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Preface

The Multidisciplinary Center for Earthquake Engineering Research (MCEER) is a national center of excellence in advanced technology applications that is dedicated to the reduction of earthquake losses nationwide. Headquartered at the University at Buffalo, State University of New York, the Center was originally established by the National Science Foundation in 1986, as the National Center for Earthquake Engineering Research (NCEER).

Comprising a consortium of researchers from numerous disciplines and institutions throughout the United States, the Center's mission is to reduce earthquake losses through research and the application of advanced technologies that improve engineering, pre-earthquake planning and post-earthquake recovery strategies. Toward this end, the Center coordinates a nationwide program of multidisciplinary team research, education and outreach activities.

MCEER's research is conducted under the sponsorship of two major federal agencies, the National Science Foundation (NSF) and the Federal Highway Administration (FHWA), and the State of New York. Significant support is also derived from the Federal Emergency Management Agency (FEMA), other state governments, academic institutions, foreign governments and private industry.

The Center's FHWA-sponsored Highway Project develops retrofit and evaluation methodologies for existing bridges and other highway structures (including tunnels, retaining structures, slopes, culverts, and pavements), and improved seismic design criteria and procedures for bridges and other highway structures. Specifically, tasks are being conducted to:

- assess the vulnerability of highway systems, structures and components;
- develop concepts for retrofitting vulnerable highway structures and components;
- develop improved design and analysis methodologies for bridges, tunnels, and retaining structures, which include consideration of soil-structure interaction mechanisms and their influence on structural response;
- review and recommend improved seismic design and performance criteria for new highway systems and structures.

Highway Project research focuses on two distinct areas: the development of improved design criteria and philosophies for new or future highway construction, and the development of improved analysis and retrofitting methodologies for existing highway systems and structures. The research discussed in this report is a result of work conducted under the existing highway structures project, and was performed within Task 106-F-3.1a "Design Procedures for Longitudinal Restrainers in Multi-Span Simply Supported Bridges" of that project as shown in the flowchart on the following page.

The overall objective of this task was to review and compare the methods currently in use for the design of cable restrainers used to tie together simple spans in multiple-span, simply-supported bridges and to recommend improvements which will provide better performance while maintain-

ing ease and efficiency of design. Two existing methods, one each developed by Caltrans and AASHTO, were compared and evaluated. Three new methods were also proposed and evaluated. All five procedures were found to be effective in preventing spans from unseating in most bridges. However, the authors recommend that the procedure known as the "proposed" method be used for future design, as it provided the best overall performance under moderate-to-large earthquakes. A companion MCEER report, "Design Procedures for Hinge Restrainers and Hinge Seat Width for Multiple-Frame Bridges," by R. DesRoches and G.L. Fenves, MCEER-98-0013, provides similar evaluations and recommendations regarding the design of cable restrainers for multiple continuous span bridges.

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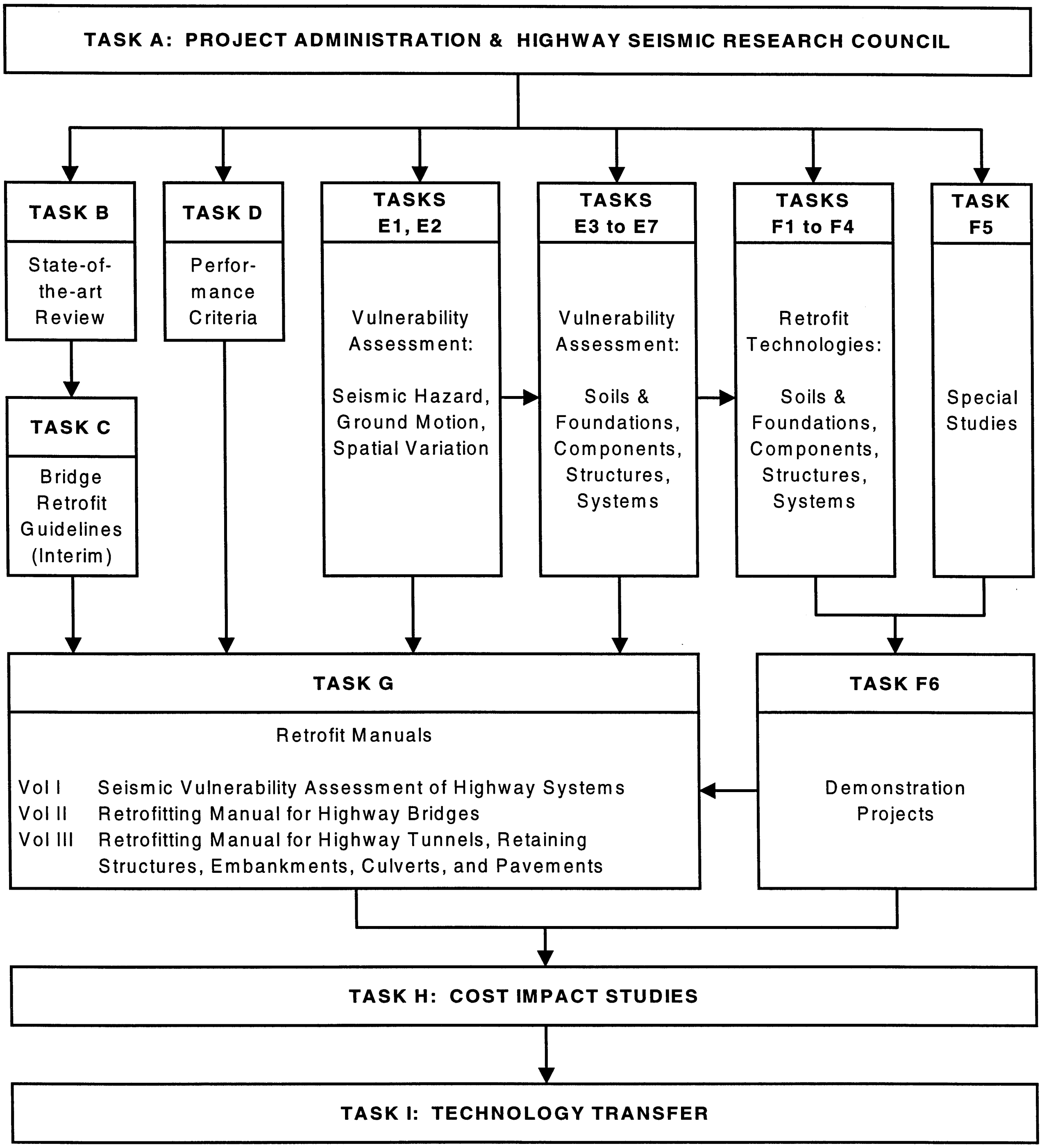

ABSTRACT

The 1971 San Fernando Earthquake in California caused highway bridges to collapse because of excessive longitudinal movement at expansion joints and supports. Since then, the California Department of Transportation (Caltrans) has installed longitudinal cable restrainers to prevent such collapse. In the study presented in this report, two existing design methods, the Caltrans and AASHTO methods, were evaluated for simply-supported bridges. In addition, three new design methods for restrainers were proposed and evaluated. The adequacy of the design methods was determined using nonlinear response history analyses of a large number of two- and five-span bridges. The models represented highway bridges that would be retrofitted with restrainers. All of the procedures considered in this study were found to be effective in preventing spans from unseating in most bridges even under strong earthquakes. In the most critical cases with narrow supports and skew, however, it was found that the Caltrans and a modified version of the Caltrans methods (a new method developed in this study) were the only procedures that prevented unseating.

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The last author who is from the University of Ljubljana, Slovenia, participated in this project in the course of a Fulbright visiting researcher award.

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SECTION 1
INTRODUCTION

1.1 Background

A bridge restrainer is any type of device that limits relative movements and prevents the loss of support. The 1971 San Fernando Earthquake in California caused many highway bridges to collapse because of excessive longitudinal movements at expansion joints and supports. Since then, the California Department of Transportation (Caltrans) has installed restrainers to prevent such collapse. Restrainers can be in the form of plates, rods, or cables. The most common type of restrainer in the U.S. is the cable restrainer connecting girders to the bent cap.

The most extensive use of restrainers in the United States (US) has been in California. Approximately 1400 bridges were retrofitted under the Caltrans Phase I retrofit program (Yashinsky, et al., 1995). The performance of cable restrainers was tested in earthquakes such as the 1987 Whittier Narrows Earthquake, the 1989 Loma Prieta Earthquake, and the 1994 Northridge Earthquake. They mostly performed well, but the collapse of bridges such as the Gavin Canyon Undercrossing and the Route 14/5 Separation during the Northridge Earthquake proved that inadequate restrainer design can have catastrophic results. While California and other western states have been retrofitting bridges for some time, other states in the U.S. have just begun to assess their seismic vulnerability and to develop seismic retrofit programs. This growing awareness of seismicity in regions other than the west will likely lead to seismic design and retrofit of more U.S. bridges with restrainers.

1.2 Objectives

The objectives of this study were:

- Review the performance of longitudinal cable restrainers on simple span bridges in past earthquakes.
- Evaluate the adequacy of current restrainer design procedures.
- Develop new restrainer design procedures that would prevent unseating.
- Evaluate the adequacy of the new restrainer design procedures.

A literature review was performed to assess the performance of cable restrainers in past earthquakes. The two current restrainer design procedures are methods described by Caltrans and AASHTO (American Association of State Highway and Transportation Officials). Three new restrainer design procedures were established entitled: the W/2 method, the Proposed method, and the Modified Caltrans method. All of these methods were evaluated in analytical studies with non-linear finite element models subjected to earthquake ground motions.

Straight, skew and non-skew, two- and five-span simply-supported highway bridges with narrow and wide seats were modeled and studied. Curved and continuous bridges were not studied. Incoherent ground motion was not considered. Ground motion was applied in the longitudinal direction for non-skew bridges and in both the longitudinal and transverse directions for skew bridges. Abutments, superstructure, columns, foundations, restrainers, bearings, and expansion joints were all included in the models. Element behavior was assumed to be non-linear with exception of the superstructure which was treated as elastic. A non-linear finite element analysis was adopted to model the bridge.

Cable restrainers were the only type of restrainer that was considered. They were assumed to connect spans to their seat in the longitudinal direction only. From the models, bridge response was observed and conclusions on the adequacy of each method were made.

1.3 Previous Studies

One of the first studies of restrainers was a full scale structure experimental test performed at the University of California at Los Angeles (Selna, et al. 1989). The purpose of the testing was to determine the strength, stiffness, and cyclic load-deflection behavior of a full-scale representative portion of a box-girder bridge in-span hinge which had been retrofitted with cable restrainers. It was found that the load-deflection behavior of restrainers was affected by the configuration of the cables around and through the diaphragm. A punching shear failure mode in the seat side of the diaphragm impacted the ultimate strength of the system. However, the tested capacity of the restrainer assembly was larger than the design capacity. Even though restrainers punching through the diaphragm was shown not to impact the vertical load carrying capacity of the hinge, it was concluded that such failure could lead to unseating of bridges with short seats.

The 1989 Loma Prieta earthquake gave researchers a chance to study the performance of in-span restrainers that were subjected to an actual earthquake (Saiidi et al., 1993). The objectives of a field study performed at the University of Nevada, Reno (UNR) was to review the performance of restrainers during the earthquake, to study and compare analytical model response to field results, to determine the effects of larger earthquakes, and to review current restrainer design procedures. The field study revealed that the majority of bridges studied activated their restrainers and in most cases the restrainers systems performed well. It was also determined that restrainers need to be treated as systems of three components: the restrainer, the connecting hardware and diaphragms, and the superstructure adjacent to the hinge. The weak link in the assembly may be predetermined depending on the seat width of the bridge. In old bridges, yielding should not be allowed in any of the components of a restrainer assembly in a maximum credible earthquake because excessive movements would result in collapse. In new bridges, yielding may be tolerated in the cables because seats are wide and cables can be easily replaced. In the analytical study, it was determined that restrainers indeed reduce hinge movements and prevent collapse of the superstructure. Also, wide seated bridges may not need restrainers because the displacements were not critical, but restrainers may be used to maintain the overall integrity of the superstructure. Bridges with relatively soft substructure and high ratio of number of hinges to number of spans are more vulnerable to loss of support. Restrainer forces are more critical when the restrainer slack is zero. It was also recommended that non-linear analysis be used for restrainer design. Further research concluded that critical abutment forces may occur when restrainer gap is maximum (Saiidi and Maragakis, 1995).

The Loma Prieta earthquake subjected bridges such as the Madrone Drive Undercrossing (O'Connor et al., 1993) and the San Gregorio Bridge (Maragakis et al., 1993) to strong ground motions. These two bridges were modeled with linear and non-linear finite element programs. The results were studied and compared with damage sustained during the Loma Prieta Earthquake. In both cases, it was determined that non-linear finite element models accurately predict damage resulting from an earthquake. Restrainers also were shown to be an effective retrofitting device. Restrainers in both bridges prevented the spans from collapse.

The damage and collapse of two-level viaducts in the Loma Prieta Earthquake prompted researchers at the University of California, Berkeley (UCB) to study their response (Singh and Fenves, 1994). Two-level viaducts were studied using linear and non-linear finite element analysis. Drain-2DX (Prakash et al., 1993) was the program used in the analysis. It was found that cable restrainers limited the out-of-phase motion between adjacent frames. The number of restrainers required by the Caltrans method was found to be excessive in terms of what was needed to keep spans from unseating. It was also shown that relative displacements at hinges were larger when ground motions were non-uniform instead of uniform. In viaducts subjected to non-uniform ground motions, an unrealistically large number of restrainers was required to limit relative displacements to less than the yield displacement of the cable.

An analytical parametric study was performed by researchers at the University of California, San Diego to investigate the influence of different bridge properties on the relative longitudinal displacements at hinges (Yang et al., 1994). The varied parameters in this study were: adjacent bridge frame stiffness, restrainer stiffness, earthquake intensity, and sliding friction and gap at the expansion joint. This study concluded that the Caltrans method did not predict the relative movement of adjacent frames accurately and led to a poor prediction of relative displacement at expansion joints. This study also showed that expansion joint gap and Coulomb friction had a small impact on the relative displacement between adjacent frames. Results showed that greater earthquake intensity increased frame displacements. When the response was elastic, relative displacements increased with increasing earthquake intensity. However, relative displacements did not change significantly with increasing earthquake intensity when non-linear behavior dominated the response.

The Northwest Connector is a bridge in southern California at the Interstate 10/215 interchange that was instrumented by the California Division of Mines and Geology. When the 1992 Landers and Big Bear Earthquakes occurred, the response of the Northwest Connector was measured (Fenves and Desroches, 1994). Researchers at UC Berkeley studied the measured response and compared it to the response of non-linear finite element models analyzed using Drain-2DX. Their models captured the longitudinal and transverse response well for both earthquakes. The Drain-2DX model represented hinge displacements well but not hinge velocities and accelerations. Their study showed that non-linear models were adequate for estimating the response of the bridge and were recommended over linear hinge models.

The 1994 Northridge Earthquake gave researchers and Caltrans another chance to evaluate seismic retrofit techniques (Yashinsky et al., 1995). Most of the bridges retrofitted with restrainers performed well. However, three bridges, the Gavin Canyon Undercrossing bridges (2 bridges) and the Route 14/5 Separation and Overhead collapsed. The Gavin Canyon Undercrossing bridges were tall and highly skewed. The site was 14.8km from the epicenter, and the peak ground acceleration was estimated at 0.6g. The collapse was due to the failure of the restrainers to limit the longitudinal movement of the bridge and due to an inadequate seat width. The Route 14/5 Separation and Overhead was subjected to an estimated peak ground acceleration between 0.7g and 1g. In addition to the shear failure of short columns, the bridge suffered abutment and hinge damage resulting from a poor restrainer design. Restrainers were grouted into steel pipes which did not allow the restrainers to stretch. They were also designed for a force equal to 25 percent of the heavier span which was too low for a site that may have experienced 1g. The failure of the restrainers was also due to the failure of the diaphragm which was shown to have the capacity to resist the force of 5 restrainer units instead of the 7 that were installed.

In January 1995 the Hanshin/Awaji Earthquake struck Kobe, Japan and caused massive destruction of highway bridges (Kawashima, 1995). Superstructures unseated from many bridges although restrainers (falling-down prevention devices) were installed. The most notable case of unseating was an approach span to the Nishinomiya Bridge on Route 5, the Hanshin Expressway. The portion of the bridge that unseated was the span before the main span crossing the Nishinomiya Channel. The unseated approach span had a length of 52m and was supported by two steel frame piers that were 25m and 24m tall. Liquefaction occurred around the bridge which resulted in lateral movement and tilting of the foundations. The restrainers were "tie bar type" and had a gap of 155mm. The bearing width was 110mm. The tilting of the foundations, the piers, the failure of the restrainers, the narrow seat, and the failure of the "pivot" bearings resulted in the unseating.

Researchers from U.S. universities and agencies traveled to Kobe and learned many lessons from the Hanshin/Awaji Earthquake also. Generally, steel bridges performed well in the Kobe earthquake except for the fact that many spans dropped due to bridge bearings and restrainers that were very stiff and brittle (Astaneh-Asl and Kanada, 1995) (Yashinsky, 1995).

Researchers from UNR determined important parameters in bridge restrainer design for seismic retrofit (Saiidi et al., 1992). The objective of this study was to determine the effects of changing the cross sectional area of restrainers and changing the slack in the restrainer on the non-linear response of a representative bridge with several hinges. A representative bridge was chosen and modeled with NEABS-86 and subjected to the 1940 El Centro, the 1954 Eureka, and the 1989 Loma Prieta Earthquakes. Only cable type restrainers were considered. Restrainer area was evaluated by changing the number of restrainers. Two different gaps of 19mm and 0mm were chosen to represent temperature change. A large number of restrainers was needed when the gap was equal to zero. A reduction in the number of cables generally increased the magnitude of stresses; thus, the most critical condition for cable stresses is when the restrainer gap is reduced to zero due to low temperature. To determine the most critical abutment forces it is necessary to analyze the bridge for both the extreme cold (maximum gap) and extreme high (minimum gap) ambient temperatures. Smaller restrainer gaps lead to larger restrainer forces but smaller abutment forces. The study concluded that the extreme low ambient temperature condition should be considered to determine the critical restrainer stresses, and that maximum relative displacements may occur for the case of zero or non-zero restrainer gap depending on the number of restrainers. The current design procedure for in-span hinges was deemed far from accurate because of its lack of consideration of the parameters discussed above.

Trochalakis, et al., 1995 from the University of Washington (UW) conducted an evaluation of current restrainer design practices for in-span hinges and developed a new procedure (Trochalakis et al., 1995). The AASHTO seat width equation provided a conservative estimate of displacement response. The Caltrans restrainer design procedure estimated a conservative number of restrainers when adjacent bridge frames had similar stiffnesses, and it led to an unconservative number of restrainers when a very flexible frame was adjacent to a very stiff frame. This study found that restrainer stiffness, restrainer gap, period ratio of adjacent frames, and stiffness ratio of adjacent frames were the important parameters in restrainer design, and incorporated these parameters into a new restrainer design procedure for in-span hinges. The procedure is based on single-degree-of-freedom response of the two adjacent frames using an equivalent stiffness. The mean response of both frames is determined and used in an empirical equation to determine the mean relative hinge displacement which must be smaller than the available seat width.

Recently, researchers from UCB also evaluated the Caltrans and AASHTO restrainer design procedures and developed a new design procedure for restrainers at in-span hinges (Fenves and Desroches, 1996). In the UCB procedure, the response of two adjacent frames is calculated as two independent single-degree-of-freedom oscillators. Non-linear frame behavior is accounted for with an effective stiffness that is based on an elastic stiffness but is reduced by a ductility factor. A modified damping ratio that is also ductility based is used. Without restrainers, the displacement response of the two oscillators is combined using the Complete Quadratic Combination (CQC) method and is compared with the available seat width. With restrainers, the relative response of the two degree-of-freedom system is calculated using CQC. The results showed that the current Caltrans design procedure required an unconservative number of restrainers for small period ratios (of adjacent frames) and required a conservative number of restrainers for high period ratios. The new procedure proved to predict the number of restrainers required to limit hinge displacement more accurately than current procedures. Another conclusion drawn was that frames with higher ductility levels exhibit in-phase motion. Frames moving in-phase will have small relative displacements and will require fewer restrainers.

A study of unseating in simply supported spans was performed recently by researchers at UW (Trochalakis et al., 1996). The study was limited to 68 analyses in which six different parameters were varied. Pier heights, abutment stiffness and strength, earthquake record and intensity, compression gap, bearing factor, and restrainer stiffness were varied. The study concluded that bearing pad friction, earthquake record and intensity, and compression gap were important parameters in determining the maximum relative displacement at a bearing support, and that bearing pad friction and earthquake record and intensity were the most important parameters affecting the maximum relative displacement at an abutment. The study also found that restrainers were ineffective in reducing relative displacements of the simply-supported bridges considered in the study. However, in this study restrainers were attached from span to span instead of span to bent cap. Span to span connection of restrainers is not a common restrainer configuration. In fact, span to span connection of restrainers is not advised and is not typically used by Caltrans (see section 2.2).

Few studies have been done on skewed bridges equipped with restrainers. The Whitewater Bridge is a 2 span skewed structure that was retrofitted with cable restrainers and subjected to the 1986 Palm Springs Earthquake (Maragakis et al., 1996). The bridge was damaged during the earthquake and was closed for a few days after the earthquake. The damage was due to the deactivation of one of the restrainers resulting in excessive forces in the other restrainers. This was probably due to the skewness of the bridge and the effects of the tie bars. The objective of the UNR study was to model the bridge non-linearly and to compare the results of the model with the damage that the structure suffered in the earthquake. Special details were considered when modeling the expansion joints. The model successfully predicted the damage.

The most recent analytical study of the response of skewed bridges was performed at the University of Washington (Bjornsson et al., 1997) although the focus was not on restrainers. In this study different models of a two-span continuous skewed bridge were analyzed with Drain 2DX. The study determined that skew bridges cause relative displacements at the abutments that are greater than non-skew bridges. Deck rotations of skewed bridges are caused by impact of the superstructure with the abutment. In-plane rotation occurs in a direction that increases the gap at acute corners. Upper and lower bounds of skew angles exist; outside of which rotation is prevented by abutments. The bounds are a function of the plan view aspect ratio of the deck. A specific skew angle exists that maximizes rotation of the bridge deck for the given aspect ratio which depends on the abutment stiffness and strength, the bridge total weight, the total stiffness and strength of the bridge, and the eccentricity between the centers of mass and stiffness. Bi-directional ground motion should be taken into account when analyzing a skewed bridge. The maximum displacements can be expected when the maximum resultant peak ground acceleration is perpendicular to the abutment face.

SECTION 2
EXISTING RESTRAINER DESIGN METHODS

2.1 Introduction

Two procedures are currently used to design restrainers in the United States. Caltrans (California Department of Transportation) and AASHTO (American Association of State Highway and Transportation Officials) each have methods to design restrainers for bridges with the potential to become unseated in an earthquake. The purpose of this chapter is to describe these methods and to discuss some of the details of each method.

2.2 The Caltrans Method

The objective of the Caltrans restrainer design method is to limit longitudinal movement of adjacent bridge elements and to keep a bridge structure tied together during severe earthquake shaking. An ideal restrainer set designed with the Caltrans method should keep spans from falling off their supports, dissipate energy, and return the structure segments to their pre-earthquake positions (Caltrans, 1994).

Caltrans first started designing and installing restrainers after the 1971 San Fernando Earthquake. Initially, restrainers were designed by the working stress method wherein a working load of 50 percent of the ultimate strength for cables plus 33 percent overstress was permitted for seismic conditions. Common cable restrainers have a diameter of 19 mm (0.75 in.) and a yield stress of 1214 MPa (176.1 ksi). This led to an allowable load of 136kN (30.6 kips) per cable. For the design which is currently used, a yield strength of 85 percent of ultimate load is used. This leads to a load of 174 kN (39.1 kips) per cable. In 1978, restrainers were designed to resist a force equal to at least 25 percent of the weight of the lighter segment of the superstructure they connected. Their resistance was based on working stress design. Substructure contribution was ignored (Degenkolb, 1978). The procedure was later modified such that a set must resist a force equal to the span weight times the site's acceleration of gravity. This method is the same as the current AASHTO method. The current Caltrans method has been used since the early eighties and is described in this section.

Either cables or bars may be used as a restraining device. Neither cables nor bars dissipate appreciable energy since they act in the elastic range. However, cables tensioned repeatedly in the elastic range will store more energy than bars. Most of Caltrans' standard restrainer details use cables rather than bars. Cables provide more flexibility for construction; they can be used in more retrofit schemes because they can be placed in many different configurations.

The current Caltrans restrainer design procedure uses an equivalent static analysis that may be performed by hand. The method is also adopted in the current Federal Highway Administration seismic retrofit manual (FHWA, 1995). The general procedure is as follows (Caltrans, 1994):

1. Compute maximum permissible restrainer displacement and limit displacement to the hinge seat width.

2. Compute maximum displacement due to earthquake.
3. Compare results from steps 1 and 2 and determine course of action.
4. Determine number of required restrainers.
5. Check displacements of the restrained system and revise restrainer design if necessary. Repeat steps 1-5 if needed.

The maximum permissible restrainer displacements are calculated with the following equations:

$$D_r = D_y + D_g \tag{2-1}$$

where:

D_r = the maximum permissible restrainer displacement
D_y = the restrainer displacement at yield
D_g = the gap in the restrainer system (slack)

$$D_y = \frac{f_y * L}{E} \tag{2-2}$$

where:

f_y = yield stress in restrainer, 1213 MPa (176 ksi) (cables only)
L = restrainer length
E = initial modulus of elasticity of restrainer, 68947 MPa (10000 ksi) (cables only)

Maximum displacements are computed from a spectral analysis. The major assumption in the Caltrans method is that the bearings have failed and that the total structure stiffness is the stiffness of the restrainers only. Thus, for simple span bridges steps 2,3, 4, and 5 are combined into one step. The designer must assume a number of restrainers and then calculate earthquake forces with restrainer stiffness, span weight, period of vibration, and Caltrans' ARS (Acceleration Response Spectra) spectra (Caltrans, 1990). The period of vibration is based on the stiffness provided from an assumed number of restrainers and the superstructure weight of the simply supported span. Restrainer stiffness is:

$$K_t = \frac{f_y * n * A_r}{D_r} \tag{2-3}$$

where:

n = number of restrainers
A_r = Area of one restrainer, 143 mm^2 (0.222 in^2) for 19mm (3/4 in.) diameter cable.

The period of vibration is calculated with:

$$T = 2\pi\sqrt{\frac{W}{K_t * g}} \tag{2-4}$$

where:

T = period of vibration
W = weight of the span under consideration
g = the gravity acceleration

Once the period of vibration is calculated, an acceleration coefficient may be taken from the ARS spectra corresponding to the bridge site soil condition. This coefficient is multiplied by the weight of the span to obtain an earthquake force which is then applied to the structure. Displacements are calculated based on the restrainer stiffness and the calculated earthquake force. Restrainer displacement is calculated from the following equation:

$$D_t = \frac{ARS * W}{K_t} \tag{2-5}$$

where:
D_t = the total restrainer displacement
ARS = ARS acceleration coefficient

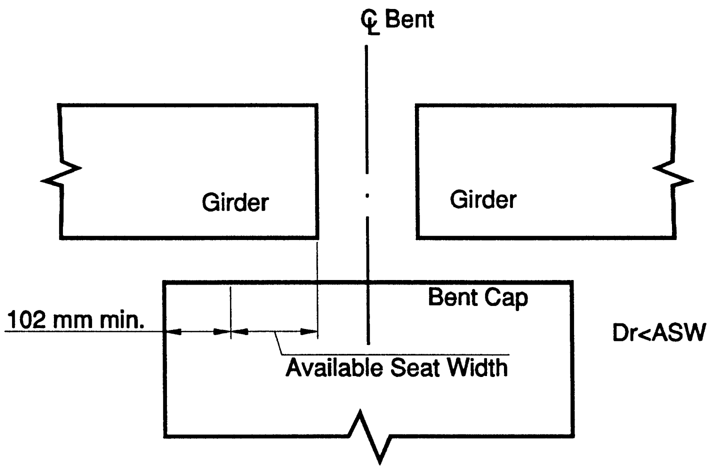


FIGURE 2-1 Available Seat Width

If the calculated restrainer displacement, D_t, is less than the maximum allowable restrainer displacement, D_r, then the analysis is complete. If D_t is greater than D_r then a larger number restrainers is assumed and a new D_t is calculated and compared to D_r. If restrainer displacement exceeds the available seat width (figure 2-1) the designer must add more restrainers. Once the displacement criteria are met, the design is complete.

2.3 Discussion of the Caltrans Method

A distinct feature of the Caltrans method is the available seat width (ASW). The available seat width is the total seat width minus 102mm (4 in.) (figure 2-1). Caltrans assumes that if a girder violates the available seat width and moves within 102mm of the edge of the bent cap, local bearing failure of the bent cap will occur and the span will fall. This limit is based on a 51-mm (2-in.) concrete cover on reinforcement for each of the girder and bent cap. When the seat width is reduced to this limit, it is assumed that the concrete cover on both the bent cap and the girder will fail and the girder will unseat.

Long restrainers and short seat widths influence the displacement limit, D_r, as computed in step one. Since restrainers are designed to remain elastic, the maximum permissible displacement is typically close to the restrainer yield displacement therefore, the available seat width must be greater than the restrainer yield displacement. Since the yield displacement is small (approximately 100 mm (4 in) for a 6m (20 ft.) cable), only a very long cable with a very short seat width would violate this criterion. If designed correctly, a restrainer for a span supported by a long seat width will not utilize most of the available seat width.

Caltrans' design method has proven effective for keeping bridges supported (Saiidi et al., 1993), but can also lead to large numbers of restrainers. Engineers and researchers have speculated that Caltrans may require a number of restrainers that is too conservative for the maximum credible event. Even though restrainers themselves are not costly, excessive quantities of restrainers can increase the retrofit cost, complicate detailing, and increase traffic delays.

With larger seat widths, designers have the option of using different lengths of cable. Changing the cable length on simply supported bridges has little effect on the number of restrainers required by the Caltrans method (See Section 5). This is due to the fact that all restrainers, no matter what length, will resist the same amount of force at yield. However, the length of restrainer has a significant effect on restrainer stiffness which may change the period of the structure and change the seismic force that the cables are required to resist.

Different restrainer gaps (slack in the cable) can be considered using the Caltrans procedure. Restrainer slack is taken into account by adding the amount of slack to the yield deflection in step 1. Slack in restrainers is required to accommodate superstructure thermal. Slack was taken as zero throughout the study because the most critical stresses in restrainers occur when there is no slack in restrainers (Saiidi et al., 1993).

Soil effects can be taken into account in restrainer design by using different spectra as provided by Caltrans. The ARS soil characteristics do not necessarily represent the stiffness of the substructure; rather, they represent the amplification and damping characteristics of the soil at the bridge site.

The Caltrans restrainer design procedure does not address skew specifically. The number of restrainers is not modified for skewed bridges; however, Caltrans uses the AASHTO equation to determine the required seat width (see Section 2.4). When the required seat width is satisfied, a designer's judgement determines whether or not a minimum number of restrainers is used or a larger number from analysis. The required seat width is typically large compared to the restrainer yield displacement, and would rarely be violated if restrainers were attached to a bridge and were designed to perform in the elastic range. Whether or not bridges that satisfy this condition have the potential to become unseated is unknown. Generally, moderate seismic events could not generate sufficient displacement demand to violate Caltrans seat width requirements.

The assumption that bearings always fail in a strong earthquake may or may not be true. Bearing structural properties vary widely from case to case. Strong bearings may not fail in a moderate to large event, and in many cases when bearings fail they still may act as a structural component in a a bridge by withstanding a certain amount of force (Mander et al., 1996). However, one can not assume that bearings will always function well in an earthquake. The Kobe earthquake proved that bearings are a seismically vulnerable element (Yashinsky, 1995). The fact is that in both older and newer bridges bearings are not explicitly designed to take seismic forces, and whether or not they will withstand seismic forces is unknown and can vary from case to case. When bearings fail, the Caltrans assumption that the bearings are non-existent may be the best estimate of the bridge properties in a worst possible case. Although Caltrans suggests that designers evaluate the adequacy of bearings for multiple simple-spans, in practice this is not typically done (Caltrans, 1990).

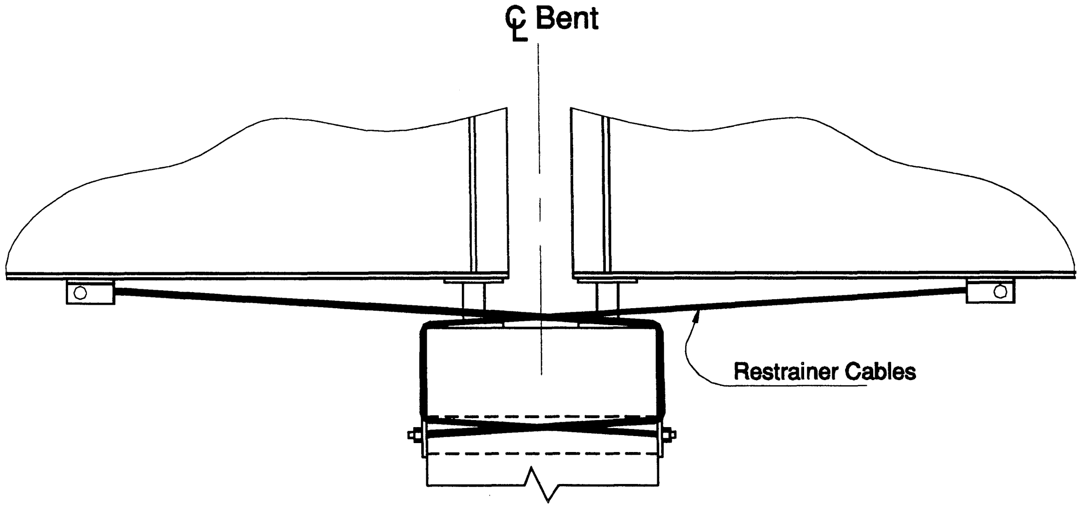


FIGURE 2-2 Girder to Column Restrainer Attachment

Caltrans also suggests different methods to attach restrainers. For spans that are supported at piers, two attachment details are available. Restrainers may attach directly girders from two adjacent spans, or may attach the girder to the pier (figure 2-2). In general it is preferred to attach restrainers in the second manner (Caltrans, 1994). Attaching restrainers from girder to girder may be preferable when the additional force applied by the restrainers to the bent cap may potentially fail the pier. This case also may be used for bridges with few spans and large seat widths. In general, however, girder to girder attachment is less desirable because the girders are not directly attached to the pier on which they are seated. Flexible columns may experience large deflections in an earthquake which may lead to unseating if the girders are not attached to the piers directly.

The Caltrans procedure is attractive because it is simple and has generally been proven to be effective when retrofitting bridges in high seismic areas that have the potential to become unseated in an earthquake (Yashinsky et al., 1995). The Caltrans method also includes effects such as site specific spectra that can have a significant effect on bridge seismic response. However, the Caltrans method leads to a large number of restrainers in some cases, and neglects the effects that bearings, columns, and footings have on the response.

2.4 The AASHTO Method

The current AASHTO method (AASHTO, 1996) for restrainer design was adopted from earlier versions of Caltrans codes and is intended for new bridge design. For retrofit of existing bridges, the current Caltrans procedure is recommended in the FHWA retrofit manual (FHWA, 1995). Similar to the Caltrans method, the objective of the AASHTO method is to design a restrainer that limits the relative displacement between the superstructure and support to the point that the superstructure remains seated during a strong earthquake. This method also allows designers to determine a number of restrainers that will keep a bridge seated with a simple hand analysis.

Rather than specifying a detailed procedure, the AASHTO code defines a method to determine the longitudinal linkage force that a set of restrainers must withstand. The force is defined as the site acceleration coefficient times the weight of the lighter of the two adjoining spans or parts of the structure (AASHTO, 1996). Restrainers designed for a simply supported bridge would resist the force generated from the span weight. The AASHTO procedure is as follows:

1. Determine the longitudinal linkage force:

$$F = A^{*} W \tag{2-6}$$

where:
F = longitudinal linkage force.
A = site's acceleration coefficient (in terms of g)
W = weight of span under consideration

2. Determine the number of restrainers:

$$n = \frac{\left(F/f_y\right)}{A_r} \tag{2-7}$$

AASHTO does not require that designers check the displacement of the restrained system versus the available seat width; however, a bridge seat width must satisfy the following criterion for seismic categories C and D which correspond to a maximum ground accelerations greater than 0.19*g (AASHTO, 1996):

$$N = (305 + 2.5L + 10H)(1 + 0.000125S^2) \tag{2-8}$$

where:
L = Length in meters of the bridge deck.
H = Column or pier height.
S = Angle of skew of the support in degrees measured from a line normal to the span.

Seismic categories A and B, which correspond to areas with smaller seismic risk, have less stringent seat width requirements. AASHTO requires that all bridges in categories C and D must have restrainers and must satisfy seat width requirements. Bridges in categories A and B must satisfy seat width requirements only.

2.5 Discussion of the AASHTO Method

The most attractive quality of the AASHTO method is that it is simple. However, the AASHTO method neglects many important elements such as seat width, substructure, abutments, bearings, and skew.

In general, Caltrans requires many more restrainers than AASHTO for a bridge with the same site coefficient. Caltrans spectra amplify the design forces, thus leading to a larger number of restrainers.

SECTION 3
NEW RESTRAINER DESIGN METHODS

3.1 Introduction

One of the objectives of this research was to develop new restrainer design procedures. In this chapter three new methods are presented and discussed. All three methods were developed in the course of the current study. The restrainers in the first method are intended to support the weight of the superstructure, W, after unseating, and the procedure is referred to as the "W/2" method. The second procedure uses modal analysis and accounts for substructure and bearing properties, gaps, etc., and is the most elaborate of the new methods. This procedure is labeled as the "Proposed" method. Finally, the Caltrans/FHWA retrofit method was modified, and is referred to in the report as the "Modified Caltrans" method.

3.2 The W/2 Method

The first new method developed was the W/2 (W over two) method. The performance criterion for this method differs from that of the others in that it assumes that the superstructure is unseated in an earthquake. After unseating, the span would hang, supported by restrainer cables until construction crews could put the span back on its support. The objective of the W/2 method is not to prevent unseating but to prevent collapse.

The main advantage of the W/2 method is its simplicity. The method is meant to be a crude design method that can be executed by hand with ease. In this method restrainers on each side of the span must resist one-half of the weight of the span. The load to be resisted by the restrainers is treated as a static force even though the dropping of the span off the support would produce a dynamic force that may be up to twice the static force. However, it is believed that the restrainer force will be less than the full dynamic force due to friction between the girder and bent cap and due to a redistribution of forces to girders that are still seated.

The number of restrainers required to resist W/2 can be determined from the following equation:

$$n = \frac{\left(\dfrac{W/2}{f_y} \right)}{A_r} \qquad (3\text{-}1)$$

where:

n = the number of restrainers
W = weight of the span under consideration
f_y = yield stress of restrainer
A_r = area of one restrainer

3.3 Discussion of the W/2 Method

The most attractive quality of W/2 is its simplicity. Like AASHTO, one may use this method to design restrainers quickly for bridges that are very complex. While the design may not take long to perform, the method neglects the same parameters as AASHTO such as substructure, abutments, skew, bearings, seat width, and interaction among different spans. The W/2 method also neglects bridge site seismicity, and any dynamic effects that the restrainers and the bridge may undergo.

The performance criterion of W/2 is very different from other existing design procedures. Letting the bridge unseat is a very plausible option for bridges in low or moderate seismic areas because retrofit costs would be minimal, and the bridge would be repairable in the event that unseating occurs.

The W/2 method is a simple method that can be used to quickly estimate the number of restrainers needed for bridge retrofit. The number of restrainers suggested by W/2 should be treated as a crude approximation of the number that is needed in larger seismic events.

3.4 The Proposed Method

The second restrainer design method developed in this research was labeled the Proposed method. The objective in this method is similar to that of Caltrans in that it attempts to limit longitudinal movement of adjacent bridge elements and to keep a bridge structure seated during strong earthquakes. To achieve this the Proposed method uses many parameters to determine the bridge response. The hypothesis is that if all of the important parameters are considered, the correlation between the actual response and predicted response will be closer than that of others. Adding such bridge parameters would certainly increase the analysis complexity. Hence, another objective of the proposed method is to add all the complexity of a bridge, but keep the procedure sufficiently simple so that an engineer may perform the analysis by hand.

The proposed general procedure is as follows:

1. Determine bridge properties.
2. Determine support location, and determine if support acts as a pin or roller (expansion bearing).
3. Determine if restrainers are needed by evaluating the bearing capacity and by checking displacements of the unrestrained system.
4. Assume a number of restrainers.
5. Check displacement of restrained system.
6. Check stresses in restrainers to ensure they remain elastic.
7. Add to or subtract from the number of restrainers as needed to achieve an optimum and safe design.

A more detailed outline of the proposed method is summarized in table 3-1.

The first step in designing restrainers by the Proposed method is to determine bridge properties. Bearing stiffness and strength may be determined by methods from Mander et al., 1996. Column stiffnesses are determined assuming cracked section properties. Cracked properties correspond to 50 percent of the gross section moment of inertia (ACI, 1996), and account for some column non-linearity associated with reinforced concrete. Footing stiffness may be calculated using the methods in FHWA, 1995, Brettmann and Duncan, 1996, Darwish et al., 1997, and Norris, 1994. Abutment stiffness and strength may be calculated using methods found in Caltrans, 1990. In addition, weights of the bridge superstructure, columns, and bent cap need to be calculated.

A distinct feature of the Proposed method is that each support is considered independently. Different numbers of restrainers may be placed on each side of a span. For example, a span supported by an abutment on one end, and a bent on the other may require different numbers of restrainers at the abutment versus the bent. At each location a designer must determine if the support is a pin or roller. If the location is supported by a pin, the next step the designer must take is to evaluate the strength of the bearing. After the bearing strength is evaluated, a number of restrainers may be determined. If the support is a roller, one must determine the unrestrained displacement. If the available seat width is exceeded by the unrestrained displacement restrainers must be added.

For example, to design restrainers for a typical two-span bridge (figure 3-1) one would start at abutment 1 and determine whether the span is pinned or supported by a roller. Note that in the figure and throughout the report a bearing that prevents longitudinal movements is referred to as "pinned" and one that allows for longitudinal movement (expansion bearing) is referred to as "roller." In figure 3-1 abutment 1 has a roller support. To determine whether or not restrainers are needed, one would determine the relative displacement between the girder supporting the first span and abutment 1. If the relative displacement exceeds the available seat width the restrainers are needed. If not, a minimum number of restrainers would be used. After the abutment restrainers were designed, then one would proceed to the pier and consider the side facing abutment 1. The bearings would be evaluated and restrainers designed. Then, one would consider the right side of the pier. This side would be facing the right abutment. Finally, the right abutment would be considered, and the design would be complete. In figure 3-1 the locations of the bridge could be abbreviated as A1-R (Abutment 1 - Roller), B2-P (Bent 2 - Pinned), B2-R, and A3-P.

The next feature of the proposed method is that it distinguishes among three different locations on a bridge: abutments, end-spans, and mid-spans. The purpose of treating these locations differently is the way the unrestrained displacements and number of restrainers are determined. Definition of these locations may be found in table 3-1. Obviously, A1-R and A3-P are classified as, "abutments." Both B2-P and B2-R are end-spans because they are supported on one side by an

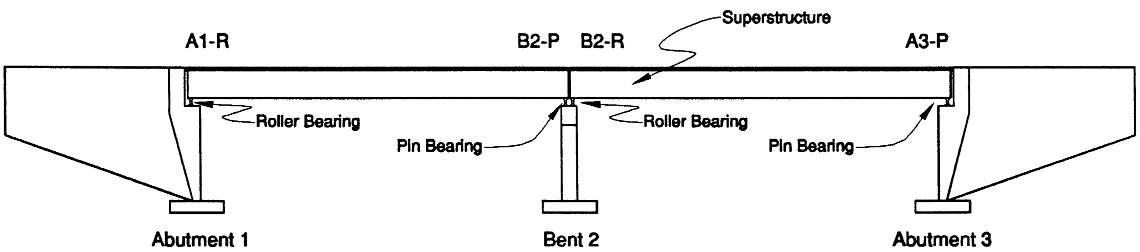


FIGURE 3-1 Typical Two-Span Bridge

abutment and by a bent on the other. Mid-spans would occur in bridges that have 3 or more spans. For any bridge one must consider the following six cases:

1. Pinned - Abutments
2. Pinned - End-spans
3. Pinned - Mid-spans
4. Roller - Abutments
5. Roller - End-spans
6. Roller - Mid-spans

All of these cases can be generalized into single-degree-of-freedom (SDOF) models with linear or non-linear springs. SDOF models are used because they can be analyzed by hand using response spectra. Both, Caltrans and AASHTO codes include response spectra to determine maximum bridge response. A SDOF analysis would work well with current codes and spectra.

3.4.1 Pinned Supports

The model of a pinned girder end at an abutment is shown in figure 3-2. Note that abutment stiffness is not included in this model since unseating occurs when the span moves away from the abutment. To find the unrestrained displacements and the force applied to the bearing, Kr would be equal to zero and the total stiffness would be equal to the bearing stiffness. A non-linear elasto-plastic force displacement relationship may be assumed for steel bearings. The post yield stiffness can be assumed to be the initial stiffness divided by 1000. Initially, typical bearings are very stiff (Mander et al.,1996).

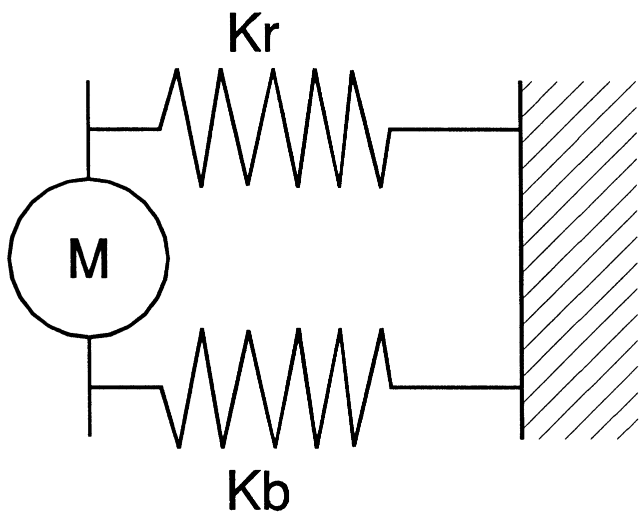


FIGURE 3-2 Pinned Abutment Single Degree of Freedom Model

To determine the force applied to the bearing one would multiply the ground acceleration by the weight of the span as shown in table 3-1, line 2.4.3. The force is then compared to the permissible bearing strength as shown in line 3.1.1. If the bearing does not fail and seat width requirements are met, a minimum number of restrainers is used. The seat width check is an extra safety measure and is not necessary if there is confidence in the estimated strength of the bearing. If the bearing fails, then the unrestrained displacements must be checked assuming the non-linear relationship for bearings. A chord stiffness is determined and used with the span weight to determine a period. Determining the correct chord stiffness is an iterative process. Chord stiffness can be determined as follows:

1. Assume a displacement value greater than the yield displacement.
2. Calculate associated force based on force displacement relationship.
3. Calculate chord stiffness by dividing the force by the displacement.
4. Calculate period of SDOF system.
5. Determine ARS coefficient.
6. Calculate seismic force by multiplying SDOF weight by the ARS coefficient.
7. If the calculated force equals the assumed force then the chord stiffness and assumed displacement are correct. If the forces are not equal then go to step 1.

