Seismic Steel Framing Systems for Tall Buildings

EGOR P. POPOV

A number of steel framing systems can be used to develop the required lateral strength, stiffness, and ductility for resisting forces caused by earthquakes. Moment-resisting frames and braced frames are by far the most widely used systems for this purpose. More recently, a number of major buildings were designed and built in California employing the concept of eccentric bracing. In this approach the line of action in a brace is deliberately offset from a common joint point. Any bracing scheme in which a component of an axial force in a brace is transferred through shear in a beam can be classified as an eccentrically braced frame. The spread K-bracing is a well-known example. A number of other bracing schemes can be devised using this concept. This type of hybrid construction is very versatile and fulfills the essential requirements for seismic design.

This paper reviews the advantages and disadvantages of the three framing systems noted above. The choice of a particular framing system depends on a large number of factors. Functional requirements or close spacing of columns may mandate moment-resistant framing. Conventional bracing may be fully acceptable in applications where moderate ductility of frames suffices. In some applications, eccentric bracing may be the most economical. Because of the newness of this system of framing, some design guidelines are given. Experimental results are provided on the behavior of the critical elements for each system of framing.

MOMENT-RESISTING FRAMES

In the design of tall buildings, moment-resisting structural steel frames are more widely used than any other type. This kind of framing results in no obstructions between the columns, allowing maximum freedom in interior planning and fenestration. The lateral integrity of such frames depends entirely on having beam-column joints of sufficient strength *and* ductility. These characteristics are best determined experimentally by subjecting building subassemblages (Fig. 1) to severe cyclic loadings simulating the effects of an earthquake. A number of such experiments have been performed at Berkeley to determine and verify adequacy of behavior required.

Typical building subassemblages that can be used effectively in experiments are shown cross-hatched in Fig. 1 on a portion of a moment-resisting steel frame.[1] For either the exterior or interior subassemblage, it is sufficiently accurate to assume columns to be pinned at their mid-heights. Moreover, since the joint is the critical element, the outer ends of test beams can also be approximated by hinges.

Joint details widely used are shown in Fig.2. One is of an all-welded type with two erection bolts; the other has the beam flanges fully welded to the column flange and the web bolted with high-strength bolts to a shop-welded shear plate. Structural steel fabricators appear to prefer the latter type for reasons of economy. The Berkeley experiments on full-size specimens subjected to severe cyclic loading indicated that well fabricated all-welded connections were somewhat better than those of the hybrid type. Nevertheless, the performance of connections with bolted webs and welded flanges in general were also found to be satisfactory. Load-deflection hysteretic loops for a subassemblage approximating an exterior joint of Fig. 1 with only four high-strength web bolts and welded flanges are shown in Fig. 3a.[2] The repetitiveness of the hysteretic loops for the same deflection indicates that no deterioration in the joint capacity had occurred for a number of severe cycle reversals. The connection reached a maximum capacity of 67.5 kips in the upward direction and 72 kips in the downward direction on two consecutive full cycles when the tip deflection of the cantilever was over five times that which occurred at first yield. The failure of this type of connection after a large number of very severe cyclic loadings tends to occur abruptly. In contrast, all-welded connections exhibit a more ductile behavior as illustrated in Fig. 3b. However, most designers are satisfied with the ductility achieved with connections using bolted webs and welded flanges, and this type of connection is now almost universally used in California.

Egor P. Popov is Professor of Civil Engineering, University of California, Berkeley, California.

This paper is based largely on a conference paper prepared for presentation at the Sino-American Symposium on Bridge and Structural Engineering, Sept. 13–17, 1982, Beijing, P.R.C.

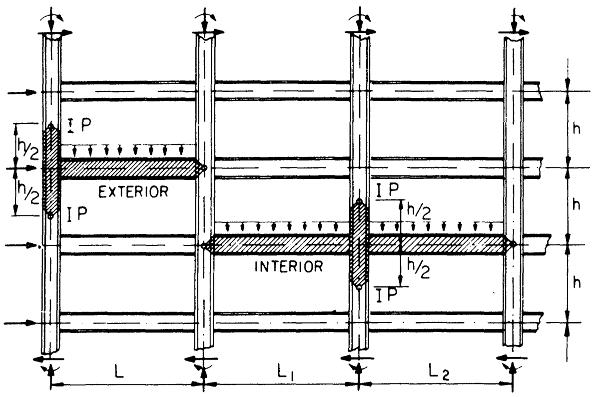


Fig. 1. A portion of a moment-resisting steel frame showing idealized subassemblages for exterior and interior columns[1]

Laboratory experiments[3] have demonstrated that similar connections to column web tend to be less ductile and preferably should be avoided for resisting significant seismic (cyclic) loads. Some of the newer developments on this problem may be found in Ref. 4.

In designing moment-resisting steel frames, consideration must also be given to the panel zone in the beam-column joint. As shown in Fig. 4,[5] the shearing stresses in the column webs can be very large. This can cause shear deformation in the panel zone, increasing the story drift as shown in Fig.5.[6] The shear deformation of the panel zone increases the number of the degrees of freedom of the frame. Many of the available computer programs overlook these important effects. Column web doubler plates are used to minimize the story drift due to the shear deformations in panel zones. A comprehensive review of this problem together with some recent full-size experiments on beam-column subassemblages with doubler plates at the panel zone may be found in Ref. 7.

