

Eccentrically Braced Frames: U.S. Practice

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SUMMARY

Current status of research and U.S. codes on Eccentrically Braced Frames (EBFs) is briefly discussed. Excerpts from the just completed tentative American Institute of Steel Construction (AISC) Specification pertaining to EBFs are given in detail. Comments on these provisions attempt to interpret them in the light of research and design practice. Based on these provisions a 6-story building employing EBFs in end walls is analyzed for seismic response under the action of several severe earthquakes. For these time-history analyses it is assumed that steel behaves as an elasto-plastic material with strain hardening. The behavior of the frames is found to be satisfactory. Several recommended details for EBFs are illustrated.

INTRODUCTION

EBFs can be considered a hybrid structural system, combining the stiffness of conventional concentrically braced frames with the ductility and energy dissipation capacity of conventional Moment Resisting Frames (MRFs). Common and recommended arrangements of EBFs are illustrated in Fig. 1. The preferred type is shown in Fig. 1(b) where potential problems with moment resisting beam-to-column connections were eliminated. The distinguishing feature of an EBF is that at least one end of every brace is connected so as to isolate a beam segment called a *link*. Link lengths are identified by the letter *e* in the figure. In a well designed EBF, inelastic activity under severe cyclic loading is restricted primarily to the links. These links must be capable of sustaining large inelastic deformations without loss of strength.

It is convenient to examine the extent and the nature of inelastic deformation by constructing an energy dissipating mechanism for a frame as shown in Fig. 2(b). (In plastic-static analyses such mechanisms are called collapse mechanisms, an inappropriate terminology for seismic design). From this diagram it can be noted that for the same story sway, a link experiences a substantially larger demand than the “links” in a MRF, Fig. 2(a). It can be noted that EBFs with the braces arranged as in Fig. 1(c) for the same θ require only one half the link rotation of that shown in Fig. 2(b).

During the early stages of research on EBFs, the links were often placed at both ends of a brace as shown in Fig. 3. A detailed study of such mechanisms showed¹ that typically only one correctly placed link at one end of a brace dissipates most of the energy, whereas the other contributes very little to energy dissipation. Therefore, unless dictated by the architectural requirements, it is less expensive to make concentric or nearly concentric brace connections at one end of braces as in Fig. 1.

The development of the EBF system for resisting seismic loads required a considerable amount of experimental and analytical work. The principal results of this research carried out at the University of California, Berkeley, are summarized below.

EXPERIMENTAL AND ANALYTICAL RESEARCH ON EBFs

After experimentally verifying the concept of eccentric bracing for seismic applications on small frames in 1977² and 1980,³ the next phase of research was directed to determining cyclic behavior of individual *short* links.^{4,5} This

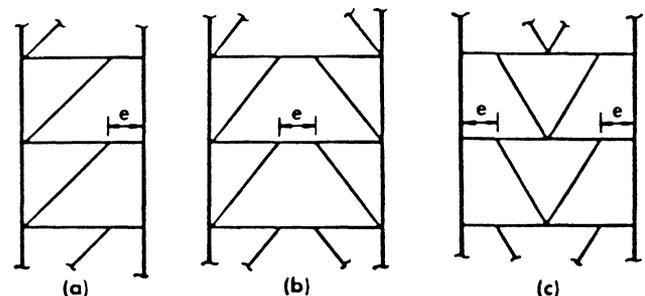


Fig. 1. Typical EBFs

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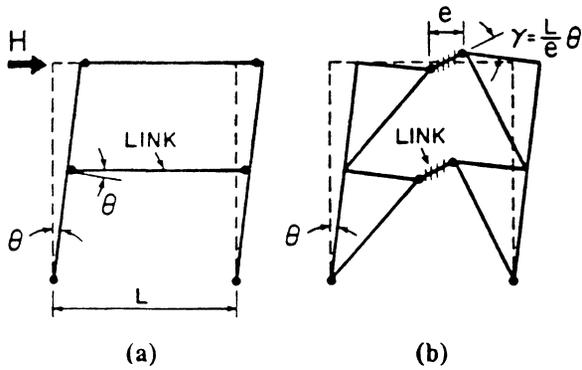


Fig. 2. Comparison of energy dissipating mechanisms for (a) MRF and (b) EBF

effort culminated in accurate criteria for web link buckling control under cyclic loads.^{1,6} An experimental study of cyclic behavior of short links in EBFs with a composite floor followed.^{7,8,9} This study, conducted on two-thirds scale models based on the full-size EBF tested at Tsukuba,¹⁰ is described below.

In the simplified model opposing vertical forces at link ends were used to simulate the brace effects on the beam. This gave good simulation for the composite link, avoiding the problems that may arise outside the link. Six different links were tested in this manner. Three of these links corresponded to links framing into columns as in Fig. 1(a); three other links corresponded to the interior links as in Fig. 1(b). These experiments clearly demonstrated that during the first application of large opposing vertical forces, the link strengths were significantly larger than those for the bare steel links. However, on the application of additional severe cycles, as local floor damage develops, the shear strengths of composite links became comparable to strain-hardened bare steel links. As shown in Fig. 4, this held true for both the interior and the exterior links. Therefore, in the computer simulation of the link behavior under severe cyclic loads, no distinction was

