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RESPONSE OF BRIDGE HINGE RESTRAINERS DURING EARTHQUAKES--FIELD PERFORMANCE, ANALYSIS, AND DESIGN

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ranged from three to eleven spans. They had different number of hinges and different substructure characteristics. The earthquake intensity also varied considerably from one bridge to another. Two groups of earthquake analyses were carried out: in one analysis the input acceleration records collected at sites near the bridges were used, and in the other a series of parametric studies with larger peak ground acceleration (PGA) was conducted. The field investigations and the analyses showed that the Loma Prieta earthquake activated the hinge restrainers in the majority of the bridges investigated in this study. Except for a few instances, the restrainers and their supporting systems performed well. It was also noted that bridges with a small ratio of number of hinges to the number of spans and in which the substructure is relatively stiff, are less likely to be susceptible to support loss.

The evaluation of the current Caltrans restrainer design method consisted of two parts: (1) a study of the effect of refinement in the current methods, and (2) a large number of nonlinear analyses of the Caltrans example bridge for different earthquakes, hinge gaps, and the number of restrainers. Based on these studies, a new method for the computation of the relative hinge movement was proposed, and demonstrated for one of the four bridges which had been the subject of detailed nonlinear analyses. It was found that the current Caltrans method for restrainer design leads to a conservative and safe design in terms of the number of restrainers. However, the degree of conservatism for different hinges is not uniform. It was also determined that a more refined method to compute relative hinge displacements can lead to fewer restrainers even in hinges with a nominal seat width of 6 in. The refined method would explicitly incorporate the nonlinearity of soil at the footings and abutments, plastic hinging of the columns, and the nonlinearity of the hinges.

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Chapter 1
INTRODUCTION

1.1 INTRODUCTION

One of the lessons learned from the 1971 San Fernando earthquake in southern California was that highway bridges with narrow seat width at hinges may be susceptible to collapse due to movements which are beyond the available seat width [1]. Following the earthquake, the California Department of Transportation (Caltrans) identified 1250 bridges (of its inventory of approximately 13,000 structures) as having vulnerable hinges [2]. To prevent excessive movements at the hinges, these bridges were retrofitted with steel hinge restrainers under Caltrans' phase I retrofit program. This phase was completed in 1985 at a cost of $55,000,000 [3]. The 1989 Loma Prieta earthquake in northern California [4] provided an opportunity to study the effects of restrainers on the response of bridges. Accordingly, a research contract was awarded by Caltrans to the University of Nevada, Reno, under the direction of the first two authors of this report to study different aspects of hinge restrainer performance and to review the restrainer design method.

This report presents a summary of the important findings. Detailed information about different parts of the study may be found in five other reports [15-19].

1.2 REVIEW OF PREVIOUS WORK

Prior to the Loma Prieta earthquake, very little research had been done on hinge restrainers. This was, in part, due to the fact that restrainers are relatively simple systems with seemingly predictable behavior. Other than in-house tests conducted at Caltrans, the only reported experimental study on restrainers was carried by Selna, et. al. under a project funded by Caltrans [5]. The primary issues considered in this study were (1) the transfer of loads from restrainers to reinforced concrete box girder superstructures and (2) the ability of restrainers to yield. The study included cyclic tests of full-scale hinge specimens and the related analytical modeling. A recommendation to reduce the number of restrainer cables resulted from this study [6].

Restrainer elements have been incorporated in analytical studies since 1973 with the introduction of computer program NEABS [7], and its expanded versions [8, 9]. The study in Ref. 6 confirmed that restrainers are essential at the hinges of highly curved bridges with narrow seats and that short tie bars are inadequate. One of the bridges included in this reference had four hinges while the other had one. The restrainers in the structure with one hinge experienced smaller stresses than the other. Imbsen, et. al. analyzed three curved bridges using NEABS [10] and found that restrainer stresses are relatively small when there is only one hinge in the superstructure. The studies in Ref. 7 and 10 were focused on curved bridges. Imbsen in Ref. 23 found that the short hinge support in combination with a large skew contributed to the collapse of a span in 1980. An analytical study of a model bridge in Ref. 24 concluded that equivalent linear models, while may be adequate for cases with relatively small hinge movements, do not accurately predict the response when hinge closures occur during an earthquake [24]. Imbsen and Penzien arrived at a similar conclusion, but noted that even under low seismic excitations segments of a bridge may collide at the hinges and result in poor correlation between a linear and nonlinear analysis [25].

The response of three bridges in the 1976 Guatemala earthquake indicated the possible effectiveness of restrainers in maintaining the integrity of bridges [11]. During the earthquake, three spans in a five-span bridge with no restrainers fell off the supports, whereas two other bridges, one a railroad bridge and the other a bridge with restrainers at its hinge and abutments survived the earthquake with minor damage. The rails in the railroad bridge appeared to have acted as restrainers. Considering the fact that many parameters such as structural and soil properties and the input ground motion can affect the hinge displacements, general observations of this nature may be considered tentative at best.

A more recent study of a two-span bridge which was damaged by the 1986 earthquake in Palm Spring, California indicated the sensitivity of restrainer stresses to the gap which is left at the ends of the restrainers to accommodate thermal movements of the superstructure [12]. Singh and Fenves [13] found that the amount of restrainer area designed based on current methods may be inadequate when non-coherent input ground motions are considered.

Although the studies mentioned above shed some light on some aspects of the design and the effectiveness of restrainers, no opportunities had developed for a comprehensive review of restrainer performance that would incorporate field investigations and detailed analytical studies of the restrainer response prior to the Loma Prieta earthquake.

1.3 OBJECT AND SCOPE

The primary objectives of the study presented in this report were: (1) to review the actual performance of bridge hinge restrainers during the 1989 Loma Prieta earthquake, (2) to simulate the earthquake on the analytical models of several selected bridges and study their responses, (3) to carry out a parametric study of these bridges to determine the effect of stronger earthquakes and the effect of changes in the restrainer gaps, and (4) to review the restrainer design procedure and identify refinements if needed.

The project started in August 1990, by which time the damaged bridges had been visited by the Caltrans maintenance staff and the majority of these bridges had been or were being repaired. Therefore, the primary source to evaluate the field performance was the damage reports compiled by the Caltrans Maintenance Division. Three bridges which had not been repaired were visited by the authors and a review of their response was done. The particular emphasis in the field visits was on the restrainer performance and its effects on the adjacent superstructure segments. Both cable restrainers and high-strength rods were included.

The selection of the bridges for analytical studies was based on the damage they experienced as well as variety in the number spans, number of hinges, substructure geometry, and skew angles. To allow for an in-depth study of the effect of restrainers in these bridges, no attempt was made to include highly curved or multiple simple span bridges. None of the bridges which were found suitable for detailed analysis were instrumented. Therefore, the actual ground motions at the site of these bridges had to be estimated based on nearby measured free field data to accomplish the second objective. Considering the uncertainties in the input earthquake record and the sensitivity of the restrainer stresses to the restrainer gap, the peak ground acceleration and the restrainer gap values were varied to determine the effects on the restrainer response. These analyses were the focus of the third objective. The review of the restrainer design approach was concentrated on the Caltrans method. This method implicitly approximates the stiffness nonlinearity of the piers and hinges. The effects of minor refinements in the method on the number of restrainers were studied. An extensive parametric study of the

Caltrans design example was carried out and important parameters were identified. An exploratory study was also carried out to design restrainers by including the nonlinearity of bridge components explicitly in a more accurate way than what is currently modeled (Sec. 5.4).

Chapter 2
FIELD INVESTIGATIONS

2.1 INTRODUCTION

Based on the Caltrans Maintenance Division records, 23 bridges which had hinge restrainers were damaged by the 1989 Loma Prieta earthquake. The level of damage was generally minor. The repair and retrofit program on some of these bridges started nearly immediately after the earthquake. The project presented in this report started many months after the earthquake, by which time many of the bridges had been repaired or were in the process of being repaired. It was not the intent of this project to conduct field survey of damage. Rather, an assessment of the performance of the bridges during the earthquake was made based on the damage reports prepared by the Maintenance Division. In three instances, the principal investigators were informed by the Caltrans project monitor that some useful information may be obtained by visiting specific bridges. The structures that were investigated in the field were the Central Viaduct, the Route 580/24/980 Separation, and the Route 92/101 Separation. This chapter presents a summary of damage reports on the 23 bridges which suffered, at least, some degree of damage and discusses the results of the field review of the three bridges which were visited.

2.2 DAMAGED BRIDGES

2.2.a. - Geometric Parameters

The Caltrans Maintenance Division identified 23 bridges which were retrofitted with restrainers and were damaged by the Loma Prieta earthquake. General structural data about these bridges were compiled by the research team based on the bridge plans at Caltrans. A summary of the main features of each bridge is shown in Table 2.1. The primary goal of developing the data base in this table was to develop a global view of the damaged bridges and to identify any clear patterns. Another objective of the data base was to assist Caltrans and the researchers select bridges for detailed analyses. Data for extremely long, multiple-span structures such as the I-880 freeway in Oakland which were the subject of other detailed studies were not generally included in the table.

It can be seen in Table 2.1 that there was considerable variation in all the major geometric and structural parameters among the damaged bridges. As expected, the skew angle in long, multiple-span bridges varied considerably even within each structure. In other bridges, the skew ranged from zero to 40 degrees. The number of spans was also highly variable starting with 3 spans in Madrone Dr. Undercrossing to one hundred or more in some of the viaducts. The maximum span length, overall length, and the overall widths were also drastically different in the damaged bridges. One common feature among all the bridges was that they were all built in or prior to 1971. Some were built as early as the 1940's. The fact that none of the structures had been constructed after the 1971 San Fernando earthquake in southern California is very significant in that it reflects the rapid responsiveness of Caltrans in correcting the weaknesses observed during that earthquake. Of primary relevance to the study summarized in this report was the hinge behavior. The number of hinges cannot be directly obtained from the bridge inventory. The number of hinges in bridges for which the complete plans were reviewed are shown in Table 2.1. It can be observed that the number of hinges was also a highly variable parameter among the damaged bridges.

Because all the major parameters for the damaged bridges are highly variable, no clear pattern can be identified among these bridges except for the year of construction which, in fact, reflects the bridge design provisions of the time.

2.2.b. General Damage

Table 2.2 list the types of damage observed in the bridges discussed in the previous section. A more detailed review of hinge damage is presented in the next section. Note that the most common damage was excessive hinge opening and cracking of columns. Large hinge movements caused cracking of the abutments in some but not all cases. The lack of reported damage in footings is, in part, attributed to the lack of inspections of the column bases. Typically footing damage during earthquakes is not well documented due to a lack of easy access.

In reviewing Table 2.2 in conjunction with Table 2.1, no clear correlation can be made in terms of the effect of number of spans on damage. For example, the damage in bridges with

11 spans or less can vary from excessive hinge opening to abutment cracking and column damage. The lack of pattern in the damage may be attributed to the fact that the intensity of the ground motion varied considerably among the listed bridges.

2.2.c. Hinge Damage

A summary of damage caused by the Loma Prieta earthquake at or near the hinges in these bridges is listed in Table 2.3. The data were obtained from the damage reports prepared by the Caltrans Maintenance Division soon after the earthquake. Note that the damage in the North Connector, E580, W580/E24 structure (Number 14 in Table 2.3) initially appeared to be due to pounding. This bridge was one of the structures which was investigated in the field by the research team. It will be shown in subsequent sections that what was believed to be the pounding damage was indeed due to the failure of the diaphragm. Another important point to consider is the fact that the damage reports are based on visible effects only. In general, no detailed inspection is conducted by the maintenance crew unless there is an obvious reason for doing so. The crew does not follow a check list. For example, restrainer connectors are not inspected for damage unless they are readily visible or there is a notable damage in the restrainer or the structure.

The data in Table 2.3 indicate that restrainers were activated in many instances, but failed only in two cases. Field observation, in general, cannot determine the extent of stresses in the restrainers if they are not damaged. The detailed analyses presented in Chapters 3 and 4 serve this purpose. The table indicates that restrainer connector damage was noted only in a few cases. Furthermore, very few diaphragms were damaged by the earthquake. Pounding at the hinges was observed in many cases which indicated relatively large movements. Associated with large hinge movements was joint seal failure that occurred in many bridges.

2.3 CENTRAL VIADUCT

2.3.a. - Brief Description of the Bridge

This bridge is the third structure listed in Tables 2.1 to 2.3. The structure is a long, multiple span system a segment of which is of double-deck reinforced concrete construction changing into a single deck composite steel/concrete construction. The reinforced concrete segment is a cast-in-place multi-cell box girder type constructed in 1957. It was retrofitted in 1972 with hinge restrainers because of its narrow hinges with nominal width of six inches. Both C-5 and C-7 [6] restrainer types had been used in different hinges.

The composite part of the bridge consists of simply-supported steel girders. Straight cables have been used to tie the girders together in the longitudinal direction of the bridge The cables pass through a one-foot galvanized pipes which are placed through the web plates in the pier caps.

2.3.b. - Earthquake Damage

The Loma Prieta earthquake caused some visible cracking of the piers and pier cap column joints in the double deck segment of the bridge. As a result, the bridge was closed to traffic and temporary shores were placed to support the damaged area. Because substructure damage could indicate large hinge movements, the hinges of the bridge were inspected. No damage had been reported in the segment of the bridge with composite construction, and the bridge was open to traffic at the time of visit.

2.3.c. - Observations

The field investigation took place on June 10, 1991. Some of the details of the field visit may be found in App. A in Ref. 14. The inspection covered the area of the bridge between the Golden Gate Ave. and Linden St. In addition, the segment between the Mission St. and South Van Ness Ave. were visited. The hinges were inspected from the top of the upper deck to determine any significant hinge rotation or opening. The hinge gaps at the east and west edges of the superstructure were measured. No significant rotation was noted except for Hinge No. 14 which is located between Fulton and Ash St. in which a differential gap of 1/2 inch was noted between the east and the west edges. This hinge and Hinge no. 16 located between McAllister St. and Golden Gate Ave. were inspected from inside the superstructure cells. The restrainer type in Hinge 14 was C-7 and the one in Hinge 16 was C-5. None of the restrainers showed any sign of damage. The uppermost cable in the east cell of Hinge 14 had left some marks on the corners of the C-drum, indicating that the cables experienced significant tension. It appeared that, as a result of tensioning and detensioning, the cables had shifted downward by approximately 1/2 inch. Because the vertical space on the drum is limited, the shift pushed the second cable from the bottom out. Figure 2.1 shows this effect more clearly. The available vertical space for restrainer cables is 5.7 inches. The total depth needed for seven cables to be stacked is 5.3 inches assuming no space in between the cables. Tensioning and detensioning of the cables can move the cables and, in the absence of vertical space, can push some of the restrainers out of vertical line and away from the drum. The effectiveness of these cables in developing their full yield strength may be questionable because of the friction that cables introduce on each other. No test data are available to quantify the effect of this friction on C type restrainers.

The second segment of the viaduct had been retrofitted with straight cables at the piers. Except for two buckled cables, all the restrainers appeared to be undamaged. The two buckled cables had no cable clips. This could have increased the unsupported length of the cables and led to buckling. Another possible cause of cable buckling could have been slippage at the swedged anchors. This could not be checked during the field investigation. Considering that restrainers are intended to act only in tension, the buckled cables do not seem to present a weakness unless the problem is caused at the restrainer anchorage.

In summary, the restrainers in the Central Viaduct did not appear to be damaged by the 1989 Loma Prieta earthquake. There are indications that the cables were mobilized during the earthquake and performed as planned. Based on the observed cable movements, it is recommended that only six of the seven cables in C-7 type restrainers be considered effective because of the tightness of the space.

2.4 ROUTE 580/24/980 SEPARATION

2.4.a. - Brief Description of the Bridge

The structure was constructed in 1970 and retrofitted with 1-¼ inch diameter high-strength rod hinge restrainers in 1980. The superstructure type is cast-in-place multi-cell reinforced concrete box girder. There are many hinges in the bridge, incorporating a variety of configurations and number of restrainers. The restrainers are generally distributed around the diaphragms. A reinforced concrete bolster was provided in skewed hinges. The structure generally had tall columns, particularly in the areas which were inspected.

2.4.b. - Earthquake Damage

Significant spalling of concrete at the bottom of the superstructure has been noted at two hinges in the East and North Connector Viaducts as a result of the 1989 Loma Prieta earthquake. The damaged hinges were on the right angle to the bridge axis and had no bolsters.

The damage in the soffit in the East Connector Viaduct was evident near the west edge of Hinge No. 20. At this hinge, two sets of restrainers were installed, one consisting of 5 rods placed in one row at 1 ft. from the top of the bottom slab, and the other consisting of 6 rods placed in two rows of three. The damage occurred in the west cell, in which the rods had been placed in one row. There was a punching type failure of the diaphragm (Fig. 2.2). The horizontal crack which is in line with the center of the rods in Fig. 2.2 indicated that a secondary beam action must have followed the punching shear cracks. No damage was evident in the cell with two rows of rods. The punching shear cracks extended to the bottom slab, thus causing the concrete spall off the soffit. This was visible from outside the cell. The crack pattern at the bottom soffit is shown in Fig. 2.3.

The damage in the soffit of the North Connector Viaduct was similar to the above damage. The hinge with significant concrete spalling was hinge number 33. Four of the nine cells in this part of the structure were retrofitted with high-strength rods. These were the first, fourth, fifth, and the eighth cell from the west edge. Six rods were placed in one row in the first cell. The other cells had each six rods placed in two rows of three rods. The damage occurred only in the first cell. Again, punching shear followed by bending action was the mode of the diaphragm failure.

2.4.c. - Observations

The field investigation took place on June 10, 1991. Some of the details of the field visit may be found in App. A in Ref. 14. The cause of damage is attributed to the inadequate punching shear strength of the damaged diaphragms. The restrainers in the damaged cells were placed in one row. This was true for both Hinge 20 and 33. The associated critical punching shear section for Hinge 20 is shown in Fig. 2.4.a. Note that the failure area is a relatively small and narrow area compared to the critical section for two rows of restrainers (Fig. 2.5.a). Tests on punching shear strength of reinforced concrete slabs have shown that the shear strength drops as the aspect ratio of the loaded area increases. In addition, the perimeter of the critical section for two rows of restrainers is considerably larger. A third factor is the thickness of the critical section. It can be seen in Figs. 2.4.b and 2.5.b that when two rows of restrainers are used, the thicker part of the diaphragm is utilized. All these three factors apparently reduced the strength of the diaphragm in the damaged cells to a point below the force demand that caused the diaphragm failure.

Based on an effective depth of 12 in. and a concrete strength of 4,000 psi, the permissible punching shear strengths are estimated to be 290 and 320 kips in Hinges 20 and 33, respectively. Because of the proximity of the rods to the bottom slab and because of the fact that the bottom slab presented a weak area, the lower part of the critical section shown in Fig. 2.4 did not actually develop in the bridge. Rather, the punching shear failure lines extended to the bottom slab (Fig. 2.3). Therefore, the actual permissible strength was even lower than these values. The total yield force is 750 kips for five high-strength rods and is 900 kips for six. It can be observed that 40 percent of the restrainer yield force is sufficient to fail the diaphragms.

It should be noted that in excess of 100 sets of restrainers are used in the structure and that damage was observed only in two of the diaphragms. It is recommended that all the cells which are retrofitted by one row of rods and which are not retrofitted by a bolster be reevaluated and the diaphragms be strengthened.

2.5 ROUTE 92/101 SEPARATION, NE CONNECTOR

2.5.a. - Brief Description of the Bridge

The 92/101 Separation consists of several bridge structures comprising the overpasses and the on- and off-ramp bridges. The superstructures are all cast-in-place box girders some of which are post-tensioned. The bridge was built in 1982.

The segment which was the subject of the field inspection was an on-ramp structure for access from Highway 101 North to I-92 East on the right structure (The North-East Connector, Bridge 35-252R). The issue that triggered the visit was excessive crack openings in the soffit of the superstructure, noted by the maintenance crew after all the earthquake damage had been repaired.

The superstructure is a cast-in-place four-cell box section supported on single-column piers of varying heights. The superstructure width is 26 ft. The hinges in the structure have a nominal 2-ft. seat and are equipped with eight sets of five 3/4 in. type 2-A cable restrainers placed in all the cells.

2.5.b. - Earthquake Damage

Caltrans Maintenance Division records indicate that both the right and the left structures suffered significant damage during the earthquake. The damage ranged from spalling of superstructure concrete near several joints to excessive hinge movements. Large spalls were also noted in some of the columns. Cracks as wide as 2 inches at the ground surface were mentioned in the damage report. All the damaged elements had been repaired by the time of the field investigation of the University of Nevada research team.

In September 1991, the researchers were notified that several wide cracks have been noted at the bottom soffit near the hinge connecting the on-ramp bridge to I-92, near Bent 32-R. There was also some spalling of concrete at the railing in the hinge area which indicated possible pounding. Because there were no access holes in the box girder cells, the bridge was not visited until November 21, 1991 after access holes were cut into the two inner cells. Figure 2.6 shows the major cracks in the bottom soffit as observed looking up. The two cracks on the left side of the access holes had extended to nearly the full height of the south exterior cell. Figures 2.7 and 2.8 show the girder cracks as observed inside the second cell from the south. The crack widths are also shown. Note that the cracks were nearly parallel with slight inclination. The cracks shown in Fig. 2.6 were observed on the bottom slab inside the cell. Some of these cracks were as wide as 0.05 inch. Because the bottom of the top slab was covered by stay-in-place forms the web cracks could not be traced through the top slab. The grooved top surface of the deck prevented the tracing of the cracks on the top deck. However, the fact that typically the vertical cracks extended over the entire web could indicate that they perhaps propagated to the top slab. The hinge gap on the edge with spalled concrete was 2 in. and on the other edge was 3 in.

2.5.c. - Observations

The fact that the cracks were nearly vertical suggested that they were not caused by shear. Because the location of the cracks was close to the hinge, the moments had to be relatively small and the cracks could not have been caused by flexure alone. The damage to one of the columns and spalling of concrete at the hinge could indicate relatively large movement of the superstructure and utilization of the restrainers. The total yield force for the restrainers was 1564 kips. The direct tensile cracking strength of the superstructure is 2300 kips based on a concrete compressive strength of 4000 psi and assuming a tensile strength of $6\sqrt{f'_c}$, in which f'_c is the concrete compressive strength. The comparison of these figures suggests that it is unlikely that restrainer forces alone have caused the cracks. However, it is possible that the restrainer tensile force in the superstructure reduced the flexural strength and helped open the flexural cracks that were developed by vertical loads.

The sensitivity of the flexural strength and the cracking moment of the superstructure to axial tension was calculated. It was determined that, for a restrainer force of one-half of the yield force (a force level expected during a moderate earthquake) the ultimate positive and negative moment capacities would drop by approximately 30 and 13 percent, respectively. When the restrainers approach the yield level, the ultimate positive and negative moment capacities drop by approximately 65 and 25 percent, respectively. Although not evident in this bridge, the axial tension could also significantly reduce the shear strength of the section [14]. In prestressed concrete members, the tension will reduce the effective prestress force. These effects are not usually considered in design, but they can be critical. It can be observed that placing too many restrainers can have detrimental effects on the superstructure.

2.6 CHARACTERISTICS OF RESTRAINER RESPONSE IN THE FIELD

The damage reports summarized in Tables 2.2 and 2.3 and the observations on the above three bridges suggest that hinge restrainers need to be treated as systems consisting of three components: (i) the restrainers, (ii) the connecting hardware and the diaphragms (if any), and (iii) the superstructure adjacent to the hinge. While restrainer systems performed reasonably well during the Loma Prieta earthquake, in a few instances they pointed out the fact that each component of the system can be damaged. Because the design and construction of restrainers has gone through an evolution leading to many variations in the restrainer systems and because the intensity of the ground motion varied for different bridges, it is not generally possible to pin-point which component presents the weak link in the field.

Restrainer systems may be designed so that the weak link is at a predetermined component. Because the function of hinge restrainers is different in old bridges (those with a nominal seat width of 6 in.) and new bridges (those with a nominal seat width of 2 ft.), the restrainer design philosophy can be different for each group. In older bridges, the function of restrainers is to avoid excessive relative movement at hinges. As a result, restrainers should be designed to avoid yielding even under the maximum credible earthquake. This is already the current Caltrans restrainer design philosophy. No yielding should be allowed in the connecting hardware and the diaphragms either. Cracking of the superstructure under tension from the restrainers may be a tolerable damage. However, the effect of the tension on the moment and shear capacity of the superstructure needs to be accounted for in design.

The purpose of restrainers in new bridges is to provide overall integrity for the superstructure. Yielding of the restrainers can result in large hinge movements but, because the seats are wide, the movement may not be critical. As long as the restrainers have a reasonable level of strain hardening, restrainer yielding may be tolerable. Furthermore, the ease of restrainer replacement after the earthquake makes restrainers a good candidate for being the weak link in the restrainer systems for new bridges.

Chapter 3
ANALYTICAL STUDIES

3.1 INTRODUCTION

The properties and damage reports for the 23 bridges listed in Table 2.1 were reviewed to select several bridges for a detailed nonlinear response history analysis. This chapter presents a summary of the selection process, the analytical models, and the assumptions made in the analyses. Detailed information about the models and the modeling procedure may be found in Ref. 16-19.

3.2 SELECTION OF BRIDGES AND EARTHQUAKE RECORDS FOR ANALYSIS

It can be seen in Table 2.1 that the features of the damaged bridges varied considerably for different bridges. The only common parameter is the era of construction. Another common feature to the majority of the bridges which is not shown in the table is that they are of seat-type abutments. The bridges may be categorized into two groups. One consists of bridges with eleven or less spans and the other group includes bridges which were part of long, multi-span freeway viaducts or major multi-structure interchanges. A nonlinear response history analysis of any of the bridges in the second group was beyond the scope and size of this project. As a result, the selected structures were from the first group.

Four bridges were chosen for nonlinear analysis. The major consideration in the selection process was to insure that the structures include different number of spans, number of hinges, superstructure type, substructure type, and the ground motion intensity. The last parameter includes the effect of geotechnical characteristics of the site and the proximity to the epicenter of the earthquake. The selected bridges were: Aptos Creek, Huntington Ave. Overhead, Madrone Dr. Overcrossing, and the San Gregorio Bridges, numbered as 2, 8, 10, and 18 in the data base, respectively. The location of these bridges is marked on Fig. 3.1. All bridges were visited to verify the general features which are shown on the blue prints, and review the repaired areas. A brief description for each bridge is presented in the following sections. More details may be found in Refs. 16 to 19.

The input ground motion records used in the nonlinear analyses were based on the acceleration data recorded during the Loma Prieta earthquake by the California Strong Motion Instrumentation Program (CSMIP) [22]. Because none of the bridges were instrumented, the input motions had to be based on the available data which were recorded in the vicinity of the bridge taking into account general soil characteristics. The input earthquakes used in the analysis may be placed in two categories: (1) the "base" records which represented the peak ground acceleration that each bridge presumably experienced during the Loma Prieta earthquake, and (2) amplified records which were the measured Loma Prieta acceleration data with an increased amplitude. The base records were used to obtain the best estimate of the effects of the Loma Prieta earthquake, and the amplified records were used to determine the response under a stronger earthquake.

3.2.a. - Aptos Creek Bridge

The bridge is located approximately six miles east of Santa Cruiz on California Highway 1. It was designed in 1946 based on the 1943 AASHO (American Association of State Highway Officials) specifications for highway bridges. The bridge was constructed and opened to traffic in 1948. The bridge is a five-span, cast-in-place girder type structure, with an intermediate hinge in the third span (Figs. 3.2 and 3.3). The girder pier-cap connections are monolithic. The superstructure is 62 ft wide, and it incorporates reinforced concrete diaphragms at the abutments and the midspans. The column bases are detailed to act as one-way pins, and are supported on combined footings. The bridge has seat-type abutments with steel bearings that allow for one inch thermal movement in both the longitudinal and transverse directions.

The intermediate hinge detail is shown in Fig. 3.4. Note that the angles are bolted to each other without any slot specified for the bolt holes, thus restraining the relative movement at the hinge. The specified hinge gap is 1 in. which is filled with expansion joint material.

The abutments are supported on vertical and battered steel piles. The implied concrete compressive strength is 3,000 psi and the grade of steel is of Grade 50.

The Aptos Creek bridge was retrofitted in 1983 with cable restrainers at the intermediate and abutment hinges (Figs. 3.5 and 3.6). In addition, pipe restrainers were installed at the intermediate hinge to limit relative transverse movement.

The Caltrans damage report indicates that the curtain walls at Abutment 1 cracked during the earthquake and some concrete spalled. The superstructure moved by 1 in. to the left of Abutment 1 and sheared off all the anchor bolts at the abutment seats. No permanent relative movement at the intermediate hinge or damage to the restrainers was reported.

3.2.b. - Huntington Avenue Overhead

The Huntington Avenue Overhead is located on I-380 connecting route 101 with the Interstate 280 in San Bruno. The structure consists of two separate, parallel bridges connected with a four-foot wide slab. The right (the south) bridge was designed according to 1969 AASHO specifications and was built in 1971 (Fig. 3.7). The left bridge was designed in 1971 and was built in 1973 according to the same specifications. Both bridges were designed to carry HS20-44 truck. The concrete used for both bridges has a specified strength of 4000 psi. Grade 50 steel was used in all bridge components. Both bridges are hybrid structures consisting of both conventionally reinforced concrete box girder sections and cast-in-place post-tensioned prestressed concrete box girder sections. The right bridge has eleven spans and the left one has twelve spans. The first spans are on tangent alignment. The superelevation of the bridge varied from one place to another. The width of both superstructures varied over the length of each bridge. The height of the superstructure is 5 ft. (Fig. 3.8). Two kinds of hinges were specified in the blue prints. The first one is a an expansion joint, and the second is a construction joint connecting the precast spans with others. In the right bridge, the expansions joints are located in the fourth and the ninth spans, while in the left bridge are located in the third, the seventh, and the tenth spans (Fig. 3.7.a).

Both bridges are supported on a seat type abutment in the west end and on a diaphragm type abutment on the east end. Abutment 1 is supported on vertical and battered piles, while the diaphragm abutment is supported on vertical piles only.

The superstructures of the left bridge and the right bridge are supported on eleven and ten three-columns bents, respectively. These columns are cast monolithically with the bent caps and the pile caps. All the bridge supports are skewed (Fig. 3.7.a). The skew angles varied from 35 degrees at the beginning of the bridge to 7 degrees at the end. The columns of bent 11 of the right bridge and bent 12 of the left bridge have constant cross section dimensions.

These columns are connected to the pile caps through a two-way pin formed by four number 11 dowels (Fig. 3.9). The rest of the columns are flared from 6 feet wide at the bottom to 12 feet wide at the top. All the flared columns are pinned to the footings except for the columns of bent 4 of the right bridge and bent 5 of the left bridge. These columns are fixed to the footing and are connected to the bent cap through a one-way pin (Fig. 3.9). All the bents are supported on pile foundations.

At the west abutment (which is seat-type), each bridge is connected to the abutment through three concrete shear keys. Both bridges are supported on elastomeric bearing pads at the west abutments and at the intermediate expansion joints. Shear keys are also present at the expansion joint hinges. Precast concrete panels cover the space in between each abutment and the adjacent bent at each end. The panels are 5 feet wide and have different heights.

Horizontal earthquake restrainer units were placed at the intermediate expansion joints during construction. The restrainers consist of seven 3/4 in. cables (Fig. 3.10).

The bridge was investigated on December 20, 1989 by Caltrans engineers. According to the damage report, the bridge did not suffer any major structural damage. The report also showed that several of the precast curtain wall panels at the east abutment were broken loose of their connection at the bridge soffit. The first four panels along the face of the abutment beginning at the right corner and the first nine panels along the right side beginning at the right corner had broken connections. The bridge was reported to be in a very good condition after the earthquake.

3.2.c. - Madrone Drive Undercrossing

The Madrone Drive Undercrossing is located approximately five miles south of Los Gatos on California State Route 17. Built in 1938, the bridge is a three-span reinforced concrete T-beam structure (FIG. 3.11). The bridge has no skew. The structure was designed in 1937 using the service load method to carry AASHO H-15 live loads. Portland cement concrete used for the structure was specified to have a minimum compressive strength of 3000 psi; reinforcement was specified as Grade 50 steel.

The T-beam superstructure is enclosed by a steel mesh and gunite soffit (Fig. 3.12). The center span is continuous over the two column bents with a short (7-foot long) cantilever section at each end. Each end span consists of a simply-supported deck section resting on the cantilever end and resting on five rocker bearings at each abutment. The columns are tapered rectangular columns (Fig. 3.12) and are supported on piles.

Similar to Aptos Creek bridge (Fig. 3.4), the intermediate hinges consist of two steel angles with 1-inch diameter shear bolts. The joint is located at mid-height of the deck section, and has a gap of 0.5 inch above the joint seat and 1 inch below the seat. The hinge is designed to allow rotation about the transverse axis of the deck, but to restrain translations and rotations in other directions.

Seat-type abutments are used which consist of a concrete seat and approximately 8-foot high concrete backwall. The wing walls are 12.75-foot long and project from either side of the backwall at a 4-to-12 slope. The abutment seat is stepped to follow the superelevation of the deck. The five rocker bearings which support the end spans are mounted on concrete pedestals. An expansion joint gap of 1 inch is specified between the abutment backwall and the deck. In 1985, the Madrone Drive Undercrossing was retrofitted as part of the Caltrans phase I seismic retrofit program (Fig. 3.13). Two reinforced concrete shear pedestals were installed at each abutment to restrain movement in both the longitudinal direction and in the transverse direction. Each shear pedestal contains two 16-inch by 18-inch shear blocks, one on either side of the first interior girder; therefore, four shear blocks act to restrain movement in the longitudinal direction while only two blocks act simultaneously in the transverse direction. The shear blocks become effective after the ½-inch expansion gap has been closed. Two additional concrete pedestals were installed to act as "landing pads" in the event of a failure of the rocker bearings. Two sets of restrainer cables were installed at each intermediate hinge, on the first interior girders at each side. Each cable unit consists of a 12-foot long ¾-inch steel cable with swaged ends and a turnbuckle (Fig. 3.13).

The Madrone Drive Undercrossing is located approximately 8 miles from the epicenter of the Loma Prieta earthquake. Ground acceleration at the bridge site was estimated at 0.65-g, and was strongly transverse [3]. During the earthquake, the north abutment underwent slight rotation, as evidenced by pavement cracking at the backwall paving notch, and the rocker bearings were knocked out of plumb. All of the retrofitted end-span shear blocks were severely damaged, producing large spalls and exposing the reinforcing steel. The retrofit, however, appears to have contained the structure and prevented a catastrophic collapse of the end spans. The damage report makes no mention of any damage to the intermediate hinges or the restrainer cables. An inspection of one of the restrainers by the research team on August 23, 1991 did not show any restrainer damage.

3.2.d. - San Gregorio Creek Bridge

This bridge was built in 1940, and is located on California Highway 1 approximately 13 miles south of Half Moon Bay and close to 40 miles from the epicenter of the Loma Prieta earthquake. The structure is a five-span cast-in-place reinforced concrete bridge of T-girder construction (Fig. 3.14). The girders are of variable depth and are cast monolithically with the pier caps (Fig. 3.15). Diaphragms are built at the abutment and at hinges. Two types of bents are used (Fig. 3.16). Bent 2 consists of three tapered columns, while the other bents consist of tapered columns and a pier wall. The columns in Bent 2 are detailed to act as a one-way pin at the base. The abutments were of seat type which supported the superstructure on concrete. The cross section of the abutment is shown in Fig. 3.17.

There is a total of three intermediate hinges in the bridge (Fig. 3.14). The cross section of the hinges is shown in Fig. 3.18. The detail is similar to that in the Aptos Creek and Madrone Dr. bridges discussed in the previous sections. Again, note that the bolts prevented relative displacement at the hinges.

Under the Caltrans phase I seismic retrofit program, the hinge in the San Gregorio bridge was retrofitted in 1982 with longitudinal and transverse cable restrainers, and the abutments were strengthened by vertical cable restrainers. Figure 3.19 shows the details of the hinge restrainers. The pipe shear keys were filled with concrete. The retrofit included the installation of shear keys at the abutments (Fig. 3.20).

According to the Caltrans damage report, both abutments suffered damage. Diagonal cracks were formed in the face of the abutment walls with a crack width of 0.125 to 0.25 in. The curtain walls in Abutment 6 also cracked and there was some concrete spalling. A major concrete spall was also noted at the base of Bent 5. Neither the damage report nor the site visit of August 23, 1991 revealed any restrainer damage.

3.3 ANALYTICAL MODELS

The primary analysis of the bridges described in the previous section was performed using computer program NEABS-86 [9] which was to evaluate the performance of the restrainers. However, to develop a general feel for the overall dynamic behavior of each bridge in the linear range and the determine the first few frequencies of vibration, an eigen value analysis of the bridges was also conducted. The first two frequencies were needed to determine the Rayleigh's damping coefficients [16-19] which were used as input to NEABS-86. The eigen-value analyses were carried out using the computer program Images-3D [21]. A brief description of Images-3D and NEABS-86 is presented in this section.

3.3.a. - Computer Program Images 3-D

The program Images-3D is a three-dimensional general-purpose finite element analysis package for personal computers [21]. It can perform static, modal, and dynamic (response history) analyses. Only linear models can be analyzed by the program. The modal analysis feature was used in this project.

3.3.b. - Computer Program NEABS-86

The earthquake response history analyses were performed using NEABS-86 (*N*onlinear *E*arthquake *A*nalysis of *B*ridge *S*ystems) computer software [8, 9]. The program is a mainframe-based software for performing nonlinear dynamic analysis of bridges. NEABS-86 evaluates the dynamic response history of the bridge subjected to applied dynamic loadings or support excitations. All the NEABS analyses were conducted on a Convex super mini-frame computer.

The program uses a step-by-step integration procedure based on either a constant acceleration or a linear acceleration method. The following element types are incorporated in the model.

1. Linear-elastic and elasto-plastic (bilinear) straight beam (flexural) elements;
2. Linear-elastic foundation spring elements, shown in Figure 3-21;
3. Linear and bilinear expansion joint elements, shown in Figure 3-22; and
4. Degrading biaxial bending elements, shown in Figure 3-23.

The degrading bending element, also called the "five-spring elements" was added to the program during an earlier study conducted at the University of Nevada[9]. This element represents a column as a group of five axial springs, each spring representing the properties of the concrete and reinforcing steel. When employed in regions of plastic hinging in columns, the five-spring element allows a more realistic simulation of the response of reinforced concrete columns subjected to biaxial bending than do linear-elastic or bilinear column elements.

3.4 COMPUTER MODELING PROCEDURE

Many simplifying assumptions were made in modeling the bridges on Images-3D and NEABS-86. The general simplifications are listed in this section. Other idealizations which were unique to each computer program are presented in the next sections.

1. The curvature in the Madrone Dr. Undercrossing was assumed to be negligible. The superstructures in other bridges were straight. Therefore, all the bridge superstructures were treated as straight line elements.
2. Bridge members with variable cross sections were treated as a series of smaller segments with constant properties over each segment.
3. Any superelevation was neglected, because the superstructure was always modeled by a series of line elements located at the centroid of the superstructure.
4. Any vertical grade of the superstructure was neglected, because the effect of vertical grade was considered to be insignificant.

3.4.a. - Modeling for Images 3-D Analysis

The computer models of the four bridges for Images-3D analyses are shown in Figs. 3.24 to 3.27. Note that all the hinges were modeled as pins because hinge elements with gaps (or any other nonlinear elements) cannot be directly modeled on a linear analysis software such as Images-3D. For the same reason, the abutments had to be modeled as rollers. Modeling of hinges as pins necessitated that the restrainers be ignored in the modal analysis model.

