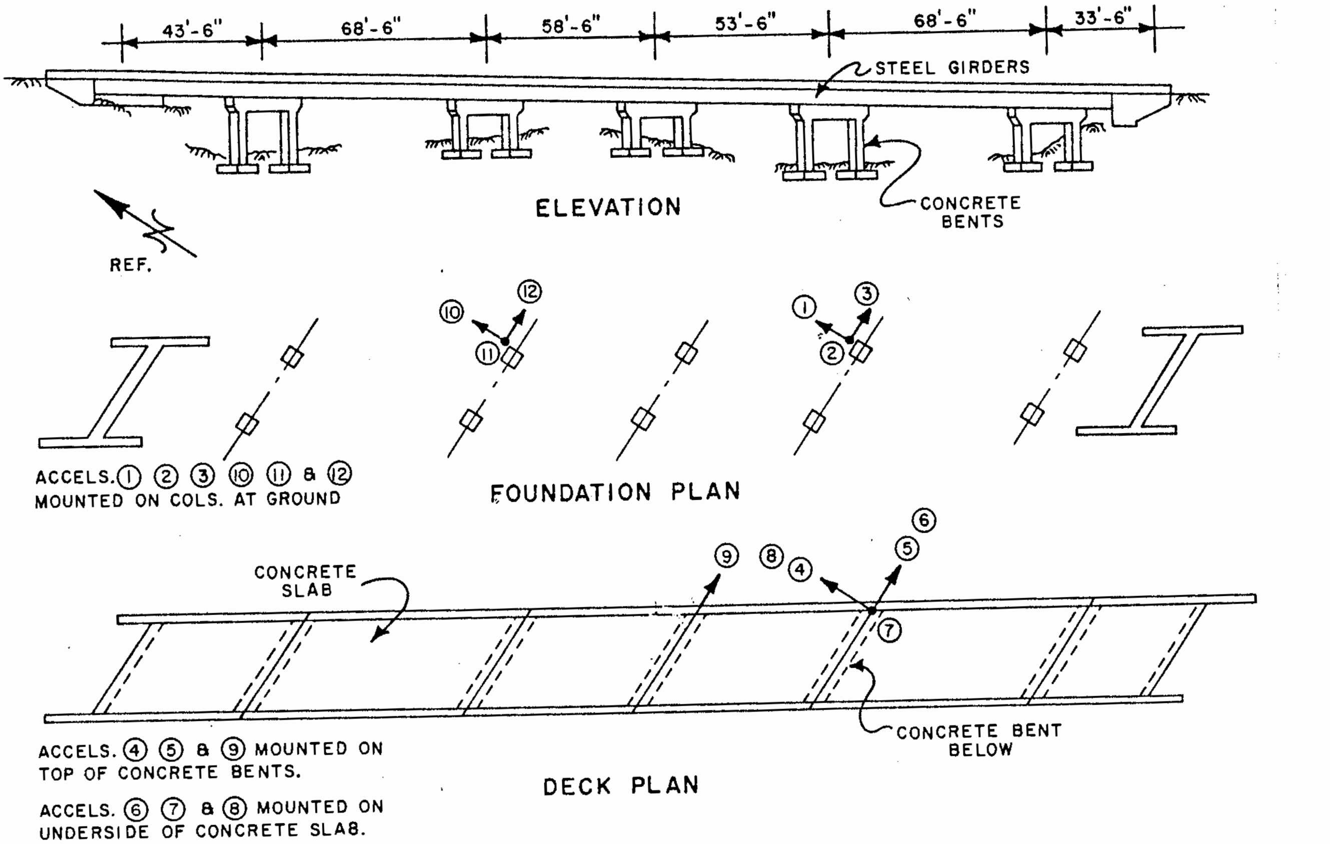


Fig. 1 Location and Orientation of Recording Channels at 101/156 Highway Separation Bridge.