Usually moment-resisting frames are well suited along the wide dimensions of buildings. However, for tall buildings the doubler plate thicknesses sometimes become excessive. This requirement, if coupled with the need for large, deep beams, also for controlling story drift, may require another type of framing.

CONCENTRICALLY BRACED FRAMES

Often it is more economical to use braces than to depend on the moment-resistance of a frame alone for resisting lateral seismic forces in the direction of the narrow dimension of a building. In most cases the center lines of braces and beams, as well as those of columns where applicable, are made to meet at a point. Frames braced in this manner are referred to as concentrically braced frames. Two examples of such framing are shown in Fig. 6.

For the zones where there is high probability of earthquake occurrence of such magnitude as to cause major damage during the life of a structure, some caution in adopting concentrically braced framing should be exercised. It is not fully appreciated that if braces repeatedly buckle under the imposed cyclic loads, a dramatic decrease in their compressive capacity can occur. As an example, hysteretic loops for axial force vs. axial displacement for a strut subjected to fully reversed loading are displayed in Fig. 7a.[8] Whereas most of the previous research was devoted to determining the capacity of columns under a monotonically applied load, the phenomenon encountered here pertains to cyclic loading. As can be seen from Fig. 7a, the axial strut capacity is reduced by more than 50% of its initial strength on repeated load reversals.

It is also noteworthy to observe that during even moderate axial displacement between the column ends, large lateral deflections develop. For example, at an axial differential displacement between the ends of δ less than 0.4 in., a lateral (sidewise) displacement Δ on the order of 6 in. (Fig. 7b) takes place.

The abrupt decrease in the axial carrying capacity of struts under fully reversed cyclic loading is due to the in-

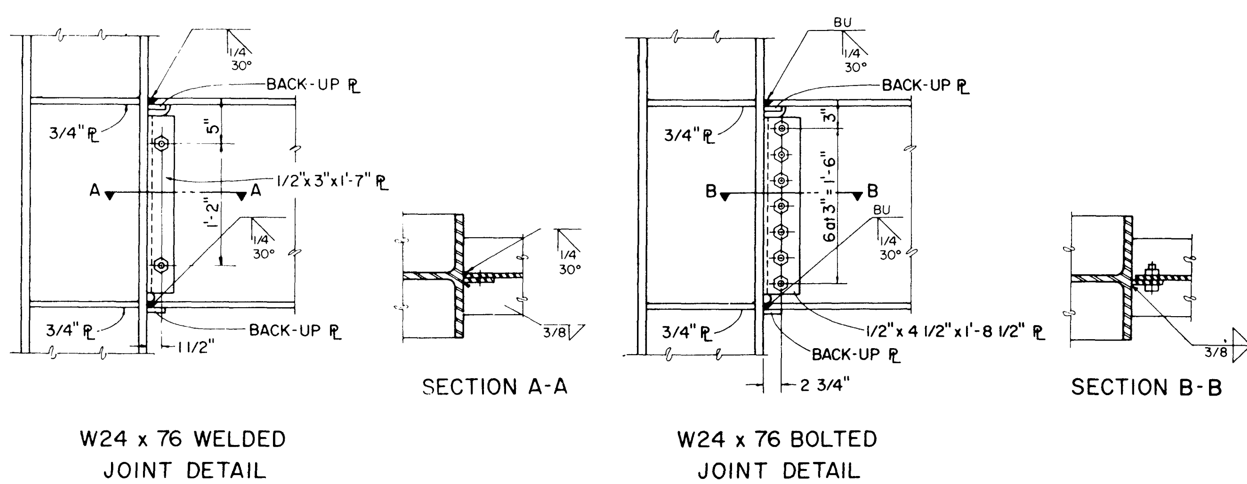
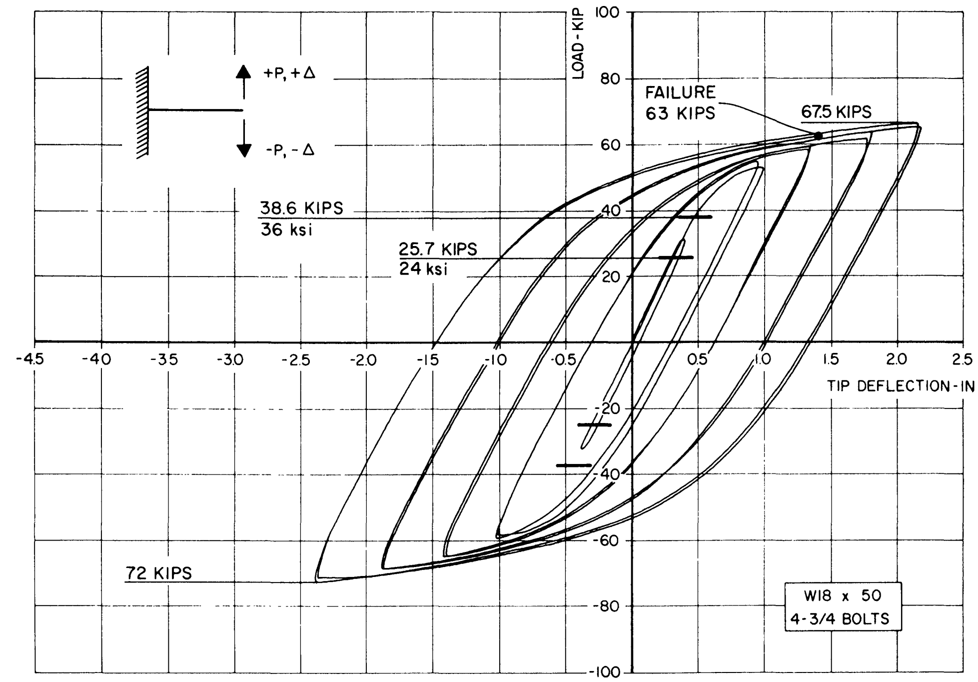
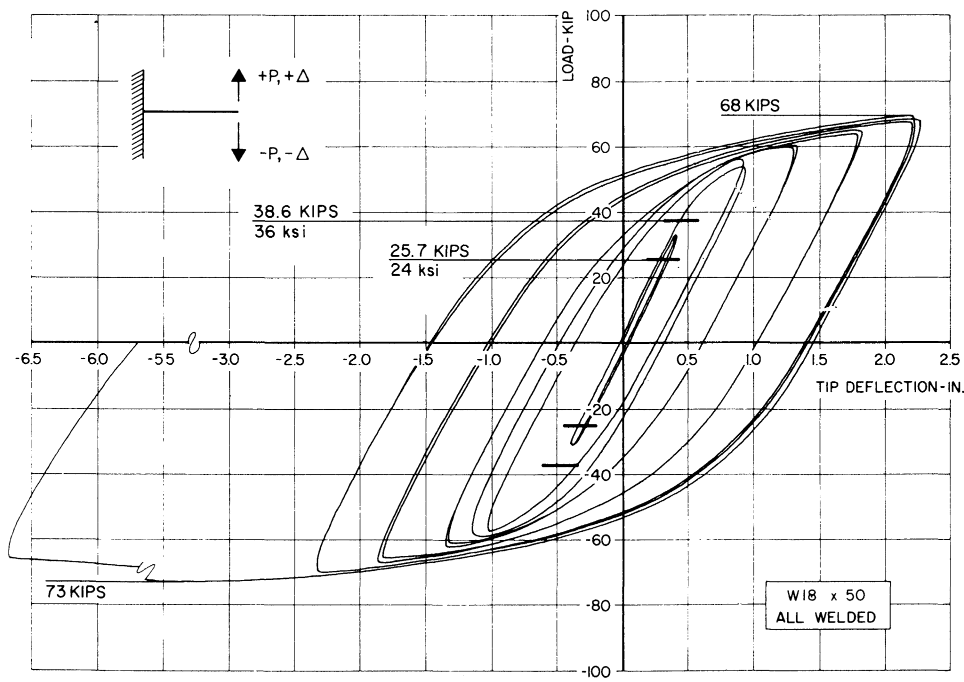


