An Update on Eccentric Seismic Bracing

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Experimental and analytical results for diagonally braced frame models with eccentric connections of the type shown in Fig. 1, first reported by Roeder and Popov in 1978,[1] aroused considerable interest in this system of bracing for resisting seismic forces. Four major buildings have now been designed and are being built in California using this concept.[2] The steel frames designed on this basis are lighter than the equivalent moment-resisting frames, and while essentially retaining the elastic stiffness of concentrically braced frames, the eccentrically braced frames are more ductile. The adoption of the eccentric frames in several projects provides strong evidence of the practicality of the system.

By employing eccentrically braced frames, the story drift control at working loads, such as recommended by SEAOC,[3] can be achieved with economy. Moreover, a reduced horizontal force factor can be justified, since at extreme lateral loadings, such as may occur during a major earthquake, these frames behave in a very ductile manner. Some specific details on the practical design of eccentrically braced frames for resisting seismic forces can be found in Refs. 4, 5, and 6. This discussion is confined to some elaborations on the design of the shear link, i.e., the beam element between the face of the column and a brace.

DESIGN OF A SHEAR LINK

No special problems arise in applying the elastic methods of analysis to eccentrically braced frames, 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. However, since the shear links play a key role in maintaining the integrity of a frame, their capacity in the inelastic range of behavior must be carefully determined and implemented in the design. Lateral torsional buckling of the links and contiguous beams must be prevented, and buckling of the flanges and webs at extreme overloads must be minimized. In the earlier AISC paper by Popov and Roeder,[6] these aspects of the design were not addressed.

To appreciate the problem of web buckling in the shear links, it is well to re-examine the data from the original one-third scale experiments with eccentrically braced

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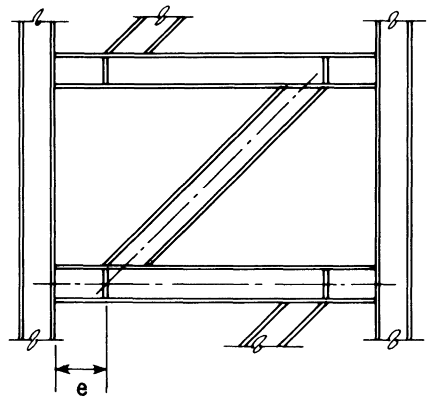

Figure 1

frames.[1] The W6 x 12 beams for these models of A36 material were selected on the basis of experiments showing their good performance in the shear link regions under extreme cyclic loadings. In the frame models the W6 x 12 shear links had an effective panel size of approximately 11 x 6 in. The webs were 0.23 in. thick, with a test yield strength of 50.4 ksi. In the prototype this translates into a non-standard W18 x 108 section with a 0.69-in. web. No standard W18 section can meet these requirements. The webs of the available sections are thinner, raising the question of the possibility of their buckling.

Some guidelines on web buckling of beams under monotonic loads are available.[7,8] However, there is a 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 original test frames.[1] This opinion is substantiated by some recent tests by the author and his associates with W8 x 13 sections having 0.23-in. webs. The shear links in the test beams buckled and lost strength at relatively low ductilities.

Based on the above reasoning, the 11 x 6 in. panels with 0.23-in. web thicknesses, which performed well in cyclic experiments, can be used for sizing other panels, and thereby determine the required stiffener spacing. As an example, consider the shear link in Ref. 6, where the beam is W14 x 53 with a web 0.37 in. thick. Neglecting the effect of any difference in web yield strengths, and using a direct ratio of the web thicknesses, for this full-size beam an unsupported panel may be made 17.7 x 9.65 in., since