The links connecting the superstructure to the piers in Madrone and Aptos were necessary because the offsets between the centroid of the superstructure and the centroid of the pier caps were significant. The section properties of the link had to be large to force the superstructure and the pier cap to deform equally at the piers. Other rigid elements had to be used to model the joint areas (the segments common to cap beams and columns). Because the pier walls in the San Gregorio bridge were modeled as line elements, rigid links had to be used to force the nodes at the top of the walls (for example nodes 23, 28, and 32 on Bent 3 in Fig. 3.27) on a straight line. Because the connecting slab between the east-bound and west-bound segments of the Huntington bridge was not detailed to transfer moment, it was modeled as a series of truss elements (Fig. 3.25). Considering the large cross section area of the slab, these truss elements were treated as rigid elements.

At least three intermediate nodes were assumed in each span. These nodes are not shown in the model for Huntington because otherwise the figure would not be clear. The methods used to determine the section properties and spring stiffnesses are consistent with the current Caltrans procedures. Details of methods used and the numerical values are presented in Refs. 16-19.

3.4.b. - Modeling for NEABS-86 Analysis

The computer models for NEABS-86 analysis of the four selected bridges are similar to the models shown in Figs. 3.24 to 3.27. In the model for Huntington, the superstructure nodes consisted only of those at the piers and those at either side of the superstructure hinges. No other superstructure nodes were used to keep the computer turn around time manageable.

The major differences between the modal analysis models and the models used in NEABS were (a) the columns, the longitudinal abutment springs, and the shear keys were modeled as nonlinear elements, and (b) the intermediate hinges were represented by the special hinge element incorporated in NEABS (Fig. 3.22). To model the potential column nonlinearity, five-spring elements (Fig. 3.23) were placed at all the column locations which were likely to develop plastic hinging. The hinge system includes an elasto-plastic restrainer sub-element. This sub-element was utilized to simulate nonlinear soil behavior at the abutments. Because the earthquake damage in some of the bridges included shear key failure and because the shear key elements in NEABS are linear and do not allow for the modeling of the shear key damage, the restrainer sub-element was also used to simulate the yielding of the shear keys. Therefore, in some of the abutments, several coincident "hinge systems" were used to simulate the nonlinearity of different components. All other boundary spring elements were modeled using the linear boundary springs incorporated in NEABS.

The Caltrans and other rational methods were used to determine the section and spring properties [6, 11]. References 16 to 19 present the procedures and the numerical values which were used in the analyses.

In carrying out the nonlinear analyses, a damping factor of five percent was assumed and the damping coefficients were determined based on the frequencies for the first two modes of vibration of each bridge. The constant acceleration method was used in conjunction with a time interval of 0.005 sec. for the solution of the differential equation of motion.

The earthquake record for the analysis of the Aptos Creek bridge was assumed to be the acceleration data obtained at the Capitola Fire Station which is located approximately four miles from the bridge site. The peak ground acceleration (PGA) in the north-south direction was 0.40g (where g is equal to 32.2 ft/s^2) and in the east-west direction was 0.47g. The north-south axis of the fire station was nearly parallel to the longitudinal axis of the bridge. Therefore, it was assumed that the north-south and the east-west components of the earthquake acted in the longitudinal and transverse direction of the bridge, respectively. Figures 3.28 and 3.29 show the acceleration records for the Aptos Creek bridge.

The nearest free field recording station to the Huntington Ave. Overhead was at approximately two miles to the southeast of the bridge located at the San Francisco International Airport with PGA of 0.24g and 0.33g in the east-west and north-south directions, respectively. A study of the geotechnical characteristics of the sites of the bridge and the airport showed that the depth of soft soil at the airport is larger than that at the bridge site and hence the PGA at the bridge site should have been smaller. This was confirmed by examining the measured acceleration record at the basement of two buildings located two miles to the west of the bridge. Five horizontal accelerometers were triggered by the earthquake and they recorded PGA's in the range of 0.11g to 0.14g. Considering the approximations involved in estimating the PGA at the bridge site, the input record was based on the free-field data at the airport with PGA reduced to 0.15g (Figs. 3.30 and 3.31).

The epicenter of the Loma Prieta earthquake was located approximately 8 miles from the Madrone Drive Undercrossing. The peak ground acceleration was estimated at 65 percent of the acceleration of gravity and was strongly in the transverse direction of the bridge [3]. Accelerograms from the nearest strong-motion detector station, Aloha Avenue in Saratoga (CSMIP station number 58065), were used as the ground motion input for the NEABS earthquake analysis. This station is located approximately 17 miles from the earthquake epicenter. Figure 3.32 illustrates the acceleration records which are scaled to the estimated 0.65g peak acceleration value. Analytical results showed that the extent of damage caused by a PGA of 0.65g would far exceed the actual damage. A reduction to a PGA of 0.5g led to results which were more in line with the observed behavior.

The epicenter of the earthquake was located approximately 40 miles south of the San Gregorio bridge. The nearest ground motion recording station to the bridge was at the Springs Reservoir Pulgas Water Temple in Upper Crystal, which is at about 44 miles from the epicenter. The PGA was 0.09g and 0.16g for the north-south and east-west directions, respectively (Fig. 3.33 and 3.34). Because the axis of the bridge was closer to the north-south direction, it was assumed that the input motion in the longitudinal and transverse direction of the bridge corresponded to the north-south and east-west directions of the recorded data.

Chapter 4
ANALYTICAL RESULTS

4.1 INTRODUCTION

A large number of computer analyses was performed for the bridges described in Chapter 3. The results of the frequency analyses are presented in Refs. 16 to 19 and are not discussed in this report because they merely provided values for input to the nonlinear analyses. The key elements in the nonlinear analyses were to determine (1) the best estimate of what the restrainers experienced during the 1989 Loma Prieta earthquake, and (2) what difference the restrainers made in the response of the bridge. In addition, a parametric study was undertaken to determine the effects of ground acceleration and the magnitude of the restrainer gap. The response parameters considered generally included the overall bridge movement; relative displacement at the hinge; the forces in the restrainers, shear keys, and abutments; and pier ductility demands. This chapter presents selected results which show the important characteristics of the responses. Detailed results are discussed in Refs. 16 to 19.

4.2 ROLE OF RESTRAINERS DURING THE EARTHQUAKE

The primary function of seismic restrainers in older bridges is to limit relative displacements below a limit beyond which a portion of the superstructure may fall off the support. Therefore, the relative hinge displacement is of primary importance. However, many other response parameters which may be affected by the presence or the absence of the restrainers are also of concern and need to be studied. The restrainer gaps used in the cases presented in this section are those specified on the blue prints for each bridge. The responses presented in this section are for the "base" earthquake records. The effects of higher PGA's are presented in the subsequent sections.

4.2.a. - Relative Longitudinal Movements at Hinges

Representative relative hinge displacement histories for the four bridges which were analyzed are presented in Figs. 4.1 to 4.5. The solid lines in all the figures show the movement with restrainers present, and the broken lines indicate the response with restrainers removed in the analytical model. Figure 1 does not include any broken lines because the relative hinge movements in this bridge were small and the restrainers were not activated. The results for other bridges indicate that the restrainers were stretched and they affected the response. It can be noted that the restrainers generally reduced the hinge movements significantly when activated. The most visible benefit of the restrainers can be seen in the San Gregorio bridge in which the presence of the restrainers reduced the hinge movement from approximately 2.6 in. to 1.25 in. The analytical results suggest that this bridge would have lost its support at Hinge 1 during the Loma Prieta earthquake had the restrainers not been installed. In the Aptos Creek bridge, however, the restrainers did not appear to affect the response, even though this bridge is close to the epicenter.

4.2.b. - Overall Bridge Displacements

The superstructure displacement histories for the four bridges are plotted in Figs. 4.6 to 4.13. The solid curves are the best estimates of the displacements that the bridges experienced during the earthquake. The dashed curves show the response would have been if the restrainers were removed. The longitudinal displacements in the Aptos Creek bridge were relatively small while the transverse displacements were as high as 4 in. This level of movement is consistent with observed abutment damage in this bridge. As it was mentioned in the last section, the restrainers were not activated in this bridge. Therefore, the bridge responses with or without the restrainers were the same. In the other three bridges, however, the restrainers were activated and generally reduced the movement of the bridge, sometimes significantly. This can be observed both in the longitudinal and the transverse directions.

4.2.c. - Restrainer Forces

The restrainers in the Aptos Creek bridge were not stretched during the Loma Prieta earthquake. A sample of force histories which represent the most critical restrainer forces for the other three bridges is shown in Figs. 4.14 to 4.16. Because of the skew in the Huntington Ave. Overhead bridge, there was significant rotation at the hinges. This led to differential forces in different cables in this bridge. In Fig. 4.14, the solid curve shows the force in the northernmost restrainer and the broken curve presents the force in the southernmost (near the edge of the right bridge) restrainer. Note that the forces in all cases were below yielding except for the cables in Madrone which yielded for a short time near t = 8 sec. (Fig. 4.15). Given the uncertainties in the defining the input earthquake and the fact that the actual hinge movement was not very large, it is not certain whether the restrainers actually yielded. The yield force for the restrainers in San Gregorio is 547 kips.

4.2.d. - Abutment Forces

To study the effects of restrainers on the abutment forces, the responses were reviewed for both conditions of the retrofitted case and unretrofitted case (Figs. 4.17 to 4.23). The lack of a broken curve in Fig. 17 for Aptos is because restrainers in this bridge were not utilized. The abutment force histories for Huntington (Fig. 4.18 to 4.21) show interesting trends. Note that Abutment 1 in both the right and the left structures is of seat type while the other abutment is of integral type, with no abutment gap specified. The abutments are active only when the bridge moves toward them. The lack of a solid curve in Fig. 4.18 indicates that Abutment 1 did not experience any force during the Loma Prieta earthquake. Had the restrainers not been installed, a small force (compared to the yield force) would have developed in this abutment. For the same structure, relatively large abutment forces were developed in the other abutment and the presence of the restrainers did not appear to make any difference in the peak force (Fig. 4.19). The trend in the left bridge was somewhat different. The seat-type abutment experienced a small amount of force even when the restrainers were present, although to a lesser degree (Fig. 4.20). The maximum force in the integral abutment, however, increased as a result of restrainer presence (Fig. 4.21), although no yielding resulted. The presence of the restrainers appear to have improved the integrity of the bridge and more utilization of the abutments which is generally desired. The difference between the behavior of the two structures is attributed to the fact that the right structure has 2 intermediate hinges while the left structure has 3. Structures with a larger number of hinges tend to apply more forces to the abutment.

The restrainers in the Madrone Dr. Undercrossing do not appear to have altered the abutment forces significantly (Fig. 4.22). This was true for both abutments. The abutments experienced large deformations during the earthquake and this is reflected in the damage report as well as the calculated response histories.

The effect of the abutment gap is also evident in the response of the San Gregorio bridge (Fig. 4.23). The abutment experienced only a small force during the earthquake because of the restrainers. When the restrainers were removed, however, significant forces were developed in both abutments, of which the force only in Abutment 1 is shown. The earthquake damage report for this bridge indicated severe cracking of the abutment back walls. This could indicate that the actual longitudinal motion experienced by the bridge was stronger than that assumed in the analytical model. The effect of stronger earthquakes is discussed in the next section.

4.2.e. - Shear Key Forces

Figures 4.24 to 4.27 show the force histories at the abutment shear keys of different bridges. Note that the "key" in Aptos represents the steel bearings and the anchor bolts, and it did not have any gaps. Other bridges had a gap of 0.25 to 1 in. between the superstructure and the shear keys. Damage reports had indicated damage in Aptos and Madrone. The analytical results show that all the abutment shear keys were activated by the earthquake and that they all yielded except in Huntington. In this bridge, the maximum shear key forces were approximately the same regardless of the presence or absence of the restrainers (Fig. 4.25). In the Madrone bridge, the shear keys yielded whether or not restrainers were present (Fig. 4.26).

The damage report for the San Gregorio bridge did not mention any shear key failure, whereas the calculated results show yielding. This could indicate that the actual transverse motion at the bridge site was not as strong as the assumed motion. Similar to other bridges, the restrainers did not affect shear key forces significantly in the San Gregorio bridge (Fig. 4.27).

4.3 EFFECT OF PEAK GROUND ACCELERATION

None of the bridges analyzed in this study were instrumented. The analyses summarized in Sec. 4.2 was based on the best estimate of the ground motion using ground acceleration records obtained at nearby stations. The actual ground motion at the bridge sites could have been different than that assumed. Because critical conditions occur during stronger earthquakes, it was decided to simulate ground motions with larger peak ground accelerations on these bridges and study the effect on different aspects of the bridge response with particular attention to hinge movements and restrainer forces. Table 4.1 lists different peak accelerations that were used under the headings of "Amplified 1" and "Amplified 2." An explanation of the reasons for selecting the PGA's shown is in the next section. Considering the fact that the focus of this study was on restrainers, this report presents the results only for hinges. The response for other elements may be found in Ref. 16 to 19.

4.3.a. - Effect of Increase in PGA on Hinge Movements

The relative displacement histories at the intermediate hinge for the Aptos Creek bridge analyzed for earthquake records amplified to 0.7g and 1.0g are shown in Figs. 4.28 and 4.29. A peak ground acceleration of 1.0g may be considered to be excessive, but it was used because the response under the 0.7g earthquake did not appear to be critical. Note that the restrainers were activated by the stronger earthquake and reduced the relative hinge movements considerably. In the case with PGA of 1.0g, the restrainers reduced the maximum movement by approximately 85 percent (Fig. 4.29). An interesting point to observe is that, despite the large PGA, the hinge movements in this bridge would not be excessive even without the restrainers.

For the Huntington Ave. Overhead bridge, the input motion was amplified to 0.3g and 0.6g in both directions. These peaks are twice and four times the estimated PGA during the Loma Prieta earthquake. Representative response histories at the hinge near Bent 7L for the 0.3g and 0.6g records are shown in Fig. 4.30 and 4.31, respectively. It can be noted that the restrainers generally reduced the local peaks. The absolute maximum in the case of 0.3g earthquake was not affected by the restrainers. However, the maximum relative displacement in the case of 0.6g motion was reduced by approximately 25 percent.

The amplified input ground motion for the Madrone Ave. Overhead bridge had a PGA of 0.65g. This figure was selected based on the estimate from Caltrans [3]. A sample hinge movement history at the hinge near Bent 2 is shown in Fig. 4.32. It can be observed that the restrainers reduced the hinge movement by approximately 60 percent. The superstructure would not survive without the restrainers because of the excessive hinge movements.

The beneficial effects of the restrainers in the San Gregorio bridge were more evident when the input PGA was raised to 0.32g and 0.60g (Fig. 4.33 and 4.34). Relative displacements at the hinges were reduced because of the restrainers. In the case with 0.6g PGA, the relative displacement would exceed the limit of 4 in. in this bridge and a part of the superstructure would lose its support if the restrainers were not included in the bridge.

4.3.b. - Effect of Increase in PGA on Restrainer Forces

The restrainer forces in the Aptos Creek bridge remained well below the yield force even with a PGA of 1.0g (Figs. 4.35 and 4.36). The plots also show that in both cases the restrainers were activated many times during the earthquakes.

The restrainers in Huntington experienced differential forces within some of the hinges because of the significant in-plane rotations. The force histories for 0.3g ad 0.6g PGA at the hinge near Bent 7L are shown in Figs. 4.37-4.40. Again, these results and those of other hinge restrainers indicate that none of the restrainers in Huntington yielded even under a PGA of 0.6g. The cable forces in the hinges of Madrone Ave. Overhead, however, showed some limited yielding nine seconds into the earthquake (Fig. 4.41).

The most critical restrainer forces in the San Gregorio bridge are shown in Figs. 4.42 and 4.43. It can be observed that large forces were developed in the cables, but the peak forces were well below the yield force of 547 kips.

4.4 EFFECT OF REDUCTION IN THE RESTRAINER GAP

Hinge restrainers are installed with a gap to allow for thermal movement of the bridge. In extreme cold conditions, this gap may be reduced to zero. At the same time the hinge gap increases, thus leaving a smaller seat width. In the Caltrans restrainer design example, a 0.75 in. restrainer gap is assumed. A study of the cable forces and the hinge movements under the condition of zero restrainer gaps was undertaken for the San Gregorio bridge. This bridge has five spans and three hinges. Results presented in previous sections showed that San Gregorio was the most susceptible bridge to hinge movements. The input ground motions were the Loma Prieta records with PGA's listed in the second and the third columns of Table 4.1. Because the cable forces present a more direct way to evaluate the effect of reduction in the gap, this report discusses only selected cable forces. More data on cable forces and hinge movements may be found in Ref. 17. The most critical cable forces are shown in Fig. 4.44. It can be seen that the restrainer forces are considerably larger when the gap is zero. In addition, the number of times that the restrainers are utilized is considerably smaller when the gap is 0.75 in. The large increase in the forces is due to the fact that, with zero restrainer gap, the restrainers are utilized as soon as there is any relative movement at the hinge, whereas when the gap is 0.75 in. the restrainers are not activated until the differential movement exceeds the gap.

4.5 OVERALL OBSERVATIONS

The results presented in previous sections of this chapter reveal the following valuable points relative to the effects of hinge restrainers on the seismic response of bridges.

1) The most important observation is the confirmation that restrainers indeed reduce hinge movements and prevent collapse of the superstructure due to support loss.

2) When restrainers were removed, the relative hinge displacements generally increased beyond the limits for narrow seats. This, of course, signifies that the restrainers are needed in these bridges. In the same bridges, had the seat width been 2 ft. (the width used in new construction), the displacements would be below the critical limits even without any restrainers. This suggests that restrainers may not generally be needed in new bridges unless a detailed nonlinear analysis shows otherwise. The current Caltrans practice which provides for a minimum number of restrainers in hinges with wide seats is aimed at improving the overall structural integrity of the bridge.

3) Of the seven bridges which were investigated in the field or analyzed, the restrainers in one bridge (the Aptos Creek bridge) do not appear to have been activated by the Loma Prieta earthquake. When the earthquake was amplified to 0.7g or higher, the restrainers were utilized and reduced hinge movement. However, even under a PGA of 1.0g, the hinge movements in this bridge were not critical. Table 4.2 lists some information about the behavior of this bridge in relationship with other bridges. Huntington is not listed because it has wide seats (with a nominal width of 2 ft.). The figures in column 3 are the precracked stiffness of the frames between hinges or a hinge and an abutment. The frames are assumed to be fixed at the base. The values in the fourth column are the axial rigidity of the restrainers divided by the axial rigidity of the superstructure. The lower and upper values of PGA's are shown in columns 5 and 6. Column 7 indicates

whether excessive hinge movements took place. To explain why Aptos did not lose support even under an earthquake with 1.0g PGA, one may consider the amount of restrainers to be a factor. Column 4 shows that the relative restrainer rigidity in this bridge was in between the value for other two bridges. Therefore, the restrainer area could not be a factor. In contrast, the frame stiffnesses in Aptos were larger than the values for other bridges, Furthermore, the Aptos Creek bridge had only one hinge in five spans (ratio of o.2) whereas the others had 2 in 3 spans (a ratio of 0.67 in Madrone) and 3 hinges in 5 spans (a ratio of 0.6 in San Gregorio). It is concluded that bridges with relatively soft substructure and high ratio of hinge to spans are more vulnerable to a loss of support at hinges.

4. Restrainer forces are more critical when the restrainer gap is zero. Only a refined nonlinear dynamic analysis should take this observation into account. This is because the maximum restrainer gap typically is 0.75 in. which is a small value, and the equivalent static analysis methods are too approximate to adequately reflect the change in the gap. This is discussed in more details in the next chapter.

Chapter 5
RESTRAINER DESIGN RESULTS

5.1 INTRODUCTION

The design of retrofit restrainers has been based on an equivalent static analysis method that incorporates many of the primary factors affecting the seismic response of bridges [6]. Many simplifying assumptions are made based on engineering judgement and observations made during past strong earthquakes. Although the performance of restrainers which have been designed using the current methodology has been generally satisfactory during recent moderate earthquakes, many aspects of the restrainer design method have not been studied in detail. The purpose of the study summarized in this chapter was to address some of these aspects of the hinge restrainer design method. More details of this study may be found in Ref. 15. This chapter also presents a brief description and an example of a suggested alternate restrainer design method that more directly accounts for the nonlinearity of the response and yields more rational results than the current method. Details of the proposed model are discussed in Ref. 17.

5.2 SENSITIVITY OF THE CURRENT METHOD TO DESIGN ASSUMPTIONS

The equivalent static analysis method for restrainer systems is intended to be a rational yet practical procedure for manual design of hinge retrofit restrainers [6]. Several simplifying assumptions are made in calculating the effective stiffness and mass. These assumptions influence the effective vibration period of different frames or groups of frames and eventually affect the number of restrainers. Furthermore, an initial restrainer gap of 0.75 in. is assumed, which corresponds to the extreme high ambient temperature. This section describes the sensitivity of the number of required restrainers to (1) the changes in the way the effective stiffness and mass are calculated and (2) the elimination of the restrainer gap. The example bridge presented in Ref. 6 (Fig. 5.1) was used to illustrate the effects. The bridge has three hinges which are numbered from left to right. The peak bed rock acceleration is assumed to be 0.6g. Only 3/4 in. cable restrainers were considered for the bridge.

5.2.a. - Influence of Changes in the Stiffness and Mass

To determine the relative movements that must be reduced by hinge restrainers, the bridge structure on each side of the hinge is considered separately. A "frame" is defined as the part of the bridge which is in between two adjacent hinges, or is between a hinge and the adjacent abutment. On each side of each hinge, the movement of the frame away from the hinge is considered. As the frame moves, it closes the gap at an adjacent hinge, thus mobilizing a second frame. If the frame continues to move in the same direction, another hinge may close which leads to the mobilization of a third frame or an abutment. The design method [6] accounts for the closure of only one adjacent hinge and permits the mobilization of only one adjacent frame. This assumption simplifies the analysis and is intended to be conservative. It is also assumed that the mass for only one frame should be used in computing the effective period of vibration for the segments, even when more than one frame is mobilized. The purpose of this section is to discuss the effect of allowing more than one adjacent frame to contribute to the stiffness and mass when displacements are sufficiently large to close the hinge gaps.

Two cable lengths of 5 ft. and 7 ft. are considered in the design example. The maximum restrainer "deflections" for these are 1.81 in. and 2.25 in., respectively, including a restrainer gap of 0.75 in. At either side of each hinge, the movement of the bridge segment is considered and the force-displacement relationship is plotted. The effective stiffness of each segment (or segments, when the gap at a second adjacent hinge closes) is the slope of the line connecting the origin to the point on the curve corresponding to the maximum restrainer elongation. Figure 5.2 shows the stiffness chart for the right side of Hinge 1 as it is presented in Ref. 6 and as refined in this study. Note that Chart (b) accounts for the fact that, at a displacement of 1.5 in., both Hinges 2 and 3 are closed and all three frames move together. As a result, the refined effective stiffnesses are 20 percent and 29 percent larger than those in Ref. 6 for the 5-ft. and 7-ft. cables, respectively. Similar trends in stiffness were observed for other hinges, the details of which are discussed in Ref. 15.

The number of required restrainers is shown in Col. 4 of Table 5.1 for different hinges. Hinge 1 did not need a restrainer when 7-ft. cables were used. It can be observed that, by changing the treatment of the mass, the number of cables is reduced drastically. The general explanation for this reduction is the fact that the added masses increased the effective period considerably. The period elongation, in turn, reduced the acceleration response spectrum (ARS) value, and the reduced ARS led to smaller displacements. Because restrainers are designed to control relative movements at hinges, smaller displacements required fewer cables.

As noted before, the intent of using the lower stiffness and masses in the current restrainer design method is to simplify the analysis and to be conservative. The comparison of the required restrainers in Columns 3 and 4 of Table 5.1 confirms that the simplifications do lead to a more conservative number of restrainers.

5.2.b. - Influence of Reducing Restrainer Gap to Zero

During extreme low ambient temperatures, the superstructure segments become shorter. As a result, the restrainer gap may diminish while the hinge and abutment gaps increase. When restrainer gap in the example bridge is reduced to zero, the gap will increase to 1.5 in., 1.375 in., and 2.375 in. in the hinges, Abutment 1, and Abutment 2, respectively. This will reduce the allowable cable "deflection" and the seat width to maintain a 3-in. minimum bearing. The effect of these changes on the number of required restrainers is presented in this section.

The maximum cable deflection is 1.06 in. for the 5-ft. cable and 1.48 in. for the 7-ft. cable. The maximum allowable movement is 1.5 in. in order to maintain a minimum seat width of 3 in. Figure 5.3 shows a sample of stiffness charts for Hinges 1 and 3. A detailed variation of stiffness changes for all hinges is illustrated in Ref. 15. It can be seen that, because of the small permissible cable deflections and because of large hinge gaps, the gaps cannot close without yielding the restrainers. This is observed for all the frames [15]. As a result, each segment of the bridge oscillates individually.

As expected, a large number of restrainers is required to limit the displacements to the above levels. Column 5 in Table 5.1 shows the results of the design. It can be seen that the numbers are substantially higher than those of other conditions.

5.2.c. - Discussion of the Current Restrainer Design Method

The results discussed in Sec. 5.2.a and 5.2.b indicate that the simplifying and judgmental assumptions made in the design of restrainers can greatly change the outcome of the design method presented in Ref. 6. An improvement in the methods to calculate the stiffness and mass leads to a reduction in the number of the restrainers, thus suggesting that the current method is conservative. However, when the restrainer gap is reduced, the number of the restrainers increases considerably, thus giving the impression that the current method in unconservative. Nonetheless, when these conclusions are combined with the field observations during the Loma Prieta earthquake and the analyses presented in Chapter 4, it appears that the current design method is safe and conservative.

The high degree of sensitivity of the results to the assumptions is due to the fact that the equivalent static method is extremely approximate for calculating relative hinge movements. It is not appropriate to modify the current restrainer design method by merely reducing the restrainer gap or other similar measures because such modification would not address the fundamental shortcoming of the method which is the computation of the relative hinge displacements. It is apparent that an alternate design method may have to be developed which accounts for the nonlinearity of the elements more accurately and yields a better estimate of hinge movements. An improved method is discussed in Sec. 5.4.

5.3 SENSITIVITY OF THE NONLINEAR RESPONSE TO DESIGN PARAMETERS

5.3.a. - General Remarks

A nonlinear analysis of the seismic response of the example bridge can yield more rational results because the analysis takes into account the instantaneous changes in stiffness during the earthquake. Because different earthquakes can affect the response differently, it is important that the analysis be carried out for a group of earthquake records, and the design be based on the envelop of the response parameters.

The example bridge in Ref. 6 was analyzed for 36 combinations of restrainer numbers, input earthquakes, and restrainer gaps using computer program NEABS-86 [9]. In all the analyses, a 5 percent damping ratio and a time interval of 0.005 second were used. The focus of the study was the effect of the change in parameters on (1) relative movements at the hinges and abutments, (2) restrainer forces and stresses, and (3) abutment movements and forces. In all the parametric studies, the piers experienced a moderate level of yielding. The maximum displacement ductility demand for all the cases ranged between 1.8 to 2.2. The ductility demand is defined as the ratio of the maximum pier top horizontal displacement divided by displacement when the base yields in flexure.

Three earthquake records were used in the nonlinear analyses. These were the north-south component of the 1940 El Centro earthquake, the north-south component of the Eureka earthquake of 1954, and the east-west component of the 1989 Loma Prieta earthquake recorded at the Saratoga station. All the records were normalized to a peak ground acceleration (PGA) of 0.6g, and were applied in the longitudinal direction of the bridge. The PGA of 0.6g is the same as that used in the example bridge in Ref. 6. Figure 5.3 shows the acceleration spectrum for the three earthquakes calculated for a damping ratio of five percent.

Two parameters were studied. One was the hinge restrainer cross sectional area and the other was the restrainer initial gap. Because only 3/4-in. cable restrainers were considered, the first parameter was studied by changing the number of restrainers. The restrainer gaps included in the study represented the condition of the bridge during the extremely high and extremely low temperatures.

Table 5.2 shows the number of restrainers for different cases. It can be seen that the restrainers were used in sets of ten cables to be consistent with the design example [6]. The number of cables in Case 1 is the same as that used in the example. Note that the abutments had no restrainers. In Cases 2 to 5, the numbers were arbitrarily reduced for all three hinges. The restrainers were completely eliminated in Case 6. The cables were all 7 ft. long and were placed in a symmetric pattern relative to the longitudinal axis of the bridge. The specified restrainer gap of 0.75 in. in Ref. 6 is for the "highest ambient temperature." The corresponding hinge gap is assumed to be 0.75 in. During the extreme low ambient temperature, the restrainer gap is expected to become very small and even approach zero. Under this condition, the hinge gap will reach 1.5 in. The cases listed in Table 5.2 were analyzed for both the extreme high (restrainer gap of 0.75 in.) and extreme low (restrainer gap of zero) ambient temperatures.

Only a sample of important results are presented in the subsequent sections. Reference 15 describes the details of the study and presents more complete results.

5.3.b - Effects of Reduction in the Restrainer Gap

The design of restrainers according to Ref. 1 is carried out for the highest ambient temperature with an assumed restrainer gap of 0.75 in. The gap can be reduced to zero during low temperatures due to the contraction of the superstructure. The results presented in this section reflect the effect of reducing the restrainer gap to zero for different restrainer areas. To facilitate the comparisons, the envelopes of the results for different earthquakes are considered.

Figure 5.4 shows the maximum relative superstructure displacements adjacent to each hinge. It can be observed that, by eliminating the restrainer gap, the displacements were sensitive to the number of cables. The results for Case 6, in which no restrainers were present, show that the relative displacements would exceed the permissible movement of 0.125 ft. at all hinges, and would potentially lead to the collapse of the superstructure. The beneficial effects of using a large number of restrainers (Cases 1 to 3) to reduce hinge movements is realized only when the restrainer gap is zero. The most critical relative hinge movements may correspond to zero or a non-zero restrainer gap depending on the number of restrainers. An improved design method would need to account for both the extreme temperature conditions to determine the maximum hinge movements.

The total restrainer forces and stresses are compared for different hinge gaps in Figs. 5.5 and 5.6, respectively. It can be seen that the forces generally tend to decrease when the number of cables is reduced regardless of the gap. The reduction in the stiffness of the connection between the adjacent superstructure segments is believed to cause the decrease in the force. The slight deviations from this trend observed in Hinge 1 and 3 are due to the fact that the bridge model is nonlinear and effects can not always be explained using elastic theory. The forces corresponding to no restrainer gap condition were always considerably higher than those in cases with a gap. Figure 5.6 shows that restrainer stresses are sensitive to the number of cables when no gap is present. A reduction in the number of cables generally increased the magnitude of stresses. The maximum stress at hinge 2 in Case 5 reached nearly 85 percent of the yield stress. The data clearly indicate that the most critical condition for cable stresses is when the restrainer gap is reduced to zero due to low temperatures.

The envelopes of the maximum closing relative displacements between the superstructure and the abutments are shown in Fig. 5.7. In all cases, the superstructure moved sufficiently to close the abutment gap. Note that the gap in Abutment 2 is nearly twice that in Abutment 1. The presence or the number of the cables did not appear to influence the movements. The relative displacements for the low temperature condition (no restrainer gap) were always higher.

This is due to the fact that the abutment gap is larger by 0.375 in. for this condition. Therefore, larger displacements are allowed before the abutment gap is closed.

The larger openings at abutments in cold temperatures resulted in generally smaller forces in the back fill soil (Fig. 5.8). This is evident in both abutments, although it is very pronounced in Abutment 2 where the gap is larger. Abutment 2 yielded regardless of the number of cables when the restrainer gap was 0.75 in. Considering the fact that the trend was not uniform at Abutment 1, it is concluded that to determine the most critical abutment forces, it is necessary to analyze the bridge for both the extreme cold and extreme high ambient temperature condition.

The results presented in this section pointed out the need to consider the extreme low ambient temperature condition (zero restrainer gap) to determine critical restrainer stresses. The maximum relative superstructure displacements at hinges and abutments, however, may develop for the case of zero or non-zero restrainer gap depending on the number of restrainers. It is therefore recommended that the design of retrofit restrainers include both the extreme high and low temperatures conditions.

5.4 SUGGESTED RESTRAINER DESIGN PROCEDURE

To obtain a realistic estimate of the maximum hinge movements, it is necessary to carry out an inelastic response history analysis of the bridge. A longitudinal analysis of the bridge is sufficient as long as the skew angle and the curvature are negligible. For highly skewed and/or curved bridges, a three-dimensional nonlinear analysis is generally necessary. When the skew angle and the curvature are small, the software to carry out the analysis can be considerably simpler than computer program NEABS or other main-frame analysis programs, and it should be possible to implement the software on work stations or desk-top computers. It should be noted that the proposed method is of tentaive nature and is subject to refinement. The primary features of the software would need to be as follows:

i) The program determines the response history of the bridge using a time-step analysis method.

ii) Nonlinearity of the soil and its cyclic response are properly modeled at the abutments and the bridge foundations.

iii) Hinge nonlinearities due to gap closure and impact are directly modeled.

iv) Provisions are made to model the yielding of the restrainers.

v) The nonlinearity of the pier response and stiffness and strength degradation effects are taken into account.

vi) The nonlinearity of the shear keys that may be activated by the longitudinal motion of the superstructure is accounted for.

Because the available programs for seismic analysis of bridges do not adequately incorporate the above features, computer program NEABS-86 was used to illustrate the proposed alternate design method which is described in the next section.

5.4.a. - Outline of the Alternate Restrainer Design Method

The primary difference between the suggested method and what is currently used [6] is the procedure to determine the maximum relative displacements at hinges. However, a computer program is necessary for the analysis. The method includes the following steps:

1) Using a nonlinear response history analysis program, determine the maximum relative hinge displacements at all the intermediate and abutment (if any) hinges. In this step, do not place any restrainers. Use an array of input earthquake records which are representative of the seismic activities at the site of the bridge. Determine the maximum relative displacement envelopes.

2) Based on the relative displacement maxima, and using the current Caltrans method, determine the number of the required restrainers at each hinge. Determine the maximum number of restrainers and use this number at all the hinges. The same number of restrainers is recommended for the purpose of uniformity of construction. Use an even number of restrainers for symmetry.

3) Reanalyze the bridge with the restrainers designed in Step 2 included. In this and subsequent analyses use zero restrainer gap and a hinge gap corresponding to the lowest ambient temperature. Determine the maximum relative displacement envelopes at all the hinges, and compare with the permissible hinge movement. If necessary, increase the number of restrainers, and repeat Steps 2 and 3 until relative displacements at all the hinges are below the permissible limits.

5.4.b. - Illustrative Example

To illustrate the above method, the San Gregorio bridge which was discussed in Chapter 3 was selected. This bridge was selected because of its relatively flexible substructure which made it more susceptible to the loss of support at hinges. Detailed information about the bridge and its seismic modeling may be found in Ref. 17.

The bridge was analyzed for the same three earthquake records used in Sec. 5.3, except that the data were amplified to a PGA of 0.7g to represent strong ground motions. All the computer runs were made with zero restrainer gap because this was determined to be the most critical condition.

Figure 5.9 shows the relative displacement maxima for the bridge without any restrainers. The dashed line shows the limit beyond which the bearing length becomes less than 3 in. Under this condition the bridge is assumed to lose its support. It can be seen that the relative displacements at all the hinges will exceed the limit when no restrainers are used. Similar to the method used by Caltrans, the number of the restrainers is determined such that the restrainer forces will reduce the hinge gap below the critical limit. The resulting values will be 6, 2, and 4 restrainers in Hinges 1, 2, and 3, respectively. The solid circles in Fig. 5.10 show the maximum relative displacements after these cables are placed in the bridge. To facilitate construction, however, the number of restrainers at all the hinges was increased to 6 (the largest number of required restrainers). The results of the nonlinear analysis are shown by open circles in the figure. It can be noted that the movement in hinge 1 for the case of El Centro record slightly exceeds the limit. In the next iteration, the number of restrainers is increased from 6 to 8 at all the hinges. The open circles with a slash in Fig. 5.10 show the results for that iteration. It can be seen that all the relative displacements are below the limit and there is no support loss.

Chapter 6
SUMMARY AND CONCLUSIONS

6.1 SUMMARY

This report presents a summary of the important findings of a study aimed at several aspects of the behavior of hinge restrainers used as a seismic retrofit measure. Details of the study are described in five other reports [15 to 19]. The study included field investigations, extensive analytical studies, and an evaluation of the restrainer design method. A more rational design method was suggested in this study. The objectives of the study were: (1) to review the actual performance of bridge hinge restrainers during the 1989 Loma Prieta earthquake, (2) to simulate the earthquake effect on the analytical models of several selected bridges and study their responses, (3) to carry out a parametric study of these bridges to determine the effect of stronger earthquakes and the effect of changes in the restrainer gaps, and (4) to review the restrainer design procedure and recommend any needed refinements.

A data base of the bridges which had incorporated hinge restrainers and which had been damaged by the 1989 Loma Prieta earthquake was formed. Twenty-three bridges were in the data base. The damage reports prepared by the Caltrans Maintenance Division were reviewed. Three bridges, namely, the Central Viaduct, the Route 580/24/980 Separation, and the Route 92/101 Separation were investigated in the field. These bridges had experienced some damage due to the earthquake, some of which was at or near hinges. Measurements were made of crack widths and patterns, and the condition of the restrainers was examined. An analysis of locally damaged components was subsequently made and conclusions were drawn based on the observations and the calculated results.

The data base of the damaged bridges was also used to select four bridges for detailed nonlinear response history analyses using computer program NEABS-86 [9], which is a modified version of program NEABS [8]. The four structures ranged from three to eleven spans. They had different number of hinges and different substructure characteristics. The earthquake intensity also varied considerably from one bridge to another. Two groups of earthquake analyses were carried out: one involved the use of input acceleration which were believed to best represent the earthquake motion that the bridge experienced during the Loma Prieta earthquake, and the other was a series of parametric studies with larger peak ground acceleration (PGA) for the input earthquake and different restrainer gaps.

The evaluation of the current Caltrans restrainer design method consisted of two parts: (1) a study of the effect of refinement in the current methods of calculating stiffness and mass on the number of restrainers, and (2) a large number of nonlinear analyses of the Caltrans example bridge for different earthquakes, hinge gaps, and the number of restrainers. Based on these studies, a new method for the computation of the relative hinge movement was proposed, and demonstrated for one of the four bridges which had been the subject of detailed nonlinear analyses.

6.2 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of the study presented in this report and supporting reports [15 to 19], the following conclusions were drawn:

1- The current Caltrans method for restrainer design leads to a conservative and safe design in terms of the number of restrainers. However, the degree of conservatism in the Caltrans restrainer design method varies considerably from one hinge to another even within a given bridge.

2- A more refined method to compute relative hinge displacements can lead to fewer restrainers even in hinges with a nominal seat width of 6 in. The refined method explicitly would incorporate the nonlinearity of soil at the footings and abutments, plastic hinging of the columns, and the nonlinearity of the hinges.

3- There is a strong suggestion that the number of restrainers in new bridges (those with a nominal seat width of 24 in.) can be substantially reduced (or eliminated) in many cases if a more refined analysis is carried out. The current practice of placing a minimum of two restrainers is appropriate in that it provides for additional redundancy when excessive hinge movements due to unforeseen circumstances occur.

4- The Loma Prieta earthquake activated the hinge restrainers in the majority of the bridges investigated in this study. Except for a few instances, the restrainers and their supporting systems performed well.

5- The analysis of the Aptos Creek bridge which has a relatively stiff substructure in the longitudinal direction indicated that the restrainers did not play a significant role during the Loma Prieta earthquake. Computer simulation of stronger earthquakes on this bridge showed that elimination of restrainers would not adversely affect the bridge seismic performance.

6- Whether restrainers can make a difference in saving the spans from falling off their supports depends on many factors including the intensity and the frequency content of the ground motion, the foundation stiffness properties, the stiffness and yielding properties of the substructure, and the superstructure characteristics. Because of the sensitivity of the relative displacements at hinges to these parameters, a reasonably accurate nonlinear response history analysis needs to be conducted before a reliable determination of the role of restrainers can be made. The equivalent static analysis method would not be an appropriate tool for this purpose.

7- In the analysis and design of hinge restrainers, it is necessary to consider the performance of the restrainer system and not merely the restrainers. The system includes (i) the connection between the restrainers and the superstructure including any diaphragms, (ii) the superstructure adjacent to the hinge, and (iii) the restrainers.

