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Evaluation of the Current CALTRANS Seismic Restrainer Design Method

Nevada Univ., Reno

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AN EVALUATION OF THE CURRENT CALTRANS SEISMIC RESTRAINER DESIGN METHOD

M. Saiidi, E. Maragakis, and S. Feng

A Report to the

California Department of Transportation
National Science Foundation
Nevada Department of Transportation

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University of Nevada, Reno

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ABSTRACT

The primary objective of this study was to develop an understanding of the implications of the current Caltrans hinge restrainer design procedure. Two aspects of the problem were studied. One was the effects of changing (a) the cross sectional area of restrainers and (b) the restrainer gap on the nonlinear response of a bridge with several hinges. The other was the sensitivity of the number of required restrainers to changes in some of the simplifying assumptions which are made in the current Caltrans restrainer design method.

Computer program NEABS-86 was used in the nonlinear analyses [3]. The focus of this part of the study was the relative displacements at the joints, restrainer forces, and restrainer stresses. Three earthquake records, the El Centro 1940, Eureka 1954, and Saratoga 1989. In addition to input earthquakes, the number of restrainers at each hinge, the restrainer gaps, and the hinge gaps were varied. It was found that when a restrainer gap of 0.75 in. is assumed, the number of restrainers does not affect the response significantly. It is recommended that the design should be based on cases with and without restrainer gaps to encompass all the critical forces, stresses, and displacements .

To study the effects of design assumptions on the required number of cables, several manual calculations of the example in the Caltrans restrainer design guidelines were carried out. The deviations from the method included the treatment of mass and stiffness of bridge segments as different hinges closed. Another variable was the simultaneous reduction in the restrainer gap and increase in the hinge gap. The results indicated that slight variation in some of the assumptions can change the number of restrainers significantly. A more streamlined design method that incorporates the nonlinear response of bridge components needs to be developed.

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Chapter 1
INTRODUCTION

1.1 Background

Since the San Fernando earthquake in 1971, restrainers have been generally incorporated in highway bridges in the State of California to avoid excessive displacements at hinges during strong earthquakes. The design of restrainers has been based on a relatively simple equivalent static analysis method that incorporates many of the primary factors affecting the seismic response of bridges [1]. Many simplifying assumptions are made some of which are based on engineering judgement and expectations, while others reflect the general 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 [2], many aspects of the restrainer design method have not been studied in detail. For example, it is not known what the effects of variation in the simplifying assumptions are. Furthermore, no studies have been conducted to evaluate the nonlinear response history of a bridge with restrainers which are designed using the current method. The purpose of the study presented in this report was to address some of these aspects of the hinge restrainer design method.

In writing this report, it was assumed that the reader is familiar or otherwise has access to Caltrans design guidelines and documents and has, at least, a general familiarity with dynamic nonlinear analysis of bridge structures.

1.2 Object and Scope

The primary objective of the research presented in this report was to provide a better insight into the implications of the current Caltrans hinge restrainer design method. Two aspects of the problem were studied. One was the effect of changing the number of restrainers and the restrainer gap on the nonlinear response of a bridge which includes several hinges. The other was the influence of changing some of the assumptions which are made in the restrainer design method on the number of required restrainers. In all the studies, the example bridge described in the Caltrans design manual was used as the reference case. Only straight bridges were considered in this study.

The focus of the nonlinear analyses was the relative displacements at the joints, restrainer forces, and restrainer stresses. The level of ductility demand in the piers was also examined to identify the extent of nonlinearity. Different earthquake records were used to insure that conclusions reflect the effects of a variety of ground excitations. In addition to the input earthquake, the number of restrainers at each hinge, the restrainer gap, and the hinge gap were also varied. Only longitudinal bridge response was considered to be consistent with the design method.

To study the effects of design assumptions on the required number of cables, several manual calculations of the design example in the Caltrans restrainer design guidelines were carried out. The general procedure was the same as that outlined in Ref. 1. The deviations from the method included a different treatment of mass and stiffness as different hinges closed as a result of earthquakes. Another variable was the simultaneous change in the restrainer and the hinge gaps.

Chapter 2

RESTRAINER DESIGN METHOD AND THE EXAMPLE BRIDGE

2.1 Summary of Restrainer Design Method

An equivalent linear static analysis method is currently used to design longitudinal hinge restrainers [1]. The primary features of the design are:

1) Restrainers are designed so that they will not yield even in the event of the maximum probable ground motion.

2) The formation of plastic hinges in the piers and the resulting reduction in the horizontal stiffness is accounted for.

3) The failure of the soil behind the abutment at certain displacements is taken into account.

4) The bridge is divided into a series of frames at the hinges. These frames may oscillate independently or in concert with adjacent frames depending on the expected maximum displacement and the hinge and restrainer gaps.

5) The displacement response of each frame is determined using the response spectrum method, based on the effective period of one or more frames. The seismicity and the characteristics of the site are accounted for in the smoothed acceleration spectra used for this purpose.

6) Restrainers are designed to avoid excessive relative displacements at the hinges so that a minimum bearing width is maintained.

A step-by-step summary of the restrainer design method extracted from Ref. 1 is presented in Appendix A.

2.2 Example Bridge

Figure 2.1 shows the elevation of the example bridge which is the subject of all the studies presented in this report. No span lengths are specified in Ref. 1 because only the superstructure weight and not the length enters the calculations. The span lengths shown in Fig.

2.1 were estimated for the purpose of the nonlinear analyses which are described in Chapter 3. The column height is 24 ft. and the bridge "active" width at the abutments is 40 ft. The superstructure depth is assumed to be 5 ft. The diaphragm at the right abutment is assumed to be 5 ft. high and is assumed to shear off at a longitudinal displacement of 2 in. The stiffness of the spring representing the abutment soil is taken as 200 K/in. per linear foot of the abutment width for an 8-ft. deep soil wedge. For the same soil wedge, the soil is assumed to fail at a pressure of 7.7 Ksf. Because the superstructure depth is only 5 ft., this stiffness and the capacity of abutment spring is reduced by a factor of 5/8.

One half of the columns in each frame are assumed to behave as pinned members at both ends due the expected inadequacies in the column bar embedment lengths, lap splice lengths, and foundation. The gross moment of inertia of the columns about the transverse axis of the bridge is assumed to be 26 ft^4. The actual compressive strength of concrete is estimated at 5000 psi. It is assumed that the bridge is at an alluvium site with a depth of 10 to 80 ft and a peak bed rock acceleration of sixty percent of gravity.

The example bridge represents the pre-1971 designs in which the typical hinge width was 6 in. The hinge gap and the restrainer gap are both assumed to be 0.75 in. Figure 2.2 shows the hinge detail and the assumed minimum required seat width.

Chapter 3
NONLINEAR MODELING OF THE EXAMPLE BRIDGE

3.1 Introductory Remarks

A series of nonlinear response history analyses were carried out on the example bridge described in the previous chapter. The purpose of the study was to determine the restrainer forces and stresses as well as relative displacements at hinges as a function of the number of restrainers, the input earthquake record, and the restrainer gap. Because the properties specified in Ref. 1 for the example bridge were suited only for an equivalent static analysis, many assumptions had to be made about geometric and section properties of different bridge components in order to perform the nonlinear analyses. This chapter first explains these assumptions and the model used in the analysis, followed by a description of the parameters which were studied.

3.2 Description of the Bridge Model

The nonlinear response history analysis of the bridge was performed using computer program NEABS-86 [3] which is a modified version of program NEABS (Nonlinear Earthquake Analysis of Bridge Systems) [4]. In the equivalent static analysis method, each frame is treated as a single-degree-of-freedom (SDOF) system which consists of a single rigid mass representing the superstructure. As a result, span lengths are not needed. To analyze the example bridge using NEABS-86, however, the span lengths and hinge locations had to be given as input. These values were estimated based on the weight of each frame and the superstructure width and height as presented in the design example. The concrete was assumed to be normal weight with a unit weight of 150 pcf. It was further assumed that the superstructure weight is uniformly distributed along its length. Each pier was assumed to consist of three square columns with a side dimension of 3.2 ft. The combined moment of inertia of each pier in the longitudinal direction is hence 26.2 ft^4 which is very close to the specified value of 26 ft^4. To determine the yield properties of the column sections, it was assumed that the longitudinal steel ratio in each column is one percent and that the steel has a yield stress of 60 ksi.

The computer model of the bridge is shown in Figs. 3.1 and 3.2. The adjacent nodes at the base of each column (Fig. 3.1) allowed the modeling of the nonlinear response at the base of the columns using the five-spring element [3]. To be consistent with the design example, all the pier footings and abutments were assumed to be fixed except for the degree of freedom of the abutments in the longitudinal direction of the bridge.

To model the nonlinear effects at the abutments, fictitious restrainer elements had to be assumed because the boundary spring elements in NEABS-86 which are intended to represent the flexibility of the abutments and foundation are linear. At each abutment, two expansion joint elements were needed, one to model the nonlinearity of the abutment soil and the other to model the impact between the superstructure and the abutment. Ordinarily, a single hinge element would be sufficient to model both the impact effect and the nonlinearity of the restrainer. This would work in hinges where opening of the hinge would activate the restrainer but closing of the hinge would activate the impact spring. In the fictitious element at the abutment, however, both the impact spring and the abutment soil spring (modeled by a restrainer) need to be activated when the gap closes. Therefore, two separate hinge elements had to be used.
In the hinge element representing the abutment soil, the input hinge node numbers were specified in such a way that closing of the gap would put the "restrainer" in tension. The impact spring in this case was deactivated. The hinge element modeling the impact effect had no restrainers. Only the intermediate hinges incorporated true hinge restrainers.

Only cable restrainers were assumed in the study with a yield stress of 176.1 ksi [1]. The stiffness of impact springs was determined based on the guidelines recommended in Ref. 5. According to these guidelines, the spring stiffness should approximately be the same as the axial stiffness of the superstructure segment on each side of the hinge. It is reasonable to use the larger of the two values when superstructure stiffnesses on both sides of a hinge are different. In the example bridge, impact stiffness values in the range of 8.4×10^5 k/ft to 9.1×10^5 k/ft were used (See Appendix B).

The differential equation of motion in NEABS-86 is formed in incremental form and is integrated using constant or linear variation of acceleration over each time interval. In the studies, the constant acceleration method was used to obtain stable results. The time interval for numerical integration can affect the convergence of the results. Because the bridge model can be highly nonlinear, the appropriate time interval was determined by attempting different values and study their effect on the relative displacements at the hinges and abutments. The relative displacement at these locations was believed to be the most critical response parameter as it can directly affect the restrainer forces. The results of the trial analyses are listed in Table 3.1. It can be seen that no appreciable improvement could be realized by reducing the time interval below 0.005 second. This time interval was used throughout the study.

A sample input data for the "reference" bridge (the bridge with the same number of cables as those specified in the Caltrans design example) used in the NEABS-86 analysis is shown in Appendix B.

3.3 Input Earthquake Records

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. 1. The first ten seconds of each record were used. Figure 3.3 shows the acceleration spectrum for the three earthquakes calculated at a damping ratio of five percent. The spectra show notable differences particularly when the vibration period is less than one second. The estimated periods for different frames in the example bridge are close to 0.5 second according to Ref. 1.

3.4 Variation of Parameters

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 3.2 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 [1]. 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. Because NEABS-86 allows for a maximum of only six restrainers at each hinge, groups of the cables had to be lumped. As a result "equivalent cables" were assigned at different hinges. The number of equivalent cables in Hinges 1, 2, and 3, is 4, 3, and 5, respectively. 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. 1 is for the "hottest 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 3.2 were analyzed for both the extreme high (restrainer gap of 0.75 in.) and extreme low (restrainer gap of zero) ambient temperatures.

Chapter 4

RESULTS OF THE NONLINEAR RESPONSE STUDIES

4.1 General Characteristics of the Dynamic Results

The bridge model discussed in Chapter 3 was analyzed for 36 combinations of restrainer numbers, input earthquakes, and restrainer gaps using computer program NEABS-86 [3]. 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 relative movements at the hinges and abutments, restrainer forces and stresses, and abutment movements and forces. Before these effects are discussed, however, sample response histories and the extent of ductility demand in the piers are discussed in this section to demonstrate the extent of nonlinearity that the example bridge experienced in the parametric studies.

It will be seen in the next section that the response at hinge 2 was typically more critical than others. Figure 4.1 shows the relative displacement response history at hinge 2 due to the El Centro record when the restrainer gap was 0.75 in. A positive value indicates that the two segments of the superstructure adjacent to the hinge move away from each other. Also shown is the restrainer force history. The number of the restrainers is the same as that in Case 1 (Table 3.2). Note that a relative displacement of 0.0625 ft. (0.75 in.) is necessary to activate the restrainers. It can be seen that during the earthquake, the restrainers experienced tension eight times, each when the relative hinge movement exceeded 0.0625 ft. The force magnitude was different depending on the relative displacement in each case. The maximum restrainer force was far below the yield force. For the same case, the relative displacement history and the force history at abutment 2 are shown in Fig. 4.2. The hinge gap at this abutment is 0.167 ft. The results show that this displacement was exceeded three times during the earthquake. At t=5.1 sec., the abutment force reached the yield level which was 1000 kips.

In all the parametric studies, the piers experienced a moderate level of yielding. The 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 moment reaches the yield level. In establishing the yield displacement, shear deformation of the column was ignored because the column aspect ratio (ratio of column height to its width) was 7.5.

4.2 Results of Parametric Studies (Cable Gap=0.75 in.)

Restrainers are used in bridges to limit the relative hinge displacements below a tolerable limit. With a restrainer gap of 0.75 in., the permissible relative displacement at the hinge is 2.25 in. (Fig. 2.2). The performance of restrainers is considered to be satisfactory when the displacement is kept below the permissible limit without any yielding of the restrainers and any damage to the hinge region. Therefore, the restrainer stresses and hinge movements are of primary concern. Also important are the movements at the abutments and the resulting forces because of the close interaction of displacement at the abutments and at hinges, even though abutments are not equipped with restrainers. This section presents hinge and abutment movements and forces for different earthquakes and different number of restrainers when the restrainer gap is 0.75 in.

4.2.1 Hinge Movements

Shown in Fig. 4.3 is the maximum relative displacement of the two segments of the superstructure adjacent to each hinge. The general observations on this figure are:

1) The relative hinge displacements were insensitive to the changes in the number of cables including Case 6 in which no restrainers had been used. This was true regardless of the input ground motion.
2) The maximum relative movement at hinges varied depending on the earthquake.
3) Hinge 2 experienced the largest relative displacement in all earthquakes. This is because the proximity of the two other hinges to the abutments helped reduce their movement and because Hinge 2 has the smallest number of restrainers. The estimated hinge movement in Ref. 1 is the smallest for Hinge 2 which indicates that a revision in the design method may be necessary.
4) The permissible relative displacement at the hinges when restrainer gap is 0.75 in. is 2.25 in. (0.1875 ft.). It can be seen in Fig. 4.3 that this limit was not reached even when no restrainers were used.

4.2.2 Restrainer Forces and Stresses

Figures 4.4 and 4.5 show the maximum restrainer forces and stresses, respectively. The forces shown are for each "equivalent cable" at each hinge. These forces need to be multiplied by 4, 3, and 5, to obtain the total restrainer forces at hinges 1, 2, and 3, respectively. The equivalent cables had to be used due to the restriction in NEABS-86 on the number of restrainers at each hinge (See Sec. 3.4). Note that Case 6 had no restrainers. The following observations are made:

1) Restrainer forces generally decreased as the number of restrainers was reduced. This was particularly clear for Hinge 2 which experienced the largest relative displacements. The reduction in restrainer forces is attributed to the reduction in the stiffness of the links between adjacent superstructure segments.
2) The maximum restrainer forces changed as the input earthquake motion varied.
3) Unlike forces, the restrainer stresses were generally insensitive to the number of cables. This is because of the reduction in cross sectional area of the restrainers as the number of cables decreased.
4) The maximum restrainer stress was well below the yield stress of 176.1 ksi in all cases.

4.2.3 Abutment Displacements

The maximum relative displacements between the abutments and the superstructure are shown in Fig. 4.6. The negative sign of the displacement amplitudes signifies the movement of the superstructure toward the abutment. Only displacements that tend to reduce or close the abutment gap were considered, because only the gap closure affects abutment forces. The following trends can be noted:

1) The maximum displacement of the superstructure relative to the abutment was not affected by the number of restrainers.
2) Different input earthquakes resulted in different maximum displacements.
3) The movement at Abutment 2 was always larger than that in Abutment 1 because the gap in the former was 2 in., while in the latter was 1 in. The larger gap allowed the superstructure to move more.

4.2.4 Abutment Forces

The maximum abutment back fill forces in the longitudinal direction of the bridge are shown in Fig. 4.7. Note that for the abutment to experience any forces, it is necessary that the hinge gap at the abutment be closed. The yield force for both abutments was 1000 kips. The figure indicates the following:

1) Except for the Eureka earthquake, the number of cables did not influence the abutment forces significantly.
2) The abutment forces were sensitive to the input ground motion. While the El Centro earthquake caused the yielding of Abutment 2, the Saratoga earthquake caused only small forces in the abutments.

4.3 Results of Parametric Studies (Cable Gap=0 in.)

All the computer analyses discussed in Sec. 4.2 were repeated for bridge models without any gap at the restrainers. A reduction in the restrainer gap led to an increase in to hinge gap to 1.5 in. (0.125 ft.). Because the total seat width is 6 in. (Fig. 2.2), to maintain a minimum of 3 in. bearing (a criterion set in the design example in Ref. 1) the maximum permissible relative displacement between the two segments of the superstructure adjacent to each hinge is 1.5 in. (0.125 ft.). The permissible limit was 2.25 in. in the cases presented in Sec. 4.2.

