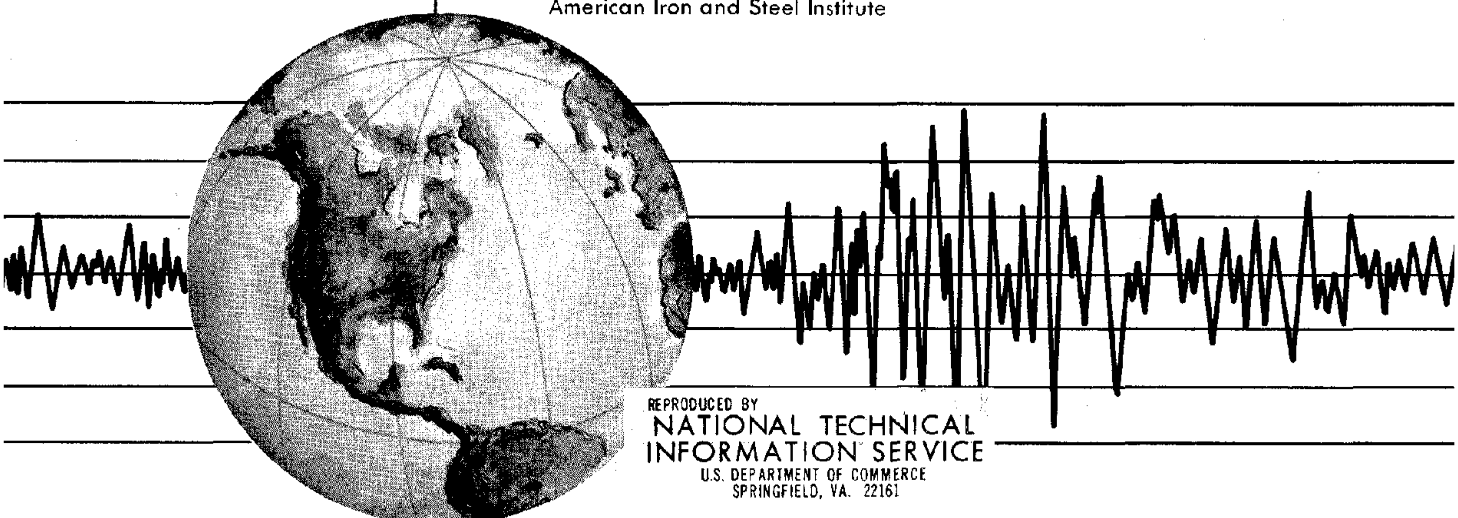
REPORT NO.
UCB/EERC-83/03
JANUARY 1983

DESIGN OF LINKS AND BEAM-TO-COLUMN CONNECTIONS FOR ECCENTRICALLY BRACED STEEL FRAMES

by

EGOR P. POPOV
JAMES O. MALLEY

Report to Sponsors:
National Science Foundation
American Iron and Steel Institute



COLLEGE OF ENGINEERING

UNIVERSITY OF CALIFORNIA · Berkeley, California

For sale by the National Technical Information Service, U.S. Department of Commerce, Springfield, Virginia 22161.

See back of report for up to date listing of EERC reports.

50272-101

REPORT DOCUMENTATION PAGE	1. REPORT NO. NSF/CEE-83010	2.	3. Recipient's Accession No. PB83 194811

4. Title and Subtitle	5. Report Date
Design of Links and Beam-to-Column Connections for Eccentrically Braced Steel Frames	January 1983
	6.
7. Author(s) Egor P. Popov and James O. Malley	8. Performing Organization Rept. No. UCB/EERC-83/03
9. Performing Organization Name and Address Earthquake Engineering Research Center 47th Street & Hoffman Blvd. Richmond, Ca. 94804	10. Project/Task/Work Unit No. 11. Contract(C) or Grant(G) No. (C) (G) CEE-81-07217
12. Sponsoring Organization Name and Address National Science Foundation, 1800 G. Street, N.W., Washington, D.C. 20550 American Iron and Steel Inst., 1000-16th Str., N.W., Washington, D.C. 20036	13. Type of Report & Period Covered 14.

15. Supplementary Notes

16. Abstract (Limit: 200 words)

This report is prepared to be Chapter 11 of the upcoming ASCE Manual on Beam-to-Column Building Connections under review by members of the Monograph Task Committee of the Committee on Structural Connections of the ASCE Structural Division.

This chapter first introduces the concept and points out some of the advantages of eccentrically braced frames (EBFs) for seismically resistant steel construction. A brief exposition of some of the available design procedures for such frames is provided. An approach is given for appraising the performance of the active links, i.e., beam segments between braces and/or columns, for achieving a stiff elastic structure together with an indication of the ductility demands placed on the links at extreme cyclic overloads. Suggestions for link length selection are made based on experimental results with some 28 full-size isolated links. Some of the obtained results in these experiments are described and selected hysteretic loops for cyclic loadings are given. Illustrated in this chapter are suggested details for the following items: link-column connections, link-brace connections, and spacing and sizing of link-web stiffeners. Possible applications of EBFs in non-seismic design are also pointed out.

17. Document Analysis a. Descriptors

b. Identifiers/Open-Ended Terms

c. COSATI Field/Group

18. Availability Statement:	19. Security Class (This Report)	21. No. of Pages
Release Unlimited		67
	20. Security Class (This Page)	22. Price

(See ANSI-Z39.18) See Instructions on Reverse OPTIONAL FORM 272 (4-77) (Formerly NTIS-35) Department of Commerce

DESIGN OF LINKS AND BEAM-TO-COLUMN CONNECTIONS FOR ECCENTRICALLY BRACED STEEL FRAMES

by

Egor P. Popov
Professor of Civil Engineering

and

James O. Malley
Research Assistant

Report to

National Science Foundation
and
American Iron and Steel Institute

Report No. UCB/EERC-83/03
Earthquake Engineering Research Center
University of California
Berkeley, California

January 1983

ABSTRACT

This report is prepared to be Chapter 11 of the upcoming ASCE Manual on Beam-to-Column Building Connections currently under review by members of the Monograph Task Committee of the Committee on Structural Connections of the ASCE Structural Division.

This chapter first introduces the concept and points out some of the advantages of eccentrically braced frames (EBFs) for seismically resistant steel construction. Because of the special features encountered in the analysis of such frames, and a very limited literature on this developing subject, a brief exposition of some of the available design procedures is provided. An approach is given for appraising the performance of the active links, i.e., beam segments between braces and/or columns, for achieving a stiff elastic structure together with an indication of the ductility demands placed on the links at extreme cyclic overloads. Suggestions for link length selection are made based on experimental results with some 28 full-size isolated links. Some of the obtained results in these experiments are described, and selected hysteretic loops for cyclic loadings are given. Together with the information given in Chapter 6 (Report No. UCB/EERC-83/02), the experimental data on links provide guidance for design and detailing of active links. Illustrated in this chapter are suggested details for the following items: link-column connections, link-brace connections, and spacing and sizing of link-web stiffeners. Possible applications of EBFs in non-seismic design are also pointed out. The chapter concludes with the needs for future research.

Three EERC reports, providing greater detail on the problems discussed herein, are in preparation, respectively, by Keith Hjelmstad, Kazuhiko Kasai, and James Malley as principal authors.

TABLE OF CONTENTS

List of Figures

CHAPTER 11 - ECCENTRICALLY BRACED FRAMES (EBFs)

By Egor P. Popov and James O. Malley
Department of Civil Engineering
University of California
Berkeley, CA 94720

11.1 INTRODUCTION

11.1.1 General

For resisting lateral loads caused by wind or earthquake, either moment-resisting or diagonally braced framing is commonly employed in structural steel design. As pointed out in Chapter 6 (Section 6.2.2), the story drift of a moment-resisting frame depends on four factors: bending of the columns, axial deformation of the columns, flexure of the beams, and the shear deformation of the column panel zone. If necessary, the deformation of a panel zone can be kept small with the use of doubler plates. The contribution of the columns to the story drift can also be contained within reasonable bounds. However, the limiting story drift due to flexure of the beams may require the use of larger beams than necessary for strength alone. Such a solution is costly. Therefore, if functional considerations make it feasible to use diagonal bracing, their choice becomes a more economical option.

Any number of diagonal bracing systems can be used. Ordinarily such bracing is so arranged that at a joint the centerlines of beams, columns, and braces meet at a point. All such systems may be referred to as concentrically braced frames (CBFs). Such systems are frequently used along the narrow dimensions of buildings, either for wind or seismic applications, since they provide an economical solution for drift control.

However, in some situations the braces cause undesirable obstructions within a bay and, for lateral loads due to an extreme seismic disturbance, their carrying capacity under cyclic loads may be poor. As has been shown by Popov and Black [1981], even initially concentrically loaded struts when subjected to severe cyclic load reversals can dramatically decrease in their compressive strength. Fortunately, such poor behavior of an individual member does not decrease the capacity of a multiply redundant frame to the same extent. Meager experimental results [Maison and Popov, 1980] nevertheless indicate that it may be difficult to achieve good overall frame ductility with the types of CBFs used in building construction.

The above problems with CBFs and moment-resisting frames suggest another possibility. By deliberately offsetting the diagonal braces at joints, a hybrid frame is obtained which can have the advantages of rigidity at moderate loads and, as has been shown experimentally [Roeder and Popov, 1977,1978a], can have good ductility at extreme overloads. Some alternative arrangements of this kind of bracing are shown in Fig. 11.1. The use of such eccentrically braced frames (EBFs) appears to have been first suggested by Spurr [1930] for architectural reasons in wind bracing. This concept can also be used to advantage for reducing the size of nominally concentric connections (see Section 11.4).

The specific use of eccentric connections in eccentric K-braces for seismic applications was studied by Fujimoto *et al.* [1972]. Experimental and analytic results on single diagonal EBFs of the type shown in Fig. 11.2 reported by Roeder and Popov [1977,1978a] provided a renewed interest in this type of framing. This chapter is primarily concerned with recent developments on eccentric connections for such applications. However, first a few remarks regarding the analysis of such bracing systems are necessary because of the special features encountered in their analysis.

11.1.2 Analysis of EBFs

Any bracing scheme in which the diagonal braces are deliberately offset from the beam-column joints (as illustrated in Fig. 11.1) can be classified as an EBF. By offsetting the diagonal braces from common joints, the axial forces from a brace are transferred to a column or to another brace through shear and bending in a portion of a beam called the active link. The diagonal braces are proportioned so as not to buckle by having greater strength than the supporting beam. As there is considerable flexibility in locating the braces in an EBF, functional requirements can be more easily met than with conventional bracing.

If an EBF is adopted for wind bracing, the usual elastic methods of analysis suffice. However, if an EBF has a primary function of resisting lateral loads caused by possible seismic disturbances, the procedure is more complex. An EBF must be designed for factored loads using plastic methods of analysis, and then be checked for code compliance (elastic behavior) at working loads. Only in this manner can proper functioning of the frame for dissipating energy through ductile behavior (see Section 6.1.2) of the active links be assured, and global column buckling prevented.

The above approach was illustrated by Roeder and Popov [1977] and Popov and Roeder [1978] on the simple EBF shown in Fig. 11.2 with prescribed loading conditions. The appropriate mechanism motions (or collapse mechanisms; see Section 6.1.2) for this frame in two possible directions are shown in Fig. 11.3. After applying the plastic moment balancing procedure [Horne, 1954; Gaylord, 1966] for the loading condition shown in Fig. 11.2, the balanced distribution of moments shown in Fig. 11.4 was found. This lower bound solution can be improved by changing the proportion of shear in the two columns. However, the obtained solution as is can be used for a preliminary selection of member sizes. If this were done, the columns would have to be considered in single curvature over three stories. Fortunately, because of the great flexibility of the plastic design method, such conditions need not arise in practice.

A direct plastic design procedure based on a generalized portal method of analysis has been developed by Kasai [1983] in which the columns assume double curvature in each story for any fixed loading condition. Using this approach, a solution of the same problem is shown in Fig. 11.5. In this solution it was assumed that plastic moments develop in the beams at the left column (M_p at the top and M_p^* in two lower beams; see definitions in Section 11.1.4), and smaller plastic moments develop on the right at both ends of the links (see definition (4) in Section 11.1.4). In this procedure, any plausible plastic beam moments can be assumed. As an example, in an improved design the moment at the upper left corner could be reduced. It is important to note, however, that in using the direct plastic design procedure, all critical moments conform to the requirements of the collapse mechanism given in Fig. 11.3, and the columns in each story are in double curvature. The obtained results give both the upper and the lower bound solutions for this plastic problem, and inelastic activity is confined to the appropriate locations. In this study, for simplicity, the effect of the axial forces acting on the links was neglected (see Section 11.5).

The static analysis approach described above is customarily used in seismic design of buildings. In reality, however, the problem is both non-deterministic and dynamic. Pauley [1983] has shown that moment-resisting frames during some instants of large earthquake motion behave very differently from what is assumed in static analysis. At some instants of time the columns in frames may be forced into single curvature over several story heights. However, the complex phenomenon of multistory column buckling under dynamic loadings is different from that occurring under static conditions and needs to be explored further. For this reason the sensitivity of EBFs to different bracing arrangements, variations in lateral loads, and their response under dynamic loading conditions are currently being studied at the University of California at Berkeley. In the process, procedures for better methods of analysis and design of EBFs are evolving. Their capability to meet the conflicting requirements of providing a stiff structure for light and moderate lateral loads and a ductile one for extreme overloads makes EBFs a viable alternative for seismic applications.

Inasmuch as the current codes, such as UBC [1982], specify elastic criteria, and there is always an interest in determining the behavior of frames at working loads, as noted earlier, plastically designed EBFs should be checked for elastic behavior at working loads.

11.1.3 Some Characteristic of EBFs

The basic characteristics of an EBF can be noted by examining the elastic as well as plastic behavior of a simple diagonally braced frame [Hjelmstad and Popov, 1982], such as shown in Fig. 11.6. The dependence of the elastic frame stiffness on the two parameters e/L and h/L is illustrated in Fig. 11.7. By varying the eccentricity ratio e/L from 0 to 1, the frame changes in character from a concentrically braced frame to a conventional moment-resisting frame. For all intermediate values of e/L, the frame becomes eccentrically braced. All three curves for different h/L's clearly show the advantages of bracing the frame to gain lateral stiffness in the system. With only a small loss of such stiffness, a brace can be placed slightly eccentric to the upper corner joint. By making the eccentricity ratio e/L large, a significant decrease in frame stiffness occurs. Hence, from the point of view of attaining a high degree of elastic stiffness, the ratio e/L should either be zero or as small as possible. If e/L is set at zero, the frame reverts to conventional concentric diagonal bracing and the desired ductility at extreme overloads may be difficult to attain. For all values of $e/L < 0.5$, the addition of bracing results in a substantial increase in stiffness. This increase becomes larger for narrower bays (e.g., $h/L = 1$), where the columns contribute less to the lateral stiffness of the frame. It is to be noted that the results shown are for a one-story frame for which member boundary conditions are different from those found in multistory frames. However, this diagram reflects qualitatively the importance of an eccentric brace on elastic frame stiffness.

Studies similar to the above [Hjelmstad and Popov, 1982] showed that the effect of shear deformation in the active links should be considered for $e/L < 0.5$. Neglecting these deformations in this range leads to an overestimate of the frame stiffness.

To gain insight regarding ductile behavior, it is necessary to examine the mechanism motion (collapse mechanism) of the frame such as shown in Fig. 11.8. This mechanism gives an indication of the extent of energy dissipation through plastic deformation and of ductility requirements in the critical regions. Recalling from Section 6.1.2 that the ultimate story drift index measured by angle θ is on the order of 1.5 to 2 percent, the kinematically compatible link deformation γ can be found. Thus, from simple considerations of frame geometry,

$$\theta L = \gamma e \quad . \tag{11.1}$$

It is to be carefully noted that since in EBFs e is usually much smaller than L, severe ductility requirements are placed on the link. The ductility demands on a beam in a moment-resisting frame, where $e = L$, are much smaller. The energy dissipation in these localized regions is critical to the performance of a frame during a major earthquake. Since in seismic design a complete lateral load reversal can be anticipated, the maximum feasible value for γ must be determined through cyclic experiments.

The correct choice of eccentricity e for an EBF can be arrived at only after carefully considering the two conflicting requirements of stiffness and ductility. In order to achieve a stiff structure, e must be small. However, for small values of e, the ductility demanded (measured by γ) of an active link may become excessive.

As an aid to the design of active links, which is the principal topic of this chapter, they are classified in the next section according to their length and location in a frame.

11.1.4 Classification of Active Links

In seismic design of EBFs, the concept of "strong columns-weak girders" is adhered to as it is for moment-resisting frames (Section 6.1.1). Except for the possible development of plastic hinges at column bases, inelastic activity is concentrated in the active links. A representative example of this condition is illustrated for the mechanism motion (collapse mechanism) of the frame in Fig. 11.3. From this figure several different kinds of plastic regions can be identified which can be classified into four types.

It is apparent that the links on the right undergo a great deal more inelastic activity than those on the left. Further, their behavior strongly depends on their length. If they are sufficiently long, plastic moment hinges form at both ends of the links. On the other hand, if these links are short they tend to yield in shear with smaller end moments. This differentiation between the two kinds of active link behavior is best illustrated with the aid of a shear-moment interaction diagram. A typical diagram for a wide-flange section is shown in Fig.

11.9 [Neal, 1961]. The relevant parameters are defined as follows:

$$M_p = F_y Z \tag{11.2}$$

$$M_p^* = F_y(d - t_f)(b_f - t_w)t_f \tag{11.3}$$

$$V_p^* = F_v^*(d - t_f)t_w \tag{11.4}$$

where M_p = plastic moment capacity of a beam;

M_p^* = plastic moment capacity of a beam reduced due to shear;

V_p^* = plastic shear capacity of a beam;

F_y = yield stress of steel;

F_v^* = shear yield stress of steel;

Z = plastic section modulus;

d = depth of beam;

t_f = flange thickness;

t_w = web thickness;

b_f = flange width.

The effect of shear is neglected in defining M_p, whereas M_p^* is based on the assumption that the web is in a plastic state and carries shear only. Equation (11.4) is written assuming that the web is yielding in shear. This equation is essentially the same as Eq. (6.3), which conforms to the AISC Specifications [1980], i.e., V_p^* very nearly equals V_p.

By considering the equilibrium conditions for an isolated link in a plastic state, one can obtain its length b* at the balance point, where M_p^* and V_p^* are reached simultaneously (see Fig. 11.9). This relation reads

$$b^* = 2M_p^*/V_p^* \quad . \tag{11.5}$$

Active links equal to or shorter than b* will yield predominantly in shear, and are called <u>shear links</u>. Those that are somewhat longer have a good deal of moment-shear interaction. The end moments of the long links will approach the plastic moment capacity M_p of the beam, and moment hinges will form at the ends of the links. Such links are referred to as <u>moment links</u>.

It is to be noted that for moment links a large increase in shear can take place with only a small change in moment. Conversely, for shear links the shear capacity remains essentially constant for a considerable range of end moments.

The inelastic behavior of the plastic regions on the left shown in Fig. 11.3 is quite different from the behavior of links on the right. The upper link, by virtue of its long length, is the classical moment hinge such as occurs in moment-resisting frames. Its moment capacity is given by Eq. (11.2). The remaining two beam links on the left require the formation of only one plastic moment hinge to develop the frame collapse mechanism. However, since large shear must be transmitted through them, their moment capacity must be limited to M_p^*, defined by Eq. (11.3). Both of these types of plastic hinges experience moderate rotation; therefore, the imposed ductility demand is small. Note that no plastic hinges form at the lower ends of the braces, which is in complete agreement with the moment diagram shown in Fig. 11.5.

The above discussion suggests the possibility of the following four types of plastic regions that may develop in an EBF:

(1) Single plastic moment hinges having the full plastic moment capacity M_p of a beam. These occur at beam-to-column connections in long

beam segments and are identical to those of moment-resisting frames.

(2) Single plastic moment hinges having reduced plastic moment capacity due to shear. These occur at beam-to-column connections of short beam links. Their capacity usually is at or near M_p^*.

