ECCENTRIC SEISMIC BRACING OF STEEL FRAMES

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SUMMARY

Unlike conventional diagonal bracing, where the brace centerlines pass through the centers of beam-column joints, braces in eccentrically braced frames are deliberately off-set with respect to such joint centers. This provides short segments of floor beams between a column and a diagonal brace. Under severe cyclic loadings, the webs of the short beam segments adjacent to the columns yield cyclically in shear and serve as energy absorbing and dissipating devices. Some adoptions of this bracing scheme have been made in practice. In this paper the general concepts of the design of eccentrically braced steel frames are reviewed.

INTRODUCTION

The use of eccentrically braced steel frames for resisting lateral loads is not entirely new. In his 1930 book on Wind Bracing Spurr [1] suggested their occasional use for architectural reasons, and it would appear that tall buildings using such bracing were designed and built in the New York area. More recently a very tall building was built in Texas using this scheme [2]. A more deliberate use of eccentric joints for seismic design may be attributed to Fujimoto [3], who performed a number of tests on eccentric K-braces. Some preliminary designs utilizing diagonal braces with eccentricities at the columns were made in 1972 by Degenkolb [4]. Experimental results on one-third scale models of eccentrically braced steel frames for the lower three stories of a 20-story building published by Roeder and Popov [5] renewed interest in this type of bracing, and some adoptions have been made in practice.

In this paper, first, the frames used in the Roeder-Popov experiments are described, followed by some selected experimental results. The extrapolation of the available information from small scale experiments to design follows. Areas of needed future research are then indicated.

ECCENTRICALLY BRACED FRAMES

A 20-story, four-bay square office building served as the prototype. The bay widths were 24 ft (7.3 m), and the story heights were 12 ft (3.6 m) for all stories except the first, which was 15 ft (4.6 m). The structure was designed using the 1976 Uniform Building Code lateral load provisions [6], and the American Institute of Steel Construction (AISC) allowable stresses [9]. An elevation of the exterior braced frame is shown in Fig. 1.

A typical eccentric bracing arrangement is shown in Fig. 2. The short segments of the beams (shear links) providing the eccentricity e are so proportioned that during plastic deformation the webs yield before

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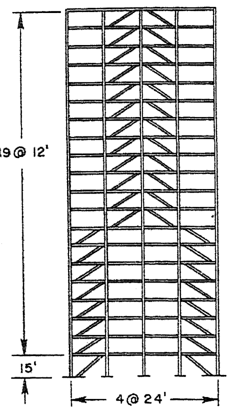

Fig. 1 Prototype Structure [5]

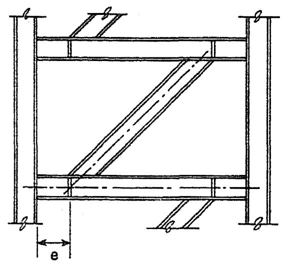

Fig. 2 Typical Eccentric Brace [10]

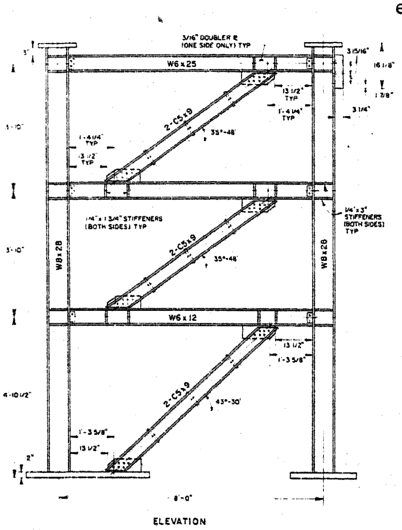

Fig. 3 Design of Test Frame 1 [5]

the plastic moment capacity of a beam is reached. Because of the cyclic yielding in the webs of the shear links during a severe earthquake, stiffeners along such links may be required. The braces are selected such that their capacity can cause yielding of the beam webs, thereby excluding the possibility of brace buckling. As is customary in the design of moment-resisting steel frames, the columns are selected using the strong column-weak beam approach.

The details for one of two one-third scale models for a braced bay for the lower three floors of the prototype, based on the concepts outlined above, are shown in Fig. 3. The behavior of the shear links at the second and third floor levels was of particular interest. These links were parts of the W6 × 12 floor beams and were 13 in (330 mm) long. However, since 2 in (50 mm) wide shear tabs were welded both to the columns and to the beam webs, the effective unsupported web panels were approximately 11 in (280 mm) long and 6 in (150 mm) high. The webs of these beams were 0.23 in (5.8 mm) thick. The design called for no vertical stiffeners along the shear link.