8. The "weak link" in the restrainer system in bridges with narrow hinge seats (approximately 6 in.) would have to be the superstructure because yielding of the restrainers or the failure of the connections would lead to excessive movement at the hinge which could result in support loss. However, in bridges with wide hinge seats (approximately 2 ft.) by allowing the restrainers to experience limited yielding and some movement at the hinges, the integrity of the bridge can still be maintained. A minimum level of strain hardening would be required for the restrainers.

9. The most critical case for restrainer design corresponds to the condition when the restrainer gap is zero. Whereas the critical abutment forces during the earthquake may occur when the restrainer gap is maximum. As a result, in a nonlinear response history analysis both conditions would need to be considered.

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Table 2.1 - Geometric Properties of Damaged Bridges Equipped with Hinge Restrainers.

	Bridge Name	Skew Angle	No of Spans	Max Span Length	Total Length	Width	No of Hinges	Year Built	Year Recons tructed	Bridge Number
1	Alemany Interchange	M V	121	160 0	11541 0	37 8		1960		34 0070
2	Aptos Creek	0 0	5	56 0	260 0	60 8	1	1948		36 0011
3	Central Viaduct									34 0077
4	China Basin Viaduct									34 0100
5	Coyote Creek	40 0	9/8	60 0	506 0	32 0	2 2	1952		37 0065L
6	Old East Connector (N980/W580/E580) New East Connector (E980/W580/E580)	M V	35	142 0	2721 0	34 0		1970		33 0304G
7	Embarcadero Viaduct									34 0055
8	Huntington Avenue Overhead	M V	11 12	140 0	1031 0	197 0	2 3	1971	1973	35 0253
9	Lawrence Expressway Overcrossing									37 0152
10	Madrone Drive Undercrossing	0 0	3	50 0	135 0	59 0	2	1938		37 0059
11	Marina Lagoon	0 0	8	62 0	619 0	61 0	2	1967		35 0193Y
12	Marina Viaduct	M V	103	56 0	4575 0	68 3		1936		34 0014
13	Mococo Overhead	0 0	10	160 0	1881 0	97 7		1962		28 0171
14	Old North Connector (580/E24) New North Connector (E580/W580/E24)	M V	42	146 0	3940 0	38 0		1970		33 0302H
15	Richardson Bay Br & Separation	M V	21	80 0	2860 0	138 0		1957	1973	27 0010
16	Route 92/101 Separation	30 0	95	208 0	4362 0	41 3		1971	1985	35 252L
17	Port of Okland Over crossing (West Grand Avenue Viaduct)	M V	83	159 0	5409 0	34 0		1966		33 126L
18	San Gregorio Creek	0 0	5	59 0	266 0	26 0	3	1941		35 30
19	Sargent Bridge and Overhead	M V	11	72 0	607 0	32 0		1950		37 0006L
20	Sidehill Viaduct	0 0	37	25 0	927 0	39 1		1940		37 0029
21	Old South Connector (580/W980) New South Connector (W580/E580/W980)	M V	36	125 0	3311 0	34 0		1970		34 0303H
22	Southern Freeway Viaduct	M V	234	132 0	21588 0	49 8		1964		34 0046
23	Terminal Separation									34 0054

Table 2.2 - Summary of Overall Damage to Bridges Equipped with Hinge Restrainers.

	Bridge Name	Reported Damage at/or Near Hinges					
		Restrainer Damage	Rest. Conn Damage	Diaphragm Damage	Pounding	Superstructure	Joint Seal Damage
1	Alemany Interchange	Stretched did not yield	None	None	Crushing		Failure
2	Aptos Creek	Stretched did not yield	None	None	None	None	Failure
3	Centeral Viaduct	None	None	None	None	None	None
4	China Basin Viaduct	Yielded	None	Severely Cracked	Crushing at Some Exp. Jt	Exposed Stee	Failure
5	Coyote Creek	None	None	None	None	None	Failure
6	Old East Connector (N980/W580 E580) New East Connector (E980/W580 E580)	None	None	None	Crushing	Concrete Spalls	None
7	Embarcadero Viaduct	None	None	None	None	Small Spalls	None
8	Huntington Avenue Overhead	None	None	None	None	None	
9	Lawrence Expressway Overcrossing	None	None	None	None	None	None
10	Madrone Drive Undercrossing						Failure
11	Marina Lagoon	None	None	None	None	None	None
12	Marina Viaduct	None	None	None	Crushing	Concrete Spalls	Failure
13	Mococo Overhead	Stretched did not yield	None	None	Crushing	Concrete Spalls	Failure
14	Old North Connector (580/E24) New North Connector (E580 W580/E24)	None	None	None	Crushing	Some Damage	None
15	Richardson Bay Br. & Separation	Failure	Failure	Failure	Crushing	Damage	Failure
16	Route 92 101 Separation	Yielded	Damage	Some Damage	Some Crushing	Exposed Steel	Failure
17	Port of Okland Over crossing (West Grand Avenue Viaduct)	Failure	Failure	None	None	Damage	None
18	San Gregorio Creek	None	None	None	Some Crushing	Exposed Steel	Failure
19	Sargent Bridge and Overhead	Stretched did not yield	None	None	Some Damage at Span 4	None	Failure
20	Sidehill Viaduct	None	None	None	None	None	Failure
21	Old South Connector (580/W980) New South Connector (W580 E580/w980)	Stretched did not yield	None	None	None	Concrete Spalls	Failure
22	Southern Freeway Viaduct	Stretched did not yield	None	None	Some Crushing	Concrete Spalls	Failure
23	Terminal Separation	None	None	None	None	Cracks	None

Table 2.3 - Summary of Hinge Damage to Bridges Equipped with Hinge Restrainers.

	Bridge Name	Hinge Opening		Abutment Crack		Column			
		2" <	> 2"	Large	Small	Column Crack	Separation	Deck Crack	Footing
1	Alemany Interchange		✓			✓	✓		✓
2	Aptos Creek			✓					
3	Centeral Viaduct					✓			
4	China Basin Viaduct	✓	✓			✓		✓	
5	Coyote Creek	✓							
6	Old East Connector (N980/W580 E580) New East Connector (E980/W580 E580)				✓			✓	
7	Embarcadero Viaduct	✓				✓		✓	
8	Huntington Avenue Overhead			✓					
9	Lawrence Expressway Overcrossing				✓			✓	
10	Madrone Drive Undercrossing		✓	✓					
11	Marina Lagoon					✓			
12	Marina Viaduct	✓				✓		✓	
13	Mococo Overhead		✓			✓			
14	Old North Connector (580/E24) New North Connector (E580 W580/E24)	✓						✓	
16	Richardson Bay Br. & Separation		✓			✓		✓	
16	Route 92/101 Separation		✓	✓				✓	
17	Port of Okland Overcrossing (West Grand Avenue Viaduct)		✓			✓	✓	✓	
18	San Gregorio Creek		✓	✓					
19	Sargent Bridge and Overhead		✓			✓		✓	
20	Sidehill Viaduct							✓	
21	Old South Connector (580/W980) New South Connector (W580 E580/w980)		✓	✓		✓		✓	
22	Southern Freeway Viaduct					✓		✓	
23	Terminal Separation					✓		✓	

Table 4.1 - Peak Ground Accelerations for different Cases in Terms of g.

(1) Bridge	(2) Loma Prieta Longitudinal	(3) Loma Prieta Transverse	(4) Amplified 1 Longitudinal	(5) Amplified 1 Transverse	(6) Amplified 2 Longitudinal	(7) Amplified 2 Transverse
Aptos	0.40	0.47	0.70	0.70	1.0	1.0
Huntington	0 15	0.15	0.30	0.30	0.60	0.60
Madrone	0.50	0.50	0.65	0.65	-	-
San Greg.	0.16	.09	0.32	0.18	0.60	0.34

Table 4.2 - Trends in Bridges with Narrow Seats

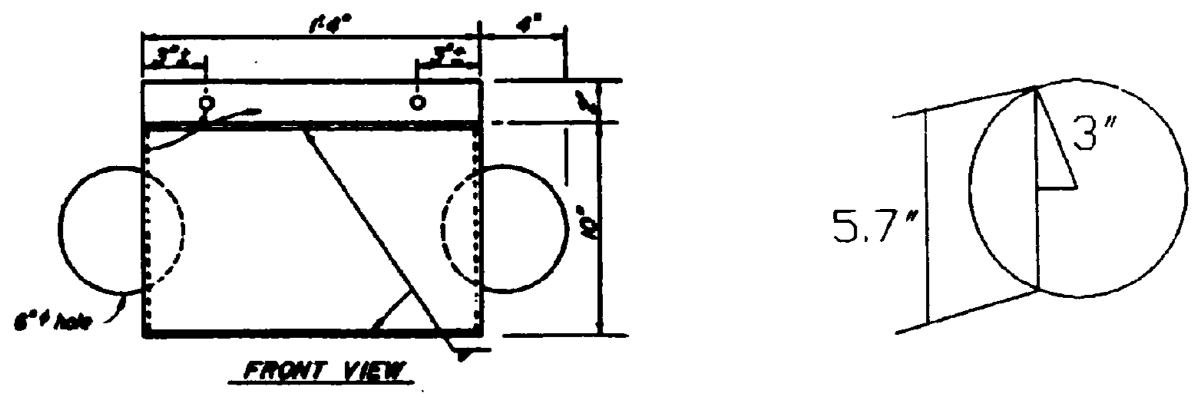
(1) Bridge	(2) No. of Hinges	(3) Frame Stiffness (k/ft)	(4) $(EA)_r/(EA)_s$ x 1000	(5) Long. PGA (g)	(6) Trans. PGA (g)	(7) Support Loss ?
Aptos	1	16000; 118000	1.04	0.4 1.0	0.47 1.0	NO NO
Madrone	2	13800	0.22	0.5 0.65	0.5 0.65	NO YES
San Greg.	3	2100; 3800	3.42	0.16 0.6	0.9 0.34	NO YES

Table 5.1 - Number of Required Ten-Cable Units for Different Analyses

(1) HINGE	(2) CABLE LENGTH (ft.)	(3) REF. 6	(4) MODIFIED EQ. STIFF. & MASS	(5) ZERO REST. GAP
1	5 7	10 8	4 -	15 13
2	5 7	12 7	7 2	15 14
3	5 7	13 11	6 2	16 15

Table 5.2 Number of Cables in Different Cases

CASE NO.	NUMBER OF CABLES		
	HINGE 1	HINGE 2	HINGE 3
1	8X10	6X10	10X10
2	7X10	5X10	9X10
3	6X10	4X10	8X10
4	5X10	3X10	7X10
5	4X10	2X10	6X10
6	0	0	0



(a) (b)

Fig. 2.1 - Cable Drum Detail. (a) Elevation View.
(b) Available Vertical Space for Restrainers

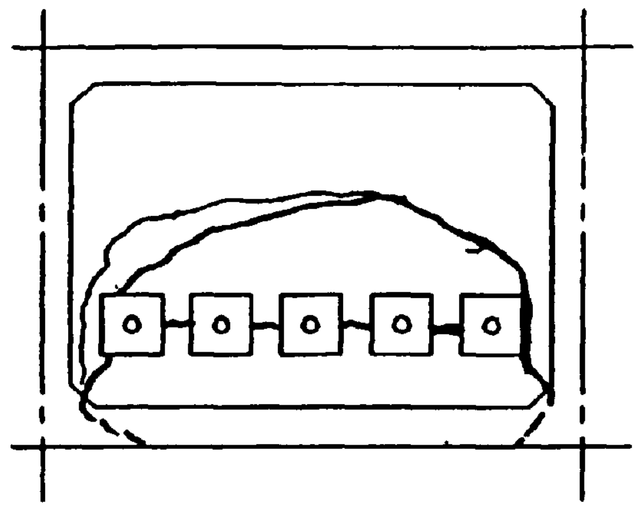


Fig. 2.2 - Crack Pattern Inside the Cell.

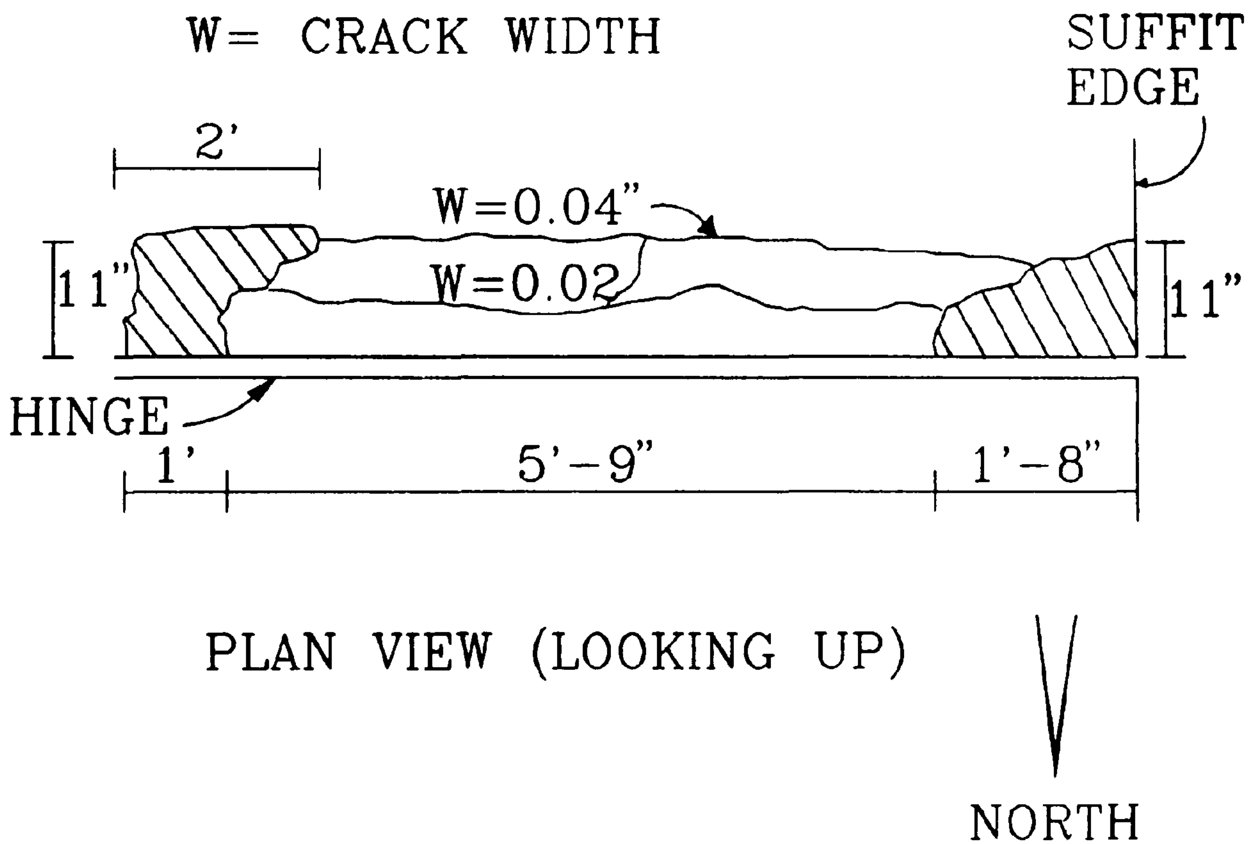
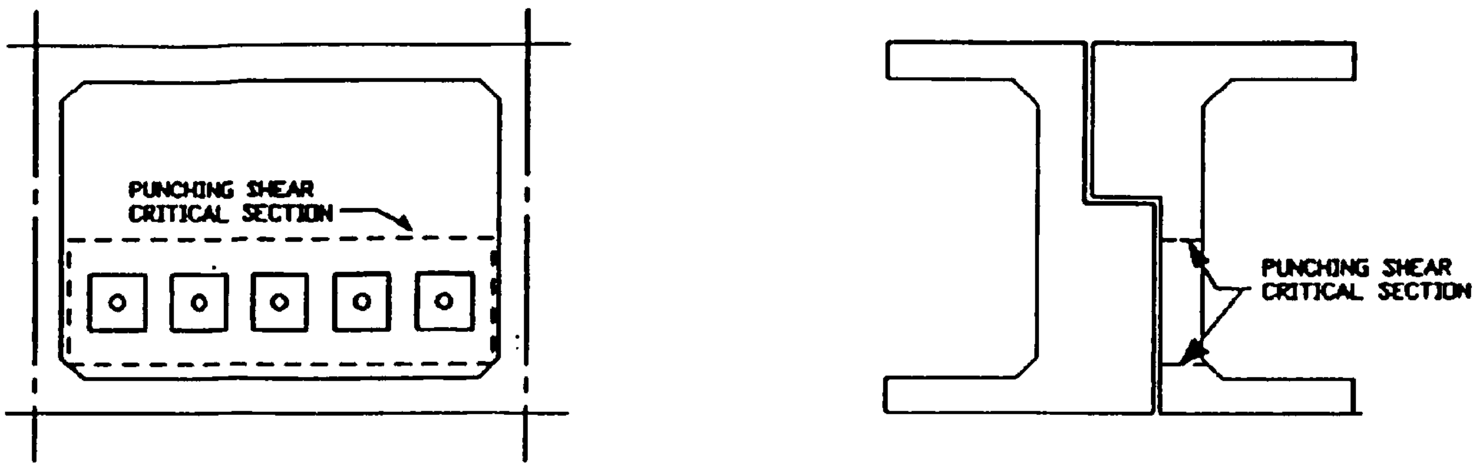
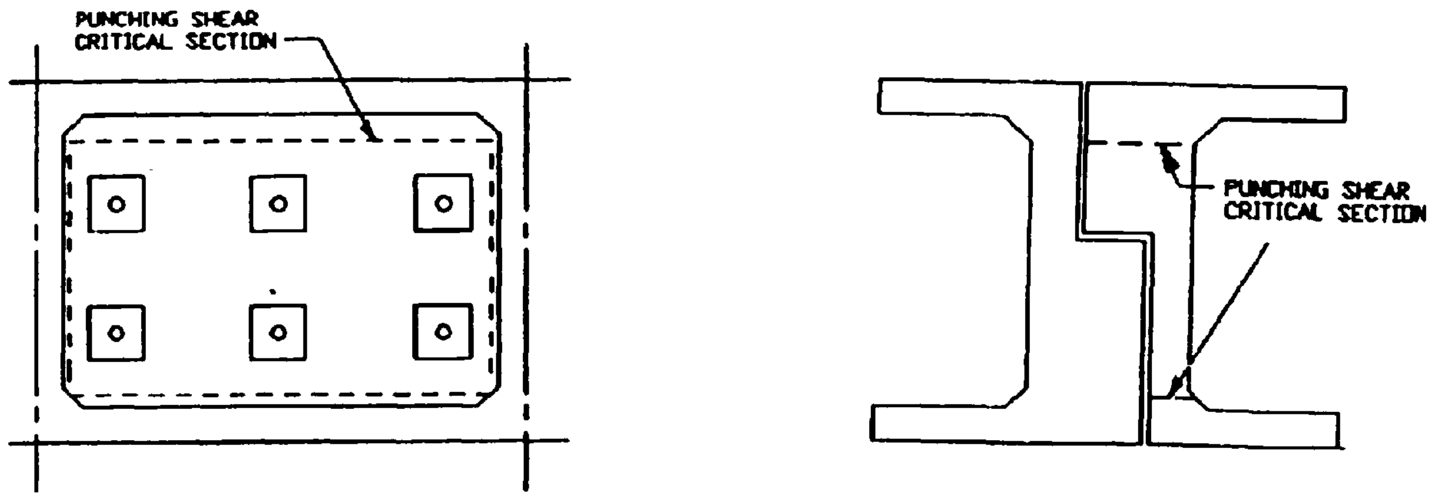


Fig. 2.3 - Crack Pattern at the Bottom Soffit Near Hinge 33.



(a) (b)

Fig. 2.4 - Critical Section for One Row of Rods. (a) Front View. (b) Section.



(a) (b)

Fig. 2.5 - Critical Section for Two Rows of Rods. (a) Front View. (b) Section.

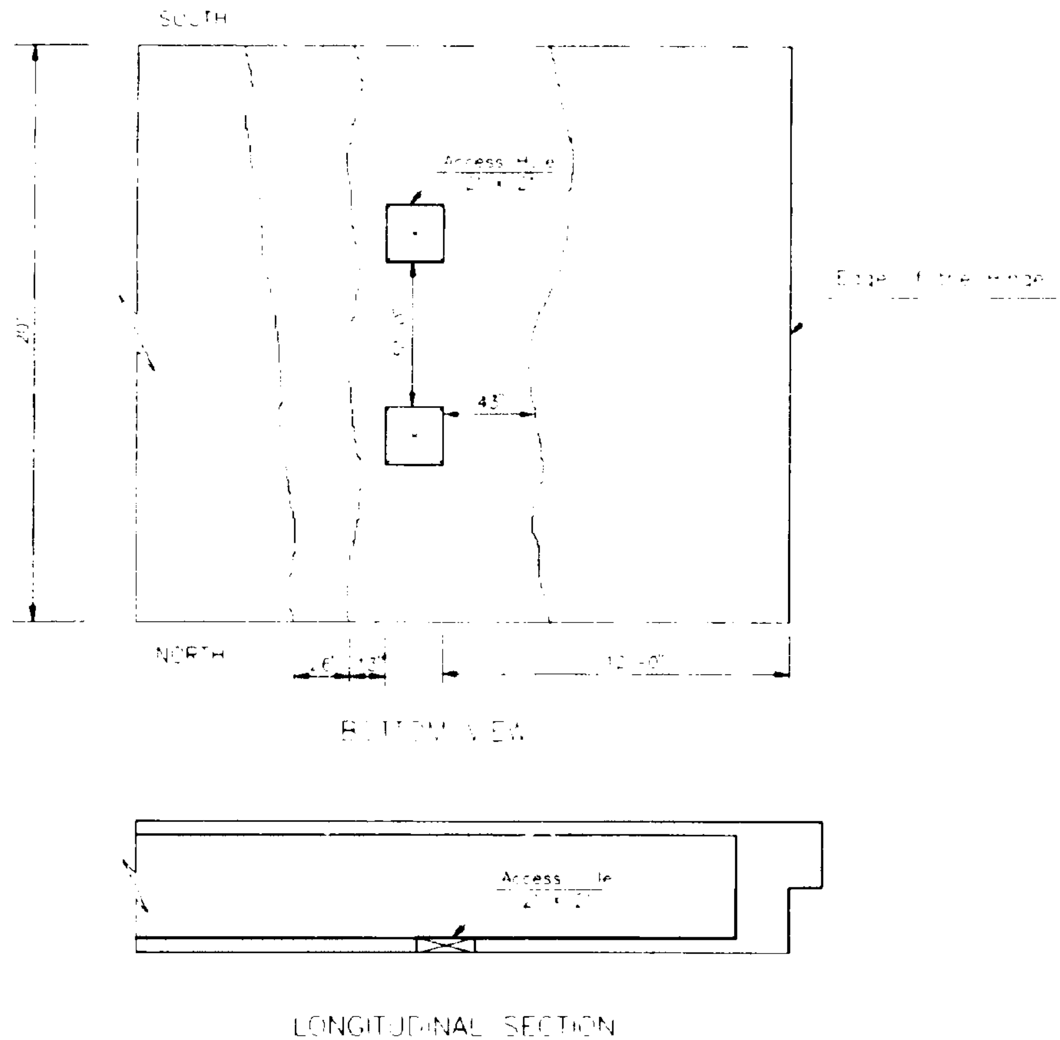


Fig. 2.6 - Crack Pattern at the Bottom Soffit of the 92/101 NE Connector.

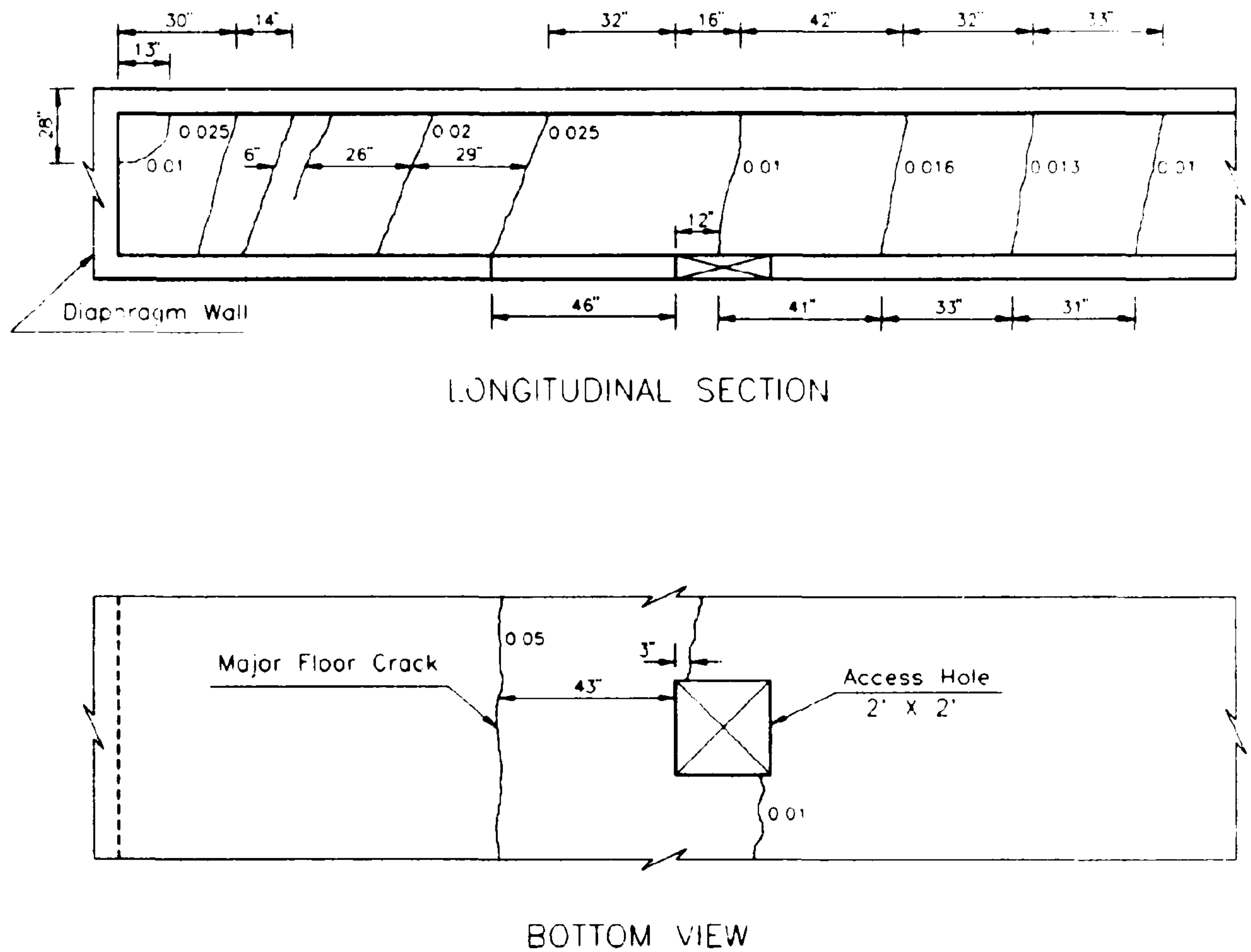


Fig. 2.7 - Girder and Bottom Slab Cracks in the South Girder.

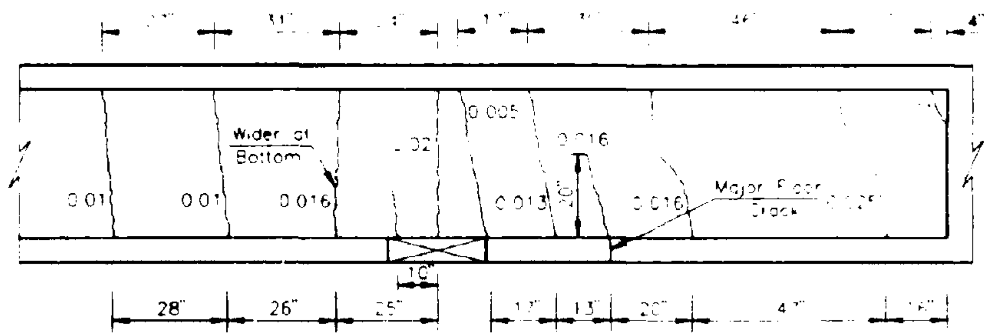


Fig. 2.8 - Girder Cracks in the North Girder.

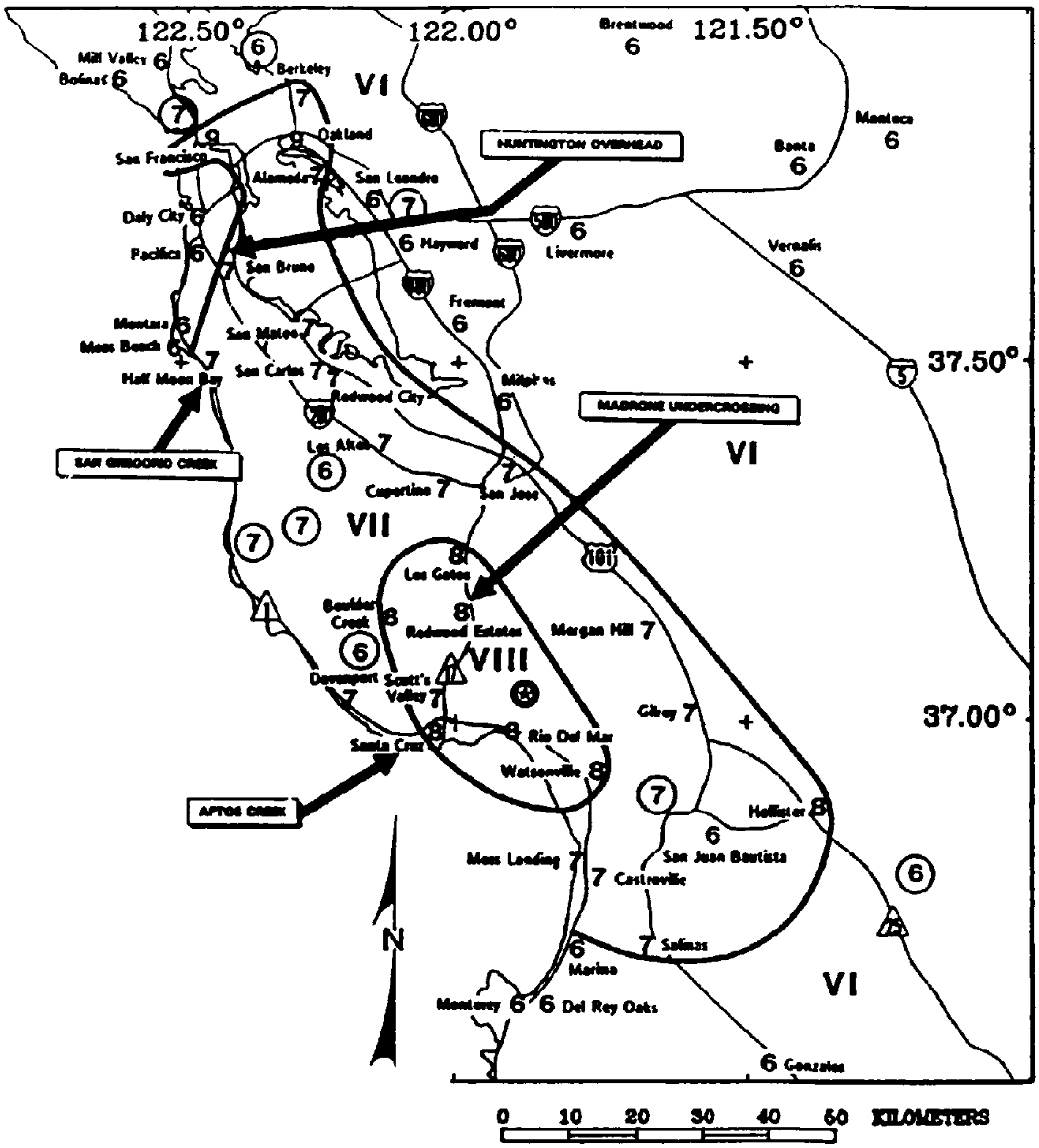


Fig. 3.1 - Location of Bridges Selected for Nonlinear Analysis (The numbers in the circles and the Roman figures show the Modified Mercalli earthquake intensities [20]).

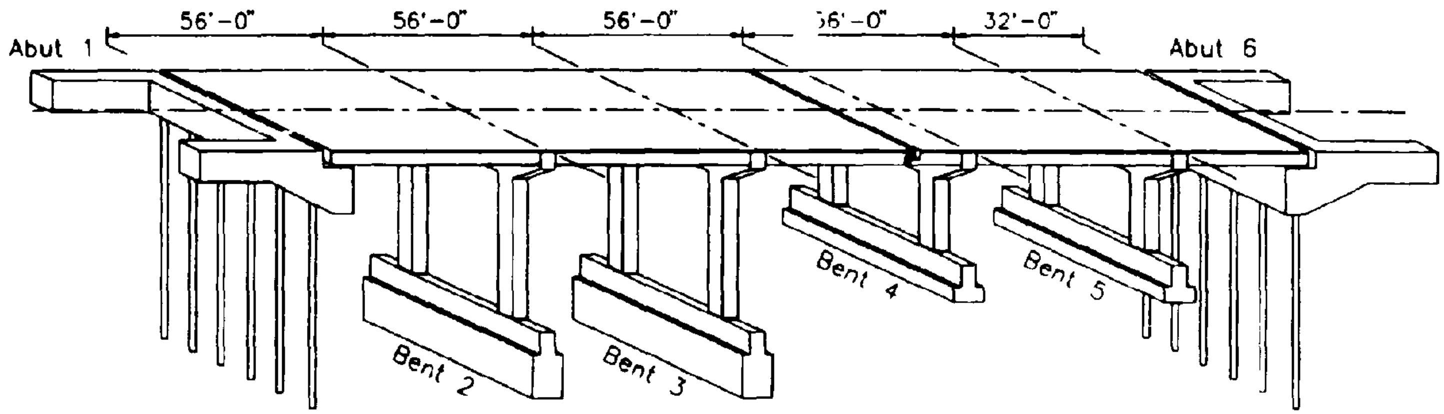


Fig. 3.2 - General view of the Aptos Creek Bridge.

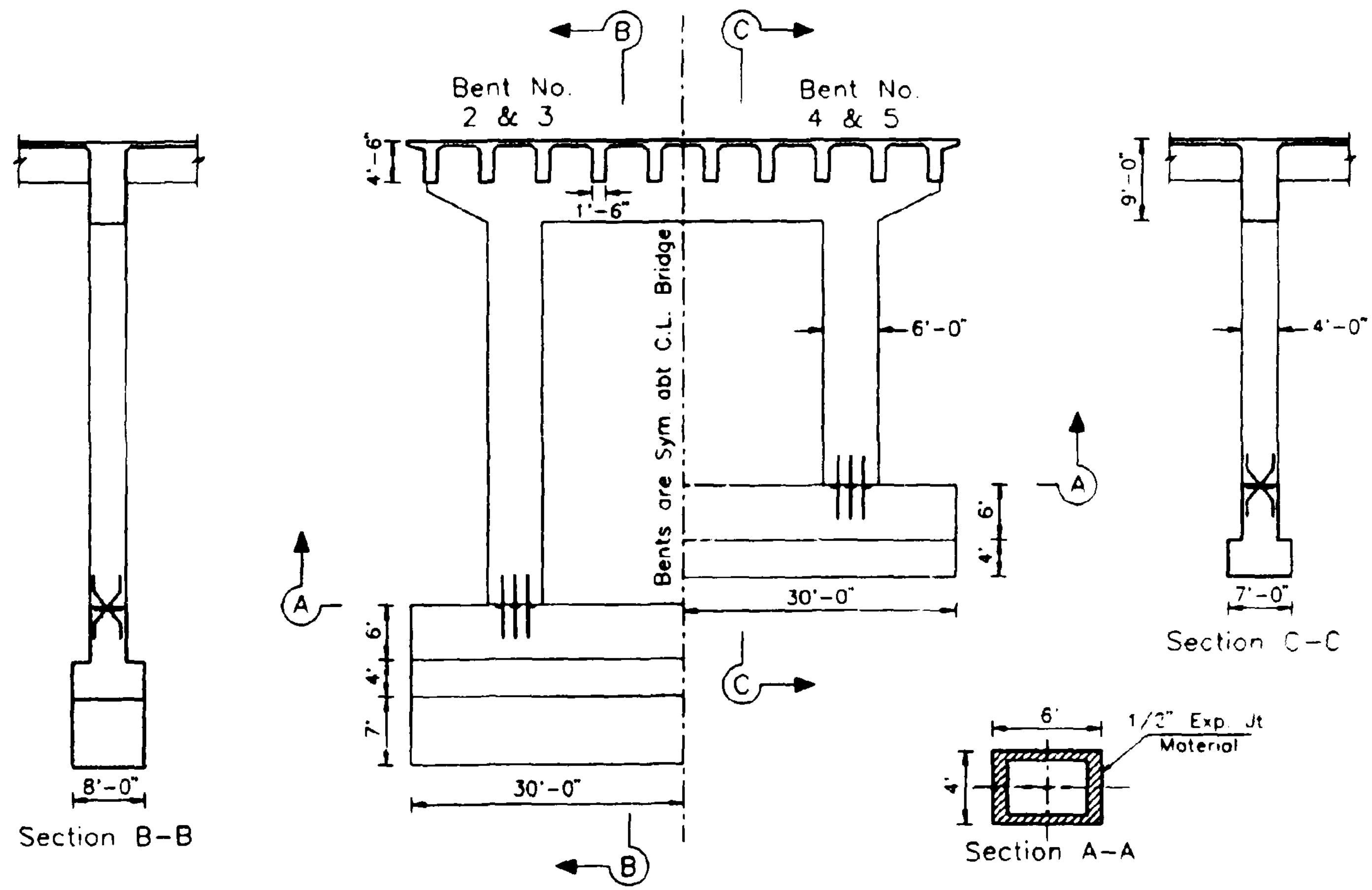


Fig. 3.3 - Cross Sections of the Aptos Creek Bridge.

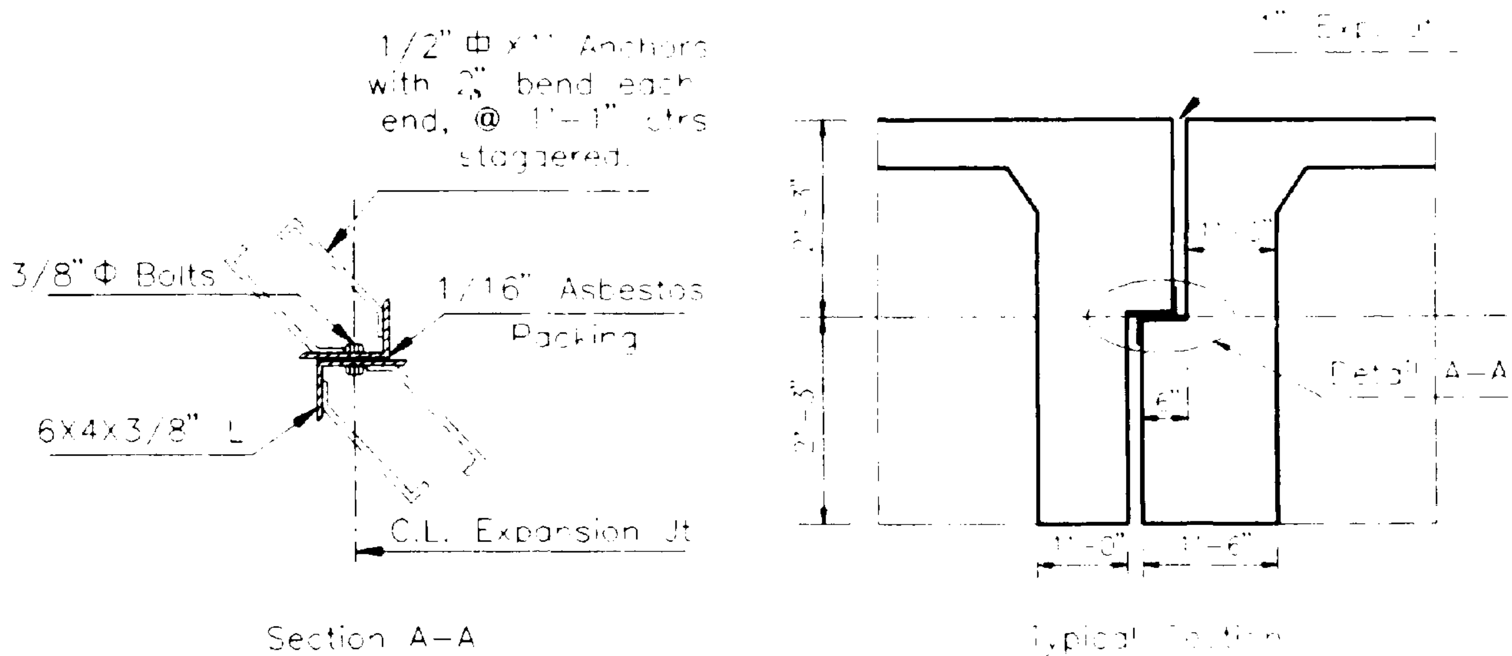


Fig. 3.4 - Detail of the Intermediate Hinge in Aptos.