4.3.1 Hinge Movements

Shown in Fig. 4.8 are the maximum relative superstructure movements at different hinges. The following general observations can be made on this figure:

1) In most cases, the relative hinge displacements increased as a result of reduction in the number of cables. When no restrainers were used (Case 6), the displacements in Hinges 2 and 3 exceeded the permissible limit. This was true regardless of the input ground motion.
2) The maximum relative movement at Hinge 1 varied depending on the earthquake.
3) Hinge 2 experienced the largest relative displacement in all earthquakes. This is because Hinge 2 is away from the abutments and movements at this hinge are not directly restricted by the abutments.

4.3.2 <u>Restrainer Forces and Stresses</u>

Figures 4.9 and 4.10 show the maximum restrainer forces and stresses, respectively. Note that the forces shown are per "equivalent cable". The results indicate the following:

1) Restrainer forces generally decreased as the number of restrainers was reduced. This was particularly clear at Hinge 2 which experienced the largest relative displacement. The reduction in restrainer forces is attributed to the reduction in the stiffness of the cables connecting the adjacent superstructure segments.
2) The maximum restrainer forces depended on the input earthquake.
3) Unlike forces, the restrainer stresses generally increased as the number of cables was reduced. This is because of the reduction in cross sectional area of the restrainers and the increase in the relative displacements (Fig. 4.8) as the number of cables decreased.
4) The maximum restrainer stresses were below the yield stress of 176.1 ksi in all cases. However, the cable stresses in Case 5 were close to 85 percent of this limit.

4.3.3 <u>Abutment Displacements</u>

The maximum relative displacements between the abutments and the superstructure are shown in Fig. 4.11. Note that the values are for displacement that tended to reduce or close the abutment gap. Because the bridge is considered to be under the extreme low ambient temperature, the gaps are 1.375 in. (0.115 ft.) and 2.375 in. (0.198 ft.) at Abutments 1, and 2, respectively. The following trends can be noted:

1) The maximum displacement of the superstructure relative to the abutment was only slightly affected by the number of restrainers.
2) Different input earthquakes resulted in different maximum displacements. The maximum displacement in most cases was sufficiently large to close the abutment gap and activate the back fill soil.
3) The movement at Abutment 2 was always larger than that in Abutment 1 because the gap in the former was 2.375 in., while in the latter was 1.375 in. The larger gap allowed the superstructure to move more.

4.3.4 Abutment Forces

The maximum abutment back fill forces in the longitudinal direction of the bridge are shown in Fig. 4.12. Note that for the abutment to experience any forces, it is necessary that the hinge gap at the abutment be closed. The figure indicates the following:

1) The effect of reduction in the number of cables on the abutment forces appears to be non-uniform. This is true for all three earthquake records, although the pattern varies from one earthquake to another.

2) The abutment forces were sensitive to the input ground motion. However, in all cases the forces remained well below the yield force of 1000 kips. The increase in the abutment gaps and the tightening of the restrainers appears to have reduced the demand on the abutments.

4.4 Effects of Reduction in the Restrainer Gap

One of the primary factors considered in the parametric studies was to determine the effect of reducing 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 reduce to zero during low temperatures due to the contraction of the superstructure. The results presented in Sec. 4.2 and 4.3 are compared and discussed in this section. To facilitate the comparisons, the envelopes of the results for different earthquakes are considered. These envelopes present the largest of the response maxima caused by different input ground motions.

4.4.1 Hinge Movements

Figure 4.13 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.

4.4.2 Restrainer Forces and Stresses

The total restrainer forces and stresses are compared for different hinge gaps in Figs. 4.14 and 4.15, 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 4.15 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.

4.4.3 Abutment Displacements

The envelopes of the maximum closing relative displacements between the superstructure and the abutments are shown in Fig. 4.16. 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.

4.4.4 Abutment Forces

The larger openings at abutments in cold temperatures resulted in generally smaller forces in the back fill soil (Fig. 4.17). 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.

4.5 Concluding Remarks

The results presented in this chapter 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 restrainers include both the extreme high and low temperatures conditions.

The current design procedure [1] predicted that Hinge 2 will experience the minimum relative displacement, and hence, the lowest number of restrainers was used at this hinge. The dynamic results presented in this Chapter, however, showed that Hinge 2 would experience the largest amount of displacement among the three hinges. The design method described in Ref. 1 needs to be modified to better represent the dynamic behavior of the bridge.

Chapter 5
SENSITIVITY OF THE EQUIVALENT STATIC DESIGN RESULTS TO CHANGES IN DESIGN ASSUMPTIONS

5.1 Introductory Remarks

The equivalent static analysis method for restrainer systems is intended to be a rational yet practical procedure for manual design of hinge restrainers [1]. The primary features of the method were discussed in Chapter 2. Several simplifying and judgmental assumptions are incorporated in calculating the effective stiffness and mass which in turn influences the effective vibration period of different frames or groups of frames. Furthermore, the initial restrainer gap is assumed to be 0.75 in. which corresponds to extreme high ambient temperature.

The purpose of the study presented in this chapter was to determine the effect of (1) the changes in the way the effective stiffness and mass is calculated and (2) the elimination of the restrainer gap. The criterion to measure the effect of these variations is the number of required restrainers. The example bridge presented in Ref. 1 (Fig. 2.1) was used to illustrate the effects. The design procedure and the properties used in the study were the same as those in Ref. 1 except as noted.

5.2 Influence of Changes in the Stiffness

To determine the movements that must be resisted 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 away from the hinge, it closes the gap at an adjacent hinge, thus mobilizing a second frame. If the displacement continues to increase, another hinge may close which leads to the mobilization of a third frame or the abutment. The design method [1] accounts for the closure of only one adjacent hinge and permits the mobilization of only one adjacent frame. This assumption is intended to be conservative and simple. 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 though more than one frame may be mobilized. The purpose of this section is to discuss the effect of allowing more than one adjacent frame to contribute to stiffness when displacements are sufficiently large to permit gap closure. The mass calculation in this section followed the procedure used in Ref. 1.

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 the 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 5.1 shows the stiffness chart for the left side of Hinge 1 as it is presented in Ref. 1 and as refined in this study. The differences between Charts (a) and (b) are:

1) Chart (b) uses a stiffness of 1100 k/in which is specified for this frame in Ref. 1, while Chart (a) rounds this stiffness down to 1000 k/in.

2) Upon closure of the gap (part "c" on the Chart), Chart (b) adds the abutment stiffness to the frame stiffness, whereas in the original design, the abutment stiffness is used for the total stiffness.

The consequence of the above modifications is that the effective stiffnesses in Chart (b) are approximately ten percent larger than those in Chart (a).

The differences are more pronounced when the movement of the bridge to the right of Hinge 1 is considered (Fig. 5.2). 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. 1 for the 5-ft. and 7-ft. cables, respectively.

A more accurate plot of stiffnesses in Fig. 5.3 for movement on the left side of Hinge 2 led to an increase in the equivalent stiffness of 26 percent and 11 percent for the 5-ft. and 7-ft. cables, respectively. The differences for movement on the right side, however, are only 4 percent or less (Fig. 5.4).

When movement of the bridge on the right side of Hinge 3 is considered (Fig. 5.5) and the 5-ft. cables are used, the equivalent stiffness is the same in both Charts. For the 7-ft. cable, however, the abutment is mobilized. In the refined analysis, the abutment stiffness was added to the frame stiffness, while in Ref. 1 the abutment stiffness was substituted for the total stiffness of the frame and the abutment. The result was an increase of 9 percent in the equivalent stiffness corresponding to the 7-ft. cable.

Following the above refined stiffnesses and the procedure used in Ref. 1, the restrainers were designed and checked to insure that the maximum displacements are within the permissible limits. The number of required ten-cable units of 3/4 in. cables is shown in Table 5.1. Column 3 in the table lists the number of units as determined in the original design. It can be seen that the refinements in the equivalent stiffnesses led to a significant reduction in the number of required cables, especially for the 7-ft. cables.

5.3 Influence of Changes in Stiffness and Mass

When hinge gaps are closed as a result of the movement of one or more frames, it is reasonable to assume that the effective period is affected by both the stiffness and mass of all the segments. In calculating the cable forces in between the segments, however, the mass of individual segments need to be used. The design method of Ref. 1 uses the mass for only one segment even in computing the effective period of two segments. To determine the effect of changing the mass to the total mass of all the contributing segments, the restrainers in the example bridge were redesigned. The modified stiffnesses discussed in Sec. 5.2 were used. The number of required restrainers is shown in Col. 5 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 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.

5.4 Influence of Reducing Restrainer Gap to Zero

During the extreme low ambient temperatures, the superstructure segments become shorter. As a result, the restrainer gap may diminish while the hinge and abutment gaps may 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 hinges, Abutment 1, and Abutment 2, respectively. (Recall that during the hottest ambient temperature, the gaps in the hinges, abutment 1, and abutment 2 were 0.75 in., 1 in., and 2 in., 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. Figures 5.6 to 5.10 show the stiffness charts for different conditions. It can be seen that because of the small permissible cable deflections and because of large hinge gaps, none of the gaps can be closed without yielding the restrainers. As a result, each segment of the bridge oscillates individually.

As expected, a large number of restrainers are required to limit the displacements to the above levels. Column 6 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, suggesting that the critical condition for the design of restrainers is when the restrainer gaps are zero.

5.5 Concluding Remarks

The results discussed in Sec. 5.2 and 5.3, 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. 1. The high degree of sensitivity of the results to these assumptions strongly suggests that an alternate design method may have to be developed.

The considerable increase in the required number of cables as a results of the elimination of the restrainer gaps shows that the worst condition for restrainer design is under low ambient temperatures. This conclusion is in complete agreement with the observations in the results of the nonlinear analyses presented in Chapter 4.

Chapter 6
SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 Summary

The primary objective of this study was to develop an understanding of the implications of Caltrans' current hinge restrainer design practice. Two aspects of the problem were studied. One was the effects of changing (a) the cross sectional area of restrainers and (b) the restrainer gap on the nonlinear response of a bridge with several hinges. The other was the sensitivity of the number of required restrainers to changes in some of the simplifying assumptions which are made in the restrainer design method. In all the studies, the example bridge described in the Caltrans design manual [1] was used as the reference case. The study was limited to the longitudinal response of straight bridges which utilized cable restrainers.

The computer program NEABS-86 was used in the nonlinear analyses [3]. The focus of this part of the study was the relative displacements at the joints, restrainer forces, and restrainer stresses. The level of displacement ductility demand in the piers was also examined to identify the extent of nonlinearity. Three earthquake records, the El Centro 1940, Eureka 1954, and Saratoga 1989, were used to insure that conclusions reflect the effects of a collection of earthquakes with different response spectra. In addition to the input earthquake, the number of restrainers at each hinge, the restrainer gap, and the hinge gap were varied. Only longitudinal bridge response was considered to be consistent with the design method.

To study the effects of simplifying assumptions incorporated in the design method on the required number of cables, several manual calculations of the example in the Caltrans restrainer design guidelines were carried out. The general procedure was the same as that outlined in Ref. 1. The deviations from the method included the treatment of mass and stiffness of bridge segments as different hinges closed. Another variable was the simultaneous the reduction in the restrainer and increase in the hinge gap. This condition would represent the effect of low ambient temperatures on the gaps.

6.2 Conclusions

The following conclusions were drawn based on the results of the study.

6.2.1 Conclusions from the Nonlinear Analyses

1) The relative hinge displacements are insensitive to the changes in the number of cables when restrainers have a gap. When the restrainer gap was eliminated, the relative displacements reduced as a result of an increase in the number of restrainers.
2) The equivalent static analysis method does not necessarily represent the dynamic response of the bridge. In the example bridge, the former predicted the least displacement at the middle hinge while the dynamic result showed that this hinge would experience the largest maximum relative displacement.
3) Unlike forces, the restrainer stresses were generally insensitive to the number of cables when a restrainer gap of 0.75 in. was present. When the gap was reduced to zero, a reduction in the number of restrainers increased the restrainer stresses.
4) The maximum displacement of the superstructure relative to the abutment was not affected by the number of restrainers regardless of the restrainer gap.
5) Abutment forces are sensitive to the input ground motion. Therefore, it is necessary that the bridge be analyzed for a variety of earthquake records representing the characteristics of the site at which the bridge is located.
6) The use of computer program NEABS for nonlinear earthquake response history analysis is very cumbersome. The nonlinearity of boundary elements has to be artificially modeled by assumptions that do not represent the true behavior. In addition, the program is outdated in terms of software development techniques. A simpler, more realistic computer program for nonlinear seismic response history analysis of bridges is urgently needed.

6.2.2 Conclusions from the Redesigns of the Example Bridge

1) When a restrainer gap of 0.75 in. was present, changes in the equivalent stiffnesses led to a significant reduction in the number of required restrainers, particularly for 7-ft. cables.

2) By changing the treatment of the mass in the restrainer design method, the number of cables were reduced drastically. This was for the restrainer gap of 0.75 in.

3) The number of required hinge restrainers became substantially higher when the restrainer gap was reduced to zero.

6.3 Recommendations

The results presented in this report 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 restrainers include both the extreme high and low temperatures conditions.

The current design procedure [1] predicted that Hinge 2 will experience the minimum relative displacement, and hence, the lowest number of restrainers was designed at this hinge. The dynamic results presented in this report, however, showed that Hinge 2 would experience the largest amount of displacement among the three hinges. The design method described in Ref. 1 needs to be modified to better represent the dynamic behavior of the bridge.

The results discussed in Sec. 5.2 and 5.3, 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. 1. The high degree of sensitivity of the results to these assumptions strongly suggests that an alternate design method may be need.

The considerable increase in the required number of cables as a result of the elimination of the restrainer gaps shows that the worse condition for restrainer design is under low ambient temperatures, if the current Caltrans restrainer is to be used. A more rational nonlinear dynamic analysis would need to consider both zero and non-zero restrainer gaps to determine the critical forces at abutments as well as restrainers.

REFERENCES

1. "Seismic Design References" State of California, Department of Transportation Division of Structure, July, 1990, and the Restrainer Design Example Update, July 1991.

2. Saiidi, M., E. Maragakis, S. Abdel-Ghaffar, and D. O'Connor, "Effect of Hinge Restrainers on the Response of Highway Bridges during the Loma Prieta Earthquake," Proceedings, Third Workshop on Bridge Engineering Research in Progress, San Diego, California, November 1992.

3. Saiidi, M., G. Ghusn, and Y. Jiang, "Five-Spring Element for Biaxially Bent R/C Columns," Journal of Structural Engineering, ASCE, Vol. 115, No. 2, February 1989, pp. 398-416.

4. "Earthquake Resistant Bridge Bearing, Vol. 2, NEABS Computer Program" U.S. department of Transportation, Federal Highway Administration, FHWA/RD-82/166, May 1983.

5. Kawashima K. and J. Penzien, "Correlative Investigations on Theoretical and Experimental Dynamic Behavior of a Model Bridge Structure" Report No. 76-26, Earthquake Engineering Research Center, University of California, Berkeley, July 1976.

Table 3.1 - Maximum Relative Displacements (ft)
(t= Time interval for integration)

Time Interval	t=0.02 (sec.)	t=0.01 (sec.)	t=0.005 (sec.)	t=0.002 (sec.)	t=0.001 (sec.)
Abut. 1	0.1803	0.1572	0.1569	0.1568	0.1568
Hinge 1	0.1237	0.07066	0.07084	0.06744	0.06718
Hinge 2	0.1038	0.08653	0.07567	0.07508	0.07499
Hinge 3	0.0751	0.08424	0.07109	0.06864	0.06848
Abut. 2	0.2176	0.2230	0.1993	0.1928	0.1931

Table 3.2 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

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

(1) HINGE	(2) CABLE LENGTH (ft.)	(3) REF. 1	(4) MODIFIED EQ. STIFF.	(5) MODIFIED EQ. STIFF. & MASS	(6) ZERO REST. GAP
1	5 7	10 8	9 5	4 -	15 13
2	5 7	12 7	9 5	7 2	15 14
3	5 7	13 11	11 10	6 2	16 15

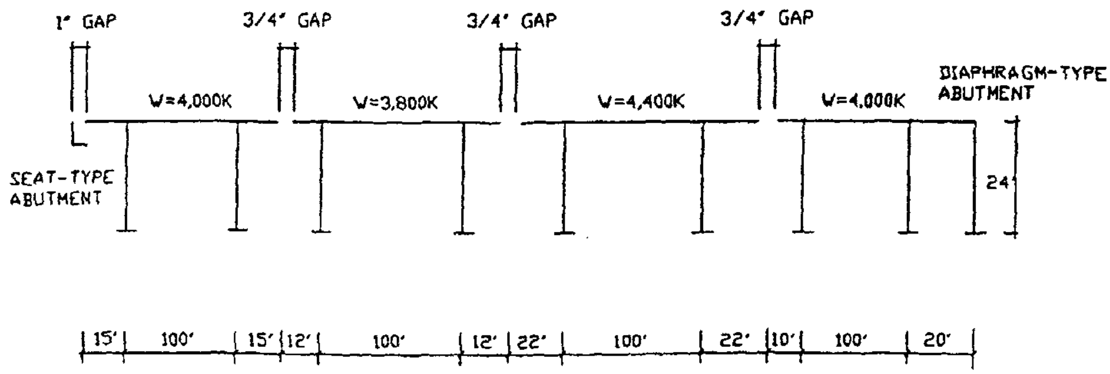


Fig. 2.1 - The Elevation of the Example Bridge (Ref. 1).