Fig. 2. Typical joint details for moment connection[2]



(a)



(b)

Fig. 3. Load-deflection (P-Δ) hysteretic loops for 8 ft W18x50 cantilevers simulating (a) exterior connection with welded flanges and bolted web, and (b) all-welded connections[2]

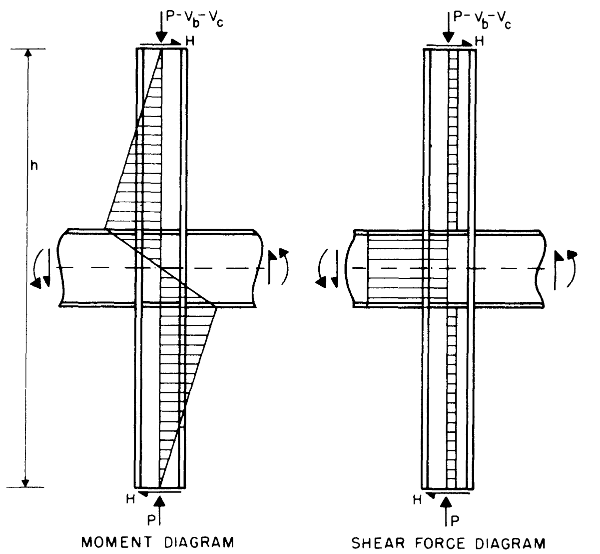


Fig. 4. Typical moment and shear diagrams for an interior column assuming hinges at column mid-heights[5]