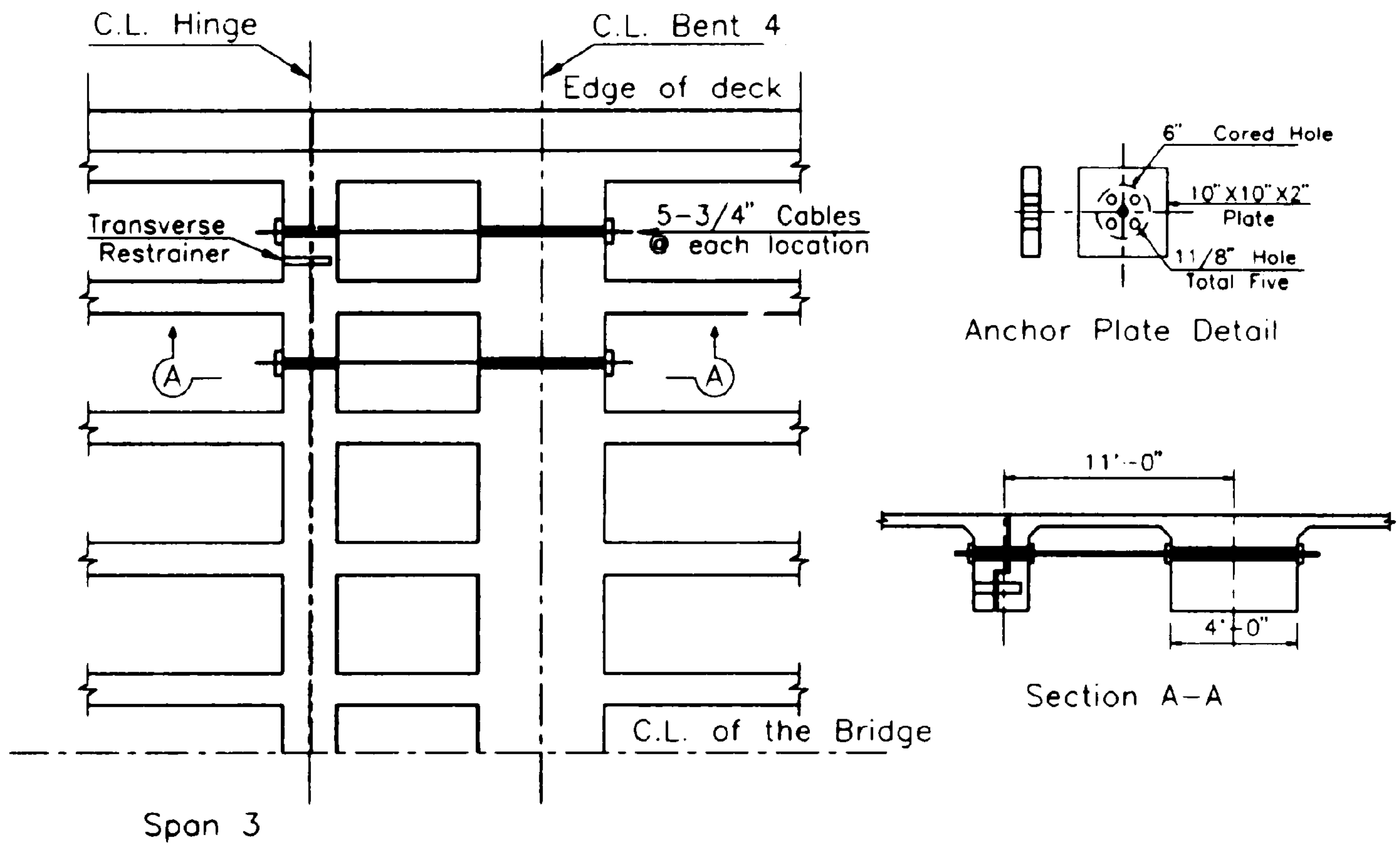


Fig. 3.5 - Details of Restrainer Retrofit in the Intermediate Hinge in Aptos.

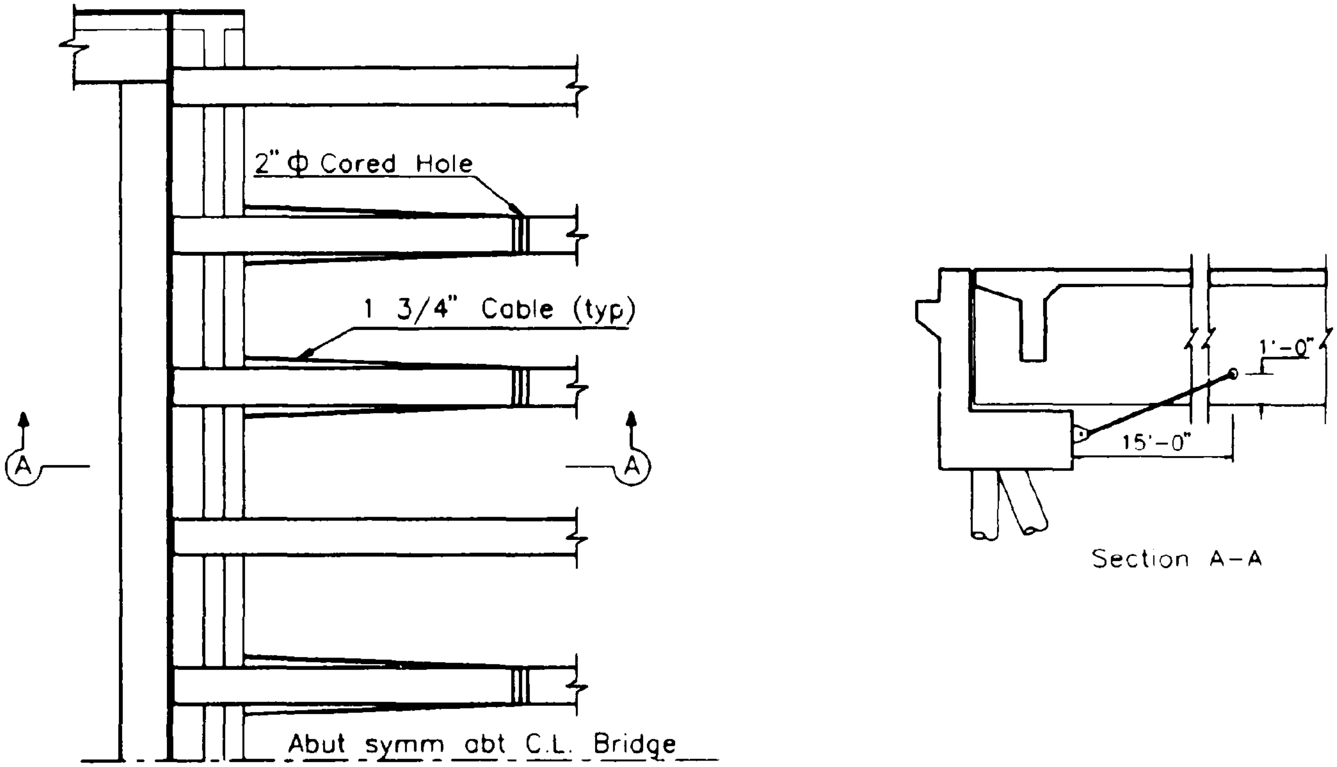


Fig. 3.6 - Details of Restrainer Retrofit at Abutments in Aptos.

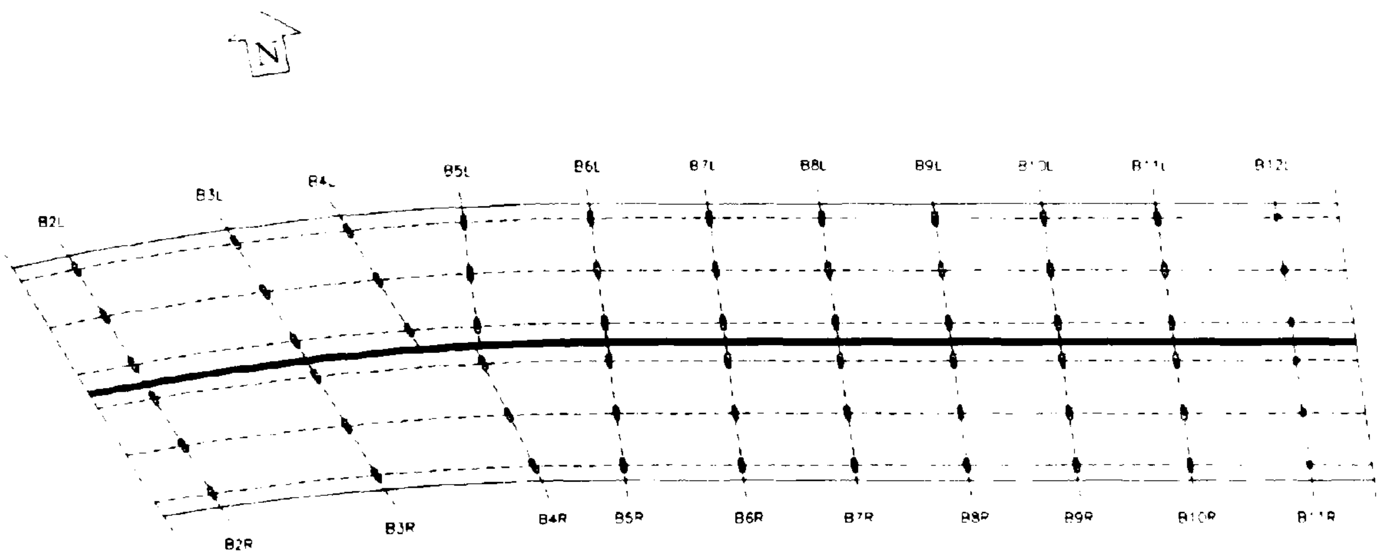


Fig. 3.7.a - Plan View of the Huntington Ave. Overhead.

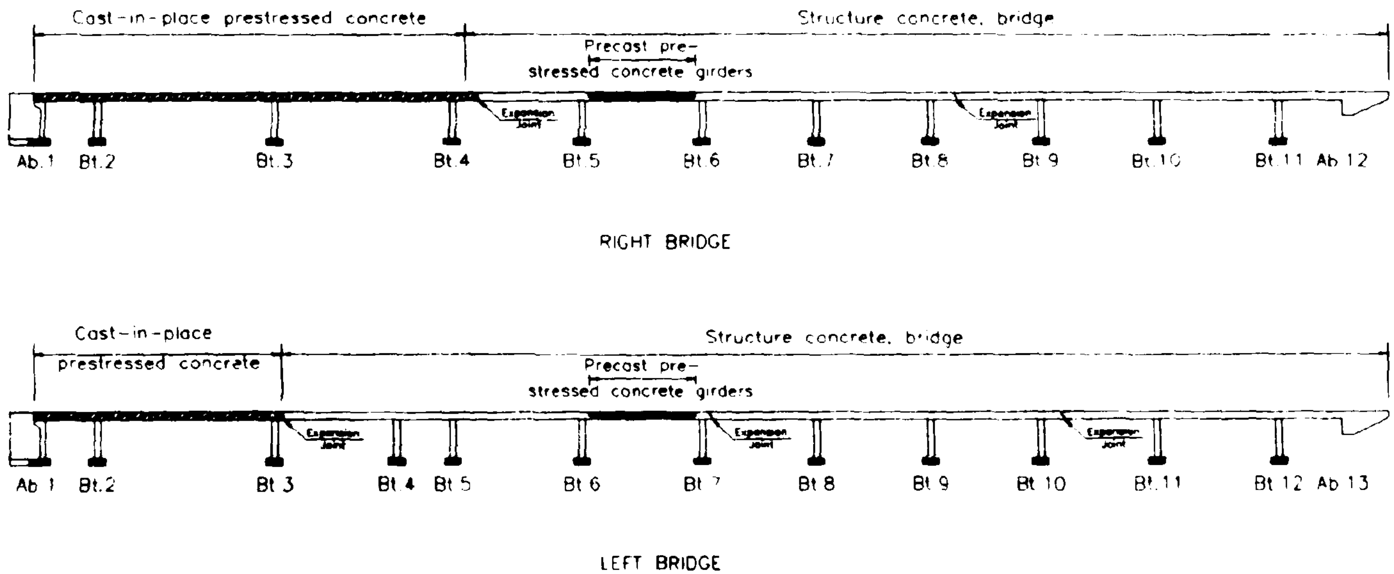


Fig. 3.7.b - Elevation View of the Huntington Ave. Overhead.

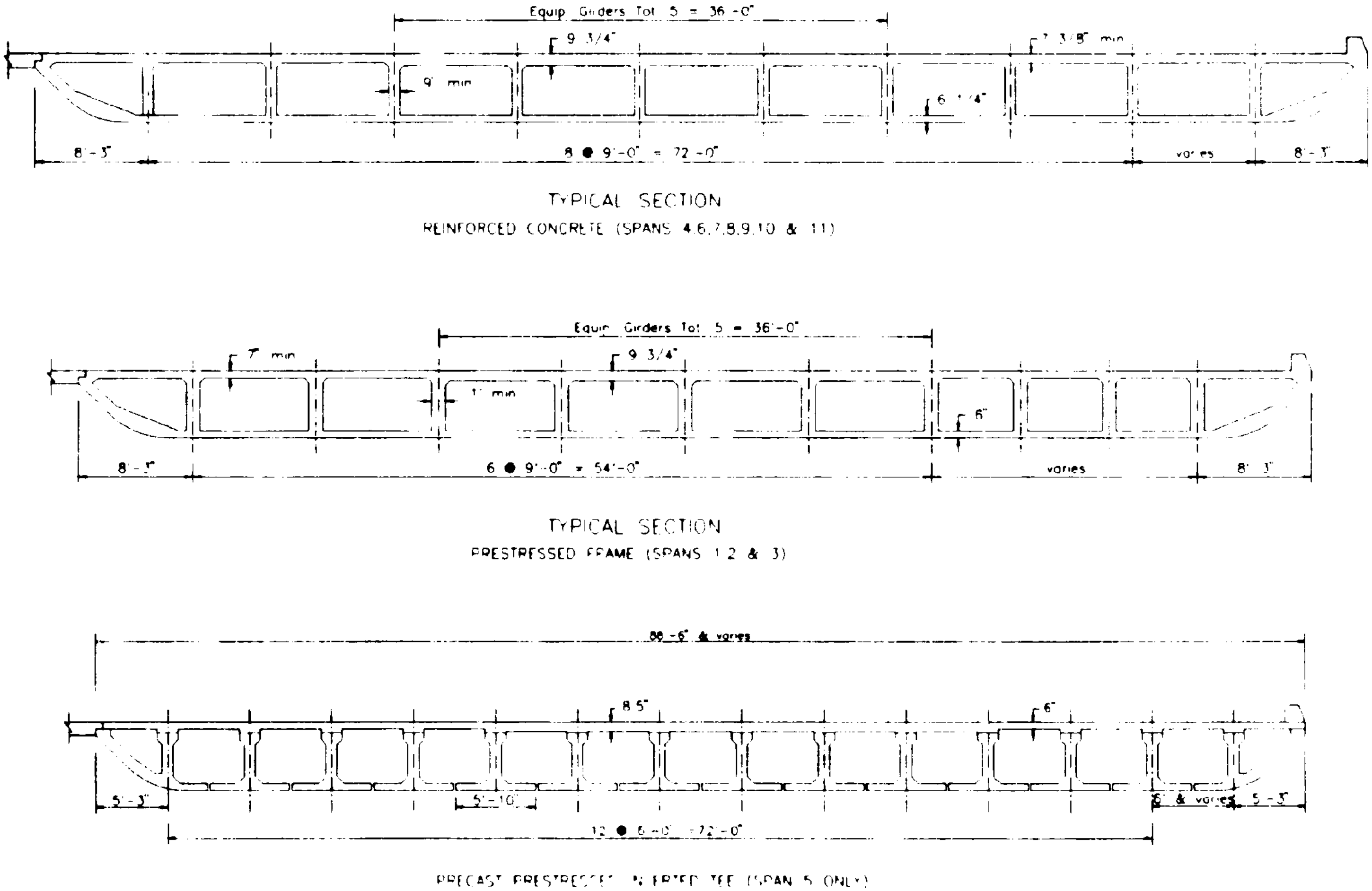


Fig. 3.8 - Superstructure Cross Section of the Huntington Ave. Overhead.

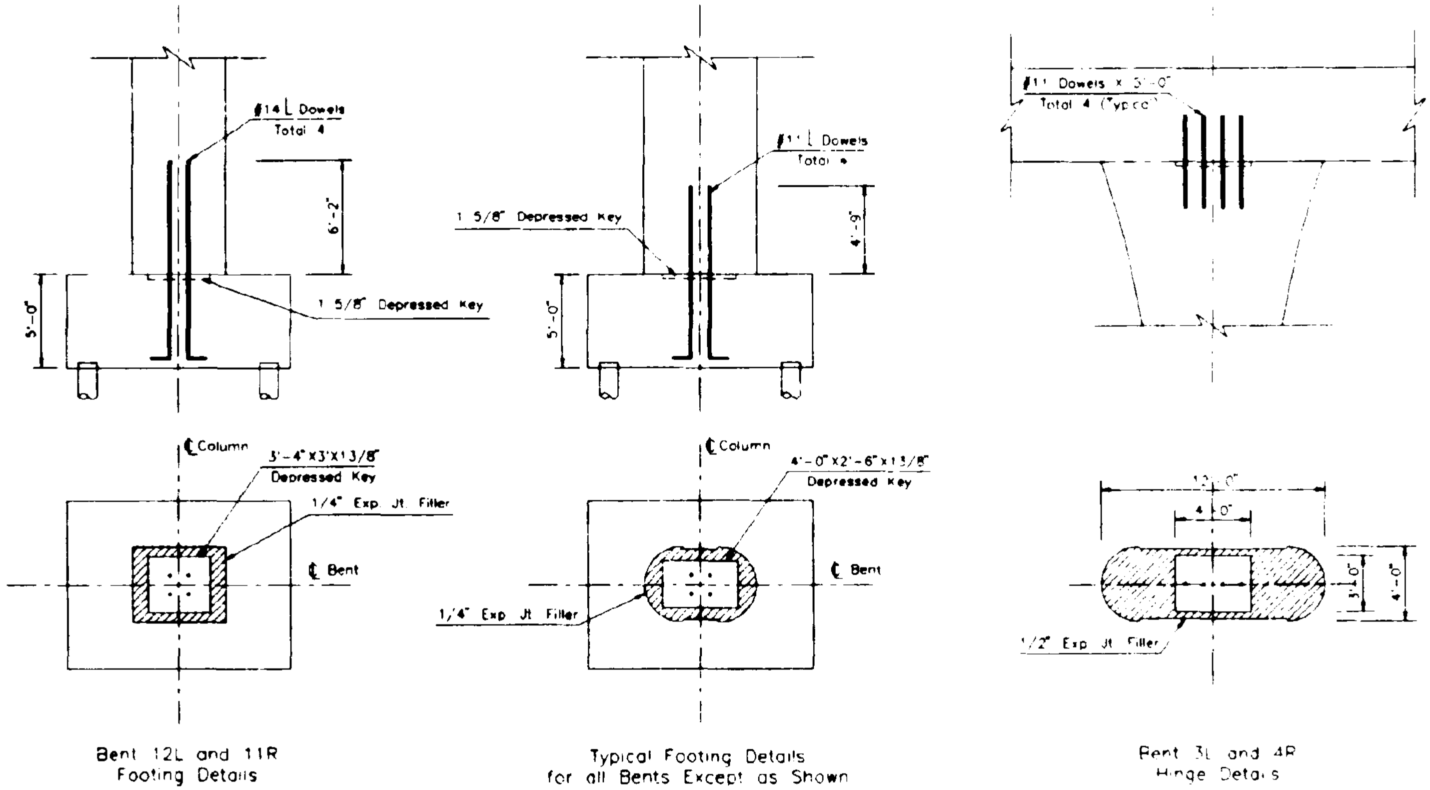


Fig. 3.9 - Pier Connection Details of the Huntington Ave. Overhead.

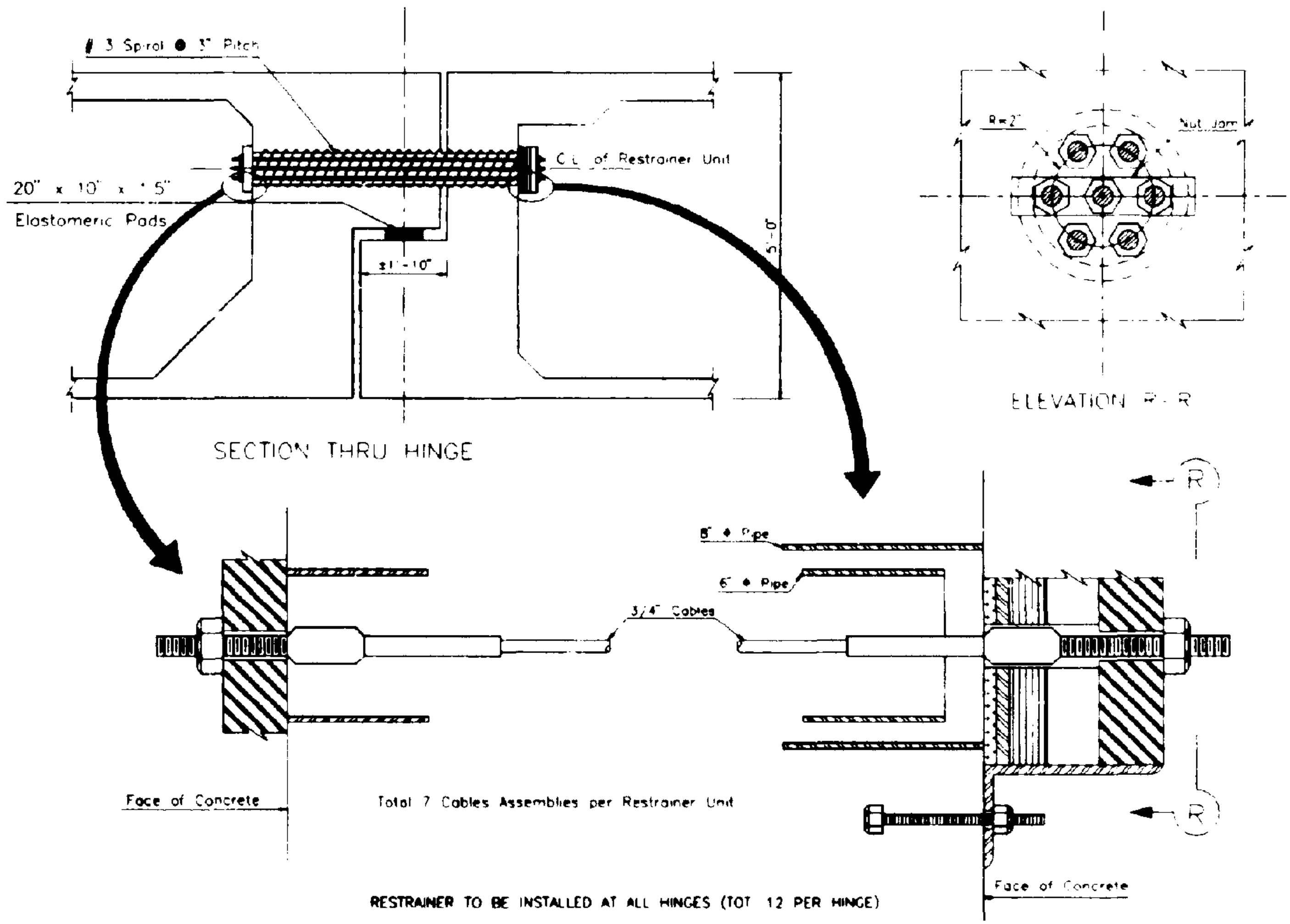


Fig. 3.10 - Hinge Details of the Huntington Ave. Overhead.

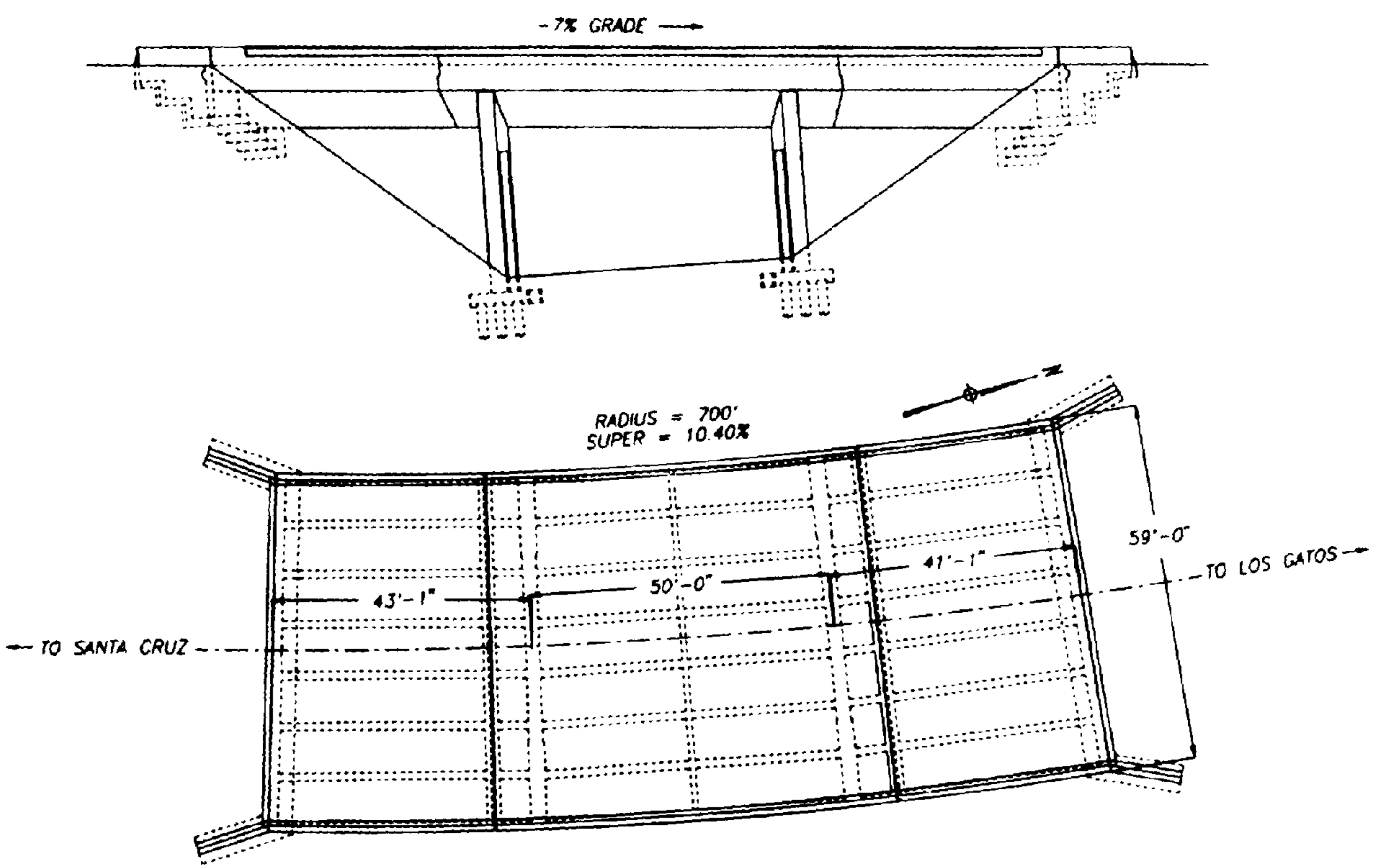


Fig. 3.11 - General Views of Madrone Undercrossing.

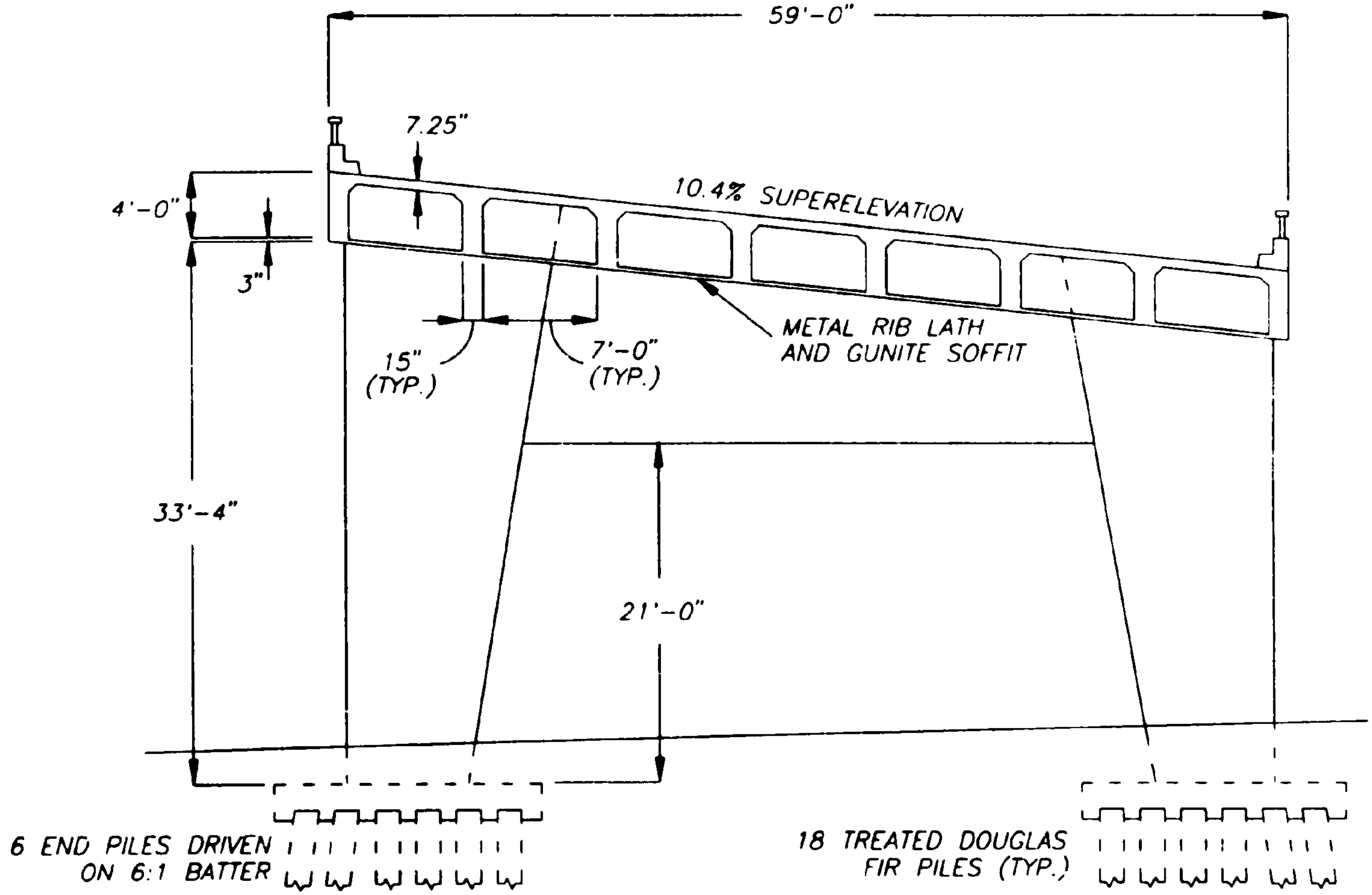


Fig. 3.12 - Transverse Section of Madrone Undercrossing.

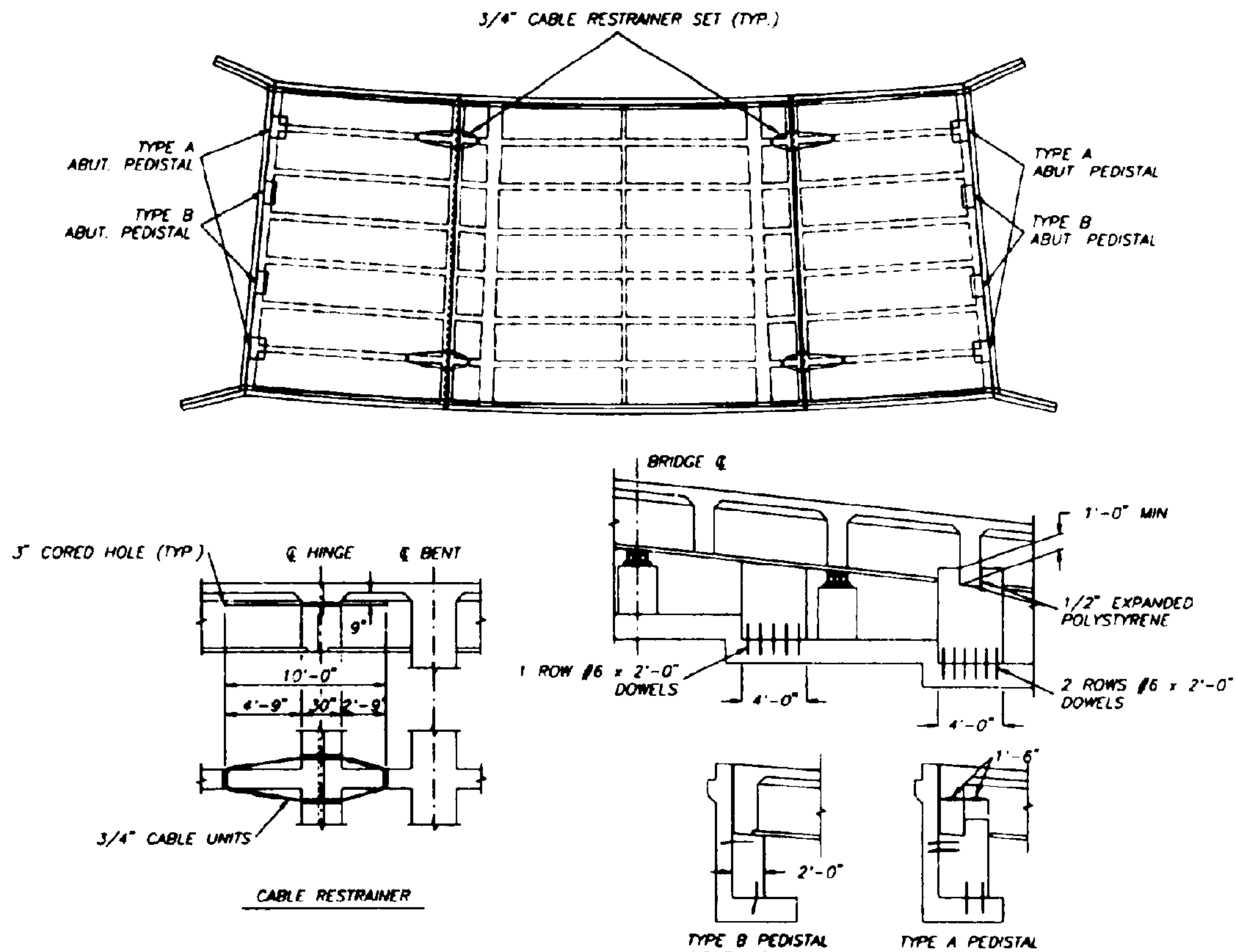
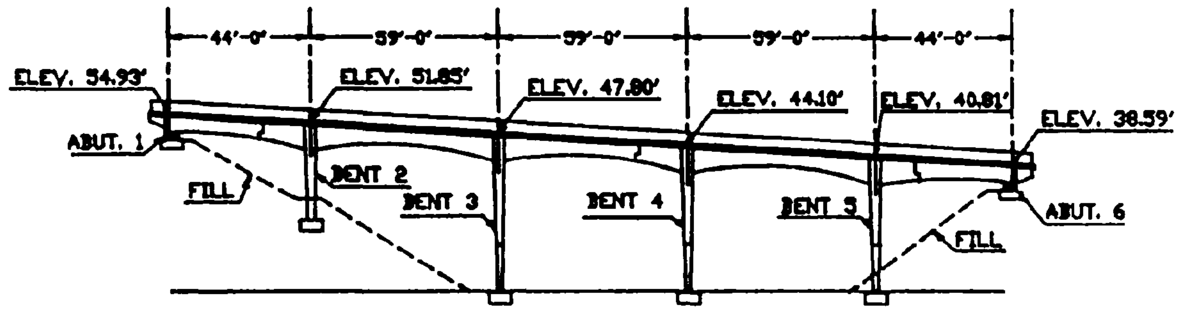
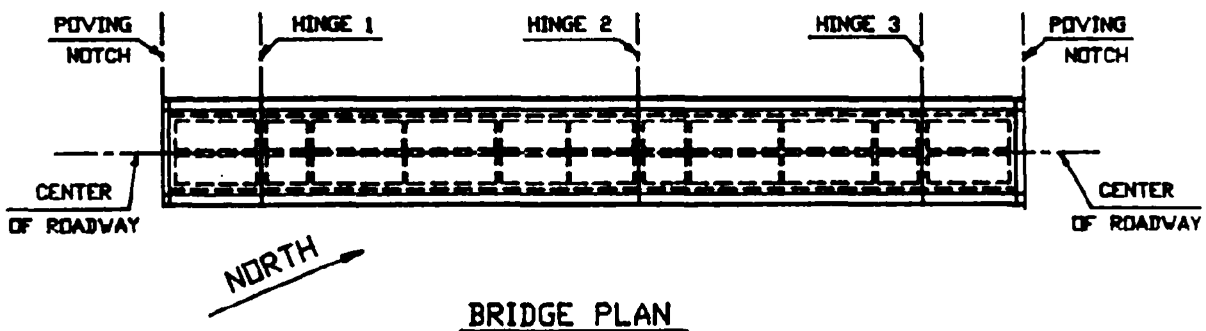


Fig. 3.13 - Seismic Retrofit Details for Madrone Undercrossing.



BRIDGE ELEVATION



BRIDGE PLAN

Fig. 3.14 - General Views of San Gregorio Bridge.

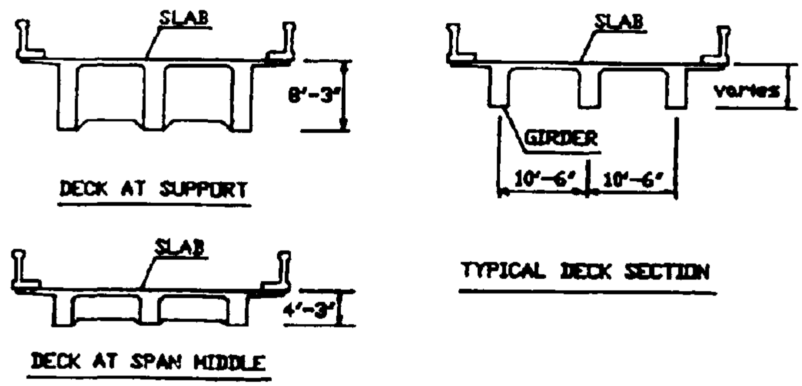


Fig. 3.15 - Transverse Sections of San Gregorio Bridge superstructure.