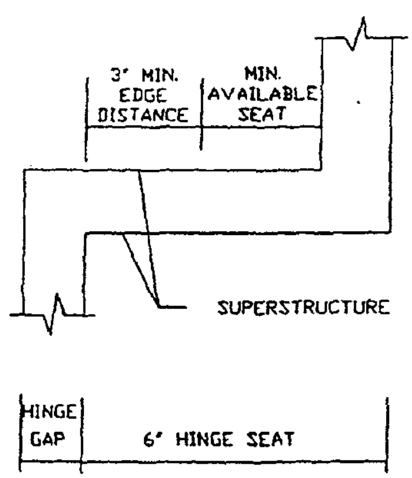


Fig. 2.2 - The Hinge Detail in the Example Bridge.

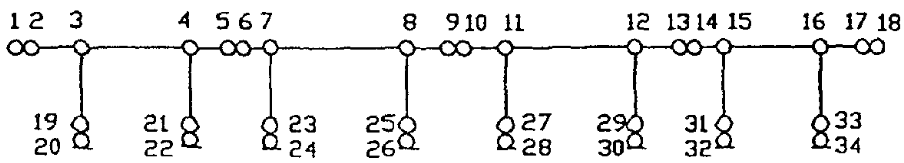


Fig. 3.1 - Node Numbers in the NEABS-86 Model.

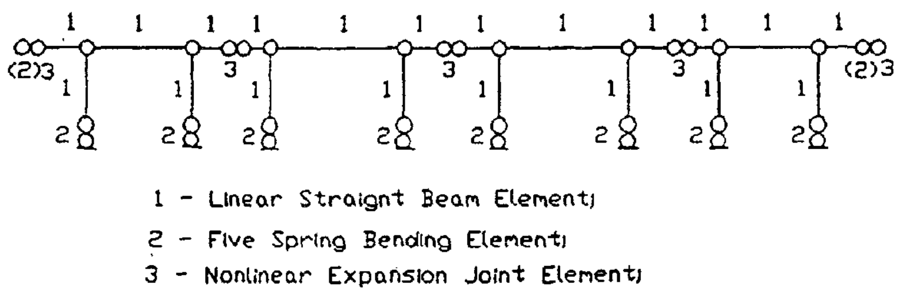


Fig. 3.2 - Element Types in the NEABS-86 Model.

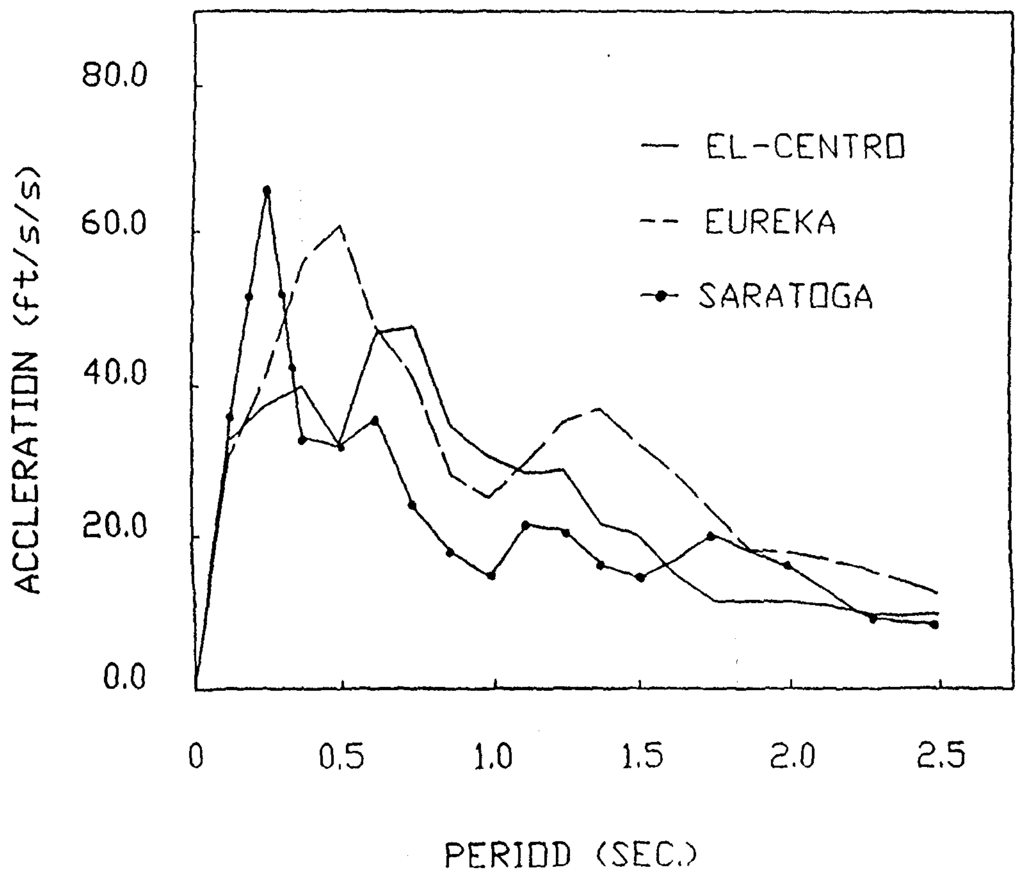
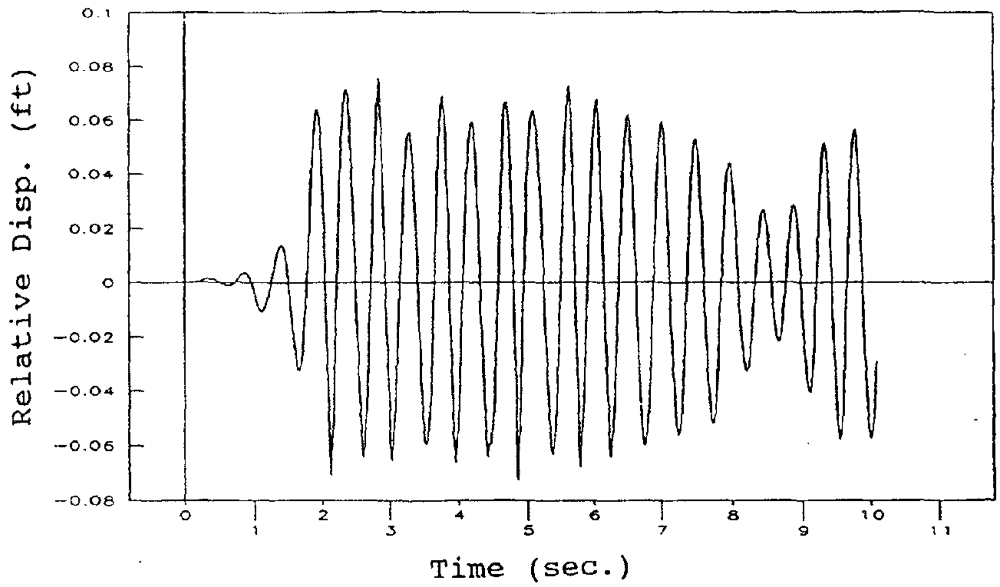
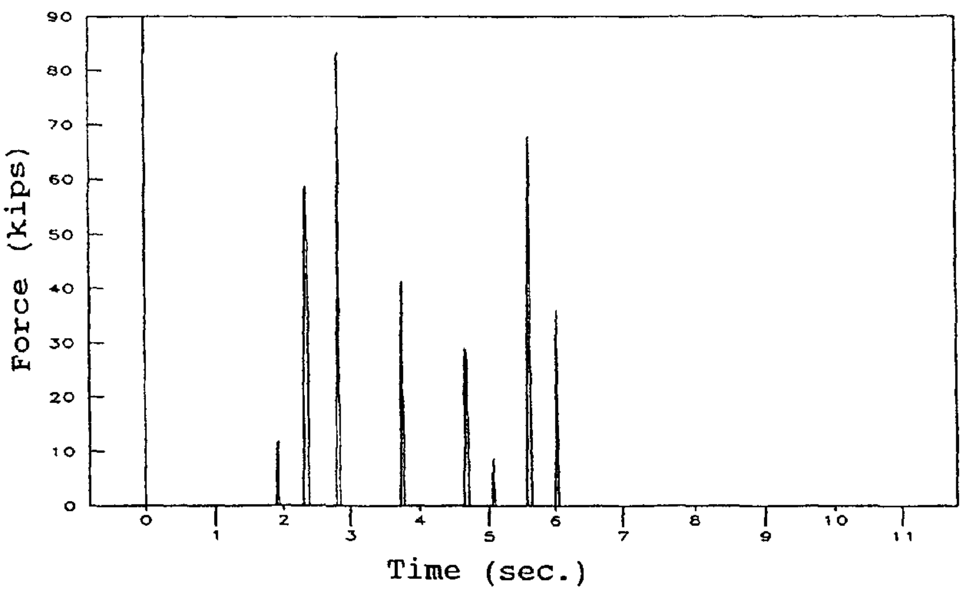


Fig. 3.3 - Response Spectra for the Input Records at 5% Damping.

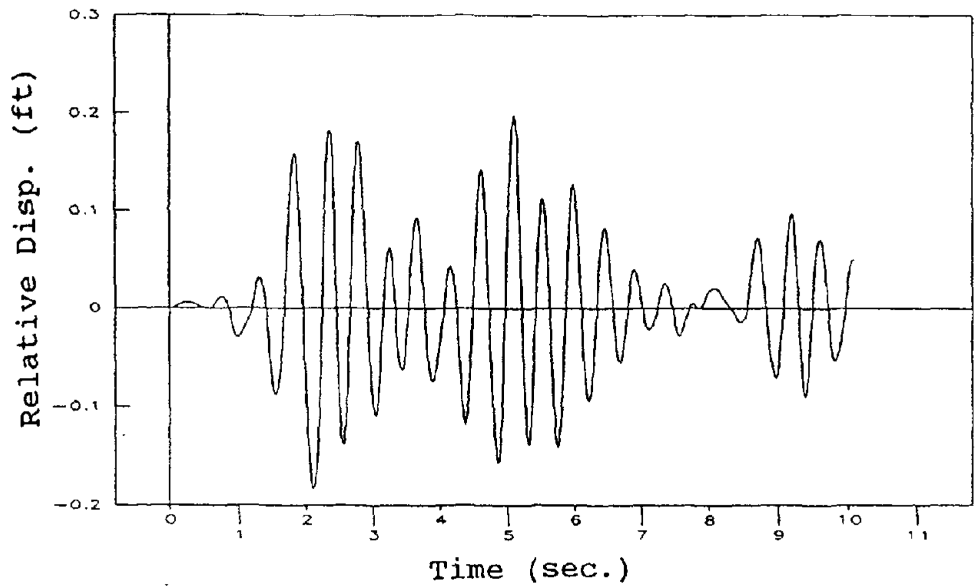


(a) Relative Displacement at Hinge 2

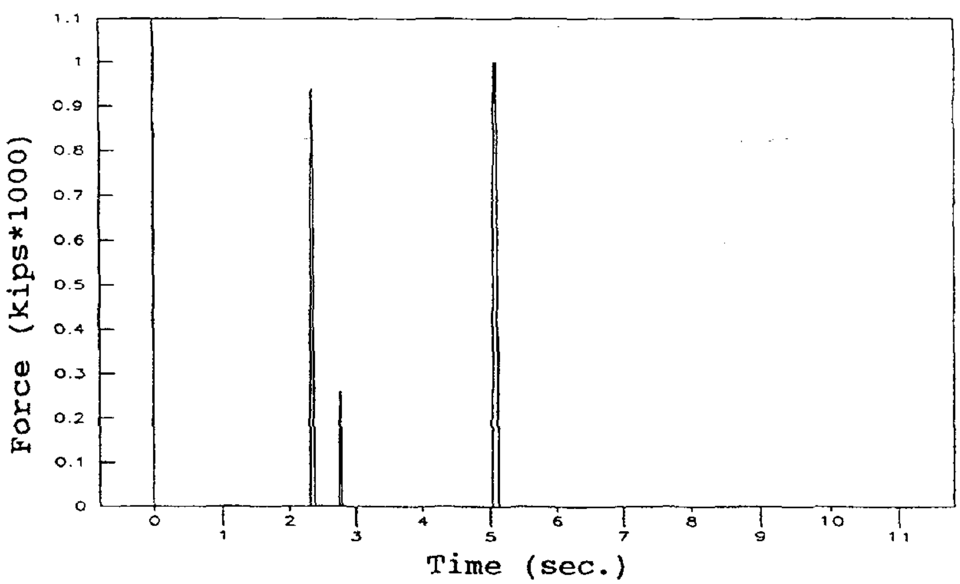


(b) Force at Hinge 2

Fig. 4.1 - Response at Hinge 2 due to the El Centro Earthquake.

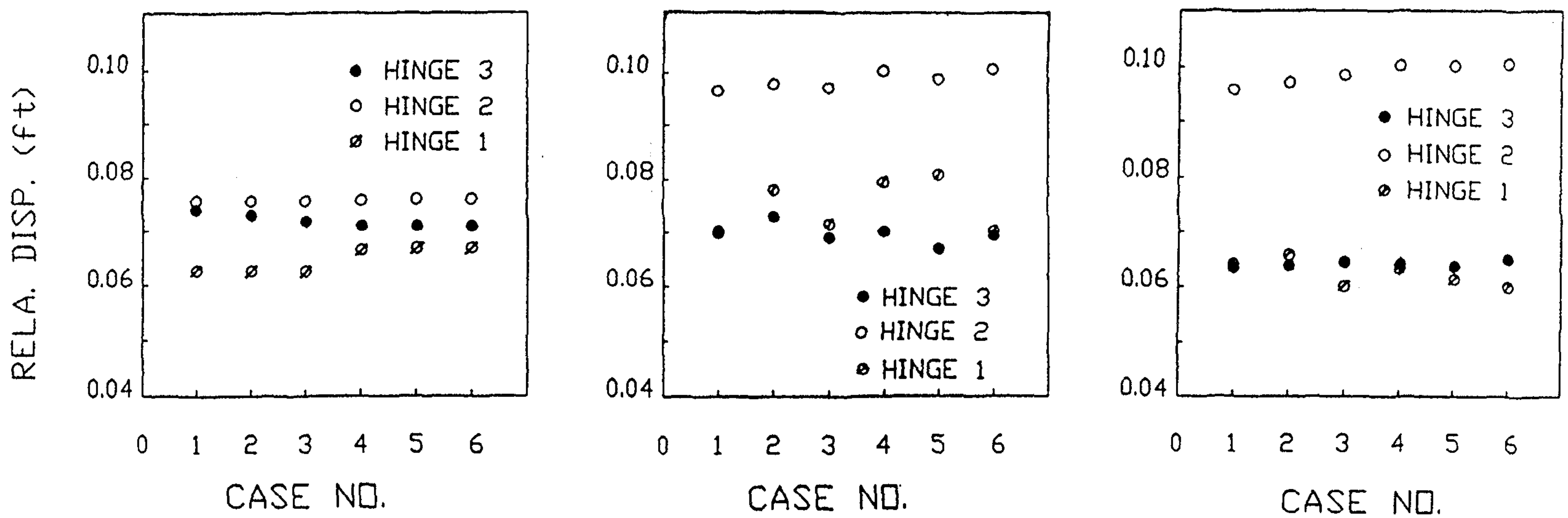


(a) Relative Displacement at Abutment 2



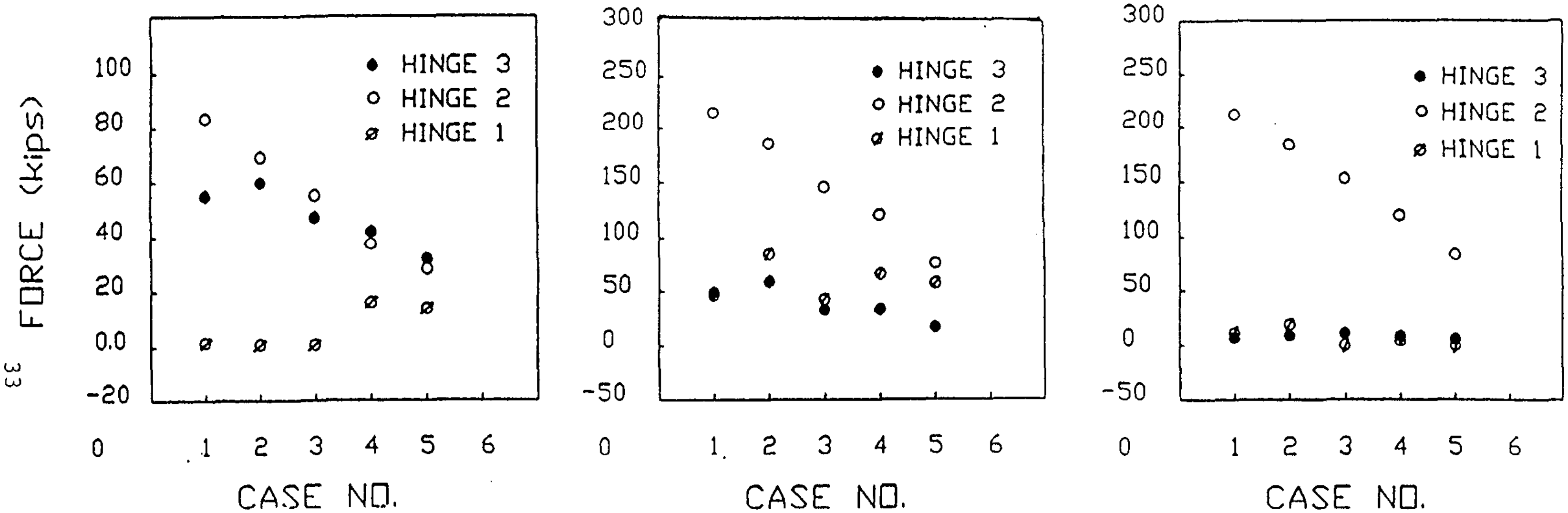
(b) Force at Abutment 2

Fig. 4.2 - Response at Abut. 2 due to the El Centro Earthquake.