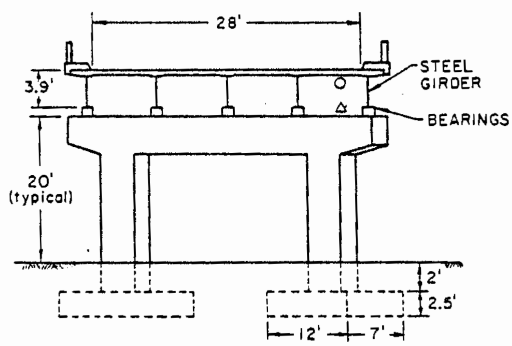
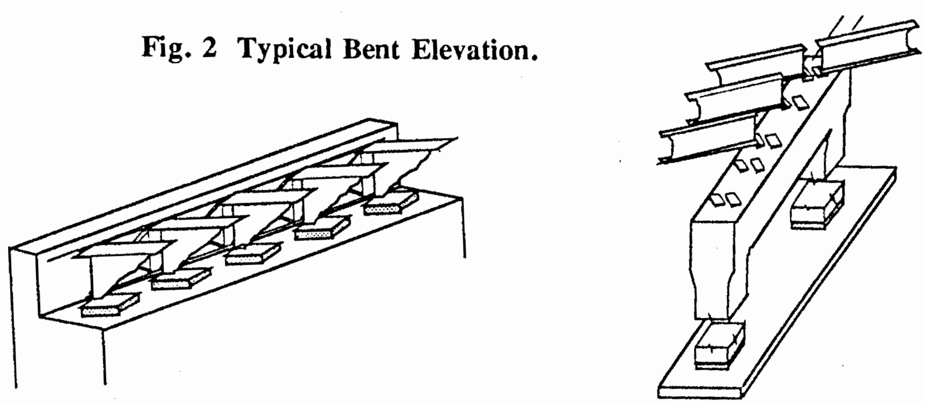


Fig. 2 Typical Bent Elevation.



a) Abutment

b) Typical Bent

Fig. 3 Skewed Bearing Supports.

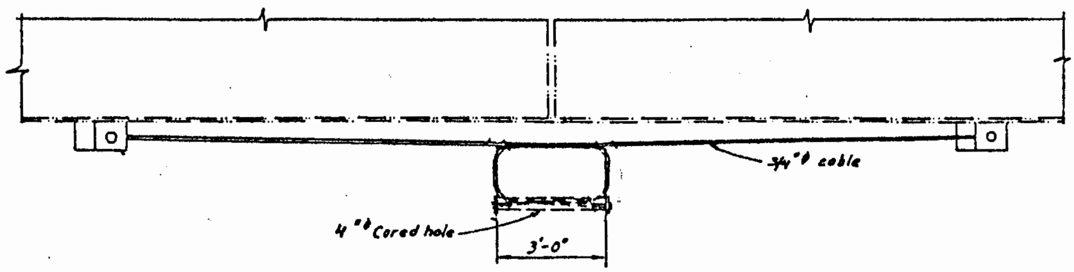
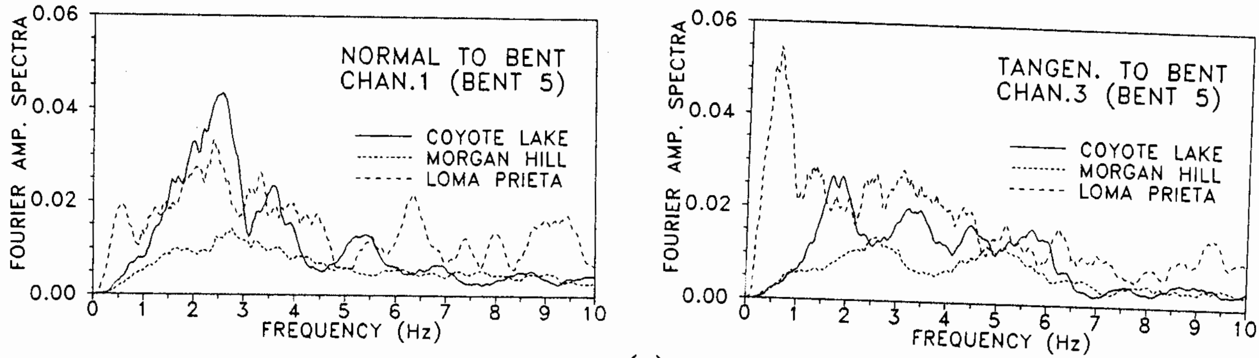
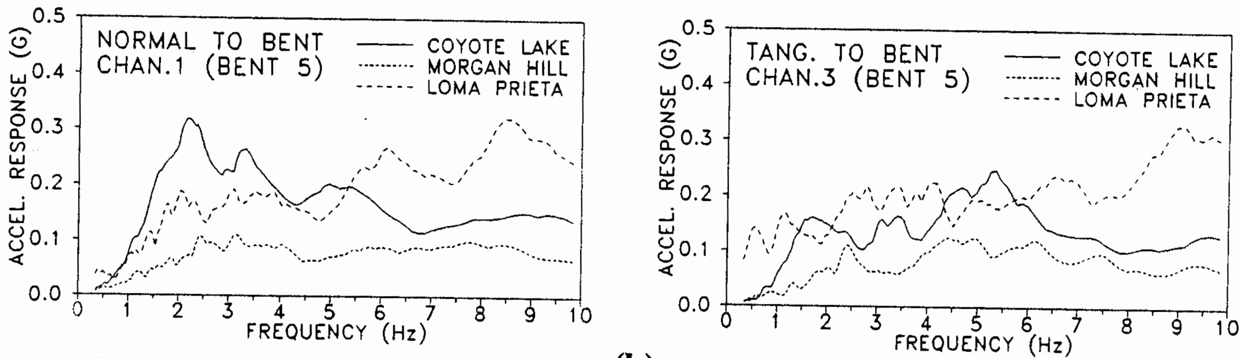