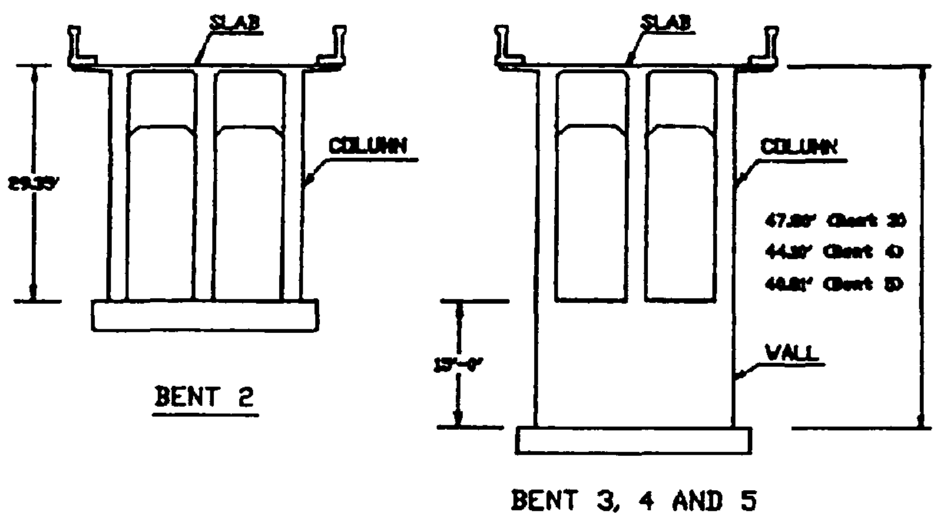


Fig. 3.16 - Bent Elevations for San Gregorio Bridge.

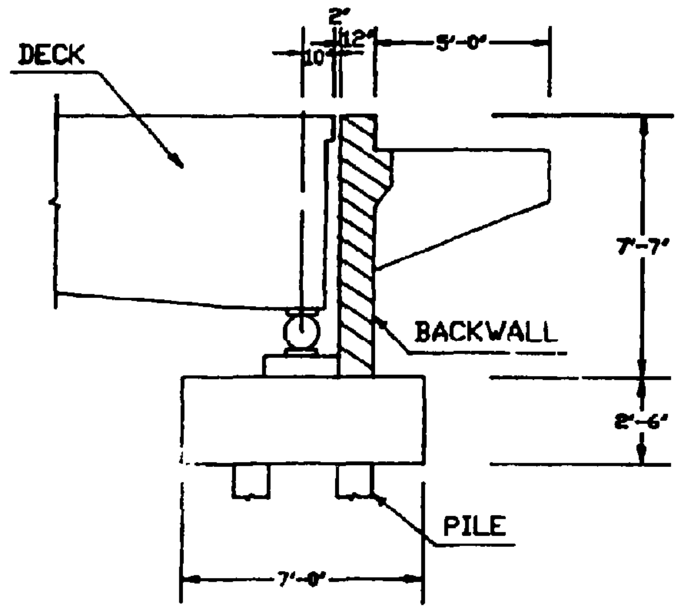


Fig. 3.17 - Abutment Cross Section in San Gregorio Bridge.

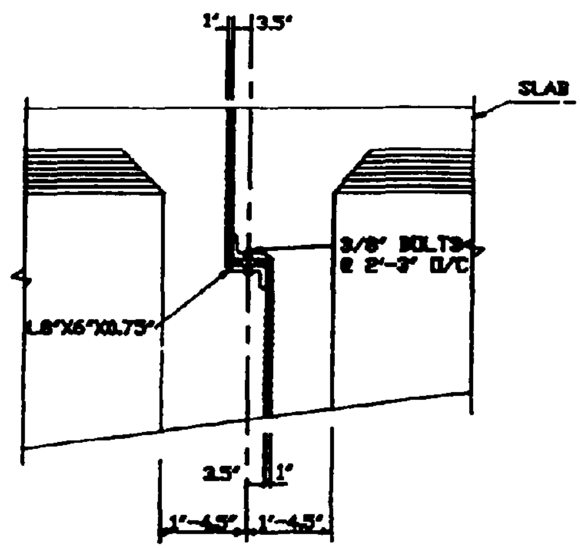


Fig. 3.18 - Detail of the Intermediate Hinges in San Gregorio Bridge.

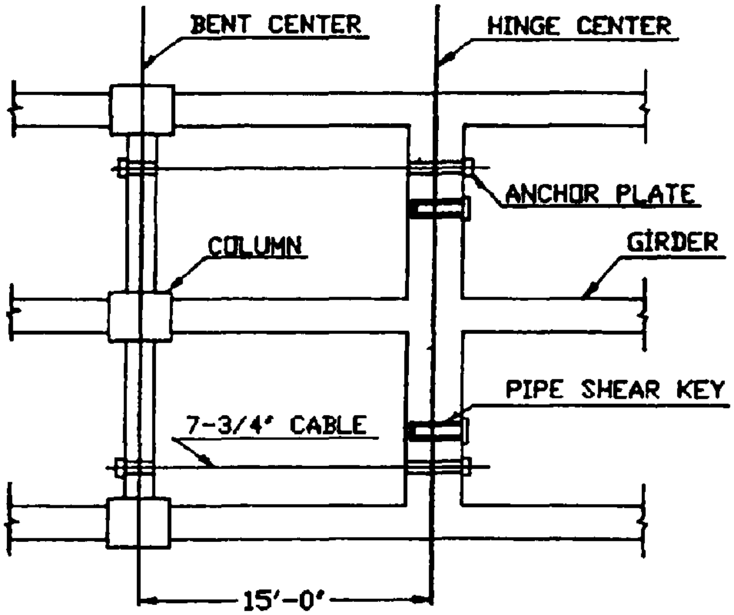


Fig. 3.19 - Hinge Restrainer Retrofits in San Gregorio Bridge.

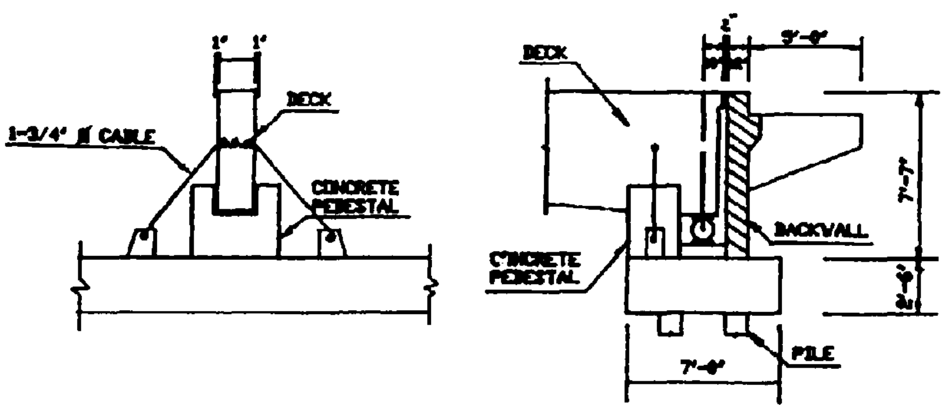


Fig. 3.20 - Abutment Retrofits in San Gregorio Bridge.

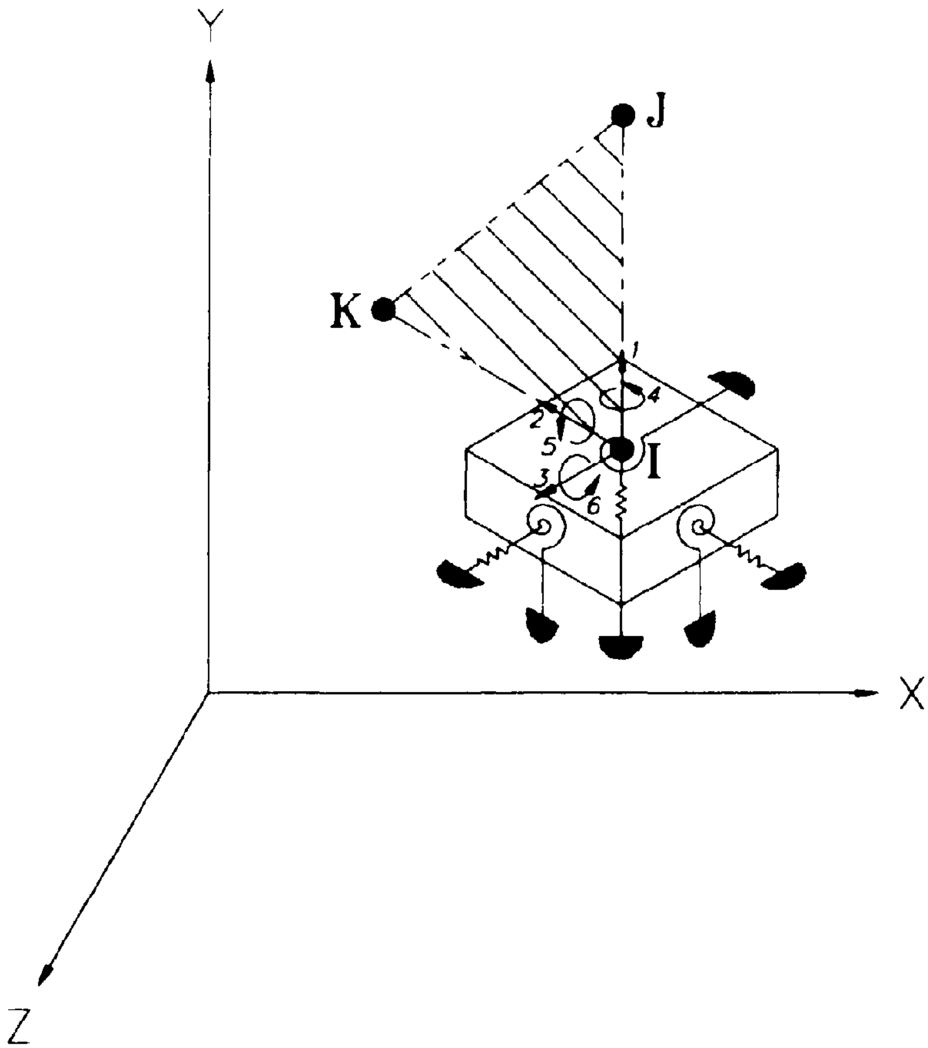


Fig. 3.21 - N ΞABS Boundary Spring System.

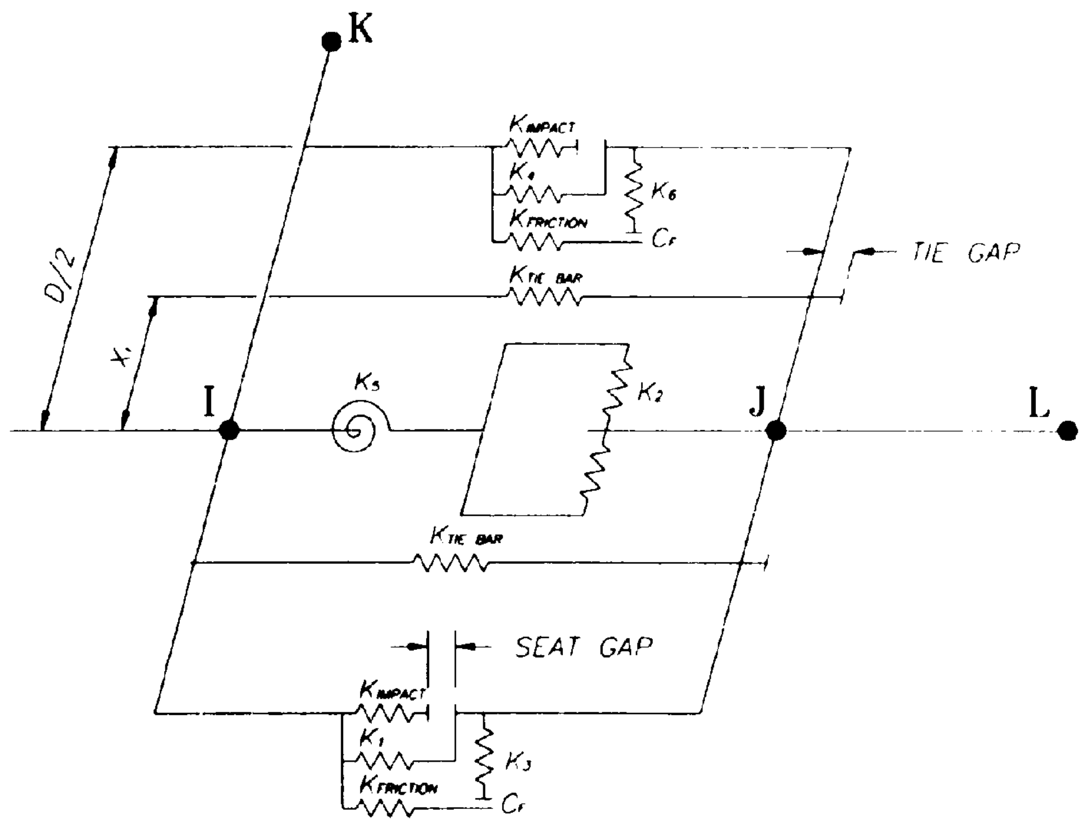


Fig. 3.22 - NEABS Expansion joint System.

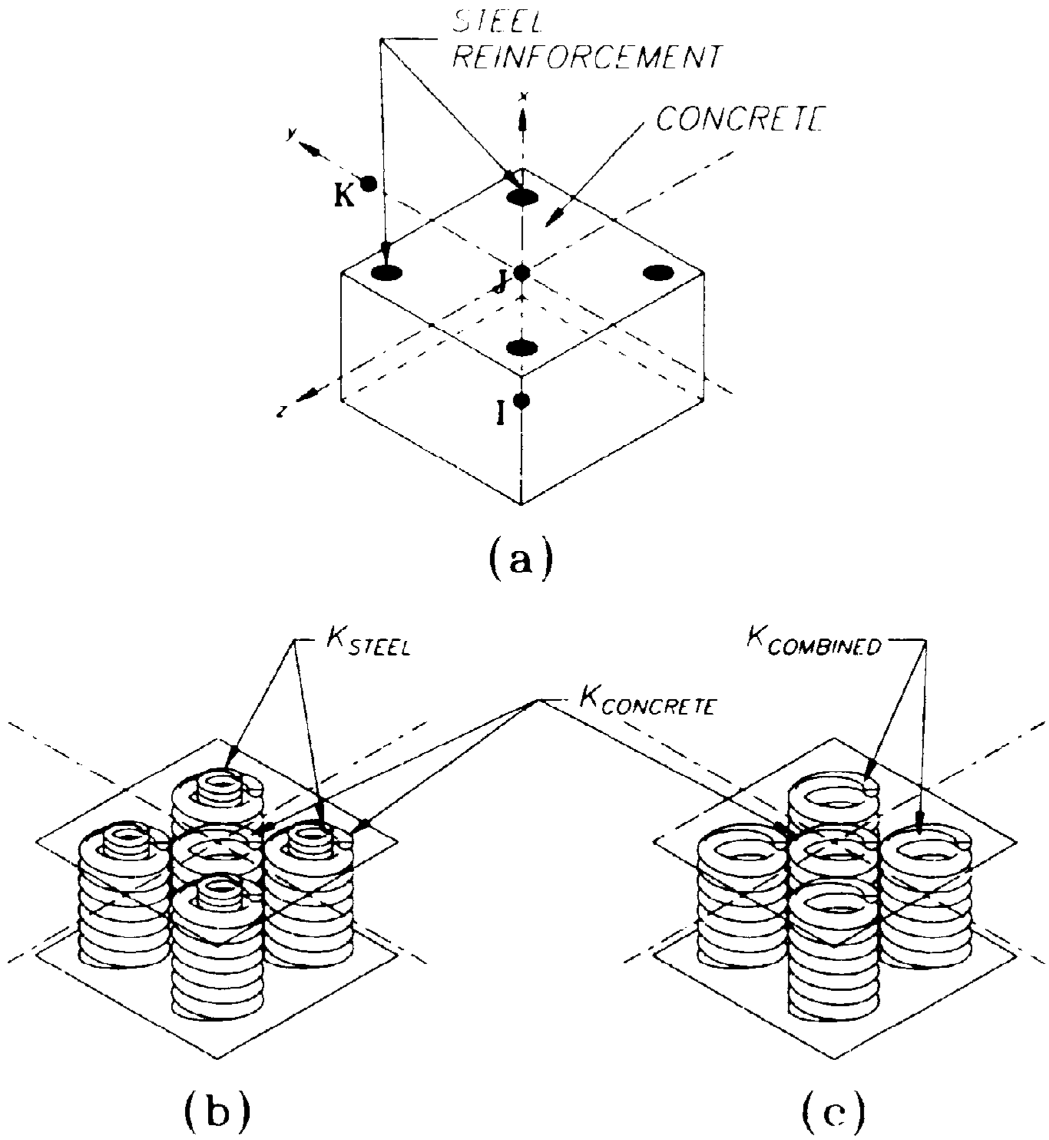


Fig. 3.23 - The Degrading Column (5-Spring) Element in NEABS-86.

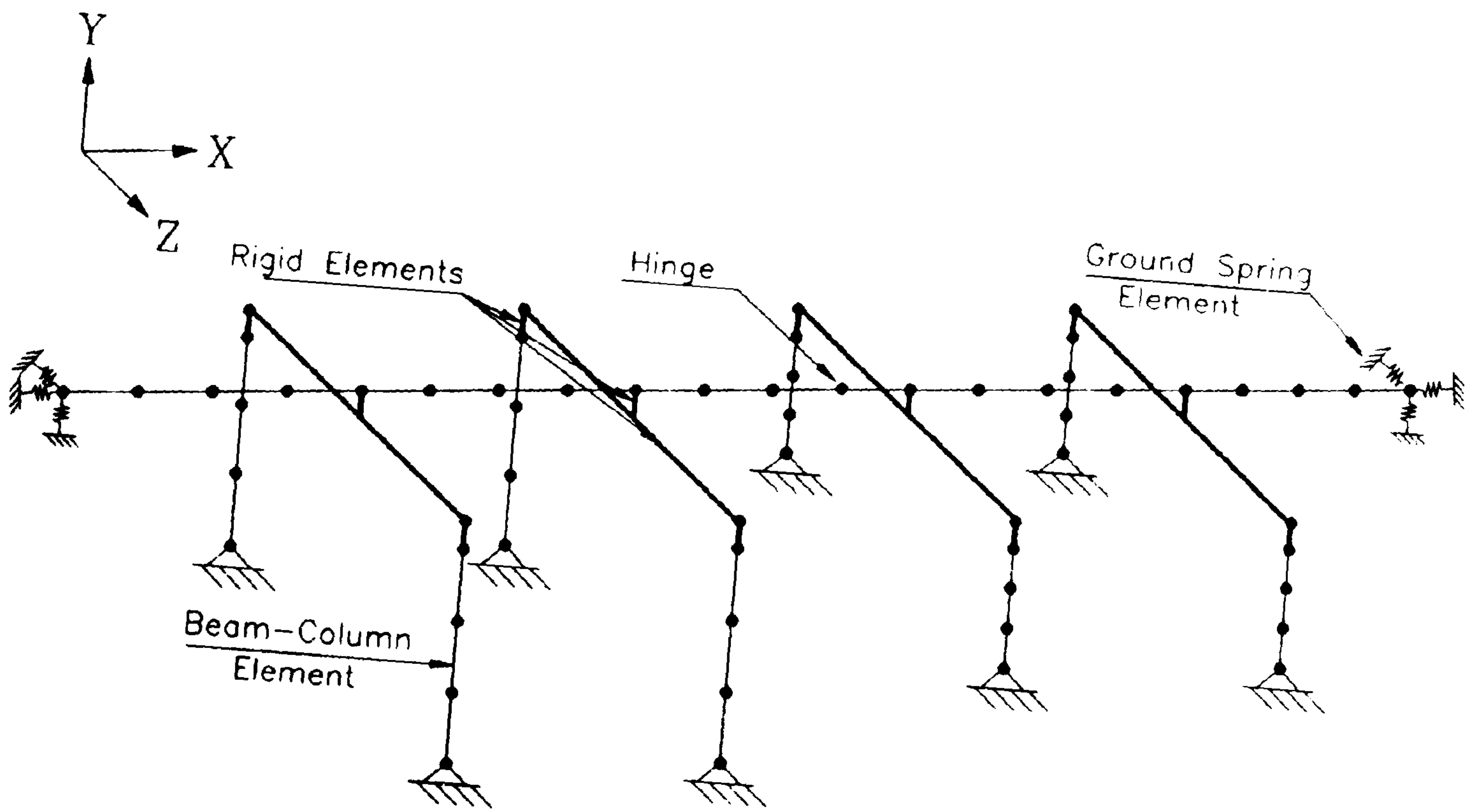


Fig. 3.24 - Images-3D Model of the Aptos Creek Bridge.

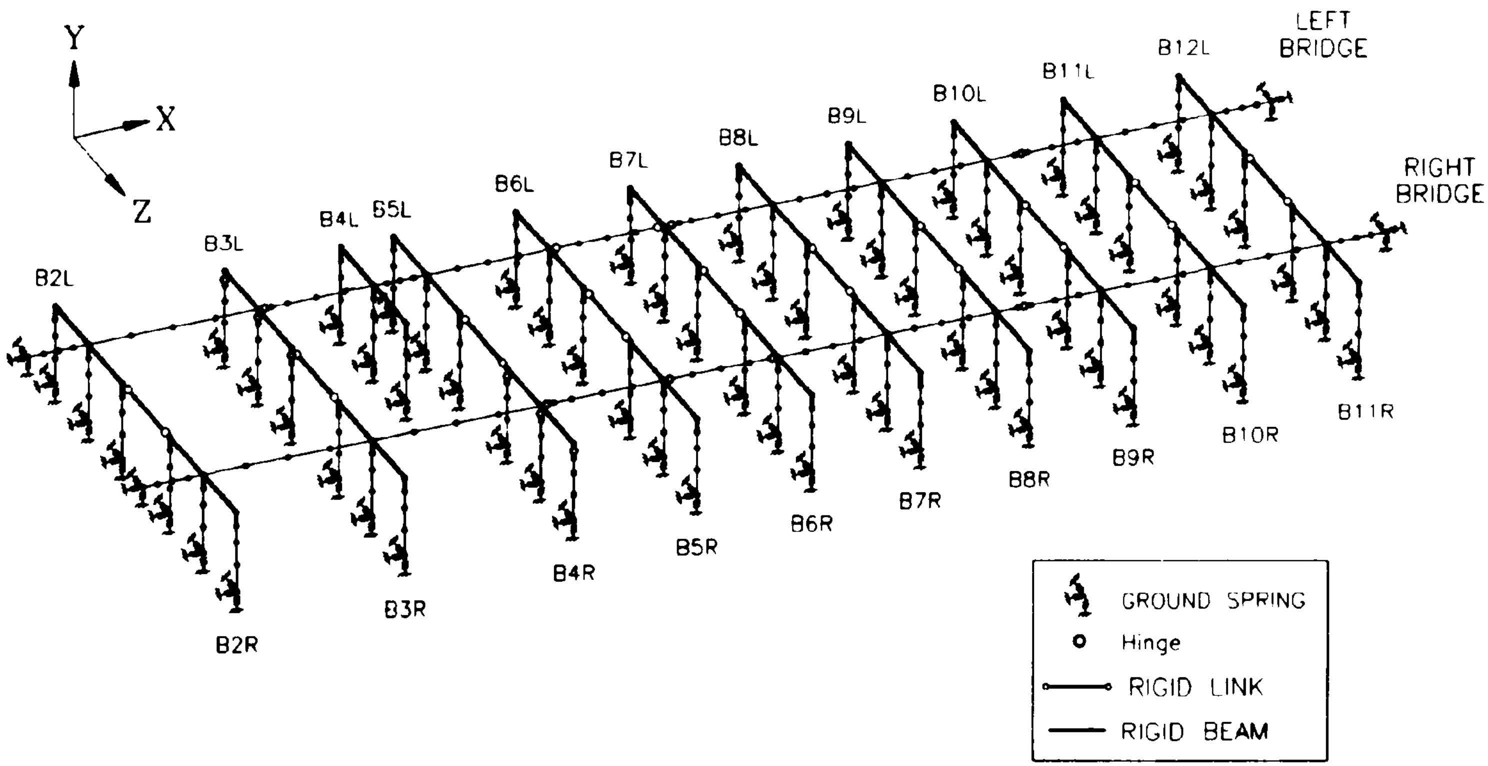


Fig. 3.25 - Images-3D Model of the Huntington Ave. Overhead.

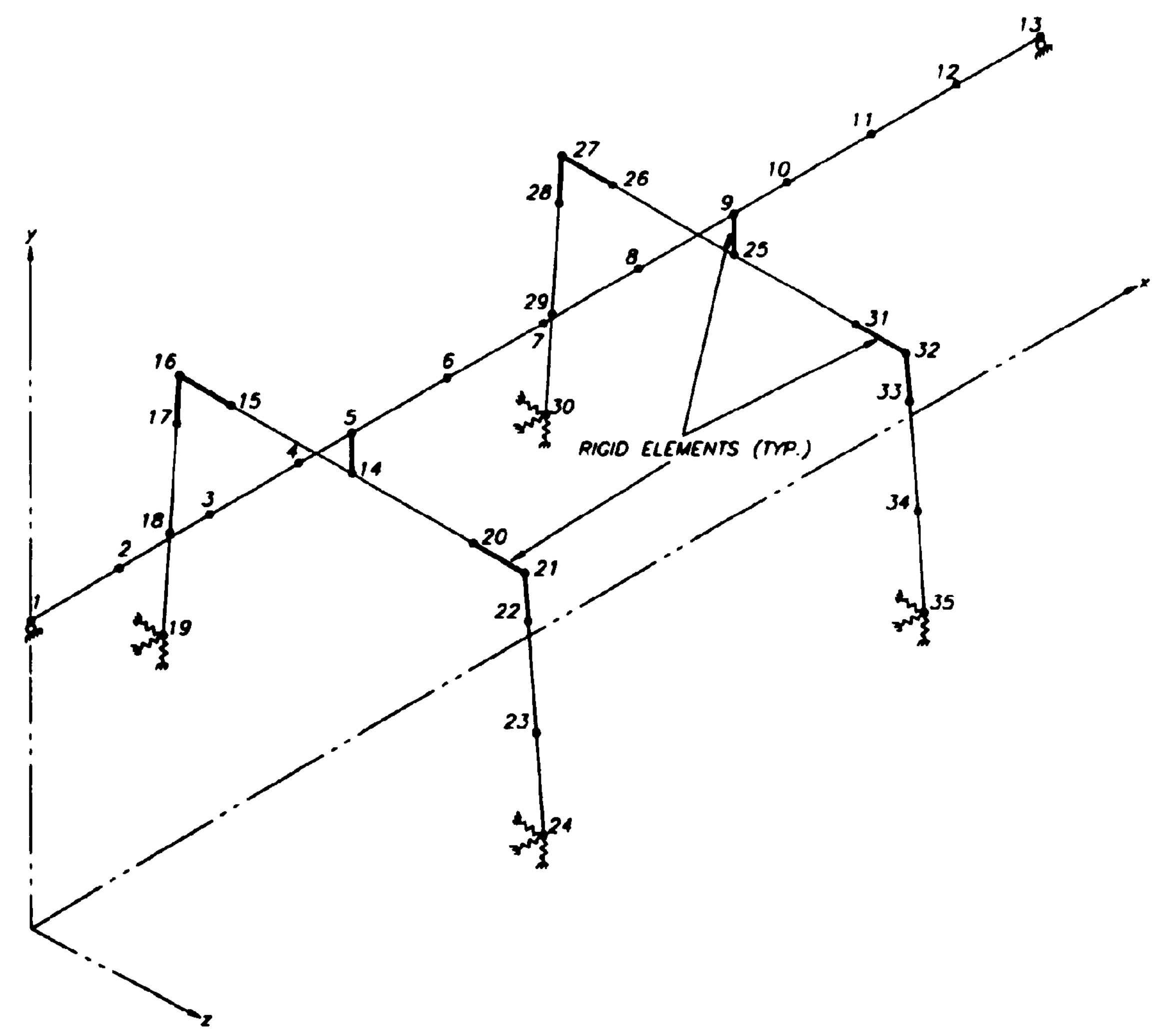


Fig. 3.26 - Images-3D Model of the Madrone Dr. Undercrossing.

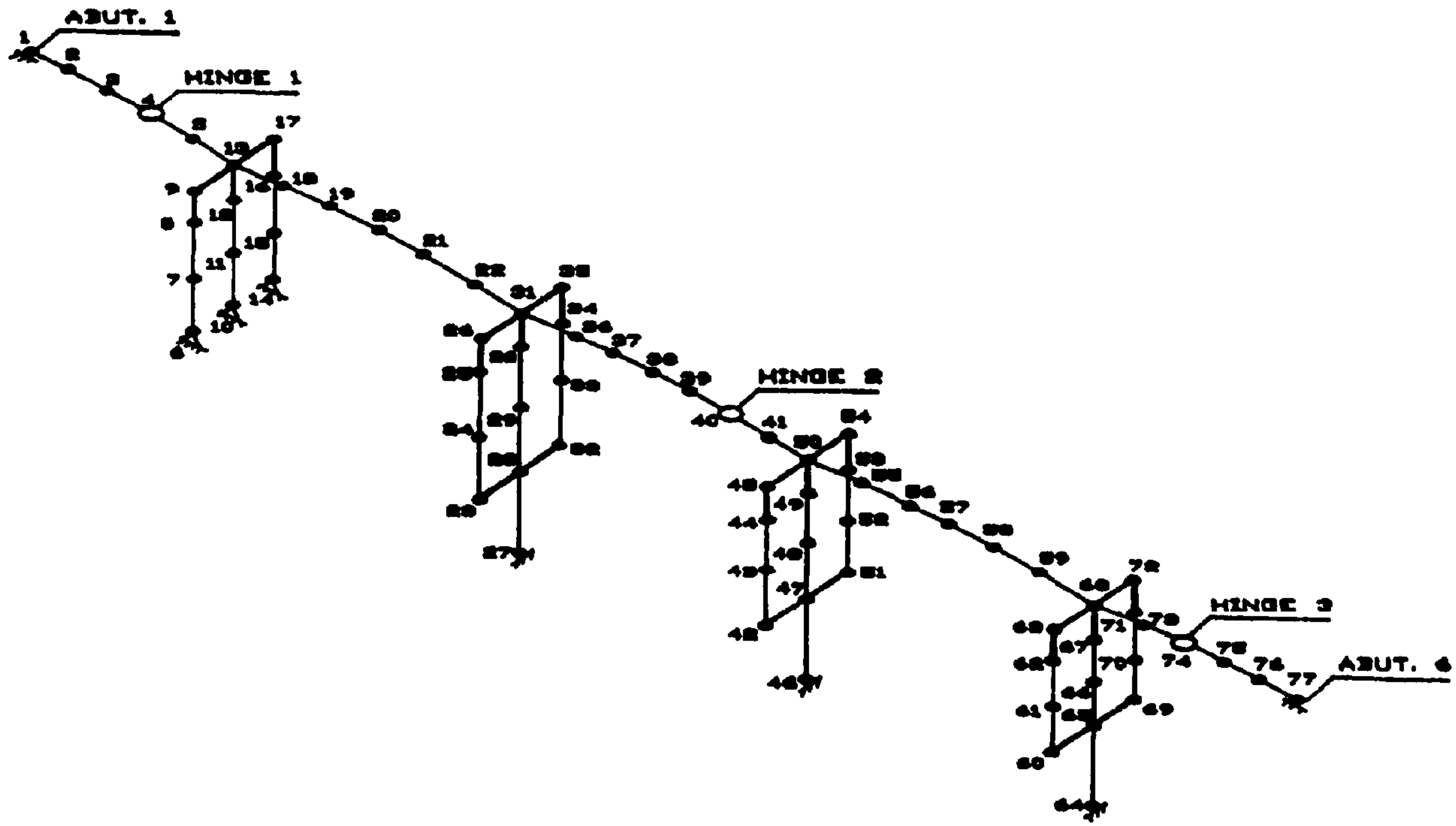


Fig. 3.27 - Images-3D Model of the San Gregorio Creek Bridge.

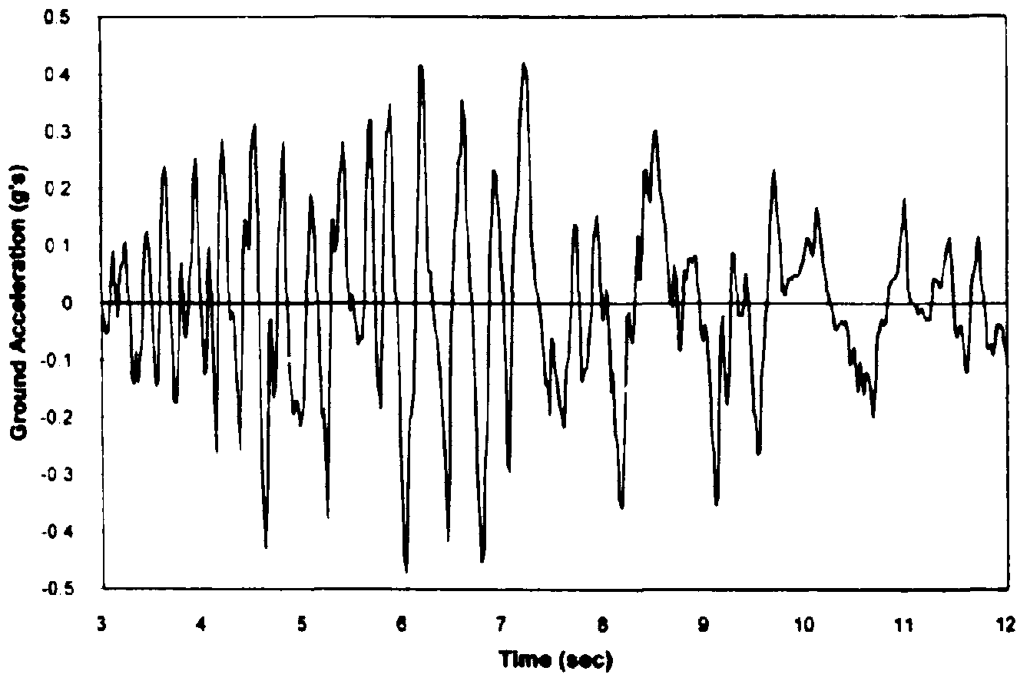


Fig. 3.28 - Input Acceleration History in the Transverse direction of Aptos.

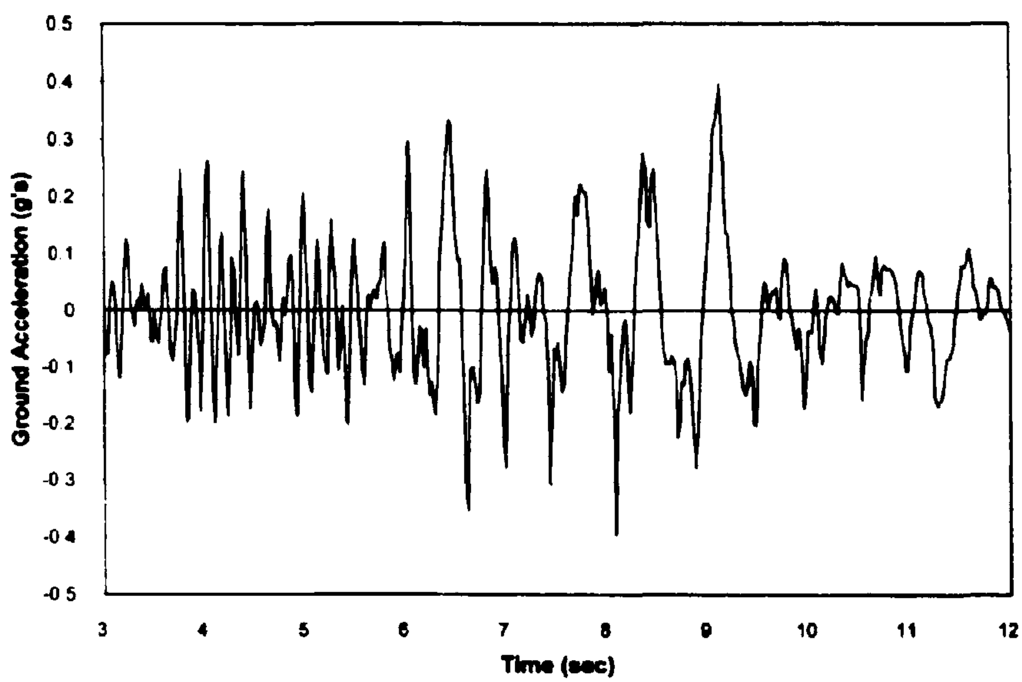


Fig. 3.29 - Input Acceleration History in the Longitudinal direction of Aptos.

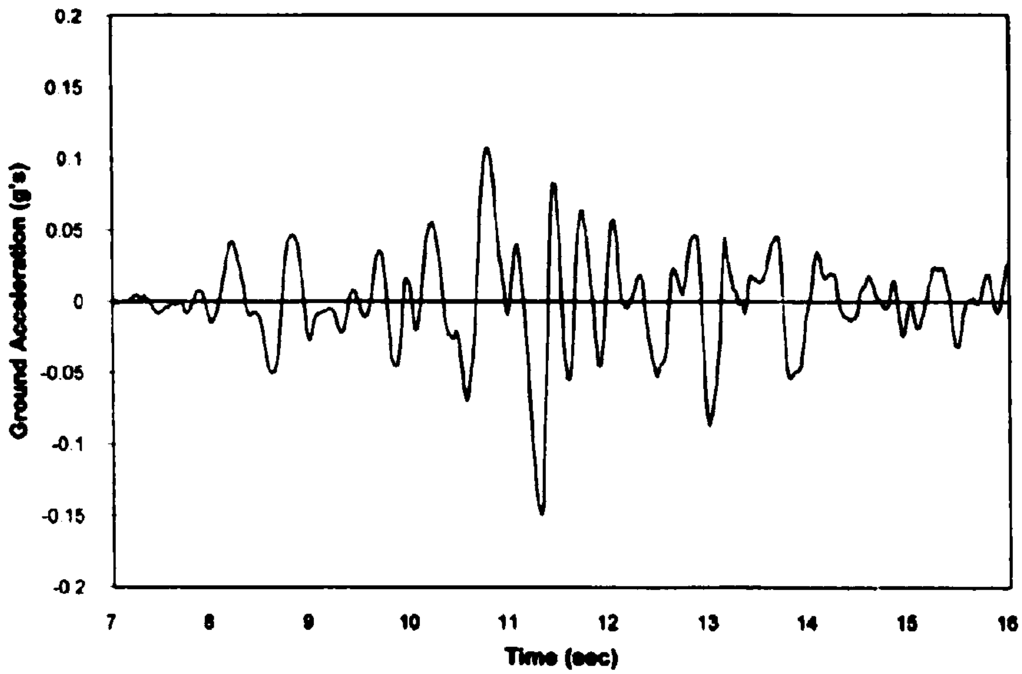


Fig. 3.30 - Input Acceleration History in the Transverse direction of Huntington.

