

Design of an Eccentrically Braced Steel Frame

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In the design of earthquake resistant structures, two basic requirements must be met. First, the structure must remain serviceable during the ordinary, frequently occurring load applications. This is usually accomplished by designing the structure so that it remains elastic and provides adequate stiffness to limit deflections. The second requirement is to preclude a disaster during a major earthquake. For such an extreme event, considerable inelastic deformation is usually allowed. Thus, structures must possess sufficient ductility and inelastic stability to withstand these extreme excitations.

Recent studies^{1,2} have shown that eccentrically braced frames offer considerable potential as seismic resistant structures. They are very stiff and can easily satisfy story drift limitations, and they can be designed to provide excellent inelastic behavior and energy dissipation characteristics. Therefore, such structures are likely to remain serviceable during the smaller, more frequently occurring earthquakes, and prevent collapse during severe, infrequent earthquakes. Eccentrically braced steel frames also appear to be very economical structures, indicating savings for some framing arrangements on the order of 30% in weight of steel over unbraced frames.¹ This unique structural system offers several design advantages, but it also employs several unusual design requirements. The purpose of this paper is to summarize these special design requirements and to show a simple example illustrating their application.

GENERAL DESIGN APPROACH

The eccentrically braced frame discussed here employs a brace which has a thrust line that does not go through the intersection of beam and column center lines, but, instead, is framed as shown in Fig. 1. The eccentric beam element provides a ductile fuse which assures good inelastic behavior and energy dissipation. Tests have shown^{1,3} that the inelastic behavior of this structural system is very good, provided the beam is designed to yield in shear. This shear yield behavior is generally attained if the clear eccentricity

e is on the order of two times the beam depth. A good design is achieved if plastic hinges form at both ends of the eccentric beam element shortly after shear yielding. Therefore, the eccentricity and plastic moment capacity must be carefully balanced to assure the proper yield mechanism.

The preliminary design of an eccentrically braced frame is best performed by employing plastic design concepts, such as given in Part 2 of the AISC Specification.⁴ The factored design forces can be determined by using the moment balancing approach.^{1,5} This is a lower bound technique, which is obtained by designing the structure for any distribution of internal forces and bending moments which satisfy statics. Using this method, an efficient design can be obtained if the desired collapse mechanism, shown in Fig. 2, is kept in mind during the distribution process. The first step in attaining a suitable distribution of moments in the

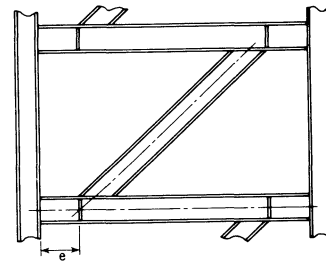


Figure 1

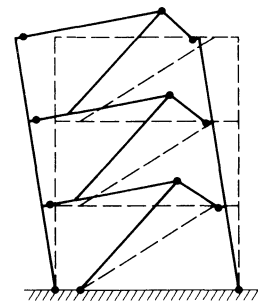


Figure 2

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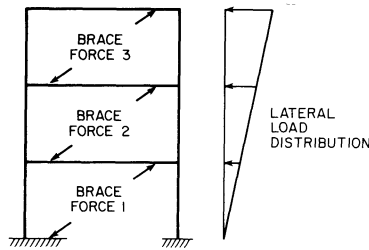


Figure 3

frame is to note that in each panel a brace provides 70 to 85% of the lateral stiffness, and so it should carry a corresponding percentage of the lateral shear. This provides a quick estimation of the force in a brace. In the subsequent moment balancing procedure, the effect of braces is replaced with their forces, as shown in Fig. 3. After this simplification, the moment balancing procedure can be performed in several different ways; an approach is shown later, in the design example.