After the chord stiffness is determined, the unrestrained displacement is found. If this displacement is smaller than the available seat width then a minimum number of restrainers is used. If the available seat width is exceeded then a number of restrainers is assumed and the system is reanalyzed. Again the bearing force is checked, but seismic forces are found with the added stiffness of the restrainers (figure 3-2). If the bearing fails, then a chord stiffness is found and the final displacement determined. To find the force in the restrainers one can divide the final restrained displacement by the restrainer stiffness. From this force a stress is determined and compared with the restrainer's yield stress. At this point the number of restrainers is adjusted, and the system reanalyzed. When the restrainer stress becomes close to the yield stress (but lower) the design is complete.

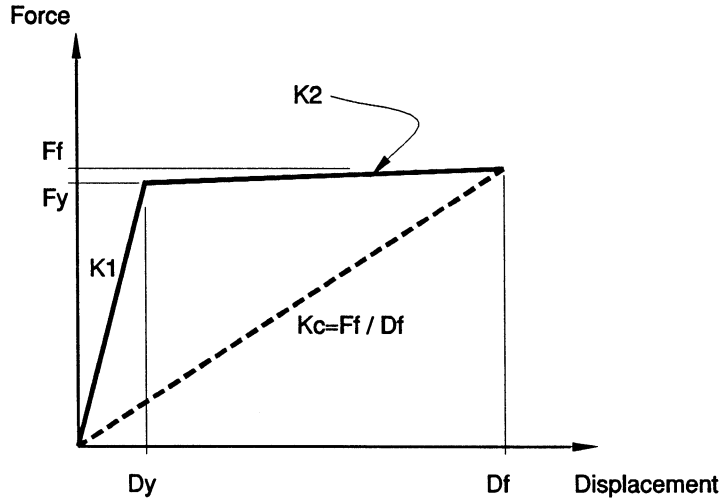


FIGURE 3-3 Non-Linear Idealization (Chord Stiffness, Kc)

The pinned end span SDOF model is shown in figure 3-4. To determine the force on the bearing without restrainers, bearing, column, and footing flexibilities are combined as shown in table 3-1 line 2.1.1.

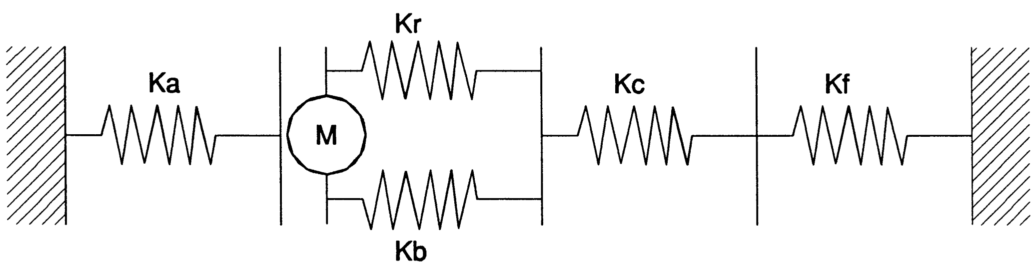


FIGURE 3-4 Pinned Endspan SDOF Model

Once the earthquake forces are found, displacements may be calculated and compared to the expansion gap between the superstructure and abutment. This assumes that the structure is moving toward the abutment. If the gap between the abutment and superstructure does not close, bearing forces are calculated and compared with the bearing strength. If the gap closes then the system is non-linear, and a chord stiffness must be calculated that accounts for the added stiffness of the abutment (figure 3-3). When determining the response with the abutment engaged, abutment forces are checked to determine whether or not the abutment fails. Forces must be checked in both the abutment and the bearings to determine which fails first. Forces must continue to be tracked until the state of the bearing is found. If the bearing does not fail and the relative displacement between the column and superstructure is less than the available seat width, then a minimum number of restrainers is recommended. If the bearing fails, then the unrestrained displacement of the system must be found and compared to the available seat width. If the available seat width is exceeded then restrainers must added. The total structure stiffness is calculated the same as without restrainers except that the bearing stiffness and restrainer stiffness are summed. The sum of the stiffness is then inverted to determine the flexibility. That flexibility is then added to the column and footing flexibility to form the total structural flexibility. Abutment and bearing non-linearities are considered in the same manner as the unrestrained system. Once the system displacements are found, the displacement in the restrainers may be found by subtracting the displacement at the top of the column from total displacement. This displacement is compared to the available seat width and then stresses are calculated. The number of restrainers is then changed to achieve an optimum design.

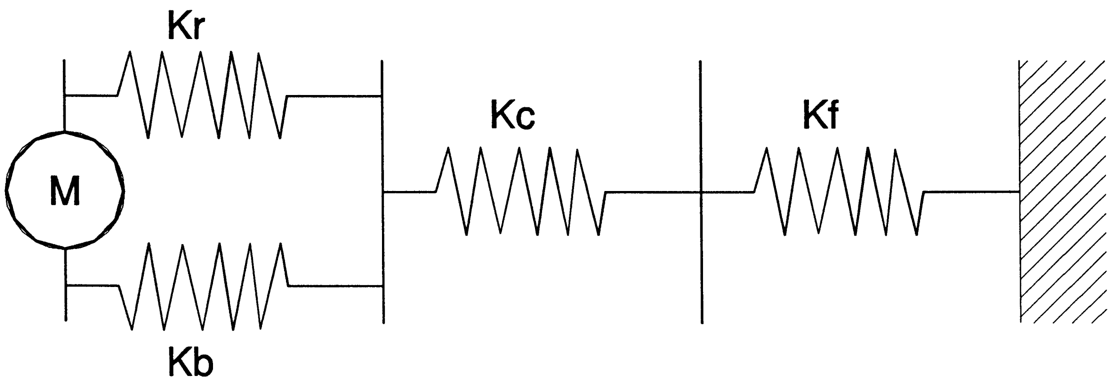


FIGURE 3-5 Pinned Midspan SDOF Model

The SDOF model for a pinned midspan is shown in figure 3-5. The only difference between a pinned end-span and a pinned mid-span is that midspans do not consider the effects of abutments or span interaction. Unrestrained and restrained displacements are calculated the same way as in pinned endspan except that abutment effects are neglected. In this case the total mass of the structure is represented by the mass of the span, bent cap and tributary mass of the column. The total relative displacement is found by subtracting the displacement of the column from the total displacement.

3.4.2 Roller Supports

Consider a seat at an abutment supporting a girder with a roller. This would be termed a "roller abutment." To determine the relative displacement of the girder relative to the abutment, the maximum relative displacement of girder A (figure 3-6) must be considered.

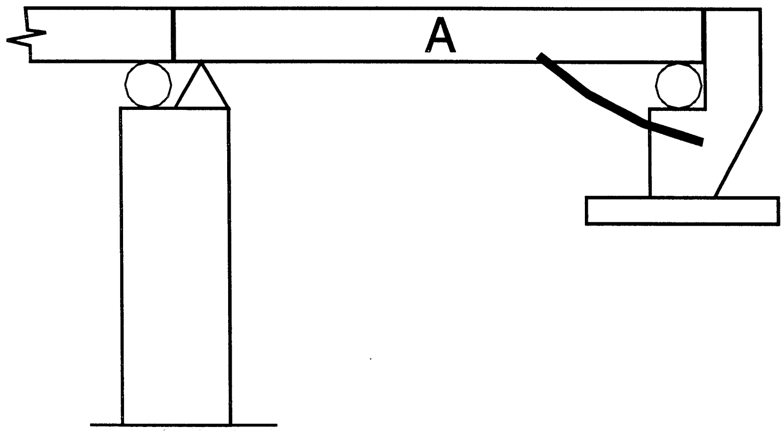


FIGURE 3-6 Typical Abutment with Roller Support

The displacement of girder A may be determined by considering the SDOF model in figure 3-6. The maximum response of the system is determined and compared with the available seat width. In this system abutments are neglected because the most critical displacement is that which corresponds to girder A moving away from the abutment.

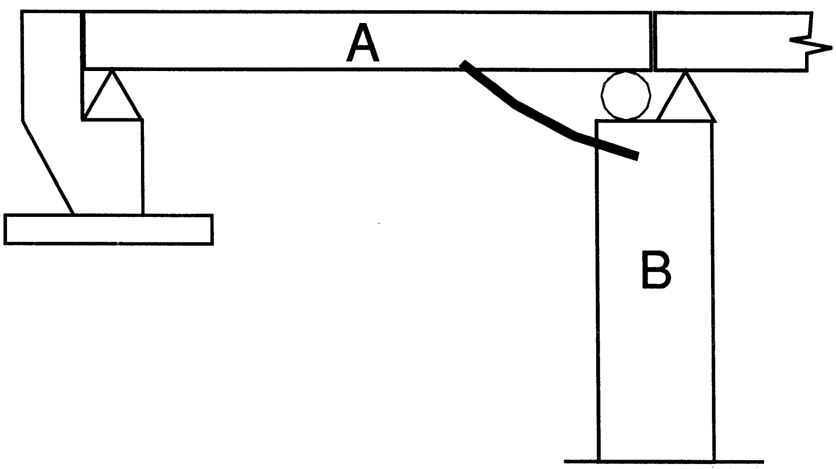


FIGURE 3-7 Typical Roller Supported Endspan

Consider a roller supporting an end span at a bent (figure 3-7). The objective is to determine the relative displacement between girder A and column B. The displacement of girder A is assumed to be zero to simplify the design. The total displacement of the frame containing column B must be calculated. The total response of the frame is determined using the SDOF system shown in figure 3-5 and methods shown in table 3-1, sections 1.1 and 1.2.

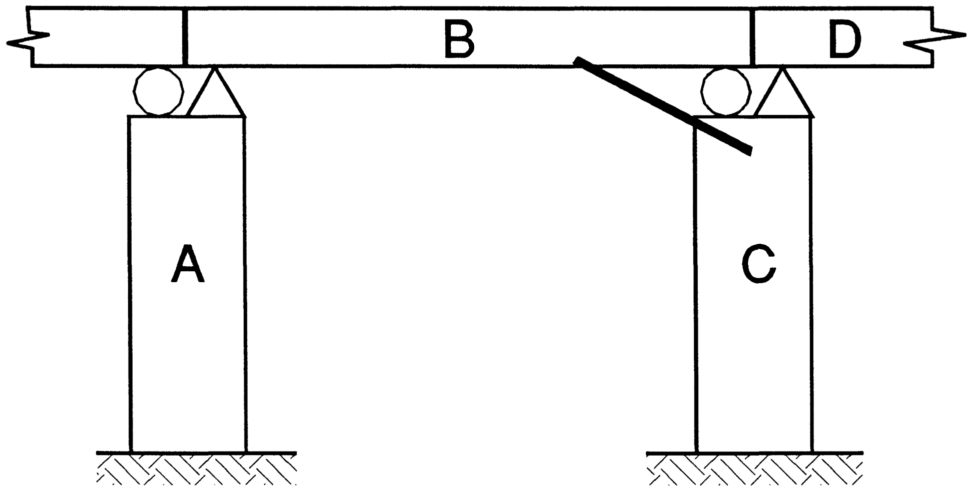


FIGURE 3-8 Typical Roller Supported Interior Span

The displacement of column B can be determined by dividing the earthquake force by the inverse of the sum of the column and footing flexibilities. The earthquake force can be determined from the chord stiffness including footing, column, and bearing elements. The displacement of column B is compared with the available seat width to determine if restrainers are needed. If restrainers are needed, the equivalent stiffness may be found by combining column, footing, and restrainer stiffnesses as shown in table 3-1, line 4.2.1.1. The response is determined with this stiffness, and stresses and displacements are checked. The number of restrainers are adjusted and the system reanalyzed until stress and seat width requirements are satisfied.

Consider a seat on the roller side of column C in figure 3-8. This roller supports an interior span. The objective is to find the relative displacement between the column C and the girder B. Unrestrained displacement of the two frames AB and CD is found using the SDOF model shown in figure 3-5 and equations in table 3-1, section 1.1. with zero restrainer stiffness. The earthquake force applied to frame CD and AB must be found considering bearing non-linearity. Once the force is determined, it is applied to only the column and footing of CD to determine the displacement at the top of the column. Once the displacement of column C and girder B are determined, they are combined using the Complete Quadratic Combination (CQC) method (Clough and Penzien, 1993). Using CQC the maximum total response of the 2-DOF system (girder B and column C) is:

$$R = \sqrt{R_1^2 + 2 * \rho_{12} * R_1 * R_2 + R_2^2} \qquad (3\text{-}2)$$

where:

R_1 = maximum modal response of girder B.

R_2 = maximum modal response of girder C.

$$\rho_{12} = \frac{8\xi^2(1+r)r^{3/2}}{(1-r^2)^2 + 4\xi^2 r(1+r)^2}$$

ξ = damping ratio

r = ratio of the smaller to the larger vibration frequencies.

The maximum relative response found with CQC can then be compared with the available seat width. If the unrestrained displacement is greater than the available seat width then restrainers are needed; if not, restrainers are not needed. When determining the number of restrainers, only frame C is considered, The SDOF system in figure 3-5 and the equation found in table 3-1, section 4.2.1.1. is used to determine response.

TABLE 3-1 Proposed Restrainer Design Procedure

Procedure for Restrainer Design for spans supported with PINNED Bearings

1 Determine Bridge Properties

1.1 Calculate bearing stiffness (k_b) & ultimate strength ($F_{b,capacity}$)

1.2 Calculate Column Properties

1.2.1 $k_c = \Sigma 3EI_{cr}/L^3$

1.2.2 $I_{cr} = 0.5 * Ig$

1.3 Calculate Footing Stiffness Properties (k_f)

1.4 For end-spans supported by bearings, calculate abutment stiffness (k_a) and yield force (F_{ua}).

1.5 Determine Superstructure(W_s), Column(W_{col}), and Bent Cap(W_{bc}) Weights.

1.6 Location Definitions:

1.6.1 End-Spans: Spans supported by a column and abutment, considering the relative displacement and restrainer at the column.

1.6.2 Interior-Spans: Spans supported by columns at both ends

1.6.3 Abutments: Spans supported by a column and abutment, considering the relative displacement and restrainer at the abutment.

1.7 Available seat width = ASW = Total Seat Width - 102 mm (4").

2 Determine Force on Bearing

2.1 Determine Initial Total Substructure Stiffness

2.1.1 $1/k_t = 1/k_f + 1/k_c + 1/k_b$ ... mid-spans and end spans.

2.1.2 k_b ... at abutments.

2.2 Determine structure period based on k_t and tributary mass ..... $T=2\pi(m/k)^{1/2}$

2.3 Determine ARS (Acceleration Response Spectra) Coefficient

2.4 Bearing Force

2.4.1 End-spans: $F_{b,\ initial} = ARS * W_s$

2.4.2 Mid-spans: $F_{b,\ demand} = ARS * W_s$

2.4.3 At abutments: $F_{b,\ demand} = A_{ground} * W_s$

2.5 Modifications for End Spans:

2.5.1 If the following conditions are met, use minimum number of restrainers. If the following conditions aren't met continue with the more rigorous analysis to determine the force on the bearing.

2.5.1.1 $F_{ua}/W_s > 2.0$

2.5.1.2 $\Delta_{ua} + \Delta_{gap} < ASW$

TABLE 3-1 Proposed Restrainer Design Procedure (continued)

2.5.1.3 Δ_{ua} = displacement corresponding to yield force of abutment; Δ_{gap} = gap between superstructure and abutment.

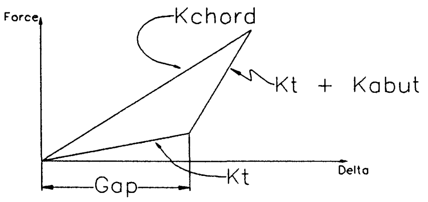
2.5.2 Before calculating period and ARS, determine Structure Displacement $(\Delta)_i$; $\Delta_i = (W_s+W_c+W_{bc})/k_t$

2.5.3 If $\Delta_i < \Delta_{gap}$, then $F_{b,\,initial} = F_{b,\,demand}$

2.5.4 If $\Delta_i > \Delta_{gap}$, then iterate to find chord stiffness (k_{chord}), based on $\Delta_{gap}<\Delta<\Delta_i$;

2.5.5 Determine Period based on k_{chord}.

2.5.6 k_{chord} will be based on structure state corresponding to the force applied. Include events such as gap closure (shown below), and abutment yielding.



2.5.7 Determine $F_{b,chord}$; ARS * $W_s = F_{b,\,chord} = F_{b,demand}$

3 **Determine if Restrainers are Needed**

3.1 Determine if bearings fail.

3.1.1 If: $F_{b,\,demand} < \phi F_{b,\,capacity}$ ..... Goto Step 3.2

3.1.2 If: $F_{b,\,demand} > \phi F_{b,\,capacity}$ ..... Goto step 5 and check displacements with available seat width.

3.1.3 $\phi = 0.7$

3.2 Seat-width (SW) requirements

3.2.1 If: SW < AASHTO RQMTS ... Goto step 5 and check displacements with available seat width.

3.2.2 If: SW > AASHTO RQMTS ... Use minimum number of restrainers.

4 **Design Restrainers**

4.1 Initial Design: $A_{restrainer} = F_{b,demand} / f_{ry}$; f_{ry} = 1213 MN/m^2 (176 ksi) (restrainer yield stress)

4.2 Minimum number of restrainers: $A_{r,min} = .35*W_s/f_{ry}$

5 **Check Displacements**

5.1 Assume slack = 0" (worst case)

5.2 Notes for determing whether or not restrainers are needed: When determining k_{chord}, for the first iteration, use Δ = ASW. If the force corresponding to Δ is greater than the effective weight times the ARS coefficient (found with k_{ch}), no restrainers are needed.

TABLE 3-1 Proposed Restrainer Design Procedure (continued)

5.3 Determine effective stiffness (k_e).

5.3.1 In mid-spans:

5.3.1.1 Initial Stiffness: $1/k_e = 1/k_c + 1/k_f + 1/(k_r + k_b)$

5.3.1.2 After bearing fails, k_e = chord stiffness.

5.3.1.3 $k_{b,fail} = k_b/1000$

5.3.2 In end span:

5.3.2.1 Initial Stiffness: $1/k_e = 1/k_c + 1/k_f + 1/(k_r + k_b)$

5.3.2.2 Include abutment stiffness as shown in 2.5.4.

5.3.2.3 Use chord stiffness when stiffness changes

5.3.3 At abutments:

5.3.3.1 Initial Stiffness: $k_e = k_r + k_b$

5.3.3.2 After bearing fails, k_e = chord stiffness

5.4 Determine maximum restrainer displacement (Δ_r) with ARS, effective span weight, and effective stiffness.

5.5 Check against available seat width (ASW) (for existing bridges) or AASHTO seat width requirements (for new bridges). ASW $< \Delta_{demand}$

6 **Check restrainer stresses**

6.1 $\sigma = F_r/A_r < f_{ry}$ ... OK

6.2 $\sigma = F_r/A_r > f_{ry}$ ... Add restrainers

7 **Reduce and increase number of restrainers as needed to satisfy displacement and stress requirements.**

__Procedure for Restrainer Design for spans supported by "Roller Bearings"__

1 **If no restrainers are required at the far end (which is pinned) and if minimum AASHTO support width at the roller support is met, or the total displacement (the current pier and the pier where the pinned end is) using CQC is less than the available seat width, no restrainers are required.**

1.1 For mid-spans:

1.1.1 At bent where pinned bearing is, consider bearing failure:

1.1.1.1 Initial Stiffness: $1/k_e = 1/k_c + 1/k_f + 1/k_b$

1.1.1.2 After bearing fails, k_e = chord stiffness.

1.1.1.3 $k_{b,fail} = k_b/1000$

1.1.1.4 $W_{total} = W_s + W_{col} + W_{bc}$

1.1.1.5 $\Delta_{pin} = (W_{total} * ARS)/ k_{chord}$

TABLE 3-1 Proposed Restrainer Design Procedure (continued)

1.1.2 At bent where roller bearing is:

1.1.2.1 When calculating ARS coefficient consider: k_c, k_f, k_b, and W_{total}

1.1.2.2 When calculating displacement consider: k_c, k_f, and W_{total}

1.1.2.3 $\Delta_{roller,col} = (W_{total}*ARS)/((k_c*k_f)/(k_c+k_f))$

1.1.3 Combine Δ_{pin} and $\Delta_{roller,col}$ using CQC.

1.2 For end-spans:

1.2.1 Determine force and displacement as in 1.1.2 and compare column displacement with ASW at bent.

1.3 For abutments:

1.3.1 At bent where pinned bearing is, consider bearing failure in force and displacement calculations:

1.3.1.1 Initial Stiffness: $1/k_e = 1/k_c + 1/k_f + 1/k_b$

1.3.1.2 After bearing fails, k_e = chord stiffness.

1.3.1.3 $k_{b,fail} = k_b/1000$, $W = W_{total}$

1.3.2 Compare calculated displacement with ASW at abutment.

2 Ignore any bearing friction. Initial restrainer design is based on $F_{demand} = W/2$.

3 Design Restrainers

3.1 $A_{restrainer} = F_{demand} / f_{ry}$; $f_{ry} = 1213$ MN/m^2 (176 ksi)

4 Check Displacements

4.1 Assume slack = 0” (worst case)

4.2 Determine effective stiffness.

4.2.1 In mid-spans & end spans:

4.2.1.1 Initial Stiffness: $1/k_e = 1/k_c + 1/k_f + 1/k_r$

4.2.2 At abutments: $k_e = k_r$

4.3 Determine maximum restrainer displacement (Δ_r) with ARS, effective span weight, and effective stiffness.

4.4 Check against available seat width (for existing bridges) or AASHTO seat width requirements (for new bridges). ASW < Δ_{demand}

5 Check restrainer stresses

5.1 $\sigma = F_r/A_r < f_{ry}$ … OK

5.2 $\sigma = F_r/A_r > f_{ry}$ … Add restrainers

6 Reduce and increase number of restrainers as needed to satisfy displacement and stress

3.5 Discussion of the Proposed Method

The main idea behind the Proposed method is that by including more of the bridge complexities, the prediction of the response will be more accurate. Bridge elements such as footings, columns, bearings, and abutments impact response significantly and should be included in some fashion when designing restrainers. Including these effects is not a simple task. Columns, footings, bearings, and abutments can be highly non-linear elements in measurable events. Opening and closing of expansion joints can also create non-linearities in the structure. All of these elements are associated with multiple degrees of freedom in many different configurations. AASHTO and Caltrans design spectra are associated with linear single-degree-of-freedom oscillators. The Proposed method attempts to account for the effects of all these bridge elements. The hypothesis that the Proposed method is built on is that simplifying the complete bridge system, which can be highly complex and non-linear, with a SDOF oscillator will predict the bridge response accurately and result in a reliable restrainer design.

The Proposed method makes the simplifying assumption that expansion bearings perform as frictionless rollers. In reality, friction forces are present when expansion bearings move (Mander, 1996), and these forces generally tend to reduce movements. However, friction forces depend on the condition of the bearing, its upkeep, presence of debris, accuracy of installation, etc. A minimum dependable friction force is yet to be established based on extensive research that would address these factors. It was therefore decided to neglect friction forces.

Span interaction is an important parameter that can affect bridge response significantly. The effects of span interaction are considered in a limited way in the Proposed method. Pounding of two adjacent spans has been evident in many of the major earthquakes of recent times (Yashinsky et al., 1995, and Saiidi et al., 1993). All of the other available design procedures neglect the impact between spans completely (Trochalakis et al., 1995). However, the integration of CQC for midspans accounts for some span interaction in the Proposed method. The implementation of this accounts for only two adjacent spans. In bridges with many spans, considering only two spans may or may not be sufficient.

Not many studies of bridge bearings subjected to seismic loads exist. The most recent experimental studies on bearings have been done for pseudo dynamic conditions (Mander et al., 1996). Bearing details are also widely varied and so are the responses to seismic loads. For the most part, prediction methods are acceptable but should be considered carefully. Once the bearings fail the relative response of the columns and girders is very difficult to predict and the response can be highly non-linear.

The Proposed procedure can take many iterations to find the number of restrainers in some cases. With non-linear analysis becoming more readily available, a complex hand procedure is less attractive; however, the Proposed procedure can be implemented on spreadsheets that can save a considerable amount of time by making iterations much faster.

As discussed in the previous chapter none of the existing restrainer design methods address bridge skew. The Proposed method does not address skew either. Skewed bridge decks have a tendency to rotate in plane (Maragakis, 1984) and could potentially increase the demand on restrainers. Skew is a parameter that needs to be included in future restrainer design procedures because it can lead to collapse (Yashinsky et al., 1995).

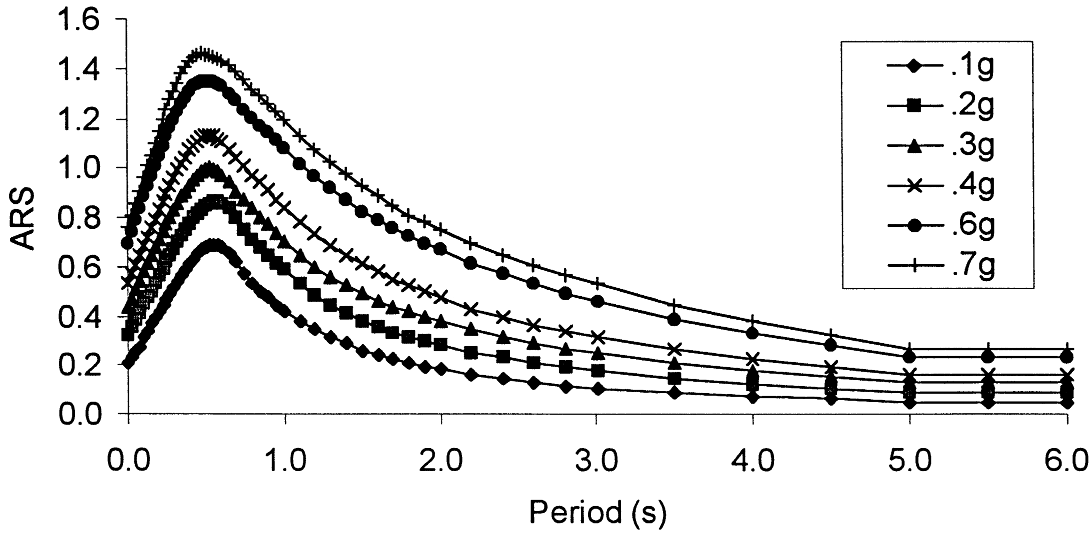


FIGURE 3-9 Caltrans ARS Spectra for 24 to 46m of Alluvium

The Proposed and Caltrans design procedures can be highly dependent on the spectra they are used with. The Caltrans design spectra that is representative of 24 to 46 m (80 to 150 feet) of alluvium is shown in figure 3-9. The Caltrans and AASHTO design spectra are similar in that they both have high amplitudes corresponding to shorter periods and lower amplitudes as the period increases. One could conclude that stiff structures will usually need larger numbers of restrainers than more flexible structures. In reality, this may not be the case; however, flexible structures have far larger displacement demands than stiff structures that may not be represented when using such spectra. Generally, the unrestrained displacement of a flexible structure will be larger than that of a stiff structure.

3.6 The Modified Caltrans Method

Similar to the Caltrans, Proposed, and AASHTO methods, the objective of the Modified Caltrans method is to limit longitudinal movement of adjacent bridge elements so that superstructure unseating does not occur during a strong earthquake. The Modified Caltrans procedure was formulated after completing much of this study, and is a hybrid of the Caltrans and Proposed methods, taking the best components of each.

The general procedure for the Modified Caltrans method is as follows:

1. Determine the adequacy of the connection between the span and superstructure and determine the unrestrained displacements of the structure.
2. If the connection is adequate and the unrestrained displacements are less than the available seat width, use the minimum number of restrainers; if not design restrainers using the Caltrans method.

At the connections between the span and superstructure, devices such as steel and elastomeric bearings are used. To evaluate the adequacy of these connections and the unrestrained displacements, one can use the same methods as used in the proposed procedure. If it is determined that restrainers are needed, one assumes that the bearings are nonexistant, and the structural stiffness comes from the restrainers only. This is the same assumption that Caltrans makes. If restrainers are needed, their number is determined using the Caltrans method.

3.7 Discussion of the Modified Caltrans Method

In simple span bridges whether or not restrainers are needed is determined mostly based on the adequacy of the bearings. Bearings are typically stiff. If they perform in the elastic range during a seismic event, they can be counted on to keep the span seated. Once the bearings fail, the behavior of the span is hard to predict because of the non-linear behavior of the bearings and the closing of the expansion joints. Because of this unpredictability, the bearings are ignored in the Caltrans method, and it is assumed that only the restrainers provide horizontal load transfer between the girder the pier. This assumption was maintained in the Modified Caltrans method.

By adding restrainers, a connection will exist between the superstructure and bent. It would then be logical to include the effects of the substructure. However, including substructure effects will usually decrease the total frame stiffness, and move the structure into a less critical location on the design spectra. Therefore, neglecting the substructure stiffness may be a better solution.

SECTION 4
ANALYTICAL MODEL

4.1 Introduction

In the past, analytical finite element models have been used by researchers to study restrainers (Saiidi et al., 1993, Trochalakis et al., 1995, and Fenves and Desroches, 1994). Analytical models have been proven to predict bridge response accurately (Fenves and Desroches, 1994, Maragakis et al. 1992, and O'Connor et al., 1993). This study used such models. Analytical models enable researchers to study complex bridge systems by studying how specific elements affect bridge response with a personal computer.

Many non-linear finite element models were analyzed in this study. The dynamic loading was in the form of acceleration records from past earthquakes. Models were formulated so that they would represent bridge response accurately. Conclusions and recommendations were drawn from analysis of the calculated responses. This chapter describes the formulation and rationale of the analytical models.

4.2 Computer Software

In this study, analytical finite element models were formulated and analyzed with the personal computer software DRAIN 3DX. DRAIN is a non-linear finite element structural analysis program. It features elements such as an inelastic truss element, an elastic beam column element, two different fiber beam column elements, inelastic connections, and an inelastic compression/tension link element (Prakash et al., 1993).

DRAIN can be used to perform many types of analyses with many different types of loading. The program can calculate mode shapes, perform spectral analysis, and perform response history analysis. Response history analysis was used in this study. One can specify loads in many different forms. For example, dynamic loads can be ground acceleration or displacement records, nodal force records, nodal initial velocity patterns, or response spectra. Ground acceleration records were used for loading of structures in this study.

Response history analysis can be performed at any time step. To determine a sufficient time step, a test structure was analyzed at time steps starting at 0.2 seconds and reduced until solutions for consecutive time steps converged. The time step used for this study was 0.02 seconds.

In DRAIN, the damping matrix is found using Rayleigh's method (Clough and Penzien, 1993) with a mass proportional damping factor and a stiffness proportional damping factor. These damping factors are constant throughout the entire analysis, and were found for this study using the following equations:

$$\alpha = \frac{2 * \omega_1 * \omega_2 * (\xi * \omega_1 - \xi * \omega_2)}{\omega_1^2 - \omega_2^2} \tag{4-1}$$

$$\beta = \frac{2*(\xi*\omega_1 - \xi*\omega_2)}{\omega_1^2 - \omega_2^2} \tag{4-2}$$

where:

α = mass proportional damping factor

β = stiffness proportional damping factor

ω_1 = circular frequency of structure corresponding to mode with the greatest contribution to response.

ω_2 = circular frequency of structure corresponding to mode with the second greatest contribution to response.

ξ = damping ratio

Constant stiffness proportional damping factors pose a problem for non-linear analysis. Damping is based on initial stiffness. Therefore, a very stiff element will have high damping forces. These forces will be unrealistically high after yield. In bearing elements, high damping forces could impact the bridge response significantly and may not represent the real behavior. To alleviate this problem, stiffness proportional damping was not considered in bearing elements. This assumption ensures a more critical response in terms of span unseating. In other elements, stiffness proportional damping was used.

Three element types were used in this study. The first is the elastic beam column element. It is a simple linear beam column element. This element requires input of material and cross-sectional properties, stiffness factor sets to account for non-prismatic elements, and rigid end zones.

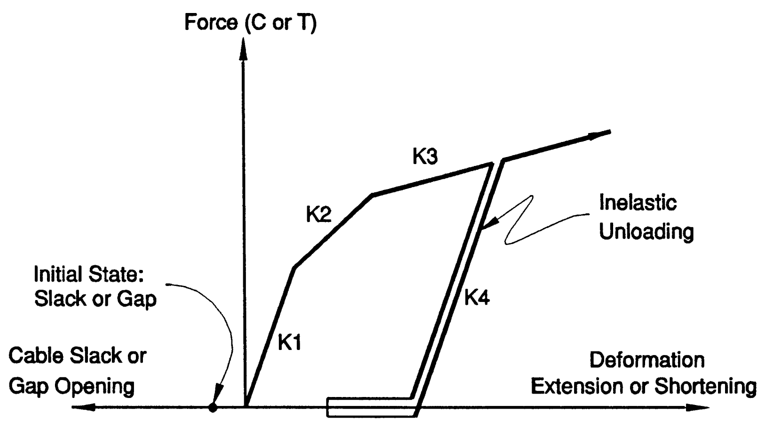


FIGURE 4-1 Tension/Compression Link Element Behavior

The next element used was the compression tension link element. This element is an inelastic truss element that resists only axial force either in tension or compression. The force displacement behavior of the element is shown in figure 4-1. The convenient aspect of this element is that it functions as either tension only or compression only; this enables analysts to model bridge elements that act in either tension or compression exclusively such as cable restrainers and expansion joints.

The third element used was the simple connection element. This is an inelastic element that models structural connections with rotational or translational flexibility. This element has zero length; it connects two nodes that have the same coordinates. The element has the capacity to connect one degree of freedom at a time, so to connect all six degrees-of-freedom of two nodes one must use six elements. The force-deformation behavior of the connection element is represented by a bilinear hysteresis model (figure 4-2).

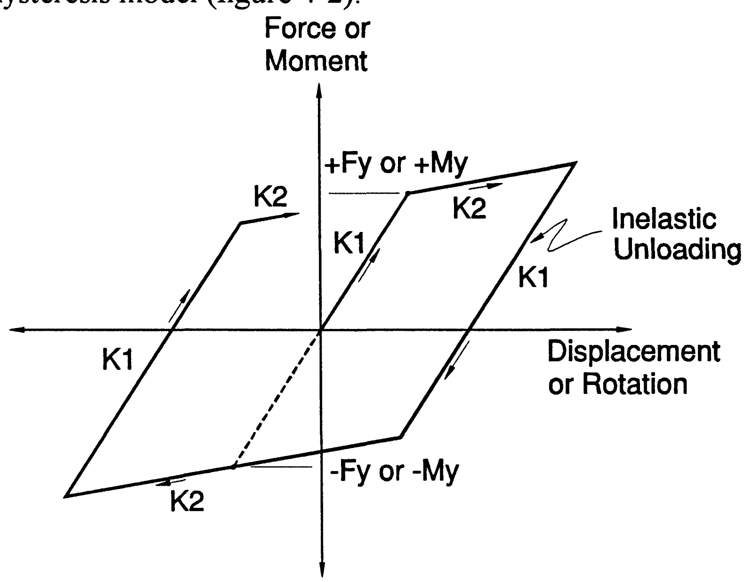


FIGURE 4-2 Simple Connection Element Behavior

4.3 The Analytical Models

The purpose of formulating an idealized model was to capture essential characteristics of a real bridge with a relatively simple structural model. The simplicity allows one to study and vary parameters with ease. All bridge elements such as columns, footings, bearings, abutments, restrainers, and expansion joints were included. It was also critical to capture the non-linear aspects of a bridge such as abutment gaps opening and closing, abutment yielding, bearing failure, and opening and closing of expansion joints. Column plastic hinging was not explicitly included in this model. Rather, its effect in reducing the effective stiffness of the substructure was implicitly accounted for by studying bridge models with a wide range of substructure stiffness.

The number of spans can affect the bridge response. This study included two-span bridges and five-span bridges. Two-span bridges are common. They also present the case with the least number of spans that unseating from pier would be a consideration. Past earthquakes have shown that single-span bridges are less susceptible to support loss (Saiidi, 1995). A five-span bridge configuration was chosen to represent multi-span, yet short, bridges. Bridges with three spans or more have interior spans that are not adjacent to abutments. This is an important difference between the bridge types. Abutments have a significant effect on the response of end spans. However, their effect is less pronounced in interior spans.

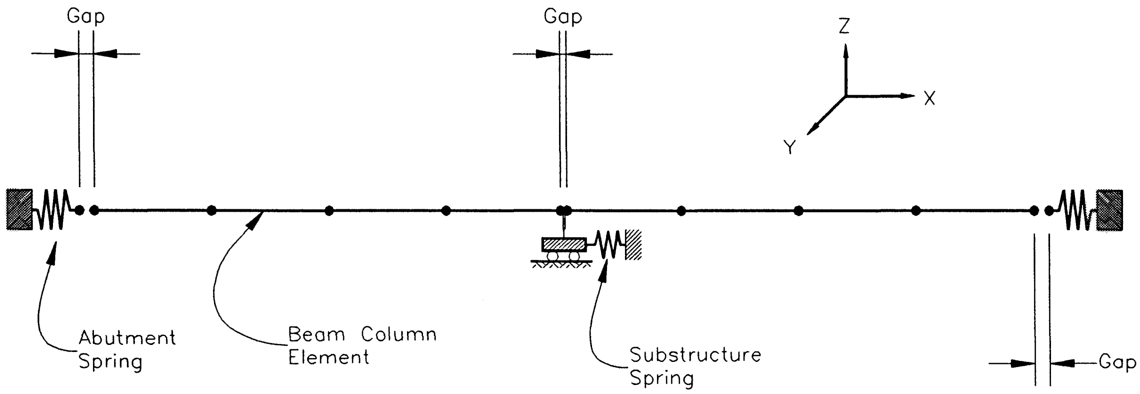


FIGURE 4-3 Two-Span Idealized Model

Figure 4-3 shows the two-span idealized model. The superstructure is represented by four elastic beam-column elements per span. Each abutment is represented by a compression link element, and the bent is modeled with an elastic simple connection element.

The detail of expansion joints used in this research is shown in figure 4-4. Two spans are supported at each bent; one on a pinned bearing and the other on an expansion or roller bearing. Expansion bearings are designed to allow free longitudinal movement. While free longitudinal movement is desirable to relieve the stresses due to thermal expansion of girders, it can lead to unseating of the superstructure during an earthquake. However, studies have shown that expansion bearings provide some force transfer between the superstructure and bent. The hysteretic behavior of expansion bearings depends on Coulomb friction, but the force and stiffness characteristics can vary greatly depending on the field conditions (Mander et al., 1996). To ignore the longitudinal stiffness and strength of expansion bearings is a conservative assumption because friction forces between the superstructure and bearing can help the bridge remain seated. For these reasons, the analytical model of the expansion bearing was constructed to be a roller. With the exception of restrainers, no longitudinal connection exists in the model between the bent cap and superstructure at expansion bearing locations.

On the side of the bent cap opposite the roller bearing, a bearing element is used to model the pinned bearing. This element has zero length, and is connected with nodes that are slaved to master nodes that represent the end of the superstructure and the top of the bent cap. To model the impact between adjacent spans, a compression link element (gap element) is connected to nodes that represent the ends of each span. Restrainers connect the top of the bent cap and the node representing the bottom of the girder that is slaved to the end of the span. The substructure is represented by a spring element that is connected to a fixed node and the node representing the top of the bent cap.

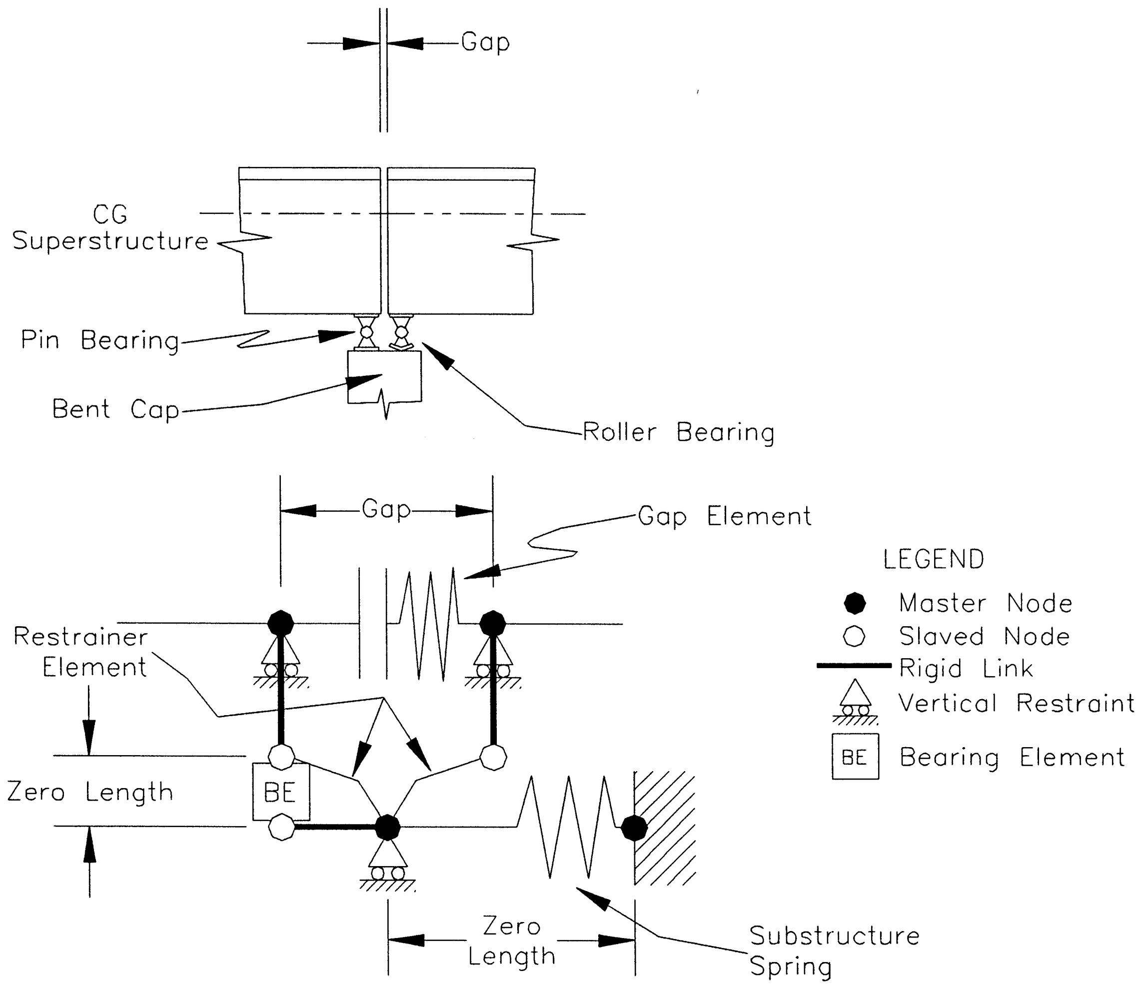


FIGURE 4-4 Expansion Joint Detail

The abutment is represented by a compression link element and is connected to a fixed node and a node representing the end of the span (figure 4-5). Spans supported by abutments can be connected to the abutment with either a roller bearing or a pinned bearing. The model detail of a pinned abutment is shown in figure 4-5. For a pinned abutment, a connection element represents the bearing and is connected to the node representing the end of the superstructure and a fixed node. A roller supported span at an abutment has no connection element, and the node representing the end of the span is restrained in the vertical direction only. Restrainers connected the fixed abutment node to the node representing the end of the superstructure.

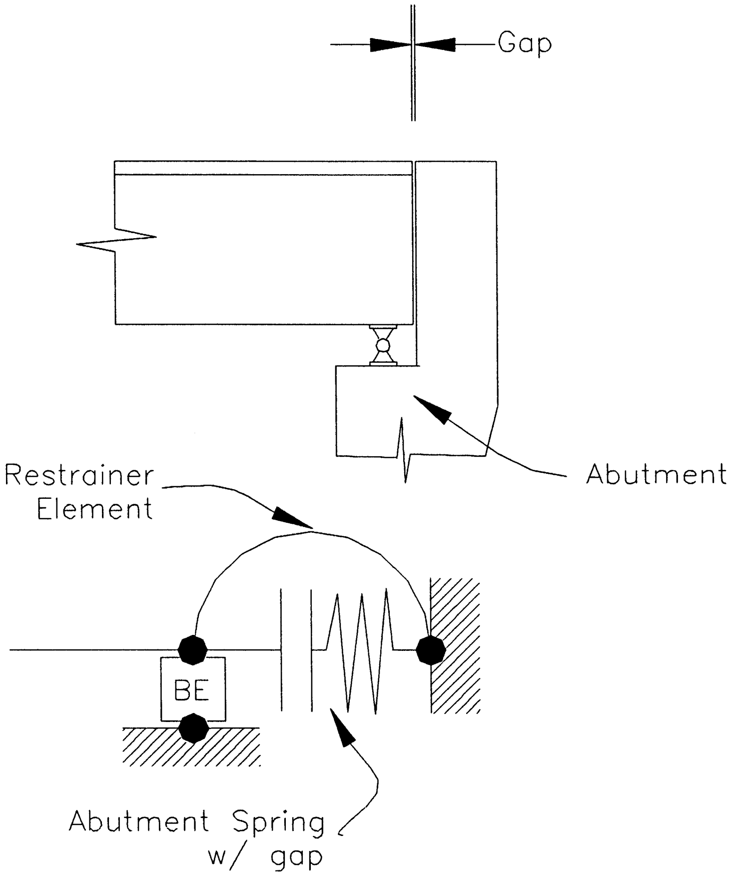


FIGURE 4-5 Abutment Detail

The model of the five-span bridge is shown in figure 4-6. The only difference between the two-span bridge and five-span bridge is the number of spans. The same expansion joint and abutment details were used in the five-span bridge.

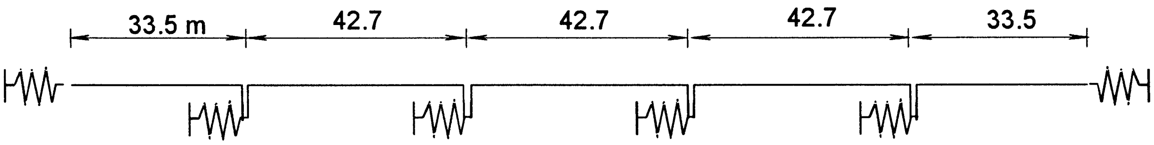


FIGURE 4-6 Five-Span Idealized Model

To study the effects of skew, changes were made to the five-span model. The skewed version of the five-span bridge model is shown in figure 4-7. At the ends of each span very stiff elements were added between nodes representing the skewed geometry of the span. Each abutment and expansion joint was represented by two springs acting perpendicular to the skew and placed on the edges of the deck.