EL-Centro Earthquake Eureka Earthquake Saratoga Earthquake

Fig. 4.3 - Maximum Relative Displacements at Hinges (Restrainer Gap=.75 in.).



EL-Centro Earthquake   Eureka Earthquake   Saratoga Earthquake

Fig. 4.4 - Maximum Restrainer Forces (Restrainer Gap=0.75 in.).

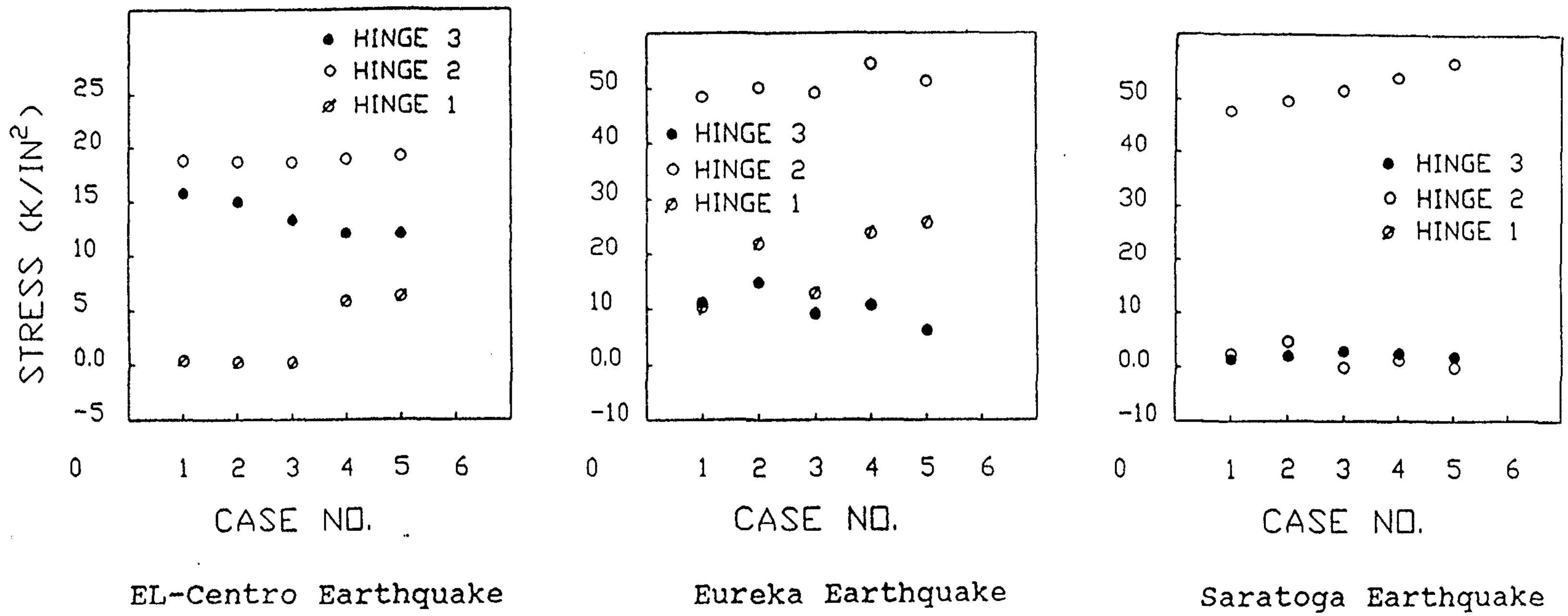


Fig. 4.5 - Maximum Restrainer Stresses (Restrainer Gap=0.75 in.).

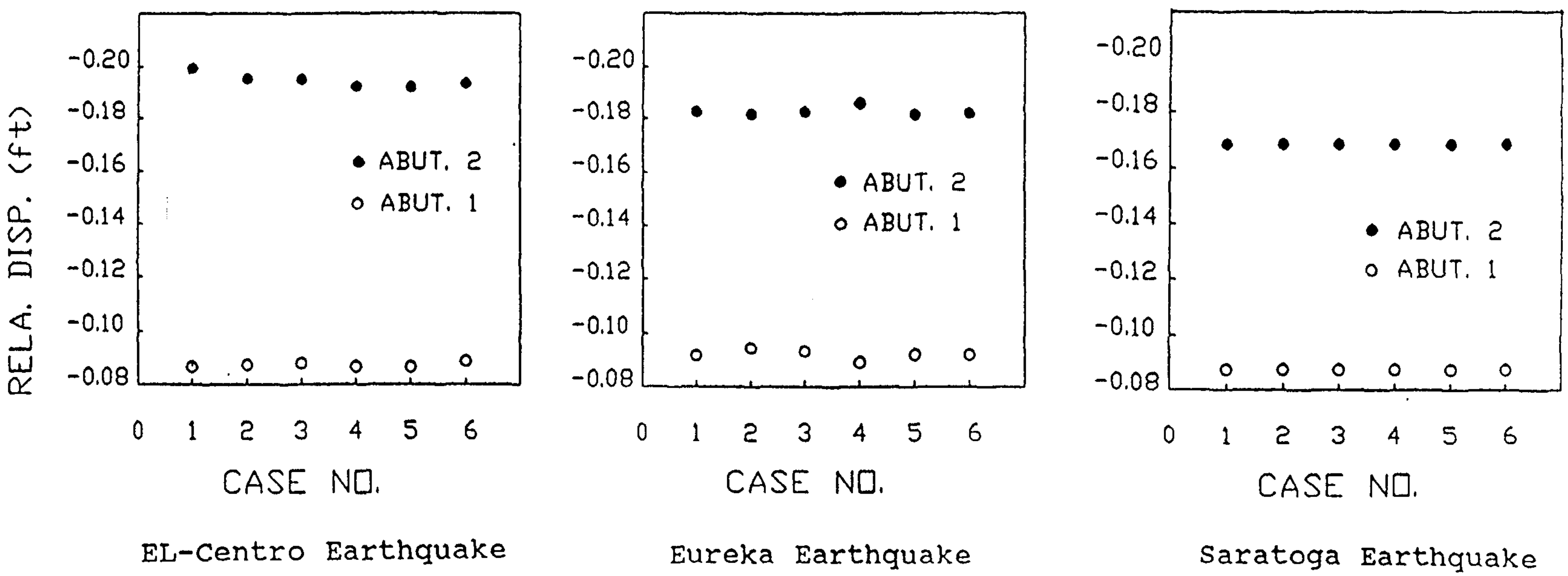


Fig. 4.6 - Maximum Relative Abutment Displacements (Restrainer Gap=0.75 in.).

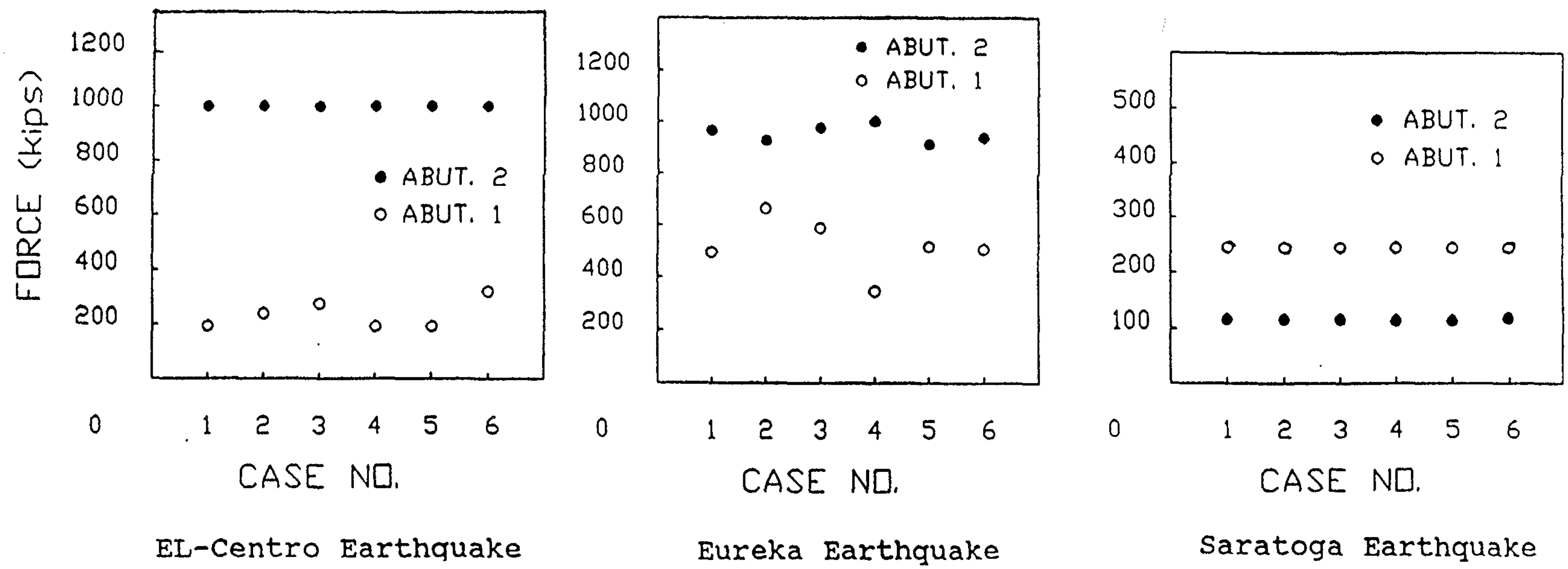
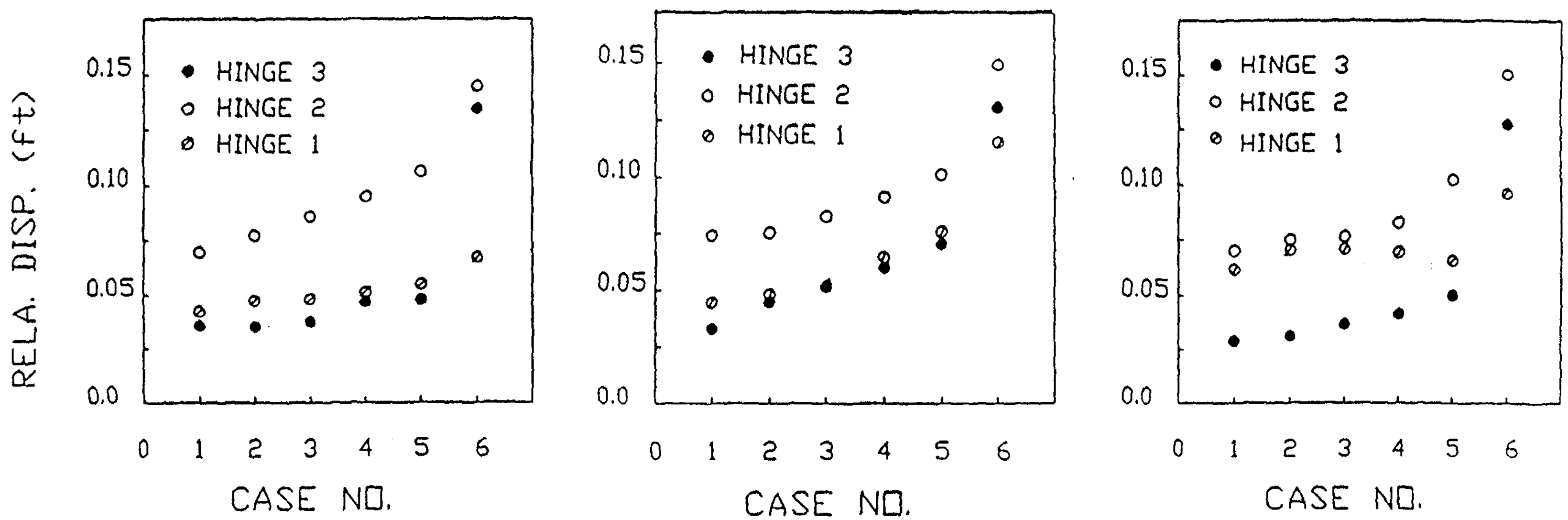
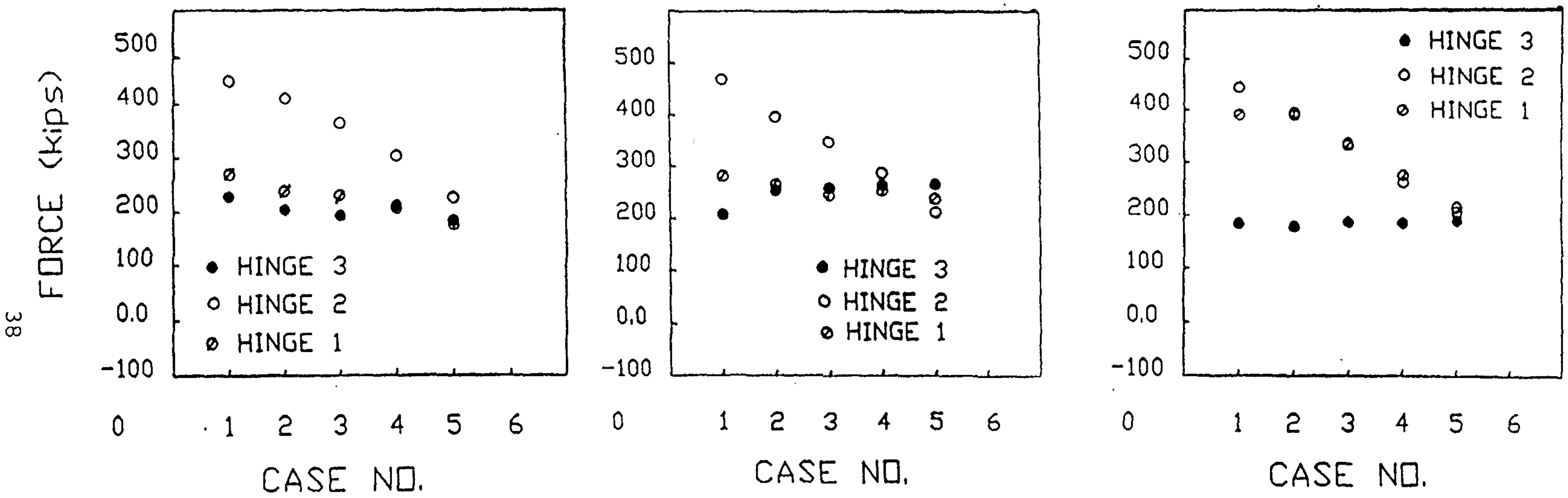


Fig. 4.7 - Maximum Abutment Forces (Restrainer Gap=0.75 in.).



EL-Centro Earthquake　　Eureka Earthquake　　Saratoga Earthquake

Fig. 4.8 - Maximum Relative Displacements at Hinges (Restrainer Gap=0 in.).



EL-Centro Earthquake   Eureka Earthquake   Saratoga Earthquake

Fig. 4.9 - Maximum Restrainer Forces (Restrainer Gap=0 in.).

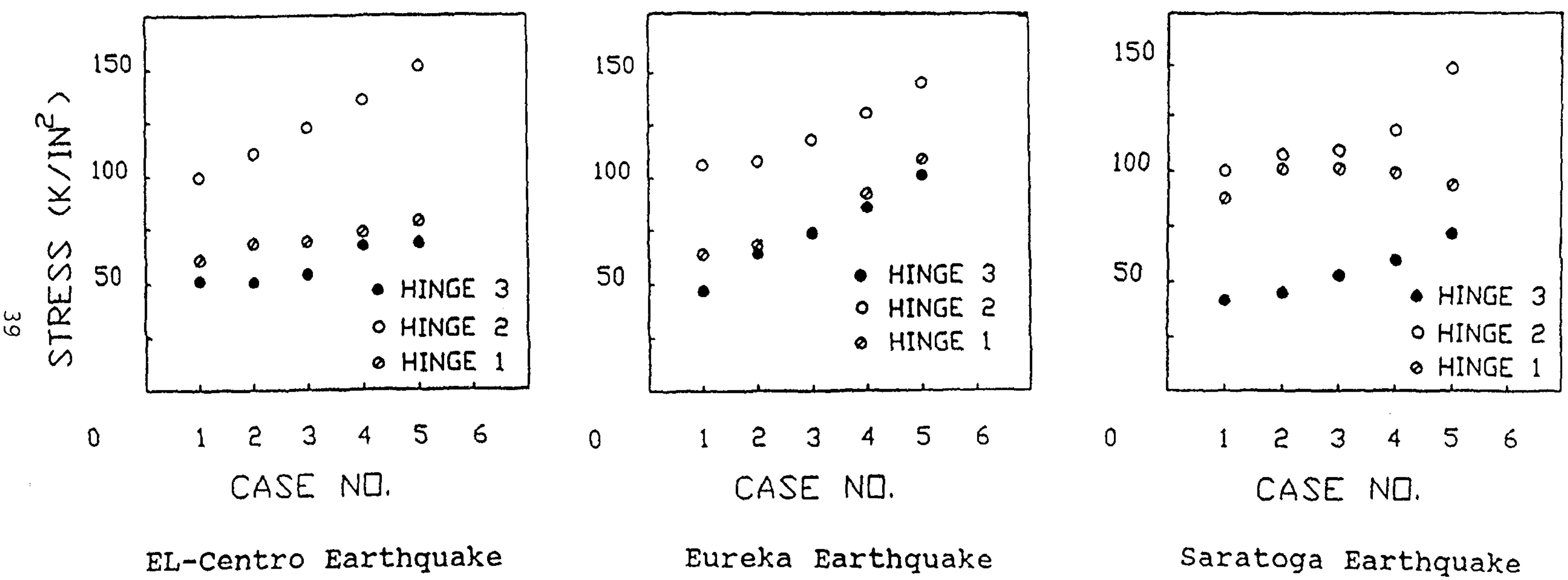


Fig. 4.10 - Maximum Restrainer Stresses (Restrainer Gap=0 in.).