The members can be designed after completion of the plastic force distribution. The beam should be designed to first yield in shear without excessive conservatism. The shear capacity, V , of a wide flange beam can be determined from the equation:

$$V = 0.55 \sigma_y t_w d \quad (1)$$

where σ_y is the yield stress of the steel, t_w is the web thickness, and d is the depth of the beam. Because of the high shear force in the eccentric beam element, instead of the conventional plastic moment capacity, M_p , the reduced moment capacity, M_p^* , given by the following formula, must be used:

$$M_p^* = t_f b \sigma_y (d - t_f) \quad (2)$$

where b is the flange width and t_f is the flange thickness. After selecting the beam for its required shear capacity, it should be checked to assure that plastic hinges form at both ends of the eccentric beam element shortly after shear yielding. This can be accomplished by requiring that $2M_p^*/e$ be somewhat larger than V . A satisfactory relationship among these quantities can be taken as $1.1 V \leq 2M_p^*/e \leq 1.3 V$. To attain the desired balance, it may be necessary to adjust eccentricity or beam size or add flange cover plates. Web doubler plates should not be used to attain the desired balance.

After the beam is designed, the column should be selected by the "weak beam-strong column" approach.⁶ Therefore, if the beam is designed to attain a moment M_p^* which is larger than required by the design forces, the column should be designed in an equally conservative manner. It is also very important to avoid buckling of the brace. This is accomplished by increasing the design axial force of the brace to account for increased brace forces induced by an overly conservative design of the beam, strain hardening of the

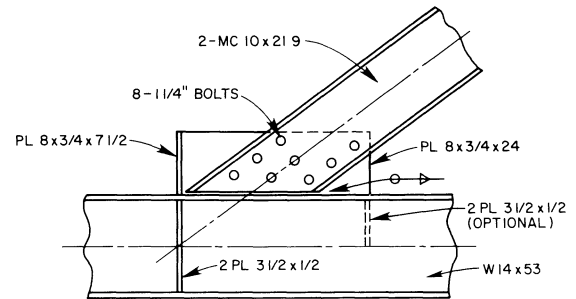


Figure 4

beam, or uncertainty in the yield stress of the beam. Thus, it is recommended that the brace be designed for a force at least 50% higher than required by moment balancing.

The brace-to-beam connection can be designed as a bolted connection, as shown in Fig. 4. The number of bolts may be determined using criteria for friction-type bolts; however, the bolt capacity should also be checked in bearing, because bolt slippage can be expected during a severe earthquake.¹ It is suggested that the bearing stresses be limited to the yield stress, σ_y , as opposed to the $1.35 \sigma_y$ allowed by AISC.^{1,4} The connection requires a pair of web stiffeners at the toe of the brace. These are required to distribute the shear stress in the web and to assist with the development of the cyclic diagonal tension field during extreme inelastic deformations.^{1,3} The gusset plate attachment for the brace can be built up to form a tee, as shown in Fig. 4. In some designs, a structural tee can be used to advantage. In either case, a tee-shaped attachment to a beam is desirable, since it reduces the eccentricity between the thrust line of the brace and the centroid of the gusset plate welds. The flange of the tee can be directly aligned with the web stiffeners to provide a direct transfer of brace force. The gusset plate must not be allowed to extend into the eccentric beam element,¹ because large deformations can be expected in this region during a severe earthquake. Experiments have shown¹ that, although not essential, a pair of partial depth beam web stiffeners are desirable. These web stiffeners are marked "optional" in Fig. 4., whereas the other stiffeners are essential.

The beam-to-column connection should be a moment resisting connection, as shown in Fig. 5. The flanges of the beam are welded to the column with full penetration welds. The beam web is bolted to the erection plate with friction-type bolts. However, because of the high shear force in the eccentric beam element, a single row of bolts cannot usually provide sufficient shear capacity, and so a fillet weld along the erection plate is added to act in combination with the friction-type bolts to develop the shear capacity of the beam.