PRINCIPAL EXPERIMENTAL RESULTS

The two test frames behaved very well during experiments. The imposed lateral displacements at the third floor level attempted to simulate displacements equal to or exceeding those which might occur during an

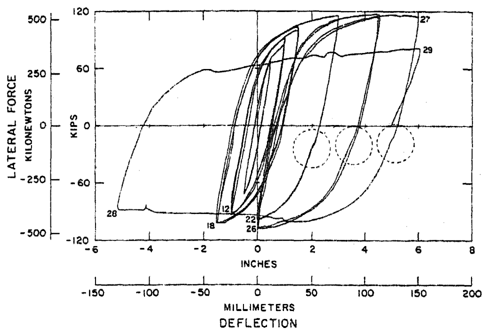

Fig. 4 Hysteretic Behavior of Test Frame 1 Lateral Force-Third Floor Deflection [5]

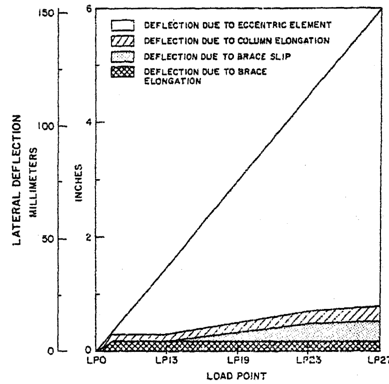

Fig. 5 Components of Lateral Deflection for Test Frame 1 [5]

El Centro and a Pacoima earthquake in tandem. Excellent hysteretic loops were observed well into the inelastic range. (See Fig. 4 for lateral force-third floor deflection loops for Test Frame 1.) The components for lateral deflection are shown in Fig. 5. After web yield begins in the shear links, the ductility of the frame is principally due to inelastic deformation in the shear links. The ultimate failure of the shear links, by tearing and buckling, occurred at very advanced stages of loading, having little practical meaning.

EXTRAPOLATIONS INTO DESIGN

The design of eccentrically braced steel frames must conform to the conventional elastic criteria stipulated in standard codes [6]. In conformity with good practice the story drift must be kept to a practical minimum. For example, the Structural Engineers Association of California recommends to limit [7] the elastic wind drift to 0.0025 times the story height, and for prescribed earthquake forces, to twice this amount. In calculating the story drift for eccentrically braced frames, it is appropriate to include the shear deformations of the links. A very useful bulletin for practical design of eccentrically braced frames has been prepared by Teal [8].

No special problems arise in applying the elastic methods of analysis, nor are there any particularly unusual problems in the design of columns and beams in the inelastic range of behavior except for the design of the shear links. The shear links themselves play a key role in maintaining the integrity of a frame, and their capacity in the inelastic range of behavior must be carefully determined and implemented in the design. Lateral torsional buckling of the links must be prevented, and buckling of the flanges and webs at extreme overloads must be minimized. The AISC lateral bracing provisions for plastic design [9] appear to be appropriate for preventing lateral torsional buckling. Usually this would require attaching the beam flanges to a column and providing a lateral brace at the other end of a link. (A less conservative bracing arrangement was found to be satisfactory in the frame tests referred to earlier [5].)

The buckling problem of the web and the flanges in the shear link is interrelated. If the suggestion [10] of setting the link length some 10 to 30% smaller than that which causes the development of plastic moments at the ends of a link is adhered to, the flanges are not likely to buckle until the web buckles. However, if the shear link length is set so that full plastic moments at the ends of a link could occur, a strong possibility of early flange buckling at extreme overloads can take place. Under such circumstances the flanges would force the thinner web to rotate and buckle. To avoid this highly undesirable situation, pairs of stiffeners spaced at approximately one-half of the buckling length of a flange should be provided at both ends of a link. Usually this would mean that stiffeners would be placed at both ends of a link at a spacing approximately equal to the flange width of a beam or less. The need for additional stiffeners along a link, if required, can be arrived at in a manner analogous to that discussed below.

If the development of plastic moment hinges at the ends of a link is prevented by reducing its length so as to cause no plastic moments, the basic problem becomes principally one of web buckling due to shear. In examining this problem it must be recognized that the AISC provisions [9] for determining shear capacities of rolled sections are directed toward monotonically applied loads. Moreover, the 1978 AISC Specifications relaxed requirements for the depth/web-thickness ratio for compact sections. Therefore, the problem of web buckling in the shear links needs to be carefully considered.