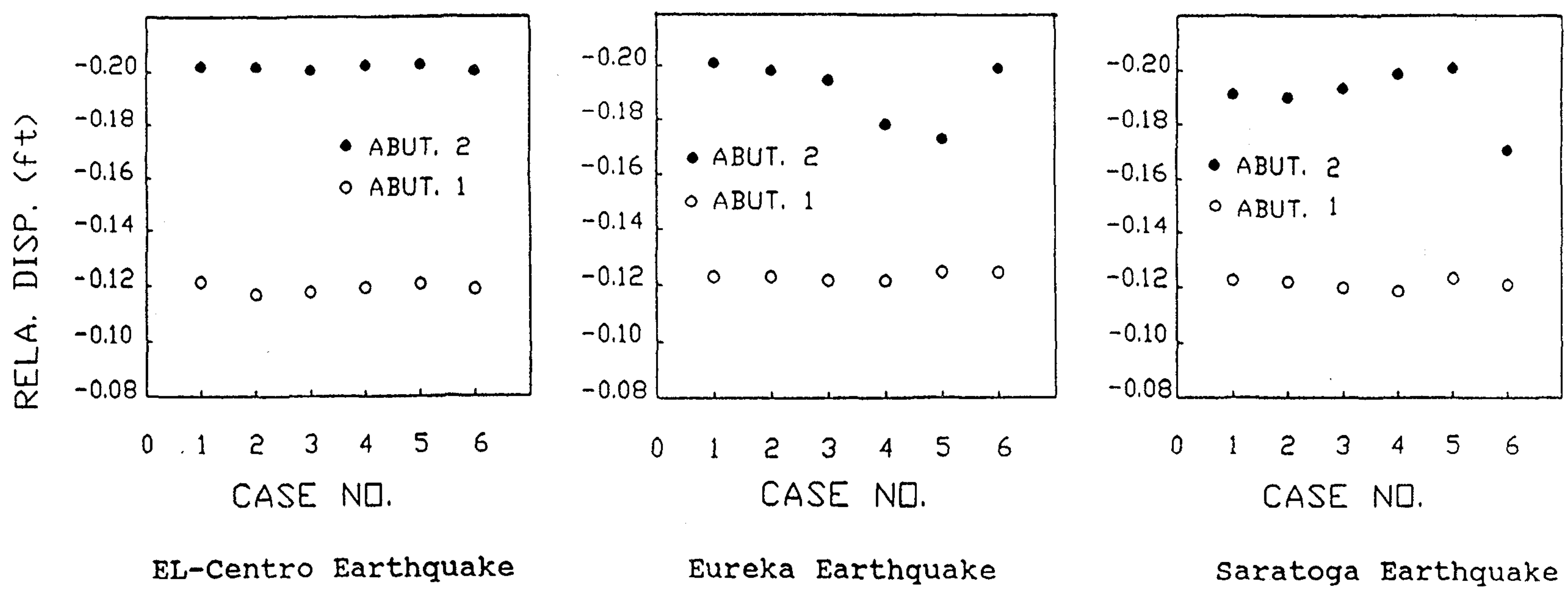
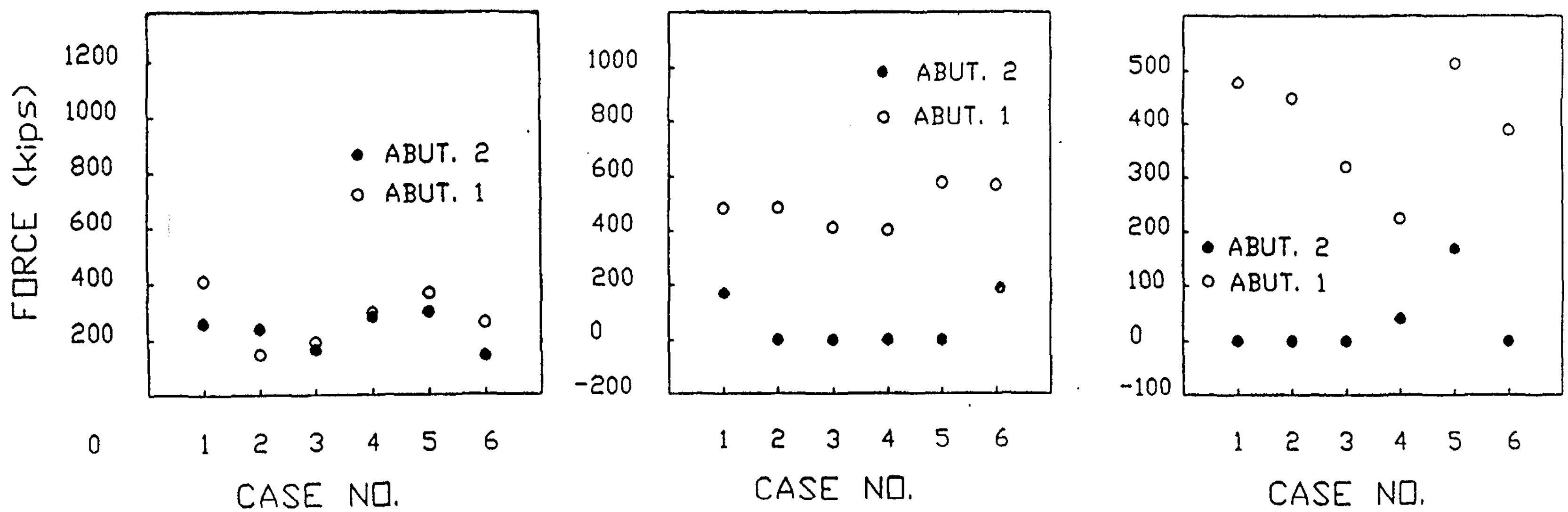


Fig. 4.11 - Maximum Relative Abutment Displacements (Restrainer Gap=0 in.).



EL-Centro Earthquake Eureka Earthquake Saratoga Earthquake

Fig. 4.12 - Maximum Abutment Forces (Restrainer Gap=0 in.).

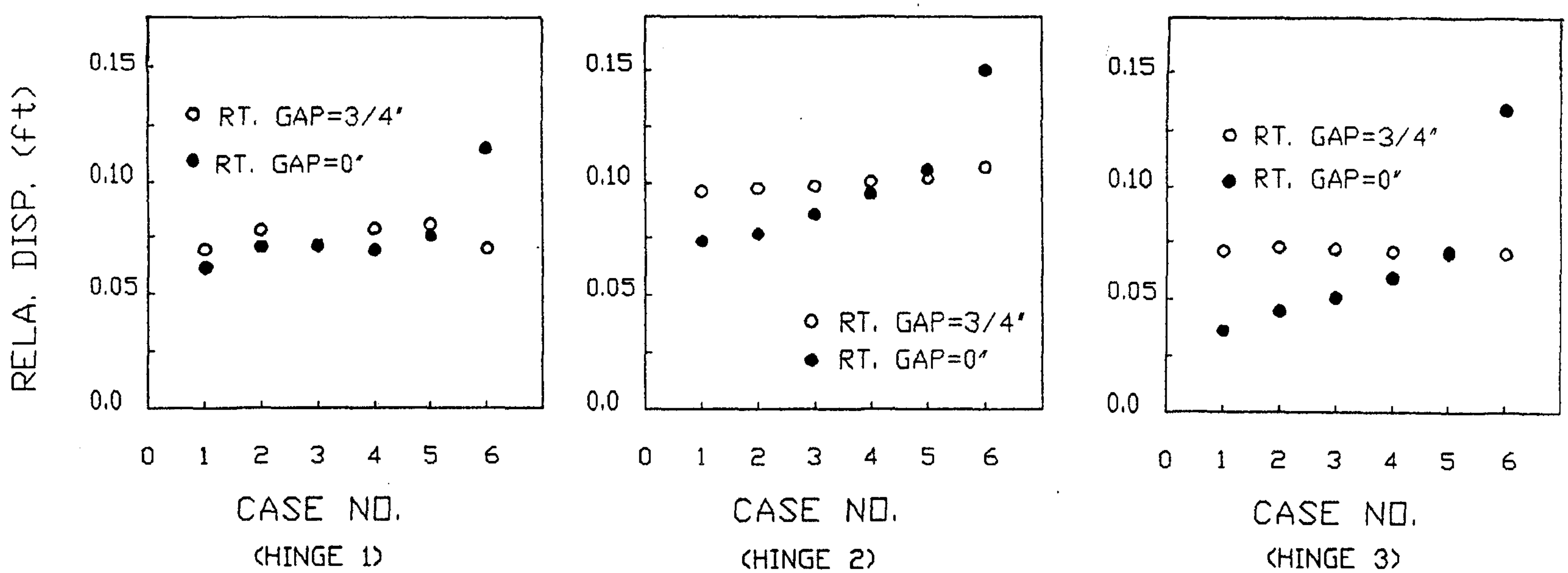


Fig. 4.13 - Maximum Relative Displacements at Hinges.

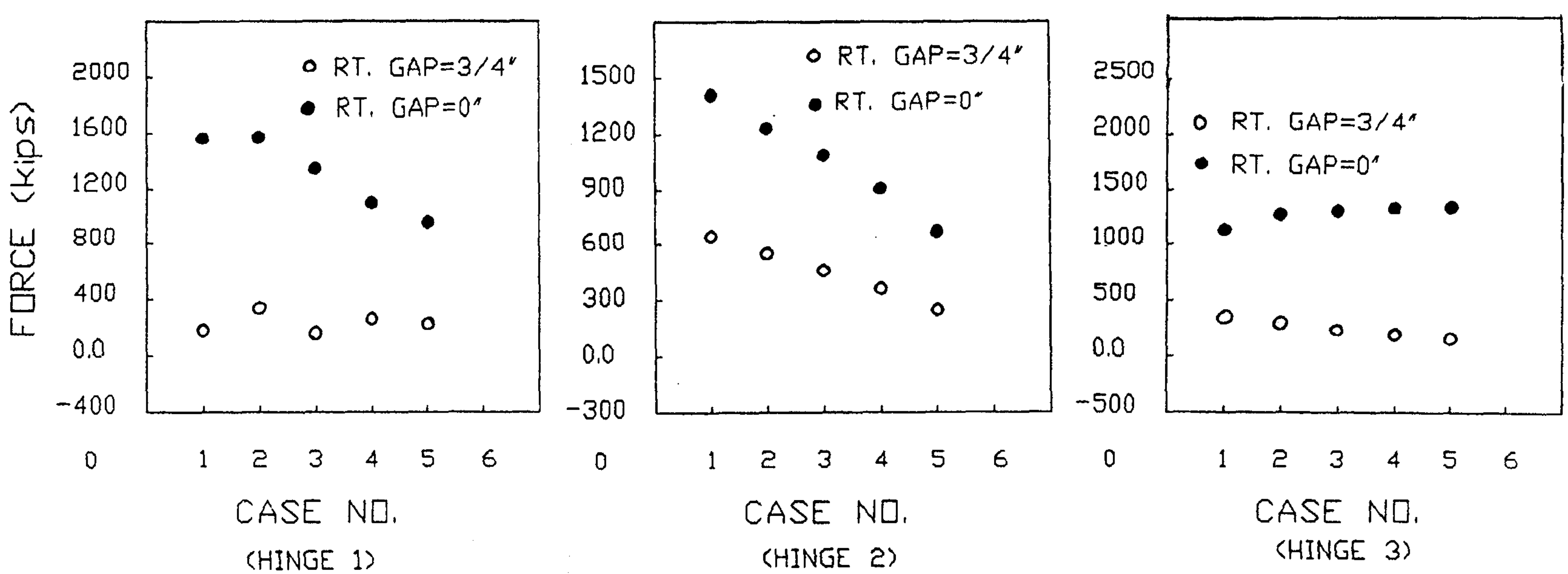


Fig. 4.14 - Maximum Restrainer Forces.

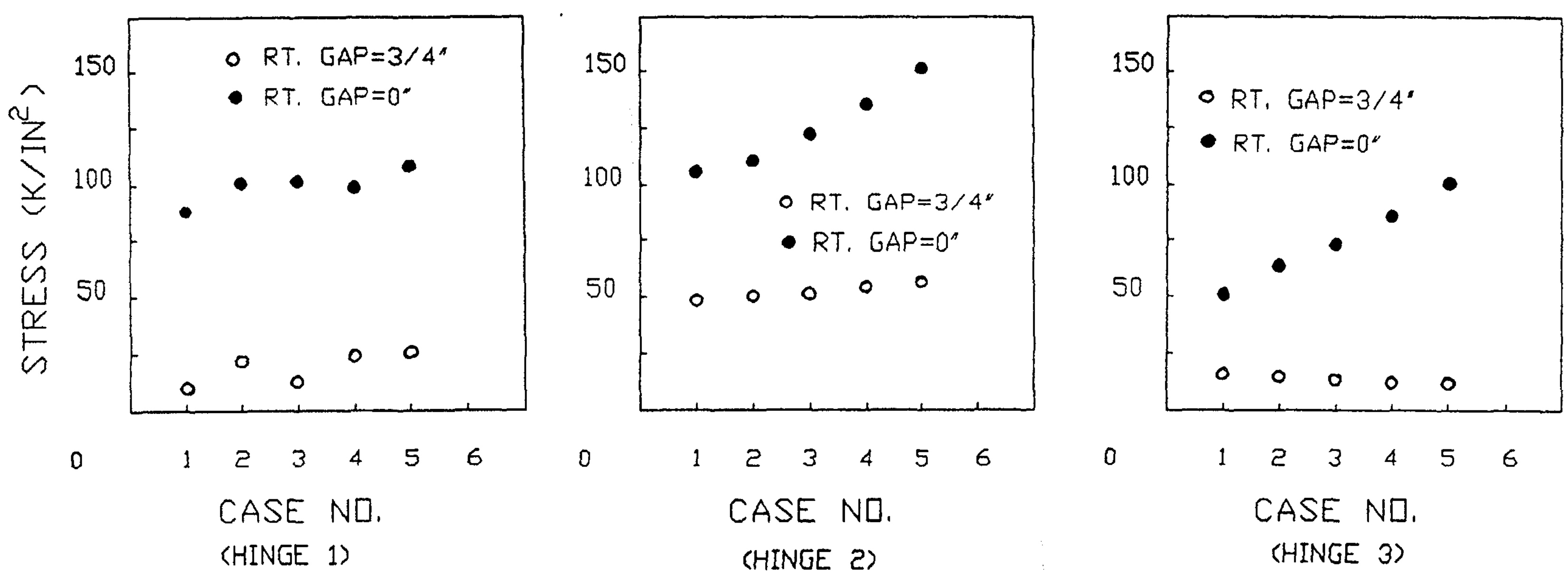


Fig. 4.15 - Maximum Restrainer Stresses.

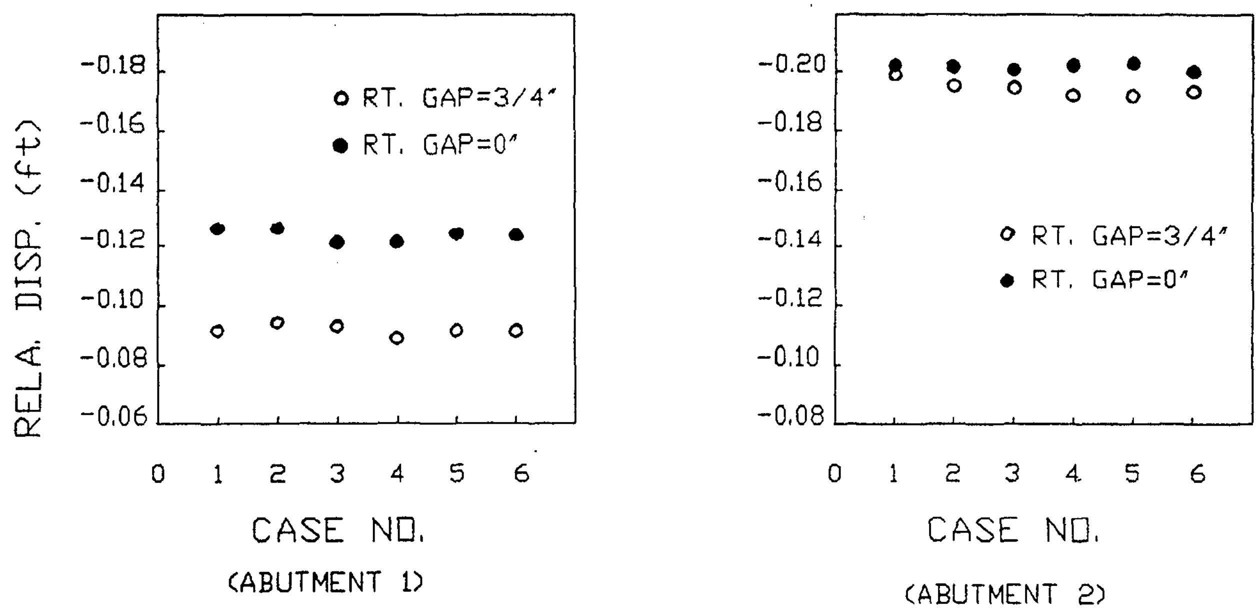


Fig. 4.16 - Maximum Relative Abutment Displacements.