The final design consideration pertains to the lateral support of the members. Due to the high shear force in the eccentric beam element, the moment gradient in it is high

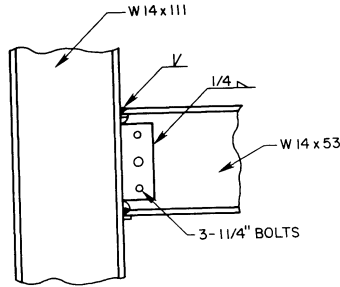


Figure 5

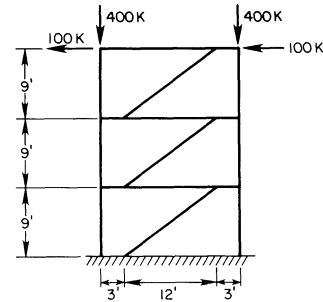


Figure 6

and little additional support is needed for this part of the beam. The moment gradient is much lower in the center beam section, and so lateral torsional buckling is more likely to be a problem in this region. The center section may also be subjected to high compressive force during a load reversal, and so lateral support is needed to prevent lateral torsional and weak-axis buckling. Tests have shown that lateral support provided at the third or quarter points of the beam span may be sufficient, and the floor joists commonly used are likely to provide adequate support.¹ In general, eccentrically braced frames require lateral support to both top and bottom beam flanges, as required by the AISC Specification.⁴

After completion of the preliminary design, the results should be checked using a linear elastic analysis. Such an analysis is useful in determining whether the deflections are acceptable, and it provides the basis for a working stress design check of the preliminary design. The results of such an analysis may warrant some adjustments in the design; in making these, care must be exercised to assure that the inelastic behavior will remain essentially unaltered. Specifically, the beam should retain its weak link feature and should yield first in shear. If the beam sizes are increased, the columns and braces must also be made larger.

PRELIMINARY DESIGN EXAMPLE

Figure 6 shows the frame geometry and loading used to illustrate a preliminary design of an eccentrically braced frame. The given loadings have been multiplied by the appropriate load factors so that one can proceed directly with the analysis. Here the vertical loads can be directly transmitted down their respective columns. It will be assumed that at each level a brace will provide 80% of the lateral stiffness. Therefore, since each brace is designed as an axially loaded member, the axial force in each brace can be set at 200 kips, as shown in Fig. 7. For the subsequent analysis for moments, the braces can be removed from the frame and replaced by an equivalent force system, shown in Fig. 3.

At this point, the moment balancing could proceed in several ways,⁵ but it is desirable to first examine each of the beams with the objective of obtaining a collapse mechanism

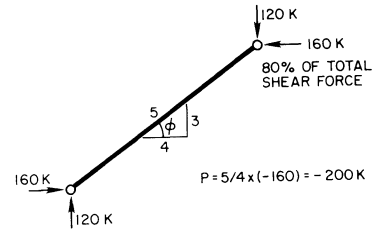
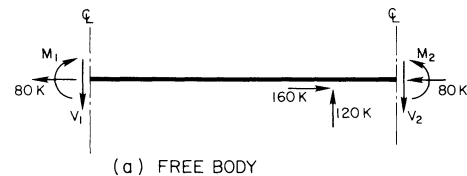
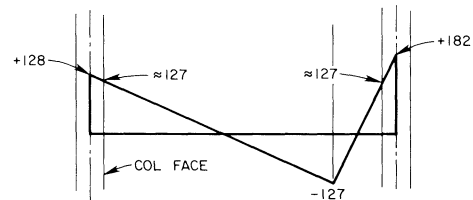


Figure 7



(a) FREE BODY



(b) MOMENT DIAGRAM (K-FT)

Figure 8

similar to that shown in Fig. 2. For example, consider the top beam, shown as a free body in Fig. 8(a). For this beam to be compatible with the desired collapse mechanism, moments at both ends and a moment at the intersection of the beam-brace center lines should form plastic hinges. However, the plastic end moments can form only at the column faces. Therefore, because of the moment gradients, the moments at the column center lines are larger. In this case, M_2 is significantly larger than the plastic moment, whereas M_1