(3) Plastic moment hinges at both ends of an active link having a plastic capacity ranging from M_p^* to M_p depending on link length. When the end moments are at or near M_p, this system forms a moment link.

(4) Plastic moment hinges at both ends of a short active link developing plastic end moments of M_p^* or less, depending on the link length, with the link web yielding in shear. This is a yielding system forming a shear link.

Since in the EBFs the inelastic activity is mainly concentrated at or in the links, their performance is critical in seismic design, and their behavior is discussed in the next section.

11.2 EXPERIMENTAL RESULTS ON LINK BEHAVIOR

11.2.1 Experimental Setup for Studying Links

The original experiments [Roeder and Popov, 1977,1978a] were made on one-third scale subassemblages. In these models the W6 × 12 shear links had an effective panel size of approximately 11 × 6 in. (280 × 150 mm). The webs were 0.23 in. (6 mm) thick. In a prototype this translates into a non-standard W18 × 108 section with an 0.69 in. (18 mm) web. No standard W18 section can meet these requirements. The webs of the available sections are thinner, raising the possibility of web buckling. Precisely such buckling was observed in the next series of frame tests [Manheim, 1982]. Some preliminary suggestions for controlling web buckling were advanced earlier by Popov [1980], but it was evident that further experiments on active links were necessary, as no information was available on the behavior of short wide-flange beams under severe cyclic loading.

Since the purpose of this investigation was specifically the behavior of the active links, the experimental model isolated the link from the remainder of the structure. The model (Fig. 11.10b) was extracted from the two possible prototype configurations shown in Fig. 11.10 [Hjelmstad and Popov, 1982]. The fully welded end plates totally restrain warping in the link cross-section, a condition simulating the prototype structure, since the link is either adjacent to a region of the beam with low shear or is welded to a column flange. Welded flanges provided excellent torsional restraint, a recommended design approach at plastic hinges [AISC Specifications Part 2, 1980].

A schematic diagram depicting the manner of applying the loading is shown in Fig. 11.11. By transferring the shear force to the specimen through the rigid L-shaped member, the imposed loading consists of a constant shear force and linear variation of bending moment with absolute maximum of opposite sign at each end. This condition approximates reasonably well the behavior of a link in the plastic range. Although actually, initially the moments at the two ends may differ significantly from each other, and this condition may persist due to strain hardening. The loads were applied quasi-statically to prescribed cyclic displacement levels.

To date, 28 full-size link specimens have been tested at Berkeley

[Hjelmstad and Popov, 1982; Malley and Popov, 1982] on sections ranging in size from W12 × 22s to W18 × 60s. Some of the results obtained in these experiments are described below. In the completed series of experiments, no axial forces were applied to the links, a subject for future research.

11.2.2 First Series of Link Tests

In the first series of tests reported by Hjelmstad and Popov [1982], 15 full-size links were subjected to quasi-statically applied cycles of relative end displacement in the plane of the specimen's web. Because of severe ductility requirements that may be imposed on the links during strong seismic excitations, the behavior of specimens undergoing relative end displacement of ±3 in. (75 mm) and more were explored. The specimens were made from W18 × 35, W18 × 40, W18 × 60, W16 × 26, and W12 × 22 sections and were either 28 in. (710 mm) or 36 in. (910 mm) long. Because of the greater advantage in stiffening a frame (see Fig. 11.7) gained from the use of short links, the emphasis was directed toward a study of shear links (see Eq. (11.5)), although some links (W16 × 26s and W12 × 22s) were in the intermediate length range. The specific objective of this test program was to determine web stiffener requirements such that a link would attain the necessary ductility γ under cyclic loading. In this series of experiments the webs were reinforced with pairs of stiffeners either 3/8 in. (10 mm) or 1/2 in. (13 mm) thick extending to the outside of link flange. Some selected experimental results are given below.

Hysteretic loops from the experiment, together with a photograph at failure for an unstiffened specimen (#1), are displayed in Fig. 11.12. Similar information for a specimen (#4) with three pairs of stiffeners is given in Fig. 11.13. Both specimens were made from a W18 × 40 section of the same material, and were 28 in. (710 mm) long. As can be seen from Fig. 11.12, the specimen with the unstiffened web experienced serious deterioration in load carrying capacity. Due to severe web buckling, characteristic dips in the hysteretic loops are observed for this specimen. A dramatic improvement in shear link behavior was achieved by stiffening the web, Fig. 11.13. The web stiffeners delayed the initiation of web buckling until the ninth severe cycle, and the specimen achieved excellent ductility before material tearing caused failure. For a large number of cycles the hysteretic loops remained full, allowing material strain hardening to continually increase the load carrying capacity.

Two specimens, similar to the two above, were designed to investigate the effect of using panel zones of different sizes between stiffeners. One specimen (#5) employed two pairs of intermediate stiffeners spaced so that the center panel (11 in. (280 mm)) was larger than the two outside panels (8.5 in. (220 mm)). This choice was made because the outside panels, in addition to carrying the same shear as the center panel, are subjected to larger bending moments. Nevertheless, the web buckling became concentrated in the center panel, causing a rapid deterioration in the energy dissipation capacity of the specimen. A similar specimen (#3) with equally spaced stiffeners behaved better. Therefore, equal sizing of panel zones appears to produce more desirable shear link behavior.

Two specimens made from W12 × 22 sections were 36 in. (910 mm) long. Since for these links b* = 24 in. (610 mm), they are of an intermediate length. Their behavior was entirely different from that of the other links. The unstiffened specimen (#12) experienced early lateral torsional buckling and failed after four cycles, reaching the maximum end displacements of ±1½ in. (38 mm). Providing two pairs of flange stiffeners for the other specimen (#15) greatly improved the link behavior, and the specimen sustained six cycles with a maximum end displacement of 2 in. (50 mm) and retained its planar alignment. Unlike the requirement for equal panel zones for shear links, pairs of flange stiffeners must be placed near the supports. Following suggestions given in the ASCE Manual 41 [ASCE-WRC, 1971], these stiffeners were placed one and one-half times the flange width away from each of the link ends.

Based on the above experiments with links under cyclic loading, the following observations can be made.

(1) Shear links are more effective energy dissipators than moment links, although in frames shear links are likely to be subjected to larger ductility demands than longer moment links.

(2) Web buckling in shear links leads to a significant loss in both load carrying capacity and energy dissipation capability. Therefore, shear links require web reinforcement. Flange buckling alone does not seriously reduce the capacity of shear links.

(3) Flange stiffeners are required for moment links of intermediate length.

(4) All links strain-harden under repeated loads; however, shear links benefit more from this effect than moment links.

11.2.3 Second Series of Link Tests

In the second series of tests reported by Malley and Popov [1982], 13 additional full-size links were tested in a manner similar to that employed previously. Again, the emphasis was placed on determining shear link behavior, rather than that of moment links, and for that reason all link specimens were 36 in. (910 mm) long, utilizing 18 in. (450 mm) deep sections. Except for two specimens made from W18 × 60 sections, all others were W18 × 40s. The objectives pursued in this study were to determine the effect of loading history, stiffener detail and spacing, and end connection details. Although the use of stiffeners in pairs was found to be effective in delaying web buckling in the previous tests, it was thought that a more economical detail could be developed. Owing to encouraging initial success, whenever webs were stiffened in this series of tests they were stiffened on only one side of the web. Either 3/8 in. (10 mm) or 1/2 in. (13 mm) thick stiffeners extended to a link's longitudinal flange edge. Some of the highlights from this series of experiments are given below.

Three specimens were used to determine sensitivity of link behavior to loading history. One of these specimens (#16), made from a W18 × 60 section, was initially subjected to two large displacement pulses to induce web buckling prior to further cycling. The other W18 × 60 specimen (#18) was first given nine ±1 in. (25 mm) cycles prior to the usual incremental cycling. Both specimens withstood extensive cycling after the initiation of web buckling, a desirable characteristic of large unstiffened panels. Two W18 × 40 specimens with two 1/2 in. (13 mm) equally-spaced web stiffeners provided further information on the behavior of links with different histories of loading. One of these specimens (#24) underwent an application of monotonically increasing load. When the displacement reached the 7.2 in. (180 mm) limit of the testing apparatus, the load resisted by the specimen had dropped only 6 percent below the maximum. The other identical specimen (#20) was cycled after a large initial cycle which initiated a visually observable web buckling. This specimen generated good hysteretic loops during the cycling process, indicating excellent capability in energy dissipation. These results indicate that properly detailed shear links can dissipate large amounts of energy regardless of the loading history.

One specimen (#17) was designed to investigate the effects of providing shear links with one-sided web stiffening. The hysteretic loops of this specimen, shown in Fig. 11.14, were remarkably similar to those of an earlier specimen (#9) which was identical except that it employed pairs of stiffeners placed on both sides of the web. This test demonstrates that providing adequate stiffeners on one side is structurally equivalent to placing stiffeners on both sides of a web. For reasons of reduced welding cost, one-sided stiffeners for shear links appear to be preferable.

Further cost reductions in providing web stiffeners for shear links can be realized by relaxing the requirement of welding the stiffeners to both beam flanges as well as to the web. The detail of omitting a weld to one of the flanges was tested on another specimen (#21), and its behavior is illustrated in Fig. 11.15. Comparison of these hysteretic loops with those for a specimen with fully welded single stiffeners showed almost identical response before the initiation of web buckling. After buckling, however, the energy dissipation decreases more rapidly in the link with stiffeners welded to the web and one flange only.

In most EBFs the links are located such that one end of the link is connected to a column. These moment connections are subjected to loading similar to those encountered in moment-resisting frames, and the procedures described for their design in Chapter 6 are essentially applicable. However, the shear link energy dissipation mechanism and the associated web buckling phenomenon are characteristics unique to EBFs and the effect they have on connection performance had not been studied. Therefore, a series of tests were made to determine the behavior of shear link connections having conventional details.

Three shear link specimens employed the all-welded connections of the type shown in Fig. 6.24b, i.e., full penetration flange welds and fillet welds to a shear tab. Cyclic tests of these specimens demonstrated that this all-welded connection detail can withstand large ductility demands without detracting from the capacity of the link. The fillet welded web connection encountered no problems, even though strain hardening caused the peak load resisted by one of the specimens to be over 80 percent above the initial yield level. In this specimen (#27), a defective weld caused sudden failure of the flange weld in the heat affected zone. Figure 11.16 illustrates the excellent behavior of a specimen (#26) with three 3/8 in. (10 mm) equally-spaced stiffeners welded to the web only.

Two specimens utilized the flange welds and bolted web connections, per Fig. 6.24a. In both these specimens, the large shear forces generated during cyclic loading induced bolt slippage in the web connection. This bolt slippage transferred large shear forces to the flanges, resulting in sudden flange failures. One of the specimens (#22) deteriorated more rapidly than a similar all-welded specimen only in the post-buckling range. On the other hand, the other specimen (#28) failed earlier than any of the other tested specimens. The hysteretic loops for this specimen are shown in Fig. 11.17. Comparing this behavior with that of the welded specimen given in Fig. 11.16 indicates the superior behavior of welded web connections in shear links.

One specimen (#25) was designed and fabricated with a connection to a column web, similar to the detail given in Fig. 6.26c. In a cyclic experiment this specimen behaved quite well, as may be seen from the hysteretic loops shown in Fig. 11.18. In this detail, the presence of a large shear in the web reduces the moment to be transmitted through the flanges to M_p^*, which is smaller than the moment that can develop in connections for longer beams (see Section 6.4.2).

11.2.4 Principal Test Conclusions

The main emphasis in the experimental study of active links was directed toward determining the cyclic behavior of shear links. These links were found to be superior to moment links as energy dissipators. The behavior of the moment links is largely determined by the full plastic moment capacity of beams and is discussed in Chapter 6. Two relevant experiments for links of an intermediate length were included in the first series of experiments. The following tentative conclusions, mainly applicable to shear links, may be drawn from the completed tests:

(1) The test results indicate that the theoretical link balance length b*, where M_p^* and V_p^* are reached simultaneously (see Eq. (11.5)) appears to underestimate the link length where shear action predominates by about 15 percent. Therefore, a better estimate of the maximum length e_{max} in which shear behavior for a link predominates can be given as [Malley and Popov, 1982]

$$e_{max} = 4b_f t_f / t_w \quad , \qquad (11.6)$$

where the meaning of the terms is the same as that in Eq. (11.3).

As e_{max} increases to $2e_{max}$, the link's behavior will approach that of a moment link. For $e > 2e_{max}$, moment link action can be expected.

(2) For monotonic displacements, the link deformation γ (see Fig. 11.8) up to 0.20 can be resisted without significant loss in load carrying capacity. For cyclic loadings, it appears reasonable to assume γ values of ± 0.10.

(3) Experimental evidence indicates that web stiffeners must be used for short links and should be equally spaced. Reinforcement of the webs by shear tabs should be judiciously excluded in determining the web panel length. Stiffeners on only one side of a web suffice to prevent premature web buckling. Moreover, good pre-buckling link behavior was observed with web stiffeners not welded to the flanges.

(4) Because of both high moment and shear in the links, beam-to-column connections of the all-welded type should be used. For severe service the bolted web connections can develop slip, resulting in sudden

premature failure. Since in shear links the plastic moment capacity is M_p^* rather than M_p, an experiment with a beam-to-column web connection showed good performance. However, the suggestions made earlier (see Section 6.4.2) for improving this detail should be followed.

(5) It is to be noted that to date, no links were tested which were simultaneously subjected to shear, bending moments, and axial force. Therefore, it is advisable to limit the axial force transfer through the links by means of design.

11.3 DESIGN AND DETAILING OF ACTIVE LINK CONNECTIONS

The results of the experimental work presented in this chapter can be combined with those obtained in Chapter 6 to provide guidance for connection design and details in EBFs. This section will outline the suggested details for the following items: (1) link-column connections, (2) link-brace connections, and (3) web stiffeners.

11.3.1 Link-Column Connections

For eccentrically braced systems in which the active links are located adjacent to columns, the designer must understand the nature of the inelastic response of the frame to adequately detail the connection. If the active links are sufficiently long to be classified as moment links (see Section 11.1.4), the rotation capacity demand on the link-column connection is likely to be small. This can be checked by determining the γ value, as in Eq. (11.1). If the γ value is not excessive, the recommendations given in Chapter 6 will apply to the link-column connection of active links.

If shear links are to be employed, the ductility demand on the active links can become quite large. Since these links yield in shear, the link-column connection must be able to develop the full yield capacity of the active link. For properly stiffened links, strain hardening can increase the shear capacity by as much as 75 percent above the initial yield level. This must be taken into account in the design of the web connection. The test results indicate that welded web connections will provide the required shear capacity and ductility. The detail shown in Fig. 11.19 (and 6.24b), which employs a fillet-web-weld to a shear tab, was found to be satisfactory for links with large ductility demands. If a more direct shear transfer mechanism is required a full penetration web weld, as shown in Fig. 11.20 (and 6.24c), can be employed. This detail is more expensive, and, unless the welding sequence is carefully worked out, it may cause large locked-in stresses due to restraint.

For shear links with low ductility demands, a bolted web detail such as that shown in Fig. 6.21 should provide adequate capacity. The shear connection should be designed conservatively to insure that the full shear capacity of the section can be resisted. In all cases, the flanges should be connected to the column flange with full penetration welds so that the full moment capacity of the section can be developed.

The connection of active links to column webs follows guidelines similar to the suggestions given for column flange connections (see Section 6.4.2). For flange moment connections, the recommendations of Driscoll and Beedle [1982], as given in Fig. 5.32 of Chapter 5, should be followed. For shear links with large ductility demands, an all-welded connection detail, such as that shown in Fig. 11.22, should be employed. For shear links with smaller ductility demands, bolted web connections should perform in a satisfactory manner.

11.3.2 Link-Brace Connections

The link-brace connection must also be designed to develop the shear capacity of the active link. If shear links are employed, strain hardening can cause significantly larger brace forces. Roeder and Popov [1977] suggest that the brace, and therefore the link-brace connection, be designed for 1½ times the yield shear.

The link-brace connection shown in Figs. 11.19 to 11.22 illustrates a detail for a brace consisting of a pair of angles. A similar detail can be adopted for square tubes. To stiffen the connection and provide for easier alignment of the weld centroid, the gusset plate detail is fabricated of two plates made into a tee section. The angles are fillet welded (or bolted) to the gusset. Another detail in which the brace is welded directly to the beam flange has also been used in design applications.

Providing eccentric bracing makes the active links susceptible to lateral torsional buckling. For this reason, the link ends at the eccentric braces must be laterally supported. The link-beam connections shown in Figs. 11.19 to 11.22 are designed to reduce the tendency for lateral torsional buckling of the link-brace assembly.

11.3.3 Web Stiffeners

By providing structurally adequate web stiffeners, the deleterious effects of active link web and/or flange buckling can be avoided. These requirements depend on the imposed ductility demand on the links and in this context three different types of links can be recognized: links with small ductility demand, shear links, and moment links. Web stiffener requirements for each link type are discussed next.

11.3.3.1 Links with Small Ductility Demand

Analysis of the plastic collapse mechanism (or mechanism motion) often indicates that certain links will not experience large inelastic deformations. These relatively inactive links, such as the left links in Fig. 11.3, may then be made as short as convenient, compatible with the requirements of a plastic frame analysis. Figure 11.23 shows a detail of such a link. Since vertical components of axial brace forces are transferred through such links, large shearing stresses are likely to develop in these links. Therefore, in order to avoid the need for intermediate stiffeners, if compatible with plastic analysis, the link length should be made less than $25t_w$, where t_w is the beam web thickness. For the detail shown in Fig. 11.23, full-length stiffeners should be provided on both sides of the web at the intersection of the brace-beam centerlines. The half-depth stiffeners located below the flange aid the transfer of the brace shear force to the beam. Since these short links have good torsional resistance, generally no lateral bracing is required.

11.3.3.2 Shear Links

The stiffeners in shear links are required to delay and control the web buckling which these members can be subjected to during large inelastic deformations. Stiffener design can be separated into three stages: spacing, sizing, and detailing.

Stiffener Spacing: Hjelmstad and Popov [1982b] presented a set of equations for determining the appropriate stiffener spacing based on the amount of energy dissipation required of a shear link. These two empirical equations are:

$$\frac{a}{t_w} = 90 - 9 \ln \frac{E_{\Sigma}^{*}}{E_e} \tag{11.7}$$

$$\frac{a}{t_w} = 94 - 14 \ln \frac{E^{*}}{E_e} \quad . \tag{11.8}$$

The topological parameters in these equations are t_w, the web thickness, and a, the minimum panel dimension (usually the distance between stiffeners). The energy dissipation parameters are E_{Σ}^{*}, the total energy dissipated prior to buckling; E_e, the elastic energy stored by a link at yield; and E^{*}, the energy absorbed during the largest single pre-buckling cycle in a cyclic experiment. It is likely that Eq. (11.8) is conservative since it is not based on monotonic tests, and for this reason it is subject to future revision.

Without the results from a series of properly modeled inelastic dynamic analyses, the energy dissipation requirements of the active links must be approximated from the estimated ductility demand. The ductility μ will be taken as:

$$\mu = \frac{v_{max}}{v_y} \quad , \qquad (11.9)$$

where v_{max} is the maximum relative end displacement of the link and v_y is the relative end displacement at initial yield. For shear links the μ values are large (see Fig. 11.8).

For cyclic loading histories, where μ_i is defined as the ductility demand in the i-th cycle, the relationship between energy dissipation and ductility demand for an elasto-perfectly plastic material undergoing predominantly shear deformation can be expressed as:

$$\frac{E^*_\Sigma}{E_e} = 2 \sum_i (\mu_i - 1) \quad . \qquad (11.10)$$

For monotonic loadings the corresponding relationship is:

$$\frac{E^*}{E_e} = 2\mu - 1 \quad . \qquad (11.11)$$

The a/t_w ratio chosen should be the lesser of the two values given by Eqs. (11.7) and (11.8). For buildings located in regions of high probability of significant seismic activity, the typical values of this ratio range between 25 and 30. If calculations show this ratio to be 20 or less, the beam size should be revised.