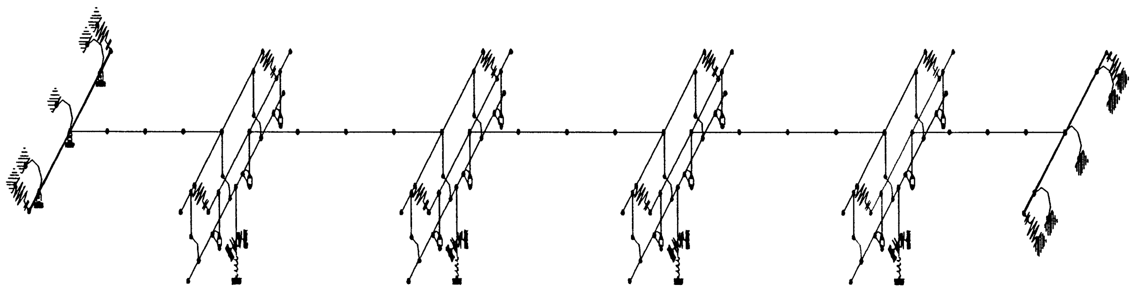


FIGURE 4-7 Skewed, Five-Span Idealized Model

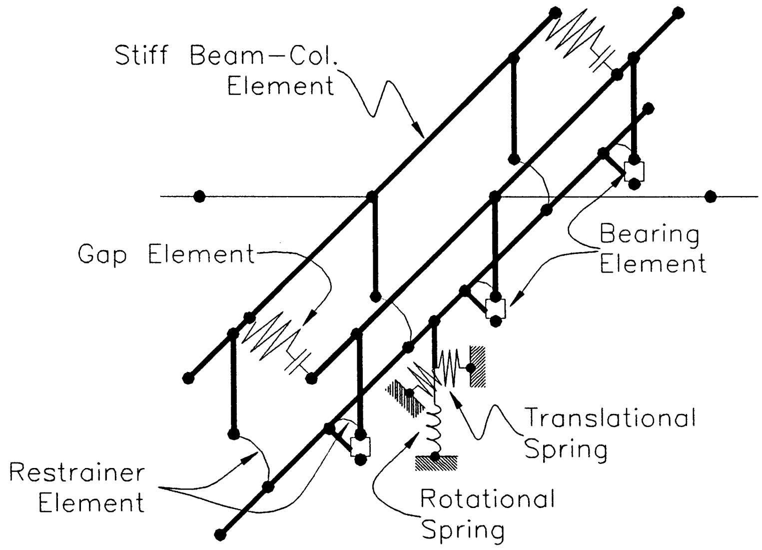


FIGURE 4-8 Bent Detail of the Skewed, Five-Span Idealized Model

Three simple connection elements were used to represent the pinned bearings, and three tension links were used to represent restrainers (figure 4-8). The links were oriented perpendicular to the skew. One rotational, and two translational connection elements were used to represent the substructure. The translational springs were oriented in the longitudinal and transverse directions of the bridge.

4.4 Restrainer Properties

Cable restrainer properties were found from ASTM bar and tension tests (figure 4-9) and from Caltrans (Caltrans, 1994). The stress strain behavior found in figure 4-9 was idealized as bi-linear. Tension link elements were used to model restrainers. The modeling of their behavior was described in section 4.2.

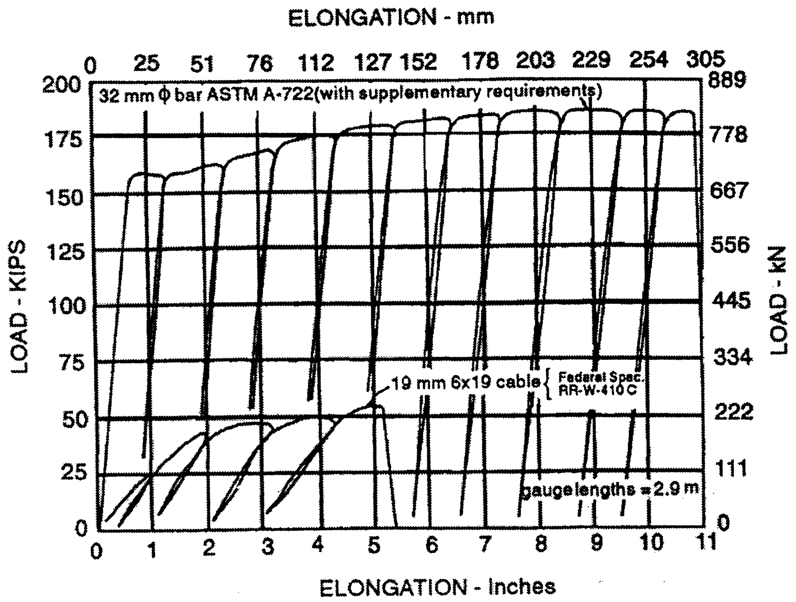


FIGURE 4-9 ASTM Cable and Bar Tension Test (Caltrans, 1994)

The initial stiffness of one restrainer is:

$$k = \frac{E^* A}{L} \tag{4-3}$$

where:

k = Stiffness of one cable restrainer in tension.
E = Young's Modulus for a cable, 68,950 MPa (10,000 ksi).
A = Area of one cable, 143 mm^2 (0.222 in^2).
L = Length of the restrainer.

The total cable stiffness for the non-skewed two- and five-span bridges was found by multiplying the stiffness of one cable by the number of restrainers. The first yield displacement was found using:

$$\Delta_1 = L^* \left(\frac{\sigma}{E}\right) \tag{4-4}$$

where:

Δ_1 = Restrainer displacement at first yield (first break point in figure 4-1).
σ = Yield stress of a cable, 1214 MPa (176 ksi)

Figure 4-10 shows the non-linear restrainer behavior used in this study. The post-yield stiffness of the restrainer was assumed to be one percent of the initial stiffness. To simplify the model, the displacement at the second break point in figure 4-1 was set to be very large so that the third loading stiffness was never reached. Restrainers unloaded inelastically with the same stiffness as the initial stiffness (figure 4-10). Stiffness proportional damping was used for the restrainer elements.

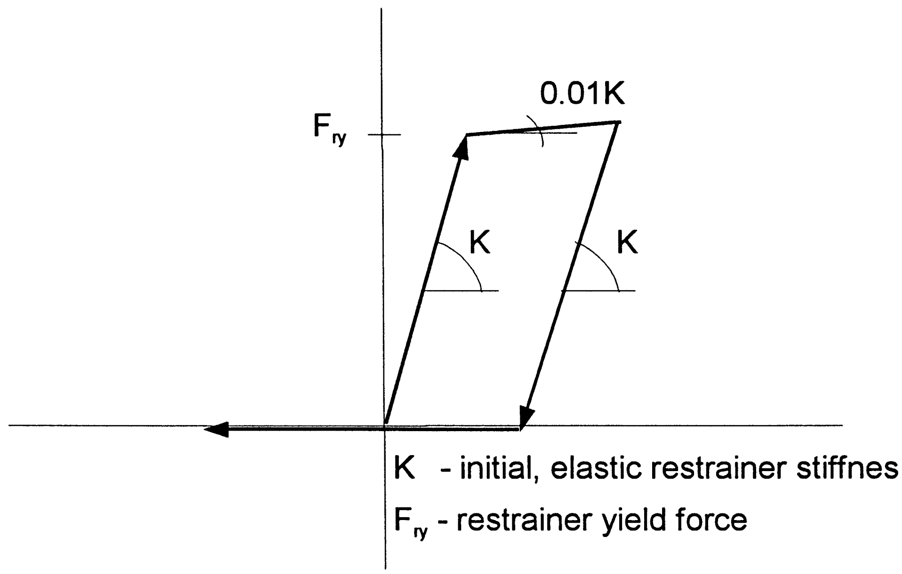


FIGURE 4-10 Non-Linear Restrainer Behavior

In cases of skewed bridges, restrainers at each bent (or abutment) were represented by three different elements. The middle element represented one-half of the total number of restrainers, and the outer elements represented one-quarter of the total number of restrainers.

4.5 Gap Properties

Gap properties are those that represented the opening and closing of expansion joints that correspond to the longitudinal movement of the bridge. Compression link elements were used to model gaps. The elements were elastic with a stiffness of:

$$k = 2 * \frac{E * A}{L} \tag{4-5}$$

where:

k = Gap element stiffness.
E = Young's Modulus for superstructure.
A = Cross sectional area of superstructure.
L = Span length.

The compression link elements were given a gap equal to the distance between the two spans at the expansion joint. The distance was equal to 25 mm (1 in.). The axial stiffness of the span (EA/L) was multiplied by two because the stiffness of both spans would be engaged when the gap was closed. Note that the compression link stiffness has to be a relatively large value to prevent node penetration. The axial span stiffness provided this large value.

Stiffness proportional damping was included with the gap elements described as in section 4.2.

In skewed models, gap properties were represented by two compression link elements located at outer edges of the deck. The elements were oriented perpendicular to the skew since the impact force acts normal to the impact surface. Each element had a stiffness equal to one-half of the stiffness found from equation 4.4.

4.6 Abutment Properties

Abutment properties were determined from equations found in Caltrans Memos to Designers (Caltrans, 1994). Bi-linear compression link elements were used to model abutments.

Abutment stiffness and strength were determined from the following equations which have shown agreement with field test results (Maroney et al., 1994):

$$k = (114.9 * B) * \frac{H}{2.44} \tag{4-6}$$

where:
k = Abutment stiffness (kN/mm)
B = Abutment width in meters
H = Abutment height in meters

$$F_y = (369 * B * H) * \frac{H}{2.44} \tag{4-7}$$

where:
F_y = Abutment yield force (kN)

The post-yield abutment stiffness was 0.1 percent of the initial stiffness. Compression link elements were given a gap equal to the distance between the abutment and girder. Stiffness proportional damping was included with this element.

In skew bridge cases, each abutment was represented by two compression link elements placed at the edges of the abutment. The elements were oriented perpendicular to the skew. Each element represented one half of the total stiffness and strength of the abutment.

4.7 Bent Properties

Bents were assumed to be elastic and were represented as a component of the spring elements that represented the total substructure. Stiffness proportional damping was included. Bent stiffness was calculated using the following equations:

$$k = \frac{3 * E * I}{L^3} + \frac{G * A_S}{L} \tag{4-8}$$

where:
k = Stiffness of one column

E = Young's modulus
I = Moment of inertia
L = Length of column
G = Shear modulus
A_S = Shear area

Assuming a rigid bent cap the total bent stiffness can be calculated using the following equation:

$$K = k^* n \tag{4-9}$$

where:
K = Total bent stiffness
n = Number of columns in the bent

The total substructure stiffness was found from the following equation:

$$K_{SUB} = \frac{1}{1/K + 1/K_F} \tag{4-10}$$

where:
K_{SUB}= Total substructure stiffness
K_F = The footing stiffness

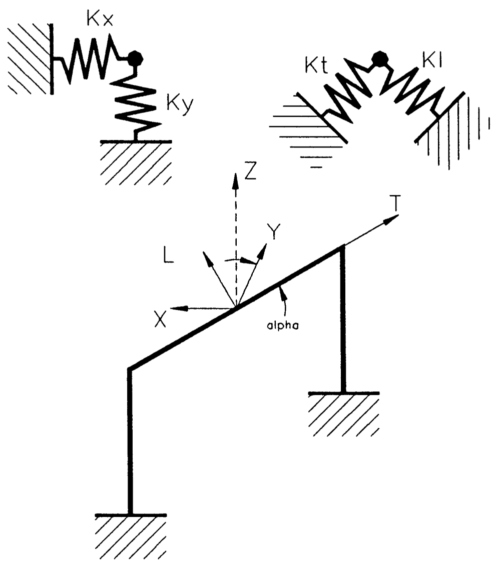


FIGURE 4-11 Skewed Bent Spring Transformation

In skewed bridges, bents were aligned with the skew as shown in figure 4-11. Substructure stiffnesses, Kt and Kl, were found parallel and perpendicular to the skew, respectively. The stiffnesses were then transformed to the global X and Y coordinates. The transformed stiffness were called Kx and Ky and were used as the bent stiffnesses for the model.

Kx and Ky were found from the following equations:

$$Kx = Kt^* \sin^2 \alpha + Kl^* \cos^2 \alpha \quad (4\text{-}11)$$

$$Ky = Kt^* \cos^2 \alpha + Kl^* \sin^2 \alpha \quad (4\text{-}12)$$

where:

Kx = Bent stiffness in the global X (longitudinal) direction.
Ky = Bent stiffness in the global Y (transverse) direction.
Kt = Bent stiffness in the local t (parallel to skew) direction.
Kl = Bent stiffness in the local l (perpendicular to skew) direction.
α = Skew angle

The torsional stiffness of the bents about the z-axis (figure 4-11) in skewed cases was calculated and input into models with rotational spring elements (figure 4-8). The total torsional bent stiffness was calculated as:

$$K\phi = K_C + K_M \quad (4\text{-}13)$$

where:

$K\phi$ = Torsional stiffness of bent.
K_C = Sum of column torsional stiffnesses about their axes.
K_M = Stiffness component due to the eccentricity of column axes.

The torsional stiffness of each column is:

$$K_C = \frac{G^* J}{L} \quad (4\text{-}14)$$

where:

G = Shear modulus
J = Polar moment of inertia
L = Length of column

By definition the rotational stiffness of a bent is the moment due to a unit rotation applied to the bent cap. One may assume the bent cap is rigid. From summing moments about the center of gravity of the bent cap, one finds that:

$$K_M = \frac{K_T^* L_b^2}{2} \quad (4\text{-}15)$$

where:

K_T = Inverse of the sum of column and footing translational flexibilities in the longitudinal direction of the bridge.
L_b = Length of bent cap.

4.8 Footing Properties

Footing stiffnesses were determined and combined with column stiffnesses as in section 4.7 to form substructure stiffness. Footing translational and rotational stiffnesses were calculated based on equations found in the Federal Highway Administration's (FHWA) Seismic Design of Highway

Bridge Foundations, Volume II (FHWA, 1986). Footing rotational flexibilities about the Z-axis in figure 4-11 were ignored.

FHWA defines the following equations to determine stiffness for a rigid embedded spread footing:

$$K = \alpha * \beta * K_O \tag{4-16}$$

where:

K = Corrected footing stiffness.
K_O = Equivalent footing stiffness.
α = Foundation shape correction factor.
β = Foundation embedment factor.

Horizontal translational stiffness can be taken as:

$$K_{OH} = \frac{8 * G * R}{2 - \nu} \tag{4-17}$$

where:

K_{OH} = Equivalent horizontal translational stiffness.
G = Shear modulus of soil.
ν = Poisson's ratio of soil.
R = Equivalent radius.

Rocking rotational stiffness can be taken as:

$$K_{OR} = \frac{8 * G * R^3}{3 * (1 - \nu)} \tag{4-18}$$

where:

K_{OR} = Equivalent rocking rotational stiffness.

Due to highly non-linear characteristics of soil the soil shear modulus, G, was selected to account for non-linear stiffnesses. Shear modulus is related to shear strain in curves by Hardin and Drnevich for sand as shown in figure 4-12, and for clay as shown in figure 4-13 (Hardin and Drnevich, 1972). Different shear strengths, S_u, for clay were chosen to reflect stiff or soft clay conditions (Carter and Bentley,1991), and different maximum shear modulus, G_{max}, were chosen to reflect loose or dense sand conditions. For constant S_u or G_{max}, one can produce curves that relate shear modulus to shear strain. With such curves and the FHWA equations one can formulate force displacement relationships for spread footings in different soil conditions from which stiffness can be evaluated.

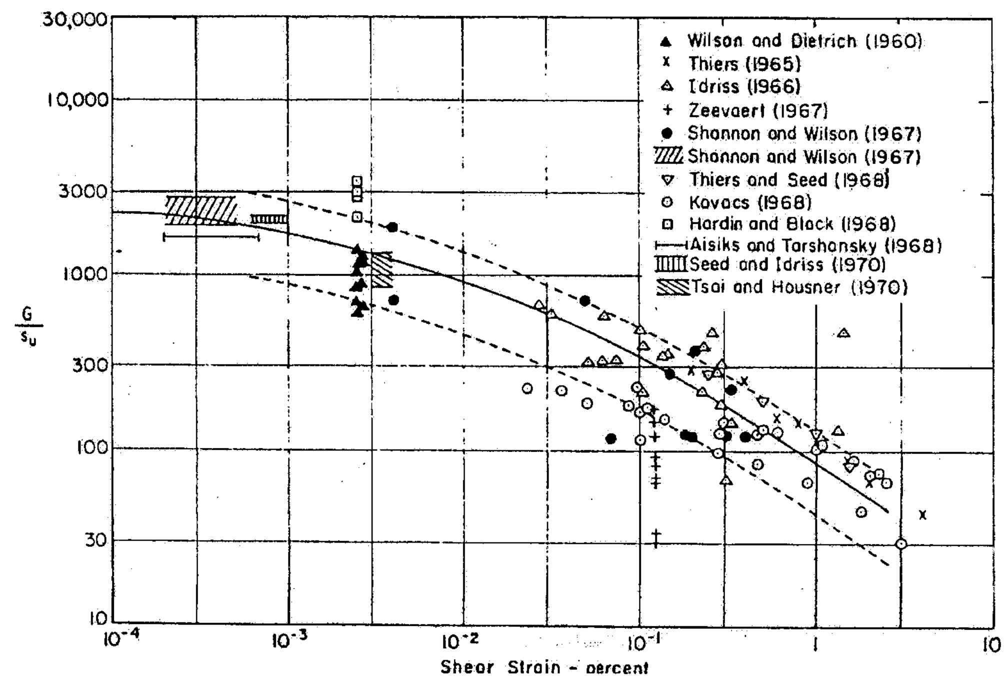


FIGURE 4-12 In-Situ Shear Modulus of Saturated Clays (Hardin and Drenvich, 1972)

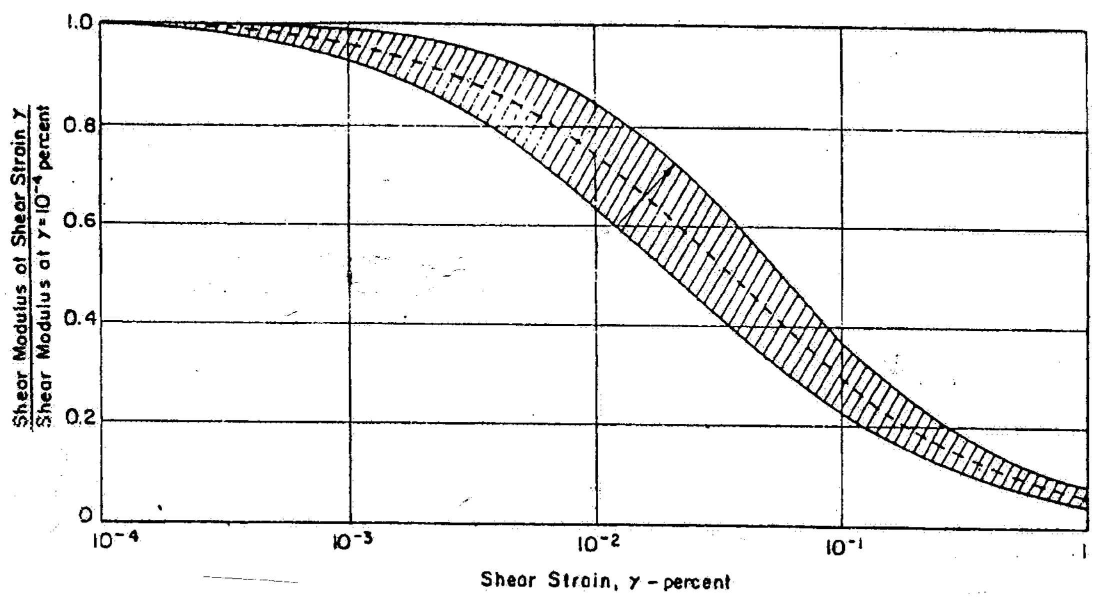


FIGURE 4-13 Variation of Shear Modulus with Shear Strain for Sands (Hardin and Drenvich, 1972)

The methods used to determine stiffness described above would not be valid for bridges at sites where soil conditions have the potential to liquefy.

4.9 Bearing Properties

Bi-linear connection elements represented the behavior of steel bearings in the longitudinal direction. Bridge superstructures were restrained in the vertical direction. For non-skewed cases no transverse bearing properties were needed. Bearing properties were determined from Mander et al., 1996. In that report, bearings obtained from actual bridges were tested under cyclic loading. Methods to determine stiffness and strength of bearings based on experimental data and failure modes found during laboratory testing were suggested. Elastic stiffness may be determined by assessing the shear and flexural flexibility of each of the bearings parts. Strength can be assessed from rigid body kinematics, upper bound mechanism analysis, or failure of anchor bolts , keeper plates, or shear keys, whichever controls.

For models analyzed in this study, it was assumed that the test results represented a range of stiffness and strength for typical steel bridge bearings. Table 4-1 lists the test results for pinned bearings (Mander et al., 1996).

TABLE 4-1 Summary of Pinned Bearing Stiffnesses (Mander et al., 1996)

Bearing Type	*Stiffness per Bearing* K_{long} (kN/mm)
Pinned Bearings	
HTF - Rte 400	193
HTF on Conc. Pedistals - Rte 400	179
LTF - Jewett-Holmwood	356
HTF - Jewett-Holmwood	380

HTF=High Type Fixed
LTF = Low Type Fixed

Strength values found for pinned bearings are summarized in table 4-2. Parameter "C" represents the ratio of horizontal strength to vertical reaction on the bearing (Mander et al., 1996).

TABLE 4-2 Summary of Pinned Bearing "C" Ratios (Mander et al., 1996)

Bearing Type	*Strength per Bearing* W (kN)	C
Pinned Bearings		
HTF - Rte 400	356	1.06
HTF on Conc. Pedistals - Rte 400	356	0.60
LTF - Jewett-Holmwood	180	3.00
HTF - Jewett-Holmwood	180	1.96

When stiffness proportional damping is specified in DRAIN, a linear viscous damping element is added in parallel with the simple connection element. The viscous element stiffness is the stiffness proportional damping coefficient multiplied by the initial stiffness of the element throughout the analysis (Prakash et al., 1993). Damping was neglected for simple connection elements that represented bearings because the initial stiffness of typical bridge bearings is very high. The high initial stiffness would overestimate damping forces once the element yields, and would lead to unrealistic damping.

In skewed bridges, bearing longitudinal strength and stiffness was represented by three simple connection elements at each pinned span location. The middle simple connection element represented bearings supporting the middle girders and was representative of one-half of the total stiffness and strength of the pinned span. The outside simple connections represented the bearings supporting the outside girders and were each representative of one-fourth of the total stiffness and strength of the pinned span. In the transverse direction, bearings were assumed to be elastic. This assumption represented transverse restraints that are ordinarily provided by shear keys. In most bridges, bearings are not aligned with the skew, so transverse and longitudinal stiffness and strength did not need to be transformed to a new coordinate system.

4.10 Earthquakes

The analytical models described above were analyzed for five earthquake acceleration records. The peak ground accelerations were scaled to either 0.7g or 1.0g. Time increments were not scaled from the accelerograms. Many earthquakes and earthquake records were considered as input. Five records were chosen:

1. Imperial Valley Earthquake, El Centro Record, North-South, 1940.
2. Loma Prieta earthquake, San Francisco Airport Record, North-South, 1989.
3. Northridge Earthquake, Sylmar County Hospital Parking Lot Record, North-South, 1994.
4. Kobe Earthquake, Kobe Station Record, East-West, 1995.
5. Artificial accelerogram which was generated based on the EC8/2 elastic spectra for medium soil and with initial acceleration record Tolmezzo North-South obtained in Italy in 1976.

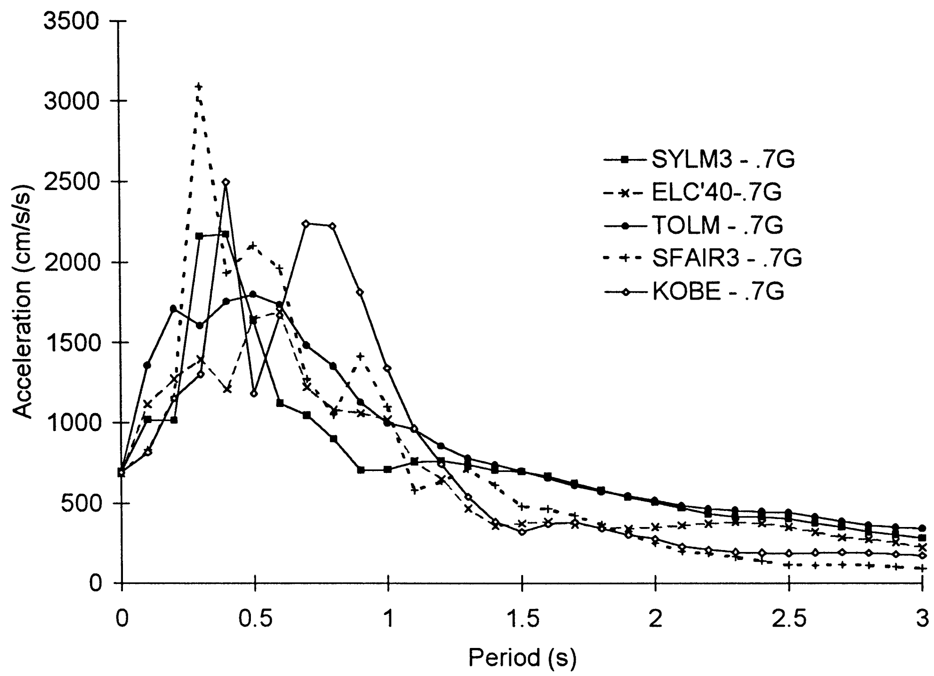


FIGURE 4-14 Single Degree of Freedom Elastic Spectra

The elastic acceleration spectra and accelerograms for the earthquakes used in this study are presented in figure 4-14 and 4-15 respectively. Earthquakes were chosen based on their amplitude, frequency content, and duration characteristics. Figure 4-14 shows that bridges with a wide range of period would be excited by the collective effect of the earthquake records.

In skewed cases, bridges were loaded in two directions, which is considered to be the most critical case (Bjornsson et al., 1997). Accelerograms used in the transverse direction corresponded to the record used in the longitudinal direction. For example, if the Kobe East-West record was used in the longitudinal direction, the Kobe North-South record was used in the transverse direction. The same scale factors were used for both directions.

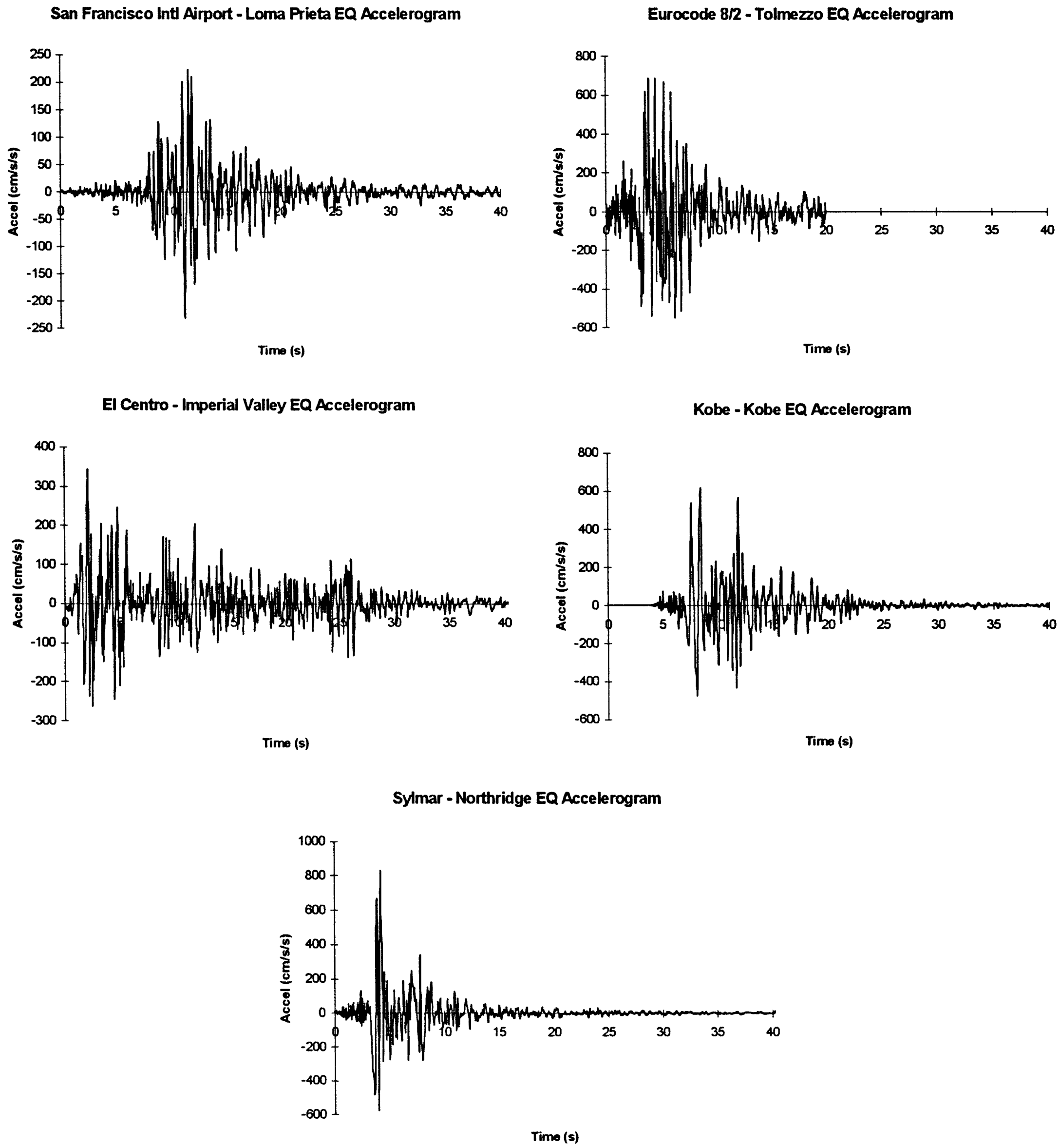


FIGURE 4-15 Earthquake Accelerograms

SECTION 5
DESCRIPTION OF BRIDGES

5.1 Introduction

The purpose of this section is to describe bridges that were studied in this research. Two-span and five span bridge configurations were studied to evaluate the effectiveness of different restrainer design methods. Some bridge parameters such as superstructure properties, span length, abutment stiffness, abutment strength, and bearing stiffness remained constant throughout the analyses; however, other parameters such as substructure stiffness, bearing strength, and restrainer design were varied. The values of all of the parameters were chosen to be realistic, although a wide range was selected for a thorough evaluation. This section describes the two- and five-span bridges and the parameters that were varied in the main and special cases.

5.2 Northridge Database

Recent seismic events, such as the 1989 Loma Prieta Earthquake, the 1994 Northridge Earthquake, and the 1995 Kobe Earthquake, have allowed researchers to investigate the effectiveness of current seismic retrofitting devices such as restrainers. In the past, researchers have compiled and tabulated summaries of the bridges subjected to such earthquakes (Saiidi et al., 1993 and Yashinsky et al., 1995). A table summarizing the performance of simply-supported bridges subjected to the Northridge Earthquake is shown in table 5-1.

TABLE 5-1 Simply Supported Bridges Inspected after the Northridge Earthquake

Name (Rte) / #, Yr.	Geometry	L (m)	Spans	Supports	Restrainer	PGA	Damage
Sun Valley OH (5) 53-1134, 1961	Skewed, Multi Col., Steel Girder	157.6	5	Simple Span	Cable	0.5g	None
LA River Bridge / Sep (5) 53-1075 R/L, 1957	Curved, Skewed, Solid Piers	194.5	5	Simple Span	Cable	0.3g	None
Providencia OH (5) 53-1085 R/L 1957	Straight Skewed	222.5	8	Simple Span	Cable	0.3g	None
Evergreen Ave POC (10) 53-0791, 1952	Straight	115.2	4	Simple Span	Cable	0.2g	None
Elysian Viaduct (5) 53-1424, 1962	Curved, Steel and RCBG	755.9	7	Continuous & Simple	Cable	0.2g	Minor Spalls @ Soffit, Shear Key Damage
Santa Clara River (5) 53-0687L/R, 1964	-	225.9	-	Simple Span	Cable	.25g	Bearing, Abutment, and Restrainer Damage
Ventura Blvd UC (101) 53-1065, 1956	-	52	-	Simple Span	Cable	.25g	None
Mulholland Ave Dr. OC (101) 53-1067, 1957	-	81	-	Simple Span	Cable	.25g	None
Las Virgenes Rd. OC (101) 53-1442	-	69	-	Simple Span	Cable	.25g	Keeper Bolts Sheared, Abutment, Pile Damage
Whittier Blvd UC (60) 53 0075L/R/S, 1965	-	79	-	Simple Span	Cable	.25g	Shear Key Spalls
Los Angeles River (2) 53-0255, 1961	-	171.3	-	Simple Span	Cable	.25g	None
Alhambra Avenue OH (5) 53-0368, 1960	-	125.6	-	Simple Span	Cable	.25g	None

The response of simply supported bridges was of interest because these bridges were equipped with restrainers, whose performance could be reviewed. The data in the table were obtained from Yashinsky et al., 1995. Note that most of the simply-supported bridges experienced no damage. This was probably due to the low peak ground acceleration that most of the bridges experienced. Restrainer damage was reported in only one bridge (Santa Clara River). However, no further information on the damage was available. Also note that steel was the most common type of superstructure. Most of the structural properties of the models in this study were taken from the first four bridges in the database because they appeared to be representative of common bridges.

5.3 Two-Span Bridges

The first type of bridge studied was a two-span bridge. The typical section of this structure is shown in figure 5-1. The bridge type was chosen based on the most common simply supported type as found in table 5-1. The superstructure was assumed to be a steel plate girder with a reinforced concrete deck. Steel bearings were assumed to support the superstructure.

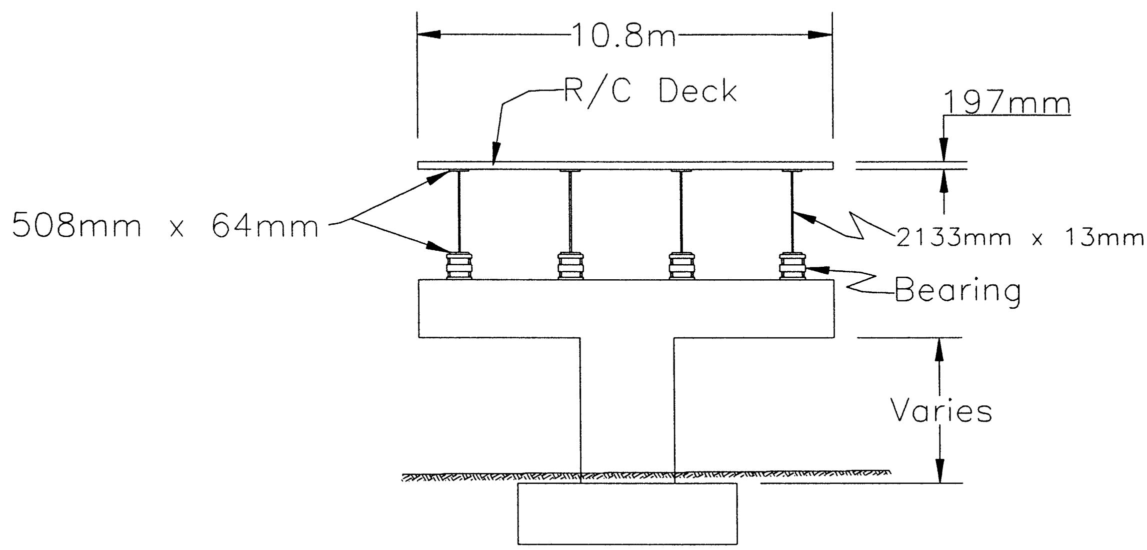


FIGURE 5-1 Two-Span Bridge Typical Section

The substructure consists of a reinforced concrete “dropped” bent cap (also known as a hammerhead) , a single rectangular reinforced concrete column, and reinforced concrete spread footings. The plan and elevation of the two-span bridge is shown in figure 5-2. Abutments were chosen to be reinforced concrete seat-type.

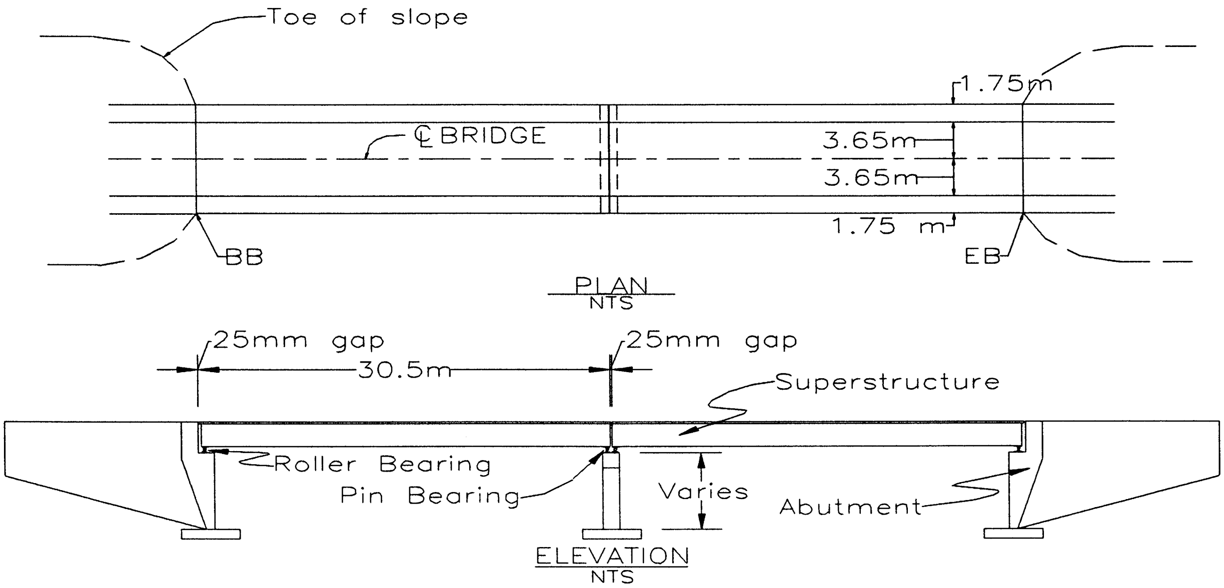


FIGURE 5-2 Two-Span Bridge Plan and Elevation

Dimensions of the deck, girders, and bent cap were based on data for bridges listed in table 5-1. The bridge width was based on two lanes with two shoulders. The reinforced concrete deck was supported by four built-up steel girders spaced at approximately 3m. The mass of the bridge was calculated from these properties and distributed among the nodes based on the tributary areas of each node. Moments of inertia and cross-sectional area of the superstructure were based on sections where steel elements were transformed to an equivalent concrete section. The modular ratio for this transformation was 7.

TABLE 5-2 Constant Two-Span Bridge Properties

Superstructure Properties

I_z (m^4)	Area (m^2)	E (MPa)	G (MPa)	Span Weight (kN)
4	4.7	27770	10440	2633

Expansion Joint Properties

K (kN/mm)
8690

Abutment Properties

K_1 (kN/mm)	K_2 (kN/mm)	Δ_1 (mm)
1550	1.5	9.8

The calculated properties of the superstructure, expansion joint, and abutment can be found in table 5-2. These properties are based on methods described in section 4.

5.3.1 Substructure Stiffness

Parameters that were varied in the two span bridge were substructure stiffness, bearing strength, restrainer design method, and earthquake loading. Table 5-3 shows the variations of substructure stiffness, K_s, and the bearing strength, F_{ub}, in terms of nine different cases. Each of these cases was analyzed with five different restrainer cases (No restrainers, Caltrans, AASHTO, W/2, and Proposed restrainers) and subjected to the five earthquake records described in section 4.10.

TABLE 5-3 Variation of 2-Span Bridge Elements

Case	Substructure Stiffness (kN/mm)	Bearing Strength (kN)
C1	145.9	395
C2	145.9	2173
C3	145.9	3950
C4	75.2	395
C5	75.2	2173
C6	75.2	3950
C7	4.4	395
C8	4.4	2173
C9	4.4	3950

To determine the range of substructure stiffnesses used in the study lower bound and upper bound stiffness cases were formulated. The upper bound case was based on the stiffness of a fixed base reinforced concrete column. The dimensions of the column cross-section were 1.2m by 2.4 m. The height of the column was 4.6m. The moment of inertia of the column was taken as 50 percent of the gross moment of inertia to account for cracking.

The lower bound case was based on the stiffness of a 15.2m column with the most flexible rotational and translational footing spring attached to the base. The footing dimensions were 2.9m by 2.9m. It was found that the most flexible footing corresponded to a soft clay soil condition. The shear strength of a soft clay is 19.2 kPa (Carter and Bentley, 1991) and was used in conjunction with figure 4-10 to construct a shear strength versus shear strain curve. The stress strain curve was then used to construct a non-linear force displacement curve and moment rotation curve. These curves were then idealized into elastic relationships that yielded the footing translational and rotational stiffnesses. These stiffnesses were combined with the column stiffness to determine the lower bound of the stiffness range.

The average of the upper and lower bound values was used to represent bridges with moderate substructure stiffness. To ensure that the three stiffness values (145.9, 75.2, and 4.4 kN/mm) were sufficient to represent all stiffnesses within the selected range, two bridges with substructure stiffnesses of 39.8kN/mm and 110.5kN/mm were analyzed using the El Centro and San Francisco Airport acceleration records. The response of these two bridges did not show new trends. From these results it was concluded that the upper, lower, and average of the upper and lower bound stiffnesses would be sufficient to represent the response for the entire stiffness range.

5.3.2 Bearing Strength

Longitudinal bearing stiffness was not varied in the study. The stiffness was taken as an average stiffness of the bearings discussed in Mander et al., 1996, and was found to be 1168kN/mm for four bearings.

Bearing strength was varied in the study as shown in table 5-3. The magnitude of bearing strength may be determined with a "C" factor as discussed in section 4.9. In Mander et al., 1996 it was shown that C varies from 0.6 to 3. In this study, C factors range from 0.3 to 3. The moderate bearing strength corresponds to a C factor of 1.65 or the average of .3 and 3. To check this range a steel bearing from the Los Angeles River Bridge on Interstate 5 in Los Angeles, California was analyzed using Mander's methods. It was found to have a C factor of approximately 0.4, which falls within the range of C factors chosen. This bearing is shown in figure 5-3.

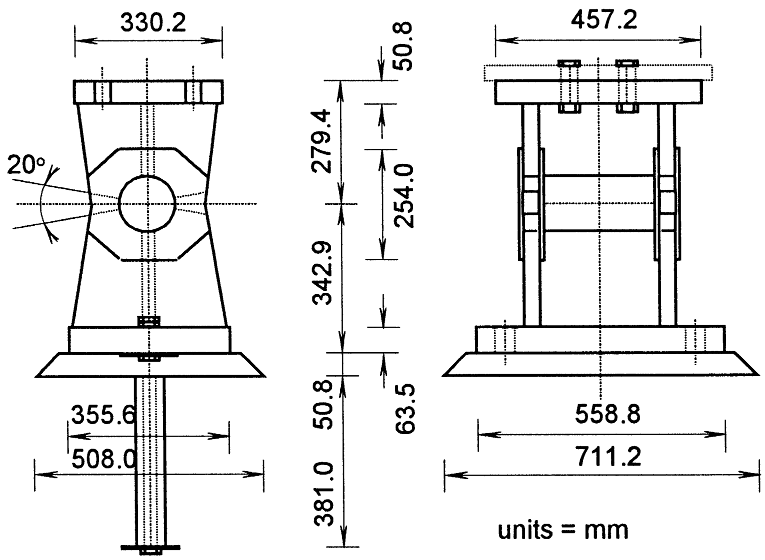


FIGURE 5-3 Typical Steel Bearing

5.3.3 Restrainers

Restrainers were designed for each case, C1 through C9, using design methods described by AASHTO, Caltrans, Proposed, and W/2. The Modified Caltrans method was used only for selected cases described in the next Section. This is because the need for the Modified Caltrans method was realized only after an extensive study of other models had been completed. Restrainer cables were assumed to have a length of 3m, and have a yield displacement of 54mm. The total number of restrainers required by each method is shown in table 5-4. The available seat width and the required seat width by AASHTO is shown in table 5-5. All of the restrainer designs correspond to a peak ground acceleration of 0.7g and an available seat width of 495 mm. The peak ground acceleration was based on the maximum design acceleration found in current design codes (Caltrans, 1994). The available seat width was based on the reinforced concrete bent cap dimensions of the bridges that were studied. In table 5-4, labels "A" and "B" indicate abutments and bents, and labels "P" and "R" pinned and roller ends, respectively. Numbers represent the bent or abutment number shown in figure 3-1.

A minimum number of restrainers corresponds to a force of 0.35*W, where W equals the weight of the span being restrained (FHWA, 1995). This minimum value applied to the Caltrans and the Proposed methods. However, a minimum number of restrainers (6 restrainers) controlled the design only in some cases of the latter method.

The Proposed and Caltrans methods require single degree of freedom spectra to determine the number of restrainers required. In order to more accurately compare the two methods, Caltrans spectra, shown in figure 5-4, were used for restrainer design. The W/2 and AASHTO methods do not require the use of spectra. AASHTO specifications also include design spectra, but were not used to design restrainers. The number of restrainers designed with AASHTO and Caltrans spectra would not be significantly different since peak ground accelerations would be the same and because both sets of spectra were constructed from earthquakes in the western United States.

TABLE 5-4 Restrainer Designs for 2-Span Bridge

Case	*C1*	*C2*	*C3*	*C4*	*C5*	*C6*	*C7*	*C8*	*C9*
A1-P									
Caltrans	28	28	28	31	31	31	22	22	22
AASHTO	11	11	11	11	11	11	11	11	11
W/2	8	8	8	8	8	8	8	8	8
Proposed	26	19	6	29	22	6	20	13	6
B2-R									
Caltrans	28	28	28	31	31	31	22	22	22
AASHTO	11	11	11	11	11	11	11	11	11
W/2	8	8	8	8	8	8	8	8	8
Proposed	6	6	6	6	6	6	6	15	15
B2-P									
Caltrans	28	28	28	31	31	31	22	22	22
AASHTO	11	11	11	11	11	11	11	11	11
W/2	8	8	8	8	8	8	8	8	8
Proposed	6	6	6	6	6	6	6	6	6
A3-R									
Caltrans	28	28	28	31	31	31	22	22	22
AASHTO	11	11	11	11	11	11	11	11	11
W/2	8	8	8	8	8	8	8	8	8
Proposed	28	6	6	31	6	6	22	22	22
TOTAL									
Caltrans	112	112	112	124	124	124	88	88	88
AASHTO	44	44	44	44	44	44	44	44	44
W/2	32	32	32	32	32	32	32	32	32
Proposed	66	37	24	72	40	24	54	56	49

It can be seen in table 5-4 that in every case, Caltrans requires the most restrainer cables. The number of restrainers required by Caltrans changes depending on the soil conditions. Different spectra were used in cases 1 to 3, 4 to 6, and 7 to 9 to account for changing soil conditions reflected in the different substructure stiffness. For example, cases 1 through 3 represent the most stiff substructure, therefore the spectrum corresponding to 0 to 3m of alluvium was used (figure 5-4). In most cases, W/2 requires the least number of restrainers, and AASHTO results are between the upper and lower bound number of restrainers. In two cases, the Proposed method requires the least number of restrainers.