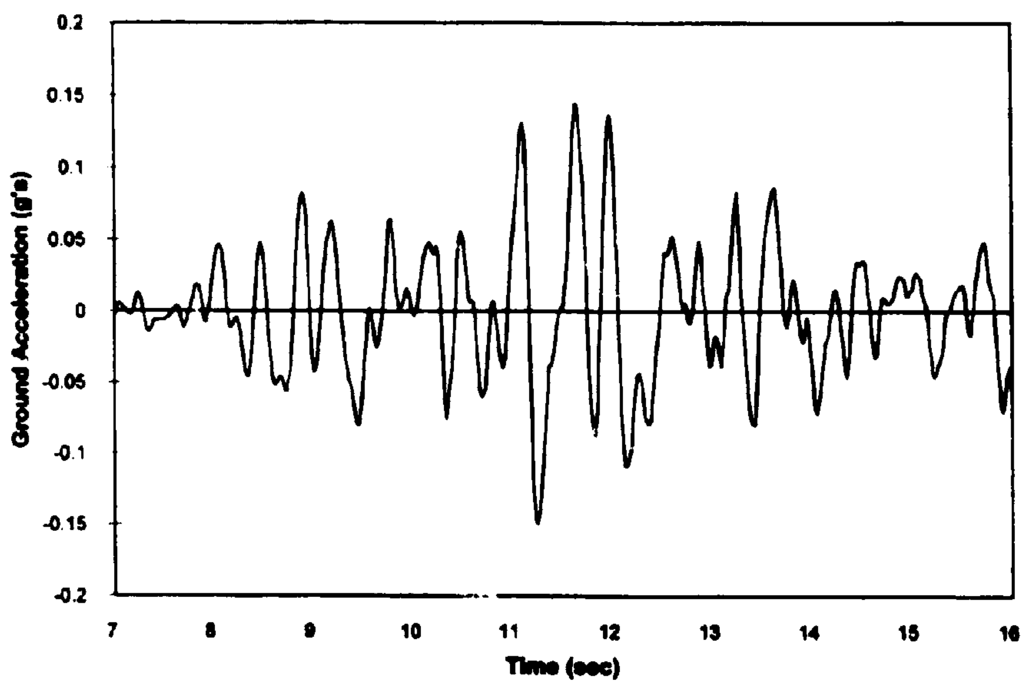


Fig. 3.31 - Input Acceleration History in the Longitudinal direction of Huntington.

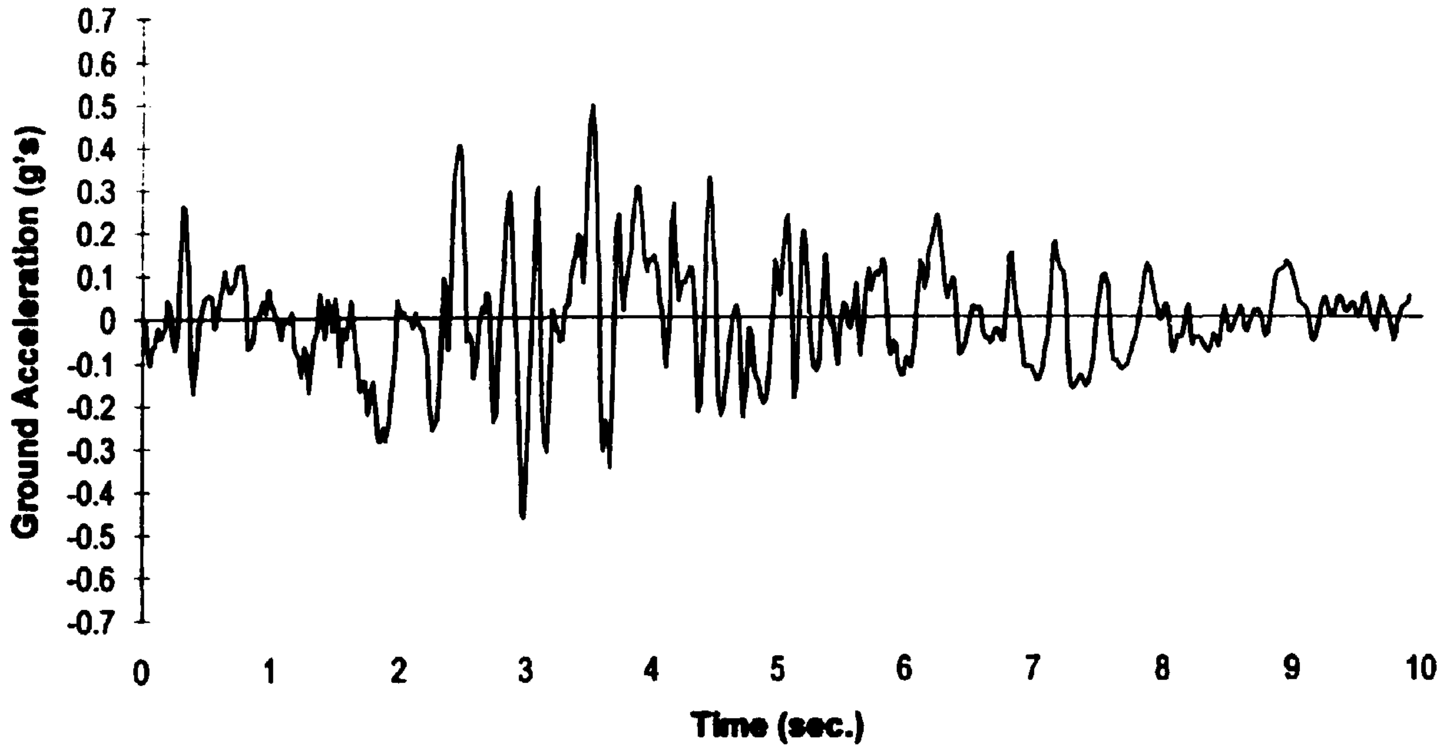


Fig. 3.32 - Input Acceleration History for the Madrone Dr. Undercrossing.

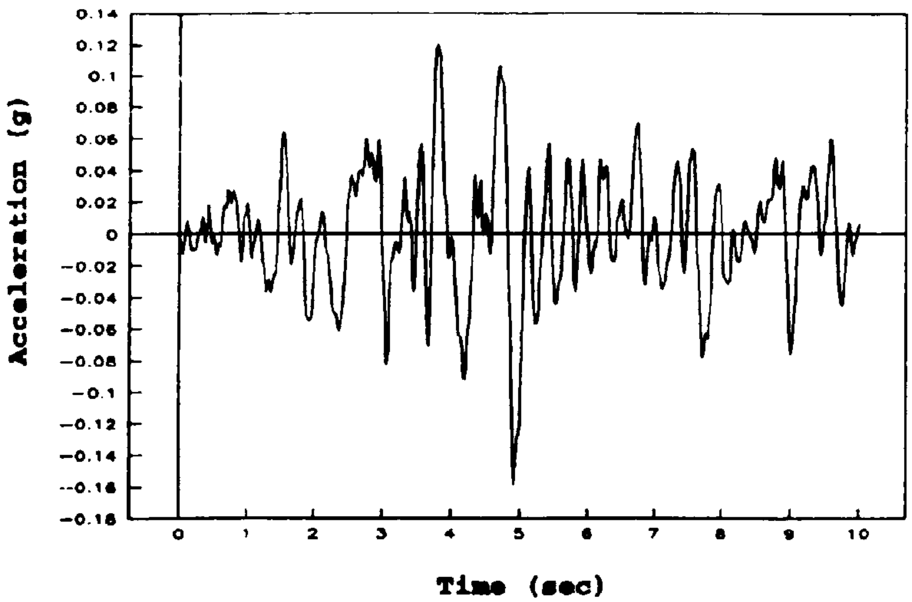


Fig. 3.33 - Input Acceleration History in the Transverse direction of San Gregorio.

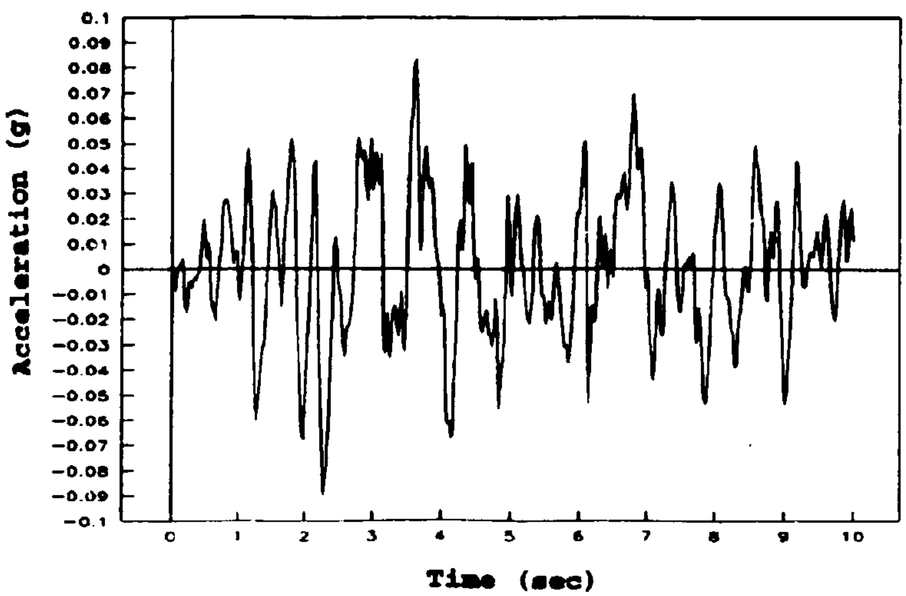


Fig. 3.34 - Input Acceleration History in the Longitudinal direction of San Gregorio.

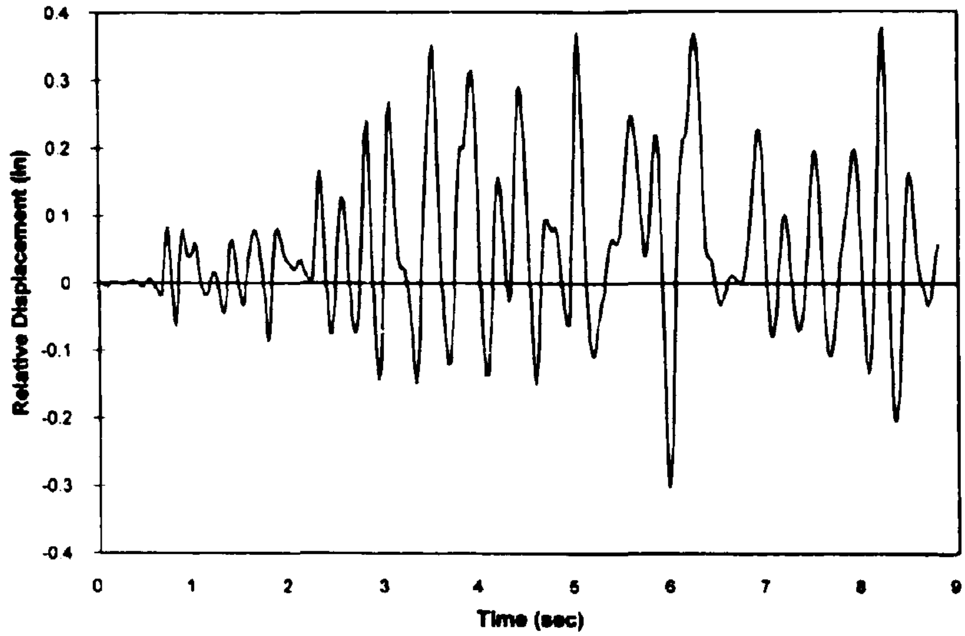


Fig. 4.1 - Relative Longitudinal Displacements at the Intermediate Hinge in Aptos.

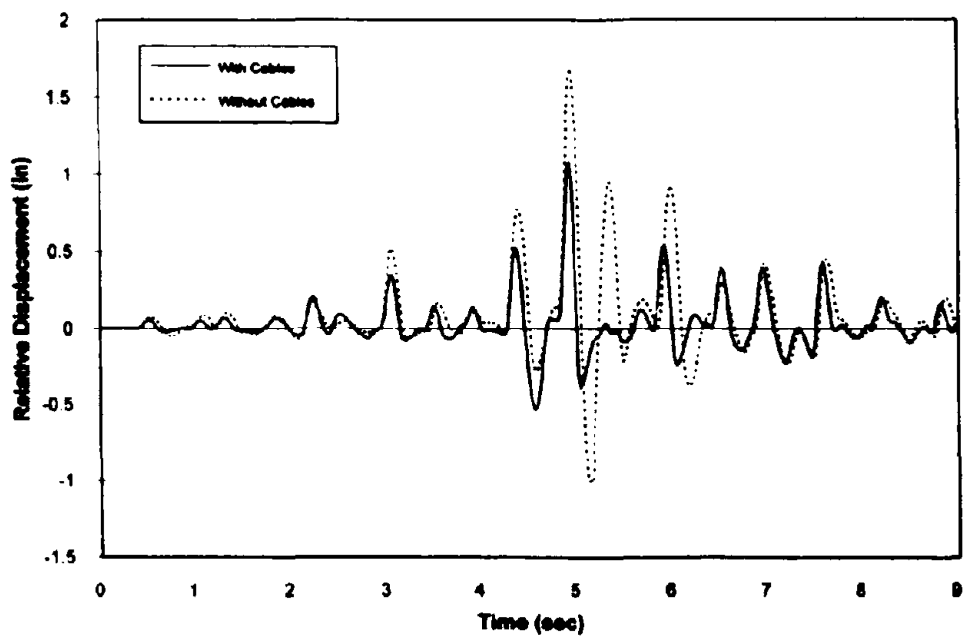


Fig. 4.2 - Relative Longitudinal Displacements at the Hinge near Bent 8R in Huntington.

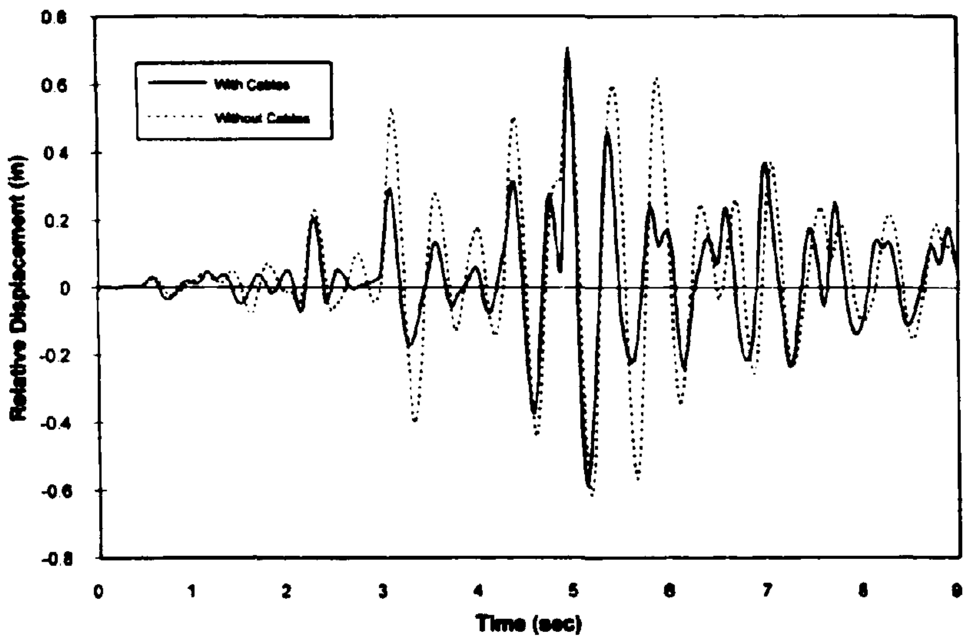


Fig. 4.3 - Relative Longitudinal Displacements at the Hinge near Bent 7L in Huntington.

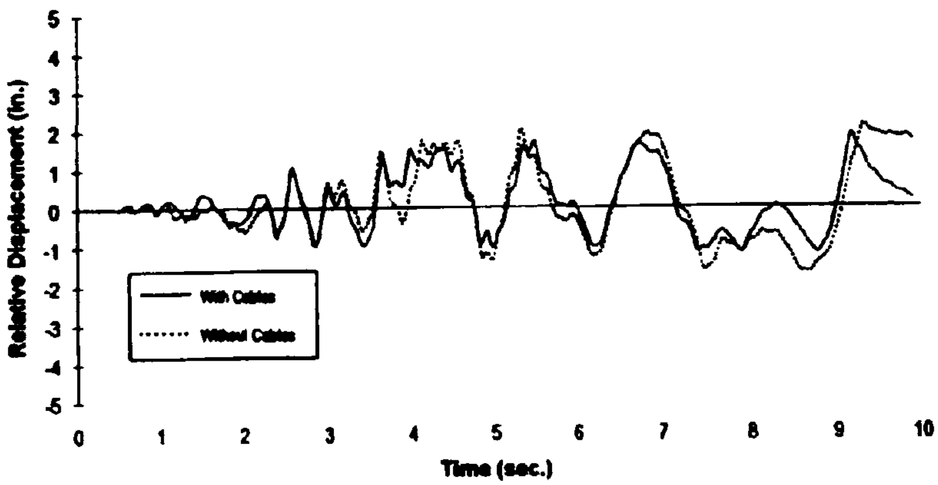


Fig. 4.4 - Relative Longitudinal Displacements at the Hinge near Bent 2 in Madrone.

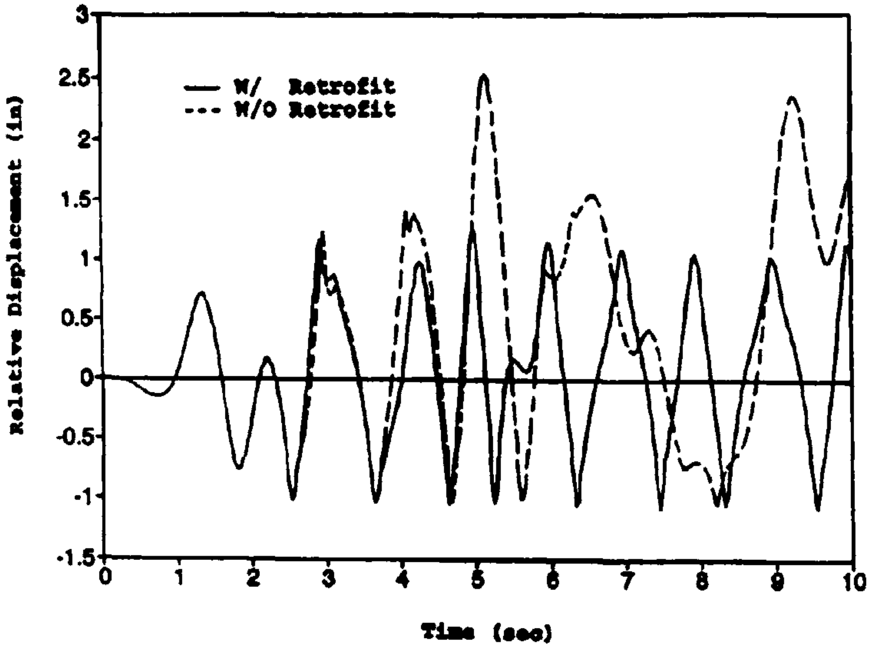


Fig. 4.5 - Relative Longitudinal Displacements at the Hinge near Bent 2 in San Gregorio.

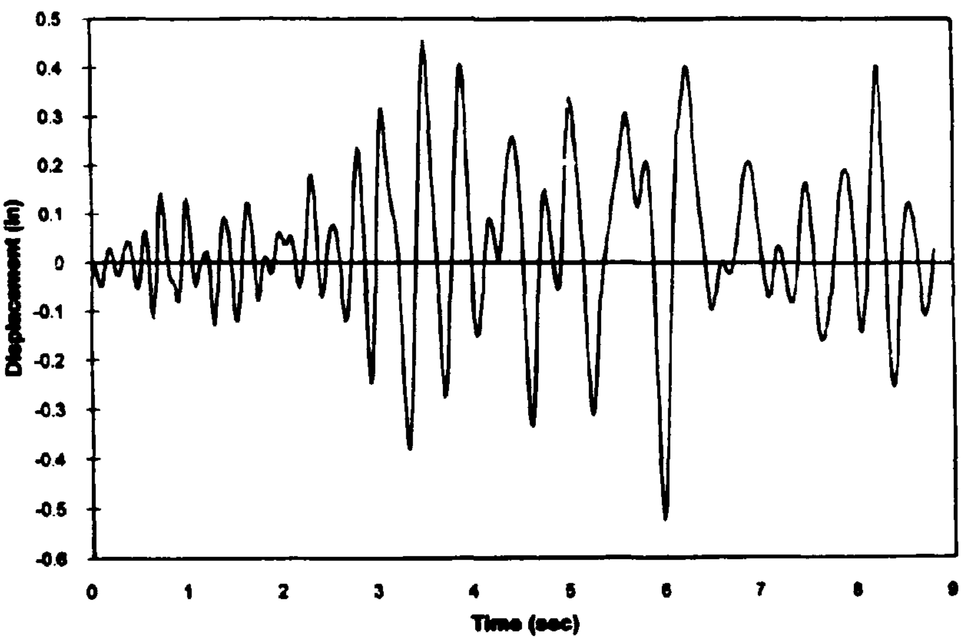


Fig. 4.6 - Longitudinal Displacement near the Hinge in the Third Span in Aptos.

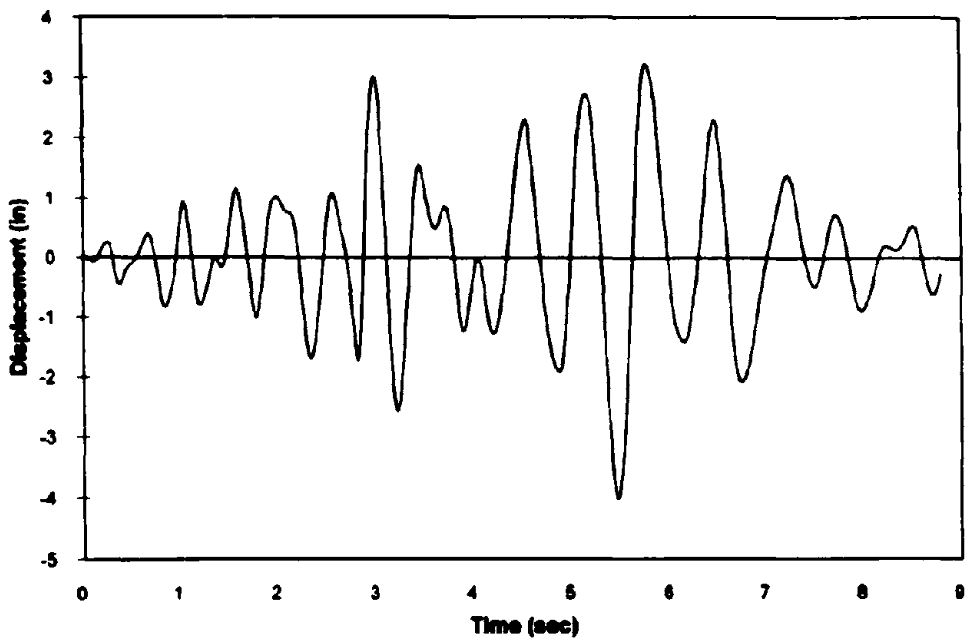


Fig. 4.7 - Transverse Displacement near the Hinge in the Third Span in Aptos.

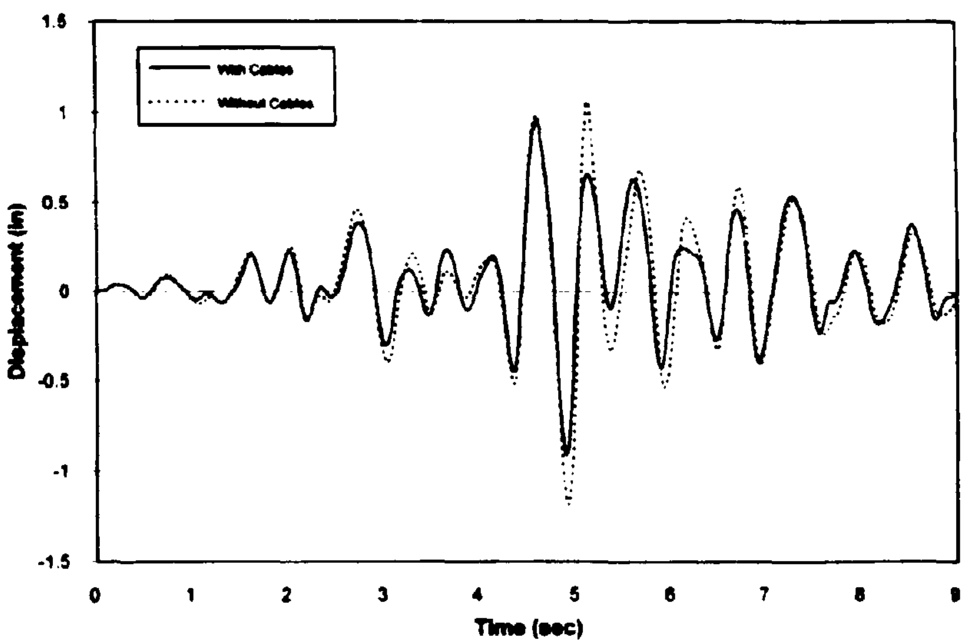


Fig. 4.8 - Longitudinal Displacement at the Top of Bent 6R in Huntington.

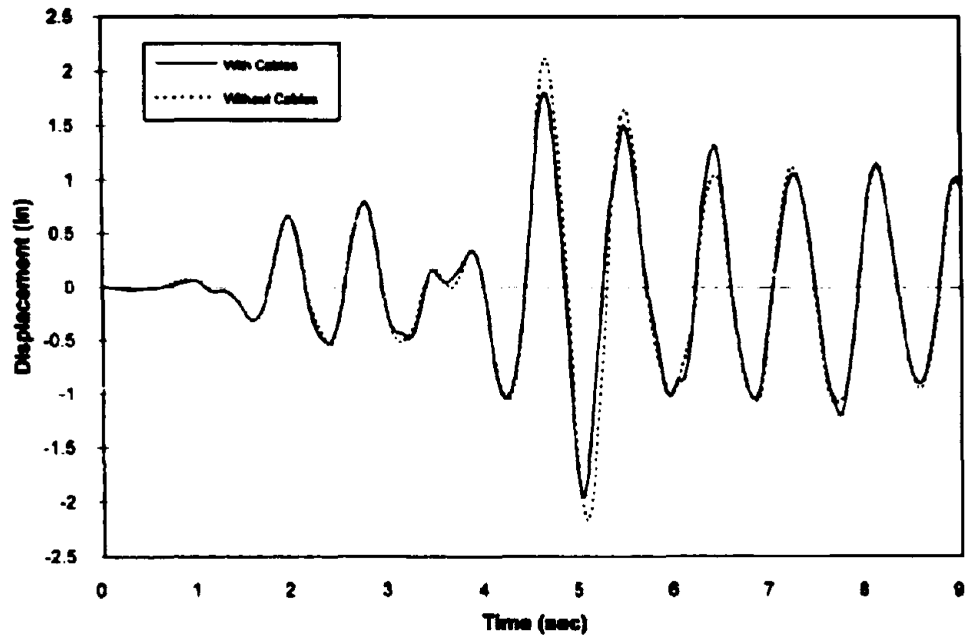


Fig. 4.9 - Transverse Displacement at the Top of Bent 6R in Huntington.

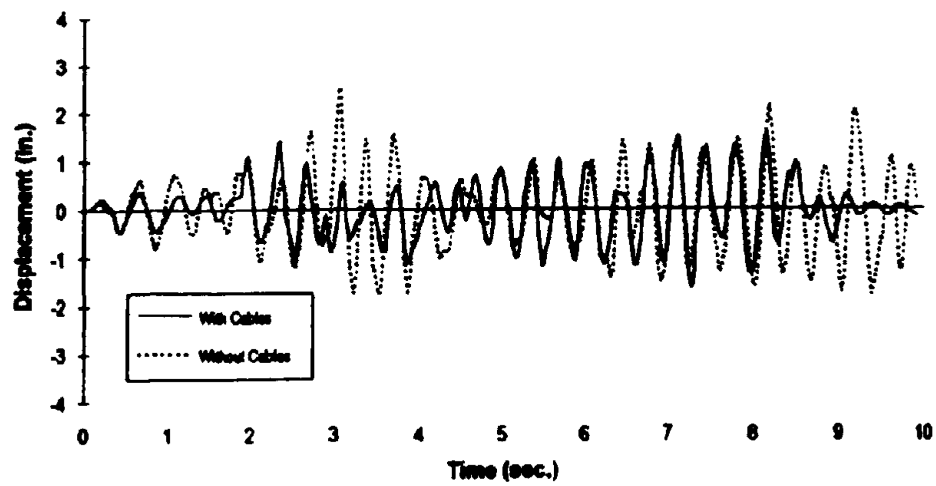


Fig. 4.10 - Longitudinal Displacement at the Center of the Middle Span in Madrone.

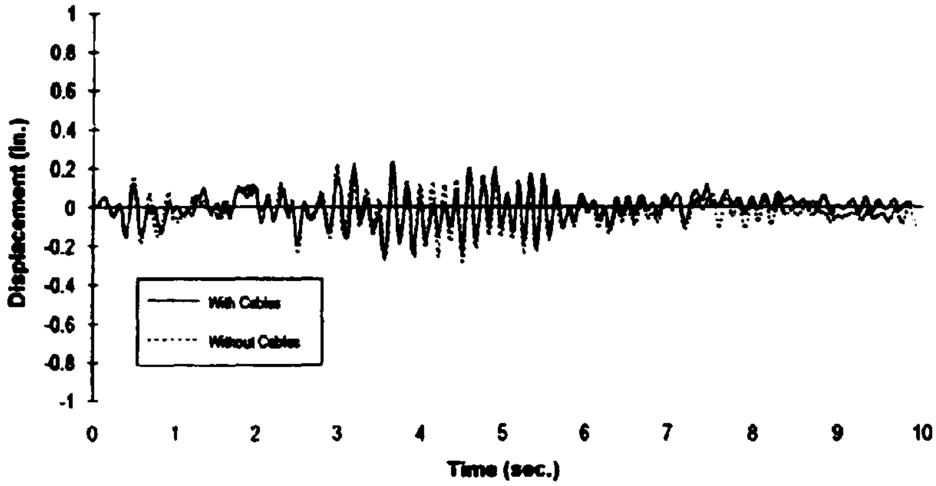


Fig. 4.11 - Transverse Displacement at the Center of the Middle Span in Madrone.

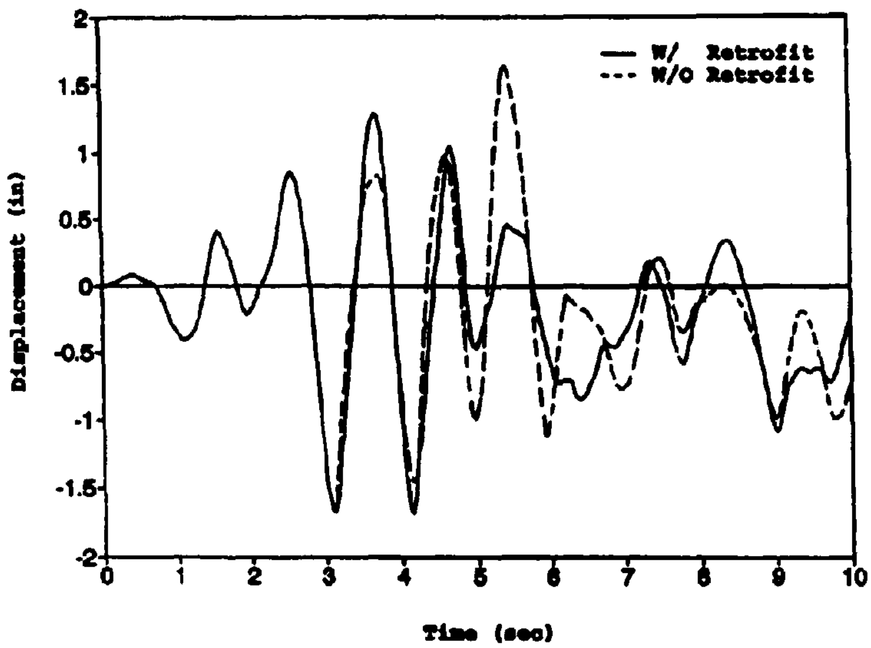


Fig. 4.12 - Longitudinal Displacement at the Center of Span 4 in San Gregorio.

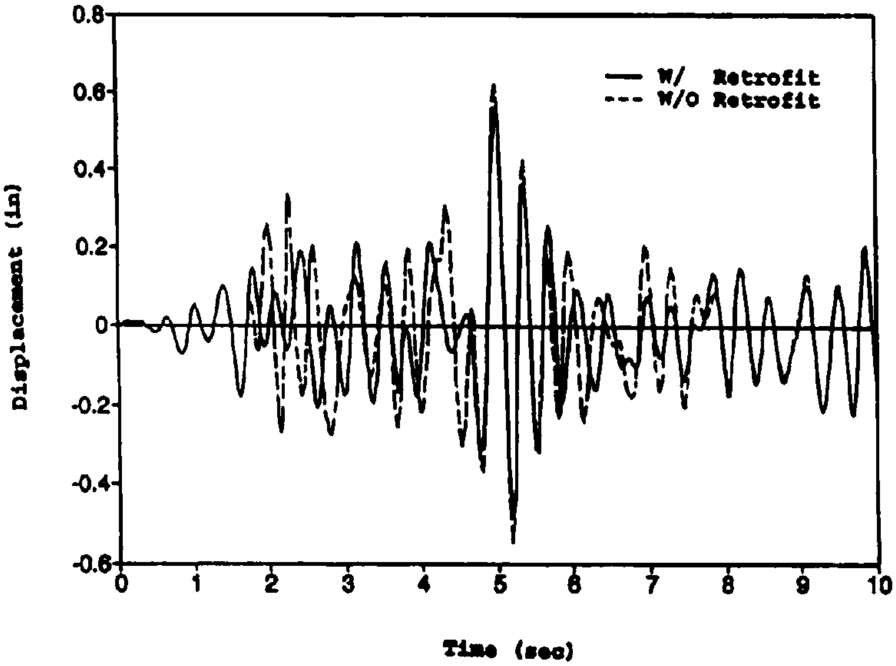


Fig. 4.13 - Transverse Displacement at the Center of Span 4 in San Gregorio.

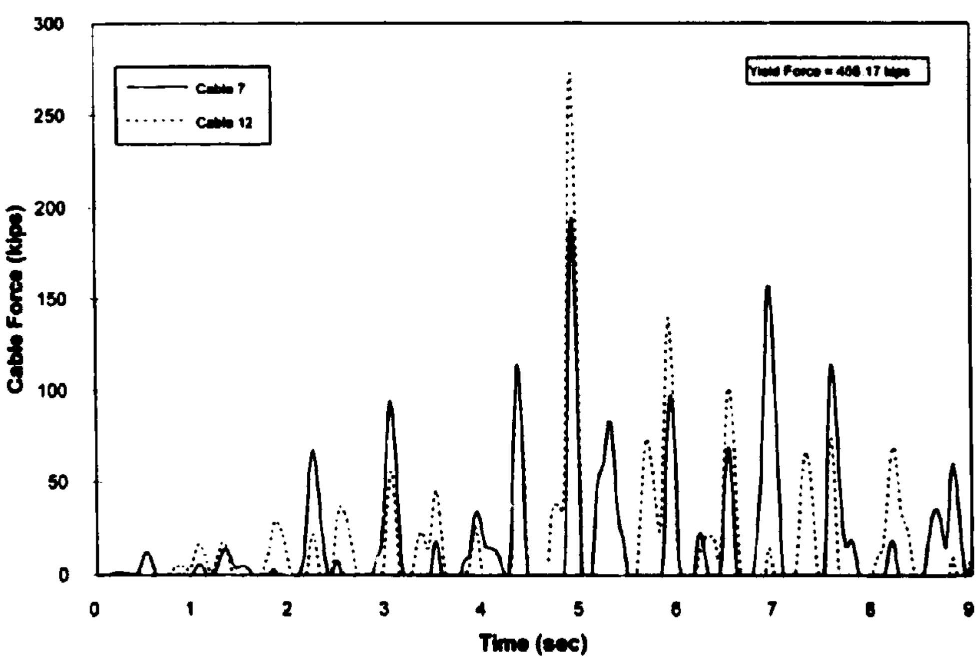


Fig. 4.14 - Restrainer Force History in the Hinge near Bent 8R in Huntington.

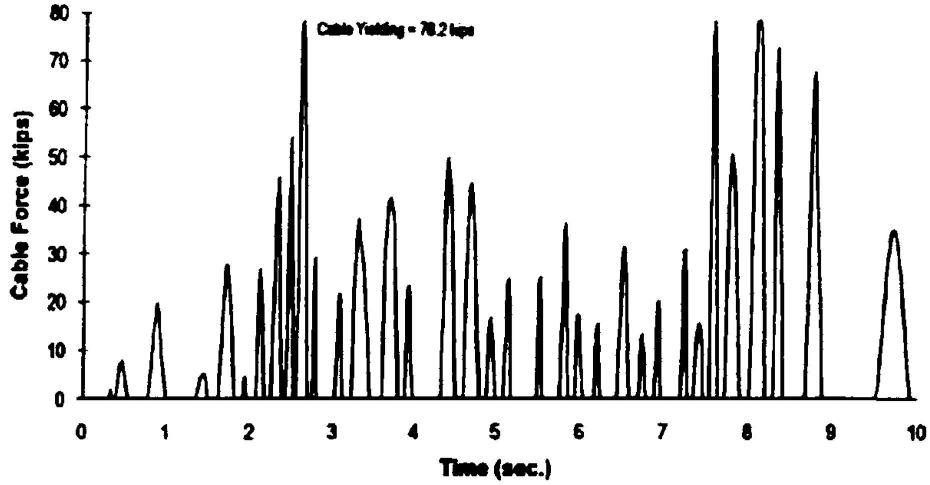


Fig. 4. 15 - Restrainer Force History in the Hinge near Bent 2 in Madrone.

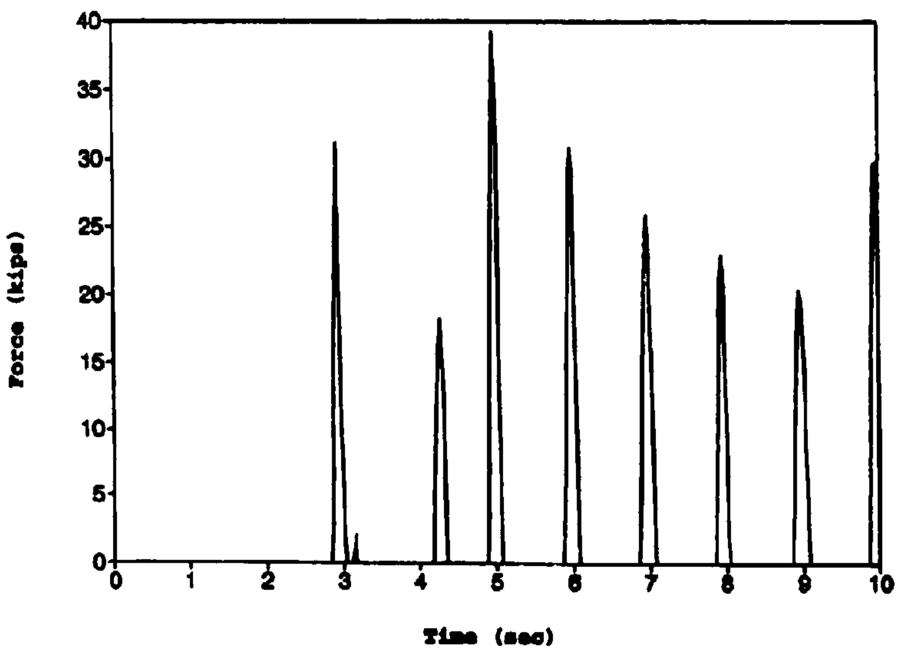


Fig. 4.16 - Restrainer Force History at the Hinge near Bent 2 in San Gregorio.