For eccentrically braced systems in which the shear links are adjacent to the columns, the stiffeners are located in the panel zone contiguous to the link-column connection. For the connection detail with the web fillet welded to a shear tab, the equally spaced stiffeners should be spaced from the erection bolt line, as shown in Fig. 11.19. For full penetration welded or bolted web connections, spacing the stiffeners from the face of the column was found to be satisfactory. A similar approach can be used when the active link is connected to the weak column direction. Figures 11.20 to 11.22 illustrate the suggested stiffener spacing in these cases.

Stiffener Sizing: After web stiffener spacing has been selected, their size must be determined to satisfy two basic design requirements. First, the stiffeners must have sufficient axial strength to permit the web to develop tension field action. Second, the stiffeners must be rigid enough to prevent buckling of the whole link web as a single panel.

AISC Specifications [1978] give equations for the design of plate girder web stiffeners based on the work of Basler [1961]. Even though plate girder stiffeners must meet requirements similar to the two listed above, these equations cannot be applied in the design of shear link web stiffeners since they are based on elastic solutions. Because of the inelastic nature of shear link web buckling, an exact solution of the web stiffener design problem would be extremely complex and impractical for applications. Therefore, an approximate method must be devised for sizing of shear link web stiffeners.

Malley and Popov [1982] adapted the tension field theory approach for sizing the web stiffeners in shear links using a formulation similar to that employed by Adams, Krentz, and Kulak [1979]. Determination of the optimum orientation of the tension field and the corresponding solution of static equilibrium results in the following equation for the web stiffener axial force, P_s:

$$P_s = F_u t_w \frac{a}{2}\left[1 - \frac{a/h}{\sqrt{1 + (a/h)^2}}\right] \quad . \tag{11.12}$$

In this equation, because of strain hardening, the diagonal tensile capacity F_u was chosen as the ultimate strength of the material. As before, t_w is the web thickness, a is the stiffener spacing, and h is the clear distance between the flanges.

Shear link test results indicated that yielding of the stiffeners does not impair their behavior, and local stiffener buckling did not occur. Therefore, allowing yielding of the stiffeners and assuming web participation equal to one-half the flange width, the required area A_{st} for two-sided web stiffeners becomes:

$$A_{st} = \frac{P_s}{F_y} - \frac{b_f t_w}{2} \quad . \tag{11.13}$$

For one-sided stiffeners, by modifying the equation for eccentricity [Basler, 1961], the required area A'_{st} is:

$$A'_{st} = 2.4\left(\frac{P_s}{F_y} - \frac{b_f t_w}{2}\right) \quad . \tag{11.14}$$

In these equations, P_s is the stiffener force defined in Eq. (11.12), F_y is the uniaxial yield stress of the stiffener material, b_f is the beam flange width, and t_w is the beam web thickness. Even though over twice as much stiffener material is required, one-sided web stiffening is likely to be more economical than the two-sided detail because of the reduced welding costs.

Typically, web stiffeners are detailed so that they do not protrude outside the longitudinal edge of the beam flanges. For stiffeners which just reach the edge of the flanges, the required thickness t_{st} of two-sided stiffeners is:

$$t_{st} = \frac{A_{st}}{b_f - t_w} \quad . \tag{11.15}$$

For similar stiffeners on one side of the web only, the required thickness, t'_{st}, becomes

$$t'_{st} = \frac{2A'_{st}}{b_f - t_w} \quad . \tag{11.16}$$

In either case, the stiffener thickness should not be less than the link web thickness, t_w.

The equations presented above provide a method for satisfying the axial force requirement. In addition, the web stiffeners must possess sufficient rigidity to prevent web buckling of the whole link web as a single panel. This requirement, first noted by Timoshenko [1936], has been studied extensively for the case of elastic buckling. The most thorough study of the elastic problem was made by Rockey and Cook [1961, 1962,1964].

The complex nature of inelastic plate buckling problems makes theoretical solutions for the required bending rigidity of shear link stiffeners impractical for design purposes. However, it is likely that the important parameters of the inelastic solution would be similar to those of the elastic problem. Some insight can therefore be gained by examining the elastic solutions, such as those presented by Wang [1947] and Stein and Fralich [1949]. These solutions demonstrate that the most important parameter in the determination of the required stiffener bending rigidity is the ratio of the smallest panel dimension to the web thickness (a/t_w). Since the a/t_w ratio for shear links is comparatively small, it can be argued that the stiffener flexural rigidity requirements are unimportant. The test results corroborate this observation, since no problems with the stiffener bending rigidity were encountered in any of the stiffened specimens. The bending stiffness provided by the axial force design method presented above appears to be sufficient to meet the bending rigidity design requirement.

Stiffener Detailing: The final step in shear link web stiffener design is determination of the proper details. As the test results presented earlier indicated, adequately designed one-sided stiffeners exhibit excellent behavior. The extra material costs are outweighed by the reduced welding costs, making one-sided web stiffening the more economical detail.

Additional savings can be realized by relaxing the requirement of fitting the stiffeners between the link flanges. The experimental work indicates that only a small reduction of energy dissipation capacity results from providing stiffeners whose length is slightly smaller than the clear distance between the link flanges. If this more economical detail is utilized, the stiffeners can be welded to the bottom flange as well as the web in an effort to restrain bottom flange buckling. In typical situations, the concrete floor can be counted upon to provide some buckling restraint for the unwelded top flange, although floor cracking may make this restraint poor in the event of a major seismic disturbance. This kind of detailing is shown in Figs. 11.19 to 11.22, where the top of an unfitted stiffener is kept a distance k below the outer face of the beam flange. Per the ASCE Manual [1978], k is taken as the distance from the outer face of flange to web toe. In all cases, the welds should be continuous fillet welds on both sides of the stiffeners, meeting AISC Specifications [1978] for minimum and maximum size.

11.3.3.3 Moment Links

As the eccentricity of the bracing elements is increased beyond e_{max} given in Eq. (11.6), the links progressively exhibit more and more moment hinge rotation with the associated problem of flange buckling, in a manner similar to that of typical moment-resisting frames. This difference in behavior changes the web stiffening requirements for the longer links.

For link lengths between e_{max} and $2e_{max}$, a transition in inelastic behavior occurs. For these intermediate lengths the shear link web stiffener design method presented in the previous section should be followed to determine the spacing and sizing. But, since moment hinge action can cause flange buckling in the regions of large moment at the ends of the link, the following modification should be included in the provision of the web stiffeners for these intermediate length links. The outer stiffeners should be placed no further than 1 to 1½ times the width of the beam flange, b_f, from the ends of these links. This suggestion is related to some work by Lay [1965] and Popov and Stephen [1972]. Since the outer stiffeners are provided to control link flange buckling, they should be placed on both sides of the beam web and welded to both beam flanges as well as to the web.

For link lengths between $2e_{max}$ and $3e_{max}$, moment hinge action will predominate, and web buckling should no longer be a problem. For links of these lengths, only the stiffeners placed a distance of 1 to 1½ times the flange width from each end of the link are necessary, as shown in Fig. 11.24. These stiffeners should be fitted between the beam flanges on both sides of the beam web since moment hinging can occur in these regions during severe inelastic action.

For links longer than $3e_{max}$, no web stiffeners appear to be necessary, since the moment hinge rotations become small.

11.3.4 Other Details

As pointed out in Chapter 6, two other aspects of steel joints, i.e., panel zones and column splices, must be considered in the design of any steel frame for resisting seismically induced lateral forces. The recommendations concerning these aspects given in Chapter 6 apply directly to EBF design. In addition, the following observations can be made. First, the panel zone deformations in columns of EBFs are likely to be small compared to those of conventional moment-resisting frames since the beam sizes should be smaller. Because of this, it is unlikely that the addition of doubler plates will be required in typical EBF applications. Second, the column splices must be designed conervatively. Because of the unknown nature of any potential seismic disturbance, and the redistribution of forces during inelastic activity in a frame, it is impossible to accurately determine the moments and shears at the splice locations (see Section 11.1.2).

11.4 NON-SEISMIC APPLICATION OF ECCENTRIC BRACING

In addition to seismic applications, EBFs may also be advantageous in other design situations. In fact, as pointed out in Section 11.1.1, this system was first suggested for architectural reasons in wind bracing [Spurr, 1930]. The flexibility of brace location inherent to EBFs can result in fewer obstructions to the architectural features of a building.

Structures designed to resist wind induced lateral forces are generally required to respond elastically. If eccentric bracing is provided, elastic analysis and design methods usually would be employed. Since active link web buckling will not occur during an elastic response, web stiffeners are not required for EBFs designed for wind loads. In addition, the suggested additional safety factor of 1.5 to preclude the possibility of brace buckling can be relaxed, since the links will not strain harden. The bolted web, welded flange link-column connection (see Fig. 6.24a) can be employed in such applications.

Another possible application for eccentric braces can arise because of the large and expensive connections which result solely from the geometrical requirement of forcing the centerlines of all the members to pass through a common working point. These large and expensive concentric connections can be avoided by using a detail such as that shown in Fig. 11.19 or 11.22. An alternative detail, shown in Fig. 11.25, shows a modified concentric connection which does not require any field welding. Moving the working point to the face of the column makes the detail more compact and less expensive.

11.5 PROJECTED RESEARCH ON EBFs

The results of the research to date have provided some of the needed information for the design of EBFs. However, there are still some aspects of analysis and design which have not been fully resolved. The ongoing and future research as well as the experience gained from design applications continue to contribute toward a better understanding of EBFs.

None of the previous experiments have addressed the problem of axial force transfer through the links. A judicious location of the links, or the use of parallel gathering beams, can minimize the transfer of large seismically induced axial forces through the links. In order to study the effects of large axial forces on active links, a number of tests are planned in which an axial force will be applied in addition to the shear force F (see Fig. 11.11).

Comprehensive analytical studies on the local active link response and the global frame behavior are being continued. These studies are intended to provide rapid analytical procedures for preliminary design of EBFs based on plastic methods. As indicated in Section 11.1.2, the elastic design methods do not account for the redistribution of forces which occur during the inelastic response of an EBF. The critical importance of the overall frame ductility requires the use of plastic analysis techniques, at least in the preliminary design process. After this initial sizing, elastic analysis can be made to check code compliance and the provision for the required frame stiffness.

Additional information on the behavior of EBFs should result from the current U.S.-Japan Cooperative Research Program [1982-1984]. This program will include a set of pseudodynamic tests on a full-size, six-story eccentrically braced steel building at Tsukuba, Japan. A scale model of this structure will subsequently be tested at Berkeley.

11.6 ACKNOWLEDGEMENTS

It is a pleasure to acknowledge with gratitude the support over several years of the research reported in this chapter by NSF (current Grant CEE-07217), as well as by AISI. During the early stages of the research an AISI Advisory Committee was most helpful in offering useful suggestions and assisted in directing the work along practical lines. Keith Hjelmstad and Kazuhiko Kasai, doctoral students in Civil Engineering at U.C. Berkeley, were very helpful with the final preparation of this chapter. Cindy Polansky provided competent service in typing the manuscript.

References

Adams, P.F., Krentz, H.A., and Kulak, G.L., "Limit State Design in Structural Steel," Canadian Institute of Steel Construction, 1979.

AISC, "Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings with Commentary," in Manual of Steel Construction, 8th ed., American Institute of Steel Construction, 1980.

ASCE-WRC, "Plastic Design in Steel," ASCE Manual 41, 2nd ed., Welding Research Council and American Society of Civil Engineers, New York, 1971.

Basler, K., "Strength of Plate Girders in Shear," J. Struc. Div., ASCE, No. ST-7, 151-180 (Oct. 1961).

Driscoll, G.D. and Beedle, L.S., "Suggestions for Avoiding Beam-to-Column Web Connection Failure," AISC Engrg. J. (1st Qtr) 19, No. 1, 16-19 (1982).

Fujimoto, M., Aoyagi, T., Ukai, K., Wada, A., and Saito, K., "Structural Characteristics of Eccentric K-Braced Frames," Trans. AIJ, No. 195 (May 1972).

Gaylord, E.H., "Plastic Design by Moment Balancing," Steel Structures Symposium, Univ. of Illinois, Urbana (October 1966).

Hjelmstad, K.D. and Popov, E.P., "Cyclic Behavior and Design of Link Beams," Preprint SC-7, ASCE Structures Congress, New Orleans, Oct. 1982. In review for publication in Journal of the Structural Division, ASCE.

Hjelmstad, K.D. and Popov, E.P., "An Appraisal of Eccentrically Braced Frames," J. Struct. Div. ASCE, in review.

Horne, M.R., "A Moment Distribution Method for the Analysis and Design of Structures by the Plastic Theory," Proceedings, Institute of Civil Engineers 3, No. 1 (April 1954).

Kasai, K., "A Plastic Design Method for Eccentrically Braced Frames," Dept. Civil Engineering, University of California, Berkeley, Report CE299 (1983).

Lay, M.G., "Flange Local Buckling in Wide-Flange Shapes," J. Struct. Div., ASCE, 91, No. ST6, 95 (Dec. 1965).

Maison, B.F. and Popov, E.P., "Cyclic Response Prediction for Braced Steel Frames," J. Struct. Div., ASCE, 106, No. ST7, Proc. Paper 15534, 1401-1416 (July 1980).

Malley, J.O. and Popov, E.P., "Design of Shear Links in Eccentrically Braced Frames," J. Struct. Div. ASCE, in preparation.

Manheim, D.N., "On the Design of Eccentrically Braced Frames," D.Eng. thesis, University of California, Berkeley (Feb. 1982).

Neal, B.G., "Effect of Shear Force on the Fully Plastic Moment of an I-Beam," J. Mech. and Engrg. Sci. 3, No. 3, 258 (1961).

Pauley, T., "Deterministic Seismic Design Procedures for Reinforced Concrete Buildings," Engineering Structures, Vol. 5, No. 1, 79-86 (Jan. 1983).

Popov, E.P., "Seismic Behavior of Structural Subassemblages," J. Struct. Div. ASCE, 106, No. ST7, 1451-1474 (July 1980).

Popov, E.P., "An Update on Eccentric Seismic Bracing," AISC Engrg. J. (3rd Qtr), 70-71 (1980).

Popov, E.P., "Recent Research on Eccentrically Braced Frames," in Proceedings, Structural Engineers Association of California, Coronado, Sept. 1981, pp. 69-80. Republished in Engineering Structures (Butterworth & Co. Ltd.), Vol. 5, No. 1, 3-9 (Jan. 1983).

Popov, E.P. and Black, R.G., "Steel Struts Under Severe Cyclic Loadings," J. Struct. Div. ASCE, 107, No. ST9, 1857-1881 (Sept. 1981).

Popov, E.P. and Roeder, C.W., "Design of an Eccentrically Braced Steel Frame," AISC Engrg. J. (3rd Qtr), 77-81 (1978).

Popov, E.P. and Stephen, R.M., "Cyclic Loading of Full-Size Steel Connections," Bulletin No. 21, American Iron and Steel Institute, New York (1972).

Rockey, K.C. and Cook, I.T., "Shear Buckling of Clamped and Simply Supported Infinitely Long Plates Reinforced by Transverse Stiffeners," Aeronaut. Qtrly XIII, 41-70 (Feb. 1962).

Rockey, K.C. and Cook, I.T., "Influence of the Torsional Rigidity of Transverse Stiffeners Upon the Shear Buckling of Stiffened Plates," Aeronaut. Qtrly XV, 198-202 (May 1964).

Rockey, K.C. and Cook, I.T., "Shear Buckling of Clamped Infinitely Long Plates - Influence of Torsional Rigidity of Transverse Stiffeners," Aeronaut. Qtrly XVI, 92-95 (Feb. 1965).

Roeder, C.W. and Popov, E.P., "Inelastic Behavior of Eccentrically Braced Steel Frames Under Cyclic Loadings," EERC Report 77-18, Earthquake Engineering Research Center, University of California, Berkeley (Aug. 1977).

Roeder, C.W. and Popov, E.P., "Eccentrically Braced Frames for Earthquakes," J. Struct. Div. ASCE 104, No. ST3, 391-412 (March 1978).

Roeder, C.W. and Popov, E.P., "Design of an Eccentrically Braced Steel Frame," AISC Engrg. J. (3rd Qtr), No. 3, 77-81 (1978).

Spurr, H.V., Wind Bracing, McGraw-Hill, New York, 1930.

Stein, M. and Fralich, R.W., "Critical Shear Stress of an Infinitely Long Simply Supported Plate with Transverse Stiffeners," NACA Technical Note 1851 (1949).

Timoshenko, S.P., Theory of Elastic Stability, McGraw-Hill, New York, 1936.

UBC (Uniform Building Code), International Conference of Building Officials, Whittier, California (1982).

Wang, T.K., "Buckling of Transverse Stiffened Plates under Shear," J. Appl. Mech. (ASME) 3, No. 4 (1947).

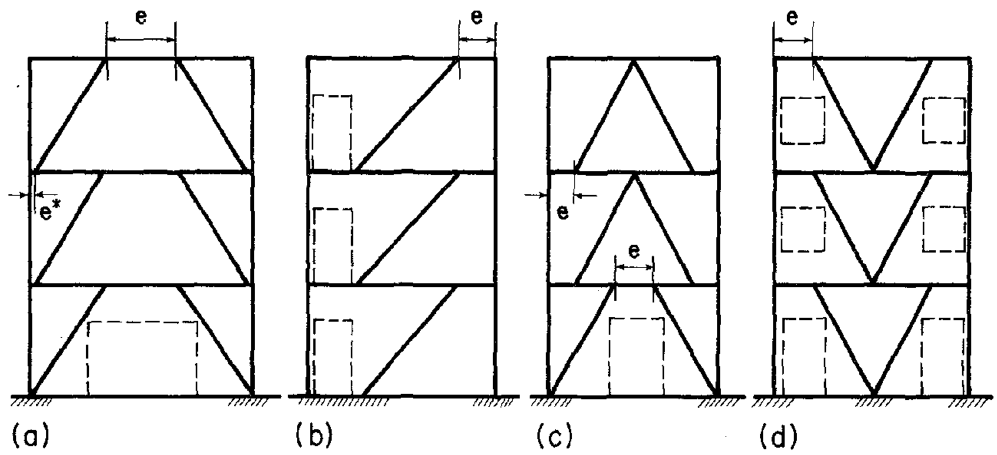


Fig. 11.1 Alternative Arrangements of Eccentric Bracing, Including Possible Locations for Architectural Openings.

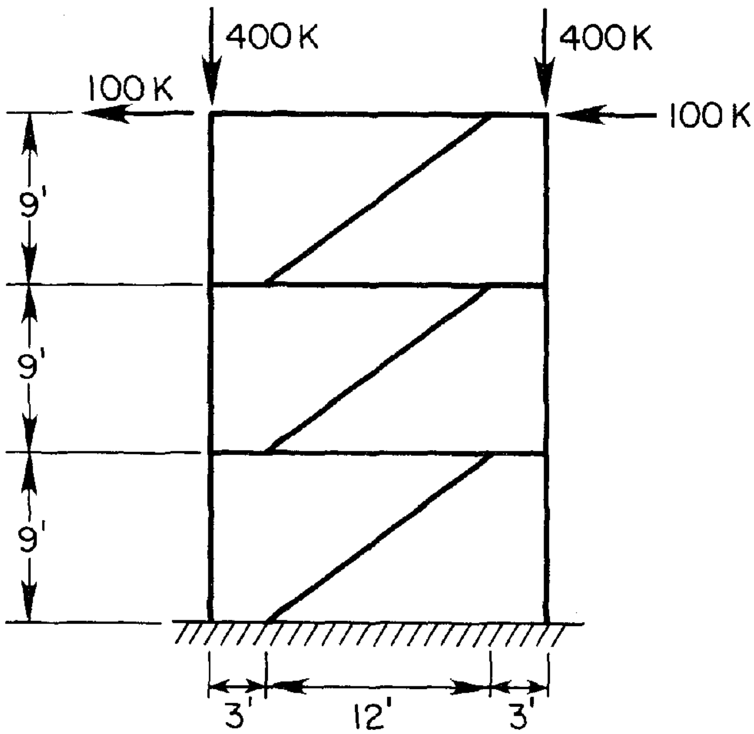


Fig. 11.2 Example Frame and Loading Used to Demonstrate the Preliminary Design Method.