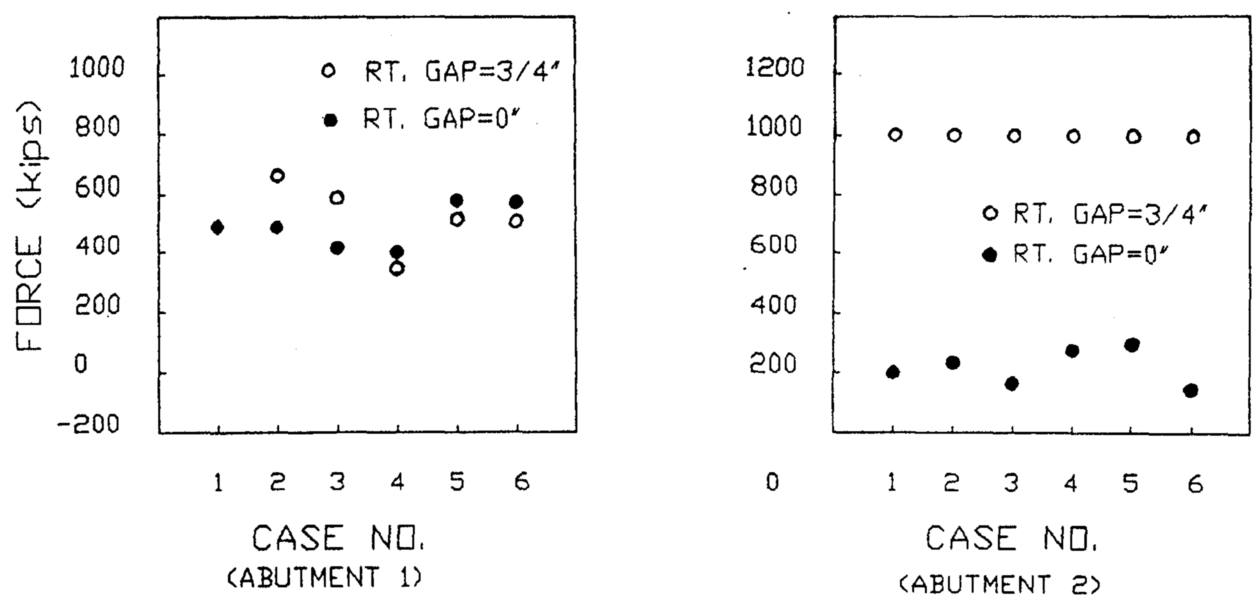
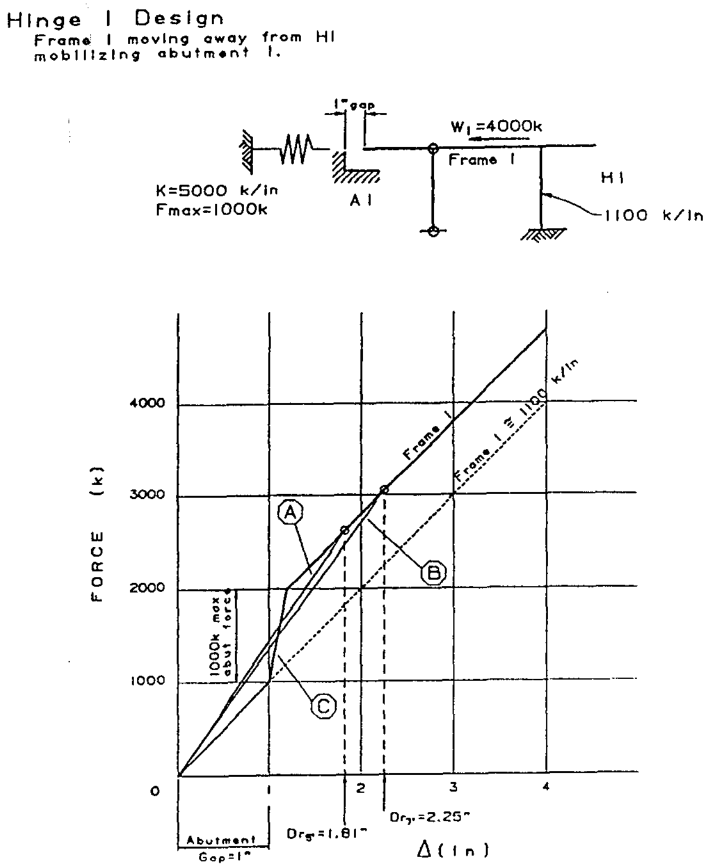


Fig. 4.17 - Maximum Abutment Forces.



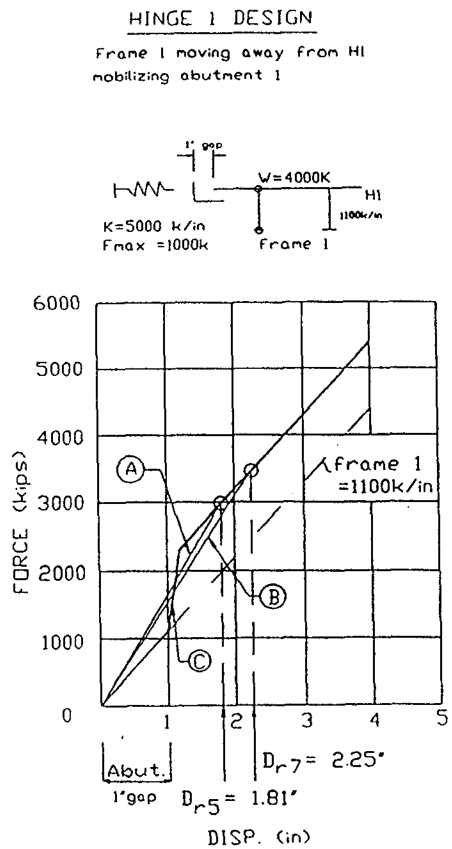
Equivalent Stiffness:

Ⓐ With 5' cables = $\frac{2700}{1.81} \cong$ 1500 k/in

Ⓑ With 7' cables = $\frac{3100}{2.25} \cong$ 1400 k/in

Ⓒ Abutment Stiffness = 5000 k/in

(a) Original Design



Equivalent Stiffness:

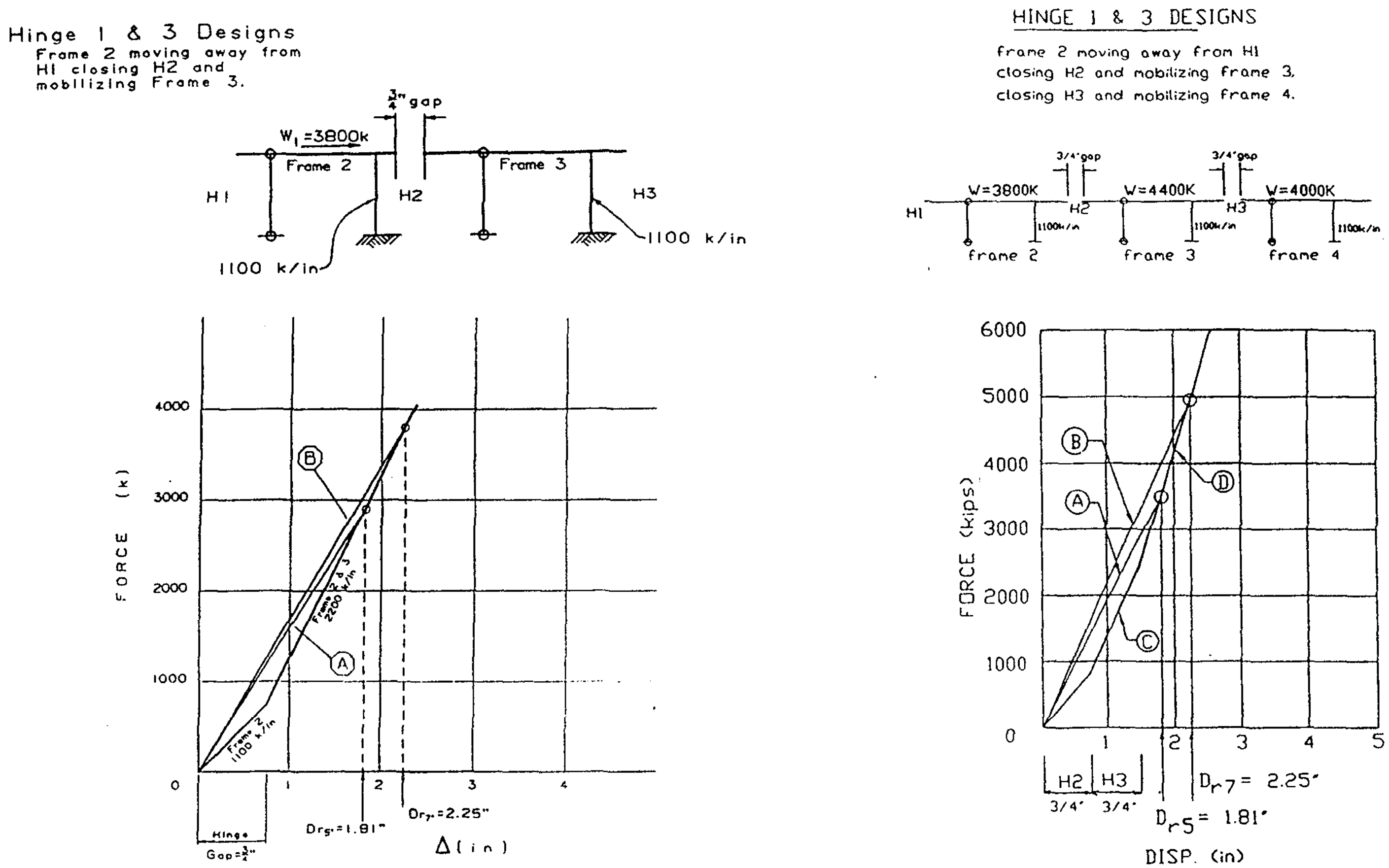
Ⓐ With 5' cables = $\frac{2991}{1.81}$ = 1653 k/in

Ⓑ With 7' cables = $\frac{3475}{2.25}$ = 1544 k/in

Ⓒ Stiffness = 6100 k/in

(b) Refined Design

Fig. 5.1 - Stiffness for Movement of the Bridge on the Left Side of Hinge 1.



Equivalent Stiffness:

(A) With 5' cables = $\frac{2900}{1.81} \cong 1600$ k/in

(B) With 7' cables = $\frac{3800}{2.25} \cong 1700$ k/in

(a) Original Design

Equivalent Stiffness:

(A) With 5' cables = $\frac{3498}{1.81} = 1933$ k/in

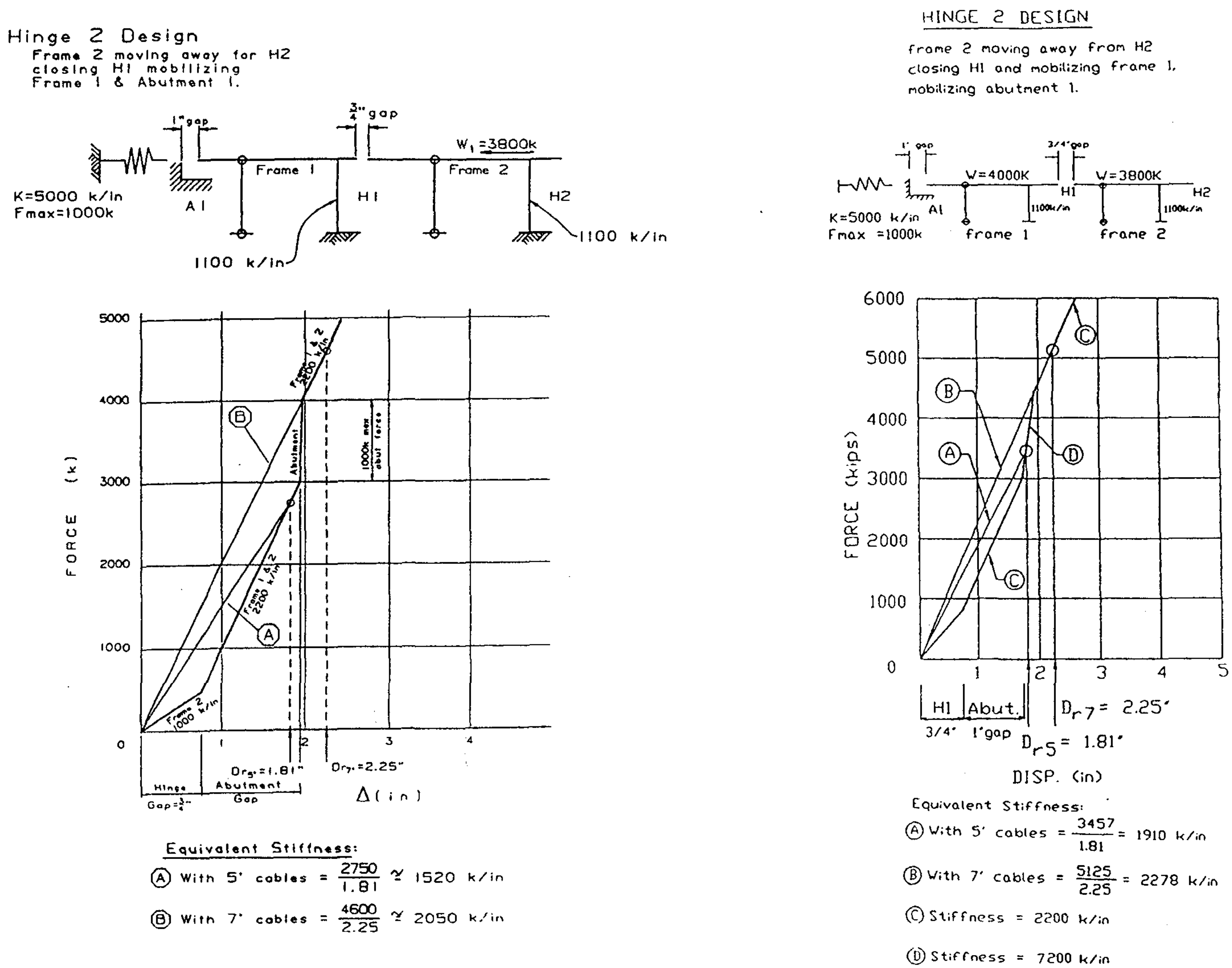
(B) With 7' cables = $\frac{4950}{2.25} = 2200$ k/in

(C) Stiffness = 2200 k/in

(D) Stiffness = 3300 k/in

(b) Refined Design

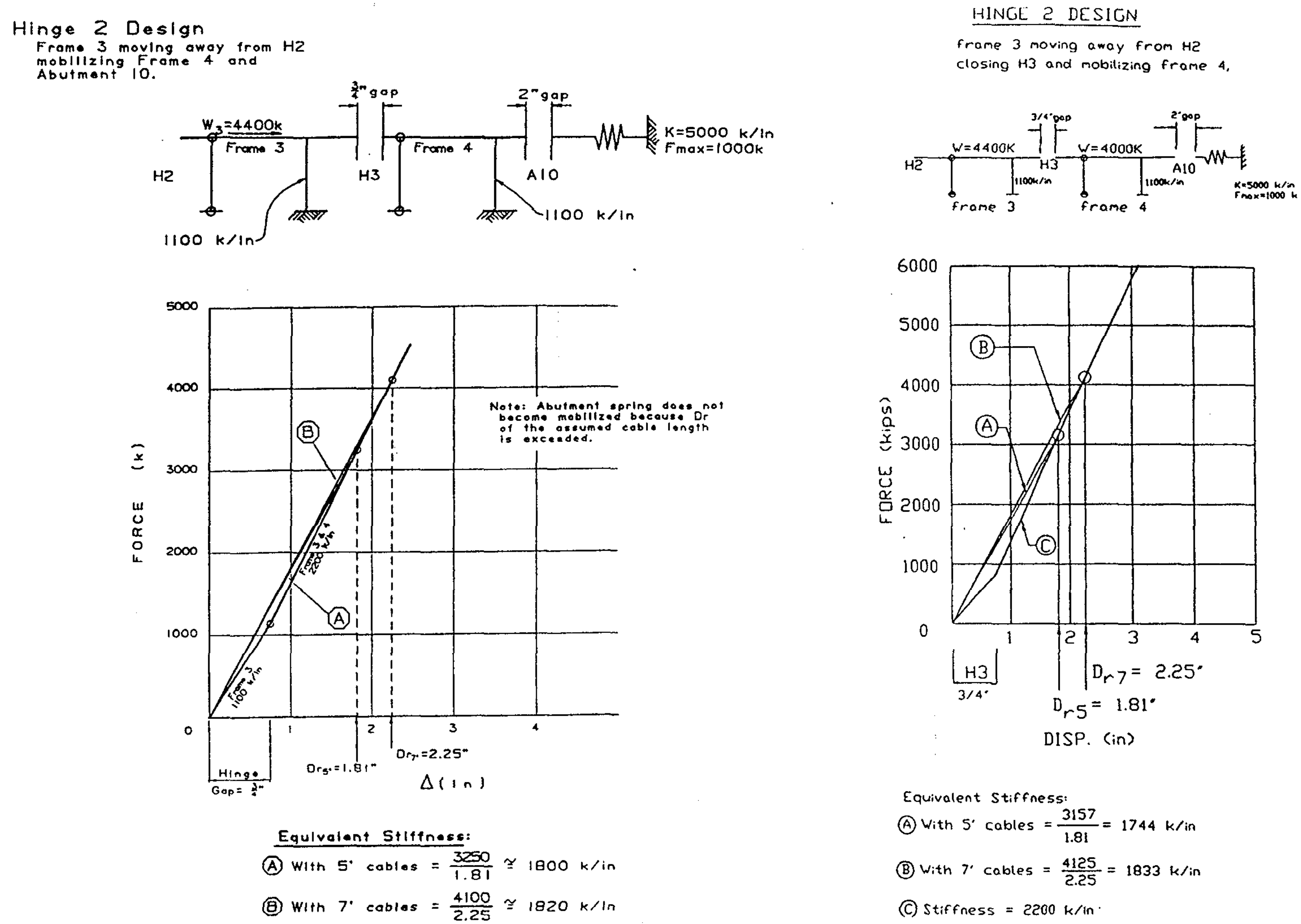
Fig. 5.2 - Stiffness for Movement of the Bridge on the Right Side of Hinge 1.