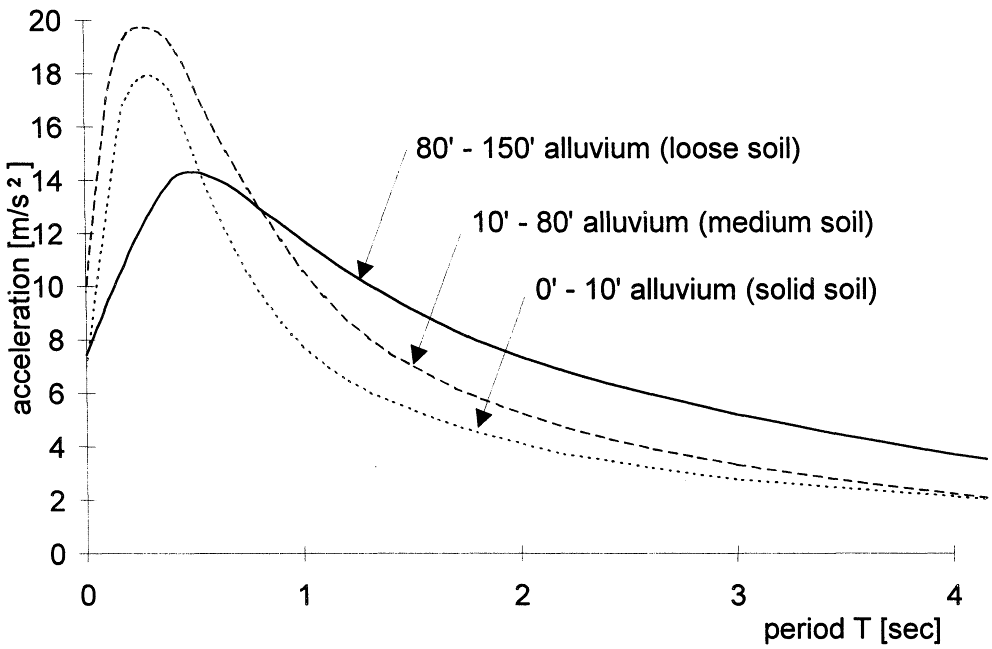


FIGURE 5-4 Caltrans Design Spectra for 0.7g

The AASHTO seat width shown in table 5-5 was calculated using equation 2.8. In this equation, skew was equal to zero, the length of the span was fixed at approximately 30m (100 ft.), and the column height varied. Column heights corresponded to the substructure stiffness and are shown along with the seat widths in table 5-5.

TABLE 5-5 AASHTO Seat Width Requirements

Case	H_{col} (m)	AASHTO Seat Width (mm)	Available Seat Width (mm)
C1	5	427	495
C2	5	427	495
C3	5	427	495
C4	10	480	495
C5	10	480	495
C6	10	480	495
C7	15	533	495
C8	15	533	495
C9	15	533	495

The required seat width is satisfied in all cases but C7, C8, and C9. Note that the required seat width increases as column height increases and stiffness decreases. It is interesting that the required seat width is independent of bearing strength. Also note that the seat width required in C1 is twenty percent smaller (approximately 100mm) than that required in C9. This implies that displacement demand for C9 would be much greater than C1 according to AASHTO.

5.4 Five-Span Bridges

The elevation of the five-span bridge is shown in figure 5-5, and a typical section can be found in figure 5-6. As in the study of the two-span bridge, nine cases were studied (cases C1 to C9) in which substructure stiffness and bearing strength were varied. The number of restrainers for each case was varied depending on the restrainer design methods described in sections 2 and 3. All bridges were analyzed for the same earthquake records that were used in the two-span bridge.

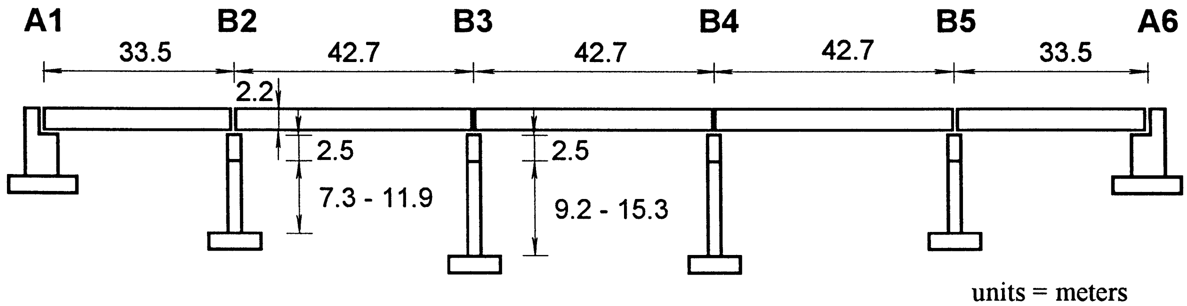


FIGURE 5-5 Five-Span Bridge Elevation

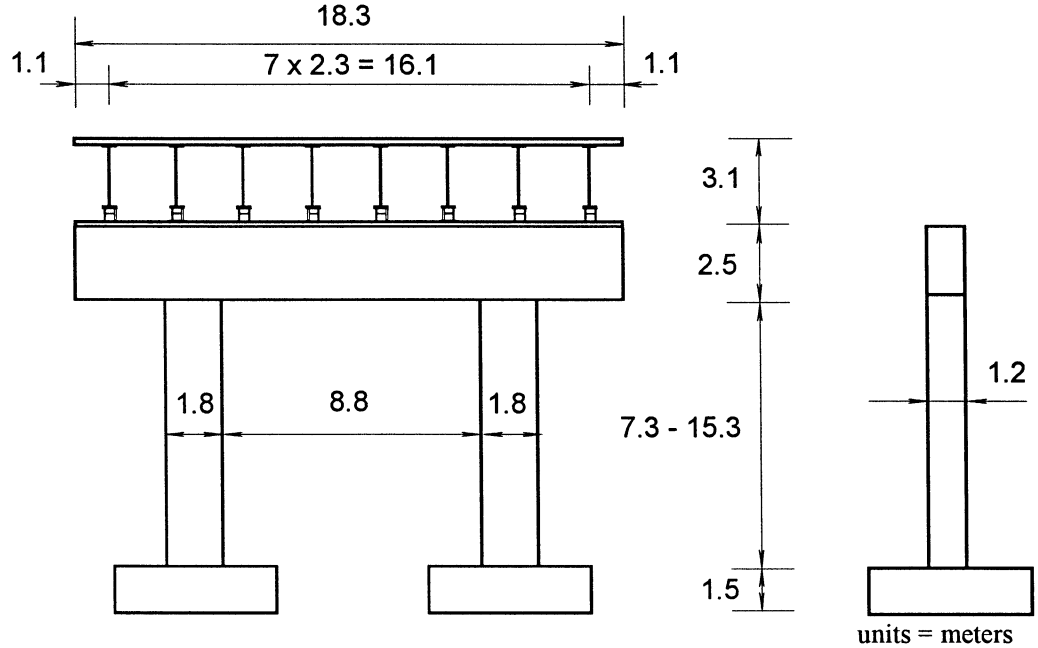


FIGURE 5-6 Five-Span Bridge Typical Section

All cases were assumed to have composite concrete/steel superstructure with constant properties (table 5-6). The slab thickness was 197mm. The moment of inertia presented in table 5-6 is about the transverse axis of the bridge. Between adjacent spans, as well as between superstructure and abutments, a gap of 25mm was used for five-span bridges.

TABLE 5-6 Constant Five-Span Bridge Properties

Superstructure Properties

I_z (m^4)	Area (m^2)	E (MPa)	G (MPa)	Span Weight (kN/m)
4.5	7.0	27770	10440	139.5

Expansion Joint Properties

K (kN/mm), B2 & B5	K (kN/mm), B3 & B4
10259	9164

Abutment Properties

K_1 (kN/mm)	K_2 (kN/mm)	Δ_1 (mm)
2626	2.6	9.8

Abutment properties were constant in all cases. Abutment stiffness and yield force were calculated assuming failure of the abutment wall. Height and length of the wall were assumed to be 3.05 m and 18.3 m, respectively. The abutment stiffness and strength properties are shown in table 5-6.

The properties calculated for expansion joint gap elements can also be found in table 5-6. Like the two-span bridge, area and moment of inertia properties were based on equivalent concrete sections with a modular ratio equal to 7.

5.4.1 Substructure Stiffness

Each bent consisted of two columns and a bent cap (figure 5-6). Properties of the bent cap were the same in all cases. The width of the bent cap was assumed to be 1.22m. The bridge width was based on four 3.7m lanes and two 1.75m shoulders. Column height was varied between 7.3 m and 11.9 m in outer bents and between 9.2 m and 15.3 m in inner bents. In calculating column stiffness, one-half of the gross moment of inertia was used to account for cracking. Column stiffness ratio between inner and outer bents was assumed to be 0.6. Footing properties were calculated the same way as that of the two span bridge, but the footing dimensions were 5.2m by 5.2m.

Table 5-7 shows the variation of substructure stiffness and bearing strength for each case. As in the two-span bridge, nine different structural types were defined and analyzed ranging from case C1 with strong fixed columns (founded on stiff soil) and weak bearings to case C9 with very flexible columns founded on flexible soil with strong bearings. Similar to the two span bridge, the average of the upper and lower bound values was used to represent bridges with moderate substructure stiffness. Three substructure stiffness cases were assumed to capture the full range of response.

TABLE 5-7 Five-Span Bridge Bent and Bearing Properties

Case	Substructure Stiffness (kN/mm) B2 & B5	Substructure Stiffness (kN/mm) B3 & B4	Bearing Strength (kN) A1 & B5	Bearing Strength (kN) B2, B3, & B4
C1	25.1	14.9	1064	1264
C2	25.1	14.9	4045	5095
C3	25.1	14.9	7022	8931
C4	13.9	8.2	1064	1264
C5	13.9	8.2	4045	5095
C6	13.9	8.2	7022	8931
C7	2.4	1.5	1064	1264
C8	2.4	1.5	4045	5095
C9	2.4	1.5	7022	8931

5.4.2 Bearing Strength

Similar to the two-span bridge, it was assumed that roller bearings did not carry horizontal force. Pinned bearing properties (horizontal yield force and stiffness) were calculated according to recommendations in Mander et al., 1996. The ratio between bearing horizontal yield force and vertical force was assumed to be 0.4 in cases C1, C4 and C7. In cases C3, C6 and C9 horizontal yield force was determined considering a relatively high ratio between horizontal and vertical force (3.0). In cases C2, C5 and C8 horizontal yield force was determined as a mean value of the previous two. Bearing stiffness was assumed to be constant (2972 kN/mm for eight bearings). In table 5-7 the total yield force for eight bearings is presented.

5.4.3 Restrainers

The diameter and length of cable restrainers was assumed to be 19mm and 6.1m, respectively. The yield displacement was 107mm. The post yielding slope in the stress-strain diagram was 1 percent. For all cases and design procedures, restrainers were designed assuming a peak ground acceleration of 0.7g. The numbers of restrainers required by different methods are presented in table 5-8.

Note that the results from AASHTO and W/2 methods are insensitive to the variation of substructure and bearing properties. Therefore, the number of restrainers was constant for the cases when these methods were used. The two methods led to a relatively close number of restrainers because they were both based on the weight of the span (W). AASHTO is based on 0.7*W, and W/2 is based on 0.5*W.

The Caltrans design procedure resulted in the largest number of restrainers in all cases. The number required by Caltrans was especially large compared to AASHTO and W/2 in cases C4 to C6. In these cases, the ratio of the number of restrainers required by Caltrans to the number required by the W/2 method was 3.4 and the same ratio between Caltrans and AASHTO was 2.6. These differences were due to very large design forces obtained from the Caltrans method.

Because the Caltrans design procedure assumes that during a strong earthquake the bearings fail, the entire seismic force must be resisted by restrainers. The Caltrans method also ignores substructure flexibility. This assumption leads to a relatively large overall stiffness, reduces the period of the structure, and increases the spectral acceleration. The spectral acceleration was near the peak value in cases C4 to C6. In other cases, the number of restrainers based on the Caltrans method was not as high as that of other cases. However Caltrans always required a number of restrainers that was significantly larger than the other design methods. Restrainer design forces were lower in structures C1 to C3 and C7 to C9 than in structures C4 to C6 because the response was larger for the medium soil design spectra which was used for C4 to C6.

TABLE 5-8 Restrainer Designs for Five-Span Bridge

	AASHTO	*Caltrans*	*W/2*	*Prop. Meth.*	*AASHTO*	*Caltrans*	*W/2*	*Prop. Meth.*	*AASHTO*	*Caltrans*	*W/2*	*Prop. Meth.*
		case:	*C1*			*case:*	*C4*			*case:*	*C7*	
B2-R	19	38	14	12	19	48	14	12	19	38	14	19
B3-R	24	48	18	12	24	62	18	21	24	49	18	12
B4-R	24	48	18	12	24	62	18	12	24	49	18	12
B5-R	24	48	18	25	24	62	18	27	24	49	18	19
A6-R	19	38	14	38	19	48	14	48	19	38	14	38
A1-P	19	38	14	12	19	48	14	48	19	38	14	38
B2-P	24	48	18	12	24	62	18	21	24	49	18	12
B3-P	24	48	18	12	24	62	18	12	24	49	18	12
B4-P	24	48	18	12	24	62	18	12	24	49	18	12
B5-P	19	38	14	12	19	48	14	12	19	38	14	12
total no.	***220***	***440***	***164***	***159***	***220***	***564***	***164***	***225***	***220***	***446***	***164***	***186***
		case:	*C2*			*case:*	*C5*			*case:*	*C8*	
B2-R	19	38	14	12	19	48	14	12	19	38	14	19
B3-R	24	48	18	12	24	62	18	12	24	49	18	12
B4-R	24	48	18	12	24	62	18	12	24	49	18	12
B5-R	24	48	18	12	24	62	18	27	24	49	18	19
A6-R	19	38	14	12	19	48	14	12	19	38	14	38
A1-P	19	38	14	12	19	48	14	12	19	38	14	21
B2-P	24	48	18	12	24	62	18	12	24	49	18	12
B3-P	24	48	18	12	24	62	18	12	24	49	18	12
B4-P	24	48	18	12	24	62	18	12	24	49	18	12
B5-P	19	38	14	12	19	48	14	12	19	38	14	12
total no.	***220***	***440***	***164***	***120***	***220***	***564***	***164***	***135***	***220***	***446***	***164***	***169***
		case:	*C3*			*case:*	*C6*			*case:*	*C9*	
B2-R	19	38	14	12	19	48	14	12	19	38	14	19
B3-R	24	48	18	12	24	62	18	21	24	49	18	12
B4-R	24	48	18	12	24	62	18	12	24	49	18	12
B5-R	24	48	18	12	24	62	18	27	24	49	18	19
A6-R	19	38	14	12	19	48	14	12	19	38	14	38
A1-P	19	38	14	12	19	48	14	12	19	38	14	12
B2-P	24	48	18	12	24	62	18	12	24	49	18	12
B3-P	24	48	18	12	24	62	18	12	24	49	18	12
B4-P	24	48	18	12	24	62	18	12	24	49	18	12
B5-P	19	38	14	12	19	48	14	12	19	38	14	12
total no.	***220***	***440***	***164***	***120***	***220***	***564***	***164***	***144***	***220***	***446***	***164***	***160***

Although the performance criterion in the Proposed method is similar to Caltrans, the new method is different and more sophisticated. Unlike the Caltrans method, it considers many important parameters that influence bridge response. For example, Caltrans only considers restrainer stiffness and site spectra when determining response. On the other hand, the Proposed procedure is sensitive to the variation of column stiffness, foundation and soil properties, bearing strength and stiffness, restrainer stiffness, and abutment abutment stiffness and strength. Therefore, the number of restrainers required by the Proposed method can be very different from Caltrans. Compared to the Caltrans design procedure, the Proposed method led to a significantly smaller number of restrainers with the exception of critical locations such as abutments. In structures with weak bearings (C1, C4 and C7) and structures with very flexible bents (C7, C8, C9), the number of restrainers at the right abutment (A6-R) was equal to that obtained from the Caltrans method. The number of restrainers at the left abutment in structures C4 and C7 was also equal for both methods. Compared to the other two methods (AASHTO and W/2), the Proposed method generally led to a smaller number of restrainers.

TABLE 5-9 AASHTO Seat Width Requirements

Case	Location	L_{span} (m)	H_{col} (m)	AASHTO Seat Width (mm)
C1	A1-P	34	7	462
C2	B2-R	34	7	462
C3	B2-P	43	7	485
	B3-R	43	9	503
	B3-P	43	9	503
	B4-R	43	9	503
	B4-P	43	9	503
	B5-R	43	7	485
	B5-P	34	7	462
	A6-R	34	7	462
C4	A1-P	34	10	486
C5	B2-R	34	10	486
C6	B2-P	43	10	509
	B3-R	43	12	533
	B3-P	43	12	533
	B4-R	43	12	533
	B4-P	43	12	533
	B5-R	43	10	509
	B5-P	34	10	486
	A6-R	34	10	486
C7	A1-P	34	12	507
C8	B2-R	34	12	507
C9	B2-P	43	12	530
	B3-R	43	15	564
	B3-P	43	15	564
	B4-R	43	15	564
	B4-P	43	15	564
	B5-R	43	12	530
	B5-P	34	12	507
	A6-R	34	12	507

The AASHTO required seat widths for the five-span bridges are presented in table 5-9. The available seat width for each girder in all five-span bridges was 495mm. In cases C1, C2, and C3, which correspond to shorter columns, seat width requirements are satisfied at A1, B2,B5, and A6. The seat widths at the middle bents, B3, and B4, do not satisfy AASHTO requirements. The only other locations that satisfy AASHTO requirements are A1 and A6 in cases C4, C5, and C6. The rest of the locations do not satisfy AASHTO requirements.

5.5 Bridges with Narrow Seats

Seat width is the most important parameter to consider when facing the problem of bridge unseating. The bridges described in previous sections are representative of typical bridges found in the United States. The seat width on these bridges was 495mm and was based on a reinforced concrete bent cap. Bridges with very large seat widths will rarely be subject to unseating. As seat width decreases the chance that the span will unseat increases. Not all bridge seat widths are greater than or equal to 495mm. Seat width is largely based on the type of bent cap that the girders are seated on. Bridges with steel bent caps usually provide a narrow seat. To make this study more comprehensive, the seat widths of the cases which were most critical in terms of unseating were reduced. A selected number of states were surveyed to determine how small of a seat width is used in bridges. The study also was used to determine the type of bridges with narrow seats.

TABLE 5-10 Narrow Seat Width Survey Data Summary

Bridge Name	Route	City, State	Average Span Length (m)	Number of Spans	Average Column Height (m)	Bent Cap Type	ASW (mm)
West Connector Overcrossing	280/17/880	San Jose, CA	35	10	9	Steel	254
Twin Bridges over the Middle Tyger River	I-85	South Carolina	12	6	11	-	102
Underpass Under US-15	I-26	South Carolina	18	5	5	-	127
Twin Bridges over Indian Creek	I-26	South Carolina	9	7	5	-	152
Bridge Over Creek	-	Georgia	6	8	3	Timber	41
Bridge Over Big Swift Creek	-	Georgia	6	9	2	Timber	41
Deegan Boulevard	-	New York	-	47	-	Steel	0
West 155th Street Viaduct, Macombs Dam Bridge	-	New York, NY	13	31	-	Steel	343
Skyway Bridge (North Approach to Fuhrmann Boulevard Structure)	-	Buffalo, NY	-	-	-	Steel	0
Horton Viaduct	-	New York	-	-	-	Steel	152

Table 5-10 shows a summary of the response to the short seat width survey. This is only a sample of the bridges that have short seat widths in the U.S. The available seat width (ASW in table 5-10) was determined for each bridge from plans. An average of available seat width of 127mm (5 in.) was found and used in the model. Table 5-11 shows a summary of the response of the narrow seat width questionnaires. It is interesting to note that most bridges in the states that were surveyed satisfy AASHTO seat width requirements.

TABLE 5-11 Narrow Seat Width Survey Questionnaire Response

State	Percentage of Bridges that Satisfy AASHTO Requirements	Percentage of Bridges With Steel Bent Caps	Steel Bent Caps In This State Have Narrow Seat Widths
Georgia	*50-75*	*0-10*	*Yes*
Pennsylvania	*75-100*	*0-10*	*No*
South Carolina	*50-75*	*0-10*	*No*
Vermont	*75-100*	*0-10*	*No*
New York	-	-	*Yes*
Illinois	*75-100*	*0-10*	*No*
New Jersey	*75-100*	*0-10*	*No*

The most critical cases from the study of the two-span and five-span bridges were chosen to study the effect of narrow seats. Case C7 and C8 were the most critical cases for both the two-span and five-span bridges. The two-span bridge was subjected to the San Francisco Airport record and the five-span bridge to the Kobe record. Earthquakes were chosen that would generate the maximum response in the two bridges. The 127mm seat width affected the bent cap detail and mass of both bridges. A steel bent cap detail was formulated from the most common properties found in plans of the bridges in the survey. A detail of the steel bent cap is shown in figure 5-7. Note that even though the plate connecting the two girders may act as a restrainer, because it was not designed for earthquake forces, its effect was ignored. The tributary mass of the bent was reduced because of the smaller weight of the steel bent cap.

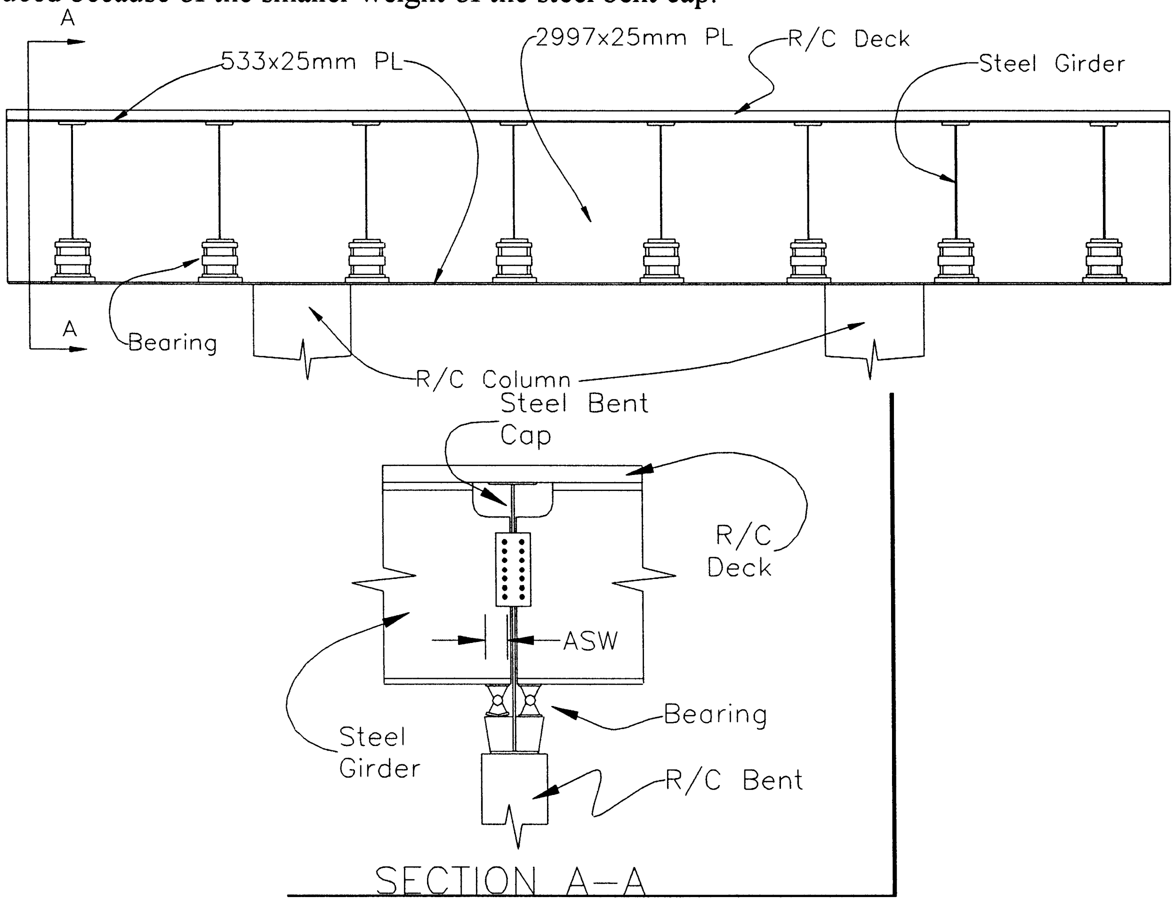


FIGURE 5-7 Short Seat Bent Cap Detail

Restrainers were redesigned for both bridges based on 0.7g and 1.0g peak ground acceleration (PGA). In recent seismic events, such as the Northridge Earthquake, ground accelerations greater than 1g were measured. Current design specifications have maximum ground acceleration of 0.7g, hence Caltrans spectral values were extrapolated using least squares regression to determine the spectrum for 1g. The extrapolated and current Caltrans design spectra are shown in figure 5-8.

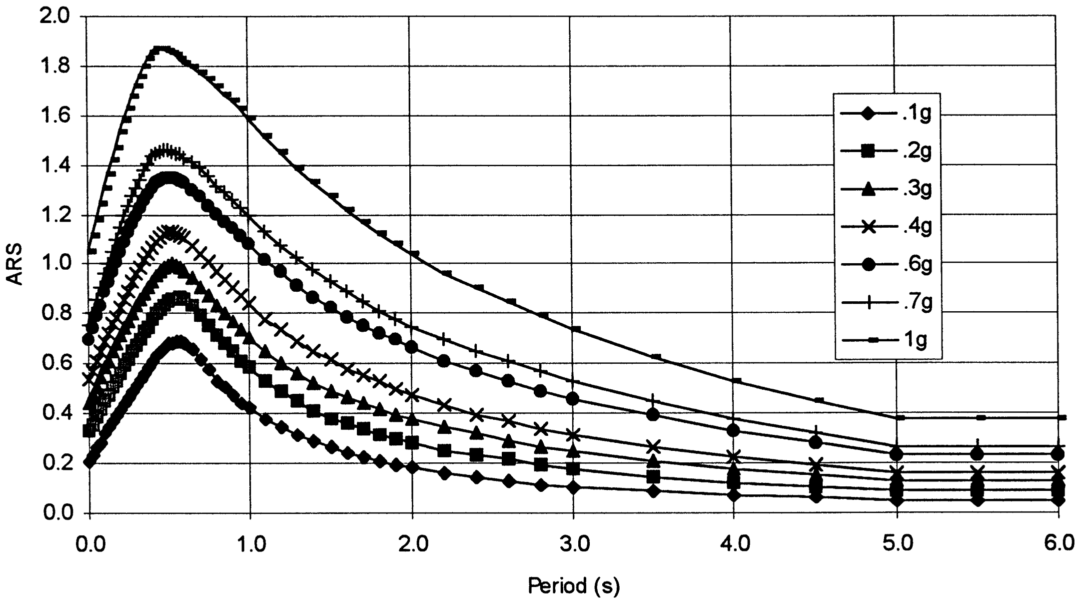


FIGURE 5-8 Extrapolated 1g Caltrans Design Spectra

The number of restrainers for the two span bridge is shown in table 5-12. In the 0.7g cases, the number of restrainers required for the wide seats (table 5-4) and the number required for the narrow seats are not that different. The Caltrans, AASHTO, and W/2 designs were not affected by the seat width. The results from the Proposed method was only changed slightly. The AASHTO and W/2 results were unchanged because they are independent of seat width. The results from the Proposed method was slightly changed due to the decreased mass of the bent cap.

TABLE 5-12 Number of Restrainers for the Two-Span Bridges with Narrow Seats

Scaled .7g Earthquake, Case: C7

	Number of Restrainers				
Method	A1-R	B2-P	B2-R	A3-P	Total
Proposed	22	6	14	20	62
Caltrans	22	22	22	22	88
AASHTO	11	11	11	11	44
W/2	8	8	8	8	32

Scaled 1g Earthquake, Case: C7

	Number of Restrainers				
Method	A1-R	B2-P	B2-R	A3-P	Total
Proposed	28	6	20	26	80
Caltrans	28	28	28	28	112
AASHTO	15	15	15	15	60
W/2	8	8	8	8	32

Scaled .7g Earthquake, Case: C8

	Number of Restrainers				
Method	A1-R	B2-P	B2-R	A3-P	Total
Proposed	22	6	14	13	55
Caltrans	22	22	22	22	88
AASHTO	11	11	11	11	44
W/2	8	8	8	8	32

Scaled 1g Earthquake, Case: C8

	Number of Restrainers				
Method	A1-R	B2-P	B2-R	A3-P	Total
Proposed	28	6	20	19	73
Caltrans	28	28	28	28	112
AASHTO	15	15	15	15	60
W/2	8	8	8	8	32

The main reason Caltrans and Proposed restrainer designs were not affected by the short seat width is that the yield displacement of a 3m restrainer is only 54mm which is much less than the available seat width of 127mm. Since restrainers are designed elastically, restrainer designs would only be impacted by seat width if the available seat width fell below 54mm.

With the exception of the W/2 method, the numbers of restrainers increased when the peak ground acceleration was increased to 1g. The number of restrainers required by the W/2 method did not change because it is independent of ground acceleration. Again Caltrans required the greatest number of restrainers, and W/2 required the least.

TABLE 5-13 Number of Restrainers for the Five-Span Bridges with Narrow Seats

Scaled .7g Kobe Earthquake, Case: C7

	Number of Restrainers										
Method	A1-P	B2-R	B2-P	B3-R	B3-P	B4-R	B4-P	B5-R	B5-P	A6-R	Total
Proposed	33	16	12	13	12	13	12	16	10	38	175
Caltrans	38	38	49	49	49	49	49	49	38	38	446
AASHTO	19	19	24	24	24	24	24	24	19	19	220
W/2	14	14	18	18	18	18	18	18	14	14	164

Scaled .7g Kobe Earthquake, Case: C8

	Number of Restrainers										
Method	A1-P	B2-R	B2-P	B3-R	B3-P	B4-R	B4-P	B5-R	B5-P	A6-R	Total
Proposed	21	16	12	13	12	12	12	16	10	38	162
Caltrans	38	38	49	49	49	49	49	49	38	38	446
AASHTO	19	19	24	24	24	24	24	24	19	19	220
W/2	14	14	18	18	18	18	18	18	14	14	164
Modified Caltrans	38	38	12	49	12	49	12	49	12	38	309

Scaled 1g Kobe Earthquake, Case: C7

	Number of Restrainers										
Method	A1-P	B2-R	B2-P	B3-R	B3-P	B4-R	B4-P	B5-R	B5-P	A6-R	Total
Proposed	45	22	18	19	12	19	12	22	10	51	230
Caltrans	51	51	65	65	65	65	65	65	51	51	594
AASHTO	27	27	34	34	34	34	34	34	27	27	312
W/2	14	14	18	18	18	18	18	18	14	14	164

Scaled 1g Kobe Earthquake, Case: C8

	Number of Restrainers										
Method	A1-P	B2-R	B2-P	B3-R	B3-P	B4-R	B4-P	B5-R	B5-P	A6-R	Total
Proposed	33	22	12	19	12	12	12	22	10	51	205
Caltrans	51	51	65	65	65	65	65	65	51	51	594
AASHTO	27	27	34	34	34	34	34	34	27	27	312
W/2	14	14	18	18	18	18	18	18	14	14	164
Modified Caltrans	51	51	65	65	12	65	12	65	51	51	488

The numbers of restrainers required by each design method for the five-span bridge are shown in table 5-13. As with the two-span bridge, the Caltrans, AASHTO, and W/2 results were unchanged for the 0.7g case. The number required by the Proposed method decreased because of the lower weight of the steel bent cap. With a PGA of 1g, all design methods except W/2 required more restrainers due to the increased seismic demand. The result from the Modified Caltrans method is also shown in table 5-13. It is shown only in case C8 because Modified Caltrans and Caltrans require the same number of restrainers in C7 due to low bearing strengths. Note that the Modified Caltrans method requires 137 less restrainers in the 0.7g case and 106 less in the 1g case than Caltrans.

5.6 Skewed Bridges

Highly skewed bridges have proven vulnerable to strong earthquakes. The Gavin Canyon Undercrossing on Interstate 5 was a highly skewed bridge that collapsed during the Northridge Earthquake in California (Yashinsky et al., 1995). The five-span bridge was modified to be skewed by 30, 45, and 60 degrees. The length of the bridge and the width perpendicular to the centerline were kept the same. The abutments, bents, and expansion joints were skewed. Plan views of the bridges that were studied are shown in figure 5-9. Case C8 was chosen to be studied because it proved to be most susceptible to unseating in the main and short seat studies.

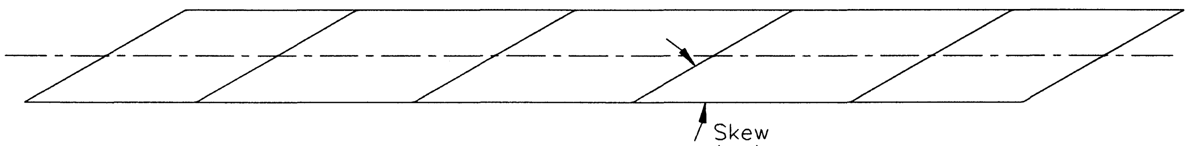


FIGURE 5-9 Plan View of Skewed Bridges

Bridge properties that had to be modified were bent mass, bent stiffness, abutment stiffness and strength, and expansion joints. Bents were aligned with the skew, which increased their length. Total weights of the bents are listed in table 5-14; they were based on the bent cap detail shown in figure 5-6. Outer bents are B2 and B5 and inner bents are B3 and B4 (figure 5-5).

TABLE 5-14 Weights of Skewed Bents

Skew (o)	Outer Bent (kN)	Inner Bent (kN)
30	658	725
45	712	778
60	827	894

The bent stiffnesses were calculated based on methods described in Section 4.7 and are shown in table 5-15. Kx, Ky, and Kϕ are the longitudinal, transverse, and torsional stiffnesses, respectively. Kt is the stiffness parallel to the axis of the skew, and Kl is the stiffness perpendicular to the skew.

TABLE 5-15 Transformed Bent Stiffnesses for Skewed Bridges

Bent	Skew (^{o})	kt (kN/mm)	kl (kN/mm)	kx(kN/mm)	ky(kN/mm)	Kϕ(kN*m/rad)
Outer	30	2.61	2.40	2.45	2.56	272650
	45	2.61	2.40	2.51	2.51	406788
	60	2.61	2.40	2.56	2.45	808535
Inner	30	1.68	1.50	1.55	1.64	173389
	45	1.68	1.50	1.59	1.59	257643
	60	1.68	1.50	1.64	1.55	509269

Abutment properties are shown in table 5-16. In this table, W is the width of the bridge measured perpendicular to its centerline, W' is the width of abutment, and H is the height of the abutment wall. Ka, Fy, and Δult are the total abutment stiffness, yield force, and yield displacement. These properties were found using the methods described in section 4.6.

TABLE 5-16 Abutment Properties for Skewed Bridges

Skew (°)	W (m)	W(m)	H (m)	ka(kN/mm)	Fy (kN)	Δult (mm)
30	18.3	21.1	3.0	3033	29663	10
45	18.3	25.9	3.0	3715	36329	10
60	18.3	36.6	3.0	5254	51377	10

TABLE 5-17 Transverse Bearing Stiffnesses (Mander et al., 1996)

Type	Stiffness (kN/mm)
Low Type Fixed	356
High Type Bolster	559
High Type Fixed	560
High Type Fixed	236
High Type Fixed on Pedistal	221
Avg. =	386.4

TABLE 5-18 AASHTO Seat Width Requirements for Skewed Cases

Skew	Location	L (m)	H (m)	No (mm)	Skew Factor	N (mm)
30	A1-P	34	12	507	1.11	565
30	B2-R	34	12	507	1.11	565
30	B2-P	43	12	530	1.11	590
30	B3-R	43	15	564	1.11	627
30	B3-P	43	15	564	1.11	627
30	B4-R	43	15	564	1.11	627
30	B4-P	43	15	564	1.11	627
30	B5-R	43	12	530	1.11	590
30	B5-P	34	12	507	1.11	565
30	A6-R	34	12	507	1.11	565
45	A1-P	34	12	507	1.25	636
45	B2-R	34	12	507	1.25	636
45	B2-P	43	12	530	1.25	665
45	B3-R	43	15	564	1.25	707
45	B3-P	43	15	564	1.25	707
45	B4-R	43	15	564	1.25	707
45	B4-P	43	15	564	1.25	707
45	B5-R	43	12	530	1.25	665
45	B5-P	34	12	507	1.25	636
45	A6-R	34	12	507	1.25	636
60	A1-P	34	12	507	1.45	736
60	B2-R	34	12	507	1.45	736
60	B2-P	43	12	530	1.45	769
60	B3-R	43	15	564	1.45	818
60	B3-P	43	15	564	1.45	818
60	B4-R	43	15	564	1.45	818
60	B4-P	43	15	564	1.45	818
60	B5-R	43	12	530	1.45	769
60	B5-P	34	12	507	1.45	736
60	A6-R	34	12	507	1.45	736

Since the skewed bridge model is three-dimensional, transverse bearing stiffness could no longer be neglected. So that a transverse mode of unseating would not occur, bearings were assumed to remain elastic in the transverse direction. To determine the transverse stiffness of the bearing, the stiffnesses from Mander et al., 1996 were averaged to a value of 386.4 kN/mm per bearing (table 5-17).

The restrainer numbers listed in table 5-13 were used for all skewed models. None of the current design procedures account for skew, therefore the same restrainer design would apply to both non-skew skew bridges. Bearing and restrainer properties were distributed to different elements in the analytical model as described in section 4.4 and 4.9, respectively.

Table 5-18 shows the AASHTO seat width requirements for the skewed bridges. In this table, L is the span length and H is the column height. N_o is the AASHTO required seat width before skew is taken into account. The skew factor is the factor that accounts for skew in the AASHTO seat width equation, and N is the required seat width for the skewed bridges at each location. Note that AASHTO requires seat widths to be wider because of skew. Note that the required seat width for the skewed bridges ranges from 11 to 45 percent greater than that of non-skewed bridges.

SECTION 6
ANALYTICAL RESULTS

6.1 Introduction

This section presents the results of the analysis of the bridge models described in the previous sections. The results are divided into two groups, one for cases C1 to C9 in which the full range of parameters were varied (main study) and the other for special cases in which the effect of narrow seats and skew were studied for a limited number of cases. The primary response parameters that were evaluated were the critical relative displacements between girders and their seats in addition to displacement ductilities of the non-linear bridge elements. Results presented in this section were obtained from over 500 different computer runs.

6.2 Main Study

The main part of this research was a parametric study of two and five-span bridges. The parameters that were varied are described in sections 5.3 and 5.4. Nine cases were studied in which substructure stiffness and bearing strength were varied. In each of the nine cases, five different restrainer designs were studied: no restrainers, Caltrans, AASHTO, W/2 and Proposed methods. Nine cases with five restrainer design cases leads to forty-five two-span and five-span models for each case. These ninety models were subjected to five different earthquake loads (section 4.10). In all, 450 runs were done in the main study. This section presents the results of the main parametric study.

6.2.1 Two-Span Bridge

The principle for judging the effectiveness of a design procedure is based on whether or not the design objective is accomplished. The objective of all restrainer design methods is to prevent unseating. To establish the relative merit of each design, the margin between the design objective and the response may be evaluated. The Caltrans design procedure assumes that bearings always fail and that the superstructure is free to move. The number of restrainers that are required depends on the forces predicted by design spectra. The AASHTO design procedure assumes that the force to cause the bridge to move and become unseated is the site maximum acceleration coefficient times the mass of the span. The AASHTO restrainer design method has no criteria for displacement response. The W/2 method has no criteria for predicting earthquake displacement or force either. The Proposed method predicts that earthquake force and displacement response are affected by all bridge elements. Spans are not allowed to unseat and restrainers are not allowed to yield in the Proposed method. Each of the restrainer design methods are generally expected to perform differently because of the different performance criteria. However, all of the design procedures have the common objective of preventing collapse. The most effective design procedure will satisfy this objective with a minimum number of restrainers.

To judge the effectiveness of each restrainer procedure, relative displacements, restrainer displacement ductilities, total displacements, effectiveness factors, abutment ductilities, and bearing ductilities were examined.

6.2.1.1 Relative Displacements at Supports

Figure 6-1 presents the envelopes of maximum relative displacements of the two-span bridge models subjected to all earthquakes. As in previous sections, "A" stands for abutment, "B" stands for bent, "P" stands for pin, and "R" stands for roller. The maximum relative displacement at any location on the two-span bridge without restrainers was approximately 110 mm and the available seat width (ASW) was 495mm, indicating that no restrainers were needed. Even in the worst case, when relative displacements were the largest, the bridge did not become unseated. In most cases, the Caltrans design led to the least relative displacement. Notice in case C7 and cases with strong bearings, restrainers increased relative displacements at pinned locations. Generally, abutments tend to have a higher displacement demand because of the restrainers. This was especially true in cases C1 and C4. Roller bearing locations also tend to have a higher displacement demand. These trends were attributed to a redistribution of forces by restrainers.

The W/2 and AASHTO methods performed similarly in case C1. They required a similar number of restrainers (8 and 11). At the abutments, the Proposed restrainer numbers were close to those of Caltrans; thus, the response was close to Caltrans. At the bent, Proposed restrainer designs were close to AASHTO and W/2, and the response was close to AASHTO and W/2. The response was inversely proportional to the number of restrainers in this case. Case C4 showed similar trends.

In general, the response of case C2 was relatively small (maximum displacement ~ 45mm). The Proposed procedure required restrainers at the pinned abutment only. At the rollers, the response for all methods was inversely proportional to the number of restrainers. Displacement demand was higher at the rollers for all methods, and the restrainers did not affect the response significantly at B2-P.

In case C3, restrainers had no effect on the response at the pinned locations because of high bearing strength. Response at the roller side was inversely proportional to the number of restrainers. Restrainers had a smaller effect on the response at the abutments than at the bent.

Restrainers had little or no effect on the response at the pinned locations in case C5. The Proposed procedure required restrainers at A3-P, even though there was no need for them. This was due to the strength reduction factor of 0.7 on the bearing capacity. AASHTO, W/2, and Proposed methods performed about the same, since they all required nearly the same number of restrainers.

With the exception of Caltrans, all of the restrainer designs performed about the same in case C6. Caltrans reduced relative displacement much more than the others. Without restrainers, the Proposed method predicted almost 330mm of relative displacement which was less than the ASW; hence, the minimum number of restrainers was required.

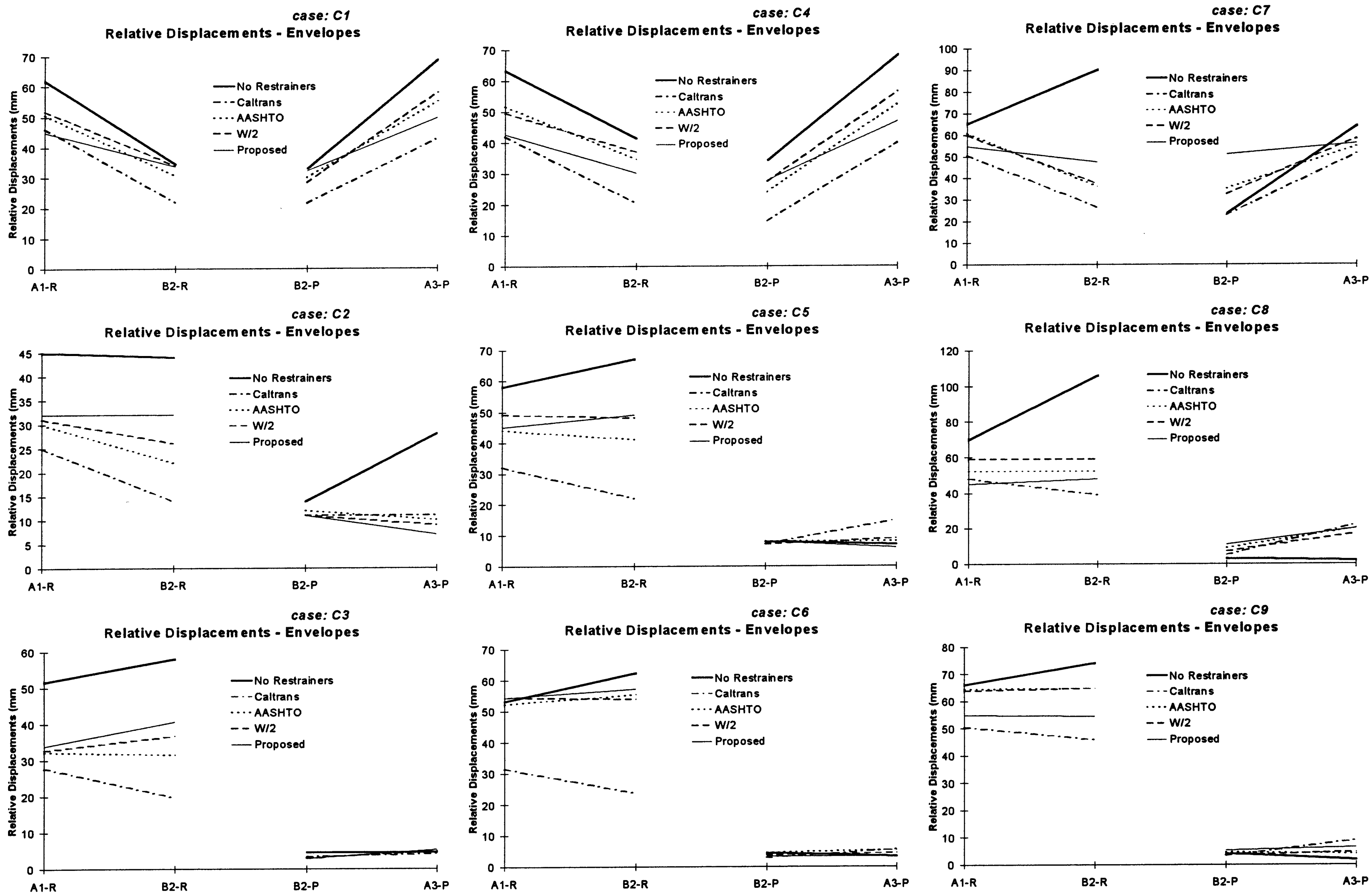


FIGURE 6-1 Envelopes of Peak Relative Displacements for Two-Span Bridges

Results from case C7 showed that restrainers at B2-P increased displacements for nearly all design methods. The AASHTO and W/2 restrainers performed nearly the same. For the case without restrainers, the displacement at B2-P seemed to be very small, but when compared with the other cases with flexible columns (C8 and C9), the displacements seemed to be reasonable. Without restrainers, displacements at B2-R were larger than at A1-R; this trend was common in other cases.

In case C8, restrainers increased relative displacements at pinned locations and reduced displacements more effectively than other cases at roller locations (restrainers reduced displacements by approximately 40 mm at the bent). The Caltrans and Proposed methods reduced displacements the most. This case had the largest relative displacements of all cases at the roller locations.

The Proposed method was the only procedure that predicted the behavior accurately at pinned locations. In case C9, restrainers had no effect on the response on the pinned sides of the bridge because the bearings were strong. The Proposed method was the only procedure that did not require restrainers at pinned locations. The performances of AASHTO and W/2 restrainers were nearly identical. The Proposed and Caltrans methods performed similarly, but Caltrans reduced displacements slightly more than the Proposed method.

Cases C1,C4, and C7 (cases with weak bearings) all experienced similar responses. They experienced the largest relative displacements at pinned locations and abutments. Cases C7, C8 and C9 (cases with the most flexible substructure) had the largest relative displacements at the roller locations and bents. Generally, restrainers did not affect the response of the bridge substantially. The maximum reduction in displacements that restrainers provided was approximately 70mm in case C7 (Caltrans).

In general, the Proposed method provided a more uniform displacement response in the bridge. Similar critical displacements occur at all locations. This indicates that when the restrainers were designed with the Proposed method, the bridge is performing as a unit. In addition, the number of restrainers in the Proposed method had a good correlation with the relative displacement.