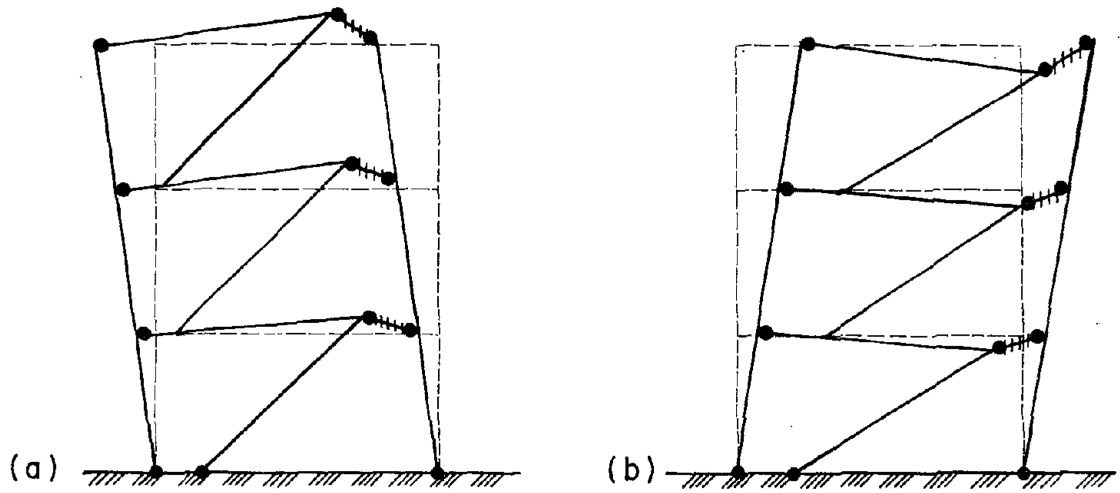


Fig. 11.3 Collapse Mechanisms (Mechanism Motions) in Opposite Directions Result in Identical Inelastic Activity.

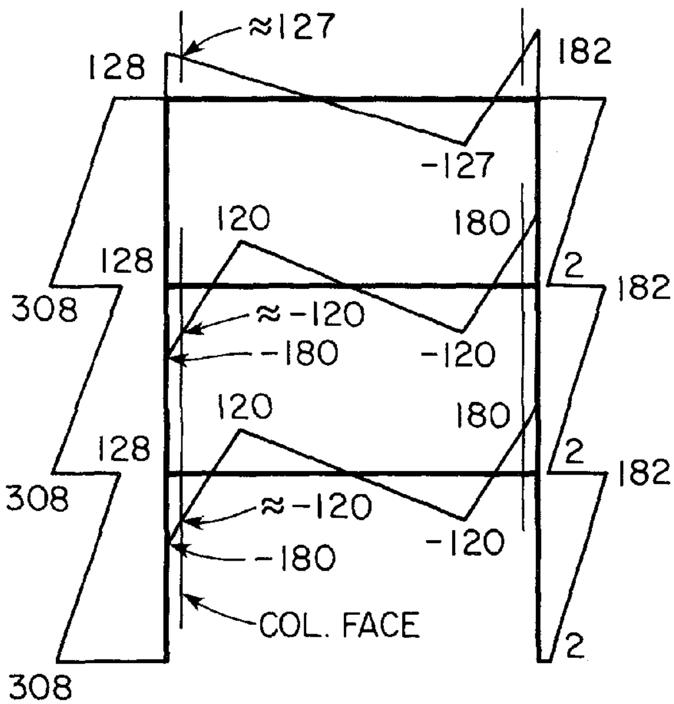


Fig. 11.4 Plastic Moment Distribution for Example Frame Assuming 20 percent of Shear Equally Distributed Between the Two Columns.

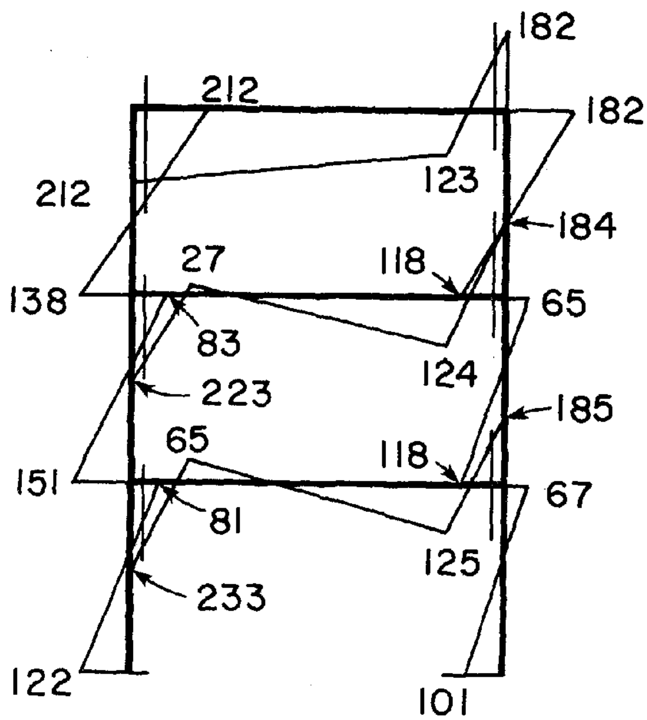


Fig. 11.5 Plastic Moment Distribution for Example Frame Obtained by Direct Plastic Design Procedure.

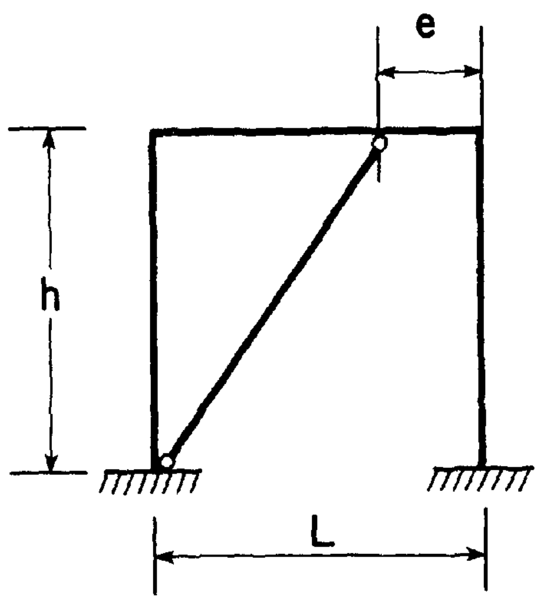


Fig. 11.6 Simple Eccentrically Braced Frame.

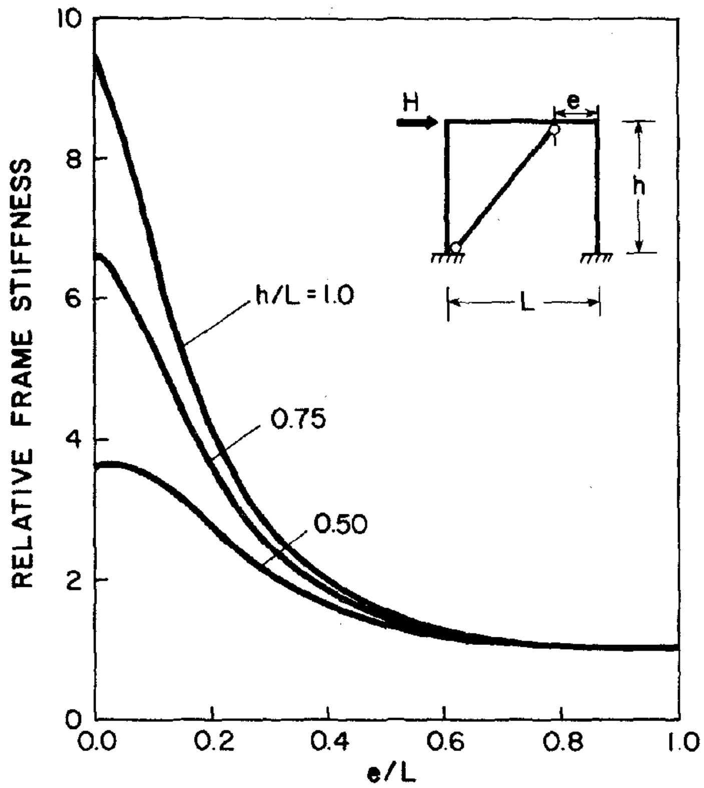


Fig. 11.7 Variation of Frame Stiffness for Different Aspect Ratios.

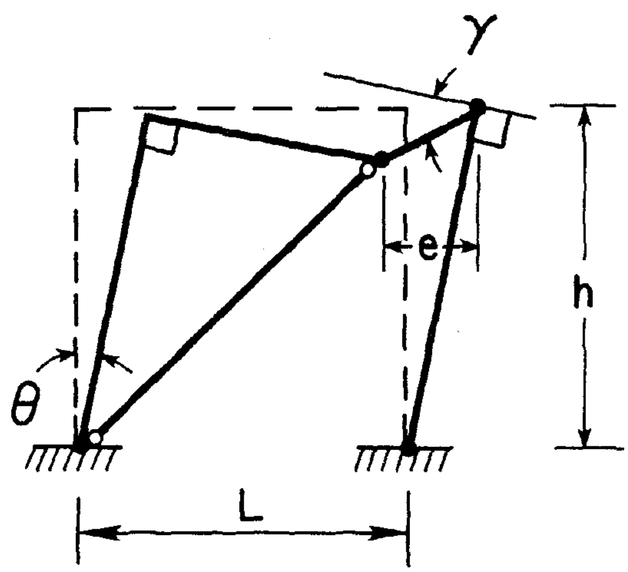


Fig. 11.8 Collapse Mechanism for the Simple Eccentrically Braced Frame of Fig. 11.6.

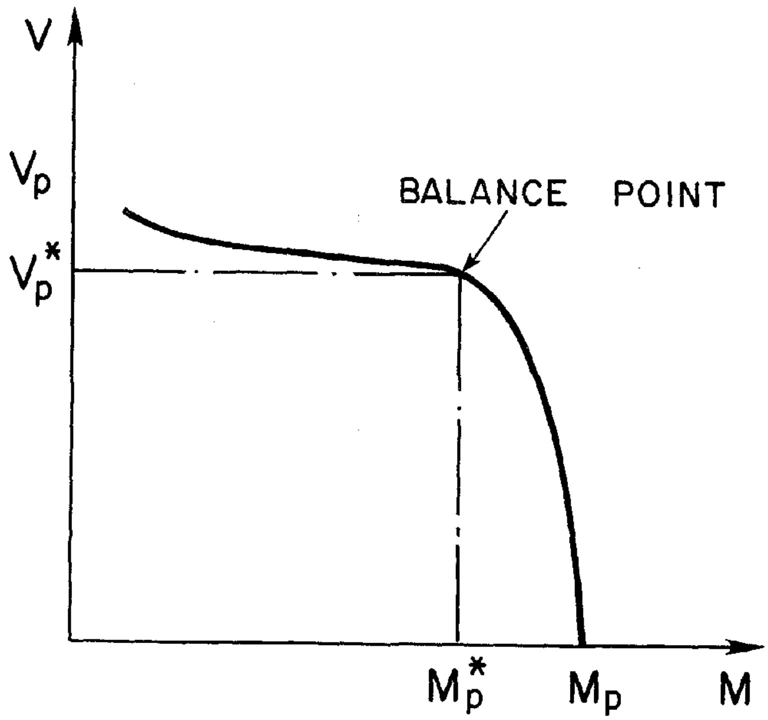


Fig. 11.9 Typical Shear-Moment Interaction Diagram for Wide Flange Sections.

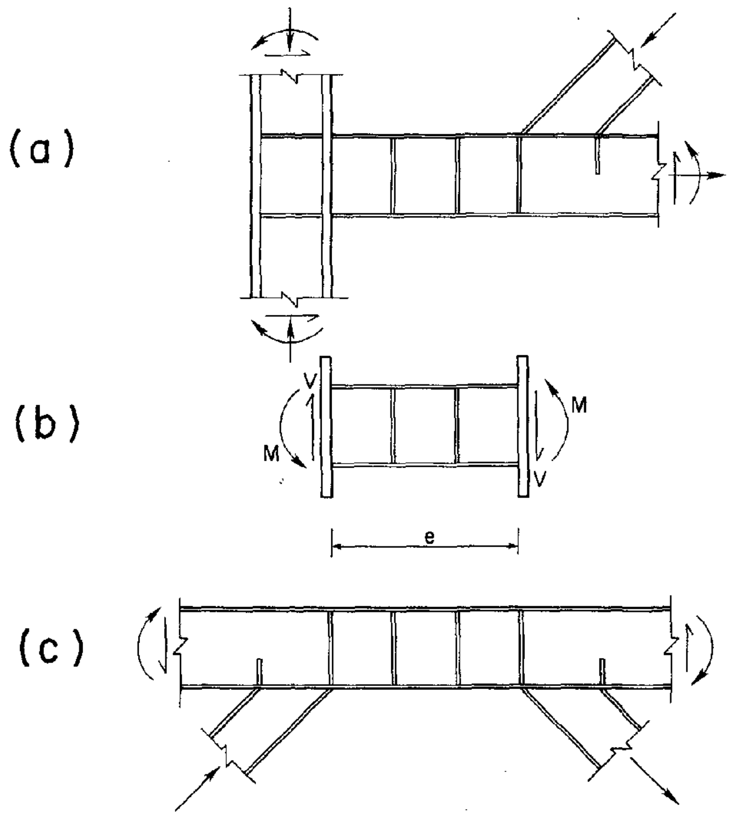


Fig. 11.10 Experimental Model (Shown in Middle) Extracted from Two Possible Prototype Configurations (Top and Bottom).

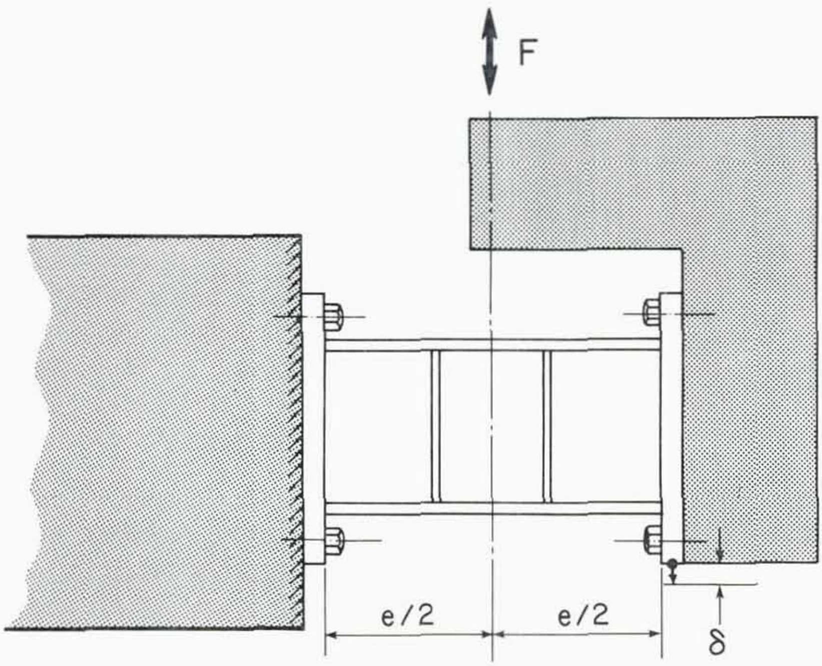
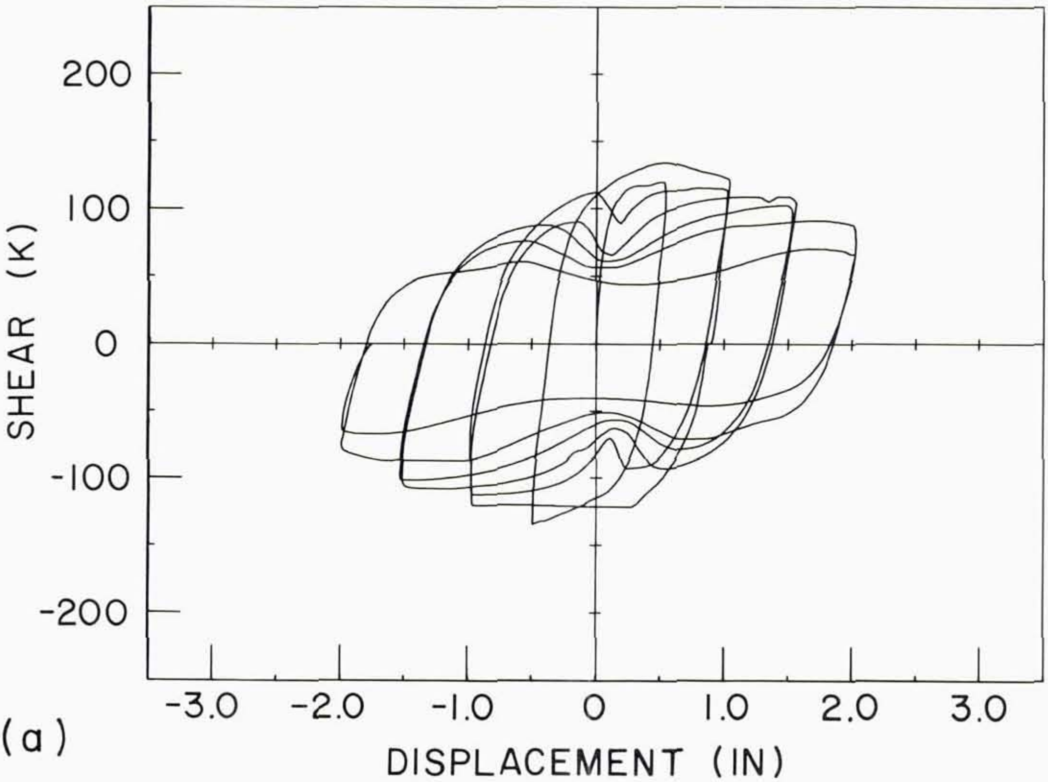
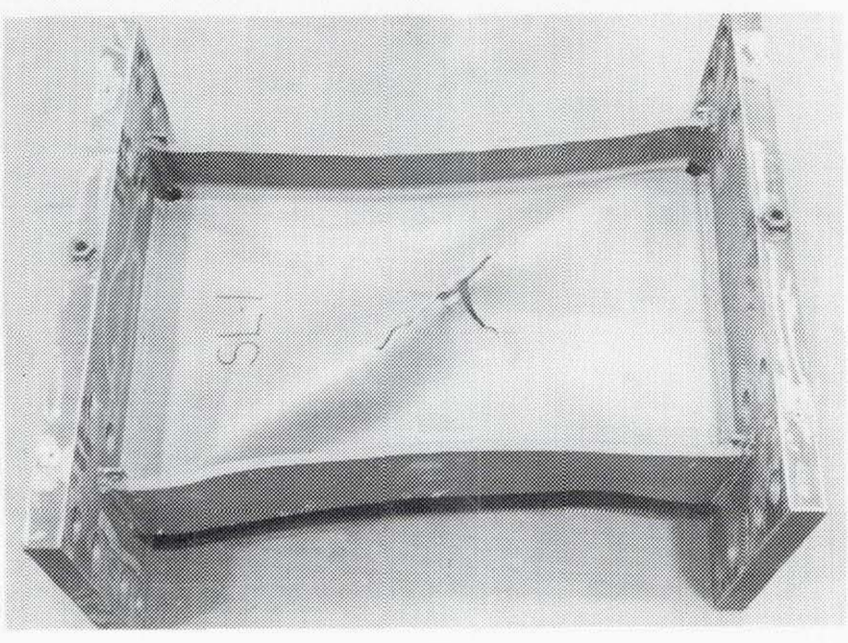


Fig. 11.11 Schematic Diagram of Test Set-up. Quasi-Static Force F is Applied Cyclically.