(a) Original Design

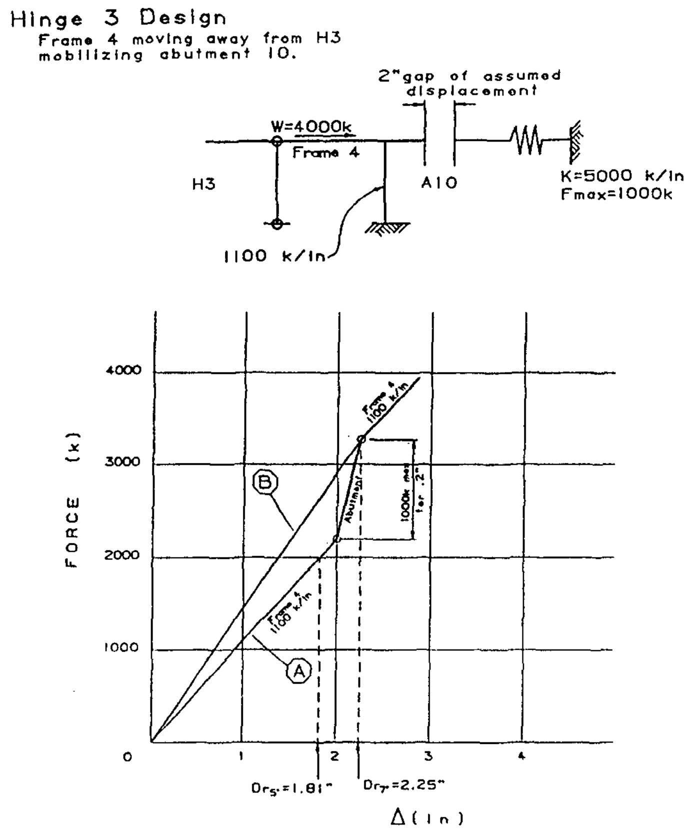
(b) Refined Design

Fig. 5.3 - Stiffness for Movement of the Bridge on the Left Side of Hinge 2.



(a) Original Design **(b) Refined Design**

Fig. 5.4 - Stiffness for Movement of the Bridge on the Right Side of Hinge 2.

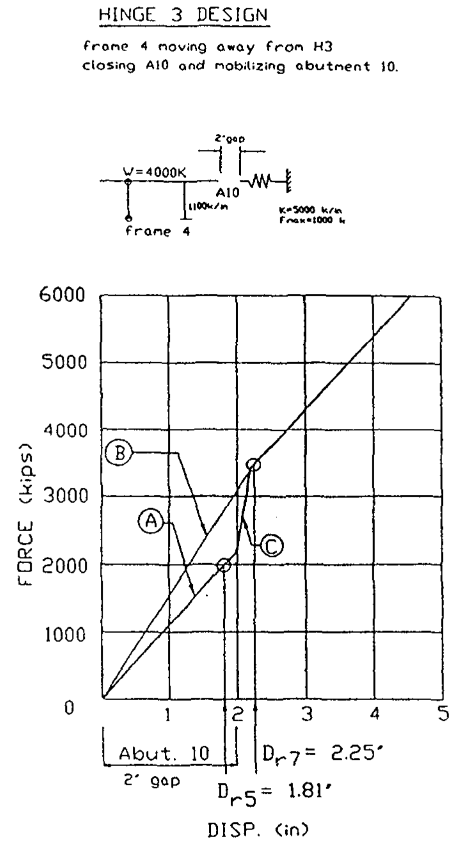


Equivalent Stiffness:

(A) With 5' cables = $\frac{2000}{1.81} \cong 1100$ k/in

(B) With 7' cables = $\frac{3200}{2.25} \cong 1420$ k/in

(a) Original Design



Equivalent Stiffness:

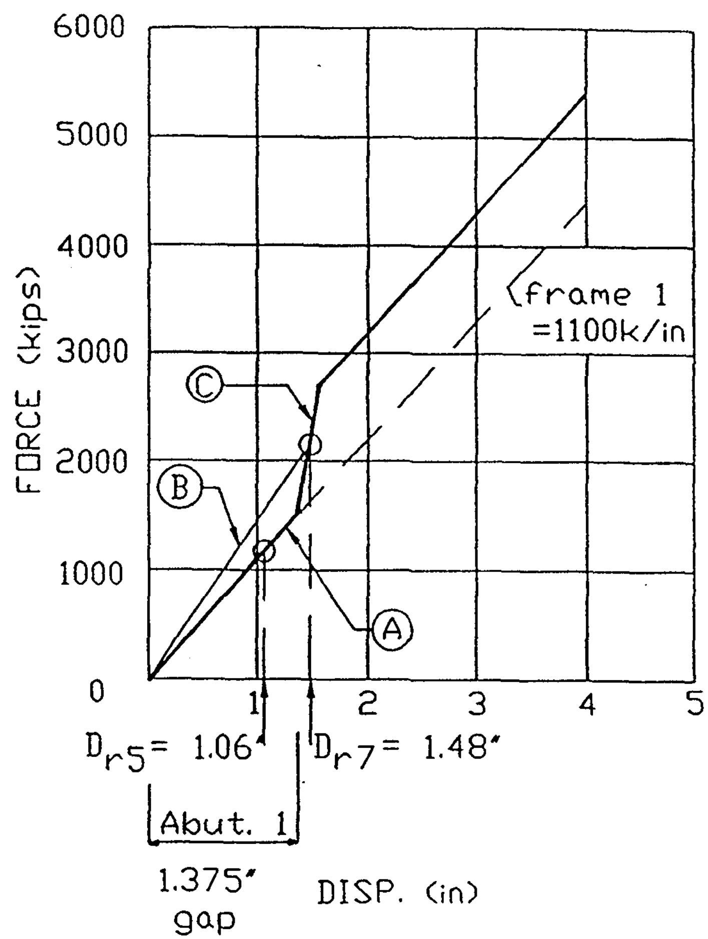
(A) With 5' cables = $\frac{1991}{1.81} = 1100$ k/in

(B) With 7' cables = $\frac{3475}{2.25} = 1544$ k/in

(C) Stiffness = 6100 k/in

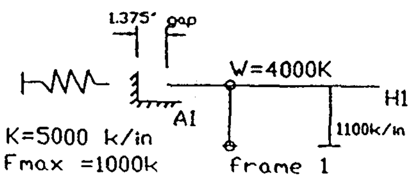
(b) Refined Design

Fig. 5.5 - Stiffness for Movement of the Bridge on the Right Side of Hinge 3.



HINGE 1 DESIGN

Frame 1 moving away from H1
mobilizing abutment 1



Equivalent Stiffness:

(A) With 5' cables $= \frac{1166}{1.06} = 1100$ k/in

(B) With 7' cables $= \frac{2154}{1.48} = 1455$ k/in

(C) Stiffness = 6100 k/in

Fig. 5.6 - Stiffness for Movement of the Bridge on the Left Side of Hinge 1 When Restrainer Gap is Zero.

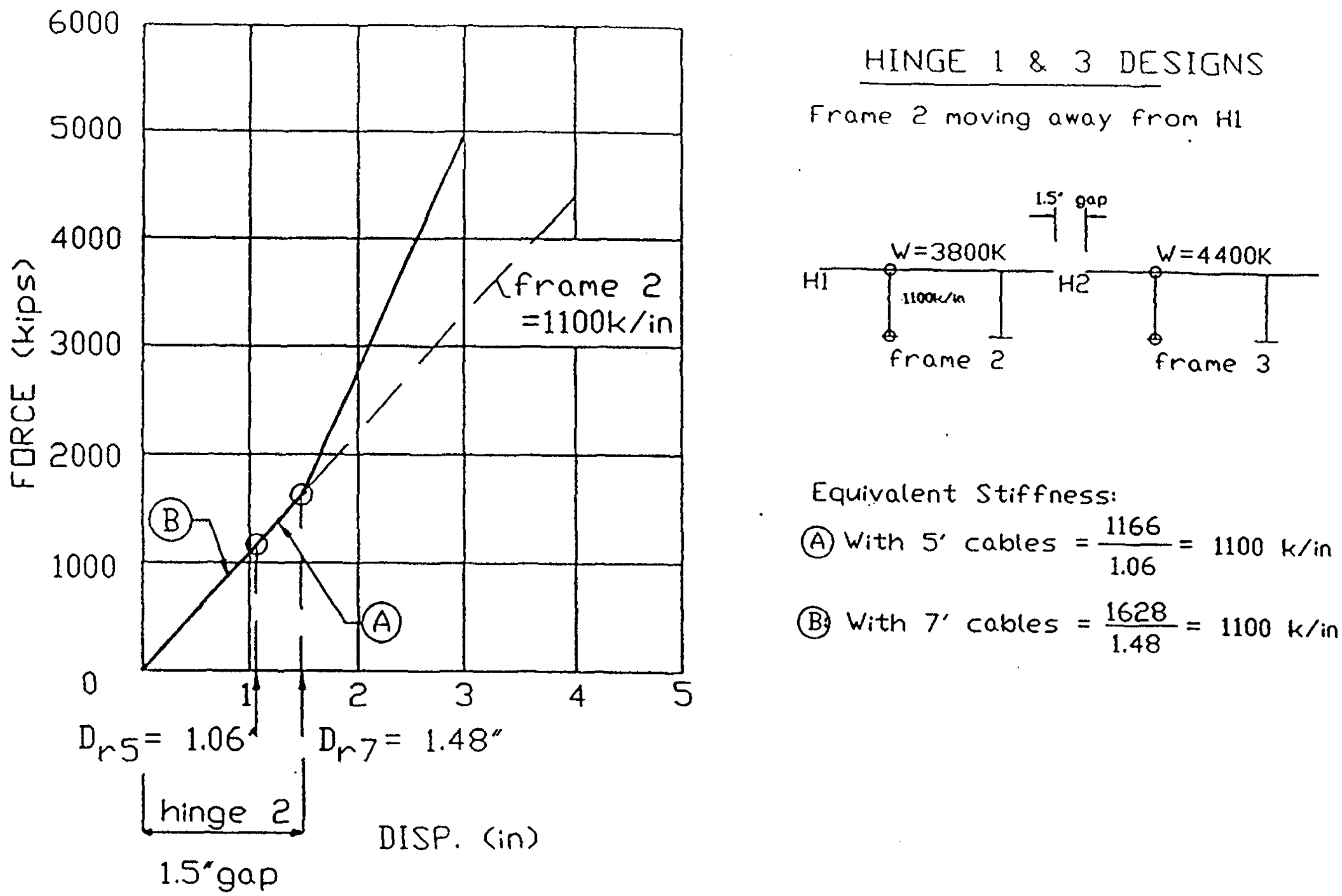
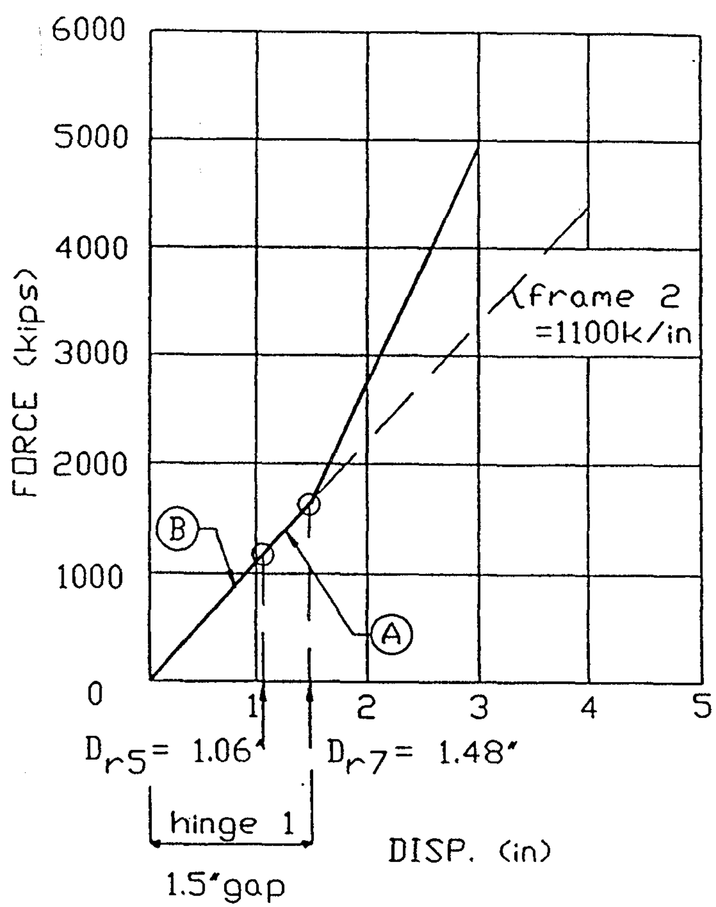
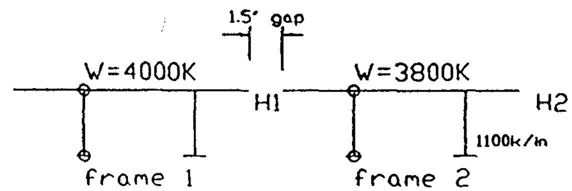


Fig. 5.7 - **Stiffness for Movement of the Bridge on the Right Side of Hinge 1 when Restrainer Gap is Zero.**



HINGE 2 DESIGNS

Frame 2 moving away from H2



Equivalent Stiffness:

(A) With 5' cables $= \dfrac{1166}{1.06} = 1100$ k/in

(B) With 7' cables $= \dfrac{1628}{1.48} = 1100$ k/in

Fig. 5.8 - Stiffness for Movement of the Bridge on the Left Side of Hinge 2 when Restrainer Gap is Zero.

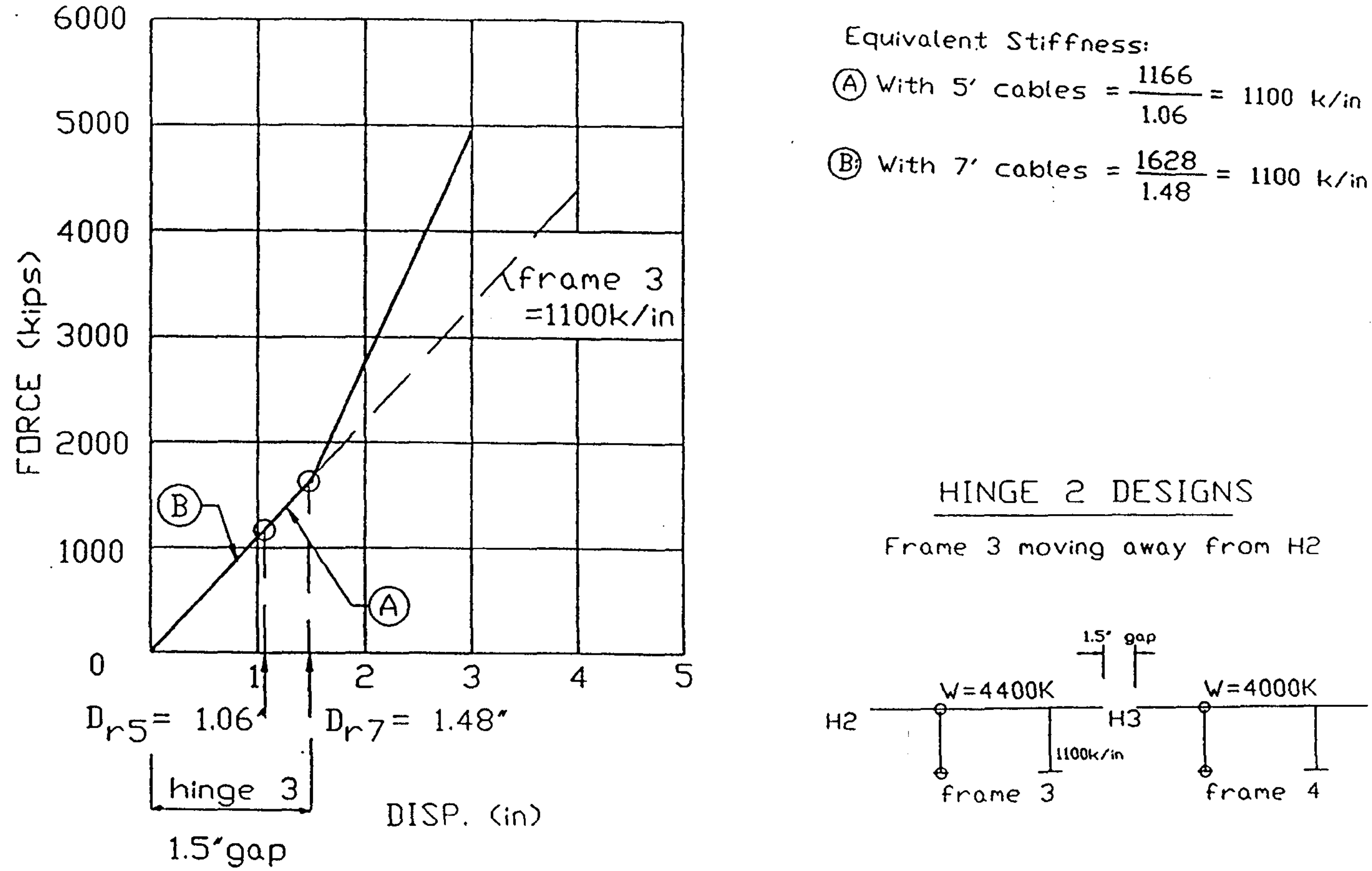


Fig. 5.9 - **Stiffness for Movement of the Bridge on the Right Side of Hinge 2 when Restrainer Gap is Zero.**