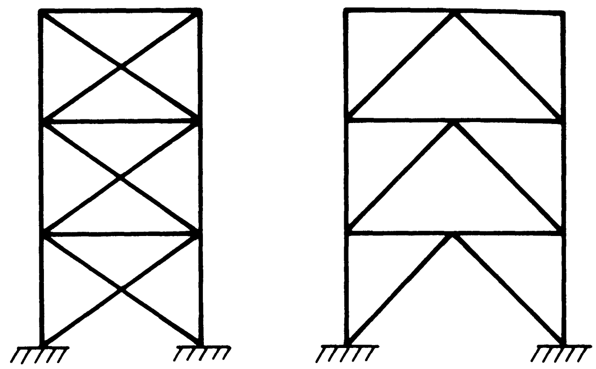


Fig. 6. Examples of concentrically braced frames

trinsic characteristic of mild steel. Under cyclic loading, a stress-strain diagram for mild steel exhibits a strong Bauschinger effect, such as shown in Fig. 8.[8] In this diagram one can note that on the initial application of a compressive stress, an elastic region followed by a plastic region can be identified. However, on reversing the applied load, as well as for all subsequent cycles, the stress-strain diagrams become strongly curved. Thus, in the relevant compressive regions of loading, the steel does not have the elastic stiffness E, but is characterized by the tangent modulus E_t. Inasmuch as it is this modulus that plays the dominant role in determining the capacity of a column, the compressive capacity of a brace can decrease, roughly speaking, by the ratio of E_t to E. A more comprehensive discussion of this problem can be found in Ref. 8.

Usually in zones of high probability of severe earthquakes, the joints of a braced frame are made moment-resistant. Therefore, the overall frame response depends on the combined action of bracing and a moment-resisting frame. For this reason, the overall behavior of such frames under lateral cyclic loading can be satisfactory. Some experimental results for a frame of this type are shown in Fig. 9.[9] This one-third scale test frame was designed as an end wall of a three-story building. The design forces and the details were typical of those employed for wind bracing. As can be seen from the figure, up to the story drift index (displacement divided by story height) of 0.005, the frame performed well. In fact, this frame behaved quite well at double the corresponding displacements. Therefore, the design of such a frame would satisfy the current Uniform Building Code (UBC),[10] but would not meet the Recommendations of the Applied Technology Council (ATC),[11] which recommends that reliable behavior be achieved for

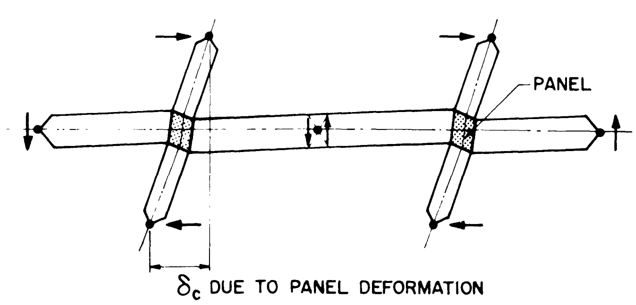


Fig. 5. Increase in story drift due to improperly designed column panel zones[4]

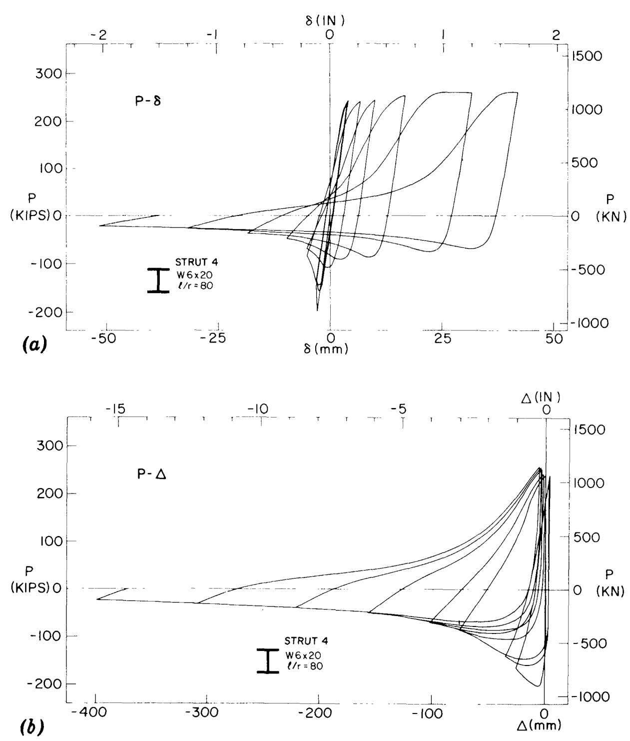


Fig. 7. (a) Hysteretic loops for axial force vs. axial displacement for a pinned-end strut; (b) axial force vs. maximum lateral deflection for strut in (a)[8]

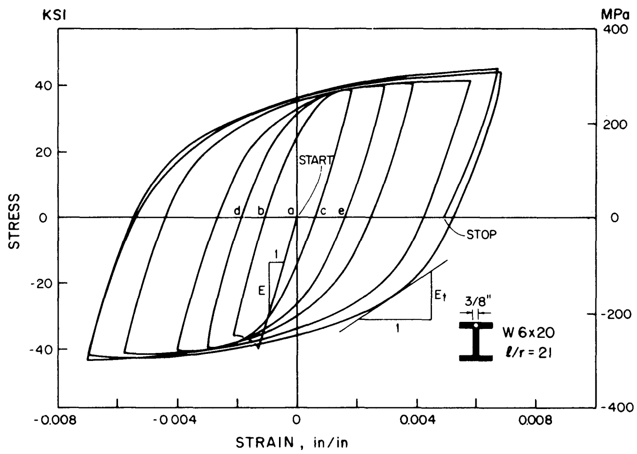


Fig. 8. Typical cyclic stress-strain diagram for steel[8]