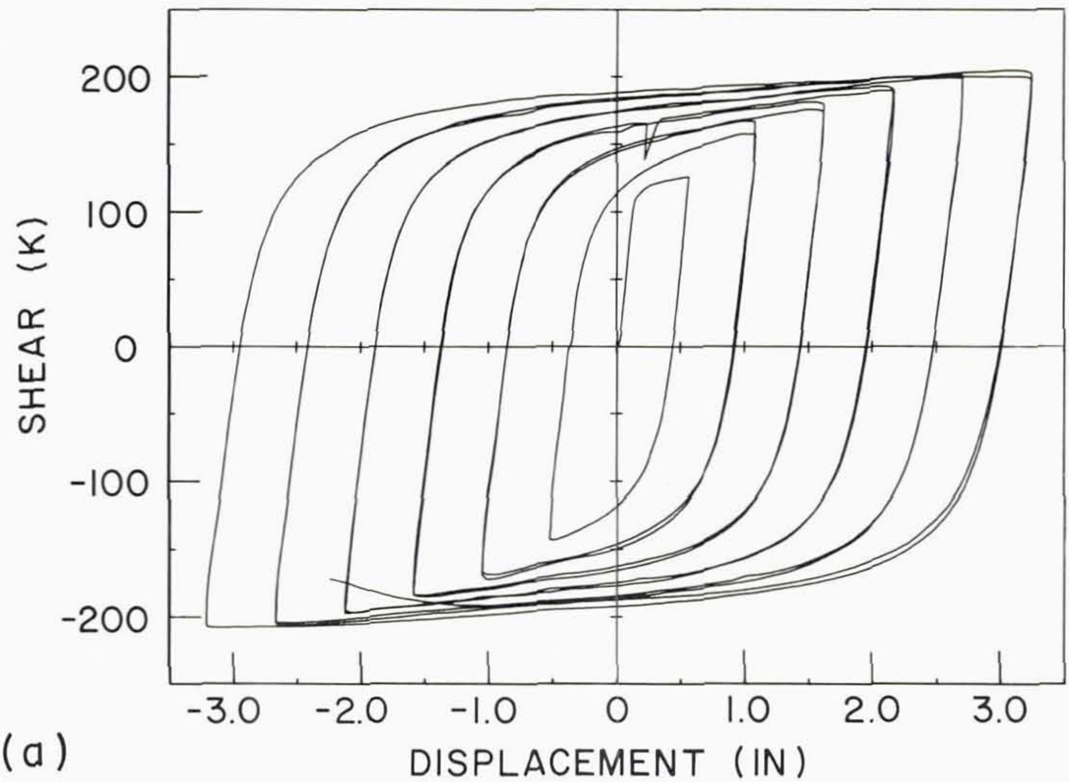


(a)

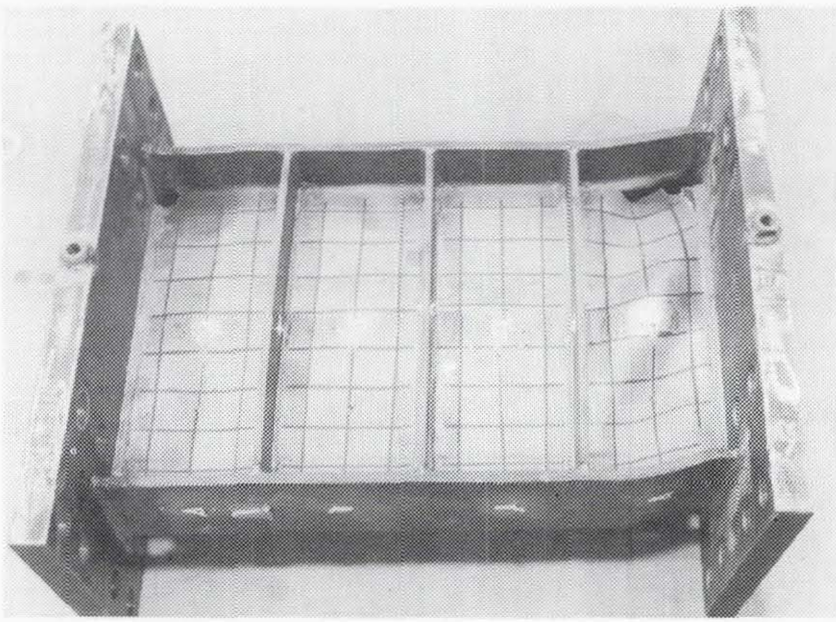


(b)

Fig. 11.12 Force-Displacement Hysteretic Loops, and Photo of Unstiffened Specimen #1 at End of Testing.

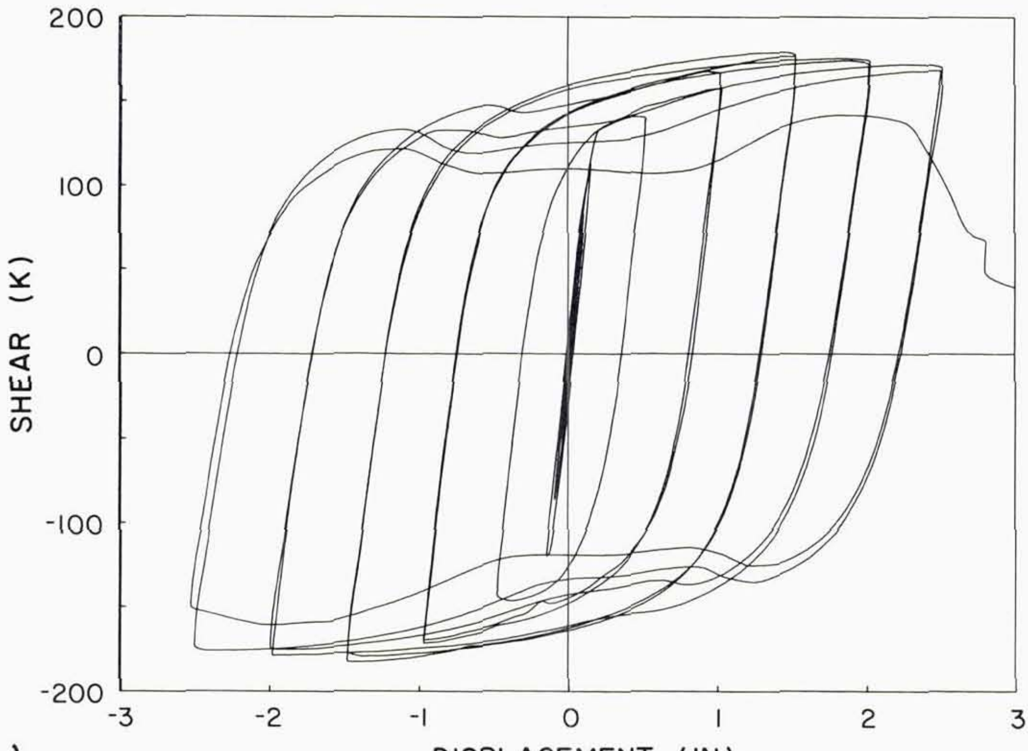


(a)

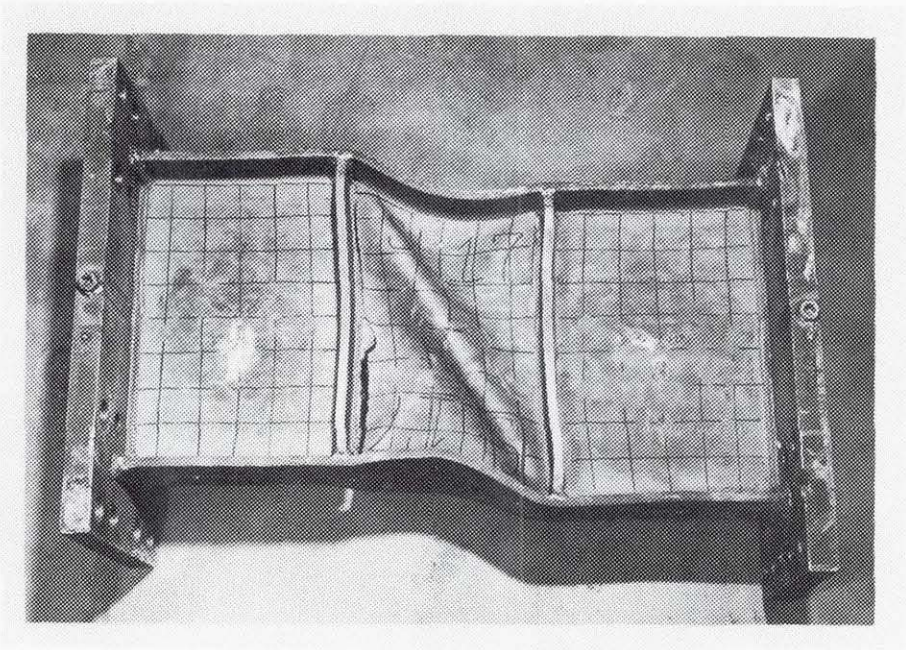


(b)

Fig. 11.13 Force-Displacement Hysteretic Loops, and Photo of Specimen #4 with Three Pairs of Equally Spaced 3/8 in. (10 mm) Stiffeners at the End of Testing.

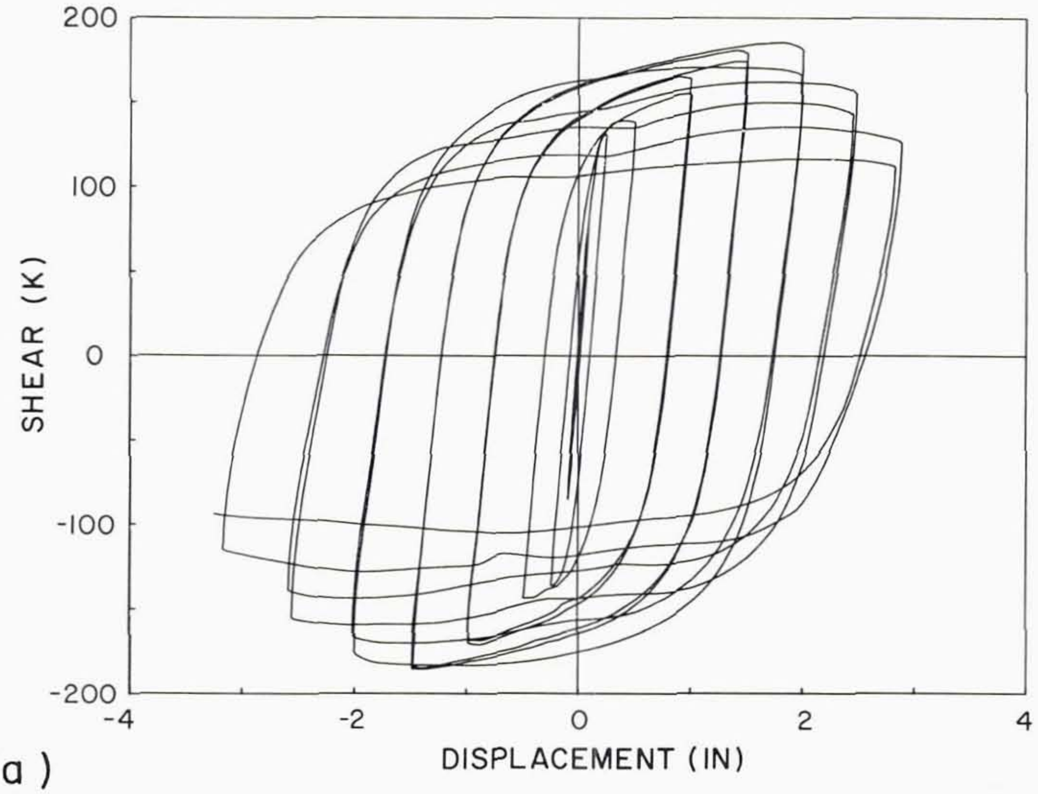


(a)



(b)

Fig. 11.14 Force-Displacement Hysteretic Loops and Photo of Specimen #17 with Two 1/2 in. (13 mm) One-Sided Stiffeners at End of Testing.



(a)

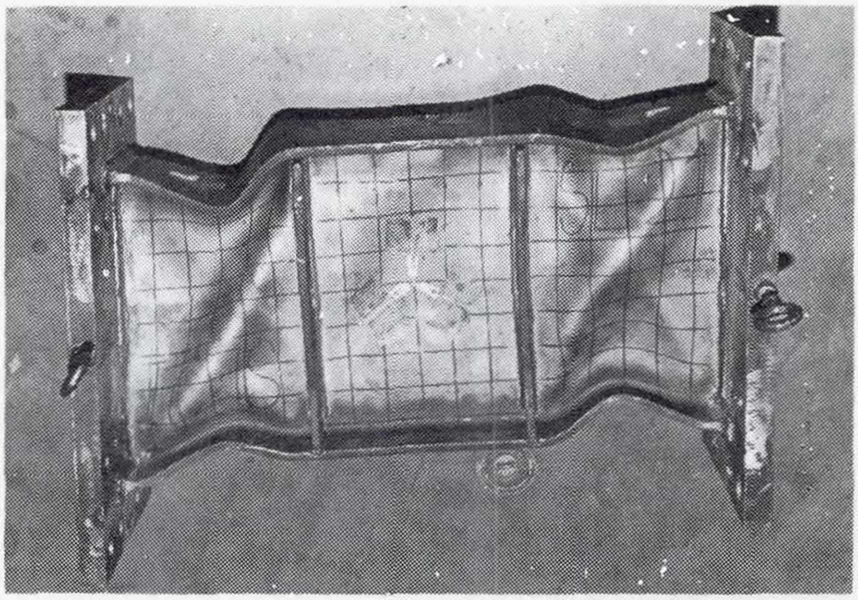
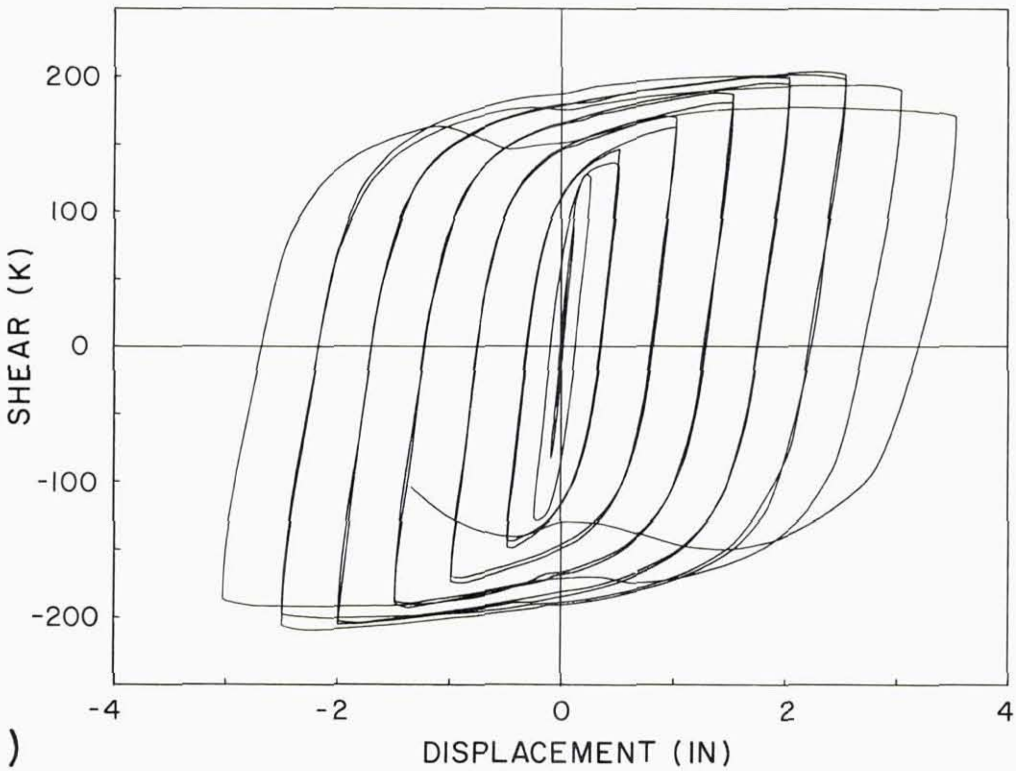
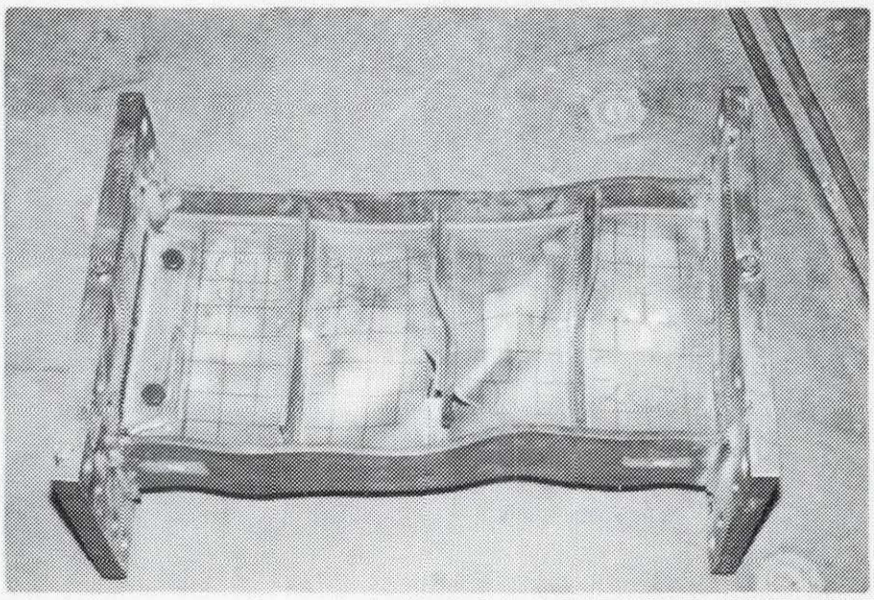

(b)

Fig. 11.15 Force-Displacement Hysteretic Loops and Photo of Specimen #21 with Two 1/2 in. (13 mm) One-Sided Stiffeners Attached to One of the Beam Flanges and the Web at End of Testing.