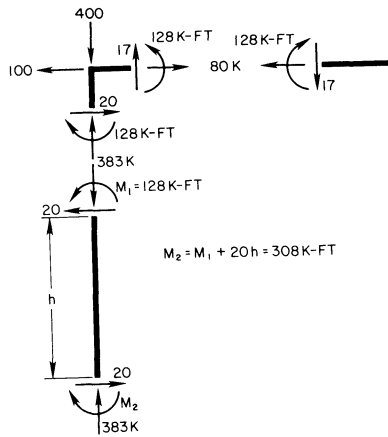


Figure 9

is nearly equal to such a moment [Fig. 8(b)]. By selecting $M_1 = 128$ kip-ft, and $M_2 = 182$ kip-ft,* and assuming columns to be W14 sections, the bending moments at the three plastic hinge locations are approximately 127 kip-ft. End moments for other beams are chosen in a similar manner.

The bending moments for the columns are determined by arbitrarily distributing the lateral shear not taken by the braces to the columns and systematically applying the conditions of statics to each of the beam-column joints. In this example, 20% of the total shear (80% was assigned to the braces) is distributed equally between the two columns. The moment balancing procedure starts at the top joints with the bending moment in the column taken equal to that in the beam (Fig. 9). The moment at the base of a column is found from the known shear force and the moment at the column top; see Fig. 9. The bending moments and shear forces for the lower columns are found by working down the structure. The results of this procedure give the bending moments satisfying the conditions of statics shown in Fig. 10(a). The axial forces are shown in Fig. 10(b); these are obtained by applying the conditions of statics at each joint. The sum of the reactions from the two diagrams keeps the frame in equilibrium.

The forces and moments shown in Fig. 10 can be used for the selection of member sizes and connections, but it should be noted that the columns will have to be designed for single curvature. This can result in an overly conservative column design, which should be avoided. The results of balancing can be substantially improved by slightly changing the proportion of lateral shear that is carried by some or all of the braces and/or columns. For example, if 5% of the shear force is assigned to the top left

* Note that a baseline in Fig. 8(b) connecting the points corresponding to moments M_1 and M_2 , together with a point -127 kip-ft at the brace, defines a moment diagram for a simply supported beam.

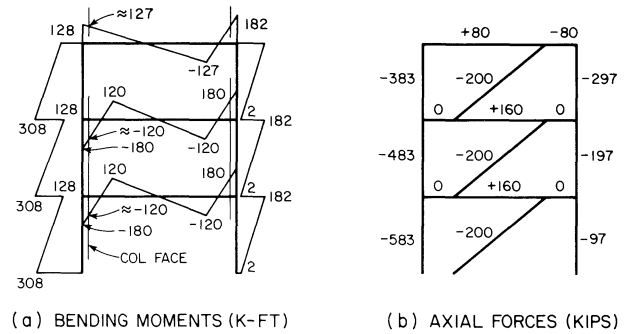


Figure 10

column, and 15% to the top right column, with all other quantities left unchanged, the maximum column design moments would be reduced from 308 to 218 kip-ft.

The selection of members can proceed after the plastic design forces, such as given in Fig. 10, are available. The requirements of beam sizes at all levels are very similar. For example, considering a beam at the first or second floor levels, one notes that the maximum shear is $(180 + 120)/3 = 100$ kips and the maximum moments are approximately 120 kip-ft, assuming W14 columns. Using A36 material, the possible beam sections, together with the relevant properties, are given in Table 1. From the sections given, W18x35 is rejected, as it does not satisfy the slenderness requirements. The W14x53 section is preferable over W16x45, because of its greater lateral stability. For compact sections, the lateral torsional buckling is believed to be dependent upon the local flange buckling, which requires yielding of the flange for a length at least equal to a full wave length.⁸ The clear eccentricity e is approximately $(36 - 14/2) = 29$ in., and, thus, neither the W16x45 nor W14x53 is likely to form flange buckles because of the very steep moment gradient. However, the W14x53 section is preferable because of its wider flanges, which makes the member more stable laterally. The W14x53 should perform satisfactorily with lateral supports applied to the top and bottom flanges at the third points.