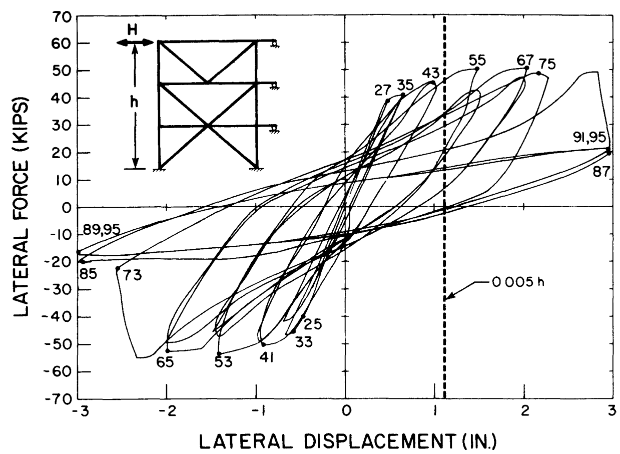


Fig. 9. Hysteretic loops for K-braced frame[9]

a story drift index of 0.015 for seismic hazard exposure in groups I and II. Because of the limited experimental data on the behavior of concentrically braced frames, conservative designs for this type of framing appear to be appropriate at this time.

ECCENTRICALLY BRACED FRAMES

By deliberately offsetting the center lines of braces from common points where the center lines of beams and columns intersect, one obtains eccentrically braced frames. Several examples of such frames are shown in Fig. 10. Many other arrangements are possible. For example, the braces in the top two stories shown in Fig. 10c may be inverted, or arranged to form a rhombic pattern. The characteristic short beam segments between the braces, or between a brace and a column, are referred to herein as beam links or simply links.

In selecting an eccentric brace configuration, it is essential to have a kinematic collapse mechanism such that each brace is connected to at least one beam link which can experience large relative displacements between its ends, as shown in Fig. 11. Since the plastic shear capacity of a link can be accurately estimated, a brace can be selected of sufficient strength to prevent it from buckling. To assure that the braces would not buckle, it is recommended that they be designed to develop 1.5 times the plastic shear capacity in the links. In this manner the behavior of an eccentrically braced frame becomes entirely different from that of a concentrically braced frame. The possibility of reduced axial capacity of a brace under cyclic loads does not enter the problem.

It should be emphasized that an earthquake loading induces a dynamic response of short duration. Therefore, a mechanism motion associated with a collapse mechanism, such as either one shown in Fig. 11, is only an indication of the extent of energy dissipation through plastic deformation and of ductility requirements in the critical regions. The meaning of the solution is completely different from that when defining the collapse of a frame under sustained loads.

In setting the lengths of links, the collapse mechanism for a frame must be studied, Fig. 11. Note that regardless of the assumed direction for the collapse, the links on the

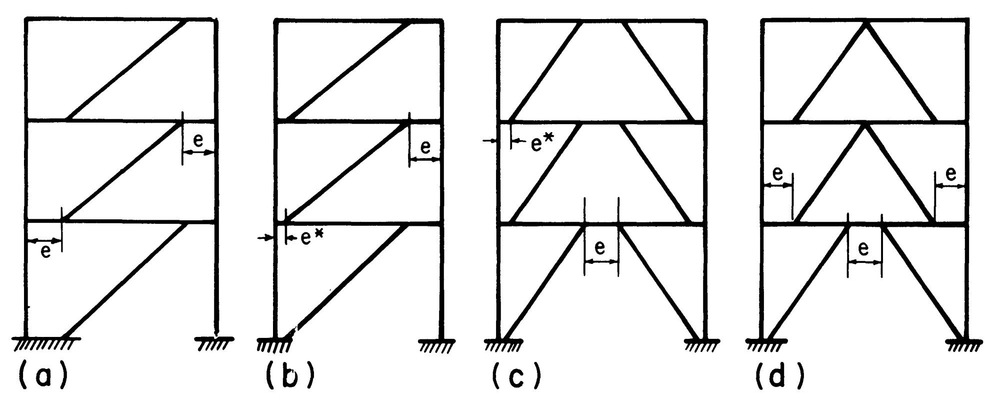


Fig. 10. Alternative arrangements of eccentric bracing[12,13]

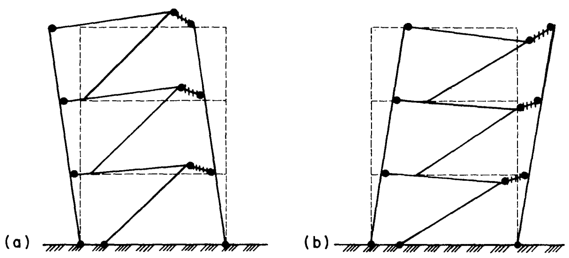


Fig. 11. Collapse mechanism for a frame in two opposite directions leading to identical conclusions[12,13]

right experience large relative displacement between their ends; whereas the relative displacement between the ends of the links on the left are small. On performing an analysis to determine whether the assumed kinematically admissible field is also a statically admissible one, it is found that only moment hinges form next to the column and experience moderate rotation. These links can be made as short as convenient without imposing severe ductility requirements on the members. This is implied in Figs. 10b and 10c, where $e \gg e^*$.