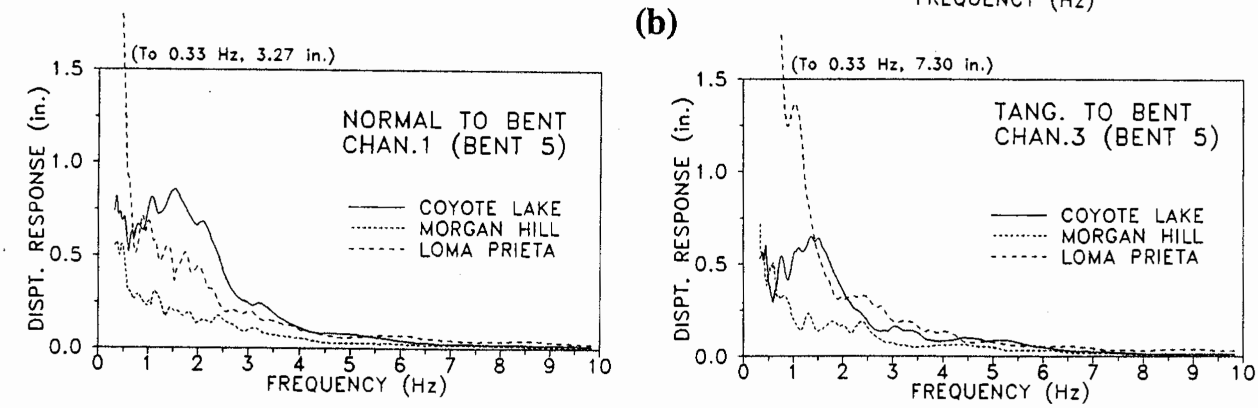
Fig. 4 Elevation View of Longitudinal Cable Restrainer.



(a)



(b)



(c)

Fig. 5 Support Acceleration Frequency Characteristics : Calculated Magnitude of (a) Fourier Amplitude Spectra, (b) Acceleration Response Spectra, and (c) Displacement Response Spectra.