Because Caltrans and Proposed methods highly depend on elastic response spectra and because the bridge system is highly inelastic, the calculated response from these methods is approximate. The displacements predicted in the Proposed procedure for cases that had no restrainers were not close to the displacements predicted using DRAIN 3DX, which is a nonlinear analysis program.

Judging from the non-linear analysis, no restrainers are needed for the two-span bridge with a ASW of 495mm. Two-span bridges with available seat widths that meet AASHTO requirements, do not need restrainers for a 0.7g earthquake. It is also interesting to note that, in most cases, the maximum critical relative displacement at any location is less than the sum of all of the expansion joint gaps (76 mm). This is significant because the bridge can move freely before the expansion joints close.

Judging effectiveness by relative displacements is deceiving because the criterion is different for all of the design procedures. For bridges equipped with restrainers designed according to the Caltrans, AASHTO, and Proposed methods, the maximum relative displacement at any location should be close to the yield displacement of a restrainer (53.7 mm). When a minimum number of restrainers is required by the Proposed procedure, the only way to judge the effectiveness is to see if the bridge will become unseated with no restrainers. Because of that the Proposed procedure was much more effective than Caltrans because in many cases the Proposed procedure required no restrainers and the bridge never unseated without restrainers.

6.2.1.2 Restrainer Ductilities

The envelopes of the maximum restrainer displacement ductility for all cases are shown in figure 6-2. The numbers in parenthesis within the legends are the number of restrainers required by each design method. For the Caltrans, AASHTO, and W/2 methods the number of restrainers is constant at all locations on the bridge. For the Proposed method, the four numbers represent restrainers at A1-R, B2-R, B2-P, and A3-P, respectively. All of the design procedures aim at keeping restrainers elastic which means that the ductility of restrainers should be less than 1. However, the more efficient procedure would use more of the capacity of the restrainer and would lead to ductilities close to one.

Note that no case experienced significant yielding. In cases with medium strength bearings (C2, C5, and C8), restrainer ductilities at pinned locations were always lower than 0.41. In cases with strong bearings (C3, C6, and C9) restrainer ductilities at pinned locations were always lower than 0.17.

The Proposed method consistently uses more of the capacity of the restrainers because the ductilities were close but not greater than one. Yielding occurred only in one case. The AASHTO and W/2 methods lead to restrainer yielding more often. At pinned locations of cases with medium and strong bearings, restrainer ductilities never reached a value greater than 0.41. Restrainers experience little force in these locations and have little impact on the response of the bridge because bearings resist the seismic forces. Excluding medium and strong bearing locations, Caltrans ductilities ranged from 0.26 - 0.94; Proposed ductilities ranged from 0.52-1.1; AASHTO ductilities ranged from 0.42-1.2; W/2 ductilities ranged from 0.48-1.2. Note that Caltrans restrainers remained elastic in all cases.

6.2.2.3 Total Displacements

Figure 6-3 shows the sum of the total maximum relative displacements at all supports for all two-span bridges. Figure 6-4 shows the summation of maximum relative displacements at all supports. On the vertical axis, both the total number of restrainers required and total maximum relative displacements are shown. The purpose of this graph was to evaluate the effect of each design method on the total bridge movement relative to the supports.

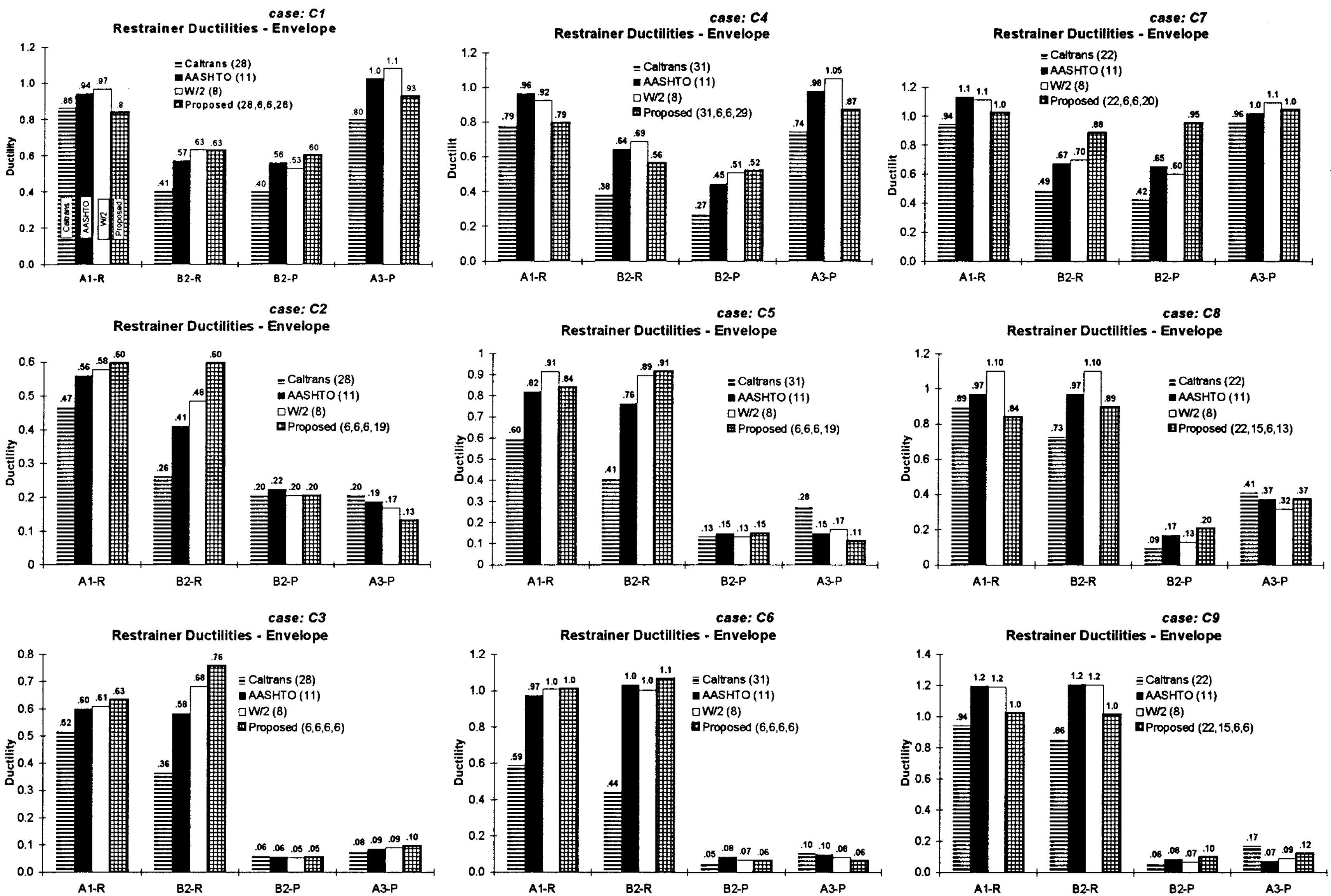


FIGURE 6-2 Envelopes of Restrainer Ductilities for Two-Span Bridges

The Proposed, AASHTO, and W/2 methods all lead to a comparable reduction in maximum relative displacement in most cases (figure 6-3). The Caltrans method reduced total relative displacement more than the others, but it required substantially more restrainers.

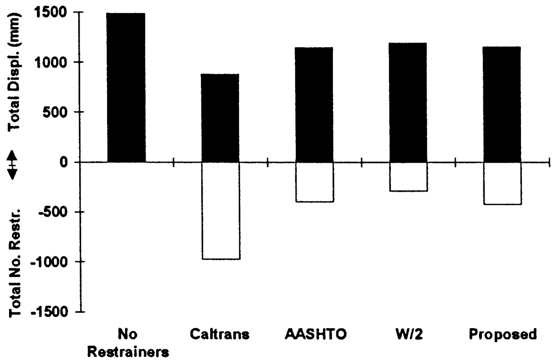


FIGURE 6-3 Sum Envelopes of Total Relative Displacements for Two-Span Bridges for All Cases

6.2.1.4 Effectiveness Factors

To provide a quantitative mean of evaluating different methods, an "effectiveness factor" was defined as shown in Eq. 6.1. This factor is the average displacement reduction per restrainer.

$$I_i = \frac{D_n - D_i}{N_i} \tag{6.1}$$

where:

I_i = effectiveness factor for method i
D_n = the total relative displacement when no restrainers are present
D_i = the total relative displacement for design method i
N_i = the number of restrainers for method i

The effectiveness factors for all two-span bridges are shown in figure 6-5. The factor indicates the reduction in relative movement per each restrainer. Larger effectiveness factors correspond to a more effective restrainer design when restrainers remain elastic and when spans remain seated. Since unseating did not occur and since restrainers only experienced limited yielding, the highest effectiveness factor corresponds to the most effective restrainer design in the two-span bridges. The W/2 method had the highest factor in five cases. The Proposed procedure had the highest factor twice. AASHTO and Caltrans each had the highest factor once.

The average effectiveness factors for all cases were as follows: W/2=1.03, AASHTO=0.85, Proposed=0.82, and Caltrans=0.64. It is interesting to note that Caltrans had about the same rating for most cases; whereas, the other cases have fluctuating ratings.

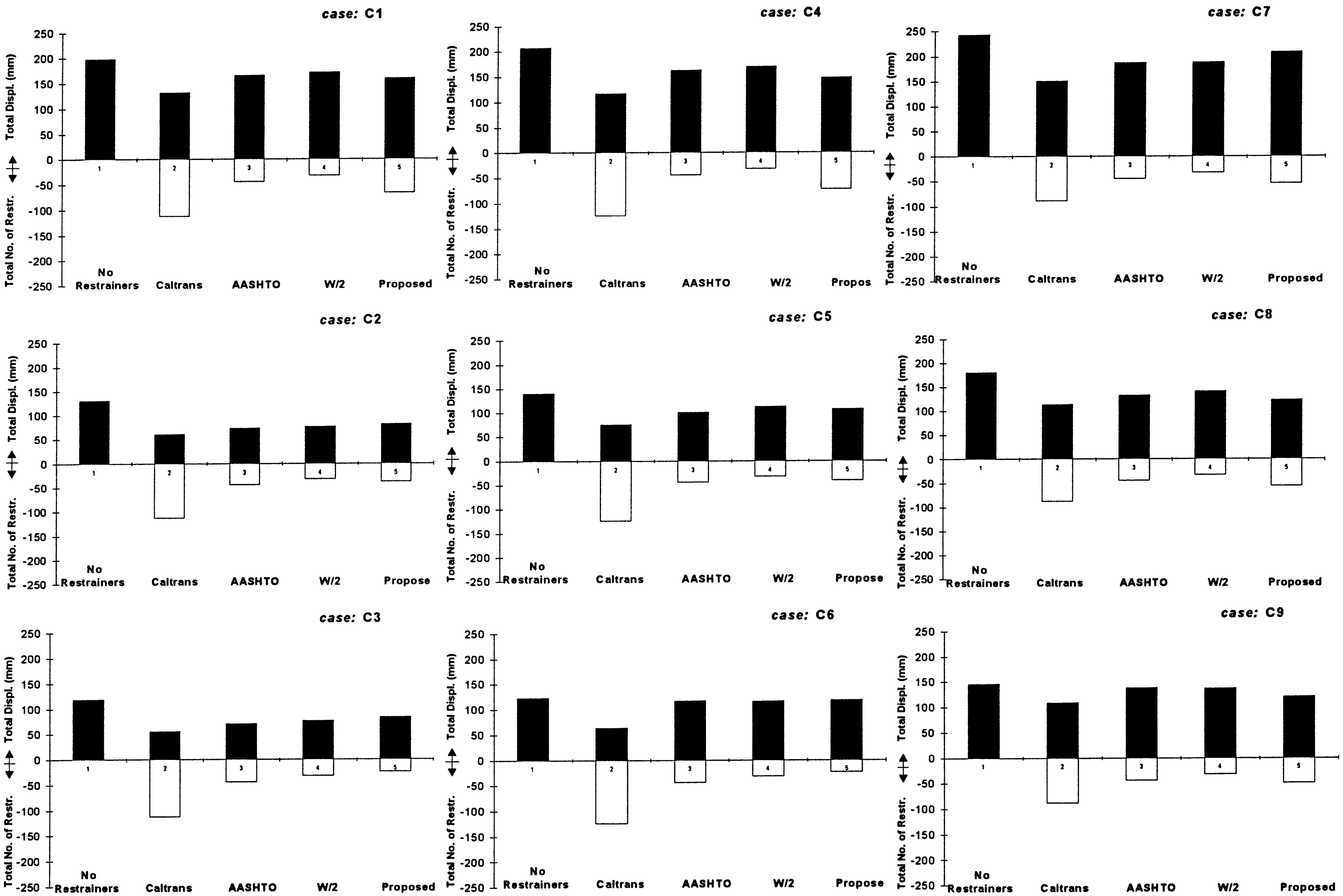


FIGURE 6-4 Envelope of Total Relative Displacements for Two-Span Bridges

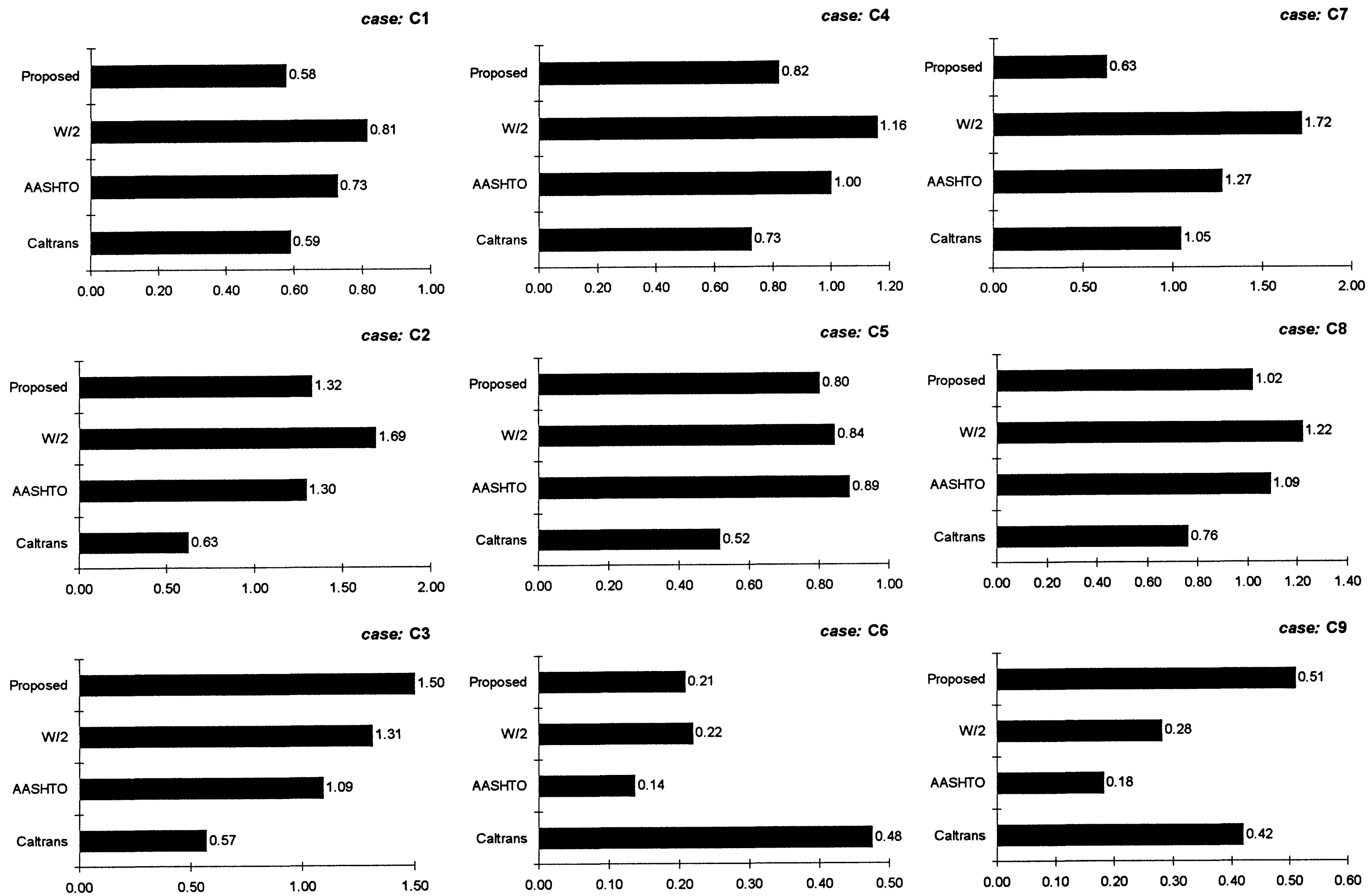


FIGURE 6-5 Effectiveness Factors for Two-Span Bridges

6.2.1.5 Bearing and Abutment Ductilities

Bearing and abutment displacement ductilities are presented in tables 6-1 and 6-2. Note that bearings in general have a relatively high elastic stiffness and that their yield displacements are very small. As a result, bearing ductilities can be very large. Higher ductilities were found with weaker bearings as expected. Weak bearing ductilities were 10-30 times larger than medium strength bearing ductilities. The values for medium strength bearings were 2-10 times larger than strong bearing ductilities. Putting restrainers on the bridge reduced the ductilities of the bearings in all cases except C9, where the ductilities remained the same. Note that these results represent the envelopes of the bearing ductilities. The ductilities in table 6-1 do not necessarily correspond to the maximum relative displacement because bearings can displace toward and away from the support. The maximum bearing ductility can correspond to the span moving toward the support, which would not be a critical relative displacement. Most cases show that the higher the number of restrainers the lower the bearing ductility demand. In case C1, the Caltrans and Proposed methods required nearly the same amount of restrainers at the abutments, and the ductilities were nearly the same. In many cases (especially C1 and C9), the bearings at the abutments had higher ductilities than the bearings at the bents. In general, putting restrainers on the bridge did not shift the higher ductility from the abutment to the bent or visa versa. In cases with strong bearings (C3,C6, and C9) and Caltrans restrainers, the bearings did not yield at the bent.

TABLE 6-1 Bearing Displacement Ductilities for Two-Span Bridges

Bearing	*No Restrainers*	*AASHTO*	*Caltrans*	*W/2*	*Proposed*
			case: C1		
B2-P	163	117	99	128	99
A3-P	203	162	126	171	147
			case: C2		
B2-P	16.3	6.2	5.7	8.4	10.6
A3-P	16.3	7.7	5.7	10.6	12.3
			case: C3		
B2-P	4.50	1.02	0.95	1.19	1.34
A3-P	7.60	3.71	1.23	4.27	4.66
			case: C4		
B2-P	155	102	72	128	92
A3-P	201	155	118	167	139
			case: C5		
B2-P	10.7	4.3	3.6	4.7	6.3
A3-P	18.6	17.1	8.4	17.5	17.6
			case: C6		
B2-P	2.24	1.35	0.81	1.58	1.61
A3-P	9.01	8.65	3.26	8.98	8.65
			case: C7		
B2-P	236	132	105	114	172
A3-P	189	161	152	173	165
			case: C8		
B2-P	22.1	4.6	2.8	4.0	5.9
A3-P	21.0	17.7	17.4	18.2	16.6
			case: C9		
B2-P	2.42	1.35	0.95	1.31	1.58
A3-P	10.59	10.67	9.05	10.61	9.78

Table 6-2 also shows abutment displacement ductilities. Note that restrainers did not necessarily reduce the demand on the abutment. For example, in cases C1 and C4, restrainers reduced the ductility demand in abutment A1 but increased it in A3.

TABLE 6-2 Abutment Displacement Ductilities for Two-Span Bridges

Abutment	*No Restrainers*	*AASHTO*	*Caltrans*	*W/2*	*Proposed*
			case: C1		
A1	1.20	1.03	0.87	1.04	0.98
A3	0.88	1.42	1.06	1.20	1.33
			case: C2		
A1	0.59	0.56	0.55	0.59	0.59
A3	0.51	0.00	0.00	0.00	0.00
			case: C3		
A1	0.73	0.53	0.31	0.54	0.66
A3	0.03	0.00	0.00	0.00	0.00
			case: C4		
A1	1.36	1.13	0.98	1.21	1.06
A3	1.08	1.64	1.07	1.49	1.30
			case: C5		
A1	0.92	1.13	0.63	0.96	0.92
A3	0.93	0.65	0.00	0.73	0.75
			case: C6		
A1	0.97	1.14	0.53	0.88	0.84
A3	0.52	0.40	0.00	0.51	0.40
			case: C7		
A1	1.69	1.65	1.57	1.48	1.48
A3	1.59	1.43	1.51	1.63	1.49
			case: C8		
A1	1.85	1.13	0.95	1.19	1.17
A3	1.39	0.77	0.73	0.86	0.56
			case: C9		
A1	1.47	1.09	0.95	1.23	1.04
A3	1.07	1.09	0.53	1.07	0.79

Restrainers did not significantly affect the ductility at A1, in case C2. Still, the demand on A1 for bridges with less restrainers was higher than the demand on the abutment for bridges with more restrainers. Abutment A3 was not even engaged when restrainers were added. Neither abutments yielded regardless of the number of restrainers.

Note that A3 was not engaged in case C3. At abutment A1, increasing the number of restrainers reduced the abutment ductility proportionally. Neither abutment yielded.

In case C5 and C6, the effect of restrainers was not uniform. However, at abutment A3, the demand was reduced when restrainers were added. The right abutment did not engage when Caltrans restrainers were added in case C5.

The ductility did not change significantly when restrainers were added in case C7, but the effect was generally significant in C8 and C9. In general, with increased flexibility the demand on the abutments increased. Bearing strength did not affect abutment ductilities significantly on the roller sides; however, on the pinned sides higher bearing strength reduced the demand on the abutment.

6.2.1.6 Other Responses

Figure 6-6 shows the effect of bearing strength and substructure stiffness on the response of bridges without restrainers. The vertical axis shows the sum of relative displacements at all supports on the two span bridge, and the horizontal axis shows the stiffness of the substructure at bent 2. Note that bearing strength affects the relative displacements more than the substructure stiffness does. For a given substructure stiffness the difference in the relative displacements between strong and weak bearings was as much as 100mm. Whereas, for a given bearing strength, the difference in relative displacements between the stiffest and most flexible cases was 50mm or less.

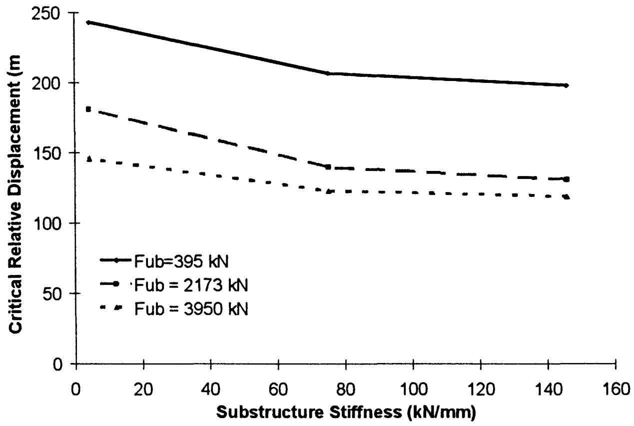


FIGURE 6-6 Envelopes of Total Relative Displacements for Two-Span Bridges Without Restrainers

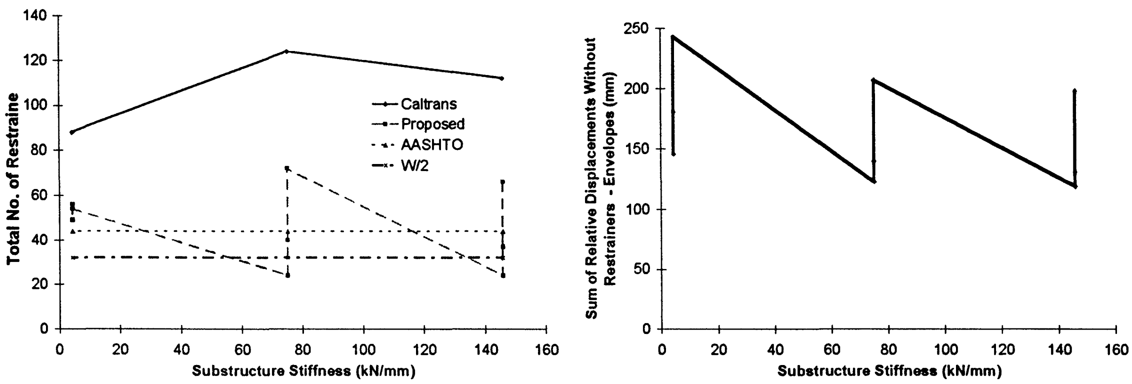


FIGURE 6-7 Effect of Substructure Stiffness on the Number of Restrainers and on Total Displacements of Unrestrained Bridges

The effects of substructure stiffness on the number of restrainers for each design method and the sum of relative displacements of the structure are shown in figure 6-7. Notice that the total number of restrainers required by AASHTO and W/2 does not change with substructure stiffness. The effect on the number of restrainers required by Caltrans is small. However, the Proposed method requires different numbers of restrainers for different stiffnesses. In fact, the number of restrainers changes with the changing displacement demand of the unrestrained structure. This is evident from the fact that the shape of the plot of the total number of restrainers required by the Proposed method closely matches the shape of the plot of the sum of relative displacements without restrainers. The close correlation indicates that the Proposed method is more rational than the others.

6.2.2 Five-Span Bridge

6.2.2.1 Relative Displacements at Supports

For all five-span bridge cases the maximum relative displacements between the girder and the support which can cause unseating were analyzed. In figures 6-8, 6-9, and 6-10, the envelopes of maximum relative displacements, including results with and without the Kobe earthquake are presented. The results without the Kobe Earthquake are included because the Kobe record controlled the envelopes in many cases. The plots that exclude Kobe were reviewed to determine if the trends were affected by the Kobe record. The solid bold line represents the maximum relative displacements for cases with no restrainers. The available seat width is presented with dotted bold line.

Generally, the relative displacements were smaller in structures with restrainers. In a limited number of cases, restrainers increased displacements at the abutments slightly. That increase was caused by the more uniform distribution of inertial forces throughout the structure with restrainers.

Since it was assumed that pinned bearings could carry a force after yielding and roller bearings did not carry any horizontal force, relative displacements were smaller at the pinned ends than at the roller sides. When the strength of the pinned bearings was increased, the difference between displacements on pinned and roller ends increased. While the larger strength of pinned bearings decreased displacements at pinned ends, it increased the displacements on the roller side.

In structures with weak bearings (C1, C4, C7), the difference in displacements on the roller and pinned ends was not very large. The differences were especially small in structures with restrainers. With the exception of abutments, distribution of displacements throughout the structures with restrainers was uniform. Generally, relative displacements were larger in structures with more flexible bents. Restrainers were effective at reducing displacements at the roller locations but were less effective at the pinned locations.

In structures with larger substructure flexibility (C4 and C7), the effectiveness of restrainers was especially large at B2-R. In all cases with weak bearings, restrainers were less effective at reducing displacements at the abutments. In case C1 the addition of restrainers increased displacements at the left abutment (A1).

All methods led to a similar responses at most supports even though the number of restrainers was different in structures with weak bearings. Larger differences were observed at the abutments. Differences in displacements among different methods were larger in cases with larger substructure flexibility. In all cases with weak bearings, the Caltrans design method led to the smallest relative displacements (with the exception of abutments) because the number of restrainers was considerably larger compared to other methods.

Generally, the Proposed method was more efficient at reducing displacements than other methods. Relative displacements were not very different compared to those obtained with the Caltrans method in spite of the fewer number of restrainers. Furthermore, the Proposed method led to a more uniform distribution of displacements throughout the bridge.

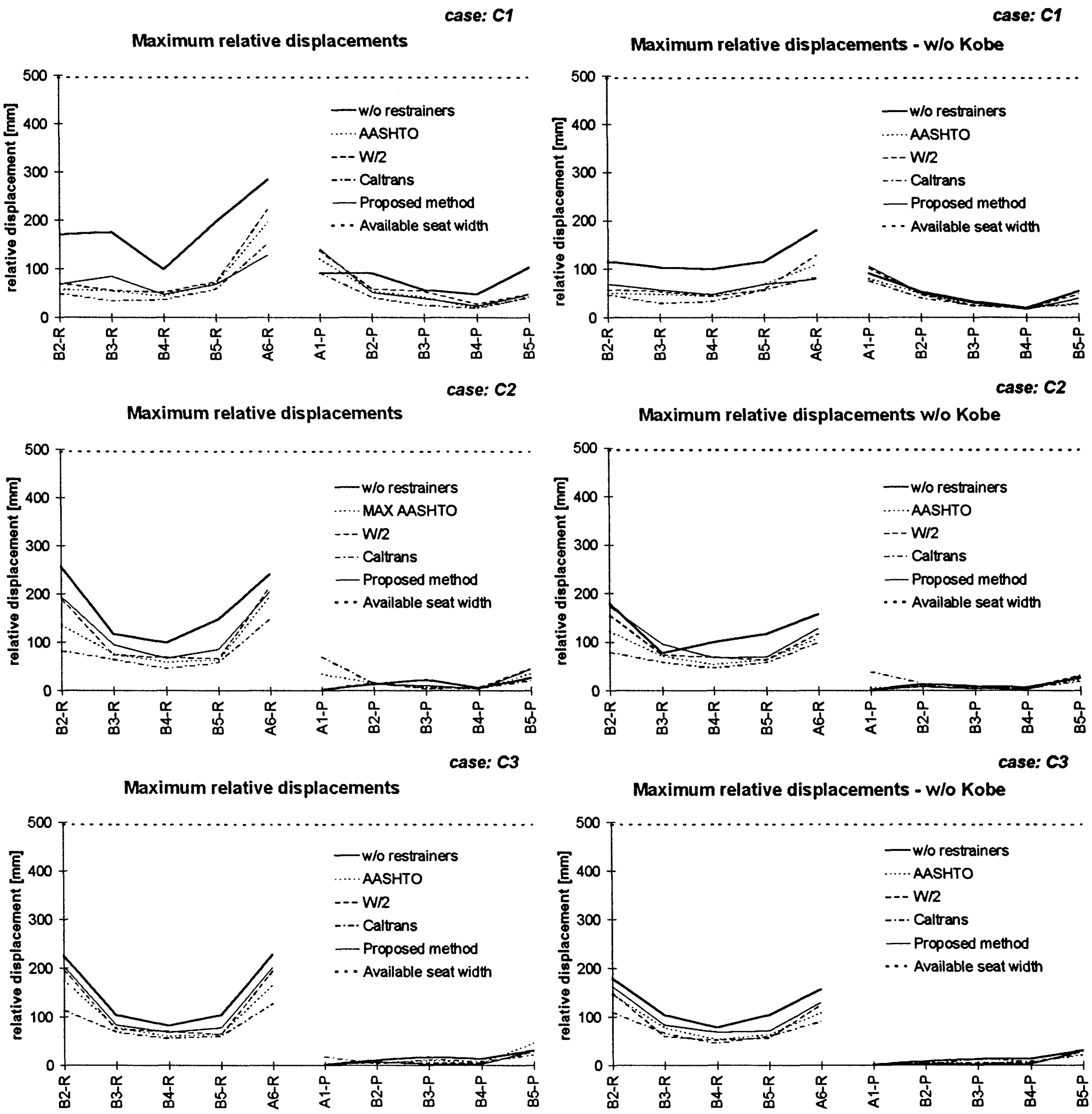


FIGURE 6-8 Envelopes of Relative Displacements in Cases C1 to C3 for Five-Span Bridges

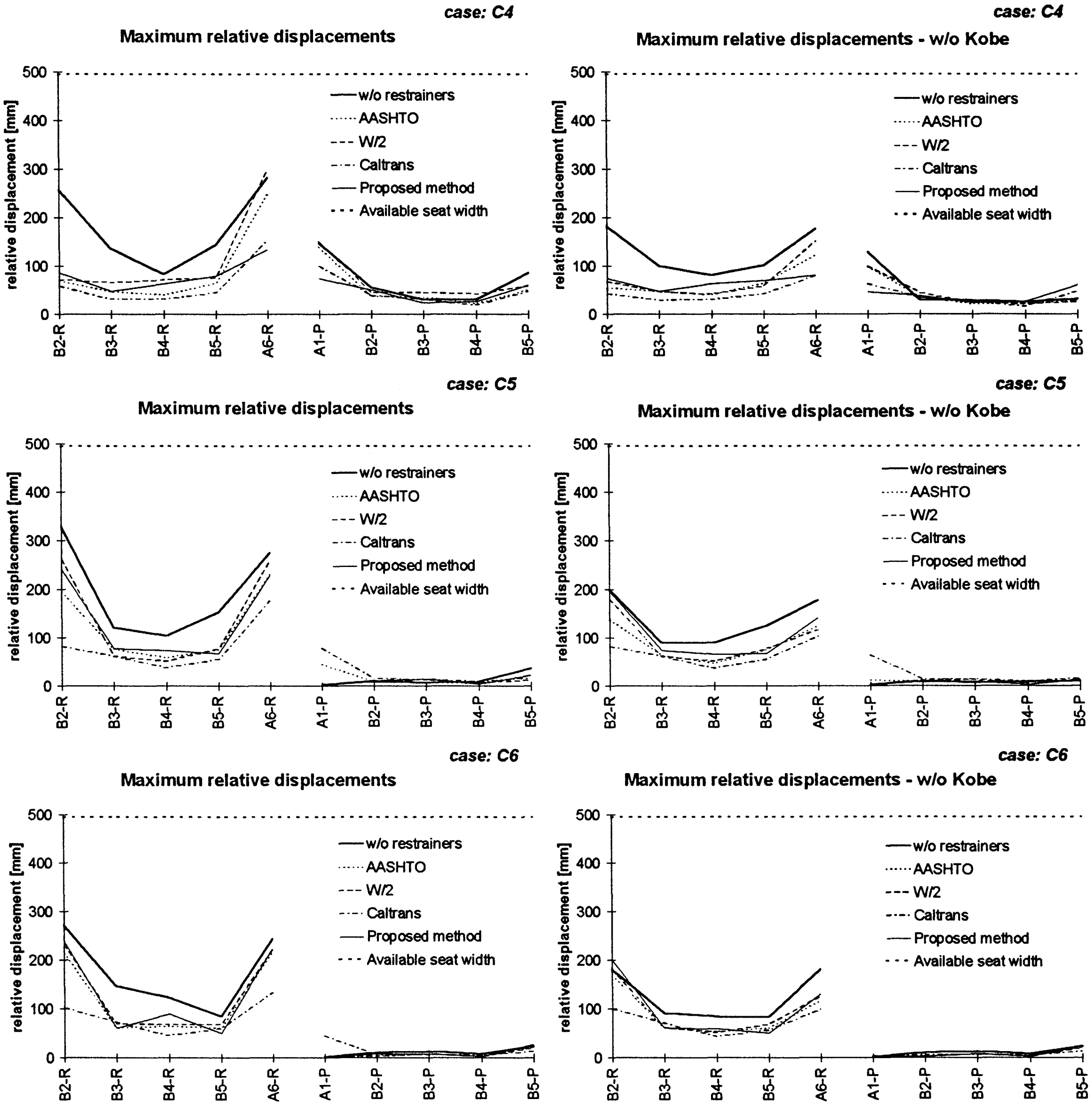


FIGURE 6-9 Envelopes of Relative Displacements in Cases C4 toC6 for Five-Span Bridges

The AASHTO and W/2 methods led to a similar response, which was not different from the response of the Caltrans and the Proposed methods. At the right abutment (A6), displacements were larger than those obtained from Caltrans and Proposed methods. However, they were still smaller than the available seat width of 495 mm.

In structures with stronger pinned bearings (C2-C3, C5-C6 and C8-C9), relative displacements at the pinned ends were small compared to those obtained at the roller ends. Relative displacements at the left abutment (A1) were significantly decreased in comparison with structures with weak bearings. Displacements at bent B2-R were increased even in structures with restrainers. Bridges with restrainers designed according to the Caltrans method led to a response that was similar to those obtained in structures with weak bearings.

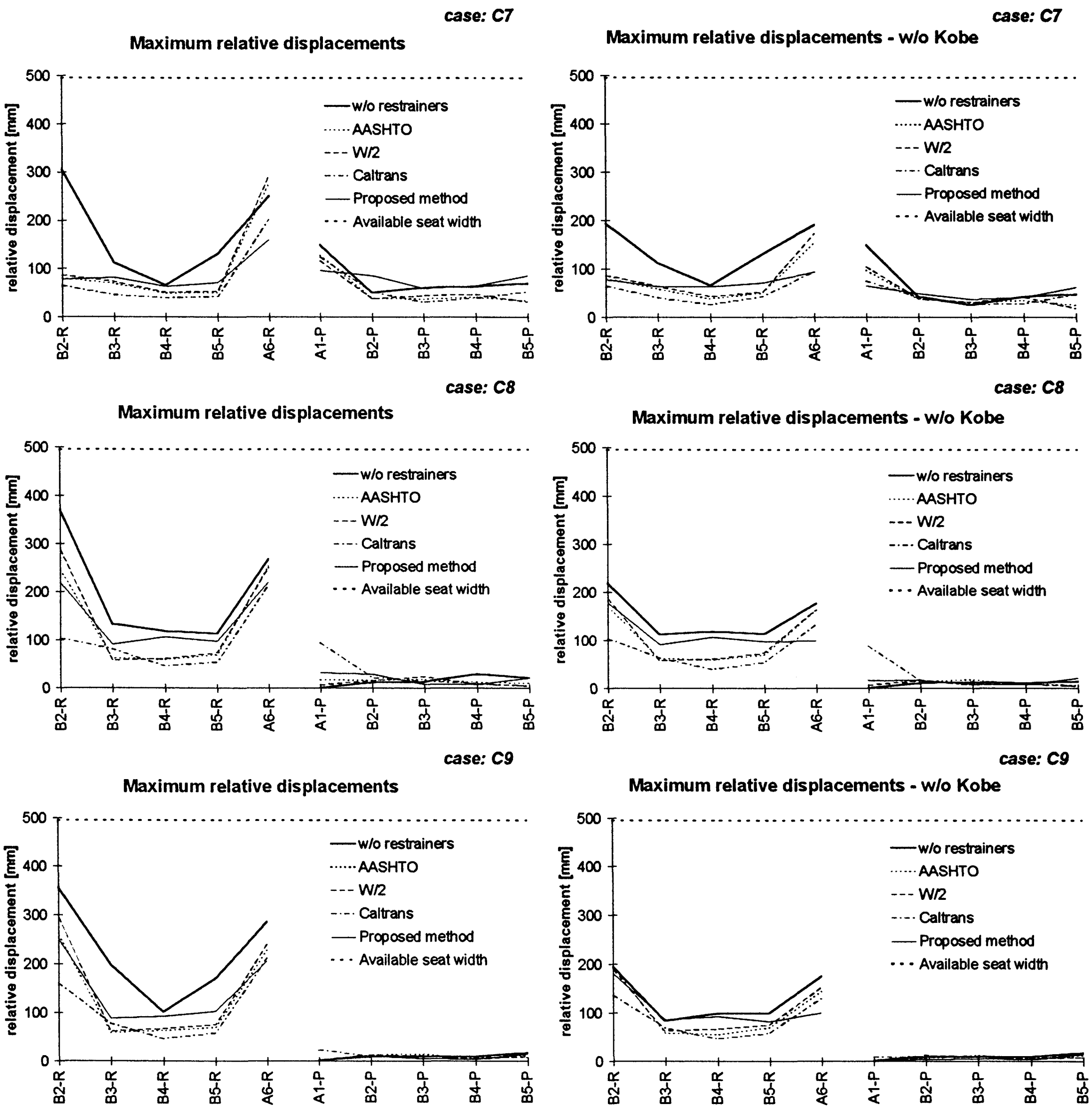


FIGURE 6-10 Envelopes of Relative Displacements in Cases C7 to C9 for Five-Span Bridges

With the exception of abutments and bent B2-R, there were no significant differences in displacements for all methods. In bridges with very flexible substructure (C8 and C9), at some locations, the Proposed method led to displacements which were larger than those obtained from other methods. Due to a large number of restrainers, the Caltrans design procedure led to the smallest displacements and was especially effective at B2-R. However the Caltrans method typically resulted in the largest displacement at the left abutment (A1-P), even exceeding the displacement for the cases with no restrainers. The other three methods led to similar responses. In most cases, the shape of the displacement line was similar to that obtained in structures with no restrainers, but displacements were smaller.

Note that in all cases including structures with no restrainers, the maximum relative displacements between the deck and substructure were smaller than the available seat width. Nonetheless the results of the study revealed the relative merit of different methods.

6.2.2.2 Restrainer Ductilities

Besides the maximum relative displacements, restrainer displacement ductilities were compared for all bridges. The maximum values of restrainer ductilities (ratio of displacements over yield displacement) are summarized in table 6-3 and 6-4.

Only at a few locations, restrainers yielded, regardless of whether or not the Kobe record was included. The extent of yielding was limited in all cases. In bridges with weak bearings, restrainers from all methods yielded at the right abutment (A6). Ductilities were larger in cases with flexible substructures (cases C4 to C9). The Proposed method led to the smallest ductility demands, with a maximum value of 1.51 (case C7). The Caltrans method led to yielding which was not larger than 1.81 (obtained also in case C7). The other two methods resulted in larger ductility demand, but those values were still relatively small (AASHTO = 2.58 in case C7 and W/2 = 2.78 in case C4).

In structures with weak bearings (C1, C4 and C7), limited yielding of restrainers was observed at the left abutment (A1-P). The Proposed method led to restrainer yielding in case C1 (1.31) and Caltrans method in case C7 (1.17). Restrainers designed according to the AASHTO and W/2 methods yielded in all three cases (C1, C4 and C7), but the extent of yielding was limited. For the AASHTO method the maximum ductility was 1.3 (case C4) and for W/2 method it was 1.41 (case C4).

In structures with stronger bearings, restrainers yielded at the right abutment and at B2 on the roller side. The AASHTO, W/2, and Proposed methods led to similar values of restrainer ductilities with a maximum value of 2.75. Ductility of restrainers designed according to the Caltrans method was limited to 1.99.

Yielding of restrainers was not observed on the pinned ends of the bridges with stronger bearings because the displacements were small. Restrainer ductilities were larger in bridges with more flexible bents, with the exception of the Proposed method which led to the maximum demand in cases with medium substructure flexibility.

If results obtained from the Kobe record were excluded, the maximum values of restrainer ductilities were smaller. In bridges with weak bearings, restrainers designed according to the Caltrans and Proposed methods did not yield in any case. The other two methods led to yielding only at the right abutment (A6-R) which was largest in the bridge with the most flexible substructure (case C7). Restrainer ductility did not exceed 1.45 for the AASHTO method and 1.6 for the W/2 method.

TABLE 6-3 Maximum Restrainer Ductilities (including Kobe) for Five-Span Bridges

	AASHTO	*Caltrans*	*W/2*	*Proposed method*	*AASHTO*	*Caltrans*	*W/2*	*Proposed method*	*AASHTO*	*Caltrans*	*W/2*	*Proposed method*
		case:	***C1***			*case:*	***C4***			*case:*	***C7***	
B2-R	0.54	0.46	0.66	0.64	0.64	0.55	0.67	0.80	0.75	0.61	0.81	0.74
B3-R	0.51	0.32	0.52	0.80	0.44	0.29	0.61	0.44	0.65	0.42	0.69	0.76
B4-R	0.41	0.34	0.48	0.45	0.37	0.29	0.67	0.59	0.46	0.37	0.48	0.60
B5-R	0.67	0.55	0.70	0.64	0.60	0.42	0.71	0.74	0.46	0.39	0.49	0.66
A6-R	***1.82***	***1.43***	***2.07***	***1.21***	***2.31***	***1.42***	***2.78***	***1.26***	***2.58***	***1.88***	***2.72***	***1.51***
A1-P	***1.14***	0.87	***1.30***	***1.31***	***1.30***	0.94	***1.41***	0.70	***1.21***	***1.17***	***1.09***	0.91
B2-P	0.51	0.38	0.55	0.48	0.34	0.36	0.42	0.46	0.36	0.49	0.35	0.80
B3-P	0.39	0.24	0.50	0.37	0.33	0.29	0.42	0.23	0.34	0.29	0.42	0.56
B4-P	0.19	0.18	0.26	0.22	0.21	0.18	0.39	0.25	0.38	0.37	0.44	0.61
B5-P	0.39	0.39	0.45	0.45	0.47	0.45	0.56	0.56	0.31	0.49	0.30	0.80
		case:	***C2***			*case:*	***C5***			*case:*	***C8***	
B2-R	***1.27***	0.77	***1.76***	***1.81***	***1.82***	0.77	***2.43***	***2.25***	***2.28***	0.97	***2.69***	***2.05***
B3-R	0.72	0.61	0.69	0.89	0.71	0.58	0.58	0.73	0.58	0.76	0.54	0.86
B4-R	0.55	0.43	0.64	0.64	0.55	0.36	0.48	0.69	0.55	0.42	0.57	1.00
B5-R	0.59	0.54	0.62	0.80	0.70	0.52	0.72	0.63	0.64	0.50	0.69	0.91
A6-R	***1.82***	***1.39***	***1.99***	***1.92***	***2.14***	***1.65***	***2.41***	***2.14***	***2.37***	***1.99***	***2.34***	***2.06***
A1-P	0.32	0.66	0.02	0.02	0.42	0.75	0.03	0.02	0.16	0.89	0.06	0.30
B2-P	0.14	0.13	0.16	0.12	0.08	0.15	0.11	0.10	0.15	0.21	0.14	0.27
B3-P	0.06	0.06	0.04	0.10	0.07	0.13	0.12	0.14	0.17	0.13	0.23	0.07
B4-P	0.03	0.05	0.05	0.04	0.05	0.09	0.06	0.04	0.11	0.09	0.07	0.08
B5-P	0.33	0.20	0.43	0.42	0.21	0.16	0.11	0.21	0.09	0.04	0.05	0.20
		case:	***C3***			*case:*	***C6***			*case:*	***C9***	
B2-R	***1.64***	***1.07***	***1.82***	***1.90***	***1.99***	0.95	***2.16***	***2.23***	***2.41***	***1.51***	***2.75***	***2.34***
B3-R	0.73	0.64	0.70	0.78	0.57	0.68	0.65	0.57	0.54	0.73	0.58	0.83
B4-R	0.56	0.52	0.66	0.64	0.60	0.42	0.64	0.84	0.59	0.43	0.62	0.86
B5-R	0.60	0.56	0.60	0.73	0.56	0.56	0.64	0.48	0.65	0.54	0.70	0.97
A6-R	***1.55***	***1.20***	***1.81***	***1.90***	***2.02***	***1.25***	***2.06***	***2.09***	***2.12***	***1.97***	***2.23***	***1.94***
A1-P	0.02	0.17	0.02	0.02	0.02	0.43	0.02	0.02	0.02	0.22	0.02	0.02
B2-P	0.05	0.03	0.07	0.07	0.03	0.05	0.03	0.08	0.12	0.09	0.12	0.11
B3-P	0.09	0.11	0.04	0.03	0.07	0.06	0.07	0.07	0.14	0.09	0.10	0.05
B4-P	0.05	0.08	0.04	0.03	0.03	0.05	0.03	0.03	0.04	0.07	0.04	0.04
B5-P	0.42	0.20	0.26	0.28	0.20	0.13	0.19	0.26	0.12	0.07	0.09	0.14

TABLE 6-4 Maximum Restrainer Ductilities (excluding Kobe) for Five-Span Bridges

	AASHTO	*Caltrans*	*W/2*	*Proposed method*	*AASHTO*	*Caltrans*	*W/2*	*Proposed method*	*AASHTO*	*Caltrans*	*W/2*	*Proposed method*
		case:	*C1*			*case:*	*C4*			*case:*	*C7*	
B2-R	0.46	0.44	0.53	0.64	0.51	0.40	0.64	0.69	0.75	0.61	0.81	0.74
B3-R	0.44	0.27	0.50	0.53	0.44	0.27	0.44	0.44	0.54	0.38	0.60	0.60
B4-R	0.41	0.31	0.41	0.45	0.37	0.29	0.39	0.59	0.35	0.25	0.40	0.60
B5-R	0.65	0.53	0.54	0.64	0.60	0.39	0.55	0.66	0.46	0.39	0.44	0.66
A6-R	***1.02***	0.78	***1.20***	0.75	***1.15***	0.74	***1.42***	0.75	***1.45***	0.89	***1.60***	0.88
A1-P	0.75	0.71	0.96	0.99	0.93	0.59	0.94	0.44	0.92	0.70	0.99	0.60
B2-P	0.46	0.38	0.44	0.47	0.34	0.32	0.42	0.36	0.35	0.38	0.35	0.45
B3-P	0.24	0.24	0.23	0.29	0.20	0.24	0.23	0.23	0.29	0.25	0.29	0.34
B4-P	0.19	0.18	0.18	0.17	0.21	0.15	0.19	0.24	0.30	0.24	0.33	0.35
B5-P	0.27	0.26	0.45	0.37	0.26	0.45	0.25	0.56	0.23	0.45	0.17	0.58
		case:	*C2*			*case:*	*C5*			*case:*	*C8*	
B2-R	***1.15***	0.74	***1.46***	***1.62***	***1.29***	0.77	***1.68***	***1.84***	***1.58***	0.97	***1.73***	***1.66***
B3-R	0.66	0.55	0.69	0.89	0.59	0.58	0.58	0.69	0.58	0.59	0.54	0.86
B4-R	0.50	0.43	0.64	0.64	0.44	0.35	0.48	0.62	0.55	0.36	0.57	1.00
B5-R	0.59	0.54	0.59	0.66	0.70	0.52	0.72	0.63	0.64	0.50	0.69	0.91
A6-R	***1.01***	0.92	***1.10***	***1.20***	***1.17***	0.97	***1.09***	***1.33***	***1.53***	***1.25***	***1.54***	0.93
A1-P	0.06	0.37	0.02	0.02	0.11	0.61	0.03	0.02	0.16	0.84	0.06	0.16
B2-P	0.07	0.13	0.09	0.08	0.08	0.13	0.11	0.10	0.13	0.14	0.14	0.17
B3-P	0.05	0.06	0.04	0.04	0.07	0.13	0.12	0.09	0.17	0.07	0.09	0.07
B4-P	0.03	0.05	0.05	0.03	0.03	0.09	0.06	0.03	0.07	0.08	0.07	0.08
B5-P	0.21	0.18	0.30	0.29	0.13	0.16	0.11	0.12	0.06	0.04	0.05	0.20
		case:	*C3*			*case:*	*C6*			*case:*	*C9*	
B2-R	***1.37***	***1.03***	***1.41***	***1.52***	***1.57***	0.93	***1.68***	***1.87***	***1.80***	***1.28***	***1.76***	***1.68***
B3-R	0.73	0.62	0.55	0.78	0.57	0.67	0.58	0.57	0.54	0.64	0.58	0.80
B4-R	0.49	0.42	0.48	0.64	0.50	0.40	0.48	0.56	0.51	0.43	0.62	0.86
B5-R	0.59	0.56	0.52	0.67	0.56	0.53	0.64	0.48	0.65	0.54	0.70	0.77
A6-R	***1.02***	0.85	***1.17***	***1.22***	***1.09***	0.92	***1.18***	***1.22***	***1.35***	***1.22***	***1.43***	0.94
A1-P	0.02	0.02	0.02	0.02	0.02	0.03	0.02	0.02	0.02	0.08	0.02	0.02
B2-P	0.05	0.03	0.03	0.03	0.03	0.05	0.03	0.03	0.06	0.09	0.12	0.04
B3-P	0.05	0.11	0.04	0.03	0.07	0.06	0.07	0.07	0.12	0.09	0.09	0.05
B4-P	0.05	0.08	0.04	0.03	0.03	0.05	0.03	0.03	0.04	0.07	0.04	0.04
B5-P	0.19	0.20	0.26	0.28	0.20	0.13	0.19	0.23	0.12	0.07	0.08	0.14

In structures with stronger bearings, yielding of restrainers was observed at the same locations as in the cases when results obtained from the Kobe record were considered. The AASHTO, W/2 and Proposed methods led to similar displacement ductility demand with a maximum of 1.87. Ductilities of restrainers designed according to the Caltrans method did not exceed 1.28.