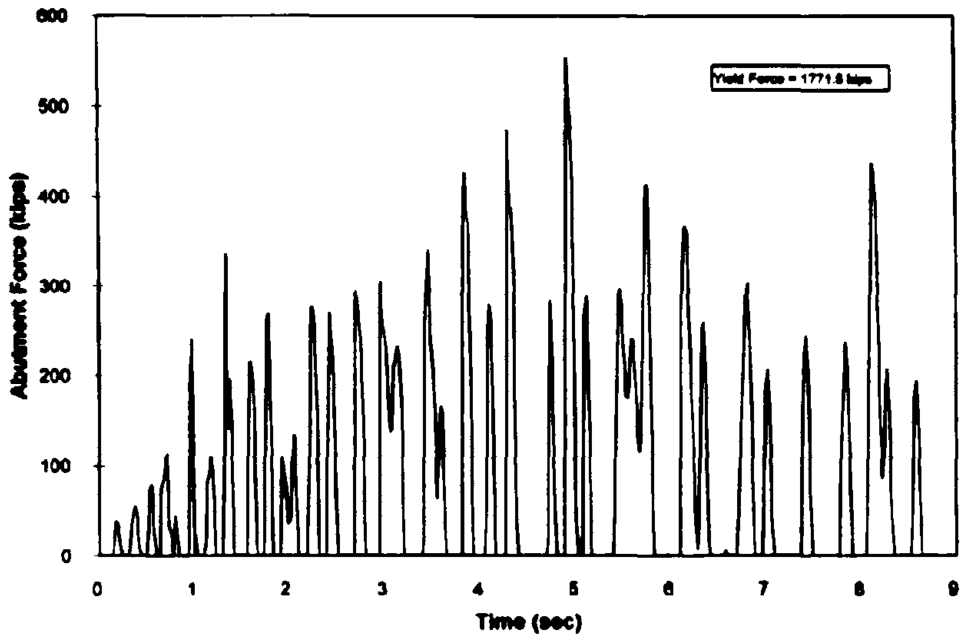


Fig. 4.17 - Force History in Abutment 1 in Aptos.

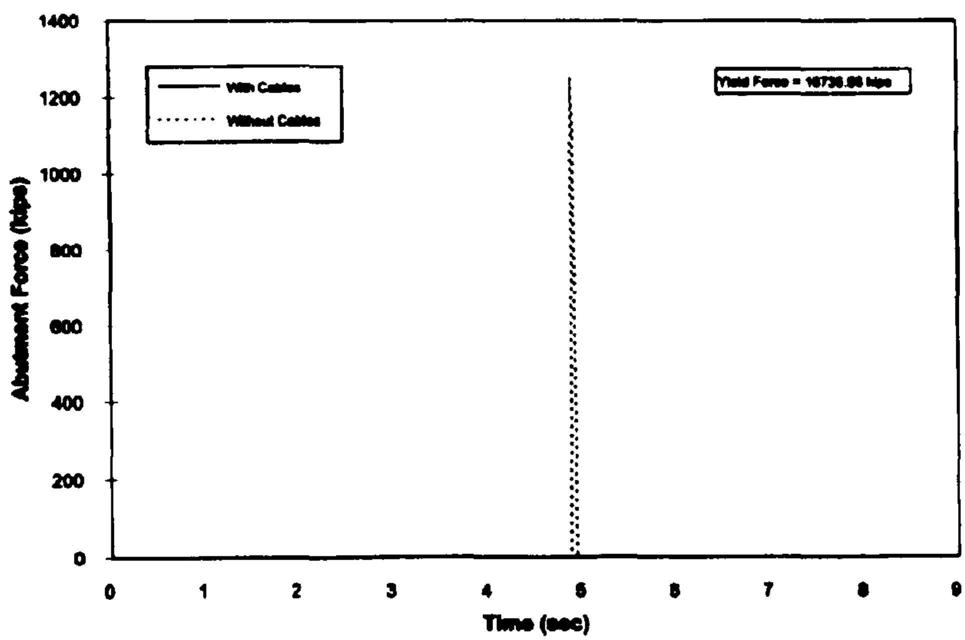


Fig. 4.18 - Force History in Abutment 1 of the Right Bridge in Huntington.

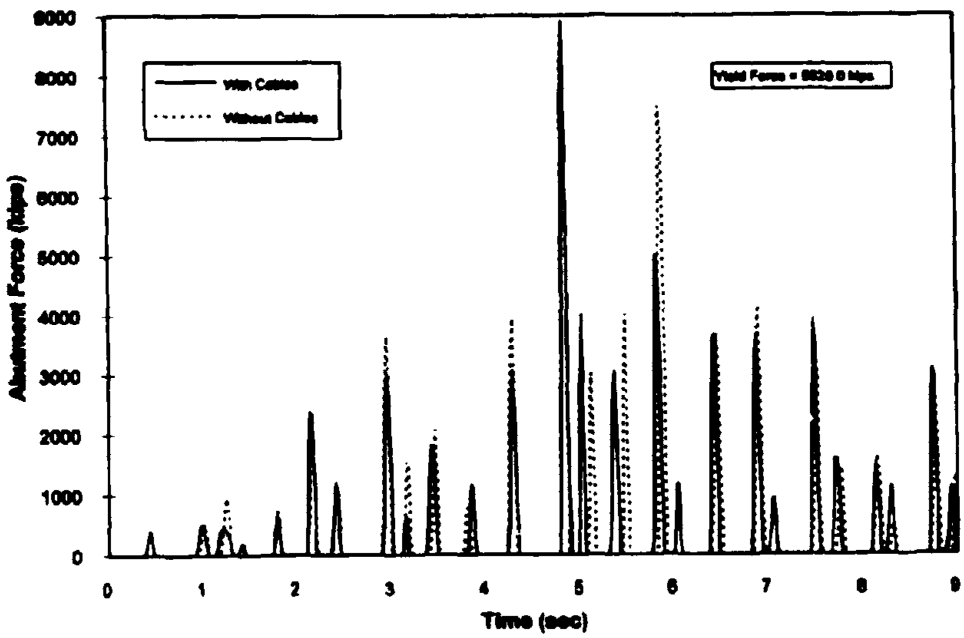


Fig. 4.19 - Force History in Abutment 12 of the Right Bridge in Huntington.

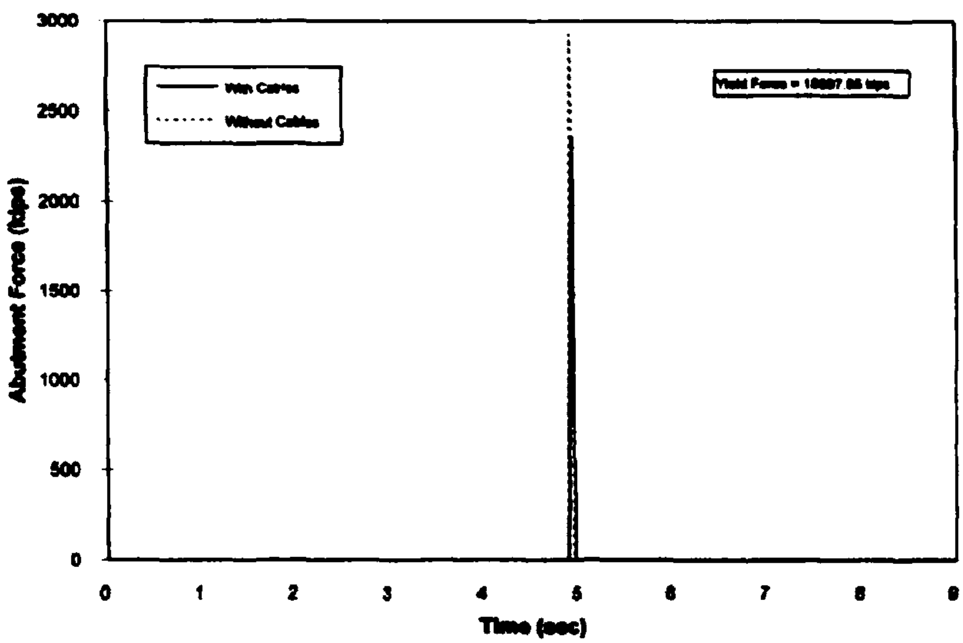


Fig. 4.20 - Force History in Abutment 1 of the Left Bridge in Huntington.

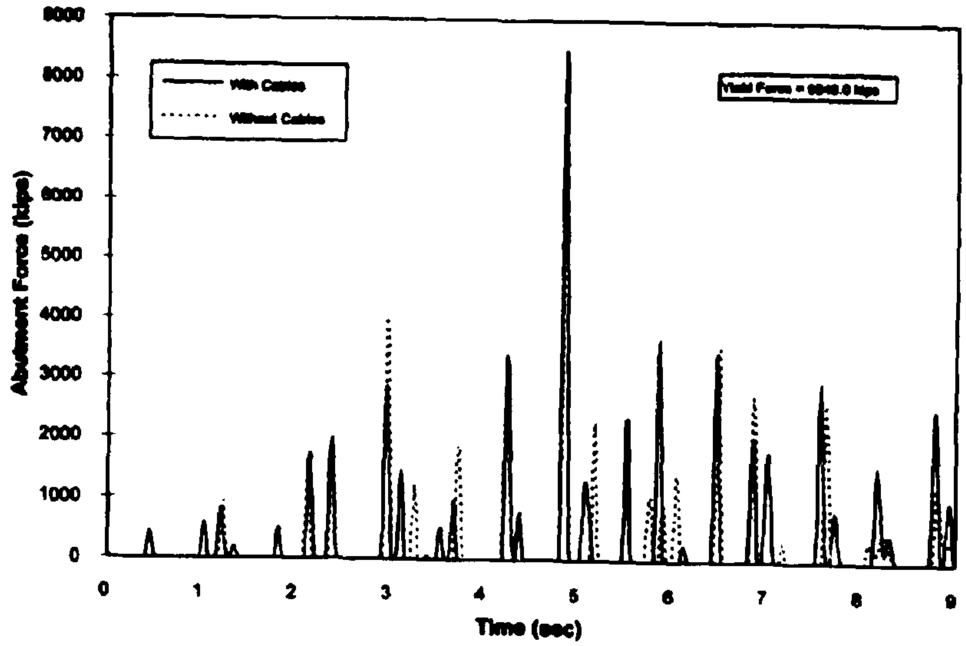


Fig. 4.21 - Force History in Abutment 13 of the Left Bridge in Huntington.

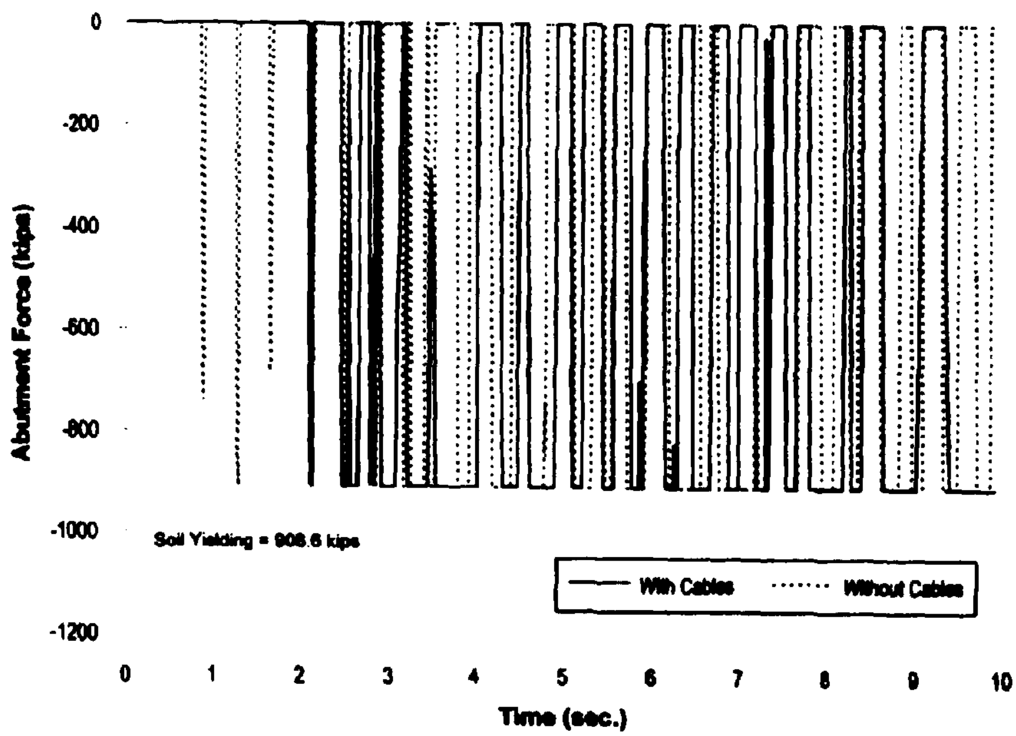


Fig. 4.22 - Force History in Abutment 1 in Madrone.

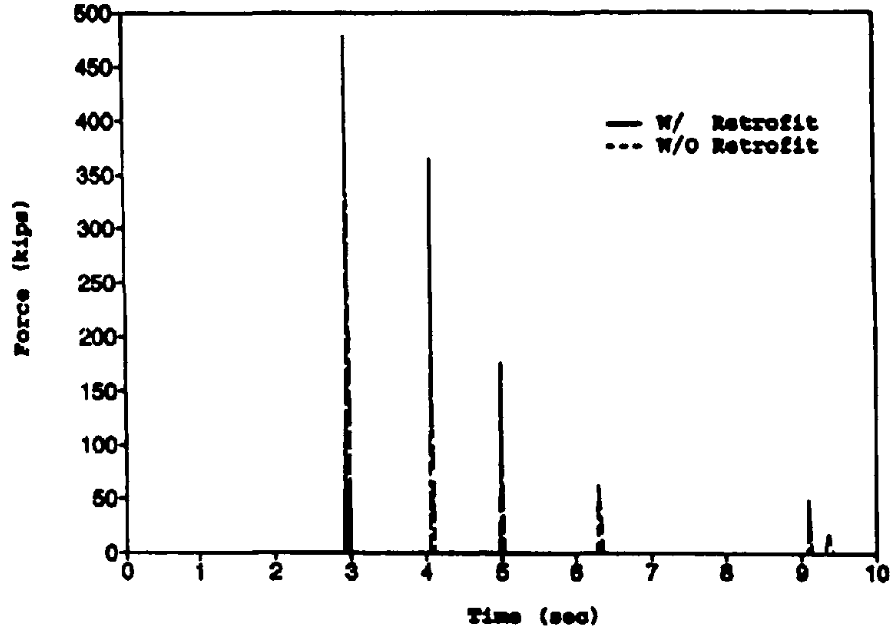


Fig. 4.23 - Force History in Abutment 1 in San Gregorio.

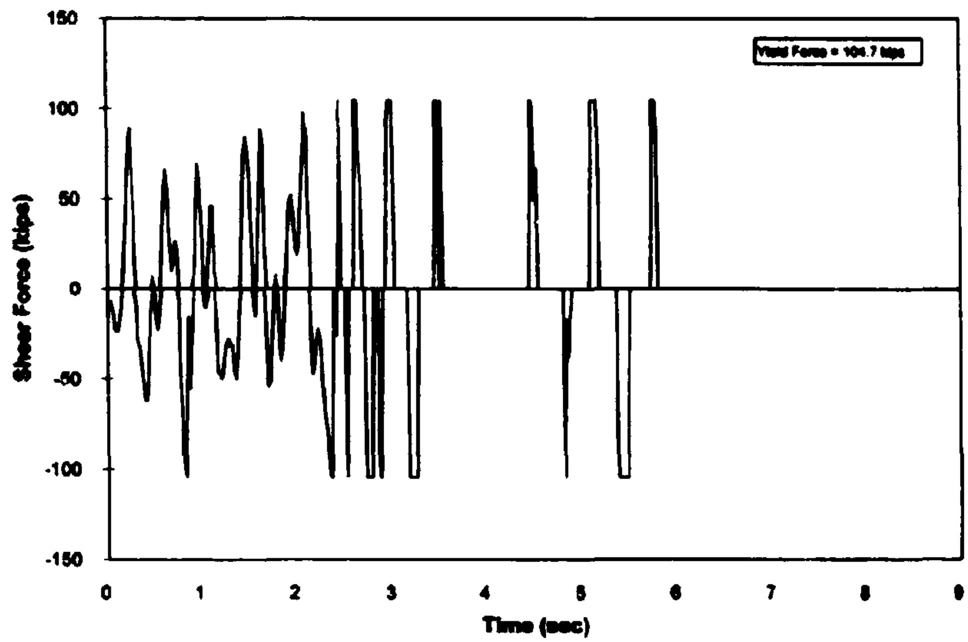


Fig. 4.24 - Force History at the Shear Key in Abutment 1 in Aptos.

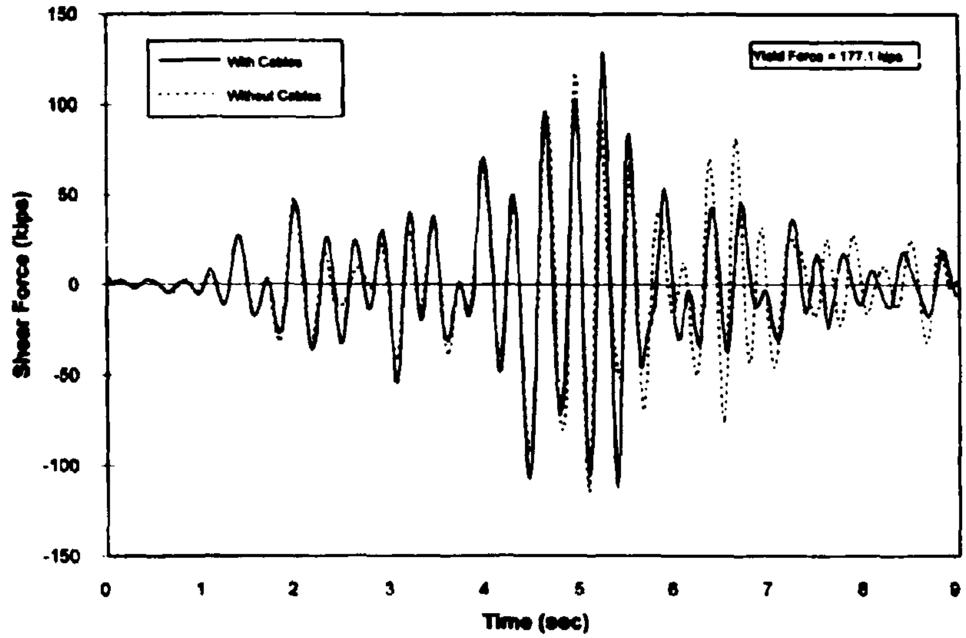


Fig. 4.25 - Force History at the Shear Key in Abutment 1 in Huntington (Right).

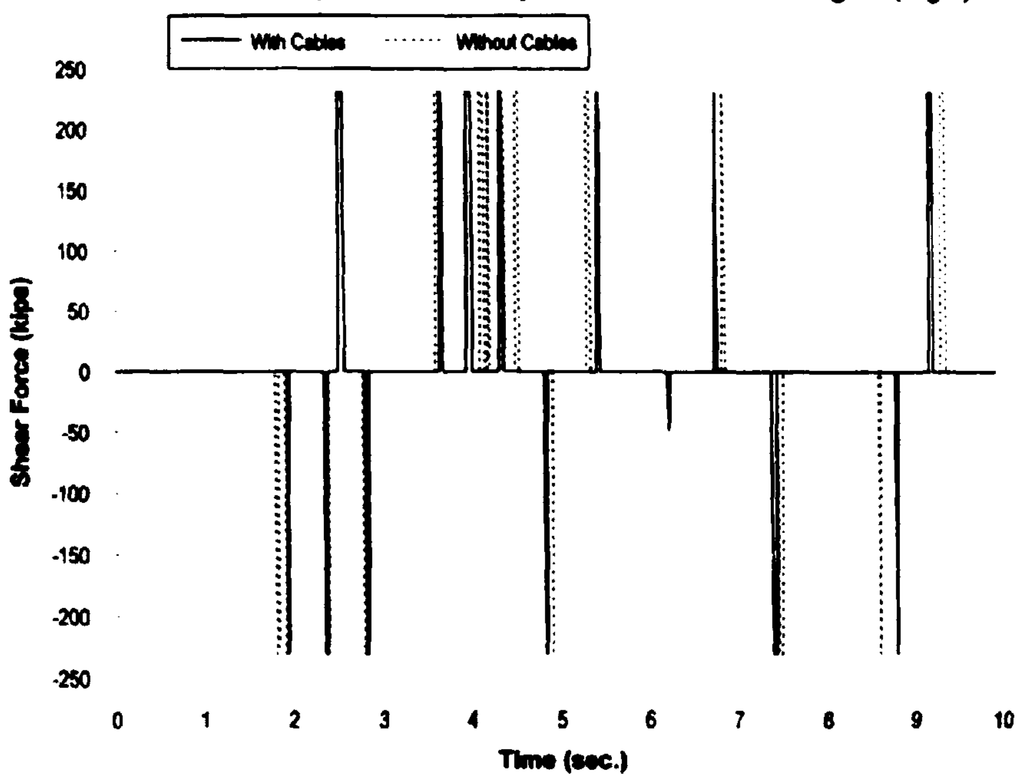


Fig. 4.26 - Force History at the Shear Key in Abutment 1 in Madrone.

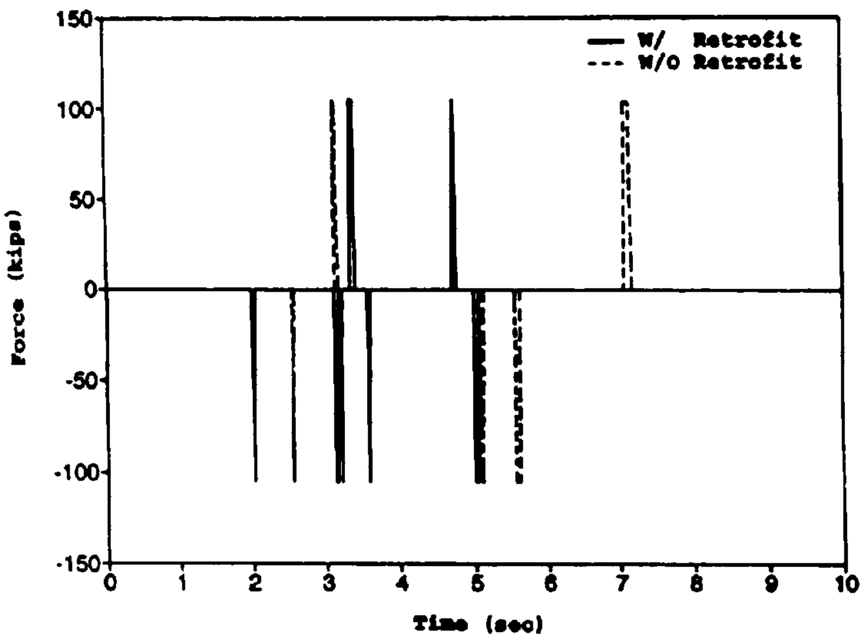


Fig. 4.27 - Force History at the Shear Key in Abutment 1 in San Gregorio.

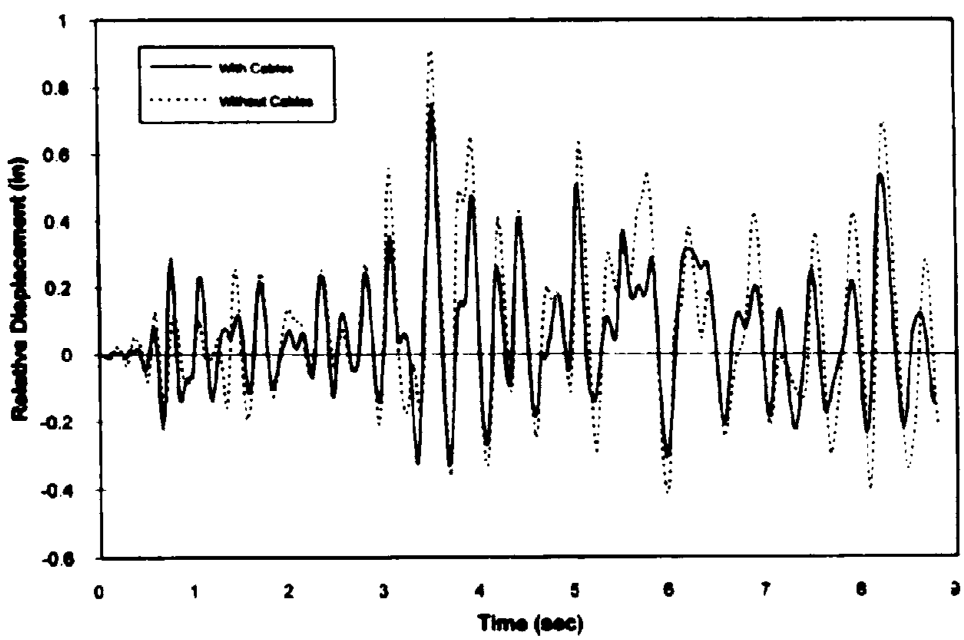


Fig. 4.28 - Relative Displacement History at the Hinge in Aptos for PGA of 0.7g.

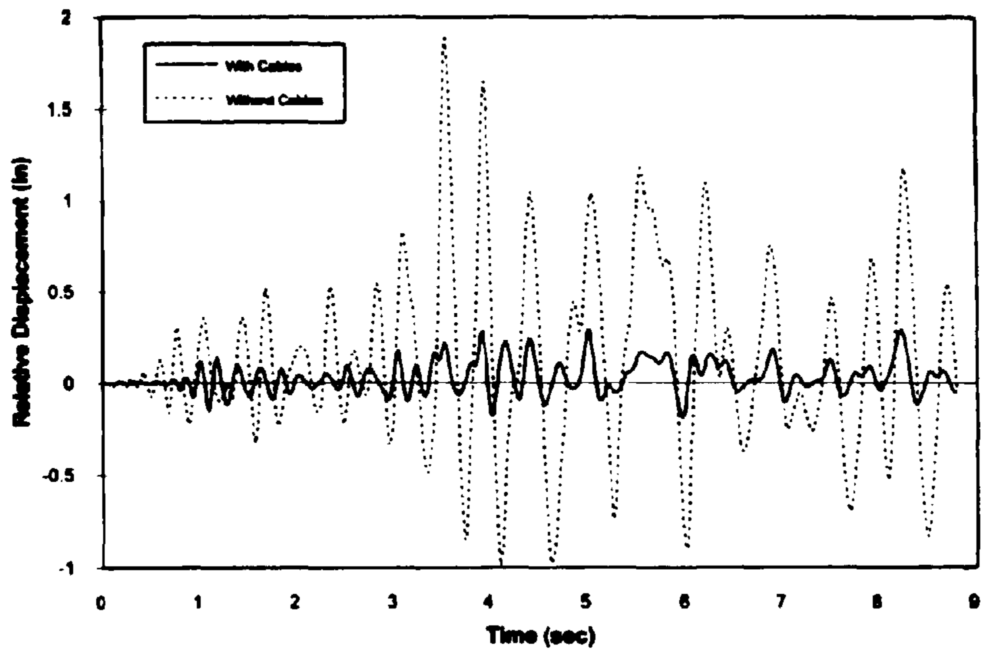


Fig. 4.29 - Relative Displacement History at the Hinge in Aptos for PGA of 1.0g.

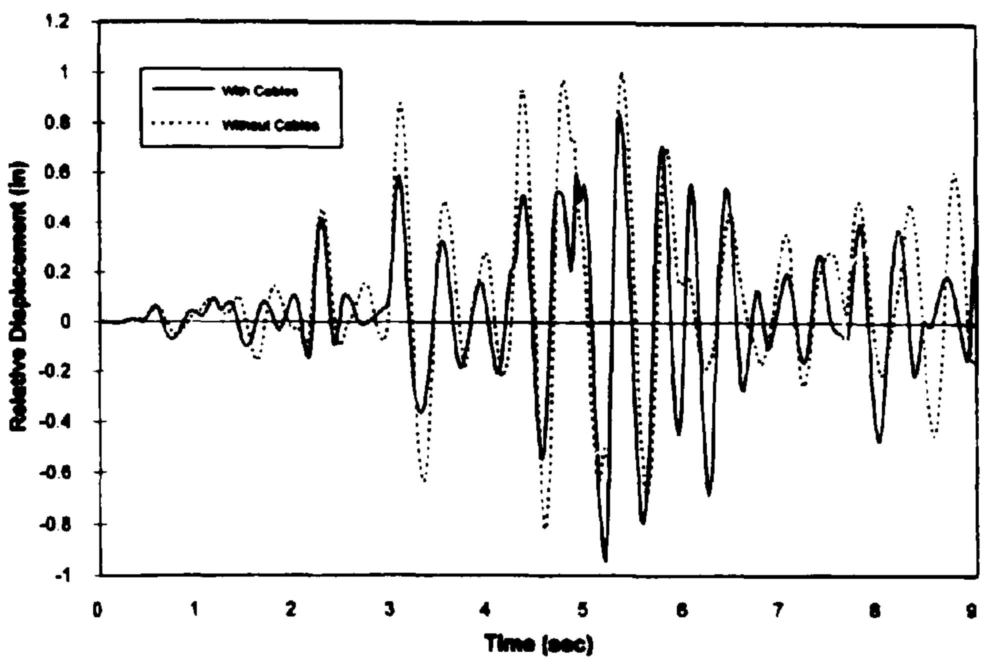


Fig. 4.30 - Relative Displacement History at the Hinge near Bent 7L in Huntington for PGA of 0.3g.

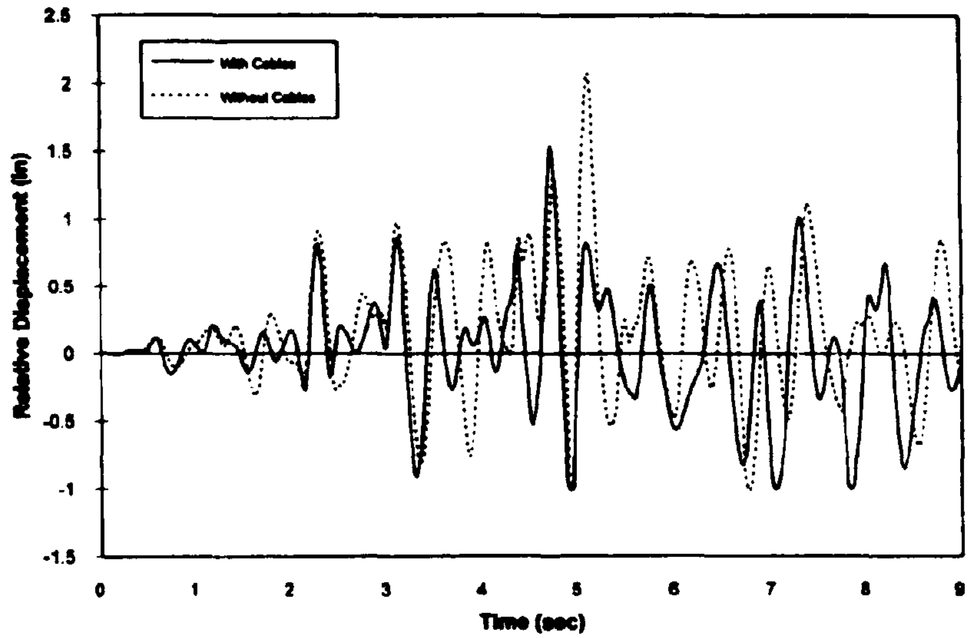


Fig. 4.31 - Relative Displacement History at the Hinge near Bent 7L in Huntington for PGA of 0.6g.

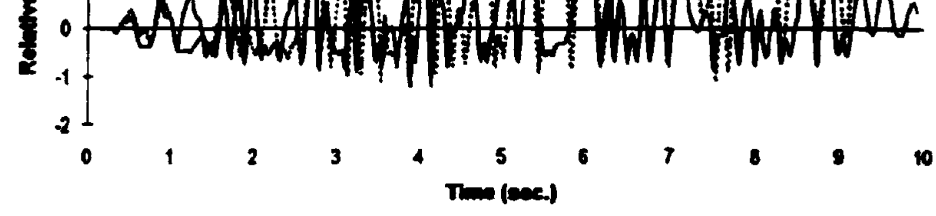


Fig. 4.32 - Relative Displacement History at the Hinge near Bent 2 in Madrone for PGA of 0.65g.

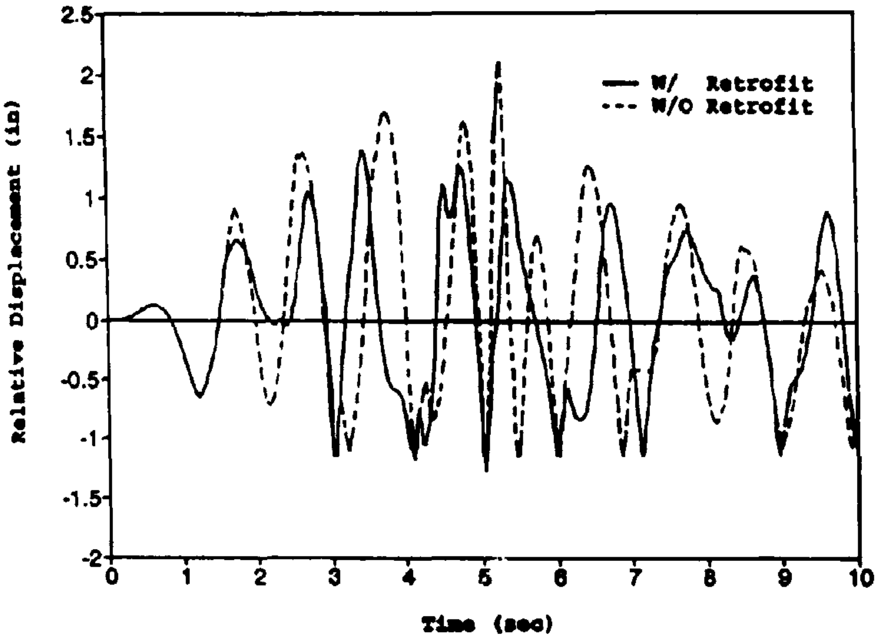


Fig. 4.33 - Relative Displacement History at the Hinge near Bent 4 in San Gregorio for PGA of 0.32g.

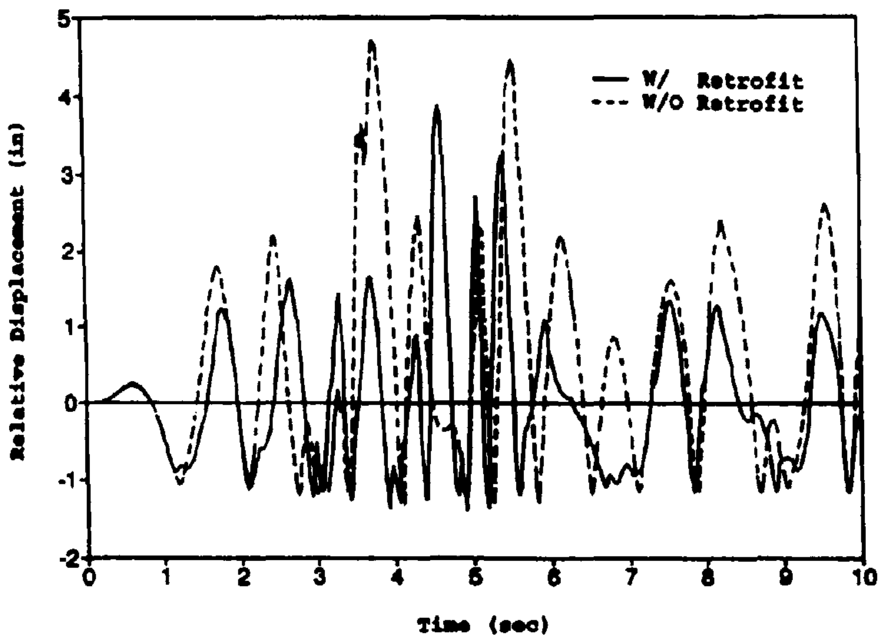


Fig. 4.34 - Relative Displacement History at the Hinge near Bent 4 in San Gregorio for PGA of 0.60g.

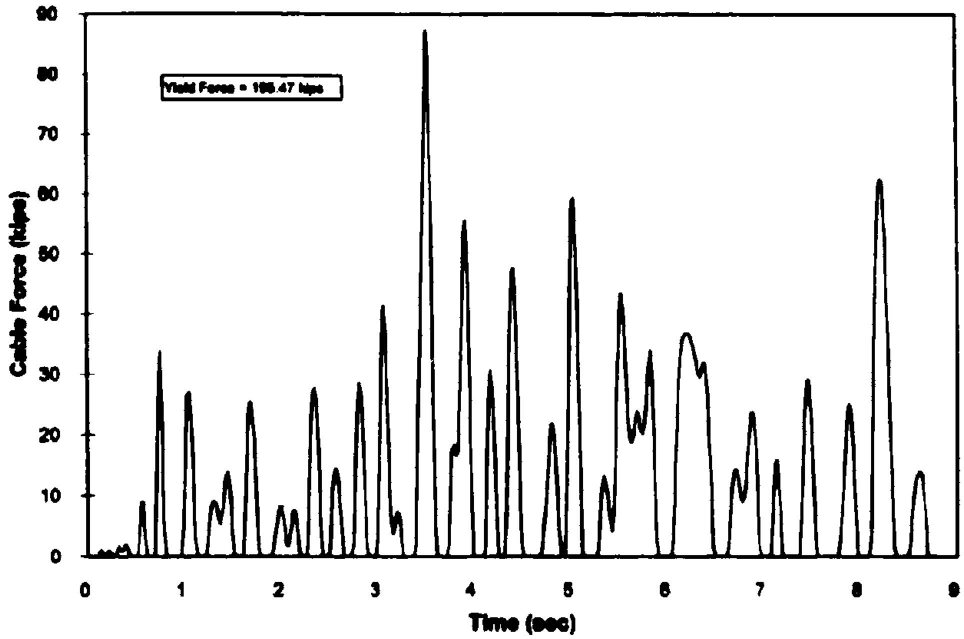


Fig. 4.35 - Restainer Force History at the Hinge in Aptos for PGA of 0.7g.

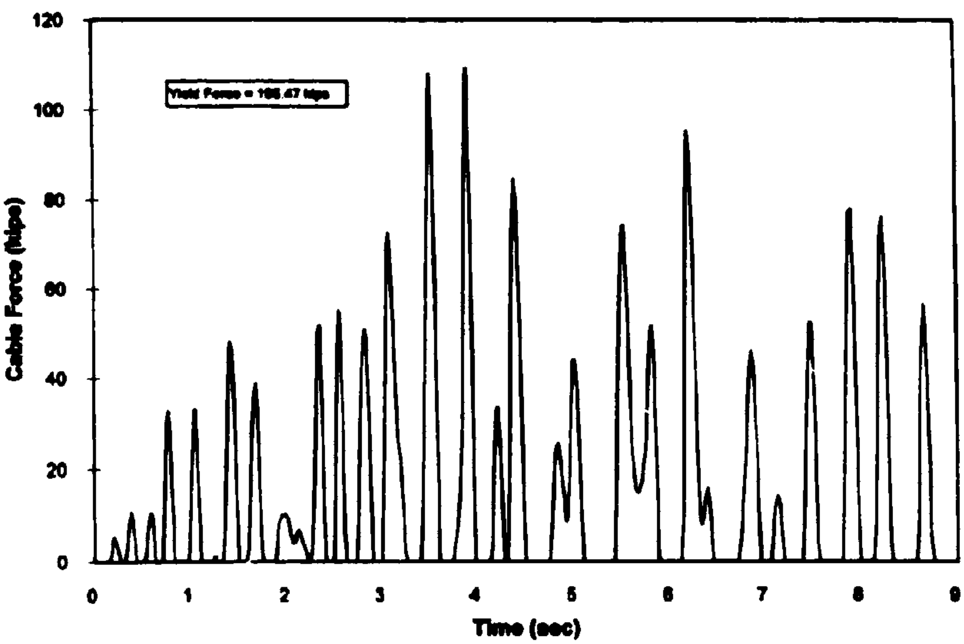


Fig. 4.36 - Restainer Force History at the Hinge in Aptos for PGA of 1.0g.