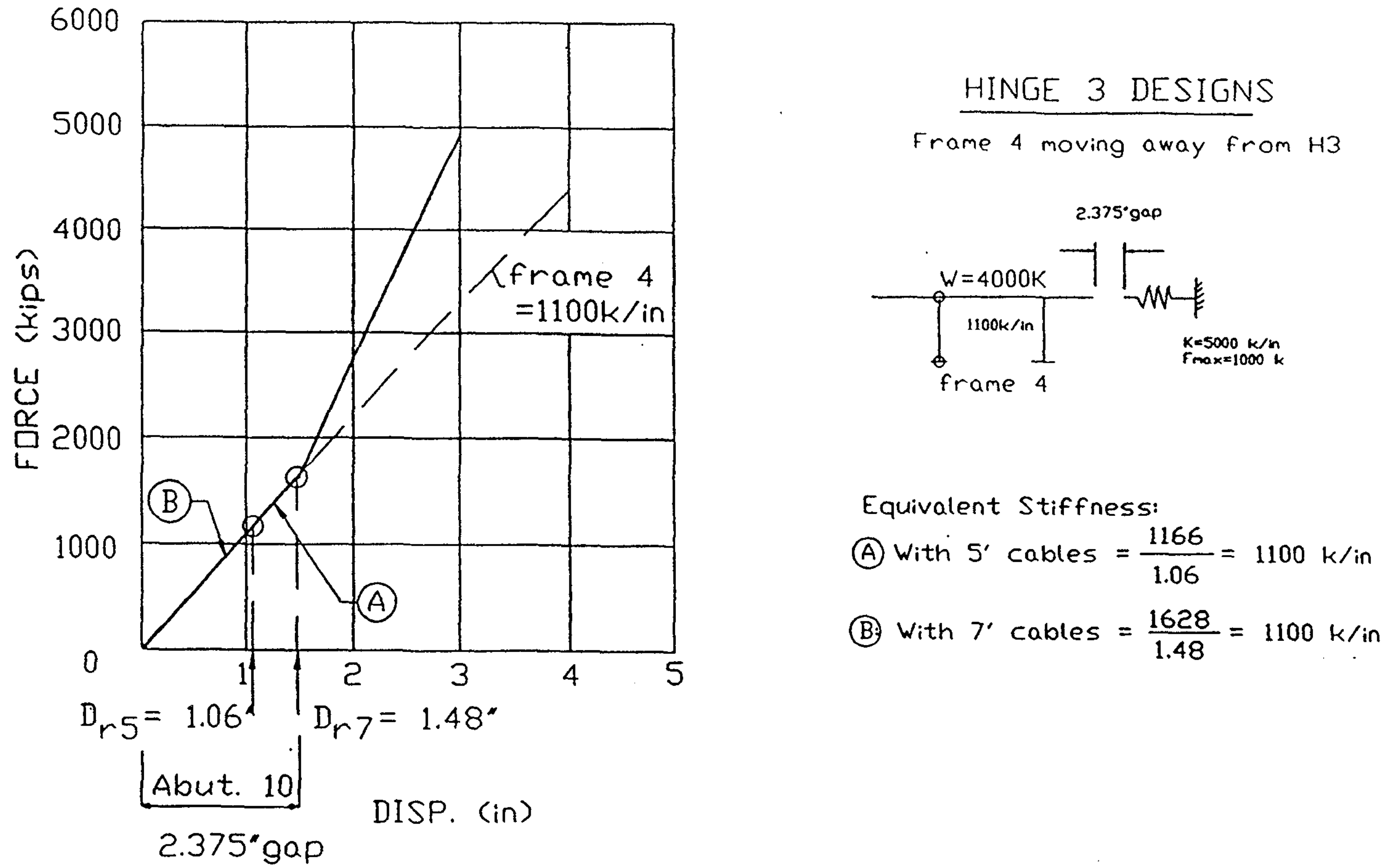


Fig. 5.10 - Stiffness for Movement of the Bridge on the Right Side of Hinge 3 when Restrainer Gap is Zero.

APPENDIX A - SUMMARY OF RESTRAINER DESIGN METHOD [1]

1. Maximum permissible restrainer deflection, Dr.

   $D_r = D_y + D_g$ D_r 5' = 1.81" D_r 7' = 2.25"

2. K_u = the unrestrained total system stiffness

   Where K_u = the equivalent stiffness of the total system considering the stiffness of all sub-structures mobilized and any gaps in the system.

3. $T = 0.32 \times \sqrt{W/K_u}$

4. Compute the longitudinal deflection $D_{eq} = ARS(W)/K_u$ (in)

   Where D_1 = the longitudinal earthquake deflection of the unrestrained system;

   ARS = the acceleration in g. for given period of vibration, T (sec). Where T = 0.32 X the square root of (W/K_u) (Ref. Bridge Design Specifications, Figures 3.21.4.3 A-D and Section 3.21.6.1);

   W = Weight of the segment (k);

   K_u = the unrestrained system stiffness (k/in)

5. Determine the number of restrainers required.

   $N_r = K_u(D_{eq} - D_r)/(F_y\ A_r)$

APPENDIX B - SAMPLE NEABS-86 INPUT FOR THE REFERENCE CASE

48	4	37	1	1	2016	1			
1	0	1	0	1	1	1	0.0	24.0	0.0
2							0.0	24.0	0.0
3							15.0	24.0	0.0
4							115.0	24.0	0.0
5							130.0	24.0	0.0
6							130.0	24.0	0.0
7							142.0	24.0	0.0
8							242.0	24.0	0.0
9							254.0	24.0	0.0
10							254.0	24.0	0.0
11							276.0	24.0	0.0
12							376.0	24.0	0.0
13							398.0	24.0	0.0
14							398.0	24.0	0.0
15							408.0	24.0	0.0
16							508.0	24.0	0.0
17							528.0	24.0	0.0
18	0	1	0	1	1	1	528.0	24.0	0.0
19							15.0	0.1	0.0
20	1	1	1	1	1	1	15.0	0.0	0.
21							115.0	0.1	0.0
22	1	1	1	1	1	1	115.0	0.0	0.0
23							142.0	0.1	0.0
24	1	1	1	1	1	1	142.0	0.0	0.0
25							242.0	0.1	0.0
26	1	1	1	1	1	1	242.0	0.0	0.0
27							276.0	0.1	0.0
28	1	1	1	1	1	1	276.0	0.0	0.0
29							376.0	0.1	0.0
30	1	1	1	1	1	1	376.0	0.0	0.0
31							408.0	0.1	0.0
32	1	1	1	1	1	1	408.0	0.0	0.0
33							508.0	0.1	0.0
34	1	1	1	1	1	1	508.0	0.0	0.0
35	1	1	1	1	1	1	0.0	24.0	100.0
36	1	1	1	1	1	1	15.0	24.0	100.0
37	1	1	1	1	1	1	115.0	24.0	100.0
38	1	1	1	1	1	1	142.0	24.0	100.0
39	1	1	1	1	1	1	242.0	24.0	100.0
40	1	1	1	1	1	1	276.0	24.0	100.0
41	1	1	1	1	1	1	376.0	24.0	100.0
42	1	1	1	1	1	1	408.0	24.0	100.0
43	1	1	1	1	1	1	508.0	24.0	100.0
44	1	1	1	1	1	1	528.0	24.0	100.0
45	1	1	1	1	1	1	528.0	24.0	−100.0
46	1	1	1	1	1	1	130.0	24.0	100.0
47	1	1	1	1	1	1	254.0	24.0	100.0
48	1	1	1	1	1	1	398.0	24.0	100.0

```
    2    20      2      0      1
    1  576000.0        0.18  0.004658
    1       30.7                                    44.4        26.0       78.0
    2      200.0                                  1405.3       416.7    26667.0
      0.0
    -32.0
      0.0
    1    19      3     36      1      1
    2    21      4     37      1      1
    3    23      7     38      1      1
    4    25      8     39      1      1
    5    27     11     40      1      1
    6    29     12     41      1      1
    7    31     15     42      1      1
    8    33     16     43      1      1
    9     2      3     36      1      2
   10     3      4     37      1      2
   11     4      5     38      1      2
   12     6      7     38      1      2
   13     7      8     39      1      2
   14     8      9     40      1      2
   15    10     11     40      1      2
   16    11     12     41      1      2
   17    12     13     42      1      2
   18    14     15     42      1      2
   19    15     16     43      1      2
   20    16     17     44      1      2
    4     2      1      1
    1    60000.0        1.0         1.0         1.0         1.0         1.0
    1     1000.0     1000.0      1000.0      1000.0      1000.0      1000.0
      10.        10.         10.         10.         10.         10.
    1     1      4     35      1      0
    2    18     16     45      1      0
    5     7      1      7
    1     1458.0        1.0     14580.0         1.0         1.0         1.0
    1        0.0        1.0      0.0625      0.0625      6337.0       792.0
909000.     4
  -20.0    -10.0       10.0        20.0
    2        0.0        1.0      0.0625      0.0625      6337.0       792.0
865000.     3
  -20.0      0.0       20.0
    3        0.0        1.0      0.0625      0.0625      6337.0       792.0
843000.     5
  -20.0    -10.0        0.0        10.0        20.0
    4        0.0        1.0         0.0      0.0833     60000.0      1000.0
   0.       1
   0.0
    5        0.0        1.0         0.0      0.1666     60000.0      1000.0
   0.       1
   0.0
```

```
    6        0.0        1.0     0.0833     0.0000         1.0     1000.0
886000.     0
    0.0
    7        0.0        1.0     0.1666     0.0000         1.0     1000.0
886000.     0
    0.0
    1    2     1    35    15    011100     1          4
      40.0          0.0
    2    5     6    46    15    000000                1
      40.0          0.0
    3    9    10    47    15    000000                2
      40.0          0.0
    4   13    14    48    15    000000                3
      40.0          0.0
    5   17    18    45    15    011100     1          5
      40.0          0.0
    6    1     2    35    15    011100     1          6
      40.0          0.0
    7   18    17    45    15    011100     1          7
      40.0          0.0
    6     8     1           1
    1      720.0     8640.0
    1      30.72   0.4   0.4
    8598.0    10934.0   32817.01     0.3071     1.9114     1
    1    20    19    36          1
    2    22    21    37          1
    3    24    23    38          1
    4    26    25    39          1
    5    28    27    40          1
    6    30    29    41          1
    7    32    31    42          1
    8    34    33    43          1


    1
    0.0050     0.6533    0.00358
    0         0.7
    0     1        0.01       0.005
    1                  1.0
```

EL CENTRO EARTHQUAKE

```
  504       5.65       0.02
 0.000   -0.108   -0.101   -0.088   -0.095   -0.102   -0.142   -0.128
-0.110   -0.085   -0.085   -0.131   -0.176   -0.194   -0.162   -0.144
-0.108   -0.082   -0.042   -0.066   -0.131   -0.190   -0.196   -0.066
 0.030    0.141   -0.049   -0.128   -0.144   -0.203   -0.260   -0.325
-0.306   -0.172   -0.197   -0.163   -0.164   -0.067    0.025    0.150
 0.236    0.252    0.336    0.463    0.492    0.419    0.359    0.271
 0.235    0.339    0.412    0.530    0.639    0.732    0.652    0.599
 0.400    0.400    0.063   -0.515   -0.787   -0.603   -0.484   -0.250
-0.059    0.134    0.308    0.499    0.710    0.995    1.219    1.529
 1.449    1.155    0.935    0.892    0.926    0.839    0.901    0.993
 1.209    0.328   -1.475   -2.066   -1.989   -2.034   -1.816   -1.752
```

-1.752	-1.753	-1.805	-1.630	-1.347	-1.087	-0.782	-0.429
-0.017	0.360	0.765	1.164	1.598	1.960	2.412	2.729
3.036	3.200	3.417	2.821	2.324	-1.198	-2.373	-1.640
-1.865	-1.095	-0.753	-0.173	0.113	0.533	0.895	1.186
1.757	0.576	-2.631	-1.547	-1.729	-1.012	-0.579	0.237
-0.670	-1.980	-1.641	-1.685	-1.481	-1.231	-1.001	-0.751
-0.523	-0.271	-0.044	0.188	-0.095	-0.433	-0.838	-0.951
-0.716	-0.599	-0.334	-0.108	0.185	0.420	0.673	-0.097
-0.372	-0.040	0.011	0.344	0.565	0.833	1.130	1.363
0.219	0.241	0.683	0.689	1.318	1.353	2.040	-0.931
-1.308	-0.692	-0.546	0.072	0.675	-1.067	-1.488	-1.071
-1.162	-0.762	-0.559	-0.215	-0.126	-0.674	-0.324	-0.337
-0.109	0.017	0.299	0.488	0.608	0.222	-0.032	-0.245
0.077	0.211	0.568	0.826	1.206	1.478	1.737	0.421
0.029	0.259	0.293	-0.055	-0.147	0.143	0.206	0.499
0.645	0.957	1.128	1.447	1.629	1.945	1.856	1.984
1.769	1.250	-1.207	-0.542	-0.384	-0.311	-1.118	-1.661
2.464	-2.205	-1.836	-1.317	-0.960	-0.325	0.154	0.816
1.319	1.818	-0.058	-0.169	0.285	0.447	0.983	1.424
1.853	2.456	1.685	-1.380	-0.999	-1.089	-0.907	-0.469
-1.250	-2.111	-1.617	-1.692	-1.306	-1.111	-0.773	-0.510
-0.544	-1.200	-1.209	-1.158	-1.145	-0.717	-0.546	0.064
-0.804	-1.634	-0.859	-0.961	-0.369	-0.147	0.319	0.648
0.876	0.472	0.198	-0.027	0.292	0.445	0.785	1.033
1.352	1.606	1.861	1.281	0.640	0.204	0.314	0.373
0.496	0.235	-0.084	-0.168	-0.113	-0.229	-0.248	-0.157
-0.069	0.147	0.379	0.579	0.255	-0.041	-0.428	-0.133
0.095	0.230	-0.129	-0.050	0.080	0.210	0.380	0.510
0.157	-0.032	-0.111	0.005	0.076	0.035	-0.095	-0.036
-0.016	0.038	0.085	-0.056	-0.304	-0.421	-0.244	-0.236
-0.177	-0.129	-0.018	0.203	-0.108	-0.091	-0.034	-0.106
-0.111	-0.099	-0.002	0.073	0.235	0.355	0.705	0.779
0.184	-0.263	-0.124	-0.042	0.159	0.048	-0.219	-0.467
-0.428	-0.216	-0.043	0.159	0.320	0.419	0.123	-0.160
-0.204	-0.082	-0.206	-0.137	-0.055	0.053	0.134	0.266
0.232	0.079	-0.008	0.200	0.435	0.492	0.191	0.092
-0.022	-0.021	0.052	0.093	0.255	0.368	0.525	0.541
0.425	0.398	0.559	0.756	0.365	0.411	0.098	-0.204
-0.249	-0.405	-0.413	-0.471	-0.433	-0.458	-0.057	0.178
-0.208	-0.492	-0.530	-0.362	-0.405	-0.308	-0.316	-0.265
-0.265	-0.269	-0.345	-0.309	-0.217	-0.078	0.087	0.281
0.031	0.358	0.341	0.358	0.287	0.305	0.112	0.214
0.136	0.384	-0.861	-1.349	-1.342	-1.354	-1.193	-1.042
-0.829	-0.651	-0.444	-0.258	-0.060	-0.091	-0.182	-0.147
0.085	0.163	0.050	0.264	0.582	0.867	1.200	1.695
1.111	-1.100	-0.366	-0.445	-0.236	-0.960	-0.656	-0.597
-0.670	-0.552	-0.027	0.378	1.072	1.669	0.947	0.408
0.667	0.132	-0.095	-0.520	-0.827	-1.152	-1.150	-0.803
-0.369	0.029	0.545	1.178	1.610	-0.270	0.034	-0.056
0.020	0.146	0.537	0.798	-0.205	-0.580	-0.169	-0.175
-0.028	0.074	0.382	0.567	0.753	0.801	0.592	0.304
0.023	0.064	-0.406	-0.451	-0.079	0.168	0.567	0.093

3

1	1	2	3			
2	1	2	3			
4	1	2	3			
6	1	2	3			
7	1	2	3			
8	1	2	3			
10	1	2	3			
12	1	2	3			
13	1	2	3			
14	1	2	3			
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19	1	2	3			
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25	1	2	3			
26	1	2	3			
41	1	2	3	4	5	6
42	1	2	3	4	5	6
43	1	2	3	4	5	6
44	1	2	3	4	5	6
45	1	2	3	4	5	6
46	1	2	3	4	5	6
47	1	2	3	4	5	6
48	1	2	3	4	5	6

3

2	1	1	2	3	4	5	6	7	8	9	10	11	12
2	2	1	2	3	4	5	6	7	8	9	10	11	12
2	3	1	2	3	4	5	6	7	8	9	10	11	12
2	4	1	2	3	4	5	6	7	8	9	10	11	12
2	5	1	2	3	4	5	6	7	8	9	10	11	12
2	6	1	2	3	4	5	6	7	8	9	10	11	12
2	7	1	2	3	4	5	6	7	8	9	10	11	12
2	8	1	2	3	4	5	6	7	8	9	10	11	12
4	1	1	2	3	4	5	6						
4	2	1	2	3	4	5	6						
5	1	1	2	3	4	5	6	7	8	9	10	11	12
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5	3	1	2	3	4	5	6	7	8	9	10	11	12
5	4	1	2	3	4	5	6	7	8	9	10	11	12
5	5	1	2	3	4	5	6	7	8	9	10	11	12

APPENDIX C - 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.
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