The links experiencing large relative displacements between their ends can be of significantly different lengths in relation to the beam depth or span. For example, the link lengths e such as shown in Fig. 10c, could be one-third of the span or even larger. Alternatively, these link lengths could be chosen to be on the order of two beam depths or less. At collapse, in the first design, *plastic moment hinges* would form at the ends of the links. This type of design tends to result in a relatively flexible structure. The second design produces a more rigid structure, with the short links deforming plastically in shear, giving rise to *plastic shear hinges*. In both instances large ductility requirements are imposed on the links. Some comments on the design of the two kinds of links follow:

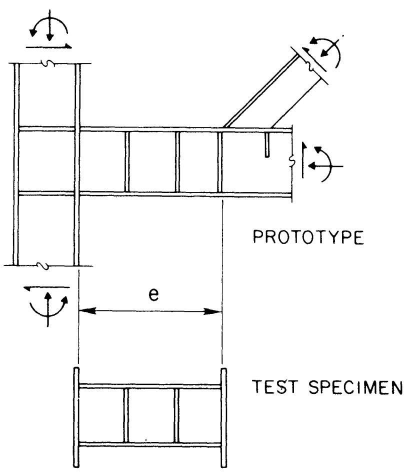


Fig. 12. Modeling of a link[14]

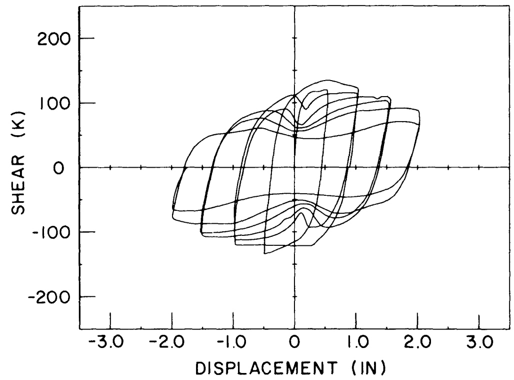


Fig. 13. Specimen 1 force-displacement history[14]

Shear Links—In the design of shear links for cyclic loading, two basic problems are encountered which were not 3 envisioned in the standard codes.[10,11] Full plastic shear capacity at connections must be developed, and the tendency for webs to buckle under cyclic loads must be inhibited or prevented. Because of the lack of experimental data on this subject, an extensive investigation has been conducted at Berkeley. For this purpose, a typical shear hinge was isolated for study from a frame as indicated in Fig. 12. To date 28 full-size experiments have been completed using W12 x 22 to W18 x 60 sections. Some representative results from two of the tests with W18 x 40 sections are shown in Figs. 13 through 16.[14] Specimen 1 was made without stiffening the web, and the web formed a clear panel of 28 x 16.85 in. The force-deformation diagram for this specimen is shown in Fig. 13. A photograph of the failed specimen is shown in Fig. 14. In this link, web buckling occurred early in the second cycle. Large amplitude lateral buckling of the web on the order of 3 in. caused significant deterioration of the load-carrying capacity of the link in the later cycles.

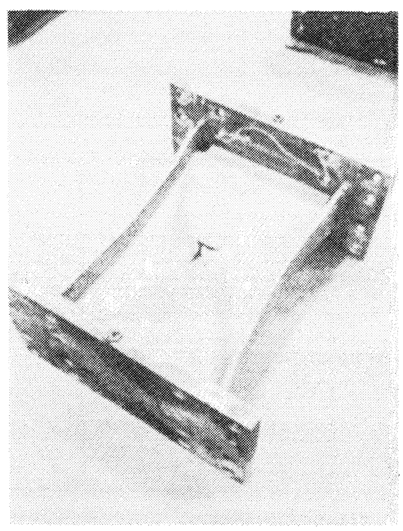


Fig. 14. Failed Specimen 1[14]

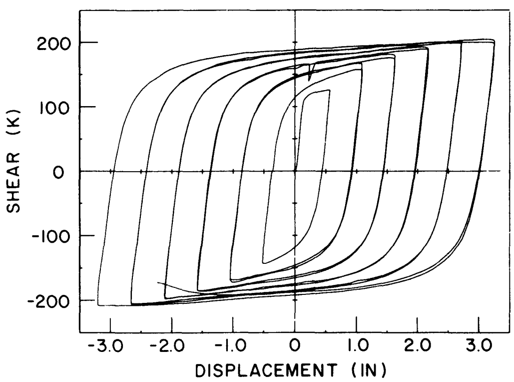


Fig. 15. Specimen 4 force-displacement history[14]

The results of an experiment on Specimen 4 with three equally spaced stiffeners subdividing the web into four 7 x 16.85 in. panel zones are shown in Figs. 15 and 16. The excellent hysteretic loops for this specimen are due to the fact that throughout most of the test the deformations in the specimen were primarily caused by inelastic shearing strains in the web. Web buckling was of small amplitude and did not occur until the ninth cycle of the test. Note that, due to strain-hardening, the shear capacity progressively increased, providing good justification for conservative design of braces, in order to activate the links.