For the member selected, bending yield will occur considerably later than shear yielding, as can be noted from $2M_p^*/e$. Bending yield would occur earlier, and the design

Table 1. Member Properties

Size	M_p (kip-in.)	M_p^* (kip-in.)	V (kips)	Kl/r	P_{cr} (kips)
W18x35	2405	1601	104	58.6	306
W16x45	2956	2216	110	45.8	422
W14x53	3132	2536	102	37.5	514
W14x111	7056	—	—	29.0	1112
2-MC10x21.9 with 3/4-in. spacing	—	—	—	80.0	360

would be improved, if the eccentricity e were increased, say, by 8 or 9 in. With this large increase in eccentricity, a new moment balancing should be performed following the procedure previously described.

For unrevised geometry and assuming W14x53 beams, the brace must be designed for an axial force of 306 kips. This follows from the following reasoning. The 200-kip axial force in the brace is increased by 2% to account for the oversized beam ($V = 102 \text{ kips} < 100 \text{ kips}$ required), and by a factor of 1.5 to assure that the brace would not buckle due to the uncertainties in the material properties of the beam, i.e., its true yield strength and strain hardening characteristics. To satisfy these requirements, a pair of channels, MC10x21.9 with $\frac{3}{4}$ -in. back-to-back separation, can be used for the brace. The bolts are designed for the full 306-kip force at their plastic design stress levels. In addition, bearing stresses on the bolts were also checked at the normal plastic design force of 204 kips to assure that they did not exceed the yield stress, as suggested earlier. Eight $1\frac{1}{4}$ -in. A325 bolts are required for this connection. A fabricated tee-section was used for the gusset plate.

The beam-to-column connection is shown in Fig. 5. It is a moment resisting connection with the beam flanges connected to the column by full penetration welds. The beam web is connected to the column by means of an erection plate with three $1\frac{1}{4}$ -in. A325 friction-type bolts, and, in addition, a small fillet weld is specified along the erection plate in order to develop the full 102-kip shear capacity of the beam.

The preliminary design is now complete. However, as noted earlier, at least an additional iteration of the plastic design is necessary. After being satisfied with the plastic design, the structure should be checked using linear elastic analysis. The latter analysis may require further revisions of the structure. In making these, the basic design principles for this structural system must be kept in mind. These may be summarized as follows:

1. The beam must be designed as the weak link which yields in shear.

2. The brace must be conservatively designed as a compression member, to avoid buckling due to strain hardening or variations in yield strength of the beam.
3. The beam-to-column connection should be designed to be a moment-resisting one capable of developing the full shear capacity of the beam.

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REFERENCES

1. Roeder, C. W. and E. P. Popov Inelastic Behavior of Eccentrically Braced Steel Frames Under Cyclic Loadings *EERC Report 77-18, University of California, Berkeley, Aug. 1977.*
2. Roeder, C. W. and E. P. Popov Eccentrically Braced Steel Frames for Earthquakes *Journal of Structural Division, ASCE, Vol. 104, No. ST3, March 1978.*
3. Roeder, C. W. and E. P. Popov Cyclic Shear Yielding of Wide Flange Beams *Journal of Engineering Mechanics, ASCE, Vol. 104, No. EM4, Aug. 1978.*
4. Manual of Steel Construction *Seventh Edition, AISC, New York, 1970.*
5. Horne, M. R. A Moment Distribution Method for the Analysis and Design of Structures by the Plastic Theory *Proceedings of Institute of Civil Engineers, Vol. 3, No. 1, April 1954.*
6. Recommended Lateral Force Requirements and Commentary *Seismology Committee, Structural Engineers Association of California, 1973.*
7. Plastic Design in Steel, A Guide and Commentary *ASCE Manual 41, ASCE, New York, 1971.*
8. Lay, M. G. Flange Local Buckling in Wide-Flange Shapes *Journal of the Structural Division, ASCE, Vol. 91, No. ST6, Dec. 1965.*