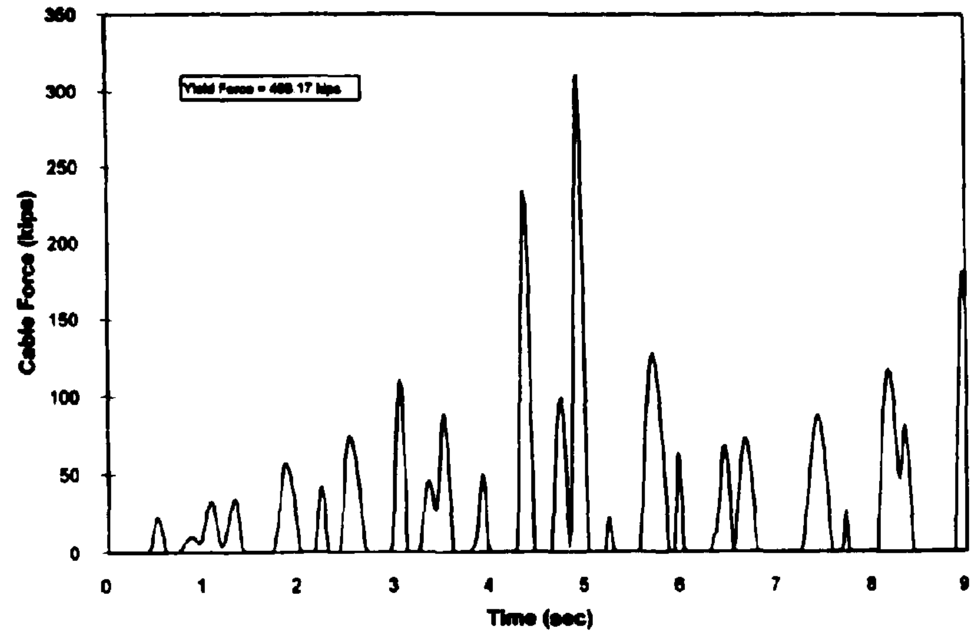


Fig. 4.37 - Restrainer Force History in the Southernmost Cable at the Hinge near Bent 7L in Huntington for PGA of 0.3g.

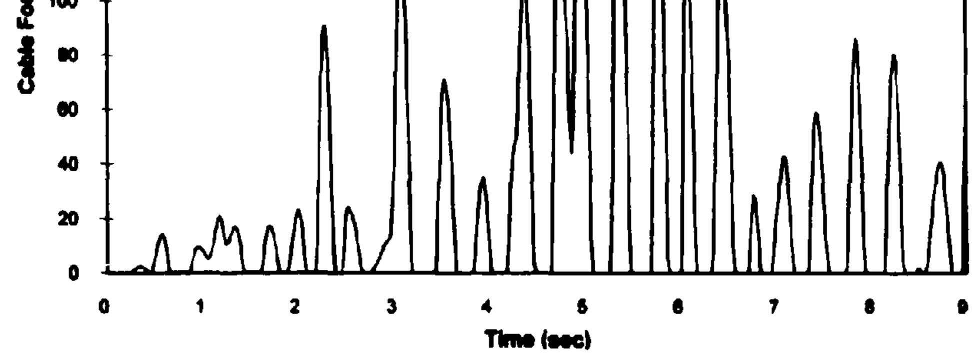


Fig. 4.38 - Restrainer Force History in the Northernmost Cable at the Hinge near Bent 7L in Huntington for PGA of 0.3g.

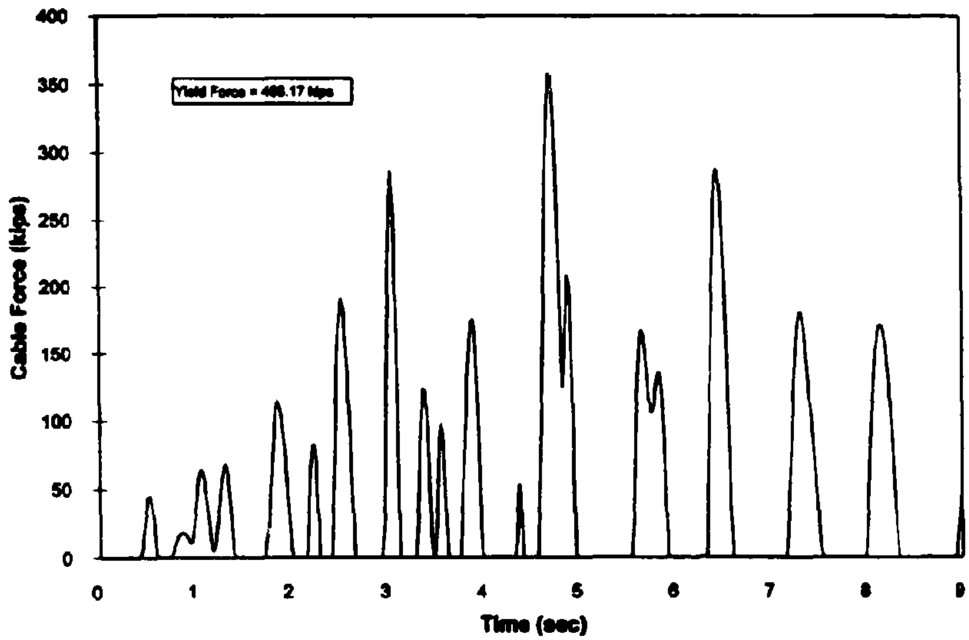


Fig. 4.39 - Restrainer Force History in the Southernmost Cable at the Hinge near Bent 7L in Huntington for PGA of 0.6g.

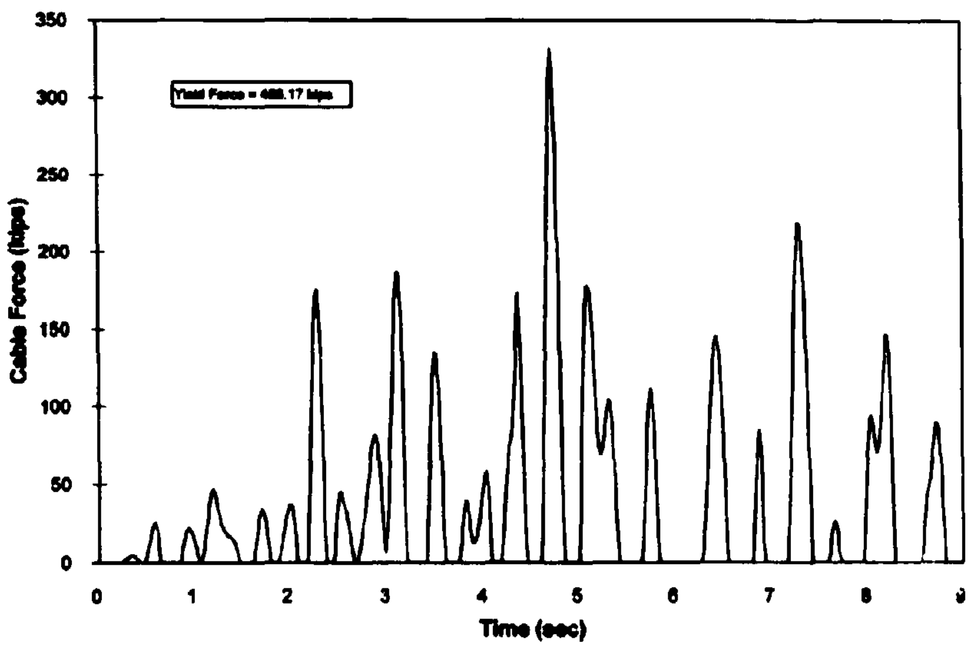


Fig. 4.40 - Restrainer Force History in the Northernmost Cable at the Hinge near Bent 7L in Huntington for PGA of 0.6g.

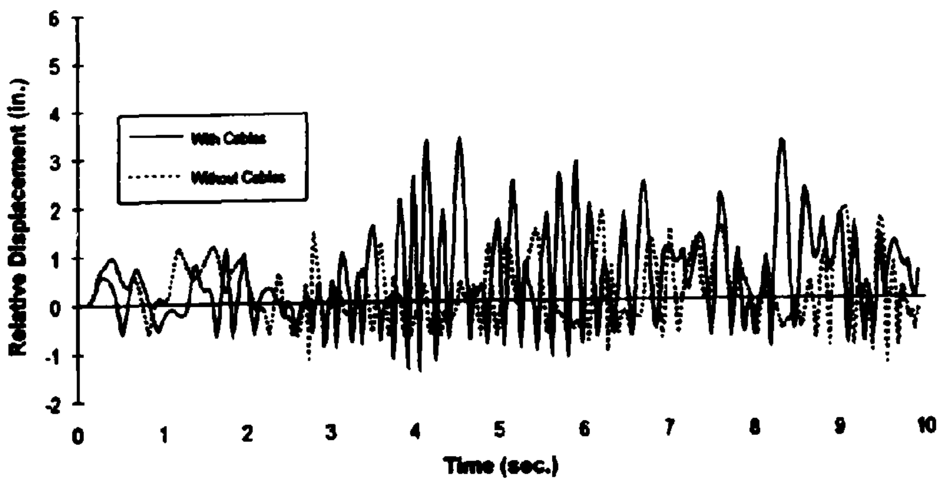


Fig. 4.41 - Restrainer Force History at the Hinge near Bent 2 in Madrone for PGA of 0.65g.

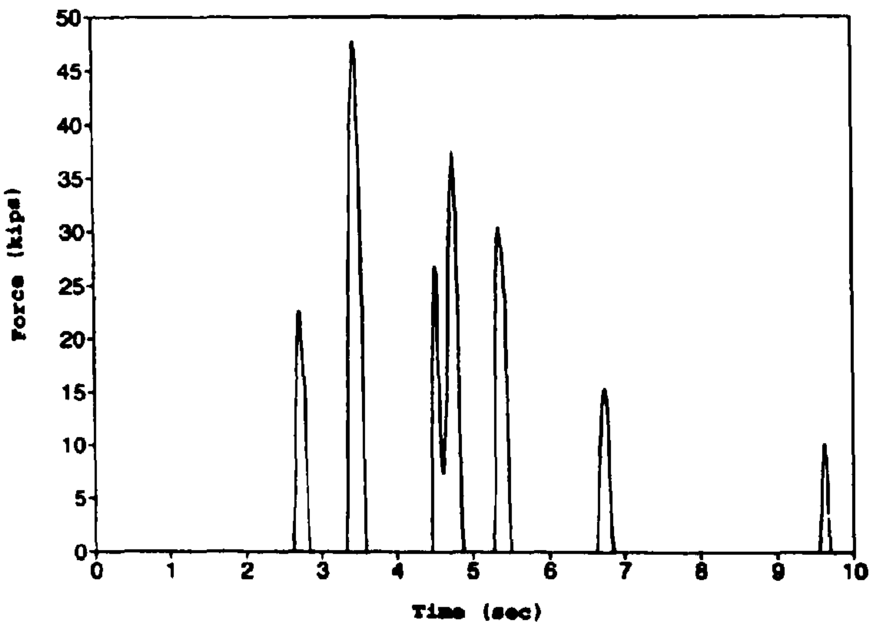


Fig. 4.42 - Restrainer Force History at the Hinge near Bent 4 in San Gregorio for PGA of 0.32g.

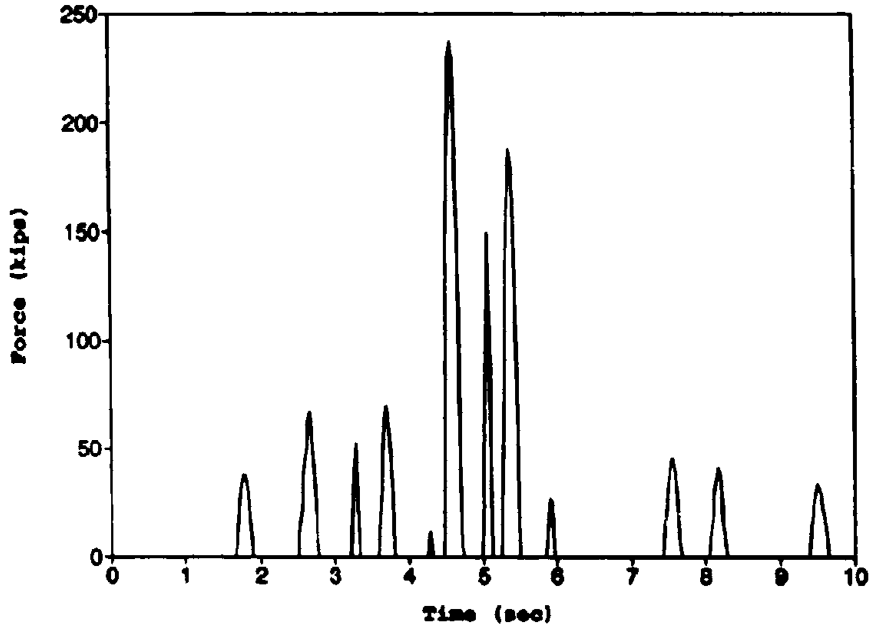


Fig. 4.43 - Restrainer Force History at the Hinge near Bent 4 in San Gregorio for PGA of 0.60g.

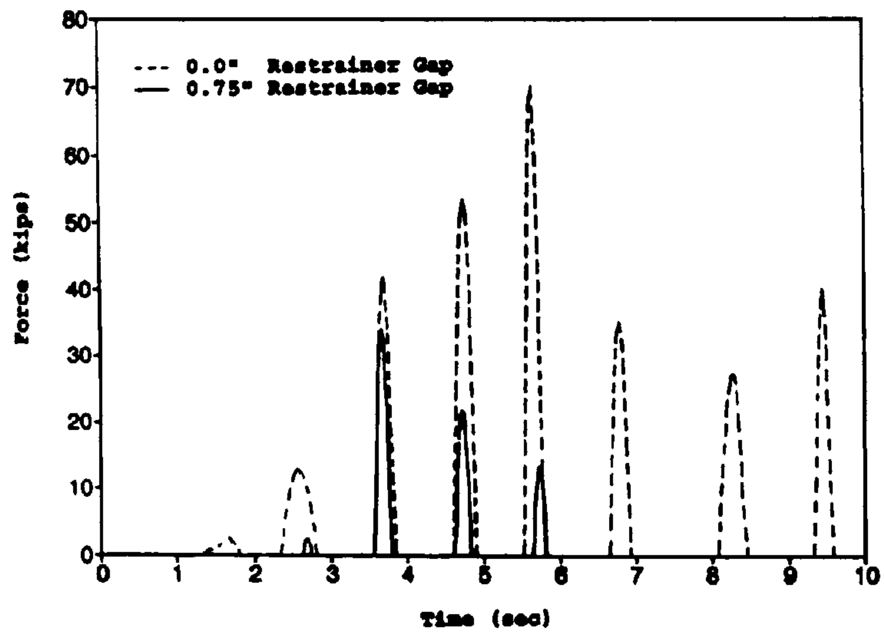


Fig. 4.44 - Restrainer Force History at the Hinge near Bent 4 in San Gregorio for different Restrainer gaps.

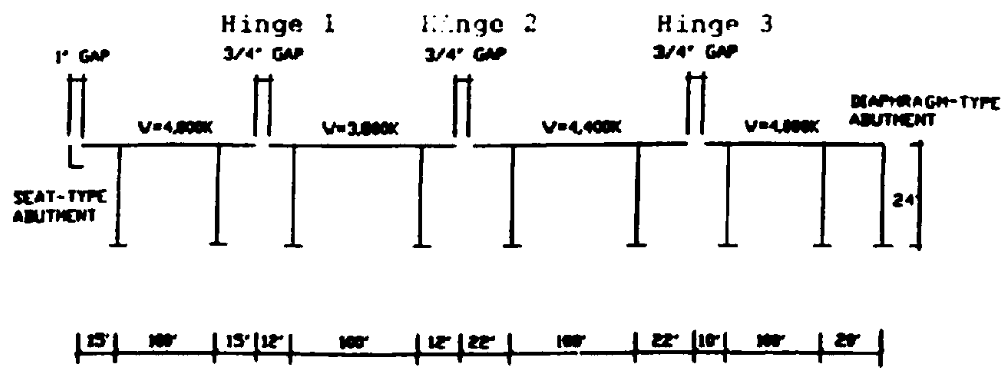
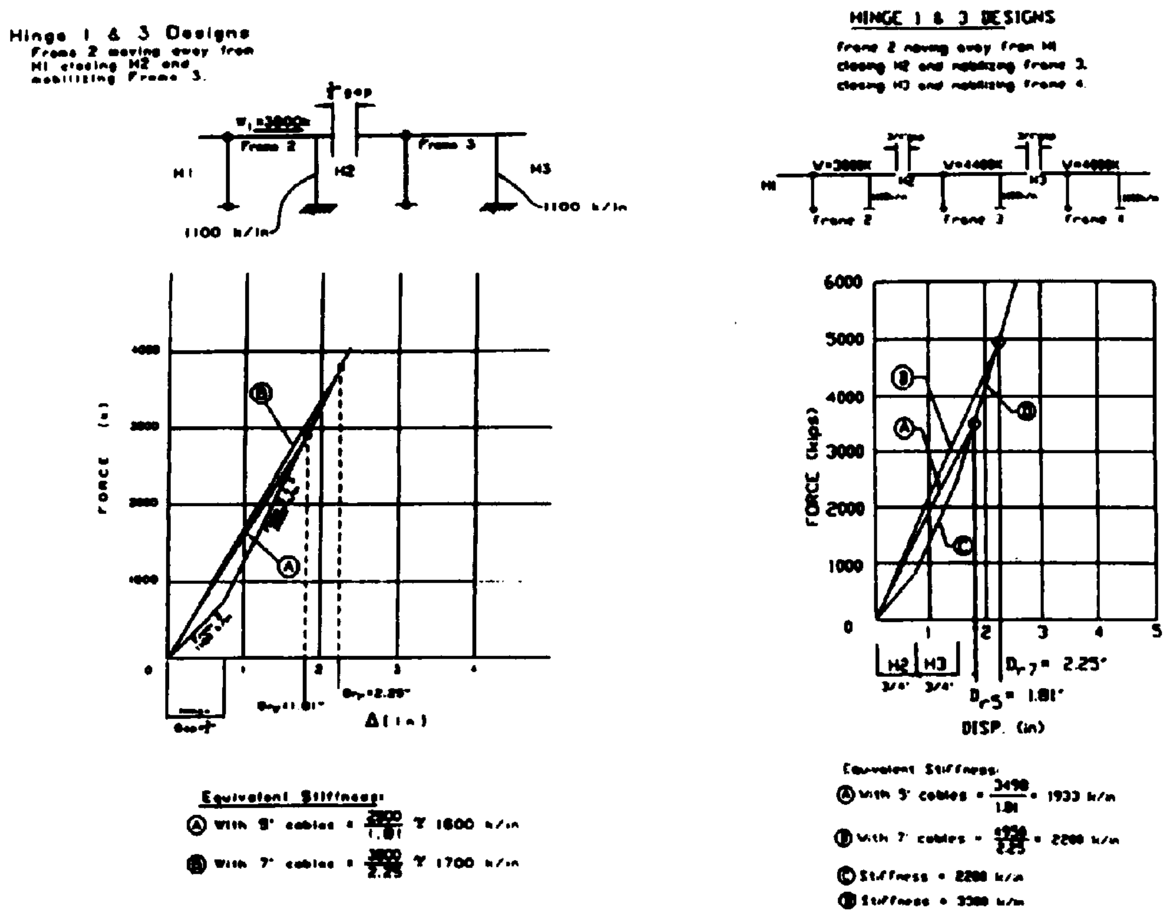


Fig. 5.1 - The Example Bridge.



(a) Original Design

(b) Refined Design

Fig. 5.2 - Stiffness for Movement of the Bridge on the Right Side of Hinge 1.

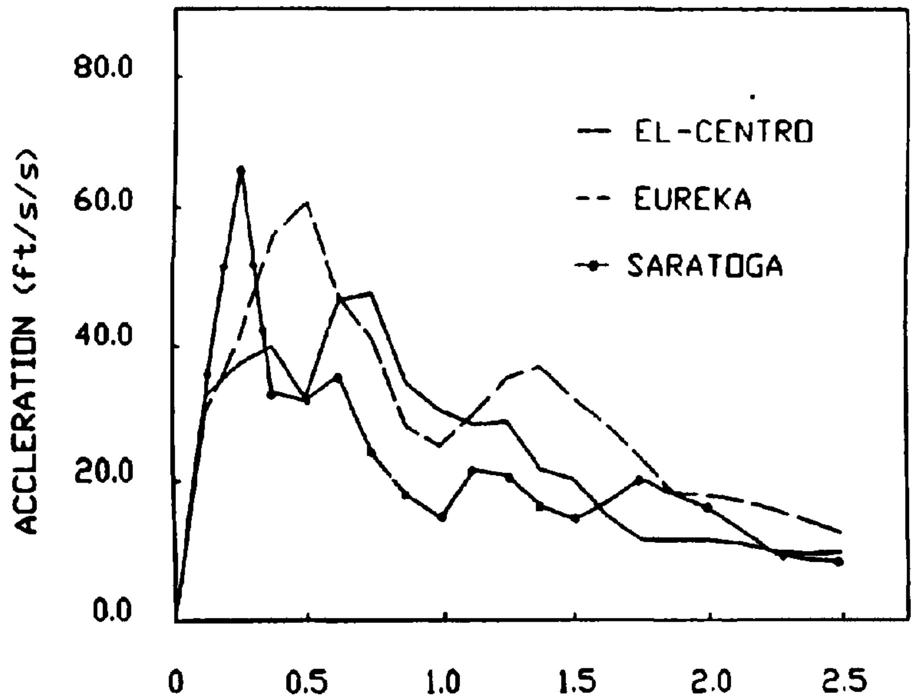


Fig. 5.3 - Acceleration Spectra for the Input Earthquakes.

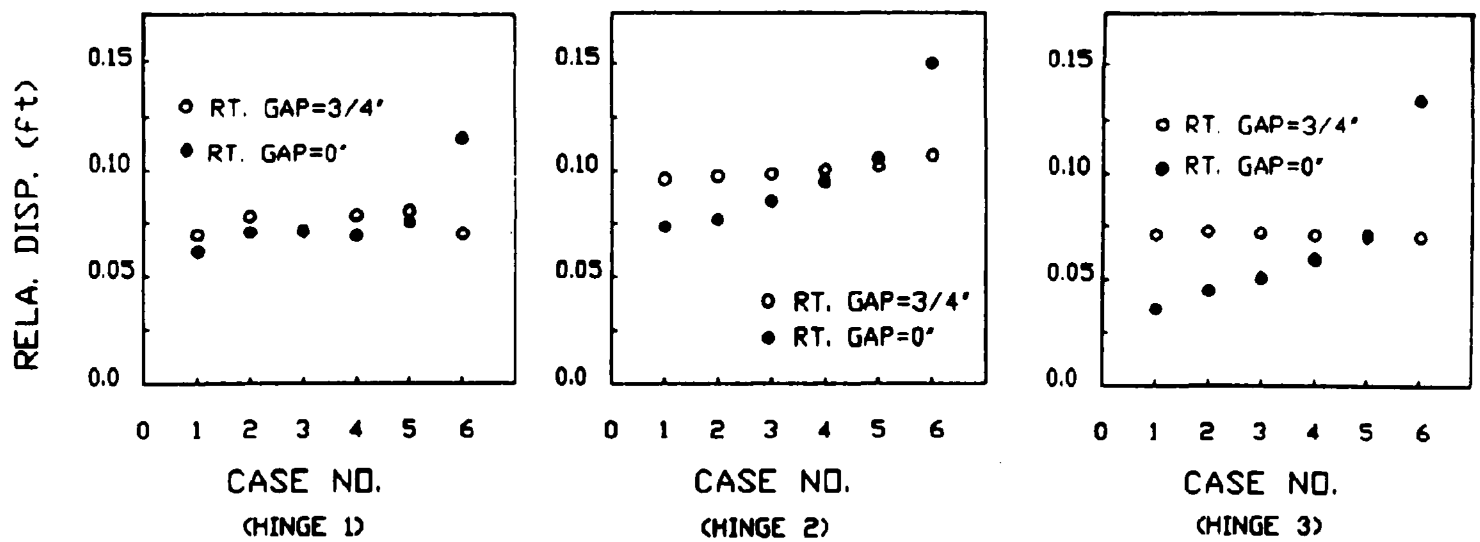


Fig. 5.4 - Maximum Relative Displacements at Hinges.

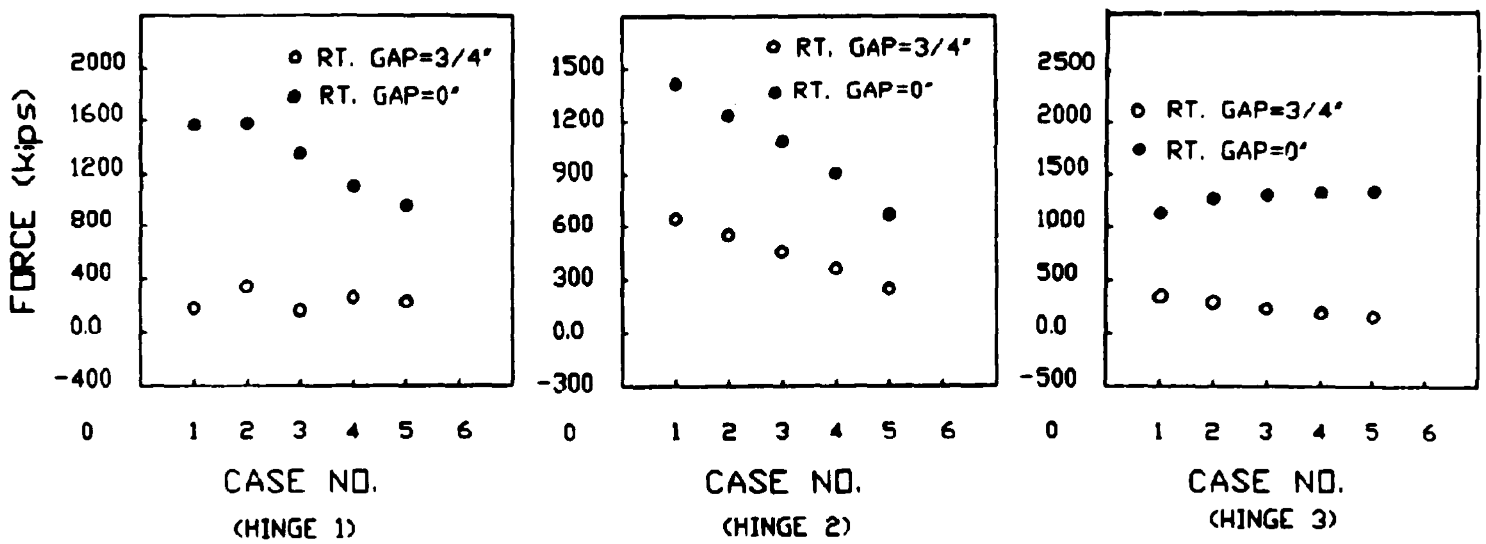


Fig. 5.5 - Maximum Restrainer Forces.

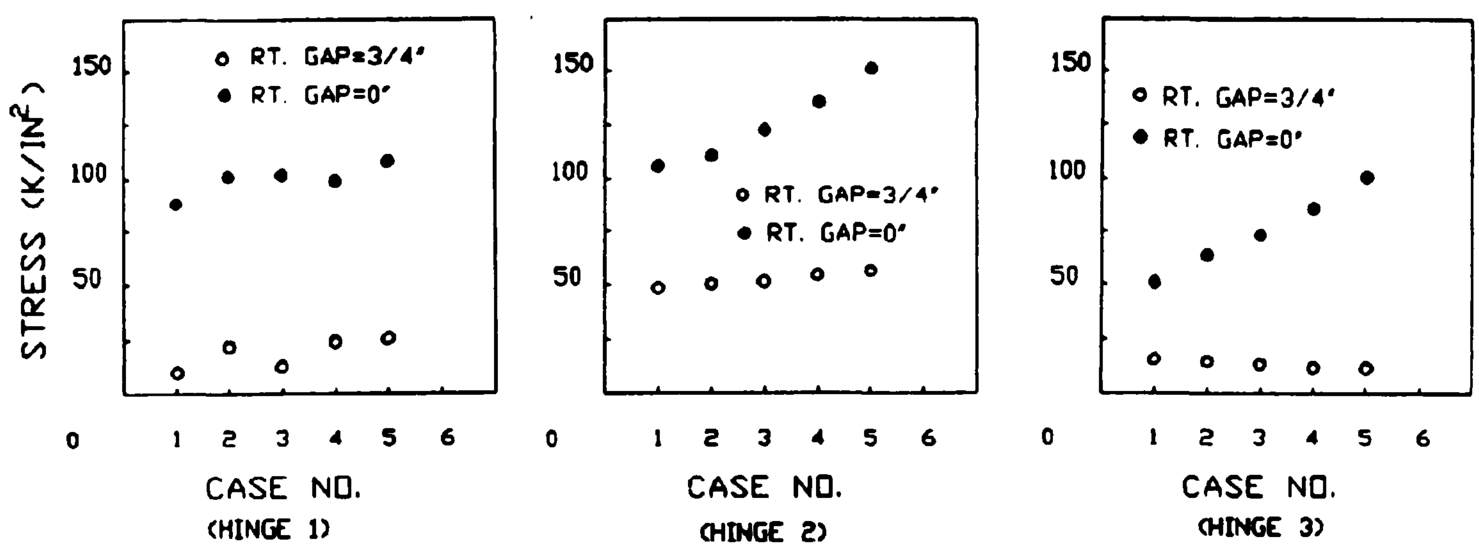


Fig. 5.6 - Maximum Restrainer Stresses.

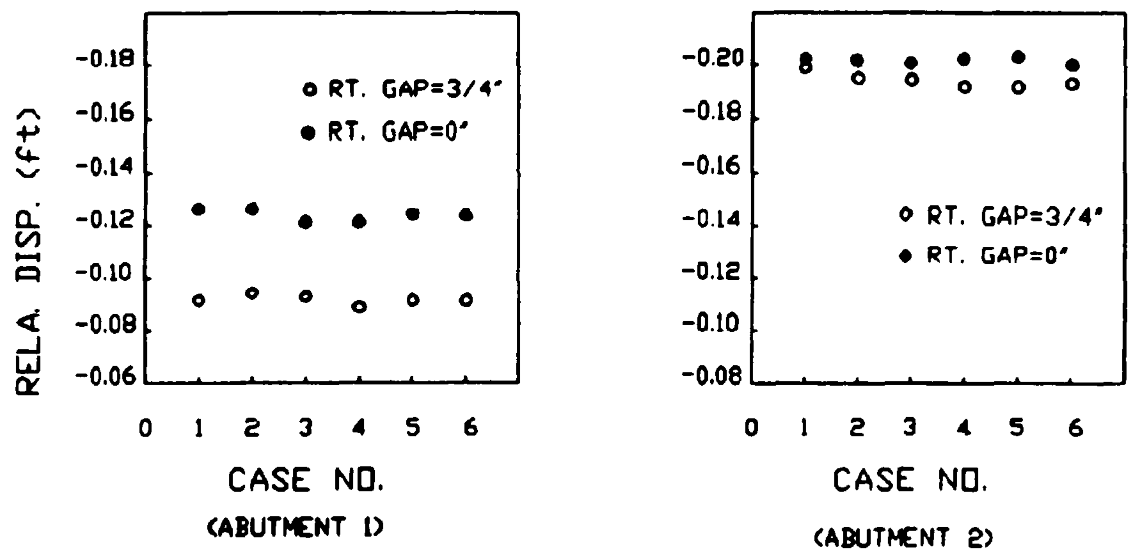


Fig. 5.7 - Maximum Relative Abutment Displacements.

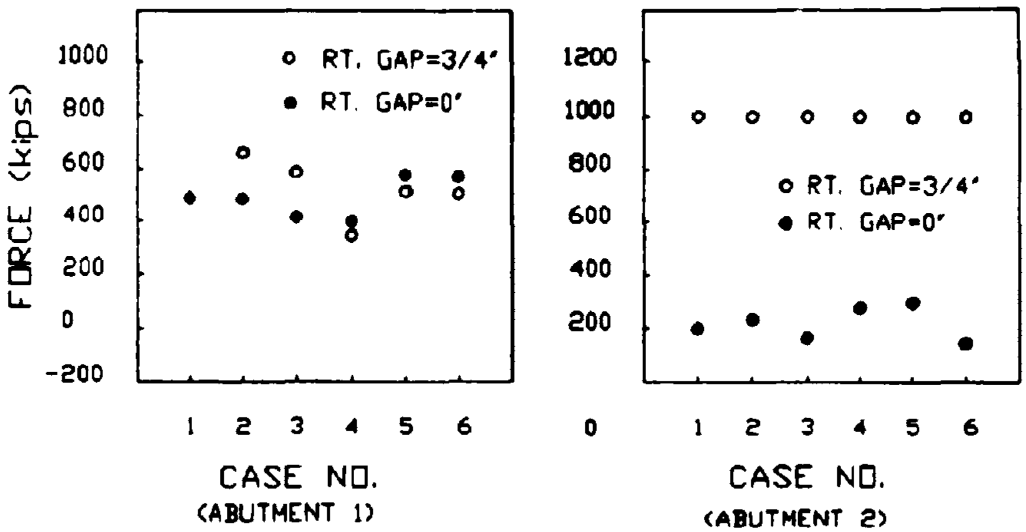


Fig. 5.8 - Maximum Abutment Forces.

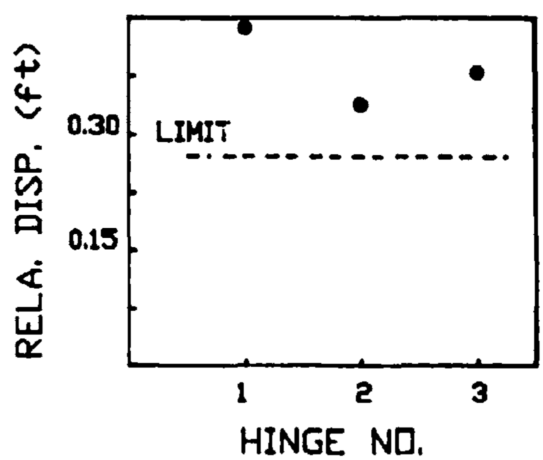


Fig. 5.9 - Maximum Hinge Movements in the Example Bridge w/o Restrainers.

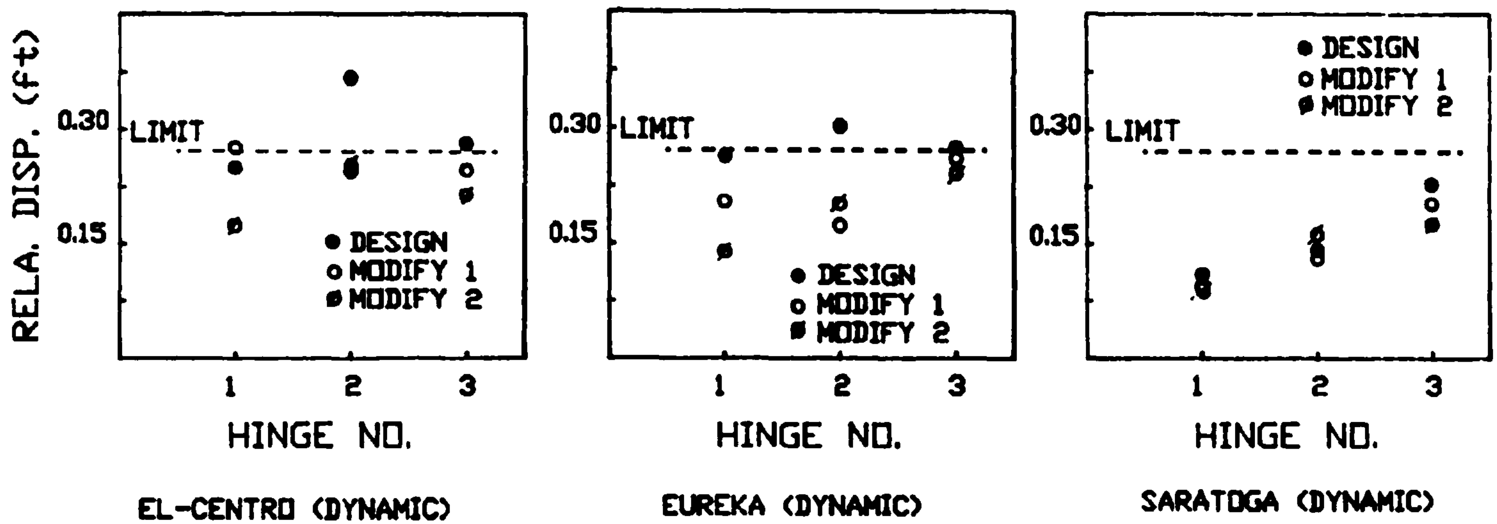


Fig. 5.10 - Results of Iterations for the Example Bridge.

LIST OF CCEER PUBLICATIONS

Report No.	Publication
CCEER-84-1	Saiidi, Mehdi and Renee A. Lawver, "User's Manual for LZAK-C64, A Computer Program to Implement the Q-Model on Commodore 64," Civil Engineering Department, Report No. CCEER-84-1, University of Nevada, Reno, January 1984.
CCEER-84-2	Douglas, Bruce M. and Toshio Iwasaki, "Proceedings of the First USA-Japan Bridge Engineering Workshop," held at the Public Works Research Institute, Tsukuba, Japan, Civil Engineering Department, Report No. CCEER-84-2, University of Nevada, Reno, April 1984.
CCEER-84-3	Saiidi, Mehdi, James D. Hart, and Bruce M. Douglas, "Inelastic Static and Dynamic Analysis of Short R/C Bridges Subjected to Lateral Loads," Civil Engineering Department, Report No. CCEER-84-3, University of Nevada, Reno, July 1984.
CCEER-84-4	Douglas, B., "A Proposed Plan for a National Bridge Engineering Laboratory," Civil Engineering Department, Report No. CCEER-84-4, University of Nevada, Reno, December 1984.
CCEER-85-1	Norris, Gary M. and Pirouze Abdollaholiaee, "Laterally Loaded Pile Response: Studies with the Strain Wedge Model," Civil Engineering Department, Report No. CCEER-85-1, University of Nevada, Reno, April 1985.
CCEER-86-1	Ghusn, George E. and Mehdi Saiidi, "A Simple Hysteretic Element for Biaxial Bending of R/C Columns and Implementation in NEABS-86," Civil Engineering Department, Report No. CCEER-86-1, University of Nevada, Reno, July 1986.
CCEER-86-2	Saiidi, Mehdi, Renee A. Lawver, and James D. Hart, "User's Manual of ISADAB and SIBA, Computer Programs for Nonlinear Transverse Analysis of Highway Bridges Subjected to Static and Dynamic Lateral Loads," Civil Engineering Department, Report No. CCEER-86-2, University of Nevada, Reno, September 1986.
CCEER-87-1	Siddharthan, Raj, "Dynamic Effective Stress Response of Surface and Embedded Footings in Sand," Civil engineering Department, Report No. CCEER-86-2, University of Nevada, Reno, June 1987.
CCEER-87-2	Norris, Gary and Robert Sack, "Lateral and Rotational Stiffness of Pile Groups for Seismic Analysis of Highway Bridges," Civil Engineering Department, Report No. CCEER-87-1, University of Nevada, Reno, June 1987.
CCEER-88-1	Orie, James and Mehdi Saiidi, "A Preliminary Study of One-Way Reinforced Concrete Pier Hinges Subjected to Shear and Flexure," Civil Engineering Department, Report No. CCEER-87-3, University of Nevada, Reno, January 1988.
CCEER-88-2	Orie, Donald, Mehdi Saiidi, and Bruce Douglas, "A Micro-CAD System for Seismic Design of Regular Highway Bridges," Civil Engineering Department, Report No. CCEER-88-2, University of Nevada, Reno, June 1988.
CCEER-88-3	Orie, Donald and Mehdi Saiidi, "User's Manual for Micro-SARB, a Microcomputer Program for Seismic Analysis of Regular Highway Bridges," Civil Engineering Department, Report No. CCEER-88-3, University of Nevada, Reno, October 1988.

CCEER-89-1 Douglas, Bruce, Mehdi Saiidi, Robert Hayes, and Grove Holcomb, "A Comprehensive Study of the Loads and Pressures Exerted on Wall Forms by the Placement of Concrete," Civil Engineering Department, Report No. CCEER-89-1, University of Nevada, Reno, February 1989.

CCEER-89-2 Richardson, James and Bruce Douglas, "Dynamic Response Analysis of the Dominion Road Bridge Test Data," Civil Engineering Department, Report No. CCEER-89-2, University of Nevada, Reno, March 1989.

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