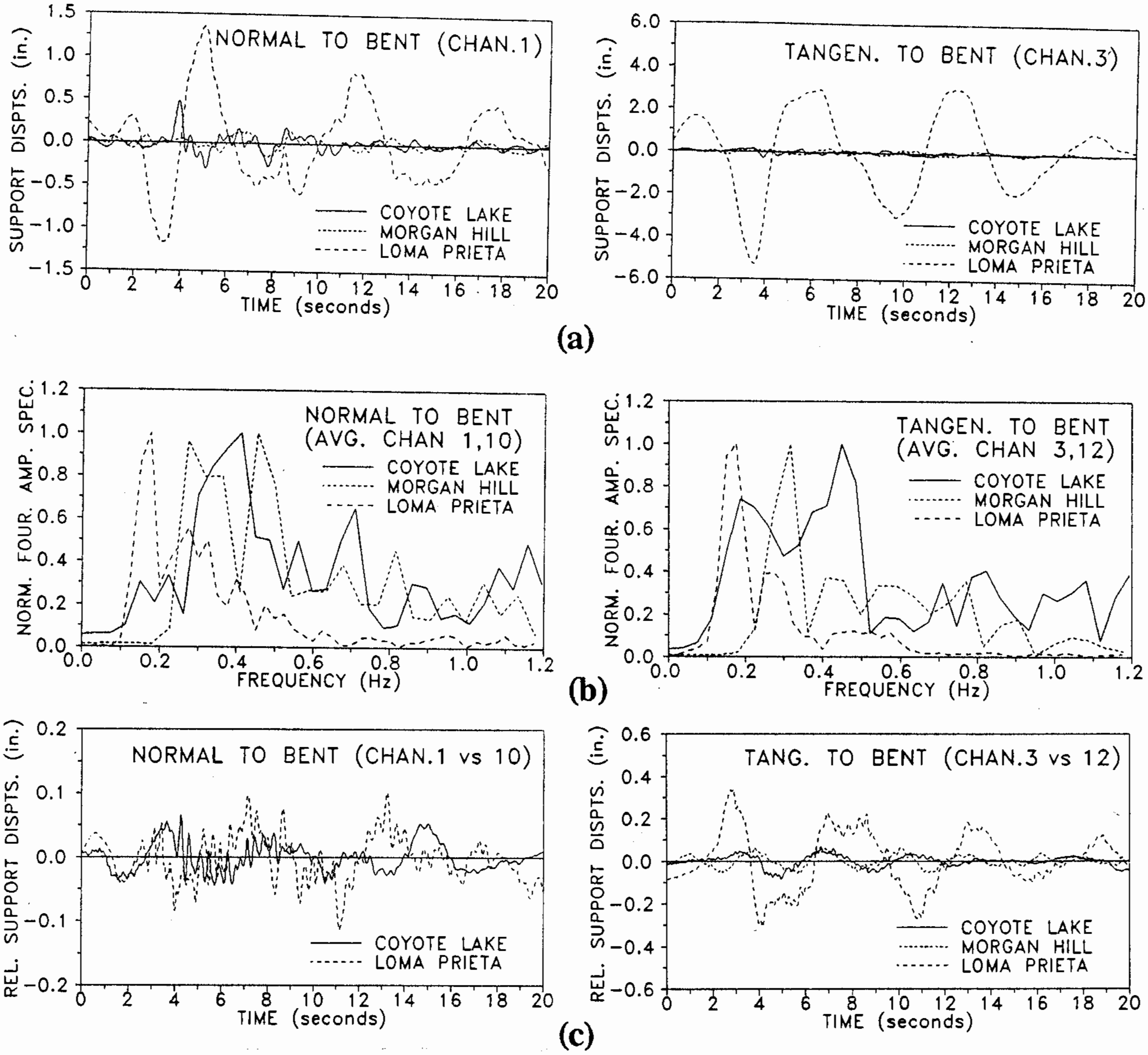


Fig. 6 Support Displacement Characteristics : (a) Support Displacement Histories, (b) Normalized Fourier Amplitude Spectra of Support Displacement, and (c) Relative Support Displacements Histories.

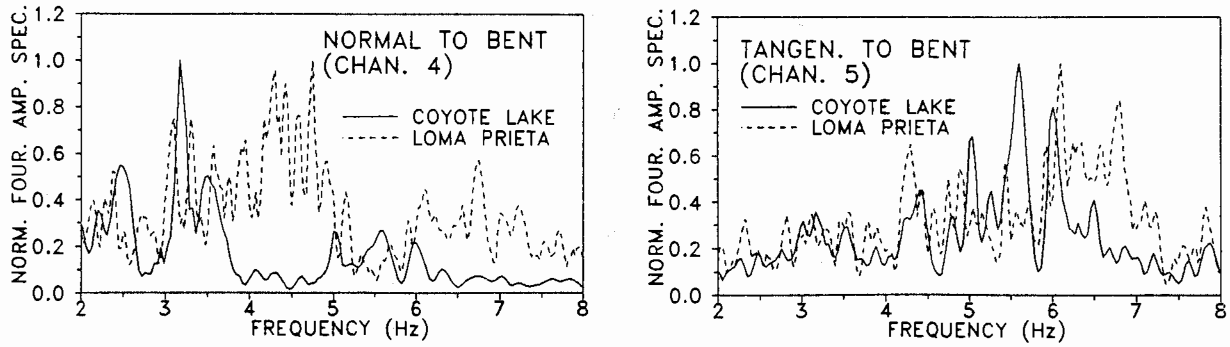


Fig. 7 Comparison of Structure Frequency Characteristics Before and After Retrofit.