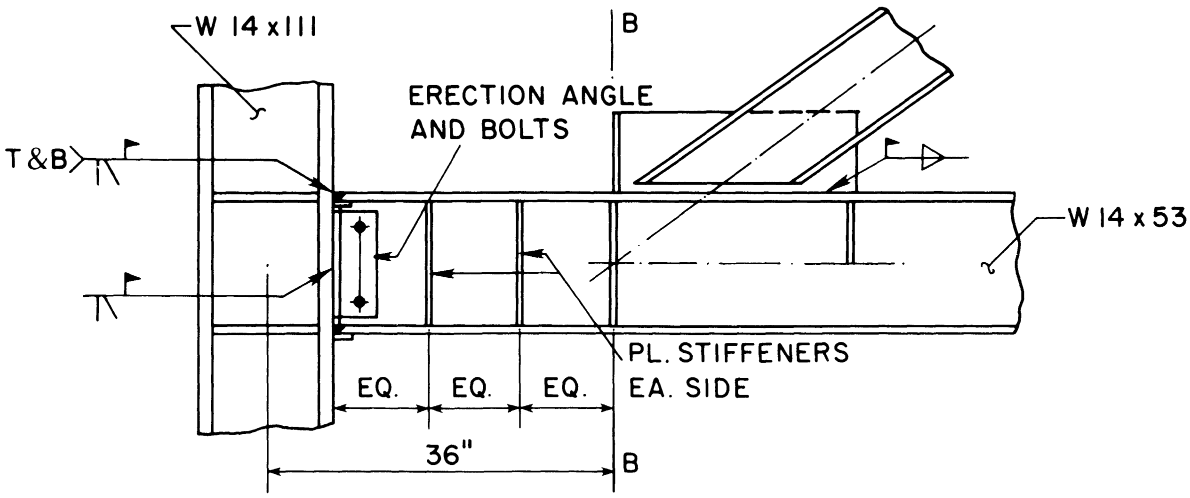

Fig. 2. Eccentric connection

(0.37/0.23) x 11 = 17.7 in., and (0.37/0.23) x 6 = 9.65 in. By taking this panel size as an upper bound, two pairs of equally spaced stiffeners along the shear link are required, as shown in Fig. 2. These additional stiffeners would prevent the development of premature buckling, thereby assuring the integrity of the frame.

ADDITIONAL CONSIDERATIONS

The buckling problem of the web and the flanges in the shear link is interrelated. If the suggestion advanced in Ref. 6 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 a very large amount of web buckling takes place. This normally would occur well beyond the useful range of frame behavior. However, if the link length is set so that full plastic moments at the ends of a link could occur first, a possibility of 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 undesirable situation, pairs of stiffeners spaced at approximately one-half of the flange buckling length should be provided at both ends of a link. In most instances this is a less desirable approach for the design of a shear link.

As required in the conventional plastic design, appropriate lateral bracing must be provided to prevent lateral torsional buckling of beams at plastic hinges. Usually this would require welding of the beam flanges to a column and providing a lateral brace at the other end of a link. Such a moment-resisting brace normal to a beam link can be conveniently attached to a web stiffener, as on line BB in Fig. 2.

CONCLUDING REMARKS

The great interest shown by designers in eccentrically braced steel frames is due to their apparent economy for resisting lateral seismic forces. The beam sizes are smaller and the frames are stiffer than one would normally obtain in conventional moment-resisting design. However, as is apparent from the above discussion, some open questions remain. There is a need for a better basis for determining the required number of web stiffeners along a link. (The procedure suggested herein may be unduly conservative.) Because shear hinge rotations beyond the elastic load capacity can be anticipated with this type of framing in a major earthquake, considerable floor damage may occur; analagous to that which would develop in a ductile moment frame. However, in minor earthquakes, elastic action and only minor non-structural damage would be anticipated. Because of the many possible variations of eccentric bracing arrangements, further analytical and experimental studies are needed. Work on these and related problems is now in progress under the sponsorship of NSF and AISI.

REFERENCES

1. *Roeder, C. W. and E. P. Popov* Eccentrically Braced Steel Frames for Earthquakes *Journal of Structural Division, ASCE, Vol. 104, No. ST3, March 1978.*
2. *Anonym.* Eccentric Bracing is Key to Seismic Resistance *Engineering News-Record, Oct. 25, 1979.*
3. Recommended Lateral Force Requirements and Commentary *Seismology Committee, Structural Engineers Association of California, 1973.*
4. *Degenkolb, H. J.* Practical Design (Aseismic) of Steel Structures *Canadian Journal of Civil Engineering, Vol. 6, No. 2, 1979.*
5. *Teal, E. J.* Practical Design of Eccentric Braced Frames to Resist Seismic Forces *Structural Steel Educational Council, AISC, Los Angeles, California, 1980.*
6. *Popov, E. P. and C. W. Roeder* Design of an Eccentrically Braced Steel Frame *AISC Engineering Journal, 3rd Quarter 1978.*
7. *Basler, K.* Strength of Plate Girders in Shear *Trans. ASCE, Vol. 128, Part II, 1963.*
8. Manual of Steel Construction *Seventh Edition, AISC, New York, 1970.*