It was shown in previous paragraphs that limited yielding of restrainers occurred at some locations even though restrainers were designed to carry seismic force without yielding. For the AASHTO and W/2 methods, restrainer yielding can be expected because seismic design force excludes the effects of many important factors on the earthquake response. It is interesting that W/2 and AASHTO only experienced limited yielding despite their crudeness.

In the Caltrans and Proposed methods, restrainer forces were determined based on the equivalent period of the structure using the Caltrans design spectra. It is evident from figures 3-9 and 4-12 that the Caltrans spectra are different from acceleration spectra used in this study. The Kobe acceleration spectrum, with two peaks, is significantly different from the Caltrans design spectra. Furthermore, the spectral methods used in the Caltrans and the Proposed methods assume a linear system with constant structural period; whereas the response history analyses used in the study account for the non-linearity of hinges and abutments. Because of different spectra and system non-linearity, the exact seismic force may be different from the design force used in the Caltrans and Proposed methods. Therefore, the actual displacements can be larger than expected and yielding can occur.

Restrainers can yield because of other reasons. The Caltrans and Proposed design methods may not determine the exact maximum response. The Caltrans design procedure neglects many important parameters that influence bridge response. Therefore, the period calculated with these methods, which is used to determine restrainers forces, is only an approximation of the actual value.

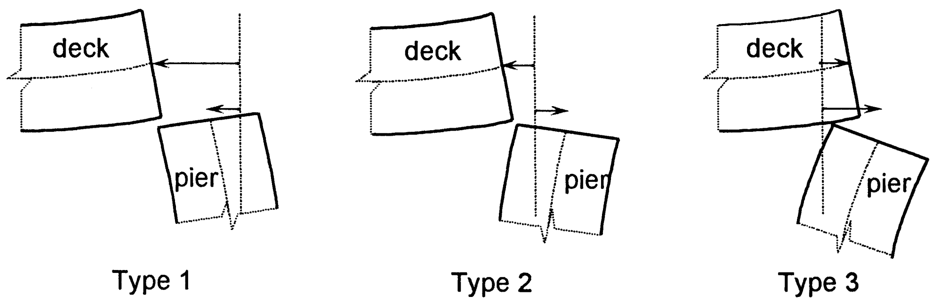


FIGURE 6-11 Possible Types of Superstructure Unseating

In the Proposed method, the estimation of structural period is more accurate but not exact. For example, in the Proposed method, impact between adjacent frames and closure of gaps is considered only at the abutments. Only one type of unseating is considered at roller locations when designing restrainers. That is when the superstructure and the substructure move in the same direction but displacement of the superstructure is larger. However, this is not the only type of unseating that can occur in the bridge. Two additional types of unseating, which are presented in figure 6-11 (Type 2 and Type 3), can also occur. These two are included in the Drain 3DX models and can be the reason for restrainer yielding.

As mentioned previously, the Caltrans method led to the smallest restrainer ductilities and the smallest relative displacements. However, the method always required substantially more restrainers than the other methods. It was also observed that the maximum displacements were always smaller than available seat width. Therefore, the large number of restrainers based on the Caltrans method did not improve the response appreciably.

6.2.2.3 Total Displacements

It was shown that the response of structures with restrainers designed according to different methods was similar. Therefore, to evaluate the relative performance of different methods, the total number of restrainers and the sum of maximum relative displacements at all supports were compared. The results are presented in table 6-5 and in figure 6-12. In this figure, the bars above the reference line show the total displacement in the structure, and bars below show the total number of restrainers. The Kobe record is included in the table and the figure.

TABLE 6-5 Total Number of Restrainers and Sum of Maximum Displacements for Five-Span Bridges

case	*total number of restrainers*				*sum of maximum displacements [mm]*				
	AASHTO	*Caltrans*	*W/2*	*Prop. meth*	*AASHTO*	*Caltrans*	*W/2*	*Prop. meth.*	*No restr.*
C1	220	440	164	159	703	552	802	705	1316
C2	220	440	164	120	624	519	685	722	935
C3	220	440	164	120	612	490	644	683	817
C4	220	564	164	225	750	555	925	647	1251
C5	220	564	164	135	723	551	756	742	1047
C6	220	564	164	144	651	491	695	714	927
C7	220	446	164	186	803	694	833	851	1260
C8	220	446	164	169	763	643	789	835	1080
C9	220	446	164	160	721	613	777	780	1159

The Proposed method led to the least number of restrainers in most cases. Even in other cases the numbers were close to the minimum. In all cases the Caltrans design procedure led to a substantially larger number of restrainers.

6.2.2.4 Effectiveness Factors

Effectiveness factors as defined in equation 6.1 for the five-span bridge are presented in table 6-6. Considering the results presented in the table, it is evident that the effectiveness of the Caltrans method was the smallest. The values of effectiveness factors obtained for the other three methods are similar. However, the Proposed method was the most effective in all cases except C7 and C8, where the W/2 method had the highest effectiveness factor. All methods had relatively high effectiveness factors in cases with weak bearings (structures C1, C4 and C7). Especially high values were obtained in structure C1 (with weak bearings and stiff columns). The Proposed method had the highest average effectiveness factor, required the least number of restrainers, and kept the bridge seated without significant yielding in restrainers. Caltrans had the lowest average effectiveness factor and required the most number of restrainers. Note that the effectiveness factors for the W/2 and

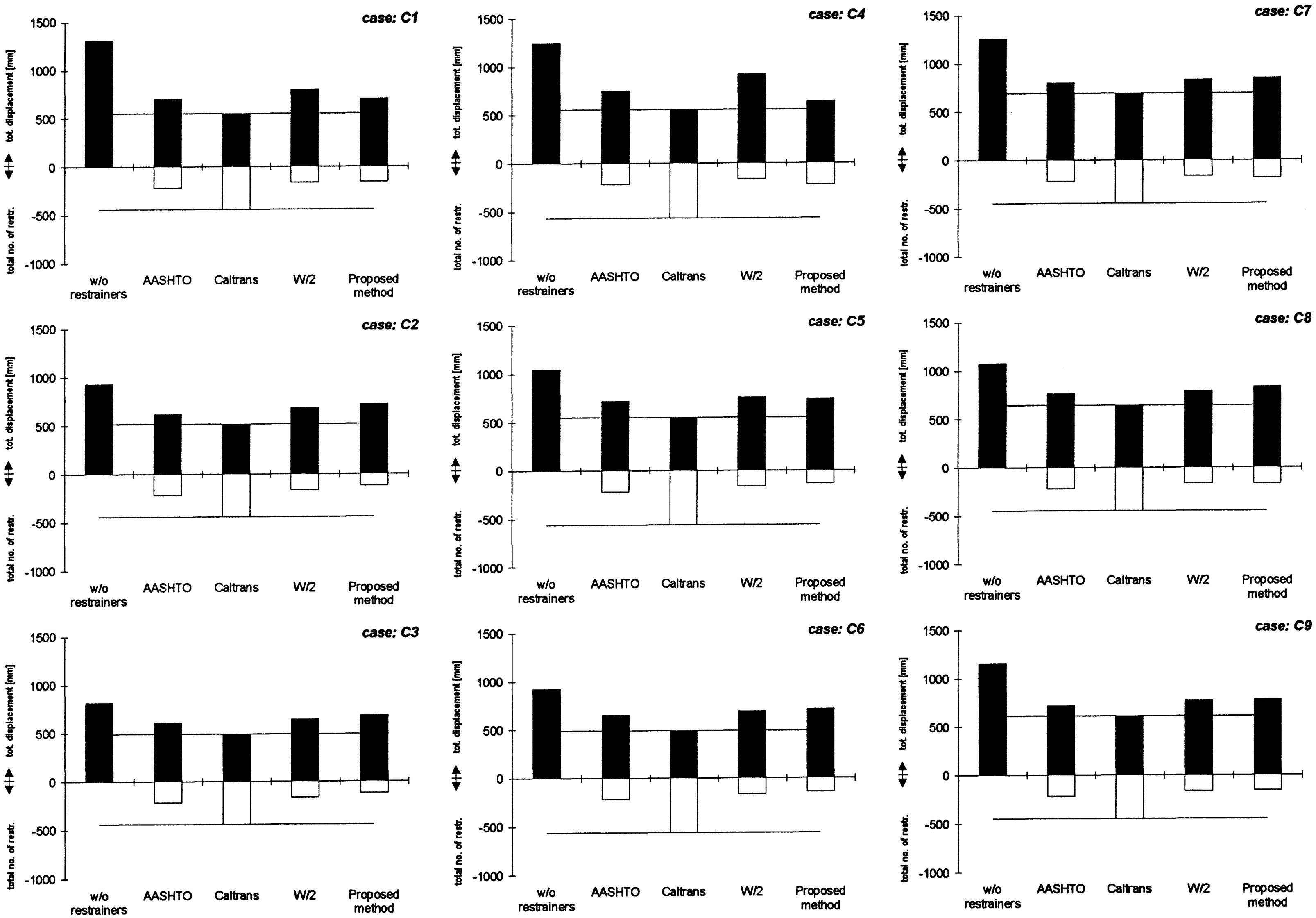


FIGURE 6-12 Envelopes of Total Relative Displacements (including Kobe) for Five-Span Bridges

Proposed methods were not that different even though the Proposed method accounts for more bridge elements.

TABLE 6-6 Effectiveness Factors for Five-Span Bridges

case	*AASHTO*	*Caltrans*	*W/2*	*Prop. Meth.*
C1	2.79	1.74	3.14	3.84
C2	1.41	0.94	1.52	1.77
C3	0.93	0.74	1.06	1.12
C4	2.28	1.23	1.99	2.69
C5	1.48	0.88	1.78	2.26
C6	1.25	0.77	1.41	1.48
C7	2.08	1.27	2.61	2.20
C8	1.44	0.98	1.77	1.45
C9	1.99	1.22	2.33	2.37
Average	**1.74**	**1.09**	**1.96**	**2.13**

6.2.2.5 Bearing and Abutment Ductilities

It was mentioned before, that abutments play an important role in the response of simply-supported bridges. Since their stiffness is usually very large, they can significantly reduce the displacements of a structure. The only restrainer design method that considers abutments is the Proposed method. Therefore, abutment ductilities were analyzed.

The maximum abutment displacement ductility demands are listed in table 6-7. When results from the Kobe record were considered, the maximum ductilities were large. Abutment A1 experienced especially large ductilities. Note that different design methods led to comparable values of abutment ductility demands in most cases. Abutment ductilities in structures with no restrainers were close to structures with restrainers. In most of the cases, the ductility obtained for the left abutment was larger than that in the right one. While variation of bearing strength had little influence on ductility of the right abutment (A6, with roller bearing), it changed the ductility demand at the left abutment (A1, with pinned bearing). Ductility of the left abutment was largest in bridges with weak bearings, because of high bearing ductility (table 6-8). When bearing strength was increased, the difference in ductility demand between the left and right abutment decreased.

Flexibility of substructure had a small influence on abutment ductility demand. However, in most cases, a larger ductility was obtained in structures with more flexible piers. Exceptions were structures with weak bearings, where the largest abutment ductility was obtained in bridges with medium pier flexibility.

When results obtained from the Kobe record were excluded, the maximum values of abutment ductility were significantly smaller. Ductilities at the left abutment were not always larger than those on the right. Abutment ductilities in bridges with and without restrainers were not significantly different. The influence of bearing and substructure stiffness was not as evident as it was for the Kobe earthquake because of the considerably smaller ductilities.

TABLE 6-7 Abutment Displacement Ductilities for Five-Span Bridges

	Including results for Kobe record					*Excluding results for Kobe record*				
abutm.	***w/o restrain.***	***AASHTO***	***Caltrans***	***W/2***	***Proposed method***	***w/o restrain.***	***AASHTO***	***Caltrans***	***W/2***	***Proposed method***
			case:	***C1***				***case:***	***C1***	
A1	12.8	13.2	10.0	12.7	11.5	4.4	3.3	4.1	3.7	3.3
A6	7.6	6.9	6.3	7.6	6.6	2.6	4.1	4.4	2.2	2.4
			case:	***C2***				***case:***	***C2***	
A1	10.2	7.2	6.0	8.3	7.9	3.8	3.1	3.6	3.2	3.0
A6	7.1	6.2	5.3	7.0	7.1	2.6	2.9	4.6	4.8	4.4
			case:	***C3***				***case:***	***C3***	
A1	8.2	6.3	4.6	6.7	6.8	3.8	2.8	3.0	2.7	2.7
A6	7.5	6.8	5.4	7.1	7.3	3.3	3.9	4.0	3.2	3.2
			case:	***C4***				***case:***	***C4***	
A1	13.5	15.9	11.8	16.7	12.2	5.0	4.0	3.6	4.0	3.5
A6	9.7	8.9	7.3	9.4	7.7	4.3	3.5	2.5	4.3	2.7
			case:	***C5***				***case:***	***C5***	
A1	12.3	9.1	6.4	11.0	8.8	3.3	3.3	3.8	3.4	3.1
A6	8.7	6.6	5.4	7.5	7.2	3.9	3.1	3.4	3.4	4.1
			case:	***C6***				***case:***	***C6***	
A1	9.6	7.0	6.1	7.4	7.7	3.1	3.2	3.3	2.9	3.1
A6	8.6	6.1	5.9	6.4	6.7	2.5	2.1	4.7	2.8	4.0
			case:	***C7***				***case:***	***C7***	
A1	11.3	14.3	14.0	15.2	11.8	4.9	3.2	3.3	2.9	3.1
A6	9.3	8.8	7.5	8.3	7.9	6.8	4.9	3.4	7.0	3.6
			case:	***C8***				***case:***	***C8***	
A1	12.7	10.0	8.6	10.3	11.8	2.7	3.4	4.0	3.2	3.0
A6	11.9	6.9	6.4	7.7	7.2	7.2	4.3	5.0	4.6	5.1
			case:	***C9***				***case:***	***C9***	
A1	13.4	8.5	6.7	9.1	11.6	2.1	2.9	4.0	2.6	2.8
A6	10.4	7.3	6.7	6.3	5.6	4.8	3.9	5.0	4.1	3.5

The maximum bearing displacement ductility demands are presented in table 6-8. Again, two maximum values are listed (with and without results obtained from Kobe earthquake). First, a large ductility demand was observed in weak bearings. Those values were due to small yield displacements which were approximately 0.4 mm (due to the large initial elastic bearing stiffness). Due to the larger yield displacements of medium and strong bearings, ductilities were smaller. For example, yield displacement in medium bearings was 1.36 mm and 1.71 mm in the outer and inner spans, respectively. Strong bearing yield displacements were 2.36 mm and 3.0 mm in the outer and inner spans, respectively.

It was also observed that displacement ductility demand in bearings at the left abutment (A1-P), was significantly larger than in bearings at the piers most of the time. Exceptions were bridges with weak bearings, bridges with large and medium substructure stiffness (C1 and C4), and bridge with no restrainers.

TABLE 6-8 Bearing Displacement Ductilities for Five-Span Bridges

	Including results for Kobe record					*Without results for Kobe record*				
bearing	***w/o restrain.***	***AASHTO***	***Caltrans***	***W/2***	***Proposed method***	***w/o restrain.***	***AASHTO***	***Caltrans***	***W/2***	***Proposed method***
			case:	***C1***				***case:***	***C1***	
A1	422	431	344	417	394	256	225	214	287	296
B2	338	178	137	176	196	151	119	102	124	160
B3	214	99	78	127	94	142	70	70	78	87
B4	157	117	76	162	127	128	85	62	94	79
B5	578	171	116	201	135	261	82	79	136	111
			case:	***C2***				***case:***	***C2***	
A1	93	72	63	79	77	47	41	45	42	41
B2	21	21	8	23	31	21	21	8	23	31
B3	20	16	11	24	20	20	16	11	24	20
B4	23	18	14	16	15	19	16	10	12	15
B5	44	26	16	35	33	19	16	14	24	23
			case:	***C3***				***case:***	***C3***	
A1	44	37	30	38	39	26	22	23	22	22
B2	6	8	7	6	8	6	8	7	6	8
B3	9	6	5	9	9	9	4	5	8	7
B4	7	6	4	7	6	4	6	3	4	4
B5	12	19	9	12	13	12	9	9	12	13
			case:	***C4***				***case:***	***C4***	
A1	439	507	393	526	405	361	278	178	281	167
B2	170	98	92	206	117	111	85	83	105	92
B3	118	88	73	106	79	118	81	69	87	79
B4	102	126	105	105	119	102	96	86	104	119
B5	355	152	134	167	169	216	102	134	125	169
			case:	***C5***				***case:***	***C5***	
A1	109	85	66	99	83	43	43	49	44	41
B2	20	17	9	19	20	20	17	8	19	20
B3	21	17	8	20	26	18	17	8	20	26
B4	21	20	16	23	18	21	20	16	23	18
B5	34	20	19	24	21	28	14	13	19	16
			case:	***C6***				***case:***	***C6***	
A1	50	40	36	41	42	23	24	24	23	23
B2	5	6	3	5	5	5	6	3	5	5
B3	7	7	2	9	10	6	7	2	6	6
B4	6	6	5	7	8	6	6	2	7	8
B5	9	9	6	9	12	9	9	6	9	11
			case:	***C7***				***case:***	***C7***	
A1	419	460	454	487	395	419	274	211	180	296
B2	173	91	124	89	201	173	88	96	114	89
B3	183	142	137	165	162	183	124	99	124	140
B4	194	142	118	158	196	194	135	114	147	136
B5	231	187	148	227	239	231	163	134	175	191
			case:	***C8***				***case:***	***C8***	
A1	111	92	82	94	105	38	44	67	42	41
B2	15	9	13	11	17	15	9	9	11	12
B3	12	20	11	29	30	10	20	11	29	30
B4	18	20	27	19	21	18	20	27	19	21
B5	31	29	28	33	37	31	29	28	30	34
			case:	***C9***				***case:***	***C9***	
A1	66	46	38	48	59	19	23	27	21	22
B2	6	4	4	5	6	5	4	4	5	6
B3	7	10	4	8	8	4	10	4	6	6
B4	6	7	6	8	9	5	7	4	8	8
B5	10	11	9	14	13	10	11	7	14	6

Maximum yielding in bearing A1 was mostly governed by the Kobe earthquake. The result from Kobe was considerably larger than the maximum ductility obtained from the other four records. In general, the maximum ductilities were obtained from the Kobe earthquake record.

Generally, different restrainer design procedures led to comparable values of displacement ductility demand in bearings, especially if results obtained for Kobe earthquake were not considered.

6.3 Special Cases

Straight, multi-span, simply supported bridges are common on U.S. highways and were represented in the previous section. However, many bridges exist that do not fall into the category that were studied in the main study. This section presents results of the analysis of special cases which include bridges with narrow seats, bridges subjected to long duration earthquakes, skewed bridges, bridges without abutment restrainers, and bridges equipped with a different minimum number of restrainers. Only a limited number of cases that were believed to be critical were included in the study of special cases.

6.3.1 Bridges with Narrow Seats

Seat width at a bent is largely based on the type of bent cap upon which a girder and bearing are seated. Seat width at an abutment is largely based on the type of abutment. In the main study, bents and abutments were assumed to be constructed of reinforced concrete. Reinforced concrete is the most common material used in abutments and bent caps; however, other types of bent caps exist. This section presents the results of the study of the narrow seat analytical model presented in section 5.5.

Case C7 and C8 in the main study had the largest relative displacements and were closest to unseating; therefore, these cases were studied in the narrow seat study. Each of these cases were subjected to the earthquake that led to a maximum response in most cases in the main study. The two-span bridge was subjected to the San Francisco Airport record, and the five span bridge was subjected to the Kobe record. These records were scaled to 0.7g and 1g.

6.3.1.1 Two-Span Bridges

Figure 6-13 shows the maximum relative displacements of the two-span bridges with narrow seats. Note that none of the bridges violated the available seat width of 127mm with or without restrainers in both the 0.7g and 1g earthquakes. In general, on the pinned sides, displacements increased except for at the abutments with weak bearings. Without restrainers, at roller locations, displacements at the bents were greater than at the abutments. With restrainers, displacements at rollers were greater at the abutments than at bents. In most cases, displacements were not significantly affected by different restrainer design methods. The maximum difference was approximately 40mm.

In comparing the displacements of this model with the models from the main study (figure 6-1), it is noticed that the trends are similar. However, bridges with narrow seats had smaller displacements due to the decreased mass of the bent cap.

In case C7 subjected to a 0.7g earthquake, the Proposed method did not reduce displacements at all at A1-R even though 22 restrainers were required. Caltrans reduced displacements more than the other methods in this case. The same trends were seen for the record with a peak ground acceleration of 1g. Restrained displacements at B2-R were not significantly different than those for the 0.7g case. The Proposed method reduced displacements the most at B2-R, and the AASHTO method the most at A1-R. The Proposed method reduced displacements the least at A1-R, in case C8 that was subjected to a 0.7g record, even though it required as many restrainers as Caltrans (which reduced displacements the most). Displacements were not critical at the pinned side. In case C8 subjected to a peak ground acceleration of 1g, the Proposed and Caltrans methods reduced displacements the most. The Proposed method required slightly more than one-half the number of restrainers that Caltrans required.

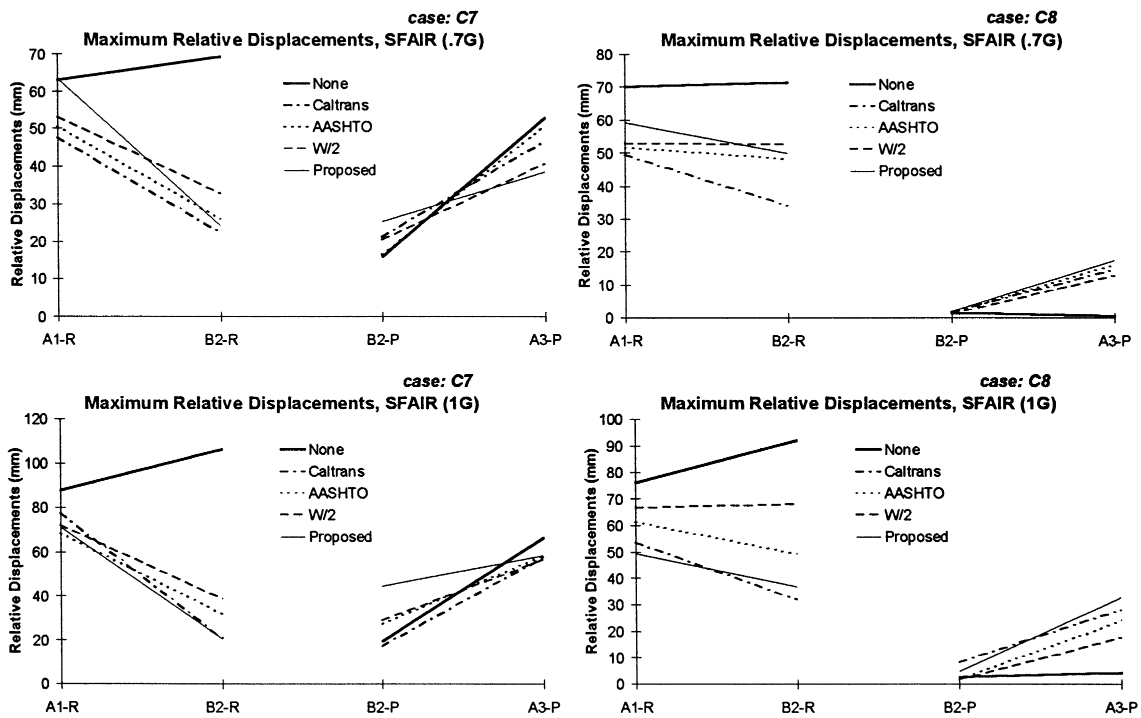


FIGURE 6-13 Relative Displacements for Two-Span Bridges with Narrow Seats

Restrainer displacement ductilities for the cases with narrow seats are shown in figure 6-14. Notice that no significant yielding occurred regardless of the design methods, and that the demand on the restrainers was very low at the pinned locations with stronger bearings (C8). For weaker bearing cases, restrainer demand at the abutments was higher than at the bent. In comparing the restrainer ductilities of this model with the models from the main study (figure 6-2), it is noticed that the demand on the restrainers was lower (0.7g cases) than those in the parametric study except for the Proposed method which experienced an increase in demand. The pinned side in case C8 also experienced an increase in demand.

In case C7 with a 0.7g peak ground acceleration, the only method with yielding restrainers was the Proposed method. The other methods performed similarly. The largest demand was seen at A1-R in case C7 with a 1g peak ground acceleration. All methods yielded at that location with the Caltrans design having the largest ductility of 1.45. Not much difference existed among the restrainer ductilities from different methods in this case. All methods experienced higher demand at roller locations. In case C8 subjected to a 1g peak ground acceleration, the Proposed method was the only method that did not show restrainer yielding. The demand for the Proposed method restrainers was much smaller than that for the others. Restrainers from the W/2 method experienced the highest ductility, 1.24 at the roller sides.

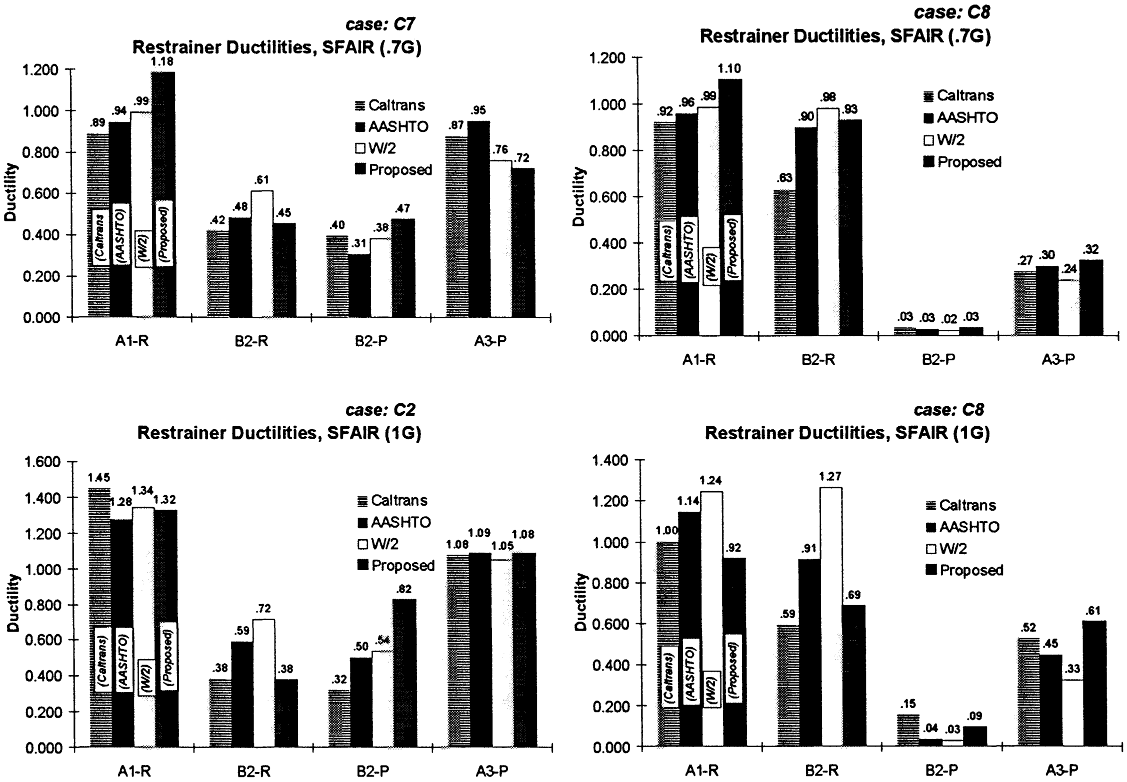


FIGURE 6-14 Restrainer Ductilities for Two-Span Bridges with Narrow Seats

The envelopes of total relative displacements for two-span bridges with narrow seats are shown in figure 6-15. As in previous cases, all of the design methods led to a similar reduction in relative displacements. Caltrans reduced displacements the most but required many more restrainers than the other methods.

Figure 6-16 shows the effectiveness factors for the two-span short seat cases. The effectiveness factors were higher in case C7 than in case C8. This is because the bearings reduced the effectiveness of the restrainers by absorbing higher forces in C8. The restrainers are not utilized as much on the pin sides in C8 as in C7. In general, the W/2 method was the most effective procedure. The W/2 method was the most effective, and Caltrans was the least effective method three out of the four times. The Proposed method was the most effective once and the least effective once. Upon comparison of the effectiveness factors in this section with those found in the main study, it was noticed that in both case C7 and C8, the W/2 method was the most effective procedure, and that the AASHTO method was the second most effective procedure in both studies.

In case C7 with a 0.7g peak ground acceleration, the Caltrans and Proposed procedures had the lowest effectiveness factors which were less than one-half of the factor corresponding to W/2. The factor corresponding to the AASHTO method fell approximately half way in between the high and low factors. The Caltrans and Proposed methods had the lowest factors when the peak ground acceleration was 1g for case C7. The factor corresponding to the W/2 method was twice that of Caltrans and Proposed methods. Again, the AASHTO method fell about half way between the high and low factors. In case C8 subjected to a 0.7g peak ground acceleration, the Proposed method had the smallest factor. The other three methods showed an interesting trend. The procedure requiring the least number of restrainers was the most effective (W/2) and the one with the most number was the least effective (Caltrans). In case C8 subjected to a 1g peak ground acceleration, the Proposed, W/2, and AASHTO methods had nearly the same factors and the Proposed method was the most effective. The Caltrans method was the least effective in this case.

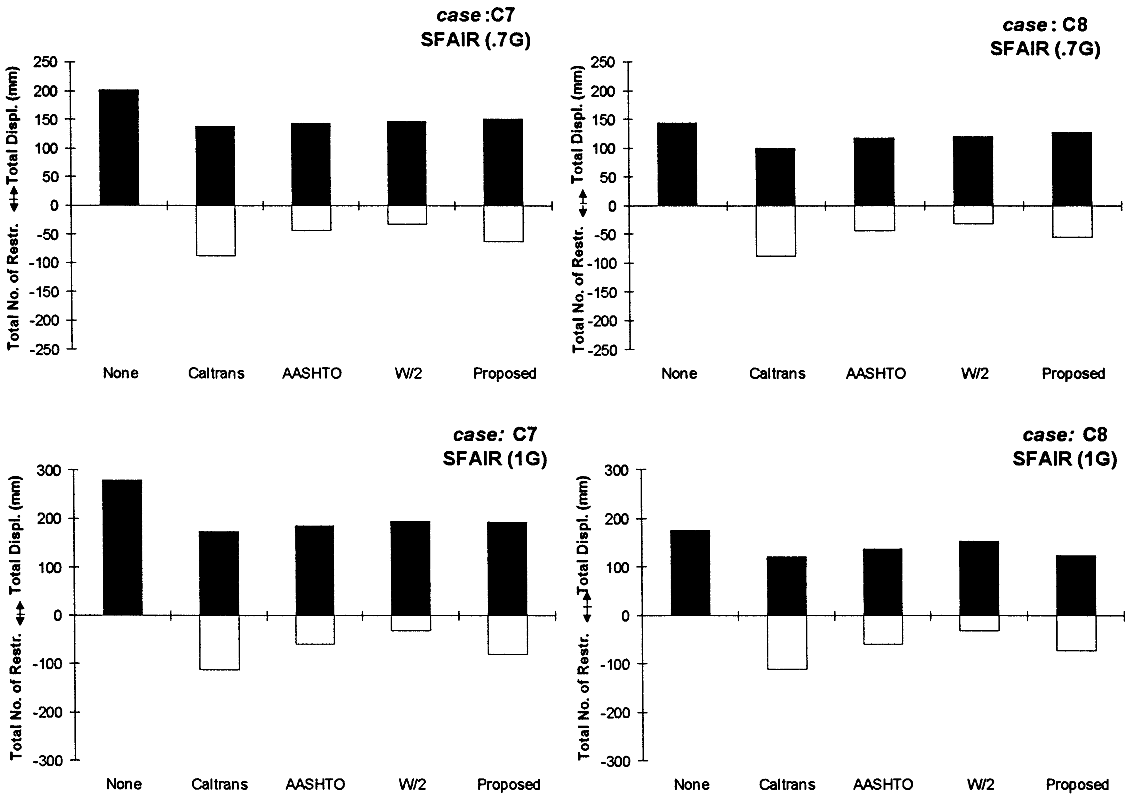


FIGURE 6-15 Envelopes of Total Relative Displacements for Two-Span Bridges with Narrow Seats

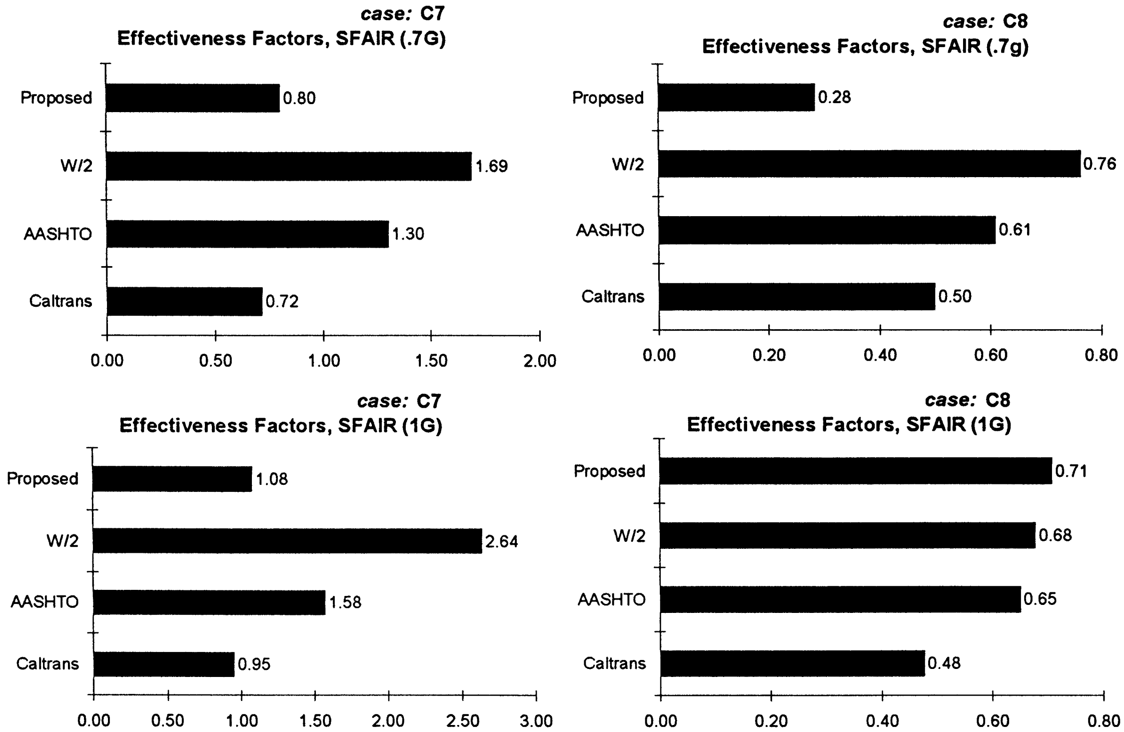


FIGURE 6-16 Effectiveness Factors for Two-Span Bridges with Narrow Seats

The abutment displacement ductility demands for the two-span bridges with narrow seats are shown in figure 6-17. In general, scaling the earthquake from 0.7g to 1g increased the demand on the abutments except for case C8 with the Proposed method restrainers. In that case the demand decreased. A pattern that described how abutments reacted to different restrainer designs could not be established. Restrainers increased abutment ductility in some cases and reduced it in others. Upon comparison of the abutment ductilities for narrow seat bridges with the main study, it was observed that the ductilities were similar but the extent of yielding was more limited in bridges with narrow seats.

In case C7 subjected to a 0.7g peak ground acceleration, generally, the abutment ductilities increased when restrainers were added except for the Proposed method where neither of the abutments yielded. For most restrainer designs, the pinned abutment experienced a higher demand than the abutment with a roller bearing. Generally, in case C7 subjected to a 1g peak ground acceleration, abutment ductilities decreased when restrainers were added. The pinned abutments experienced a higher demand than the roller abutments in most cases. Adding restrainers generally reduced the demand on abutments in case C8 subjected to a peak ground acceleration of 0.7g. Without restrainers, the demand was higher on the pinned sides, and with restrainers the demand was higher at roller locations. In case C8 for a peak ground acceleration of 1g, restrainers reduced the demand on the abutments for all of the restrainer designs.

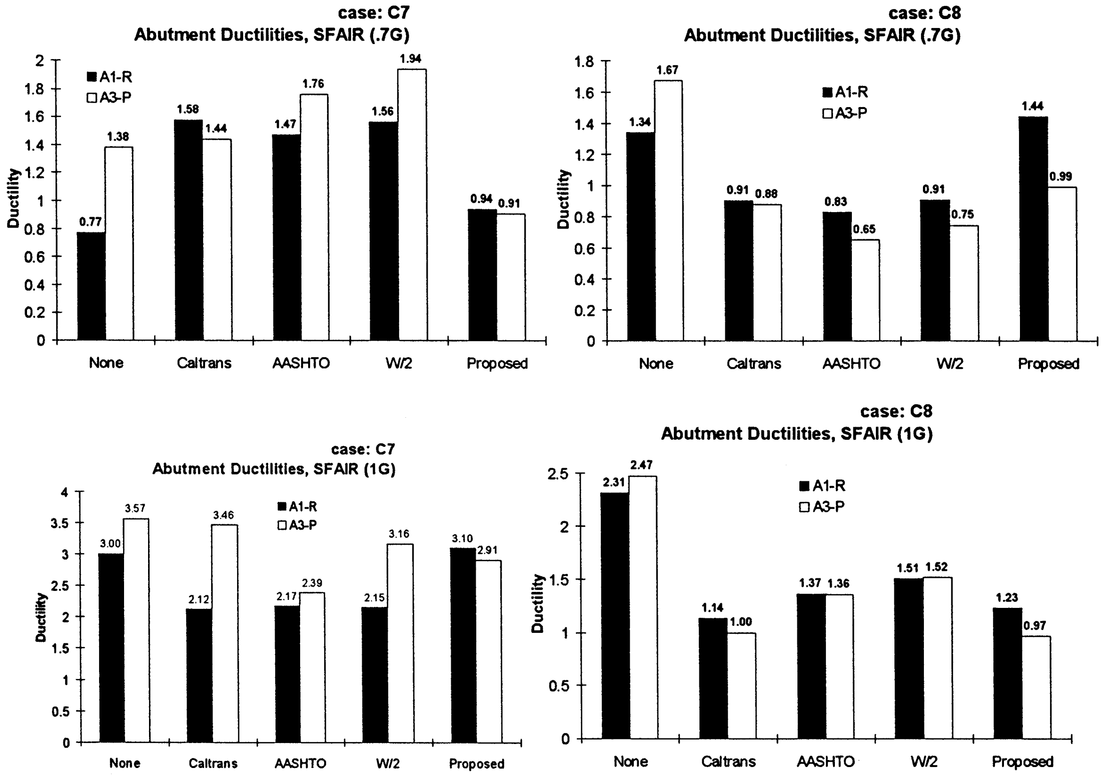


FIGURE 6-17 Abutment Ductilities for Two-Span Bridges with Narrow Seats

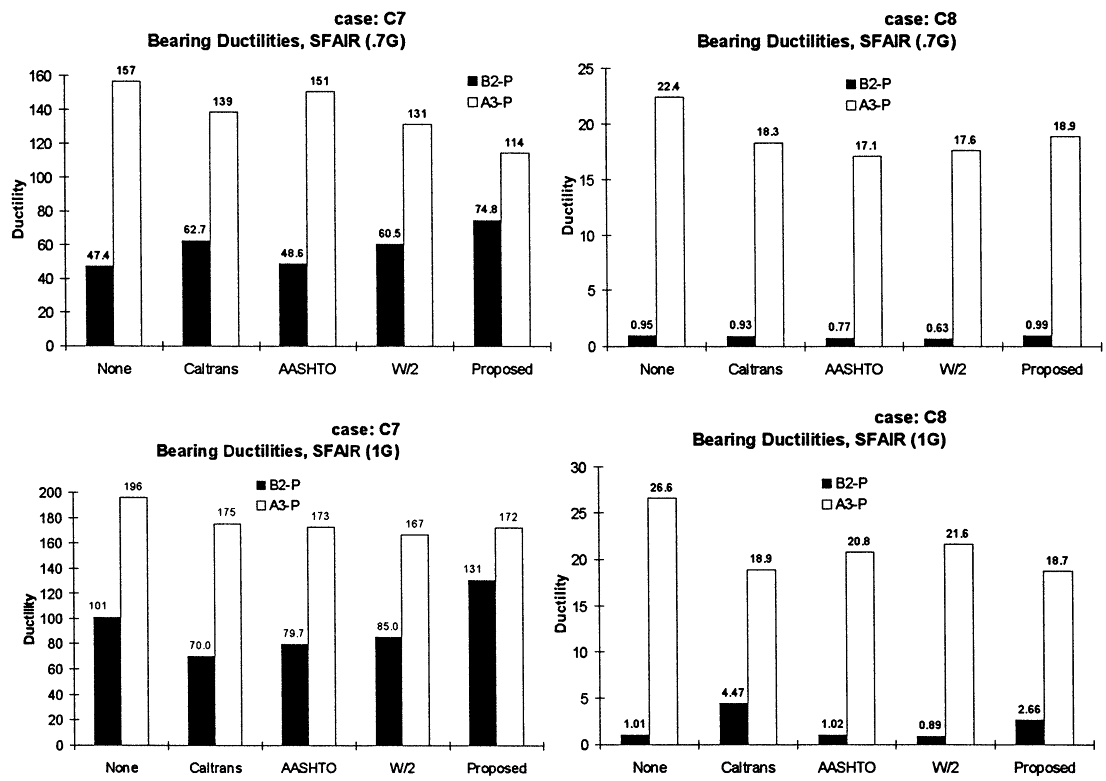


FIGURE 6-18 Bearing Ductilities for Two-Span Bridges with Narrow Seats

Bearing displacement ductilities are presented in figure 6-18. In all cases, bearings at the abutments had considerably higher ductilities than the bearings at the bents. Generally, increasing the earthquake peak ground acceleration increased the demand on the bearings as expected. Ductilities did not change substantially with or without restrainers or among the different methods in most instances. Compared to the main study, in which adding restrainers to the bridge reduced bearing ductilities, adding restrainers to the bridges with narrow seats did not reduce ductilities.

6.3.1.2 Five-Span Bridges

Figure 6-19 shows the critical relative displacements of the five-span bridges with narrow seats. Note that the maximum displacements occurred at the abutments and end-span bents (A1,A6, B2-R). The bridge became unseated without restrainers in all cases, and with restrainers in C8. Displacements at the pinned locations were relatively small. The only critical pinned location was A1-P. The cases with W/2, AASHTO and Proposed restrainers became unseated in case C8 at B2-R in both earthquakes (.7g and 1g). The bridge with the Proposed restrainers also became unseated at B4-R in case C8 with a peak ground acceleration of 1g. Bridges with Caltrans restrainers did not become unseated. All of the restrainer designs performed similarly in case C7.