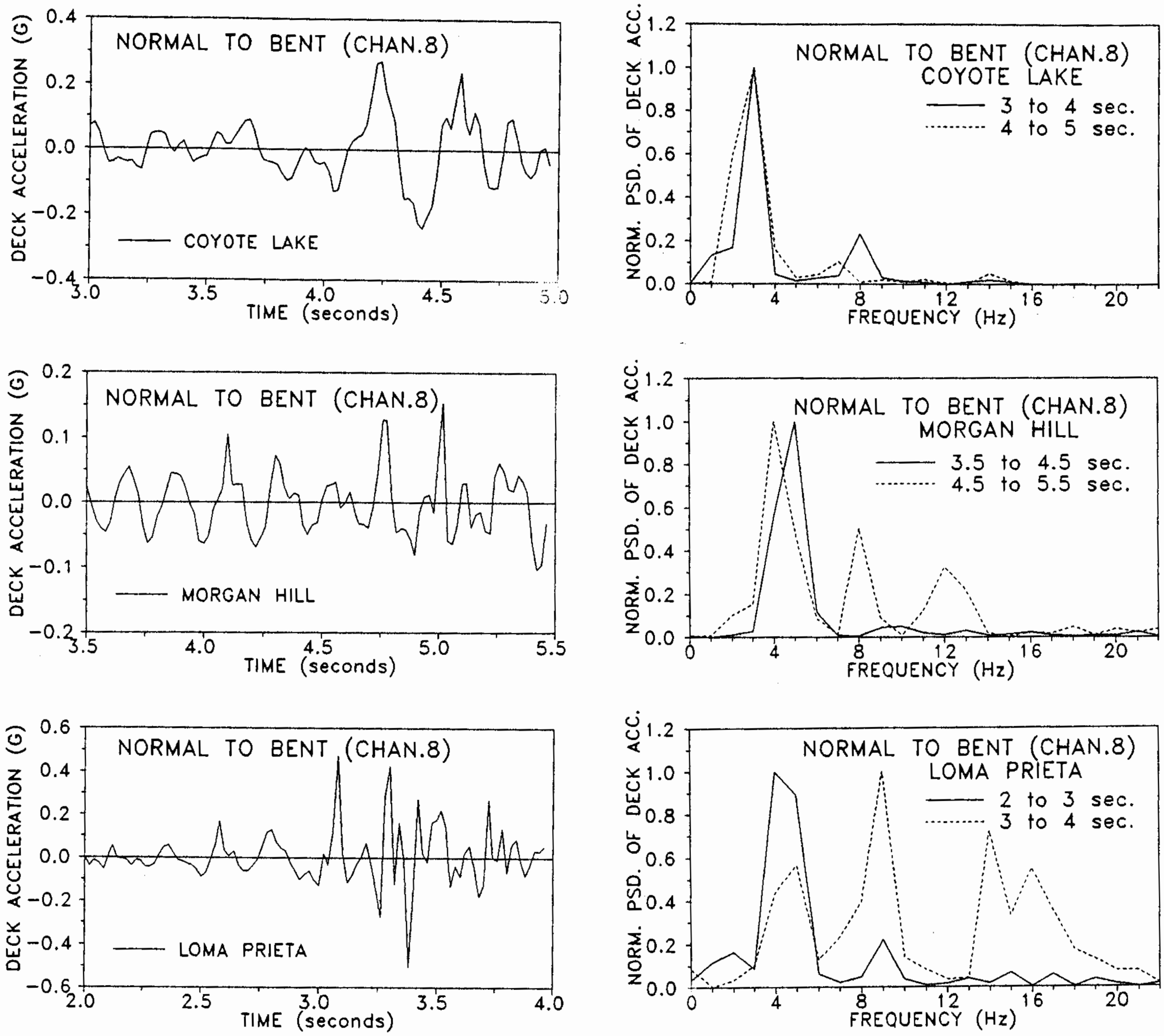


Fig. 8 Changes in Frequency Contents of Study Bridge During Earthquake.