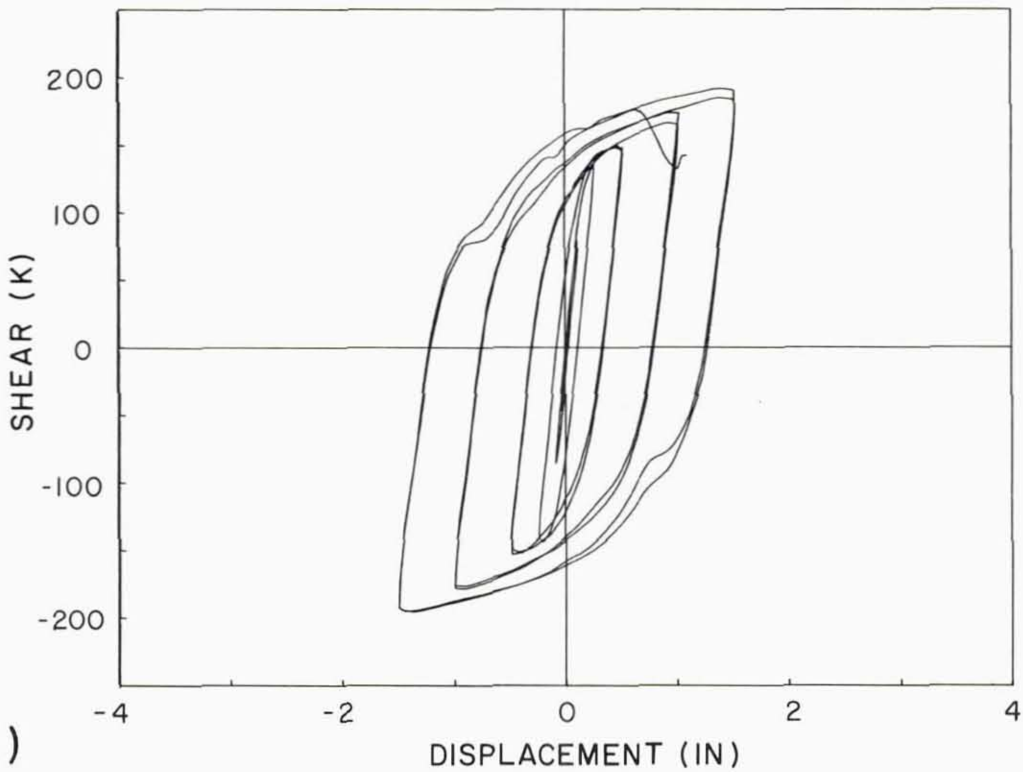


(a)

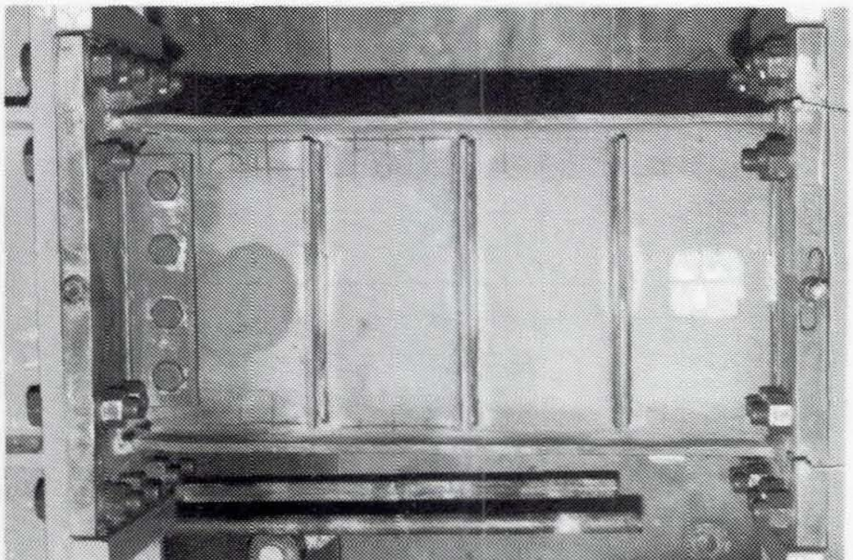


(b)

Fig. 11.16 Force-Displacement Hysteretic Loops and Photo of Specimen #26 with a Fully Welded Connection Detail and Three 3/8 in. (10 mm) One-Sided Stiffeners at End of Testing.

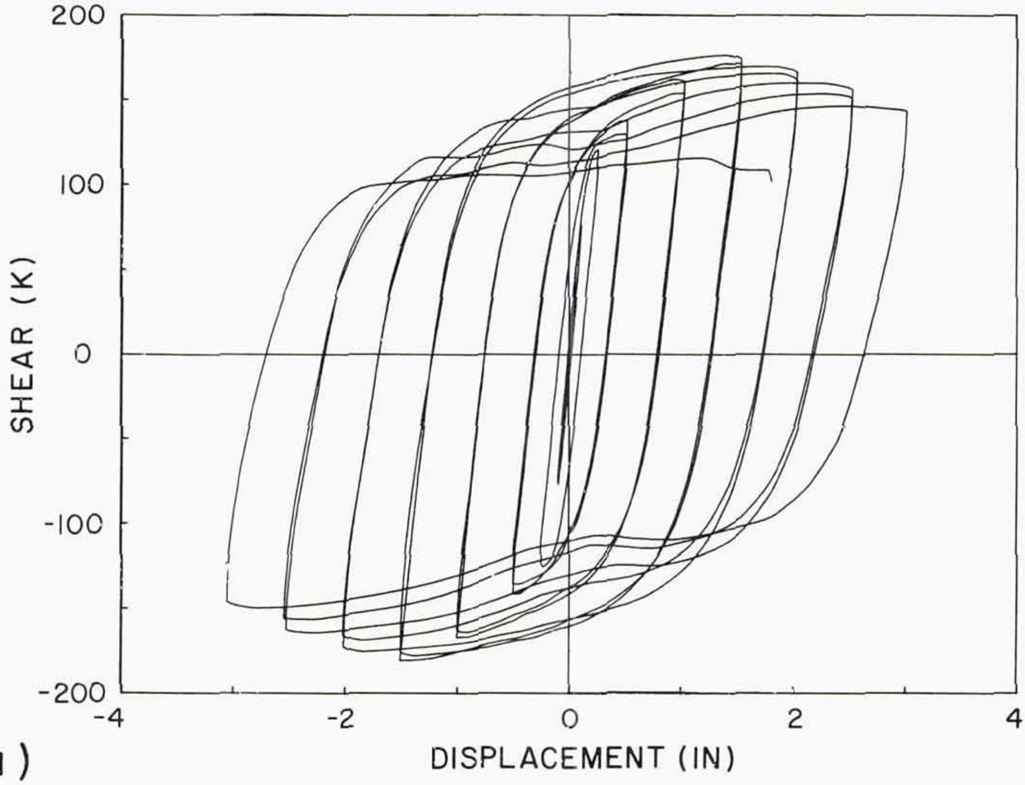


(a)

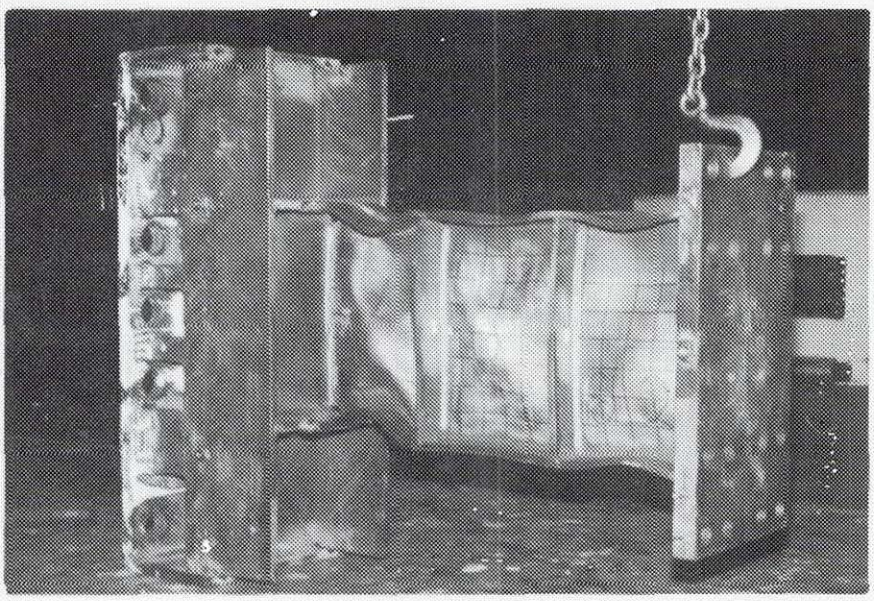


(b)

Fig. 11.17 Force-Displacement Hysteretic Loops and Photo of Specimen #28 with a Bolted Web-Welded Flange Connection Detail and Three 3/8 in. (10 mm) One-Sided Stiffeners.



(a)



(b)

Fig. 11.18 Force-Displacement Hysteretic Loops and Photo of Specimen #25 with Fully Welded Column-Web Connection and Two 1/2 in. (13 mm) Full Length One-Sided Stiffeners at End of Testing.

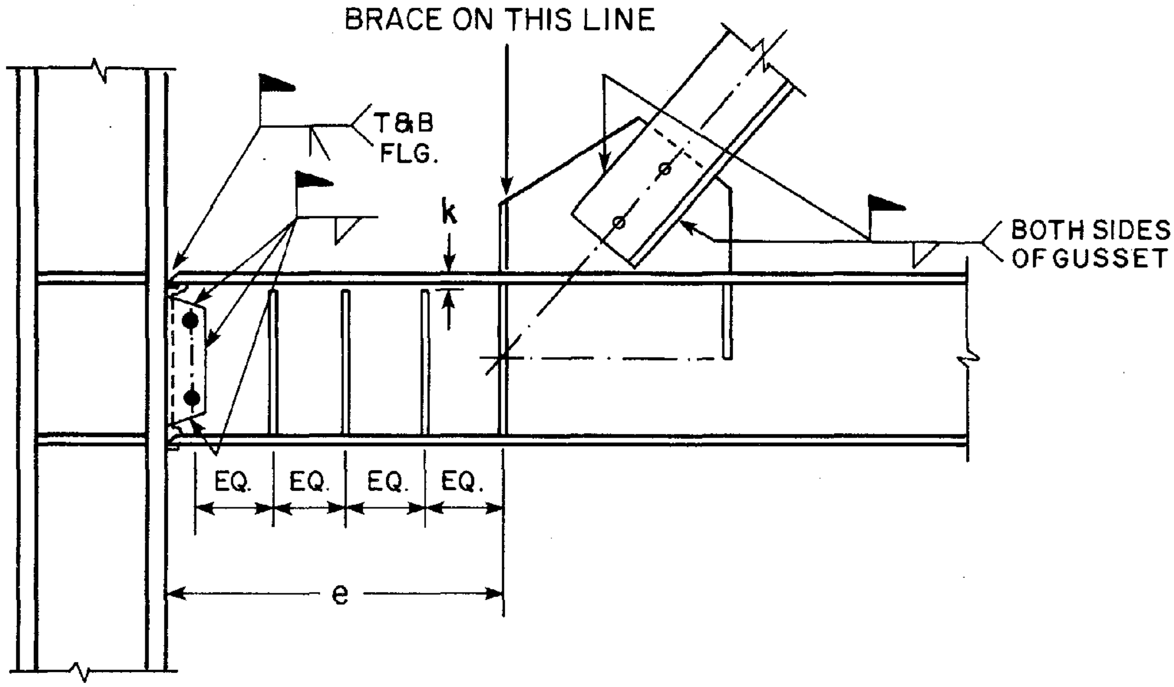


Fig. 11.19 Fully Welded Connection of Shear Link to Column Flange with Fillet Welds on Shear Tab Showing Stiffener Spacing.

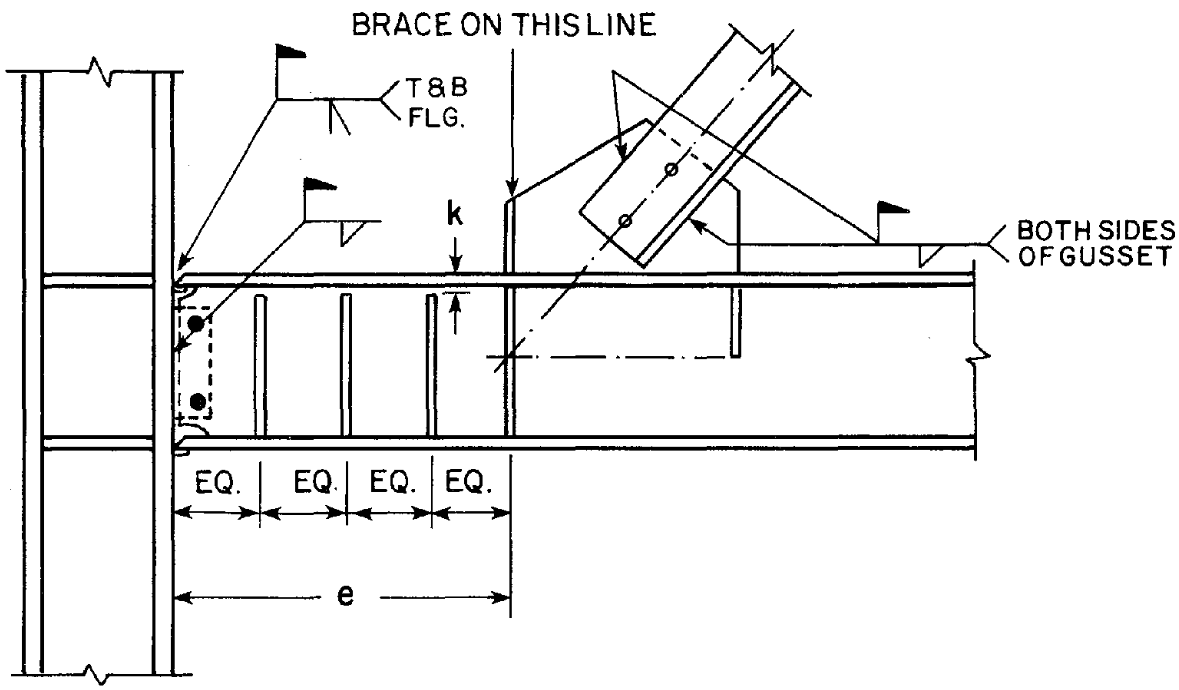


Fig. 11.20 Fully Welded Connection of Shear Link to Column Flange with Full Penetration Web Weld Showing Stiffener Spacing.

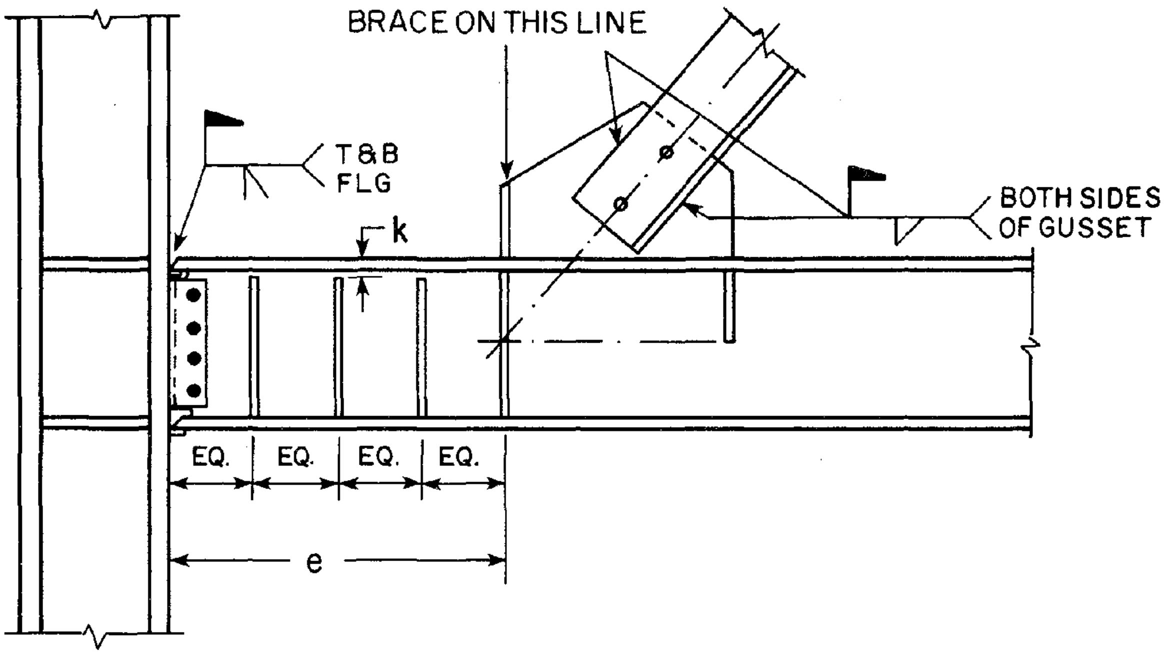


Fig. 11.21 Bolted Web, Welded Flange Connection of Shear Link to Column Flange Showing Stiffener Spacing.

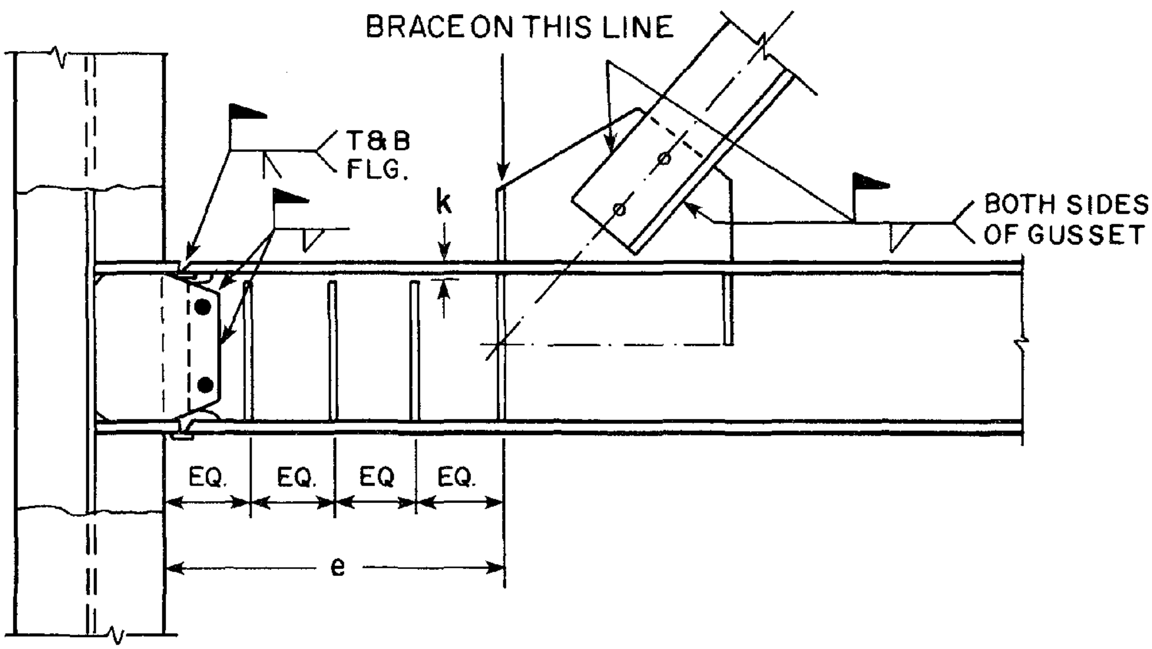


Fig. 11.22 Recommended Fully Welded Connection of Shear Link to Column Web.

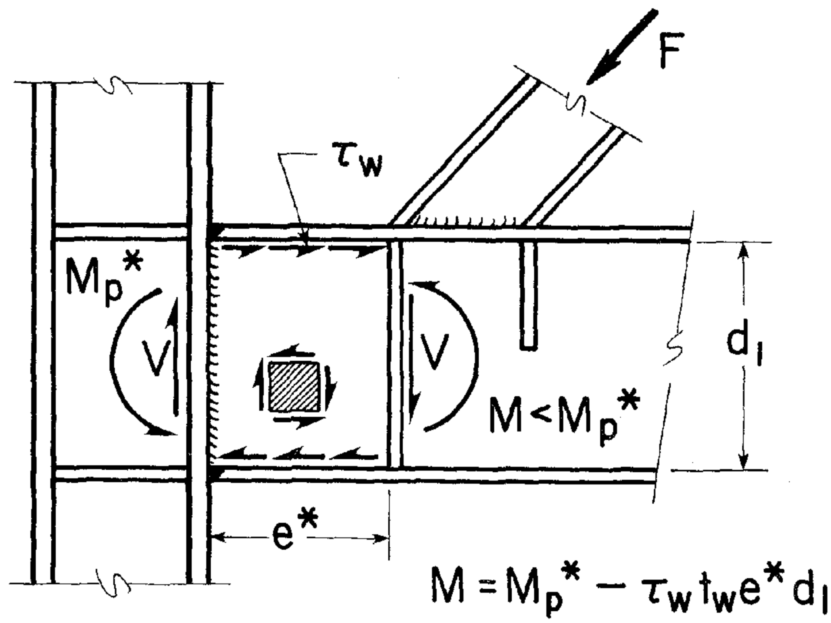


Fig. 11.23 Detail of a Short Shear Link Connection to Column Flange.