When comparing the displacements of this model with the models from the main study, it was noticed that responses were similar in case C7. However, the response in the bridge with the concrete cap (main study) was slightly larger than the response of the bridge with the steel cap due to the larger bent cap mass. In case C8, the responses were similar also. The larger bent cap caused a larger response in the main study model except for the Proposed method which yielded a larger response at B2-R. The bridge did not unseat in the main study. However, unseating occurred in several cases with narrow seats.

In case C7 with a peak ground acceleration of 0.7g, the bridge violated the ASW and became unseated at B2-R and B5-R when restrainers were not installed. Restrainers from all methods reduced displacements below the ASW. The difference among all the methods was small. The AASHTO and W/2 restrainers did not reduce displacements significantly at A6-R. Displacements at pins were small, and restrainers had little or no effect on displacements at pinned locations. In case C7 subjected to a 1g peak ground acceleration and without restrainers, displacements violated the ASW at all roller locations except A6-R. With restrainers in C7, displacements did not violate the ASW; thus, all methods performed well. All the methods performed similarly except at the abutments. At abutments, the Proposed method reduced displacements the most.

Without restrainers, in case C8 subjected to a peak ground acceleration of 0.7g, the bridge violated the ASW at B2-R,B3-R, and B5-R. Displacements at pinned locations were considerably smaller than the ASW. With restrainers, the only method that did not violate the ASW was Caltrans. The AASHTO, W/2, and Proposed methods all violate the ASW at B2-R. Caltrans was the only method that performed satisfactorily in this case. In case C8 subjected to a 1g peak ground acceleration, displacements were not critical at pinned locations. The AASHTO, W/2, and Proposed methods all violated the ASW at B2-R, and the Proposed method violated the ASW at B4-R also. Caltrans was the only method that did not violate the ASW.

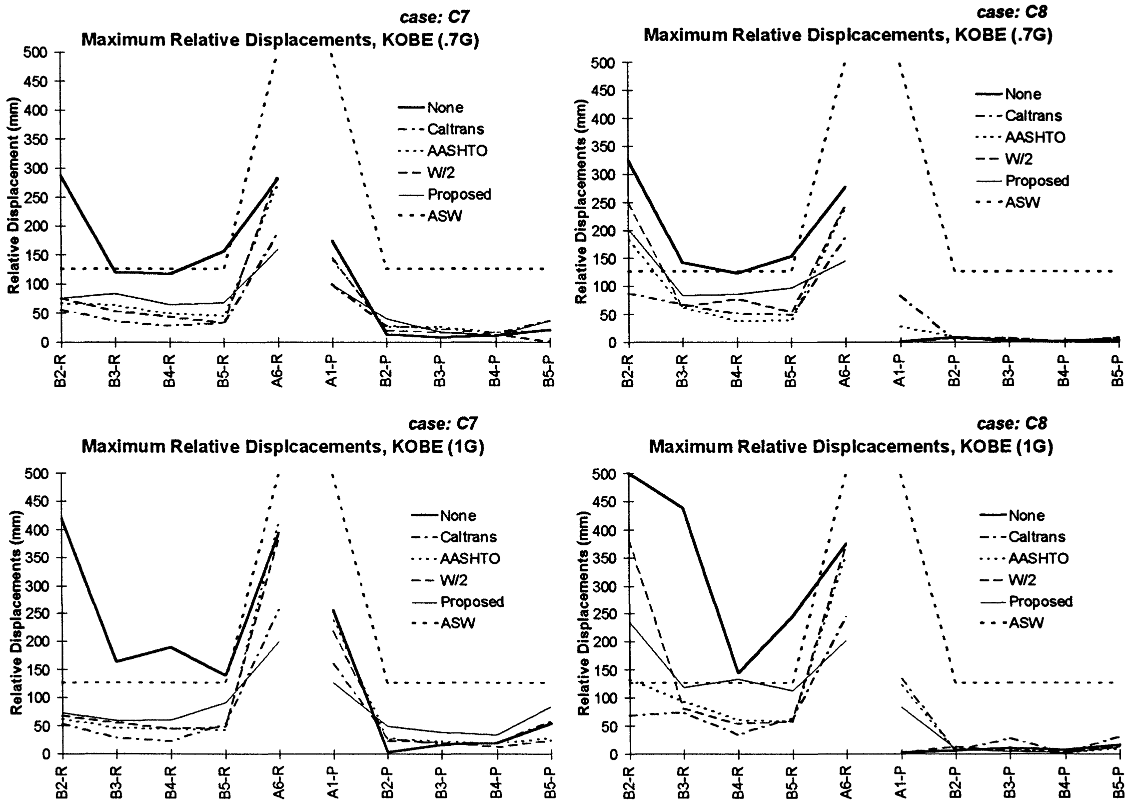


FIGURE 6-19 Relative Displacements for Five-Span Bridges with Narrow Seats

Restrainer displacement ductilities for the five-span model with narrow seats are presented in table 6-9. No yielding occurred at pinned locations at bents. In the cases with a 0.7g peak ground acceleration, restrainers experienced limited yielding but lower than a ductility of 2.65. All methods yield at least once. With the peak ground acceleration raised to 1g the ductility demand increased to four at abutments and at B2-R.

The restrainer ductilities for the bridges with narrow seats were similar to those of wide seats. The values at most locations were similar in the case with a 0.7g peak ground acceleration. The result for case C8 was similar in both studies with the exception of the Proposed method. The Proposed method experienced a higher demand when the seat was narrow. Generally, ductilities were smaller with narrow seats due to the smaller bent cap mass.

In case C7 with a 0.7g and 1g peak ground accelerations, restrainer yielding occurred only at abutments. The Proposed method had the lowest demand at the abutments. Generally the Proposed method experienced ductilities closer to one than other methods indicating a more effective design.

Most of the restrainers yielded at A6-R and B2-R in case C8 with a peak ground acceleration of 0.7g. On the roller sides, all restrainers yielded at A6-R. At B2-R, W/2 restrainers experienced the highest ductility (2.31), but Caltrans restrainers did not yield. On the pinned sides, no method experienced significant demand except for Caltrans at A1-P (.78). Raising the peak ground acceleration from 0.7g to 1g in case C8 did not change the trend. Restrainers at roller abutments yielded significantly (as high as 3.43). The Proposed restrainers yielded in all roller locations.

TABLE 6-9 Restrainer Displacement Ductilities for Five-Span Bridges with Narrow Seats

Case C7 - .7g*KOBE

	Caltrans	AASHTO	W/2	Proposed
B2-R	0.53	0.62	0.71	0.71
B3-R	0.34	0.59	0.50	0.78
B4-R	0.27	0.45	0.41	0.61
B5-R	0.31	0.42	0.32	0.64
A6-R	**1.73**	**2.52**	**2.65**	**1.50**
A1-P	0.95	**1.29**	**1.33**	**0.92**
B2-P	0.27	0.25	0.20	0.38
B3-P	0.22	0.25	0.15	0.16
B4-P	0.15	0.16	0.12	0.11
B5-P	0.34	0.19	0.01	0.35

Case C8 - .7g*KOBE

	Caltrans	AASHTO	W/2	Proposed
B2-R	0.81	**1.68**	**2.31**	**1.88**
B3-R	0.63	0.59	0.59	0.77
B4-R	0.47	0.35	0.72	0.79
B5-R	0.45	0.36	0.50	0.90
A6-R	**1.72**	**2.23**	**2.25**	**1.35**
A1-P	0.78	0.27	0.01	0.02
B2-P	0.04	0.08	0.06	0.09
B3-P	0.06	0.06	0.07	0.04
B4-P	0.01	0.01	0.01	0.01
B5-P	0.07	0.09	0.08	0.05

Case C7 - 1g*KOBE

	Caltrans	AASHTO	W/2	Proposed
B2-R	0.49	0.58	0.64	0.68
B3-R	0.27	0.42	0.51	0.56
B4-R	0.22	0.41	0.41	0.56
B5-R	0.49	0.39	0.44	0.86
A6-R	**2.38**	**3.81**	**3.54**	**1.87**
A1-P	**1.46**	**2.22**	**2.02**	**1.18**
B2-P	0.27	0.25	0.22	0.45
B3-P	0.17	0.21	0.19	0.36
B4-P	0.19	0.18	0.12	0.32
B5-P	0.54	0.27	0.23	0.78

Case C8 - 1g*KOBE

	Caltrans	AASHTO	W/2	Proposed
B2-R	0.64	**1.25**	**3.55**	**2.19**
B3-R	0.68	0.87	0.75	**1.10**
B4-R	0.32	0.57	0.50	**1.24**
B5-R	0.60	0.54	0.54	**1.05**
A6-R	**2.26**	**3.28**	**3.43**	**1.89**
A1-P	**1.23**	**1.12**	0.02	0.77
B2-P	0.05	0.05	0.12	0.09
B3-P	0.27	0.09	0.05	0.05
B4-P	0.01	0.02	0.03	0.02
B5-P	0.30	0.09	0.13	0.11

Envelopes of total relative displacements and effectiveness factors for the five-span models with narrow seats are shown in figures 6-20 and 6-21, respectively. From figure 6-20 one may observe that all of the methods reduced displacements substantially compared to the unrestrained bridges. The Caltrans method always required substantially more restrainers than the other methods but did not reduce displacements proportionally.

The W/2 method had the highest effectiveness factors in three out of four cases. However, the W/2 method was not the most effective procedure because W/2 restrainers experienced significant yielding in several cases and because they led to unseating in case C8. The Caltrans method had the lowest effectiveness factors in all cases, but it always prevented the bridge from becoming unseated. When comparing the effectiveness factors in figure 6-21 with the effectiveness factors found in the main study, it was noticed that both case C7 and C8 follow the same trend in both studies.

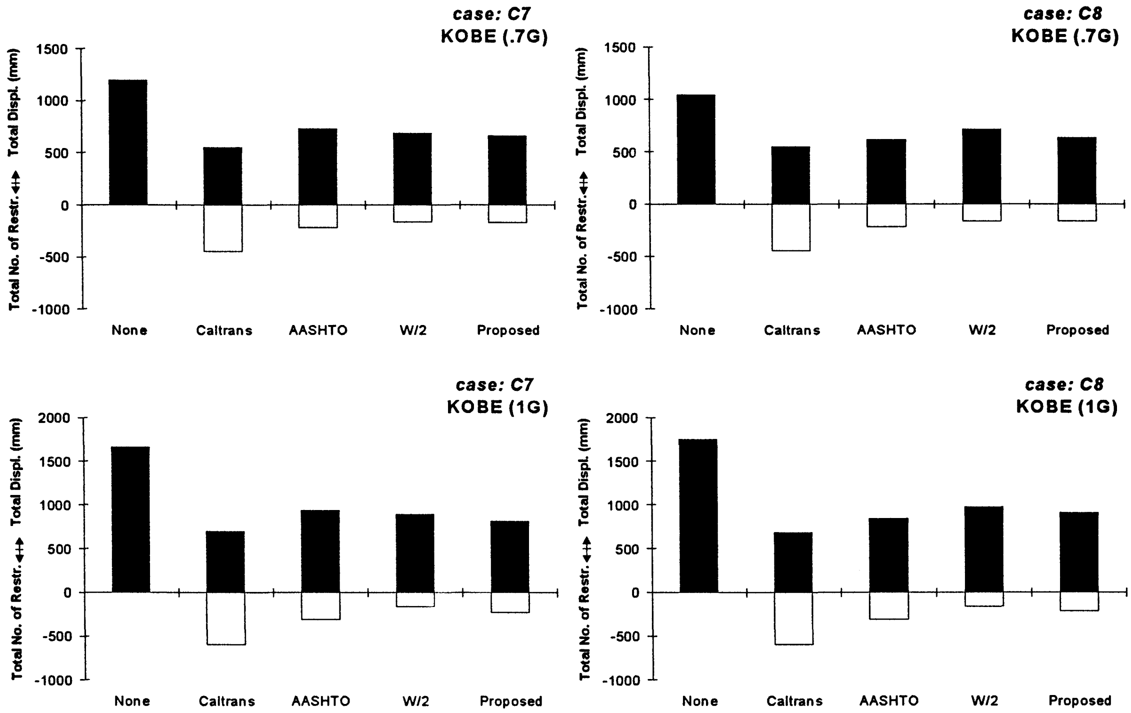


FIGURE 6-20 Envelopes of Total Relative Displacements for Five-Span Bridges with Narrow Seats

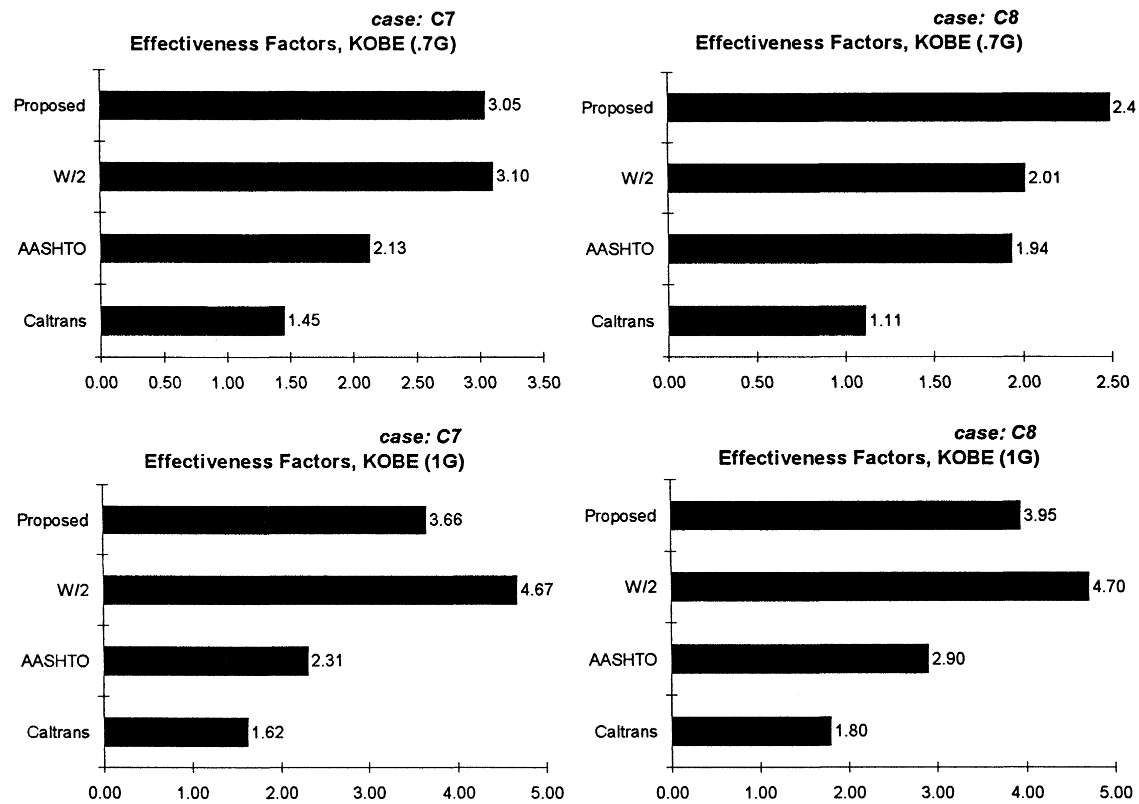


FIGURE 6-21 Effectiveness Factors for Five-Span Bridges with Narrow Seats

In case C7 with a peak ground acceleration of 0.7g, the Proposed and W/2 method effectiveness factors were nearly the same. The AASHTO method effectiveness factor fell between the high and low, and the Caltrans method effectiveness factor was the lowest. The same trends applied to cases C7 and C8 with a peak ground acceleration of 1g, but the difference between the W/2 and Proposed method effectiveness factors was greater. The AASHTO and W/2 method effectiveness factors were similar in case C8 with a peak ground acceleration of 0.7g, but the Proposed method had the highest factor.

Figure 6-22 shows the maximum abutment ductilities for the five-span bridges with narrow seats. Note that abutments yielded in all cases with ductilities well in excess of one. The demand on roller abutments was always higher than the demand on pinned abutments. Adding restrainers to the bridge seemed to increase the difference in ductilities between A1-P and A6-R in many cases. The ductility demand increased with the earthquake peak ground acceleration as expected.

Table 6-10 shows the bearing displacement ductility demands. Note that the bearings at abutments always had considerably higher ductilities than the bearings at bents. Generally, increasing the earthquake from 0.7g to 1g did not always increase bearing ductilities except at the abutments. At the bents, the addition of restrainers did not have a uniform effect on ductilities. When comparing the bearing ductilities from narrow seats with those from the main study, note that in both studies ductilities at A1 were larger than the bents. In general, the values for narrow seats were smaller due to the lower bent cap mass.

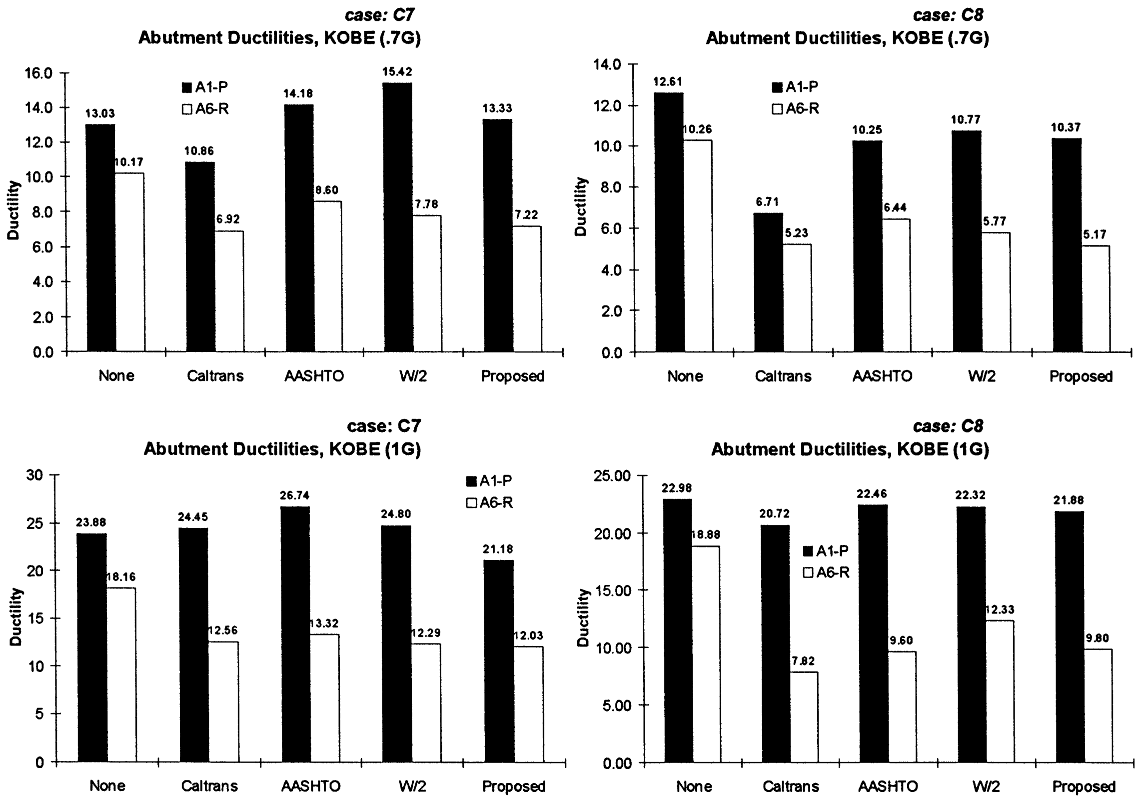


FIGURE 6-22 Abutment Ductilities for Five-Span Bridges with Narrow Seats

TABLE 6-10 Bearing Displacement Ductilities for Five-Span Bridges with Narrow Seats

Case C7 - .7g*KOBE

	None	Caltrans	AASHTO	W/2	Proposed
B2-P	63.2	68.8	63.9	50.4	95.5
B3-P	57.5	54.7	63.5	48.1	41.0
B4-P	60.7	77.1	40.6	85.8	81.7
B5-P	64.3	102.9	58.4	88.9	105.7
A1-P	489.9	366.5	457.1	490.9	433.9

Case C8 - .7g*KOBE

	None	Caltrans	AASHTO	W/2	Proposed
B2-P	4.3	2.7	5.3	3.7	5.8
B3-P	7.1	3.9	3.8	4.3	3.1
B4-P	7.6	10.1	8.4	11.1	13.9
B5-P	1.9	5.9	6.9	6.7	4.0
A1-P	109.0	66.7	92.1	95.8	92.9

Case C7 - 1g*KOBE

	None	Caltrans	AASHTO	W/2	Proposed
B2-P	104.8	67.0	61.9	55.8	114.5
B3-P	74.9	74.4	53.5	73.1	89.8
B4-P	118.8	56.5	72.1	79.6	82.8
B5-P	148.8	163.4	89.7	93.3	235.2
A1-P	721.2	737.0	799.3	746.4	647.7

Case C8 - 1g*KOBE

	None	Caltrans	AASHTO	W/2	Proposed
B2-P	5.9	3.9	3.1	7.7	5.4
B3-P	5.2	16.7	5.4	6.1	4.9
B4-P	8.6	6.1	15.3	6.6	15.7
B5-P	11.3	23.8	6.8	10.3	8.6
A1-P	183.4	167.2	179.6	178.6	175.5

The Modified Caltrans method (Section 3.6) was used for case C8 with narrow seats. Figure 6-23 shows the relative displacements for different methods including the Modified Caltrans restrainer design. Note that the Modified Caltrans restrainers kept the bridge seated in both cases as well as the Caltrans restrainers with far fewer restrainers (309 versus 446). Figure 6-24 shows effectiveness factors. Note that the effectiveness factors for the Modified Caltrans method were at least 15 percent higher than Caltrans indicating a more efficient design.

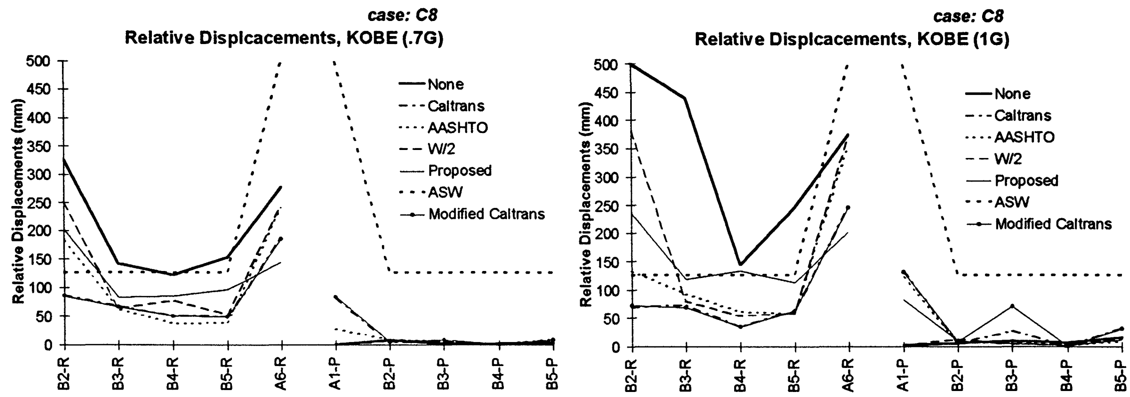


FIGURE 6-23 Relative Displacements for Five-Span Bridges with Narrow Seats Including Modified Caltrans Design

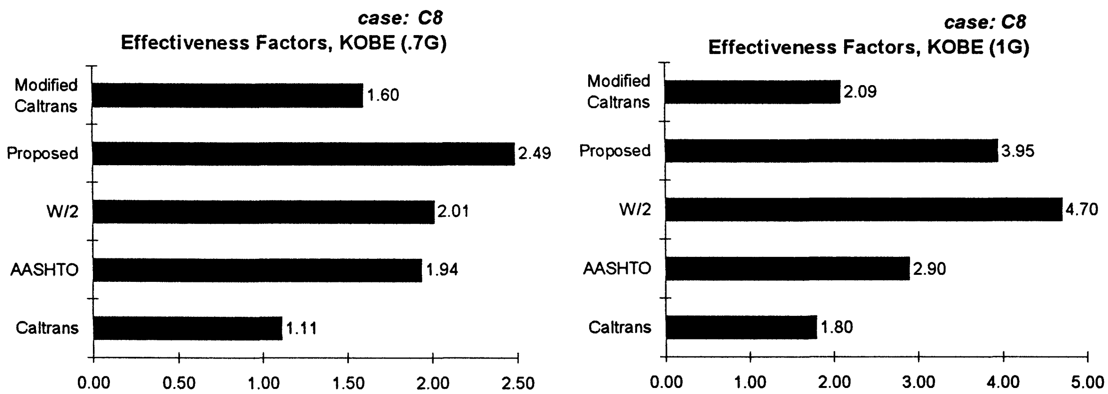


FIGURE 6-24 Effectiveness Factors for Five-Span Bridges with Narrow Seats Including Modified Caltrans Design

6.3.2 Effect of Earthquake Duration

Frequency content, amplitude, and duration are the three main parameters that characterize strong ground motion (Jennings, 1983). Different earthquake frequency contents were represented by using five different earthquakes and by varying structural vibration frequency properties such as substructure stiffness. Peak ground accelerations of 0.7g and 1g were used to represent earthquake amplitude. Earthquake duration can be important in non-linear structural analysis because longer duration of strong shaking can lead to the accumulation of plastic deformations. In this study, case C7 of both the two-span and five-span bridges with narrow seats were subjected to a long duration earthquake. Case C7 was chosen because it had the weakest bearings. Weaker bearings would yield sooner than stronger bearings; thus, weaker bearings are subject to large plastic deformations which would be more critical in a long duration earthquake. The results of these analyses are presented in this section.

The La Villita North-South record was chosen as the long duration earthquake that would be used in the analysis. The accelerogram for LaVillita North-South is shown in figure 6-25. The LaVillita station is located in Mexico and was part of the Guerrero Network established in the 1980s (Anderson et al., 1987). The original record had a peak ground acceleration of approximately 125 cm/s^2. The duration used in the analysis was 60 seconds. The acceleration record was scaled to 0.7g so that the analytical results could be compared with those obtained in the main and short seat studies. The response spectra for the both north-south and east-west components of the LaVillita record are shown in figure 6-26. The north-south record was chosen for analysis because it had a wider frequency band.

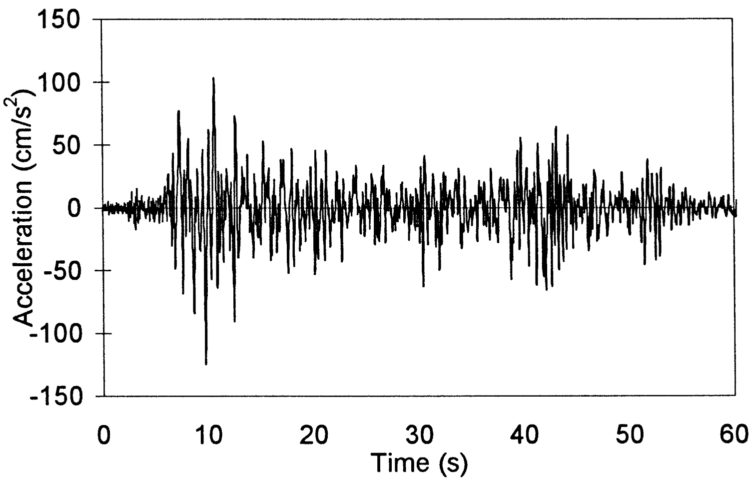


FIGURE 6-25 La Villita North-South Earthquake Accelerogram

A long duration earthquake is characterized by relatively large amplitudes that last over a long period of time (Jennings, 1983). Note in figure 4-13 that the duration of strong ground motion in the earthquakes shown lasts for less than 15 seconds in most cases. The El Centro record shows strong ground motion for approximately 25 seconds. When scaled, the La Villita record shows nearly 50 seconds of strong ground motion (figure 6-25). This can be characterized by five peak regions in the acclerogram. The peak ground acceleration occurs at approximately 10 seconds and was scaled to 0.7g. Scaling the peak leads to other acceleration spikes of 0.4g at 20, 30, and 40 seconds.

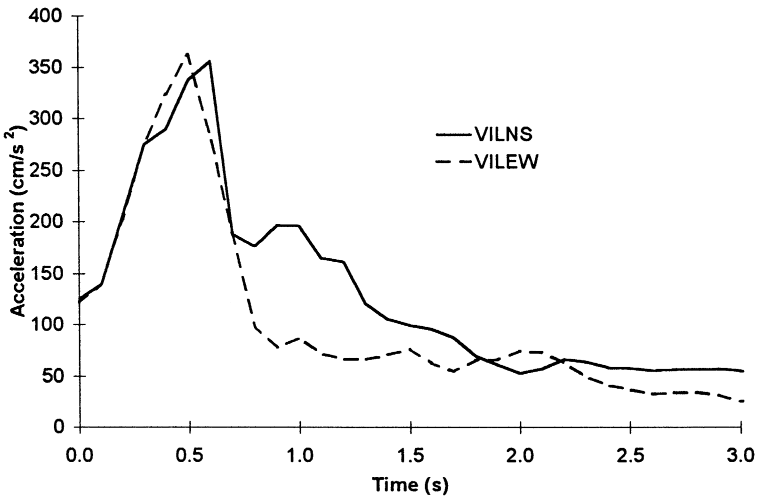


FIGURE 6-26 La Villita Earthquake Acceleration Spectra

6.3.2.1 Two-Span Bridge

The maximum relative displacements for the two-span bridge subjected to the LaVillita earthquake record are shown in figure 6-27. Generally, relative displacements for the case without restrainers are higher for LaVillita than San Francisco Airport at roller locations. For cases with restrainers, displacements are not affected significantly. Caltrans restrainers reduced displacements the most. The Proposed restrainers performed better in the LaVillita earthquake than in the San Francisco Airport earthquake.

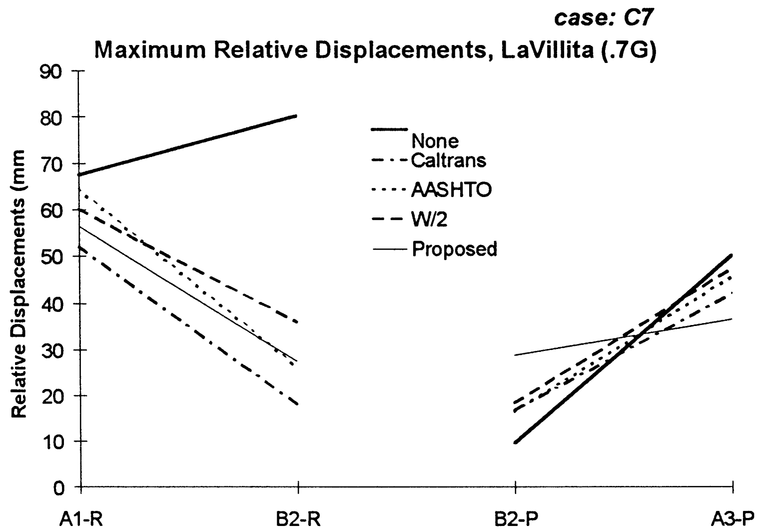


FIGURE 6-27 Relative Displacements for Two-Span Bridge Subjected to a Long Duration Earthquake

Restrainer ductilities are presented in figure 6-28. Ductility values were not significantly different than those obtained with other earthquakes.

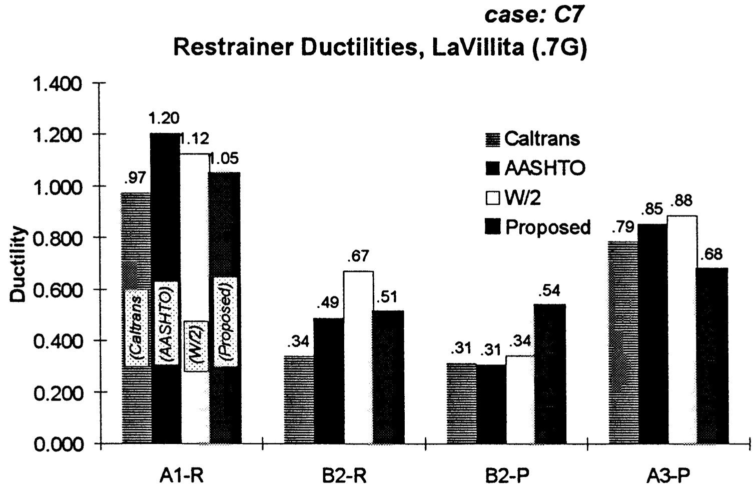
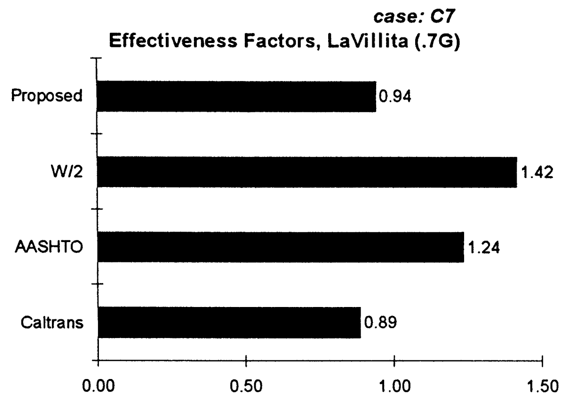


FIGURE 6-28 Restrainer Ductilities for Two-Span Bridge Subjected to a Long Duration Earthquake



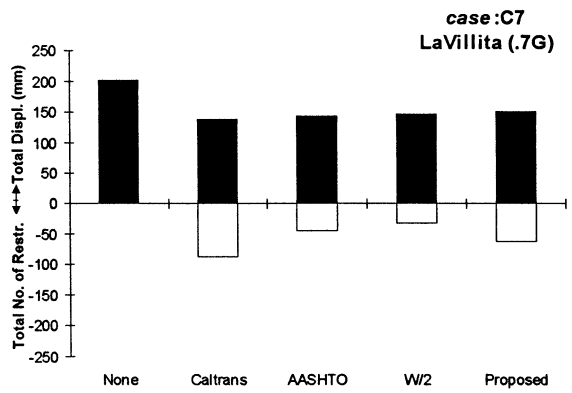


FIGURE 6-29 Effectiveness Factors and Envelopes of Total Relative Displacements for Two-Span Bridge Subjected to a Long Duration Earthquake

Effectiveness factors and envelopes of total relative displacements are shown in figure 6-29. The effectiveness factors in both studies followed the same pattern. The W/2 method had the highest effectiveness factor and Caltrans the lowest. Generally, values of effectiveness were similar to those corresponding to bridges with narrow seats.

Abutment ductilities are shown in figure 6-30. Abutment ductility values were not significantly different from the short seat study although the patterns of high and low values were different than in the short seat study.

Bearing ductilities are also shown in figure 6-30. Bearing ductilities were not significantly different from those that occurred in the narrow seat bridges. Ductilities of bearings at abutments were consistently higher than those for the bearings at the bents. This trend was the same as the trend found in narrow seat bridges.

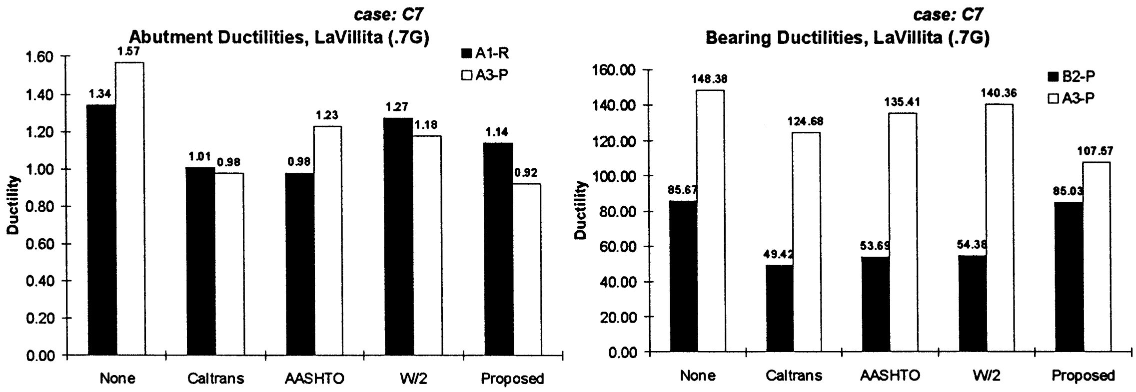


FIGURE 6-30 Abutment and Bearing Ductilities for Two-Span Bridge Subjected to a Long Duration Earthquake

6.3.2.2 Five-Span Bridge

The maximum relative displacements for the five-span bridge subjected to the LaVillita earthquake record are shown in figure 6-31. Critical relative displacements were smaller without restrainers in LaVillita than in Kobe. This was especially true at critical locations such as B2-R, A6-R, A1-P; however, without restrainers the bridge violated the ASW criterion. With restrainers, displacements were not significantly different from the bridges with narrow seats with the exception of the abutments. All design methods reduced displacements.

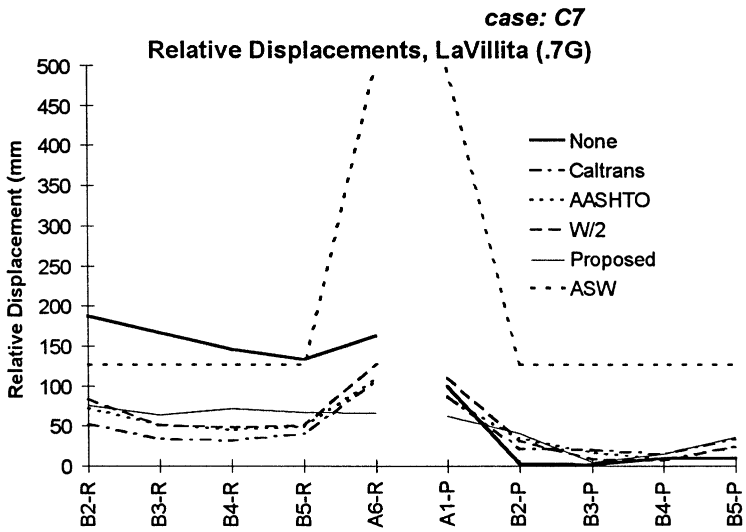


FIGURE 6-31 Relative Displacements for Five-Span Bridge Subjected to a Long Duration Earthquake

Restrainer ductilities for the five-span bridge subjected to the longer duration earthquake are shown in table 6-11. At critical locations, restrainer ductilities were smaller in La Villita than Kobe, but in most other locations, ductilities were close.

The effectiveness factors and envelopes of total relative displacements are shown in figure 6-32. The trends were similar for both Kobe and LaVillita. Both the Proposed and W/2 methods performed the best in both cases. The effectiveness factors were smaller for LaVillita than for San Francisco Airport.

TABLE 6-11 Restrainer Ductilities for Five-Span Bridge Subjected to a Long Duration Earthquake

Case C7 - .7g*LaVillita

	Caltrans	AASHTO	W/2	Proposed
B2-R	0.48	0.67	0.78	0.71
B3-R	0.31	0.48	0.47	0.60
B4-R	0.29	0.42	0.45	0.67
B5-R	0.37	0.46	0.47	0.62
A6-R	**1.00**	**1.01**	**1.19**	0.61
A1-P	0.82	0.80	**1.04**	0.58
B2-P	0.20	0.32	0.30	0.38
B3-P	0.19	0.16	0.08	0.04
B4-P	0.14	0.08	0.05	0.13
B5-P	0.31	0.22	0.23	0.33

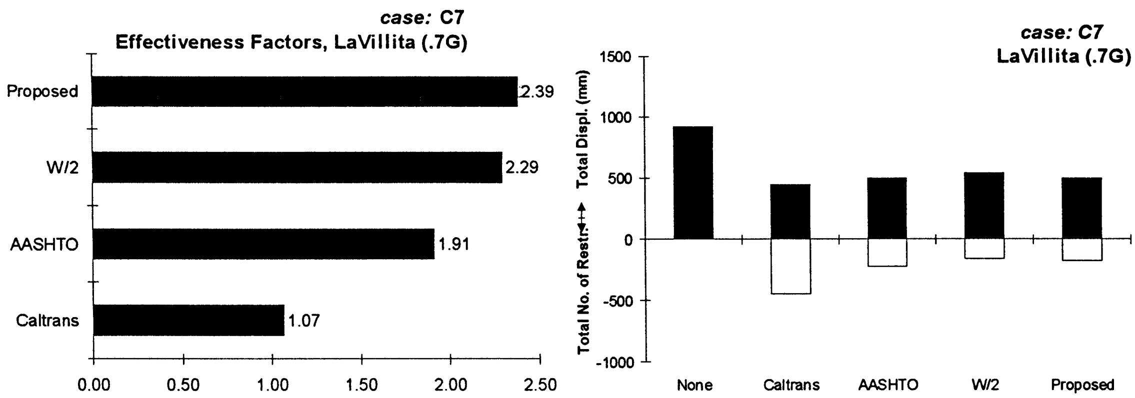


FIGURE 6-32 Effectiveness Factors and Envelopes of Total Relative Displacements for Five-Span Bridge Subjected to a Long Duration Earthquake

Abutment ductilities are shown in figure 6-33. Abutment ductilities were smaller than bridges with narrow seats, and the demand at A6-R was higher than at A1-P in all cases which was the opposite of the trend found in the other bridges with narrow seats.

Bearing ductilities are shown in table 6-12. Without restrainers, bearing ductilities were higher at bents and smaller at abutments for LaVillita than for Kobe. With restrainers, bearing ductilities were similar for both cases.

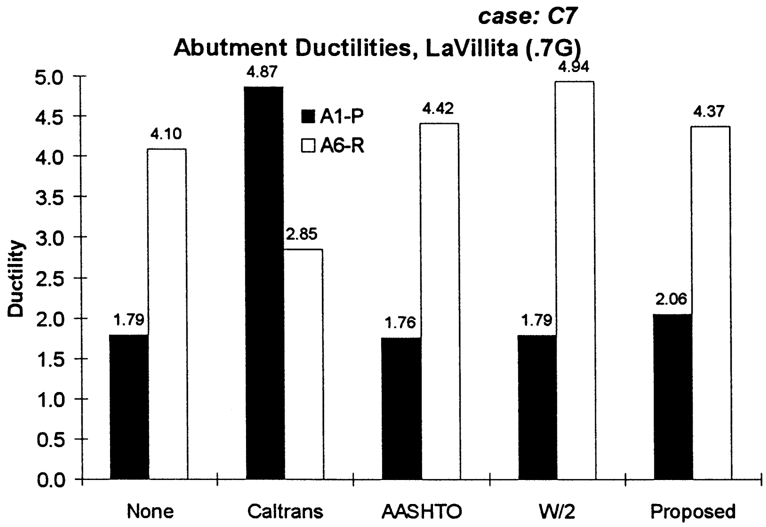


FIGURE 6-33 Abutment Ductilities for Five-Span Bridge Subjected to a Long Duration Earthquake

TABLE 6-12 Bearing Ductilities for Five-Span Bridge Subjected to a Long Duration Earthquake

Case C7 - .7g*LaVillita

	Bearing Ductilities				
	None	Caltrans	AASHTO	W/2	Proposed
B2-P	137.7	49.7	81.9	74.5	96.9
B3-P	152.3	47.4	87.4	95.7	72.9
B4-P	174.9	80.9	81.6	92.2	84.1
B5-P	90.3	94.4	67.0	68.0	100.2
A1-P	278.6	246.8	241.1	310.6	173.5

6.3.4 Skewed Bridges

In the past, skewed bridges have proven to be particularly vulnerable to seismic loads due to their tendency to rotate in the plane of the bridge deck. Skewed analytical bridge models were constructed and analyzed as described in section 5.6. Analysis results from the skewed bridge models are presented in this section. Bridges were subjected to ground motion in both the longitudinal and transverse directions. The Kobe east-west record scaled to 0.7g was used in the longitudinal direction and the Kobe north-south record with the same scale factor resulting in a peak ground acceleration of 0.9g was used in the transverse direction. The first part of this section describes the performance of restrainer designs that do not account for skew. The second part describes a method that modifies a number of restrainers to account for skew and presents the results.

6.3.4.1 Unmodified Restrainer Designs

The critical relative displacements for the skewed models are shown in figure 6-34. In these plots "B2-RA" is the relative displacement at bent two on the roller side at the acute corner. On the other hand, "B2-RO" stands for bent two roller side at the obtuse corner. Results for all skew angles are shown in the figure. Note that all bridges without restrainers unseated at roller locations and remained supported at pin locations. All cases equipped with restrainers violated the available seat width at least once. Compared to the response of the unskewed bridge, the skewed bridge corner displacements were as much as four times greater.

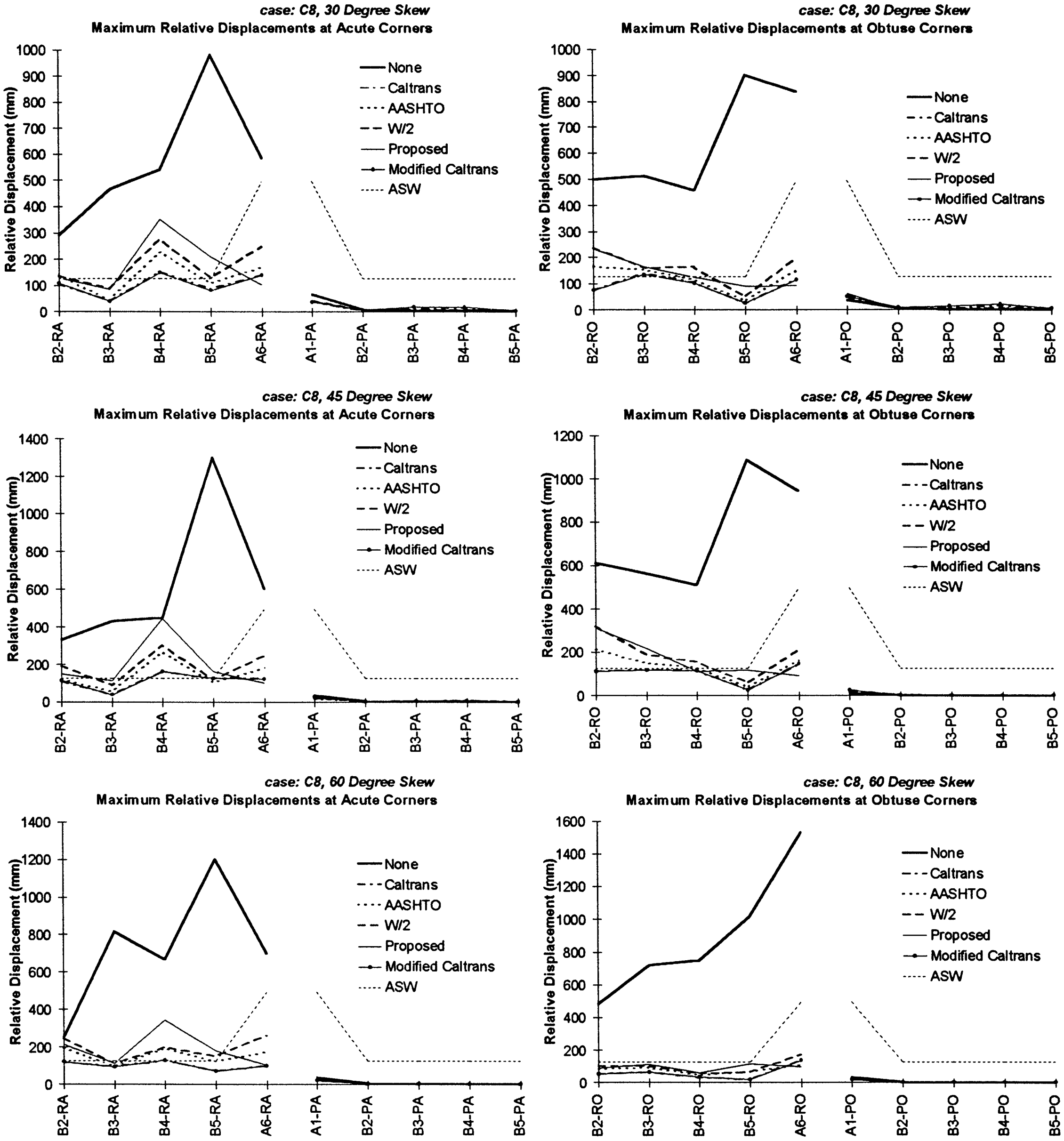


FIGURE 6-34 Relative Displacements of Five-Span Skewed Bridges

In the 30 degree skew case all restrainer designs reduced displacements, but in many cases the reduction was not sufficient to keep the bridge seated. Generally, restrained displacements were smaller at obtuse corners than at acute corners. All of the restrainer designs failed to keep the bridge seated at B4-RA and B3-RO. At B2-RA both the Proposed and W/2 methods failed to keep the bridge seated, and at B5-RA the Proposed method again allowed the bridge to unseat.