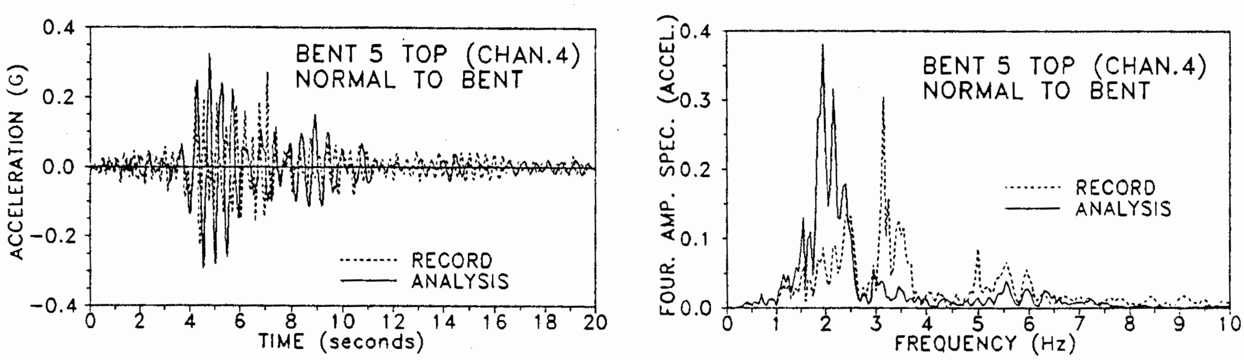


Fig. 9 Recorded and Calculated Acceleration Response (Model I) for Coyote Lake Earthquake.

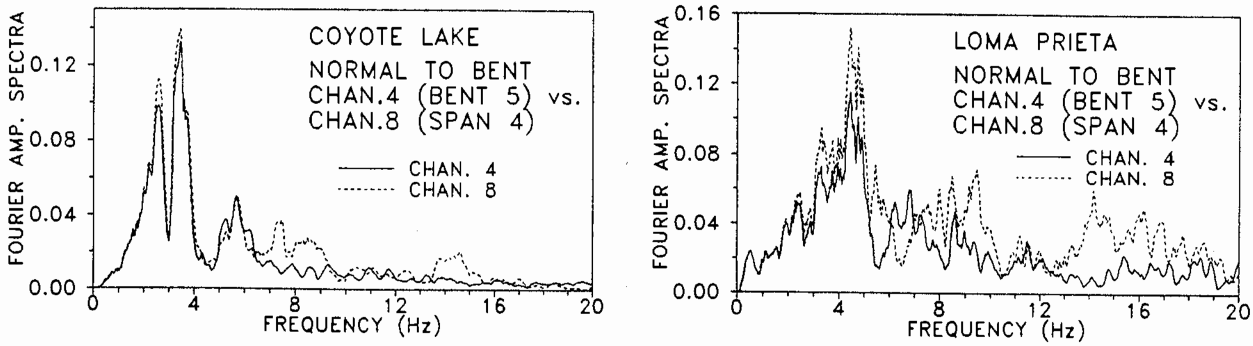


Fig. 10 Frequency Content Recorded for Coyote Lake Earthquake and Loma Prieta Earthquake.