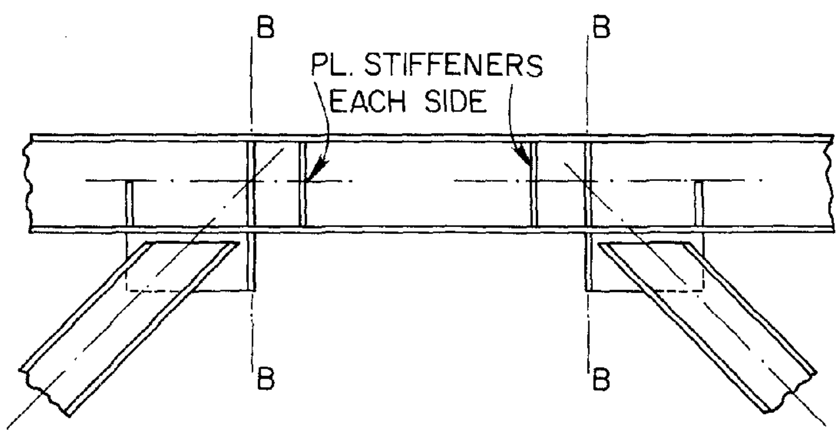


Fig. 11.24 Detail for Typical Interior Link of Moderate Length. Provide Lateral Braces on Lines B-B.

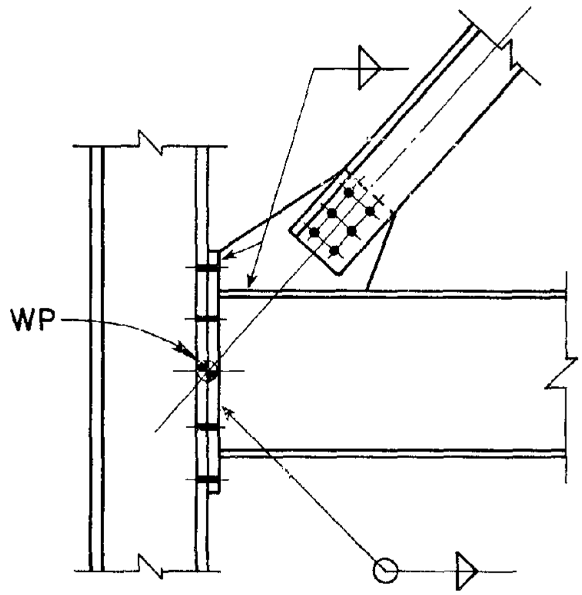


Fig. 11.25 Modified Concentric Detail.

EARTHQUAKE ENGINEERING RESEARCH CENTER REPORTS

NOTE: Numbers in parentheses are Accession Numbers assigned by the National Technical Information Service; these are followed by a price code. Copies of the reports may be ordered from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia, 22161. Accession Numbers should be quoted on orders for reports (PB --- ---) and remittance must accompany each order. Reports without this information were not available at time of printing. The complete list of EERC reports (from EERC 67-1) is available upon request from the Earthquake Engineering Research Center, University of California, Berkeley, 47th Street and Hoffman Boulevard, Richmond, California 94804.

UCB/EERC-77/01 "PLUSH - A Computer Program for Probabilistic Finite Element Analysis of Seismic Soil-Structure Interaction," by M.P. Romo Organista, J. Lysmer and H.B. Seed - 1977 (PB81 177 651)A05

UCB/EERC-77/02 "Soil-Structure Interaction Effects at the Humboldt Bay Power Plant in the Ferndale Earthquake of June 7, 1975," by J.E. Valera, H.B. Seed, C.F. Tsai and J. Lysmer - 1977 (PB 265 795)A04

UCB/EERC-77/03 "Influence of Sample Disturbance on Sand Response to Cyclic Loading," by K. Mori, H.B. Seed and C.K. Chan - 1977 (PB 267 352)A04

UCB/EERC-77/04 "Seismological Studies of Strong Motion Records," by J. Shoja-Taheri - 1977 (PB 269 655)A10

UCB/EERC-77/05 Unassigned

UCB/EERC-77/06 "Developing Methodologies for Evaluating the Earthquake Safety of Existing Buildings," by No. 1 - B. Bresler; No. 2 - B. Bresler, T. Okada and D. Zisling; No. 3 - T. Okada and B. Bresler; No. 4 - V.V. Bertero and B. Bresler - 1977 (PB 267 354)A08

UCB/EERC-77/07 "A Literature Survey - Transverse Strength of Masonry Walls," by Y. Omote, R.L. Mayes, S.W. Chen and R.W. Clough - 1977 (PB 277 933)A07

UCB/EERC-77/08 "DRAIN-TABS: A Computer Program for Inelastic Earthquake Response of Three Dimensional Buildings," by R. Guendelman-Israel and G.H. Powell - 1977 (PB 270 693)A07

UCB/EERC-77/09 "SUBWALL: A Special Purpose Finite Element Computer Program for Practical Elastic Analysis and Design of Structural Walls with Substructure Option," by D.Q. Le, H. Peterson and E.P. Popov - 1977 (PB 270 567)A05

UCB/EERC-77/10 "Experimental Evaluation of Seismic Design Methods for Broad Cylindrical Tanks," by D.P. Clough (PB 272 280)A13

UCB/EERC-77/11 "Earthquake Engineering Research at Berkeley - 1976," - 1977 (PB 273 507)A09

UCB/EERC-77/12 "Automated Design of Earthquake Resistant Multistory Steel Building Frames," by N.D. Walker, Jr. - 1977 (PB 276 526)A09

UCB/EERC-77/13 "Concrete Confined by Rectangular Hoops Subjected to Axial Loads," by J. Vallenas, V.V. Bertero and E.P. Popov - 1977 (PB 275 165)A06

UCB/EERC-77/14 "Seismic Strain Induced in the Ground During Earthquakes," by Y. Sugimura - 1977 (PB 284 201)A04

UCB/EERC-77/15 Unassigned

UCB/EERC-77/16 "Computer Aided Optimum Design of Ductile Reinforced Concrete Moment Resisting Frames," by S.W. Zagajeski and V.V. Bertero - 1977 (PB 280 137)A07

UCB/EERC-77/17 "Earthquake Simulation Testing of a Stepping Frame with Energy-Absorbing Devices," by J.M. Kelly and D.F. Tsztoo - 1977 (PB 273 506)A04

UCB/EERC-77/18 "Inelastic Behavior of Eccentrically Braced Steel Frames under Cyclic Loadings," by C.W. Roeder and E.P. Popov - 1977 (PB 275 526)A15

UCB/EERC-77/19 "A Simplified Procedure for Estimating Earthquake-Induced Deformations in Dams and Embankments," by F.I. Makdisi and H.B. Seed - 1977 (PB 276 820)A04

UCB/EERC-77/20 "The Performance of Earth Dams during Earthquakes," by H.B. Seed, F.I. Makdisi and P. de Alba - 1977 (PB 276 821)A04

UCB/EERC-77/21 "Dynamic Plastic Analysis Using Stress Resultant Finite Element Formulation," by P. Lukkunapvasit and J.M. Kelly - 1977 (PB 275 453)A04

UCB/EERC-77/22 "Preliminary Experimental Study of Seismic Uplift of a Steel Frame," by R.W. Clough and A.A. Huckelbridge 1977 (PB 278 769)A08

UCB/EERC-77/23 "Earthquake Simulator Tests of a Nine-Story Steel Frame with Columns Allowed to Uplift," by A.A. Huckelbridge - 1977 (PB 277 944)A09

UCB/EERC-77/24 "Nonlinear Soil-Structure Interaction of Skew Highway Bridges," by M.-C. Chen and J. Penzien - 1977 (PB 276 176)A07

UCB/EERC-77/25 "Seismic Analysis of an Offshore Structure Supported on Pile Foundations," by D.D.-N. Liou and J. Penzien 1977 (PB 283 180)A06

UCB/EERC-77/26 "Dynamic Stiffness Matrices for Homogeneous Viscoelastic Half-Planes," by G. Dasgupta and A.K. Chopra - 1977 (PB 279 654)A06

UCB/EERC-77/27 "A Practical Soft Story Earthquake Isolation System," by J.M. Kelly, J.M. Eidinger and C.J. Derham - 1977 (PB 276 814)A07

UCB/EERC-77/28 "Seismic Safety of Existing Buildings and Incentives for Hazard Mitigation in San Francisco: An Exploratory Study," by A.J. Meltsner - 1977 (PB 281 970)A05

UCB/EERC-77/29 "Dynamic Analysis of Electrohydraulic Shaking Tables," by D. Rea, S. Abedi-Hayati and Y. Takahashi 1977 (PB 282 569)A04

UCB/EERC-77/30 "An Approach for Improving Seismic - Resistant Behavior of Reinforced Concrete Interior Joints," by B. Galunic, V.V. Bertero and E.P. Popov - 1977 (PB 290 870)A06

UCB/EERC-78/01 "The Development of Energy-Absorbing Devices for Aseismic Base Isolation Systems," by J.M. Kelly and D.F. Tsztoo - 1978 (PB 284 978)A04

UCB/EERC-78/02 "Effect of Tensile Prestrain on the Cyclic Response of Structural Steel Connections, by J.G. Bouwkamp and A. Mukhopadhyay - 1978

UCB/EERC-78/03 "Experimental Results of an Earthquake Isolation System using Natural Rubber Bearings," by J.M. Eidinger and J.M. Kelly - 1978 (PB 281 686)A04

UCB/EERC-78/04 "Seismic Behavior of Tall Liquid Storage Tanks," by A. Niwa - 1978 (PB 284 017)A14

UCB/EERC-78/05 "Hysteretic Behavior of Reinforced Concrete Columns Subjected to High Axial and Cyclic Shear Forces," by S.W. Zagajeski, V.V. Bertero and J.G. Bouwkamp - 1978 (PB 283 858)A13

UCB/EERC-78/06 "Three Dimensional Inelastic Frame Elements for the ANSR-I Program," by A. Riahi, D.G. Row and G.H. Powell - 1978 (PB 295 755)A04

UCB/EERC-78/07 "Studies of Structural Response to Earthquake Ground Motion," by O.A. Lopez and A.K. Chopra - 1978 (PB 282 790)A05

UCB/EERC-78/08 "A Laboratory Study of the Fluid-Structure Interaction of Submerged Tanks and Caissons in Earthquakes," by R.C. Byrd - 1978 (PB 284 957)A08

UCB/EERC-78/09 Unassigned

UCB/EERC-78/10 "Seismic Performance of Nonstructural and Secondary Structural Elements," by I. Sakamoto - 1978 (PB81 154 593)A05

UCB/EERC-78/11 "Mathematical Modelling of Hysteresis Loops for Reinforced Concrete Columns," by S. Nakata, T. Sproul and J. Penzien - 1978 (PB 298 274)A05

UCB/EERC-78/12 "Damageability in Existing Buildings," by T. Blejwas and B. Bresler - 1978 (PB 80 166 978)A05

UCB/EERC-78/13 "Dynamic Behavior of a Pedestal Base Multistory Building," by R.M. Stephen, E.L. Wilson, J.G. Bouwkamp and M. Button - 1978 (PB 286 650)A08

UCB/EERC-78/14 "Seismic Response of Bridges - Case Studies," by R.A. Imbsen, V. Nutt and J. Penzien - 1978 (PB 286 503)A10

UCB/EERC-78/15 "A Substructure Technique for Nonlinear Static and Dynamic Analysis," by D.G. Row and G.H. Powell - 1978 (PB 288 077)A10

UCB/EERC-78/16 "Seismic Risk Studies for San Francisco and for the Greater San Francisco Bay Area," by C.S. Oliveira - 1978 (PB 81 120 115)A07

UCB/EERC-78/17 "Strength of Timber Roof Connections Subjected to Cyclic Loads," by P. Gülkan, R.L. Mayes and R.W. Clough - 1978 (HUD-000 1491)A07

UCB/EERC-78/18 "Response of K-Braced Steel Frame Models to Lateral Loads," by J.G. Bouwkamp, R.M. Stephen and E.P. Popov - 1978

UCB/EERC-78/19 "Rational Design Methods for Light Equipment in Structures Subjected to Ground Motion," by J.L. Sackman and J.M. Kelly - 1978 (PB 292 357)A04

UCB/EERC-78/20 "Testing of a Wind Restraint for Aseismic Base Isolation," by J.M. Kelly and D.E. Chitty - 1978 (PB 292 833)A03

UCB/EERC-78/21 "APOLLO - A Computer Program for the Analysis of Pore Pressure Generation and Dissipation in Horizontal Sand Layers During Cyclic or Earthquake Loading," by P.P. Martin and H.B. Seed - 1978 (PB 292 835)A04

UCB/EERC-78/22 "Optimal Design of an Earthquake Isolation System," by M.A. Bhatti, K.S. Pister and E. Polak - 1978 (PB 294 735)A06

UCB/EERC-78/23 "MASH - A Computer Program for the Non-Linear Analysis of Vertically Propagating Shear Waves in Horizontally Layered Deposits," by P.P. Martin and H.B. Seed - 1978 (PB 293 101)A05

UCB/EERC-78/24 "Investigation of the Elastic Characteristics of a Three Story Steel Frame Using System Identification " by I. Kaya and H.D. McNiven - 1978 (PB 296 225)A06

UCB/EERC-78/25 "Investigation of the Nonlinear Characteristics of a Three-Story Steel Frame Using System Identification," by I. Kaya and H.D. McNiven - 1978 (PB 301 363)A05

UCB/EERC-78/26 "Studies of Strong Ground Motion in Taiwan," by Y.M. Hsiung, B.A. Bolt and J. Penzien - 1978 (PB 298 436)A06

UCB/EERC-78/27 "Cyclic Loading Tests of Masonry Single Piers: Volume 1 - Height to Width Ratio of 2," by P.A. Hidalgo, R.L. Mayes, H.D. McNiven and R.W. Clough - 1978 (PB 296 211)A07

UCB/EERC-78/28 "Cyclic Loading Tests of Masonry Single Piers: Volume 2 - Height to Width Ratio of 1," by S.-W.J. Chen, P.A. Hidalgo, R.L. Mayes, R.W. Clough and H.D. McNiven - 1978 (PB 296 212)A09

UCB/EERC-78/29 "Analytical Procedures in Soil Dynamics," by J. Lysmer - 1978 (PB 298 445)A06

UCB/EERC-79/01 "Hysteretic Behavior of Lightweight Reinforced Concrete Beam-Column Subassemblages," by B. Forzani, E.P. Popov and V.V. Bertero - April 1979(PB 298 267)A06

UCB/EERC-79/02 "The Development of a Mathematical Model to Predict the Flexural Response of Reinforced Concrete Beams to Cyclic Loads, Using System Identification," by J. Stanton & H. McNiven - Jan. 1979(PB 295 875)A10

UCB/EERC-79/03 "Linear and Nonlinear Earthquake Response of Simple Torsionally Coupled Systems," by C.L. Kan and A.K. Chopra - Feb. 1979(PB 298 262)A06

UCB/EERC-79/04 "A Mathematical Model of Masonry for Predicting its Linear Seismic Response Characteristics," by Y. Mengi and H.D. McNiven - Feb. 1979(PB 298 266)A06

UCB/EERC-79/05 "Mechanical Behavior of Lightweight Concrete Confined by Different Types of Lateral Reinforcement," by M.A. Manrique, V.V. Bertero and E.P. Popov - May 1979(PB 301 114)A06

UCB/EERC-79/06 "Static Tilt Tests of a Tall Cylindrical Liquid Storage Tank," by R.W. Clough and A. Niwa - Feb. 1979 (PB 301 167)A06

UCB/EERC-79/07 "The Design of Steel Energy Absorbing Restrainers and Their Incorporation into Nuclear Power Plants for Enhanced Safety: Volume 1 - Summary Report," by P.N. Spencer, V.F. Zackay, and E.R. Parker - Feb. 1979(UCB/EERC-79/07)A09

UCB/EERC-79/08 "The Design of Steel Energy Absorbing Restrainers and Their Incorporation into Nuclear Power Plants for Enhanced Safety: Volume 2 - The Development of Analyses for Reactor System Piping,""Simple Systems" by M.C. Lee, J. Penzien, A.K. Chopra and K, Suzuki "Complex Systems" by G.H. Powell, E.L. Wilson, R.W. Clough and D.G. Row - Feb. 1979(UCB/EERC-79/08)A10

UCB/EERC-79/09 "The Design of Steel Energy Absorbing Restrainers and Their Incorporation into Nuclear Power Plants for Enhanced Safety: Volume 3 - Evaluation of Commercial Steels," by W.S. Owen, R.M.N. Pelloux, R.O. Ritchie, M. Faral, T. Ohhashi, J. Toplosky, S.J. Hartman, V.F. Zackay and E.R. Parker - Feb. 1979(UCB/EERC-79/09)A04

UCB/EERC-79/10 "The Design of Steel Energy Absorbing Restrainers and Their Incorporation into Nuclear Power Plants for Enhanced Safety: Volume 4 - A Review of Energy-Absorbing Devices," by J.M. Kelly and M.S. Skinner - Feb. 1979(UCB/EERC-79/10)A04

UCB/EERC-79/11 "Conservatism In Summation Rules for Closely Spaced Modes," by J.M. Kelly and J.L. Sackman - May 1979(PB 301 328)A03

UCB/EERC-79/12 "Cyclic Loading Tests of Masonry Single Piers; Volume 3 - Height to Width Ratio of 0.5," by P.A. Hidalgo, R.L. Mayes, H.D. McNiven and R.W. Clough - May 1979(PB 301 321)A08

UCB/EERC-79/13 "Cyclic Behavior of Dense Course-Grained Materials in Relation to the Seismic Stability of Dams," by N.G. Banerjee, H.B. Seed and C.K. Chan - June 1979(PB 301 373)A13

UCB/EERC-79/14 "Seismic Behavior of Reinforced Concrete Interior Beam-Column Subassemblages," by S. Viwathanatepa, E.P. Popov and V.V. Bertero - June 1979(PB 301 326)A10

UCB/EERC-79/15 "Optimal Design of Localized Nonlinear Systems with Dual Performance Criteria Under Earthquake Excitations," by M.A. Bhatti - July 1979(PB 80 167 109)A06

UCB/EERC-79/16 "OPTDYN - A General Purpose Optimization Program for Problems with or without Dynamic Constraints," by M.A. Bhatti, E. Polak and K.S. Pister - July 1979(PB 80 167 091)A05

UCB/EERC-79/17 "ANSR-II, Analysis of Nonlinear Structural Response, Users Manual," by D.P. Mondkar and G.H. Powell July 1979(PB 80 113 301)A05

UCB/EERC-79/18 "Soil Structure Interaction in Different Seismic Environments," A. Gomez-Masso, J. Lysmer, J.-C. Chen and H.B. Seed - August 1979(PB 80 101 520)A04

UCB/EERC-79/19 "ARMA Models for Earthquake Ground Motions," by M.K. Chang, J.W. Kwiatkowski, R.F. Nau, R.M. Oliver and K.S. Pister - July 1979(PB 301 166)A05

UCB/EERC-79/20 "Hysteretic Behavior of Reinforced Concrete Structural Walls," by J.M. Vallenas, V.V. Bertero and E.P. Popov - August 1979(PB 80 165 905)A12

UCB/EERC-79/21 "Studies on High-Frequency Vibrations of Buildings - 1: The Column Effect," by J. Lubliner - August 1979 (PB 80 158 553)A03

UCB/EERC-79/22 "Effects of Generalized Loadings on Bond Reinforcing Bars Embedded in Confined Concrete Blocks," by S. Viwathanatepa, E.P. Popov and V.V. Bertero - August 1979(PB 81 124 018)A14

UCB/EERC-79/23 "Shaking Table Study of Single-Story Masonry Houses, Volume 1: Test Structures 1 and 2," by P. Gülkan, R.L. Mayes and R.W. Clough - Sept. 1979 (HUD-000 1763)A12

UCB/EERC-79/24 "Shaking Table Study of Single-Story Masonry Houses, Volume 2: Test Structures 3 and 4," by P. Gülkan, R.L. Mayes and R.W. Clough - Sept. 1979 (HUD-000 1836)A12

UCB/EERC-79/25 "Shaking Table Study of Single-Story Masonry Houses, Volume 3: Summary, Conclusions and Recommendations," by R.W. Clough, R.L. Mayes and P. Gülkan - Sept. 1979 (HUD-000 1837)A06

UCB/EERC-79/26 "Recommendations for a U.S.-Japan Cooperative Research Program Utilizing Large-Scale Testing Facilities," by U.S.-Japan Planning Group - Sept. 1979(PB 301 407)A06