The AASHTO, W/2, and Proposed methods allow loss of support at B2 and B3-RO, and W/2 allows unseating at B4-RO. Overall, in the 30 degree case, the Caltrans and Modified Caltrans methods were the most effective at restraining displacements. However, no method reduced displacements sufficiently to keep the bridge seated at all supports.

The restrained and unrestrained displacement response of the 45 degree skew case followed the same pattern as the 30 degree skew case. The amplitudes in the 45 degree skew case were higher than the 30 degree case, but the patterns were similar. Again all of the procedures allowed the bridge to become unseated at B4-RA. The Proposed method did not reduce displacements significantly at B4-RA.

The 60 degree skew response was the greatest out of all three cases. Displacements without restrainers were as great as 1.5m (4.9 feet) which would result in collapse. All restrainer designs performed well and kept the bridge seated at its obtuse corners. Displacements were reduced more than 600mm (2 feet) in many locations. At acute corners, the only methods that kept the bridge seated were the Caltrans and Modified Caltrans methods. The Proposed and W/2 restrainers allowed the bridge to become unseated in three locations, and AASHTO allows unseating at two locations. Overall, in all three skew cases the Modified Caltrans and Caltrans methods performed the best since they result in a number of restrainers that keep the bridge seated (or nearly so) most of the time.

Tables 6-13, 6-14, and 6-15 present the restrainer ductilities for all three skew cases. Note that in all of the pinned locations restrainers did not experience high ductilities because the pinned bearing helped keep the bridge seated. No restrainer elements saw displacement ductilities over 4.15, and in most cases did not see ductilities over 2 which represents a displacement of approximately 214mm (8.4 in.). A ductility of 1.18 indicates a displacement greater than the ASW.

In the 30 degree skew case, ductilities were lower than 3.3. In general, ductilities were largest at the ends of the bridge (B2-R, A6-R, and A1-P). This was the same trend as seen in the other studies of the five span bridge. Caltrans and Modified Caltrans performed similarly as expected. The restrainers designed based on these methods yielded the least number of times (5 locations each). The restrainers from the W/2 method yielded the most times (12 locations). It is interesting to note that the Modified Caltrans and Caltrans methods performed nearly the same even though the Modified Caltrans method required far fewer restrainers.

The highest ductility of 4.15 was in the case of the Proposed method with 45 degree skew. Again, Caltrans and Modified Caltrans restrainers performed similarly. Both of them yielded eight times which was three times more than the 30 degree skew case. Both the Proposed and W/2 restrainers experienced significant yielding in more cases than the other methods. Again, restrainers at pinned locations saw little action.

TABLE 6-13 Restrainer Displacement Ductilities, 30 Degree Skew Bridge

30 Degree Skew

	Caltrans	AASHTO	W/2	Proposed	Modified Caltrans
Roller Support, Acute Corners					
B2-RA	**1.02**	**1.30**	**1.27**	**1.20**	**1.01**
B3-RA	0.38	0.42	0.83	0.81	0.37
B4-RA	**1.43**	**2.14**	**2.58**	**3.28**	**1.41**
B5-RA	0.84	**1.02**	**1.23**	**1.94**	0.78
A6-RA	**1.31**	**1.60**	**2.33**	0.95	**1.31**
Roller Support, Centerline					
B2-RM	0.51	**1.01**	**1.32**	**1.16**	0.51
B3-RM	0.68	0.75	**1.00**	**1.03**	0.66
B4-RM	0.73	**1.12**	**1.38**	**1.84**	0.72
B5-RM	0.42	0.57	0.71	**1.07**	0.39
A6-RM	0.68	**1.29**	**1.90**	0.58	0.69
Roller Support, Obtuse Corners					
B2-RO	0.70	**1.55**	**2.23**	**2.18**	0.72
B3-RO	**1.30**	**1.44**	**1.49**	**1.54**	**1.27**
B4-RO	0.98	**1.07**	**1.53**	**1.16**	0.95
B5-RO	0.23	0.34	0.48	0.86	0.26
A6-RO	**1.11**	**1.44**	**1.86**	0.87	**1.08**
Pinned Support, Acute Corners					
A1-PA	0.37	0.38	0.40	0.38	0.36
B2-PA	0.04	0.05	0.07	0.05	0.06
B3-PA	0.13	0.08	0.05	0.07	0.18
B4-PA	0.14	0.07	0.07	0.06	0.19
B5-PA	0.04	0.03	0.03	0.03	0.04
Pinned Support, Centerline					
A1-PM	0.19	0.24	0.22	0.26	0.19
B2-PM	0.05	0.05	0.06	0.05	0.07
B3-PM	0.09	0.06	0.04	0.06	0.14
B4-PM	0.13	0.07	0.06	0.05	0.18
B5-PM	0.04	0.03	0.03	0.03	0.04
Roller Support, Obtuse Corners					
A1-PO	0.47	0.54	0.51	0.58	0.47
B2-PO	0.08	0.06	0.06	0.05	0.10
B3-PO	0.08	0.04	0.04	0.05	0.14
B4-PO	0.15	0.09	0.06	0.05	0.21
B5-PO	0.06	0.03	0.02	0.03	0.05

TABLE 6-14 Restrainer Displacement Ductilities, 45 Degree Skew Bridge

45 Degree Skew

	Caltrans	AASHTO	W/2	Proposed	Modified Caltrans
Roller Support, Acute Corners					
B2-RA	**1.05**	**1.16**	**1.80**	**1.42**	**1.06**
B3-RA	0.38	0.52	0.84	**1.05**	0.38
B4-RA	**1.52**	**2.52**	**2.85**	**4.15**	**1.52**
B5-RA	**1.19**	0.97	**1.12**	**1.53**	**1.17**
A6-RA	**1.18**	**1.70**	**2.31**	0.94	**1.17**
Roller Support, Centerline					
B2-RM	0.55	**1.07**	**1.97**	**1.82**	0.54
B3-RM	0.56	0.79	0.99	**1.25**	0.55
B4-RM	0.78	**1.36**	**1.55**	**2.29**	0.78
B5-RM	0.61	0.49	0.58	0.82	0.60
A6-RM	0.75	**1.34**	**2.10**	0.55	0.75
Roller Support, Obtuse Corners					
B2-RO	**1.05**	**2.00**	**3.00**	**2.89**	**1.05**
B3-RO	**1.12**	**1.39**	**1.77**	**2.01**	**1.10**
B4-RO	**1.08**	**1.20**	**1.44**	**1.04**	**1.08**
B5-RO	0.26	0.38	0.56	**1.10**	0.26
A6-RO	**1.38**	**1.52**	**2.01**	0.88	**1.38**
Pinned Support, Acute Corners					
A1-PA	0.27	0.14	0.21	0.19	0.27
B2-PA	0.04	0.04	0.04	0.05	0.04
B3-PA	0.06	0.05	0.04	0.05	0.07
B4-PA	0.07	0.04	0.05	0.04	0.10
B5-PA	0.03	0.03	0.03	0.04	0.03
Pinned Support, Centerline					
A1-PM	0.10	0.07	0.03	0.09	0.10
B2-PM	0.04	0.05	0.04	0.05	0.04
B3-PM	0.04	0.05	0.04	0.04	0.05
B4-PM	0.07	0.04	0.05	0.04	0.09
B5-PM	0.03	0.03	0.03	0.04	0.03
Roller Support, Obtuse Corners					
A1-PO	0.24	0.21	0.19	0.24	0.24
B2-PO	0.02	0.04	0.03	0.03	0.02
B3-PO	0.02	0.03	0.04	0.04	0.03
B4-PO	0.01	0.02	0.03	0.02	0.01
B5-PO	0.02	0.02	0.02	0.03	0.03

TABLE 6-15 Restrainer Displacement Ductilities, 60 Degree Skew Bridge

60 Degree Skew

	Caltrans	AASHTO	W/2	Proposed	Modified Caltrans
Roller Support, Acute Corners					
B2-RA	**1.09**	**1.82**	**2.33**	**2.01**	**1.12**
B3-RA	0.87	0.91	**1.05**	**1.04**	0.87
B4-RA	**1.21**	**1.79**	**1.85**	**3.22**	**1.21**
B5-RA	0.66	**1.16**	**1.43**	**1.67**	0.65
A6-RA	0.92	**1.62**	**2.47**	0.97	0.92
Roller Support, Centerline					
B2-RM	0.59	**1.11**	**1.49**	**1.24**	0.59
B3-RM	0.41	0.41	0.48	0.53	0.41
B4-RM	0.60	0.93	0.99	**1.75**	0.60
B5-RM	0.33	0.64	0.75	0.92	0.33
A6-RM	0.67	**1.12**	**1.73**	0.45	0.66
Roller Support, Obtuse Corners					
B2-RO	0.50	0.77	0.93	0.84	0.49
B3-RO	0.61	0.85	0.93	**1.02**	0.61
B4-RO	0.31	0.45	0.53	0.56	0.32
B5-RO	0.18	0.63	0.60	**1.11**	0.18
A6-RO	**1.30**	0.99	**1.64**	0.91	**1.29**
Pinned Support, Acute Corners					
A1-PA	0.22	0.25	0.30	0.26	0.21
B2-PA	0.02	0.03	0.03	0.03	0.03
B3-PA	0.03	0.03	0.02	0.03	0.03
B4-PA	0.02	0.03	0.03	0.02	0.03
B5-PA	0.03	0.03	0.04	0.04	0.04
Pinned Support, Centerline					
A1-PM	0.17	0.17	0.25	0.22	0.16
B2-PM	0.03	0.03	0.03	0.03	0.03
B3-PM	0.03	0.03	0.03	0.03	0.03
B4-PM	0.03	0.03	0.03	0.02	0.03
B5-PM	0.03	0.03	0.03	0.04	0.03
Roller Support, Obtuse Corners					
A1-PO	0.21	0.20	0.31	0.27	0.20
B2-PO	0.03	0.04	0.03	0.03	0.04
B3-PO	0.03	0.03	0.03	0.03	0.03
B4-PO	0.04	0.03	0.03	0.02	0.04
B5-PO	0.04	0.03	0.03	0.03	0.04

In the 60 degree skew case, the Modified Caltrans and Caltrans restrainers only yielded in three locations. It was interesting to note that the 60 degree skew case had the highest displacements without restrainers, but the demand on the restrained cases was generally lower than the demand in the 30 and 45 degree cases. Except for at A1-P, restrainers saw very little demand at pinned locations.

Table 6-16 shows abutment displacement ductilities for all skew cases. Note that abutment ductilities in the skewed cases were slightly lower than those in the non-skew bridges with narrow seats (figure 6-22). The ductility values of zero indicate that in the 60 degree case the pinned abutment was not even engaged.

TABLE 6-16 Abutment Displacement Ductilities for Five-Span Skewed Bridges

30 Degree Skew

	None	Caltrans	AASHTO	W/2	Proposed	Modified Caltrans
A1-PA	0.94	0.41	2.29	0.43	0.33	1.30
A1-PO	2.21	1.81	1.22	3.02	3.31	0.91
A6-RO	1.70	2.80	0.61	2.90	1.87	1.55
A6-RA	0.44	0.67	0.65	0.64	0.68	0.94

45 Degree Skew

	None	Caltrans	AASHTO	W/2	Proposed	Modified Caltrans
A1-PA	0.52	0.12	0.01	0.16	0.20	0.13
A1-PO	1.73	1.43	0.86	1.28	1.09	1.44
A6-RO	2.19	2.87	1.27	1.79	1.39	2.81
A6-RA	1.57	0.45	0.70	0.66	0.50	0.44

60 Degree Skew

	None	Caltrans	AASHTO	W/2	Proposed	Modified Caltrans
A1-PA	0.00	0.00	0.00	0.00	0.00	0.00
A1-PO	0.00	0.00	0.00	0.00	0.00	0.00
A6-RO	3.22	1.57	1.32	1.52	1.43	1.58
A6-RA	0.67	0.33	0.45	0.44	0.43	0.33

Since the Kobe earthquake record generated responses that were much higher than other records in the main study, the response of a skew bridge subjected to a less severe earthquake was also studied. The San Francisco Airport record generated the second most critical bridge response for five span-bridges in the main study; therefore, the 60-degree skew case was subjected to both components of this record. The 60-degree skew angle was chosen because larger unrestrained displacements were observed with this skew angle than other skew angles.

The relative displacements of the bridge subjected to the San Francisco Airport record are presented in figure 6-35. In the figure, RSW is the required seat width as determined with the AASHTO seat width equation (see section 2.4). Note that the unrestrained displacements exceeded the available seat width at both the obtuse and acute corners indicating loss of support. Also note that the unrestrained displacements exceeded the required seat width in some locations indicating that the AASHTO equation underestimates displacements. All of the restrainer designs reduce relative displacements enough to keep the bridge supported except at B4-RA, where the Proposed method fails to keep the bridge seated.

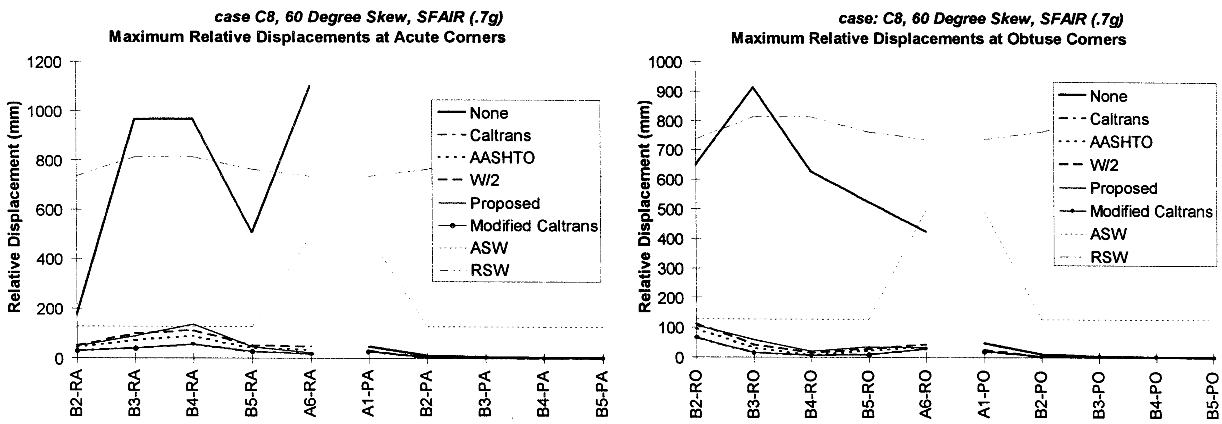


FIGURE 6-35 Relative Displacements of Five-Span Skewed Bridges Subjected to the San Francisco Airport Earthquake Record

6.3.4.2 Restrainers Modified with the Skew Factor

Since skew bridges equipped with restrainers designed with current methods were subject to loss of support (see section 6.3.4.1), a factor (the Skew Factor) that increases the required number of restrainers was developed. In developing the factor, it is noted that the longitudinal displacement, Δ, is a component of displacement perpendicular to the skewed support, $\Delta/\cos\alpha$, where α is the skew angle (figure 6-36). Because restrainers are designed to control longitudinal movement, and because the restrainer force is proportional to displacement, the design force perpendicular to the skewed support is increased by $1/\cos\alpha$. This increase would lead to an increase in the restrainer area by a Skew Factor of $1/\cos\alpha$.

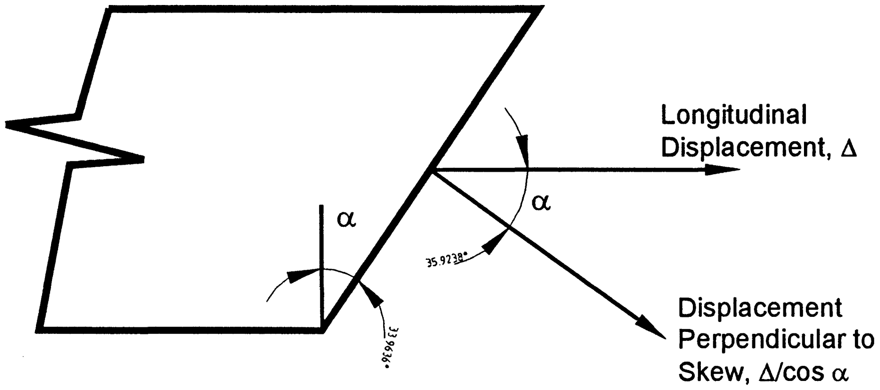


FIGURE 6-36 Displacement in Skewed Bridges

The original number of restrainers on skewed bridges is increased by the Skew Factor. For example, a 30-degree skew would require a 15 percent increase in the number of restrainers. Table 6-17 presents the number of restrainers required by each method for the 60 degree skew five-span bridges. Since the cosine of 60 degrees is one-half, the original number of restrainers required by each method was increased by a factor of two.

TABLE 6-17 Modified Restrainer Designs for 60 Degree Skew Five-Span Bridges

	Number of Restrainers										
Method	A1-P	B2-R	B2-P	B3-R	B3-P	B4-R	B4-P	B5-R	B5-P	A6-R	Total
Proposed	42	32	24	26	24	26	24	32	20	78	328
Caltrans	76	76	98	98	98	98	98	98	76	76	892
AASHTO	38	38	48	48	48	48	48	48	38	38	440
W/2	28	28	36	36	36	36	36	36	28	28	328
Modified Caltrans	76	76	24	98	24	98	24	98	24	76	618

The 60 degree skew case was analyzed with restrainer numbers increased by the skew factor. The Kobe record was used for loading as in section 6.3.4.1. Figure 6-37 shows the relative displacements for bridges equipped with the modified number of restrainers. Restrainers reduced relative displacements in all cases, but the AASHTO, W/2, and Proposed method restrainers still

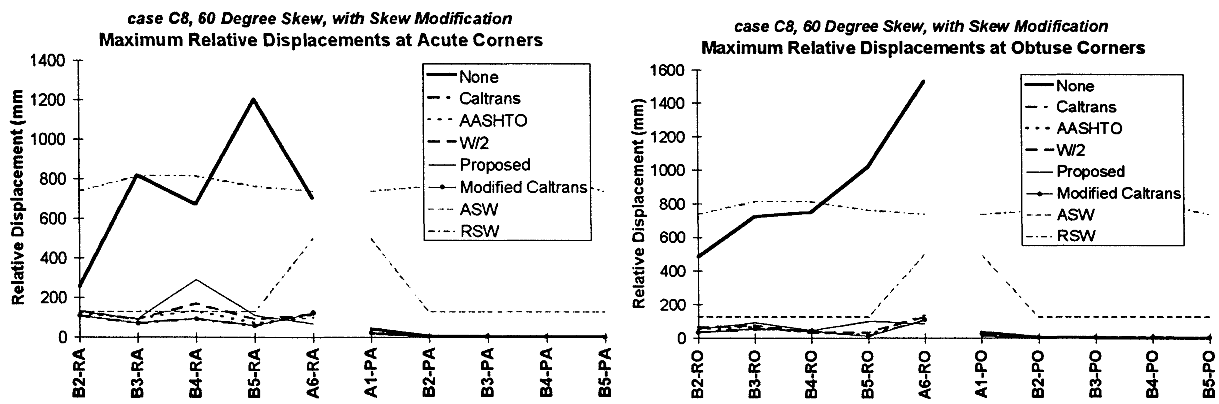


FIGURE 6-37 Relative Displacements for 60 Degree Skew Bridge with Restrainer Designs Modified for Skew

did not keep the span supported at B4-RA. The Modified Caltrans and Caltrans restrainers prevented loss of support in all locations. Also note that the unrestrained displacements exceeded the AASHTO seat width requirements (RSW).

6.3.5 Abutment Restrainers

It was shown in section 6.2 that large relative displacements at the abutments can be expected, yet restrainers are not installed at the abutments frequently, in part, because restrainers are difficult to install at abutments. Therefore, the influence of abutment restrainers on the response of the five-span bridge from the main study was investigated. The maximum displacements for case C1

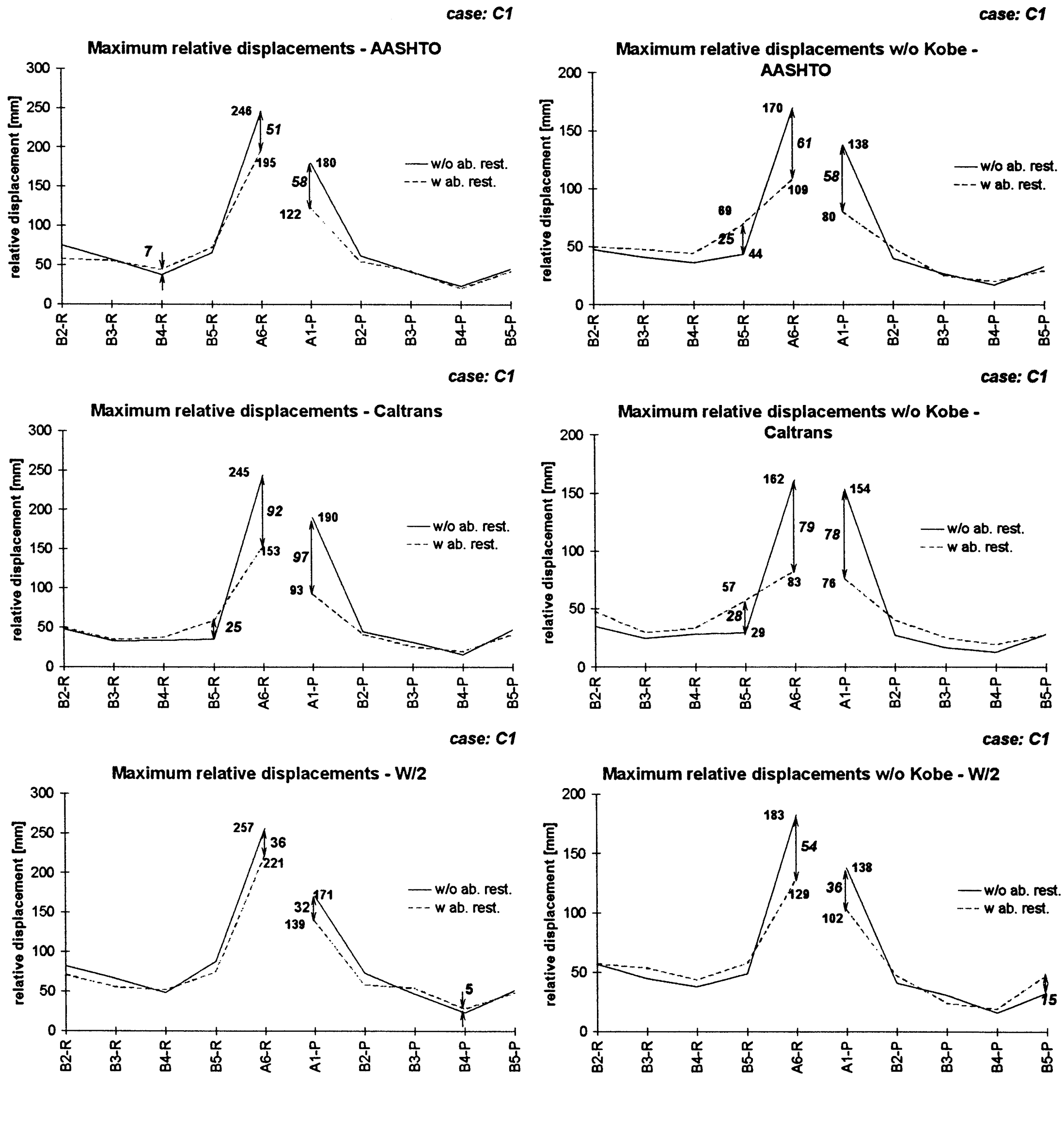


FIGURE 6-38 Maximum Relative Displacements in Five-Span Bridges With and Without Abutment Restrainers

It is evident from figure 6-38, that abutment restrainers had a large influence on the maximum (weak bearings and stiff substructure) with restrainers designed according to AASHTO, Caltrans and W/2 methods were determined (figure 6-38).

relative displacements only at the abutments. Displacements were smaller if a larger number of abutment restrainers were used. Therefore, the largest reduction was observed in the structure with restrainers designed according to the Caltrans method. In that case, the maximum relative displacements at the abutments were decreased by nearly 50 percent. The reduction of relative displacement at the abutments was smaller, but still significant for the other two methods.

6.3.6 Minimum Number of Restrainers

In FHWA, 1995, the minimum restrainer design force is 0.35 multiplied by the dead load of the superstructure. This minimum design force is used for Seismic Performance Category B in AASHTO, by Caltrans on bridges at sites with low seismic risk, and in the Proposed method when unrestrained displacements are less than the available seat width. In this section, bridges retrofitted with the minimum number of restrainers were studied. It was assumed that the minimum number of restrainers was required at all locations of the bridge. It was shown in

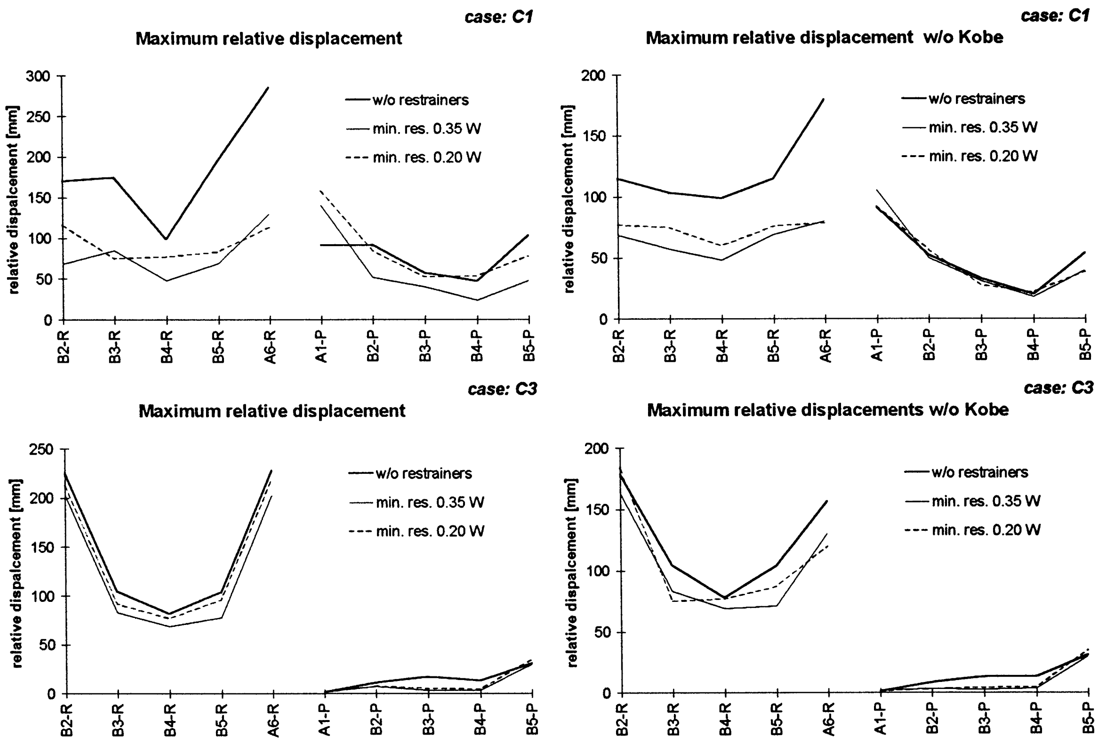


FIGURE 6-39 Maximum Relative Displacements in Five-Span Bridges with a Different Minimum Number of Restrainers

section 6.2 that no significant yielding in restrainers was observed in structures with a minimum number of restrainers. Therefore, a possible reduction of the minimum number of restrainers was studied to determine its influence on the response.

A reduction of the minimum design force to 0.2W was investigated in two different cases. Since bearings were observed to have a large effect on response, cases C1 and C3 were chosen because of their different bearing properties. The five-span bridge from the main study was investigated. Response was evaluated in terms of critical relative displacements for cases with no restrainers, and both minimum numbers of restrainers. Figure 6-39presents envelopes of displacements for cases including and excluding the use of the Kobe record.

A minimum number of restrainers is only used at locations where a span is not likely to unseat. Another possible use of minimum restrainers is in areas of low seismic risk as a precaution. A reduction in the minimum number of restrainers would not be advised if there were a large difference in response between the bridge with the reduced minimum number of restrainers and the bridge with restrainers designed to resist 0.35*W.

It is evident from the figure that a different a minimum number of restrainers did not affect the response significantly. A smaller minimum number of restrainers led to larger displacements that were still smaller than displacements in structures with no restrainers in most cases. Unseating was not a possibility due to the large seat that corresponds to the main study model. Since the difference in response between restrainers designed for 0.2*W and 0.35*W was insignificant, a reduction in the minimum number of restrainers is recommended.

SECTION 7
SUMMARY AND CONCLUSIONS

7.1 Summary

Since the 1971 San Fernando earthquake in California, restrainers have been used to prevent spans from becoming unseated in earthquakes. The most common type of restrainer for simply-supported bridges in the U.S. is a cable restrainer that is attached from the bent cap to the span. Caltrans and AASHTO currently have guidelines under which cable restrainers may be designed.

Many studies have been performed on cable restrainers including field and analytical studies that have mostly shown that cable restrainers are effective in reducing unwanted relative displacements between spans and their seats. Many studies have also shown that non-linear finite element models may be used to predict bridge response to ground motion. In recent earthquakes, such as the Northridge and Loma Prieta earthquakes in California, restrainers have been proven to prevent bridges from collapse; however, bridges such as the Gavin Canyon Overcrossing and the Northwest Connector on Interstate 14 have collapsed because of inadequate restrainer performance.

Currently, Caltrans uses an equivalent static analysis to design restrainers for their simply supported bridges. This procedure uses response spectra and assumes that the only structural component that functions in an earthquake is the restrainer. The AASHTO restrainer design procedure states that the force that a restrainer set must withstand is the site's acceleration coefficient multiplied by the weight of the span. The main objective of this study was to evaluate the current design procedures and to formulate and evaluate more efficient alternatives to the current procedures.

Three new design procedures were developed: the Proposed method, the W/2 method, and the Modified Caltrans method. The Proposed method is similar to Caltrans in that it uses response spectra to determine the force a restrainer set must withstand. However, the Proposed method includes the effect of column stiffness, footing stiffness, bearing stiffness and strength, and abutment stiffness and strength, to determine an equivalent vibration period that is used with response spectra to determine restrainer forces. In the W/2 method, restrainers must withstand one-half of the weight of the span that they are restraining. The Modified Caltrans method combines part of the Proposed method with the Caltrans procedure.

The study was mainly on the Caltrans, AASHTO, Proposed, and W/2 methods. The four design methods were evaluated in this study with non-linear finite element models using the computer program DRAIN-3DX. The finite element bridge models were developed from a data base of typical bridges. Typical two- and five-span simply supported bridges were included. Properties of these bridges were varied in a parametric study. The ranges of the parameters were selected so that a large number of bridges were represented. The bridge models were subjected to five different ground motions. The results were tabulated and observations made. Nearly 500 analyses were performed in this study.

The research was divided into a main and supplementary study. In the main study, none of the bridges became unseated without restrainers because of the wide seat in both the two- and five-span bridges. The Caltrans method generally reduced displacements the most but required disproportionally the largest number of restrainers. In the two-span bridge, the W/2 method often required the least number of restrainers and was effective at reducing critical relative displacements. An effectiveness factor was defined to indicate the reduction in relative displacement per restrainer. The W/2 method had the highest effectiveness factors in most of the two-span bridges. The Proposed method performed well in the main study of the two-span bridges, and it performed the best out of all of the methods in the five-span bridges. The Proposed method always required the least number of restrainers and reduced displacements satisfactorily in the five-span cases. The most notable qualities of the Proposed method were that it predicted locations of high displacement demand on the bridges and that it predicted locations where restrainers were not needed due to the sufficient strength of bearings. The drawback of the Proposed method is that it requires more computation than the other methods.

In the supplemental study the Modified Caltrans procedure was formulated and evaluated. In this procedure the connection between the span and bent must be evaluated first. If the connection has strength under seismic forces, a minimum number of restrainers is used. If the connection is insufficient, restrainers are designed according to the Caltrans method.

Special bridge cases were also investigated under the supplemental study. A limited number of bridges with narrow seats, bridges subjected to long duration ground motion, skewed bridges, bridges with and without abutment restrainers, and bridges with different minimum numbers of restrainers were studied.

In bridges with narrow seats, all of the methods prevented unseating in the two-span model, but all of the methods except Caltrans and Modified Caltrans led to unseating in some supports of the five-span model. A long duration earthquake had no significant effect on plastic deformations or the maximum response of both models. In the skewed models, all of the restrainer designs failed to keep the bridge seated unless the number of restrainers was increased by a skew factor proposed in this study. Abutment restrainers were found to affect relative displacements significantly at abutments, and a smaller minimum number of restrainers was found to have little effect on the response of the five-span bridge.

7.2 Observations

- Of the restrainer design procedures evaluated in this study the Proposed method is the only procedure that is capable of predicting whether or not the bridge will become unseated without restrainers.

- In the main study, in all cases, the bridge never becomes unseated without restrainers. The Proposed method is the only procedure that predicts this in many locations.

- Bridges perform more uniformly with restrainers designed with the Proposed procedure. Critical relative displacements were similar at abutments and at bents in both the two- and five-span models in the main study.

- The Proposed method utilizes restrainers more effectively than other methods do. This was shown with displacement ductilities that were closer to one more often than the other methods.

- The Proposed method predicts locations of high displacement demand, but in some cases it does not provide sufficient restrainers to keep the superstructure seated. This problem could be addressed with an additional number of restrainers in critical locations.

- The Caltrans, AASHTO, and W/2 methods do not predict locations of high displacement demand.

- The Modified Caltrans method is effective in reducing displacements at locations that are subject to loss of support. In bridges with narrow seats, the Modified Caltrans method required as much as 30 percent fewer restrainers than Caltrans and kept the bridge seated. It can prevent unseating in the largest number of cases in a more efficient manner than the Caltrans method.

- With the exception of skew bridges, Caltrans requires a number of restrainers that is excessive. In the main study of the two- and five-span bridges, the reduction in relative displacements provided by Caltrans was not substantially different than that provided by other methods even though the number of restrainers required was considerably greater.

- Skewed bridges with Caltrans restrainers were subject to loss of support.

- For most bridges, the AASHTO and W/2 methods are acceptable for design, but for cases where seat widths are narrow and the bridge location is known to have high seismic risk or cases where pinned bearings have high strength and bents are very flexible, a more rigorous or conservative method is recommended.

- All of the design methods that neglect skew effects allowed spans to become unseated in skew bridges. However, skewed spans never unseated at locations where girders were supported by pinned bearings.

- When modified by the Skew Factor, the number of restrainers required by the Caltrans and Modified Caltrans methods was sufficient to keep all spans seated.

- Bearings need to be included in restrainer design. Bearing strength significantly affects the response of simply supported bridges and the design of restrainers.

- Relative displacements were much smaller than the available seat width at locations with medium and high strength bearings in all cases. A minimum number of restrainers was sufficient at such locations.

- The two-span bridges in this study had seat widths much smaller than AASHTO requirements and still never were subject to unseating. Five-span bridges that satisfied AASHTO seat width requirements never unseated.

- The AASHTO seat width equation overestimates the amount of seat width necessary for strait non-skewed bridges and underestimates the amount of seat width necessary for skewed bridges.

- Longer duration earthquakes can affect the maximum response of a bridge and its elements. However, in this study, a longer duration input motion had little or no impact on the effectiveness of restrainers. Duration of input earthquakes should be considered carefully when evaluating bridge response using non-linear finite element analysis.

- The reduction in critical relative displacements provided by restrainers is not necessarily proportional to the total number of restrainers on a bridge or at a location. Putting restrainers on a bridge can increase critical relative displacements. An increase in displacements was observed at some pinned supports, abutments, and bents adjacent to abutments.

- Abutment restrainers generally reduce displacements at abutments. Their impact on the response of the other parts of a bridge is limited.

7.3 Conclusions

- The Modified Caltrans procedure is an effective method for designing restrainers to limit relative displacement at supports and prevent unseating. It is recommended for use in all cases. In cases where seats are narrow, restrainers should be designed with the Modified Caltrans method. In skew cases, the number of restrainers should be modified by the Skew Factor.

- The Proposed method is recommended for all cases except for skew bridges and for structures with narrow seats when the substructure is very flexible.

- The Caltrans method is an effective and satisfactory method for designing restrainers to limit relative displacement at supports and prevent unseating. In cases where seats are narrow Caltrans will keep the bridge seated. In skew cases, the number of restrainers should be modified by the Skew Factor.

- The W/2 and AASHTO methods are effective and satisfactory procedures for designing restrainers to prevent unseating. However, their use should be limited to bridges that have wide seats (say, 450 mm (18 in.) or more) and bridges that are not skewed or have a small skew angle (say, 20 degrees or less).

- All restrainer design methods included in the study are satisfactory for designing restrainers for 2-span bridges that are not skewed.

- Bearing strength has a great influence on the relative displacements at superstructure seats and should be considered in restrainer designs. Restrainers are not needed at locations that have pinned bearings with sufficient strength.

- The minimum number of restrainers can be reduced to 0.2*W.

- Straight, non-skewed two- and five-span bridges do not need restrainers to prevent unseating if AASHTO seat width requirements are met.

- Abutment restrainers are recommended when abutment seats are narrow

SECTION 8
REFERENCES

American Concrete Institute (ACI), (1995), " Building Code Requirements for Reinforced Concrete and Commentary," ACI 318-95 and ACI 318R-95, 1995.

American Association of State Highway and Transportation Officials (AASHTO), (1996), "Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials, Sixteenth Edition, 1996.

Anderson, John G., Quaas, R., Almora, D., Velasco, J.M., Guevara, E., Eduardo de Pavia R., Luis, Gutierrez R., A., and Vazquez L., R., (1987),"Guerrero, Mexico, Accelerograph Array: Summary of Data Collected in the Year 1985," Seismological Laboratory, Mackay School of Mines, University of Nevada - Reno, Reno, Nevada, September 1987.

Astaneh-Asl, A., and Kanada, M., (1995), "Seismic Behavior of Steel Bridges During the 1994 Northridge and the 1995 Hanshin Earthquakes," Proceedings of the National Seismic Conference on Bridges and Highways, San Diego, California, December 1995.

Bjornsson, S., Stanton, J., and Eberhard, M., (1997), "Seismic Behavior of Skewed Bridges," Report No. SGEM 97-1,Department of Civil Engineering, University of Washington, Seattle, Washington, June 1997.

Brettmann, T., and Duncan, J.M., (1996), "Computer Application of CLM Lateral Load Analysis to Piles and Drilled Shafts," American Society of Civil Engineers, Journal of Geotechnical Engineering, June 1996, pp. 496-498.

California Department of Transportation (Caltrans), (1994), "Memo to Designers, 20-3, 20-4," California Department of Transportation, May 1994.

Caltrans, (1990), "Bridge Design Aids," California Department of Transportation, June 1990

Caltrans, (1990), "Bridge Design Specifications," California Department of Transportation, June 1990.

Carter, M., and Bentley, S. P., (1991), "Correlations of Soil Properties," Pentech Press, London, 1991.

Clough, R. W., and Penzien, J., (1993), Dynamics of Structures, Second Edition, McGraw-Hill, Inc., 1993.

Darwish, I., Saiidi, M., Norris, G., and Maragakis, E., (1997), "Determination of In-Situ Footing Stiffness Using Full-Scale Dynamic Field Testing," Report No. CCEER-97-3, Civil Engineering Department, University of Nevada, Reno, October 1997.

Degenkolb, Oris, (1978), " Bridge Notes, Retrofitting Techniques for Highway Bridges," State of California - Department of Transportation, Office of Structures, Vol. 20-No.1, August 1978.

Eurocode 8, (1994) "Design Provisions for Earthquake Resistance of Structures, Part 2 - Bridges," ENV 1998-2, October 1994.

Federal Highway Administration (FHWA), (1995), "Seismic Retrofitting Manual for Highway Bridges," Publication No. FHWA-RD-94-052, U.S. Department of Transportation Research and Development, Turner - Fairbank Highway Research Center, May 1995.

Federal Highway Administration, (1986), "Seismic Design of Highway Bridge Foundations, Volume II: Design Procedures and Guidelines," Publication No. FHWA-RD-86-102, U.S. Department of Transportation Research and Development, Turner - Fairbank Highway Research Center, June 1986.

Fenves, G., and Desroches, R., (1994), "Response of the Northwest Connector in the Landers and Big Bear Earthquakes," Report No. UCB/EERC-94/12, Earthquake Engineering Research Center, University of California, Berkeley, December 1994.

Fenves, G., and Desroches, R., (1997), "Design Procedure for Intermediate Hinge Restrainers for Multiple Frame Bridges," National Seismic Conference on Bridges and Highways, Sacramento, California, July 1997, pp. 667-681.

Hardin, B.O., and Drnevich, V.P., (1972), "Shear Modulus and Damping in Soils: Design Equations and Curves," Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Engineers, Vol. 98, No. SM7, July 1972.

Jennings, P.C., (1983), "Engineering Seismology," Reprinted From Earthquakes: Observation, Theory, and Interpretation, 1983.

Kawashima, Kazuhiko, (1995), "Impact of Hanshin/Awaji Earthquake on Seismic Strengthening of Highway Bridges," Report No. TIT/EERG 95-2, Department of Civil Engineering, Tokyo Institute of Technology, December 1995.

Mander, J.B., Kim, D-K, Chen, S. S., and Premus, G. J., (1996), "Response of Steel Bridge Bearings to Reversed Cyclic Loading," Report No. NCEER-96-0014, National Center for Earthquake Engineering Research, State University of New York at Buffalo, November 1996.

Maragakis, E.A., Saiidi, M., and Abdel-Ghaffar, S., (1992), "Evaluation of the Response of the Whitewater Bridge During the 1986 Palm Springs Earthquake," Proceedings of the Eighth U.S.-Japan Workshop on Bridge Engineering, Chicago, Illinois, May 1992, pp. 336-350.

Maragakis, E.A., (1984), "A Model for the Rigid Body Motions of Skewed Bridges," Report No. EERL 85-02, Earthquake Engineering Research Laboratory, California Institute of Technology, Pasadena, California. December 1984.

Maroney, B., Kutter, B., Romstad, K., Chai, Y.H., and Vanderbilt, E., (1994), "Interpretation of Large Scale Bridge Abutment Test Results," Caltrans Third Annual Seismic Research Workshop, Sacramento, California, June 1994.

Norris, G.M., (1994), "Seismic Bridge Pile Foundation Behavior," Proceedings of the International Conference on Design and Construction of Deep Foundations, December 1994, pp. 27-134.

O'Connor, D.N., Saiidi , M., and Maragakis, E.A., (1993), "Effect of Hinge Restrainers on the Response of the Madrone Drive Undercrossing during the Loma Prieta Earthquake," Report No. CCEER-92-9, Civil Engineering Department, University of Nevada, Reno. February 1993.

Prakash, V., G.H. Powell, and S. Campbell, (1993), "Drain-2DX base program description and user guide : Version 1.10 /," Structural Engineering, Mechanics and Materials, Department of Civil Engineering, UCB/SEMM-93/17, University of California, Berkeley, June 1993.

Prakash, V., Powell, G.H., and Filippou, F.C., (1993), "DRAIN-3DX: Base Program User Guide," Structural Engineering, Mechanics and Material, Department of Civil Engineering, University of California, Berkeley, November 1993.

Saiidi, M., (1995), "Guam Earthquake Reconnaissance Report – Bridges," Earthquake Spectra, Journal of Earthquake Engineering Research Institute, Vol. 11 (Supplement B), April 1995, pp. 45-62.

Saiidi, M., and Maragakis, E.A., (1995), "Effectiveness of Hinge Restrainers as Seismic Retrofit Measure," Fourth International Bridge Engineering Conference Proceedings, Transportation Research Board, San Francisco, California, August 1995

Saiidi, M., Maragakis, E.A., Abdel-Ghaffar, S., Feng, S., and O'Connor, D.N., (1993), "Response of Bridge Hinge Restrainers During Earthquakes -- Field Performance, Analysis, and Design," Report No. CCEER-93-6, Civil Engineering Department, University of Nevada, Reno, May 1993.

Saiidi, M., Maragakis, E.A., and Feng, S., (1992), "An Evaluation of the Current Caltrans Seismic Restrainer Design Method," Report No. CCEER-92-8, Civil Engineering Department, University of Nevada, Reno, October 1992.

Saiidi, M., Maragakis, E.A., Feng, S., (1996), "Parameters in Bridge Restrainer Design for Seismic Retrofit," American Society of Civil Engineers, Journal of Structural Engineering, January, 1996, 122(1):55-61.

Seed, H.B., and Idriss, I.M., (1969), "Influence of Soil Conditions on Ground Motions During Earthquakes," Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 95, SM1, January 1969, Pp. 99-135.

Selna, L. G., Malvar, L. J., and Zelinski, R. J., (1989), “Bridge Retrofit Testing: Hinge Cable Restrainers,” American Society of Civil Engineers, Journal of Structural Engineering, April 1989, (115)4: 920-934.

Singh, S.P., and Fenves, G., (1994), “Earthquake Analysis and Response of Two Level Viaducts,” Report No. UCB/EERC-94/11, Earthquake Engineering Research Center, University of California, Berkeley, October 1994.

Trochalakis, P., Eberhard, M. O., and Stanton, J.F., (1995), “Evaluation and Design of Seismic Restrainers for In-Span Hinges,” Report No. WA-RD 387.1, Washington State Transportation Center, Seattle, Washington, August 1995.

Trochalakis, P., Eberhard, M. O., and Stanton, J.F., (1996), “Unseating of Simply Supported Spans During Earthquakes,” Report No. WA-RD 387.2, Washington State Transportation Center, Seattle, Washington, May 1996.

Yang, Y.- S., Priestley, M.J.N., and Ricles, J.M., (1994), “Longitudinal Seismic Response of Bridge Frames Connected by Restrainers,” Report No. SSRP-94/09, Structural Systems Research, University of California, San Diego, August 1994.

Yashinsky, M., (1995), "Lessons Learned from the January 17, 1995 Kobe Earthquake," Proceedings of the National Seismic Conference on Bridges and Highways, San Diego, California, December 1995.

Yashinsky, M., Hipley, P., and Nguyen, Q., (1995), “The Performance of Bridge Seismic Retrofits During the Northridge Earthquake,” Caltrans Office of Earthquake Engineering, June 1995.

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