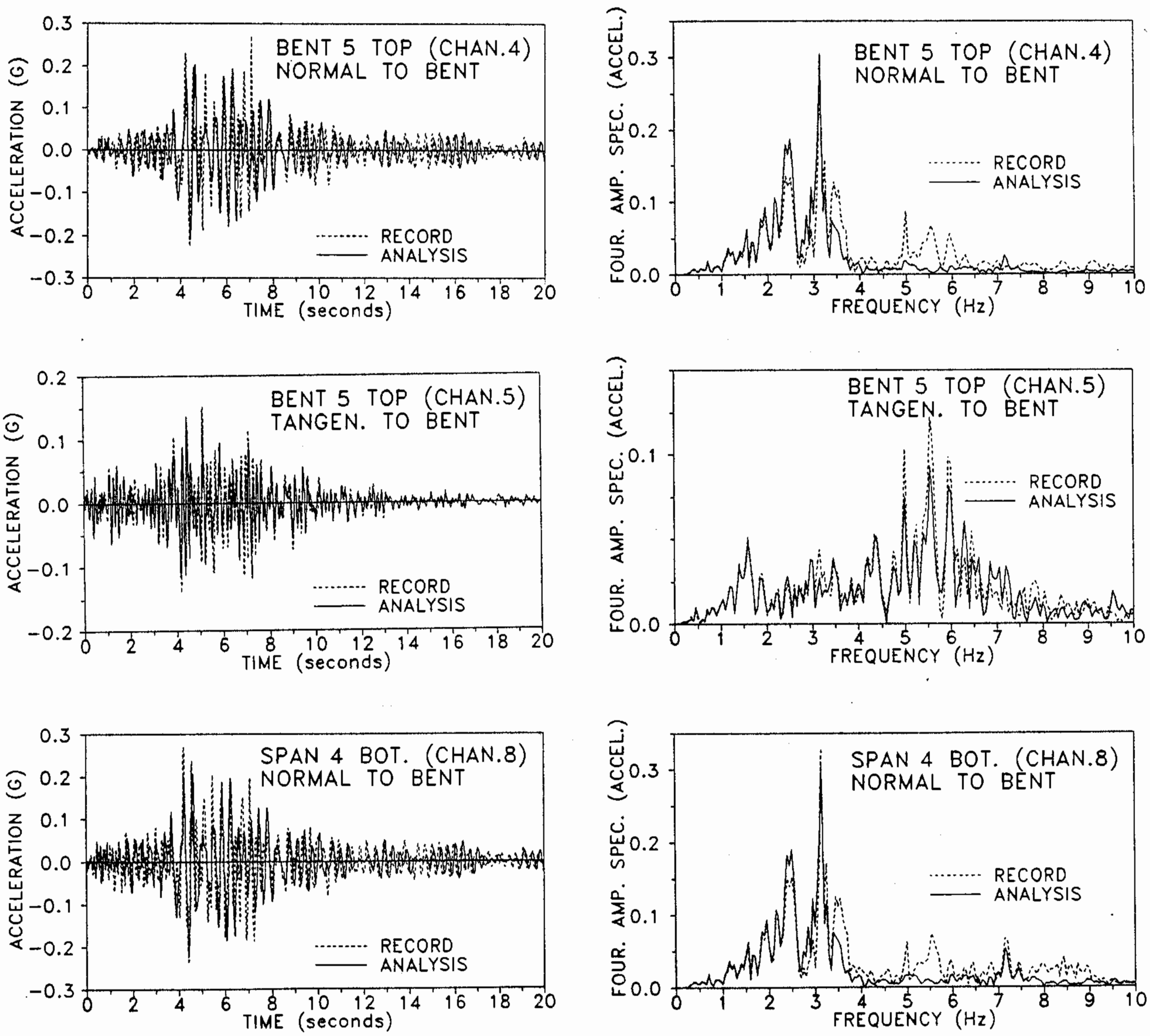
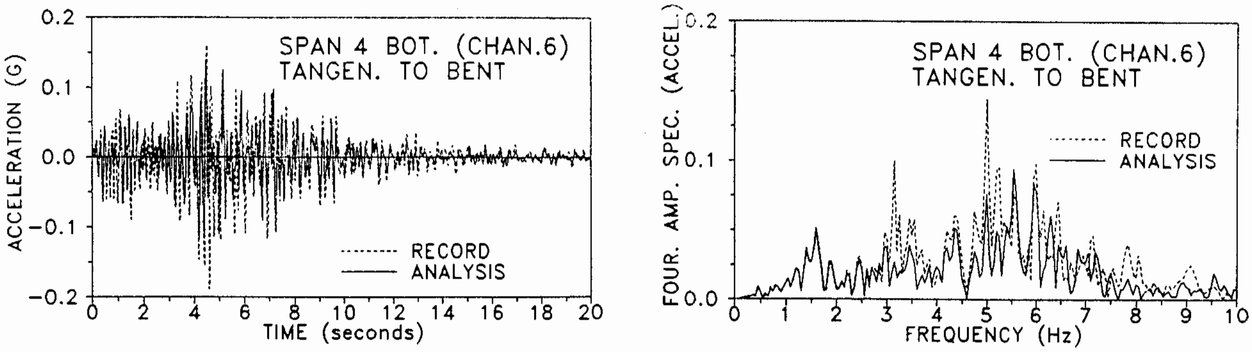


Fig. 11 Recorded and Calculated Acceleration Response (Model II) for Coyote Lake Earthquake.