UCB/EERC-79/27 "Earthquake-Induced Liquefaction Near Lake Amatitlan, Guatemala," by H.B. Seed, I. Arango, C.K. Chan, A. Gomez-Masso and R. Grant de Ascoli - Sept. 1979(NUREG-CR1341)A03

UCB/EERC-79/28 "Infill Panels: Their Influence on Seismic Response of Buildings," by J.W. Axley and V.V. Bertero Sept. 1979(PB 80 163 371)A10

UCB/EERC-79/29 "3D Truss Bar Element (Type 1) for the ANSR-II Program," by D.P. Mondkar and G.H. Powell - Nov. 1979 (PB 80 169 709)A02

UCB/EERC-79/30 "2D Beam-Column Element (Type 5 - Parallel Element Theory) for the ANSR-II Program," by D.G. Row, G.H. Powell and D.P. Mondkar - Dec. 1979(PB 80 167 224)A03

UCB/EERC-79/31 "3D Beam-Column Element (Type 2 - Parallel Element Theory) for the ANSR-II Program," by A. Riahi, G.H. Powell and D.P. Mondkar - Dec. 1979(PB 80 167 216)A03

UCB/EERC-79/32 "On Response of Structures to Stationary Excitation," by A. Der Kiureghian - Dec. 1979(PB 80166 929)A03

UCB/EERC-79/33 "Undisturbed Sampling and Cyclic Load Testing of Sands," by S. Singh, H.B. Seed and C.K. Chan Dec. 1979(ADA 087 298)A07

UCB/EERC-79/34 "Interaction Effects of Simultaneous Torsional and Compressional Cyclic Loading of Sand," by P.M. Griffin and W.N. Houston - Dec. 1979(ADA 092 352)A15

UCB/EERC-80/01 "Earthquake Response of Concrete Gravity Dams Including Hydrodynamic and Foundation Interaction Effects," by A.K. Chopra, P. Chakrabarti and S. Gupta - Jan. 1980(AD-A087297)A10

UCB/EERC-80/02 "Rocking Response of Rigid Blocks to Earthquakes," by C.S. Yim, A.K. Chopra and J. Penzien - Jan. 1980 (PB80 166 002)A04

UCB/EERC-80/03 "Optimum Inelastic Design of Seismic-Resistant Reinforced Concrete Frame Structures," by S.W. Zagajeski and V.V. Bertero - Jan. 1980(PB80 164 635)A06

UCB/EERC-80/04 "Effects of Amount and Arrangement of Wall-Panel Reinforcement on Hysteretic Behavior of Reinforced Concrete Walls," by R. Iliya and V.V. Bertero - Feb. 1980(PB81 122 525)A09

UCB/EERC-80/05 "Shaking Table Research on Concrete Dam Models," by A. Niwa and R.W. Clough - Sept. 1980(PB81 122 368)A06

UCB/EERC-80/06 "The Design of Steel Energy-Absorbing Restrainers and their Incorporation into Nuclear Power Plants for Enhanced Safety (Vol 1A): Piping with Energy Absorbing Restrainers: Parameter Study on Small Systems," by G.H. Powell, C. Oughourlian and J. Simons - June 1980

UCB/EERC-80/07 "Inelastic Torsional Response of Structures Subjected to Earthquake Ground Motions," by Y. Yamazaki April 1980(PB81 122 327)A08

UCB/EERC-80/08 "Study of X-Braced Steel Frame Structures Under Earthquake Simulation," by Y. Ghanaat - April 1980 (PB81 122 335)A11

UCB/EERC-80/09 "Hybrid Modelling of Soil-Structure Interaction," by S. Gupta, T.W. Lin, J. Penzien and C.S. Yeh May 1980(PB81 122 319)A07

UCB/EERC-80/10 "General Applicability of a Nonlinear Model of a One Story Steel Frame," by B.I. Sveinsson and H.D. McNiven - May 1980(PB81 124 877)A06

UCB/EERC-80/11 "A Green-Function Method for Wave Interaction with a Submerged Body," by W. Kioka - April 1980 (PB81 122 269)A07

UCB/EERC-80/12 "Hydrodynamic Pressure and Added Mass for Axisymmetric Bodies," by F. Nilrat - May 1980(PB81 122 343)A08

UCB/EERC-80/13 "Treatment of Non-Linear Drag Forces Acting on Offshore Platforms," by B.V. Dao and J. Penzien May 1980(PB81 153 413)A07

UCB/EERC-80/14 "2D Plane/Axisymmetric Solid Element (Type 3 - Elastic or Elastic-Perfectly Plastic) for the ANSR-II Program," by D.P. Mondkar and G.H. Powell - July 1980(PB81 122 350)A03

UCB/EERC-80/15 "A Response Spectrum Method for Random Vibrations," by A. Der Kiureghian - June 1980(PB81 122 301)A03

UCB/EERC-80/16 "Cyclic Inelastic Buckling of Tubular Steel Braces," by V.A. Zayas, E.P. Popov and S.A. Mahin June 1980(PB81 124 885)A10

UCB/EERC-80/17 "Dynamic Response of Simple Arch Dams Including Hydrodynamic Interaction," by C.S. Porter and A.K. Chopra - July 1980(PB81 124 000)A13

UCB/EERC-80/18 "Experimental Testing of a Friction Damped Aseismic Base Isolation System with Fail-Safe Characteristics," by J.M. Kelly, K.E. Beucke and M.S. Skinner - July 1980(PB81 148 595)A04

UCB/EERC-80/19 "The Design of Steel Energy-Absorbing Restrainers and their Incorporation into Nuclear Power Plants for Enhanced Safety (Vol 1B): Stochastic Seismic Analyses of Nuclear Power Plant Structures and Piping Systems Subjected to Multiple Support Excitations," by M.C. Lee and J. Penzien - June 1980

UCB/EERC-80/20 "The Design of Steel Energy-Absorbing Restrainers and their Incorporation into Nuclear Power Plants for Enhanced Safety (Vol 1C): Numerical Method for Dynamic Substructure Analysis," by J.M. Dickens and E.L. Wilson - June 1980

UCB/EERC-80/21 "The Design of Steel Energy-Absorbing Restrainers and their Incorporation into Nuclear Power Plants for Enhanced Safety (Vol 2): Development and Testing of Restraints for Nuclear Piping Systems," by J.M. Kelly and M.S. Skinner - June 1980

UCB/EERC-80/22 "3D Solid Element (Type 4-Elastic or Elastic-Perfectly-Plastic) for the ANSR-II Program," by D.P. Mondkar and G.H. Powell - July 1980(PB81 123 242)A03

UCB/EERC-80/23 "Gap-Friction Element (Type 5) for the ANSR-II Program," by D.P. Mondkar and G.H. Powell - July 1980 (PB81 122 285)A03

UCB/EERC-80/24 "U-Bar Restraint Element (Type 11) for the ANSR-II Program," by C. Oughourlian and G.H. Powell July 1980(PB81 122 293)A03

UCB/EERC-80/25 "Testing of a Natural Rubber Base Isolation System by an Explosively Simulated Earthquake," by J.M. Kelly - August 1980(PB81 201 360)A04

UCB/EERC-80/26 "Input Identification from Structural Vibrational Response," by Y. Hu - August 1980(PB81 152 308)A05

UCB/EERC-80/27 "Cyclic Inelastic Behavior of Steel Offshore Structures," by V.A. Zayas, S.A. Mahin and E.P. Popov August 1980(PB81 196 180)A15

UCB/EERC-80/28 "Shaking Table Testing of a Reinforced Concrete Frame with Biaxial Response," by M.G. Oliva October 1980(PB81 154 304)A10

UCB/EERC-80/29 "Dynamic Properties of a Twelve-Story Prefabricated Panel Building," by J.G. Bouwkamp, J.P. Kollegger and R.M. Stephen - October 1980(PB82 117 128)A06

UCB/EERC-80/30 "Dynamic Properties of an Eight-Story Prefabricated Panel Building," by J.G. Bouwkamp, J.P. Kollegger and R.M. Stephen - October 1980(PB81 200 313)A05

UCB/EERC-80/31 "Predictive Dynamic Response of Panel Type Structures Under Earthquakes," by J.P. Kollegger and J.G. Bouwkamp - October 1980(PB81 152 316)A04

UCB/EERC-80/32 "The Design of Steel Energy-Absorbing Restrainers and their Incorporation into Nuclear Power Plants for Enhanced Safety (Vol 3): Testing of Commercial Steels in Low-Cycle Torsional Fatigue," by P. Spencer, E.R. Parker, E. Jongewaard and M. Drory

UCB/EERC-80/33 "The Design of Steel Energy-Absorbing Restrainers and their Incorporation into Nuclear Power Plants for Enhanced Safety (Vol 4): Shaking Table Tests of Piping Systems with Energy-Absorbing Restrainers," by S.F. Stiemer and W.G. Godden - Sept. 1980

UCB/EERC-80/34 "The Design of Steel Energy-Absorbing Restrainers and their Incorporation into Nuclear Power Plants for Enhanced Safety (Vol 5): Summary Report," by P. Spencer

UCB/EERC-80/35 "Experimental Testing of an Energy-Absorbing Base Isolation System," by J.M. Kelly, M.S. Skinner and K.E. Beucke - October 1980(PB81 154 072)A04

UCB/EERC-80/36 "Simulating and Analyzing Artificial Non-Stationary Earthquake Ground Motions," by R.F. Nau, R.M. Oliver and K.S. Pister - October 1980(PB81 153 397)A04

UCB/EERC-80/37 "Earthquake Engineering at Berkeley - 1980," - Sept. 1980(PB81 205 874)A09

UCB/EERC-80/38 "Inelastic Seismic Analysis of Large Panel Buildings," by V. Schricker and G.H. Powell - Sept. 1980 (PB81 154 338)A13

UCB/EERC-80/39 "Dynamic Response of Embankment, Concrete-Gravity and Arch Dams Including Hydrodynamic Interaction," by J.F. Hall and A.K. Chopra - October 1980(PB81 152 324)A11

UCB/EERC-80/40 "Inelastic Buckling of Steel Struts Under Cyclic Load Reversal," by R.G. Black, W.A. Wenger and E.P. Popov - October 1980(PB81 154 312)A08

UCB/EERC-80/41 "Influence of Site Characteristics on Building Damage During the October 3, 1974 Lima Earthquake," by P. Repetto, I. Arango and H.B. Seed - Sept. 1980(PB81 161 739)A05

UCB/EERC-80/42 "Evaluation of a Shaking Table Test Program on Response Behavior of a Two Story Reinforced Concrete Frame," by J.M. Blondet, R.W. Clough and S.A. Mahin

UCB/EERC-80/43 "Modelling of Soil-Structure Interaction by Finite and Infinite Elements," by F. Medina - December 1980(PB81 229 270)A04

UCB/EERC-81/01 "Control of Seismic Response of Piping Systems and Other Structures by Base Isolation," edited by J.M. Kelly - January 1981 (PB81 200 735)A05

UCB/EERC-81/02 "OPTNSR - An Interactive Software System for Optimal Design of Statically and Dynamically Loaded Structures with Nonlinear Response," by M.A. Bhatti, V. Ciampi and K.S. Pister - January 1981 (PB81 218 851)A09

UCB/EERC-81/03 "Analysis of Local Variations in Free Field Seismic Ground Motions," by J.-C. Chen, J. Lysmer and H.B. Seed - January 1981 (AD-A099508)A13

UCB/EERC-81/04 "Inelastic Structural Modeling of Braced Offshore Platforms for Seismic Loading," by V.A. Zayas, P.-S.B. Shing, S.A. Mahin and E.P. Popov - January 1981(PB82 138 777)A07

UCB/EERC-81/05 "Dynamic Response of Light Equipment in Structures," by A. Der Kiureghian, J.L. Sackman and B. Nour-Omid - April 1981 (PB81 218 497)A04

UCB/EERC-81/06 "Preliminary Experimental Investigation of a Broad Base Liquid Storage Tank," by J.G. Bouwkamp, J.P. Kollegger and R.M. Stephen - May 1981(PB82 140 385)A03

UCB/EERC-81/07 "The Seismic Resistant Design of Reinforced Concrete Coupled Structural Walls," by A.E. Aktan and V.V. Bertero - June 1981(PB82 113 358)A11

UCB/EERC-81/08 "The Undrained Shearing Resistance of Cohesive Soils at Large Deformations," by M.R. Pyles and H.B. Seed - August 1981

UCB/EERC-81/09 "Experimental Behavior of a Spatial Piping System with Steel Energy Absorbers Subjected to a Simulated Differential Seismic Input," by S.F. Stiemer, W.G. Godden and J.M. Kelly - July 1981

UCB/EERC-81/10 "Evaluation of Seismic Design Provisions for Masonry in the United States," by B.I. Sveinsson, R.L. Mayes and H.D. McNiven - August 1981

UCB/EERC-81/11 "Two-Dimensional Hybrid Modelling of Soil-Structure Interaction," by T.-J. Tzong, S. Gupta and J. Penzien - August 1981(PB82 142 118)A04

UCB/EERC-81/12 "Studies on Effects of Infills in Seismic Resistant R/C Construction," by S. Brokken and V.V. Bertero - September 1981

UCB/EERC-81/13 "Linear Models to Predict the Nonlinear Seismic Behavior of a One-Story Steel Frame," by H. Valdimarsson, A.H. Shah and H.D. McNiven - September 1981(PB82 138 793)A07

UCB/EERC-81/14 "TLUSH: A Computer Program for the Three-Dimensional Dynamic Analysis of Earth Dams," by T. Kagawa, L.H. Mejia, H.B. Seed and J. Lysmer - September 1981(PB82 139 940)A06

UCB/EERC-81/15 "Three Dimensional Dynamic Response Analysis of Earth Dams," by L.H. Mejia and H.B. Seed - September 1981 (PB82 137 274)A12

UCB/EERC-81/16 "Experimental Study of Lead and Elastomeric Dampers for Base Isolation Systems," by J.M. Kelly and S.B. Hodder - October 1981 (PB82 166 182)A05

UCB/EERC-81/17 "The Influence of Base Isolation on the Seismic Response of Light Secondary Equipment," by J.M. Kelly - April 1981 (PB82 255 266)A04

UCB/EERC-81/18 "Studies on Evaluation of Shaking Table Response Analysis Procedures," by J. Marcial Blondet - November 1981 (PB82 197 278)A10

UCB/EERC-81/19 "DELIGHT.STRUCT: A Computer-Aided Design Environment for Structural Engineering," by R.J. Balling, K.S. Pister and E. Polak - December 1981 (PB82 218 496)A07

UCB/EERC-81/20 "Optimal Design of Seismic-Resistant Planar Steel Frames," by R.J. Balling, V. Ciampi, K.S. Pister and E. Polak - December 1981 (PB82 220 179)A07

UCB/EERC-82/01 "Dynamic Behavior of Ground for Seismic Analysis of Lifeline Systems," by T. Sato and A. Der Kiureghian - January 1982 (PB82 218 926)A05

UCB/EERC-82/02 "Shaking Table Tests of a Tubular Steel Frame Model," by Y. Ghanaat and R. W. Clough - January 1982 (PB82 220 161)A07

UCB/EERC-82/03 "Experimental Behavior of a Spatial Piping System with Shock Arrestors and Energy Absorbers under Seismic Excitation," by S. Schneider, H.-M. Lee and G. W. Godden - May 1982

UCB/EERC-82/04 "New Approaches for the Dynamic Analysis of Large Structural Systems," by E. L. Wilson - June 1982 (PB83 148 080)A05

UCB/EERC-82/05 "Model Study of Effects of Damage on the Vibration Properties of Steel Offshore Platforms," by F. Shahrivar and J. G. Bouwkamp - June 1982

UCB/EERC-82/06 "States of the Art and Practice in the Optimum Seismic Design and Analytical Response Prediction of R/C Frame-Wall Structures," by A. E. Aktan and V. V. Bertero - July 1982 (PB83 147 736)A05

UCB/EERC-82/07 "Further Study of the Earthquake Response of a Broad Cylindrical Liquid-Storage Tank Model," by G. C. Manos and R. W. Clough - July 1982 (PB83 147 744)A11

UCB/EERC-82/08 "An Evaluation of the Design and Analytical Seismic Response of a Seven Story Reinforced Concrete Frame - Wall Structure," by F. A. Charney and V. V. Bertero - July 1982

UCB/EERC-82/09 "Fluid-Structure Interactions: Added Mass Computations for Incompressible Fluid," by J. S.-H. Kuo - August 1982

UCB/EERC-82/10 "Joint-Opening Nonlinear Mechanism: Interface Smeared Crack Model," by J. S.-H. Kuo - August 1982 (PB83 149 195)A05

UCB/EERC-82/11 "Dynamic Response Analysis of Techi Dam," by R. W. Clough, R. M. Stephen and J. S.-H. Kuo - August 1982 (PB83 147 496)A06

UCB/EERC-82/12 "Prediction of the Seismic Responses of R/C Frame-Coupled Wall Structures," by A. E. Aktan, V. V. Bertero and M. Piazza - August 1982 (PB83 149 203)A09

UCB/EERC-82/13 "Preliminary Report on the SMART 1 Strong Motion Array in Taiwan," by B. A. Bolt, C. H. Loh, J. Penzien, Y. B. Tsai and Y. T. Yeh - August 1982

UCB/EERC-82/14 "Shaking-Table Studies of an Eccentrically X-Braced Steel Structure," by M. S. Yang - September 1982

UCB/EERC-82/15 "The Performance of Stairways in Earthquakes," by C. Roha, J. W. Axley and V. V. Bertero - September 1982

UCB/EERC-82/16 "The Behavior of Submerged Multiple Bodies in Earthquakes," by W.-G. Liao - September 1982

UCB/EERC-82/17 "Effects of Concrete Types and Loading Conditions on Local Bond-Slip Relationships," by A. D. Cowell, E. P. Popov and V. V. Bertero - September 1982

UCB/EERC-82/18 "Mechanical Behavior of Shear Wall Vertical Boundary Members: An Experimental Investigation," by M. T. Wagner and V. V. Bertero - October 1982

UCB/EERC-82/19 "Experimental Studies of Multi-support Seismic Loading on Piping Systems," by J. M. Kelly and A. D. Cowell - November 1982

UCB/EERC-82/20 "Generalized Plastic Hinge Concepts for 3D Beam-Column Elements," by P. F.-S. Chen and G. H. Powell - November 1982

UCB/EERC-82/21 "ANSR-III: General Purpose Computer Program for Nonlinear Structural Analysis," by C. V. Oughourlian and G. H. Powell - November 1982

UCB/EERC-82/22 "Solution Strategies for Statically Loaded Nonlinear Structures," by J. W. Simons and G. H. Powell - November 1982

UCB/EERC-82/23 "Analytical Model of Deformed Bar Anchorages under Generalized Excitations," by V. Ciampi, R. Eligehausen, V. V. Bertero and E. P. Popov - November 1982

UCB/EERC-82/24 "A Mathematical Model for the Response of Masonry Walls to Dynamic Excitations," by H. Sucuoğlu, Y. Mengi and H. D. McNiven - November 1982

UCB/EERC-82/25 "Earthquake Response Considerations of Broad Liquid Storage Tanks," by F. J. Cambra - November 1982

UCB/EERC-82/26 "Computational Models for Cyclic Plasticity, Rate Dependence and Creep," by B. Mosaddad and G. H. Powell - November 1982

UCB/EERC-82/27 "Inelastic Analysis of Piping and Tubular Structures," by M. Mahasuverachai and G. H. Powell - November 1982

UCB/EERC-83/01 "The Economic Feasibility of Seismic Rehabilitation of Buildings by Base Isolation," by J. M. Kelly - January 1983

UCB/EERC-83/02 "Seismic Moment Connections for Moment-Resisting Steel Frames," by E. P. Popov - January 1983

UCB/EERC-83/03 "Design of Links and Beam-to-Column Connections for Eccentrically Braced Steel Frames," by E. P. Popov and J. O. Malley - January 1983