These and other experiments clearly show the need for web stiffeners along plastic shear links. A suggested detail is shown in Fig. 17. To minimize the ductility demand on links, which may be expected to experience significant inelastic displacement between their ends, they should be made as long as possible. Usually their length can be set at about 1.8 to $2d$, where d is the beam depth. A more accurate criterion of determining their length is given as $5b_f$, where

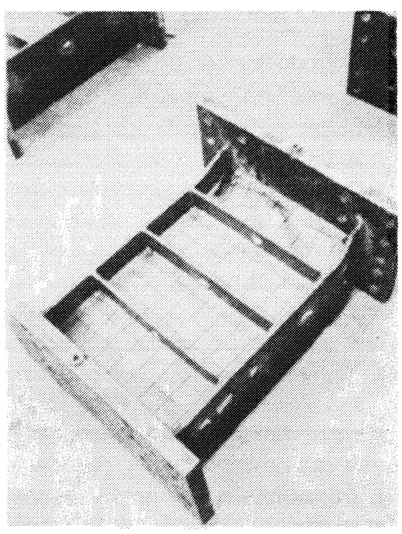


Fig. 16. Failed Specimen 4[14]

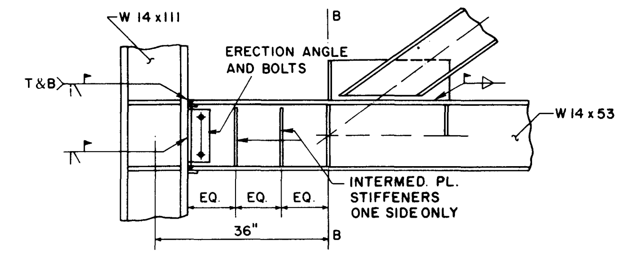


Fig. 17. Suggested detail for a shear link with web stiffeners; lateral brace on Line B-B

b_f is the beam flange width. The latter expression approximates rather well the ratio between the plastic moment and shear capacities of a beam, which is the fundamental basis for establishing the optimum length of a shear link.

It would appear that web stiffener spacing between $24t_w$ and $30t_w$, where t_w is the thickness of a web, is appropriate for applications for the severe service conditions considered in this paper. With a decrease in the plastic ductility requirements, the intermediate stiffeners may be spaced farther apart. For the elastic range, the current AISC specifications, usually requiring no stiffeners for rolled sections, appear to be adequate.

In the initial test specimens for the envisioned applications, the intermediate fitted stiffeners were placed on both sides of a web. Subsequent experiments have shown, however, that stiffeners on only one side of a web sufficiently inhibit the web buckling, resulting in satisfactory shear links. Further, it was determined that the intermediate stiffeners need not be closely fitted to the beam flanges, as plastic moment hinging does not develop in these regions. This differs from the requirements for a pair of fitted web stiffeners at the brace where moment hinging can occur. Therefore, conservatively, only gaps between the intermediate stiffeners and the top beam flange are indicated. Locating these gaps at the top is based on the premise that usually the floor slab would offer some buckling restraint to the flange. Documentation on the Berkeley web stiffener research is in preparation and will be released shortly.

Links with Moment Hinges—Links longer than twice the beam depth or $5b_f$ tend to develop plastic moment hinges at the ends. For such links of moderate length, i.e., length

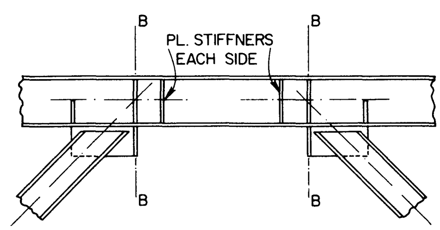


Fig. 18. Typical link of moderate length with plastic moment hinges at ends; lateral braces on Lines B-B[13]

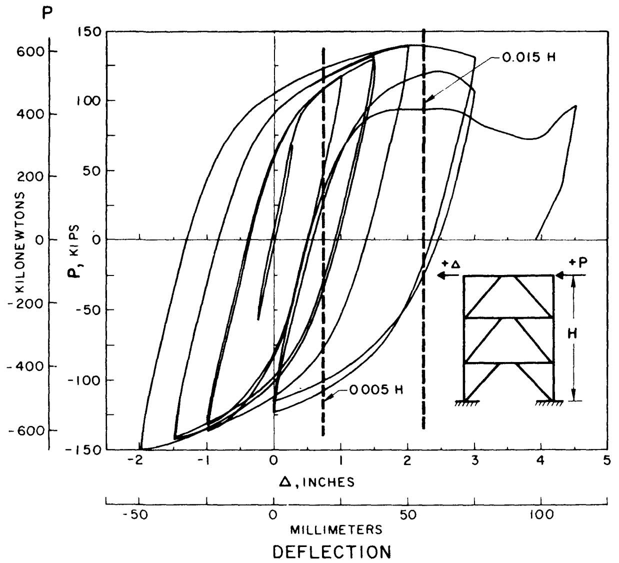


Fig. 19. Hysteretic loops for one-third size eccentrically braced frame[13]

greater than $2d$ and less than $4d$, the same web stiffener requirements as for shear links apply, except that the outer fitted stiffeners should be placed at a distance no greater than the width of the beam flange from the ends of a link (Fig. 18). For moment links of length between $4d$ and $6d$, only end stiffeners at a distance equal to the width of the beam flange at both ends of a link need be provided, as shown in Fig. 18. No web stiffeners appear to be necessary for moment links longer than $6d$. For links of intermediate length, future research may relax some of the above requirements.