(Fig. 11 Continued)

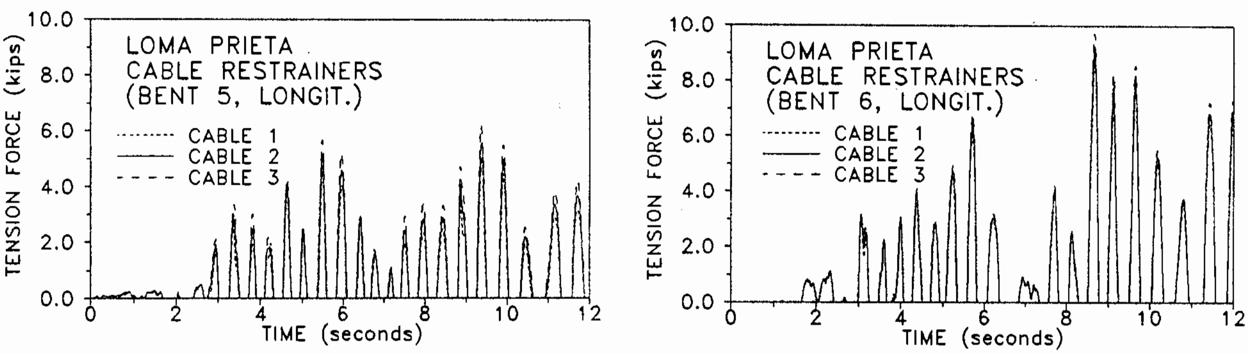


Fig. 12 Analytically Obtained Tension Force Histories of Cable Restrainers.

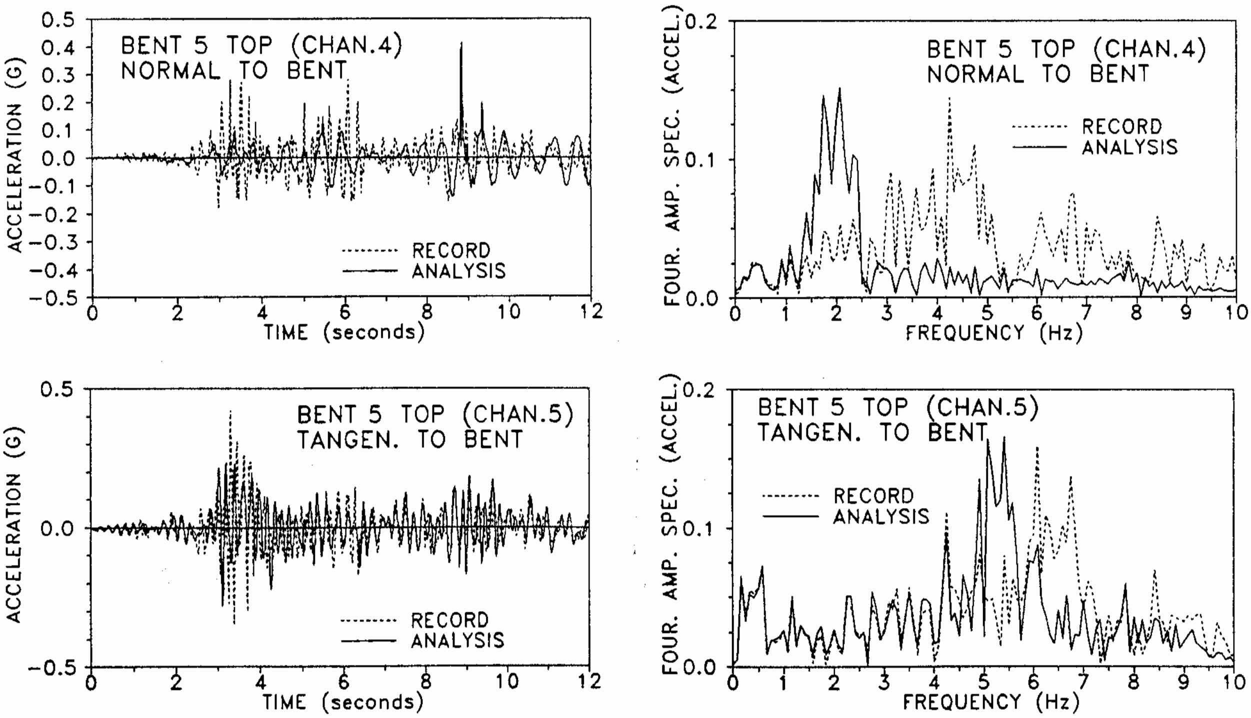


Fig. 13 Recorded and Calculated Acceleration Response for Loma Prieta Earthquake.