As stated earlier in the experiments on one-third scale models, the W6 × 12 shear links had an effective clear panel size of approximately 11 × 6 in (280 × 150 mm). The webs were 0.23 in (5.8 mm) thick. In the prototype this translates into a non-standard W18 × 108 section with a 0.69 in (17.5 mm) web. No standard W18 section can meet these requirements. The webs of the available sections are thinner, indicating a possibility of web buckling.

Some guidance on web buckling of beams under monotonic loads is available [11,12]. However, there is dearth of data as to the behavior of yielding webs under cyclic loading. Therefore, for the present it would seem reasonable to determine the required stiffeners along a link based on the satisfactory performance of the links in the test frames. This can be done by using direct geometric proportions. For example, consider a 36 in (900 mm) long shear link as part of a W18 × 65 floor beam with a 0.403 in (10.2 mm) thick web. The clear web panel size for the web of the given thickness could then be taken approximately as 19 × 10.5 in (480 × 270 mm), i.e., in direct ratio to the web thicknesses; thereby requiring three pairs of double vertical stiffeners along a link. Instead of equal spacing of these stiffeners, a slightly closer spacing near the ends of a link than in the middle can be rationalized. In unusual cases some consideration of transferring an axial force through a shear link must also be given.

ADDITIONAL CONSIDERATIONS

The extent to which one should stiffen the webs of the shear links is tied-in with the required hinge rotation. If one designs for a maximum credible earthquake, and the extent of the permitted inelastic strain reversal is small, less stringent requirements on web stiffening may be justified.

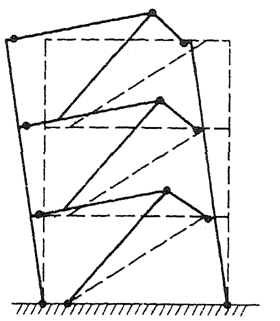

Fig. 6 Plastic Collapse Mechanism [10]

It is also of interest to note that a collapse mechanism for an eccentrically braced frame can be visualized as shown in Fig. 6. For the diagonal bracing scheme shown, large rotation demands are placed on the right links, whereas there is little of inelastic activity at the left links. For this reason, in some cases, it may be advantageous to significantly reduce the eccentricity of the left links as shown in Fig. 7(b).

Several other arrangements of eccentric connections are possible. An example is shown in Fig. 7(a) [3]. A different concept or using coupling beams between two conventionally designed braced frames is shown in

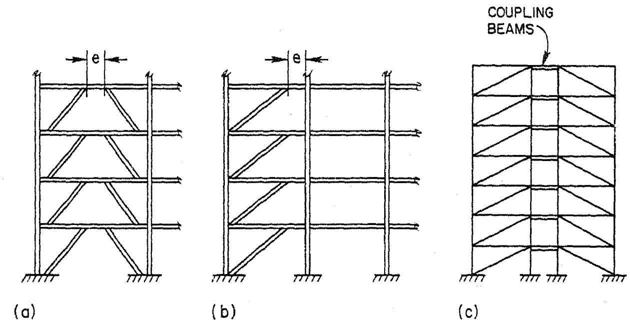

Fig. 7 Bracing Arrangement Schemes

Fig. 7(c). This approach is analogous to that of coupled reinforced concrete shear walls, and may have a particular advantage in reducing the axial forces in columns due to lateral loads.

CONCLUDING REMARKS

The great interest shown by designers in eccentrically braced steel frames is due to their apparent advantage in resisting lateral forces. This conclusion is reached mainly on the basis of elastic analyses. The beam sizes are smaller and the frames are stiffer than one would readily obtain in conventional moment-resistant designs. The simultaneous participation of the numerous shear links contributes to the efficiency of the system.

For code level seismic design [6] of eccentrically braced steel frames on elastic basis the same conclusions as above clearly apply. In the ductile range of frame behavior, however, some open questions remain. There is need for a better basis for determining the required web stiffeners along a link. Full-size experiments together with an appropriate theory on cyclic web buckling are needed. In some situations large shear hinge rotations can be anticipated. The extent of acceptable floor damage must be made more precise.

To place the highly promising eccentrically braced frame system on a firmer basis, further analytical studies are required. These must include elasto-plastic analyses for static and dynamic cases of different type buildings. To attain the status comparable to that of moment-resisting frames, experiments on a shaking table also appear to be very desirable.

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