Regardless of the kind of link employed, top and bottom flanges of the beams should be braced at the ends of the link beams to prevent lateral-torsional buckling. The axial forces which may develop in the links must also be considered in design.

Test frames designed on the above basis showed excellent behavior; an example of overall hysteretic behavior of one such frame may be noted from Fig. 19. This frame sustained not only the story drift of $0.005H$ prescribed by UBC,[10] but also that of $0.015H$ recommended by ATC.[11]

CONCLUSIONS

State-of-the-art of seismic design for three types of structural steel framing systems was discussed. Each type has its particular advantages and disadvantages. These may be summarized as follows:

Moment-Resisting Frames provide the greatest flexibility in exterior and interior planning by having no obstruction between the columns. The ductility demand at the beam-column connections is moderate and can be easily implemented with welded flanges and bolted webs. However, for some structures, this sytem of framing tends to be relatively flexible under the action of lateral loads and during a severe earthquake excessive nonstructural damage can occur. To prevent this, story drifts must be controlled, requiring larger beams than necessary for strength alone. Moreover, doubler plates in the panel zone of columns are often necessary. For some buildings these requirements may make moment-resisting framing uneconomical.

Concentrically Braced Frames provide an excellent structural system for regions of low or moderate probability of strong earthquake occurrence. This structural system is very efficient for resisting lateral loads, usually requiring smaller columns and beams than those for moment-resisting frames. The resulting structure is stiff laterally, a very desirable attribute. At locations where braces are placed some architectural problems with locating windows and doors may be encountered. With this kind of a structura system, non-structural damage during a moderate shak would be expected to be small. Some reservations for th use of concentrically braced frames in regions with a hig probability of severe earthquakes can be expressed. Unde severe cyclic reversal of loads, the brace capacities ca significantly decrease. Because of their large latera buckling displacements, the braces could cause damage i their surrounding area and the frame may become inef fective in dissipating energy.

Eccentrically Braced Frames appear to offer sever advantages over concentric bracing, although, like the latte they cause some obstruction in the bays. A properly de signed eccentrically braced frame is very stiff at light an moderate lateral loads, and is very ductile at extrem overloads such as may occur during an extraordinaril strong earthquake. This enables the framing to absorb ar dissipate large amounts of energy and California co struction experience has shown these frames to be ec nomical. Whereas ductility comparable to that of m ment-resisting frames is achieved, beam sizes are sma Thereby the expensive details of doubler plates in colum panel zones is avoided. Such frames being intrinsically sti they would in general tend to reduce the nonstructur damage. However, at active links where considerable di placement between their ends can take place, fracturing the floor slabs should be anticipated. Such effects are like to rapidly attenuate with story height.

In addition to seismic applications, eccentrically brac frames may be used to advantage in non-seismic desig In some cases, very large and expensive connections r sulting purely from the geometrical requirements associa with forcing the center lines of all members of a braci connection to pass through a common working point c be avoided. This can be achieved using a detail such as tl shown in Fig. 17, with significantly relaxed requireme for stiffeners and their fit-up. Further, by adopting concept of eccentric bracing connections for non-seisr applications, one can design modified concentrically bra frames using, for example, the detail designed for fi bolting shown in Fig. 20. By moving the working poin the column face, the detail becomes compact.

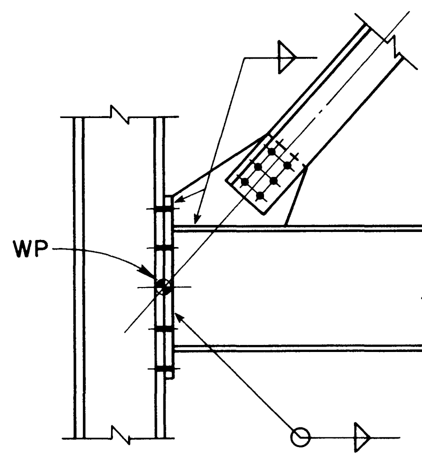


Fig. 20. Modified "concentrically braced" frame connection

In designing any structural frame for randomly reversing lateral forces causing ineleastic activity at joints, the possibility of columns going into single curvature should be carefully checked and accounted for. This problem, related to the global frame stability, has received very scant attention to date.

ACKNOWLEDGMENTS

In developing the point of view expressed in this paper, the author is indebted to his former and present students and colleagues. Their names are interspersed throughout the references. Major support for conducting the research was provided by the National Science Foundation, current Grant No. CEE 81-07217. The American Iron and Steel Institute also provided significant aid. This support is gratefully acknowledged. Any opinions, findings, conclusions and recommendations expressed in this paper are those of the author and do not necessarily reflect the views of NSF nor AISI.

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