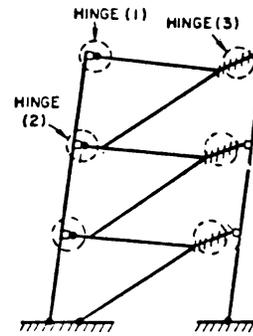


Fig. 3. Hinges 1 and 2 dissipate less energy than hinges 3.

made between the composite and the bare steel links.⁷ A computer program based on these and earlier findings was developed for the analysis of EBFs.^{7,12}

The effectiveness of a composite deck on restraining the lateral-torsional buckling of links was also investigated in these experiments [Fig. 5(a)]. The link twist angle ϕ for a link laterally braced at both ends, when unloaded, essentially returned to its initial position, as shown in Fig. 5(b). However, for a laterally unbraced link, as shown in Fig. 5(c), the link twist angle ϕ continued to increase during consecutive cycles, and the test was discontinued for fear of damaging test equipment. This link behavior was corroborated by a recent series of tests.¹³

The experiments described above were largely performed on individual links with the exception of the Tsukuba tests¹⁰ and the subsequent tests on a one-third scale model on the Berkeley shaking table.¹⁴ In the recent series of tests noted above,¹³ 14 complete two-third scale subassemblies, such as those shown in Fig. 6, were investigated. These experiments provided a large data base on moderate length links, and on the behavior of the beam outside the link. A number of different brace-to-beam connections were also studied.

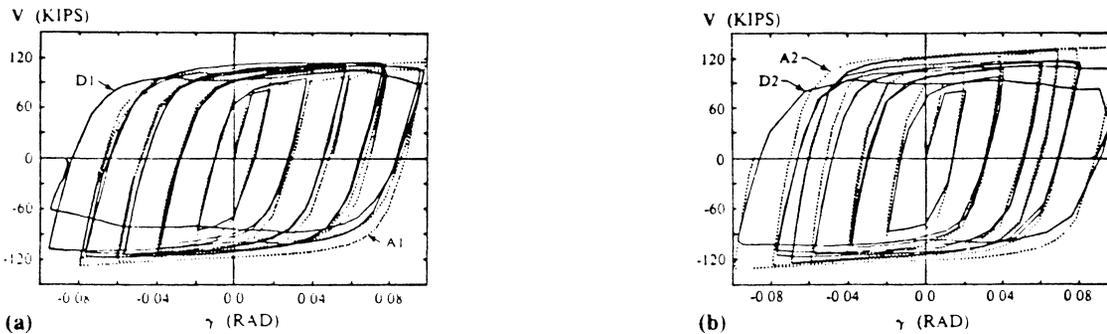


Fig. 4. Comparison of hysteretic loops for shear capacity vs. rotation angle for composite (dotted) and bare (solid lines) links. (a) Interior and (b) exterior link. Specimens D1 and D2 are bare links; A1 and A2 are composite links.

These experiments showed that links of moderate length connected to column flanges behaved reasonably well as shown in Fig. 7(a), although the maximum inelastic rotation, γ_p , was only half of that provided by a well stiffened short link.⁶ Long links in similar applications behaved poorly, Fig. 7(b); most of the inelastic action is concentrated at the ends, placing large demands on the flexural capacity of a connection.

Poor behavior of subassemblages was also observed where a brace forms a small angle on the order of 30° or less with the floor beam (Fig. 8). The beam outside the link and the brace in such arrangements provide inadequate stiffness for forming a hinge in the link.

These experiments also have shown that large forces can develop in lateral bracing (Fig. 9). This is particularly likely to occur when beam flanges distort, giving rise to lateral-torsional buckling at the link ends.

The last series of tests also provided considerable insight into the behavior of various brace to beam connections. A full report on this research is in preparation.¹³

The above experimental and analytical research provided the bases for practical applications, and together served in the development of code provisions for EBFs.

CODE DEVELOPMENT

Almost immediately upon publication of the first research report² on EBFs, several important applications of this concept were made for major buildings. It is interesting to note that the reasons for adopting this system of framing were entirely different in each of the three cases. In one, a forty-seven story building, it was determined

that the originally designed frame was too flexible under high winds. In a sense, EBFs were adopted as a retrofit for this project. On another project a municipality was unwilling to accept concentric bracing for a nineteen story building for the lack of ductility in such systems. In the third case, for a four story hospital, it was determined that EBFs provided a rational solution for poor foundation conditions. By means of links, the column loads could be brought within acceptable bounds. These projects were scattered throughout California.

The versatility of EBFs shown by the above applications provided the driving motivation for further research in this area. Because of the research noted in the previous section, and considerable competence acquired by the professionals in the design of EBFs, the need for developing a code for such systems became clearly evident. Therefore the Structural Engineers Association of California (SEAOC) embarked on the development of recommended seismic provisions for EBFs.

After several years of work, the formidable task of updating the recommended seismic provisions for buildings was completed by SEAOC in January 1988.¹⁵ These provisions contain a section on EBFs. With minor revisions these recommendations were accepted for inclusion in the 1988 Uniform Building Code (UBC).¹⁶ Paralleling this development, the Building Seismic Safety Council (BSSC) included in its 1985 edition of the National Earthquake Hazards Reduction Program (NEHRP) Recommended Provisions for the Development of Seismic Regulations for New Buildings, tentative provisions for the design of EBFs.¹⁷ This document is an update on the Applied Tech-

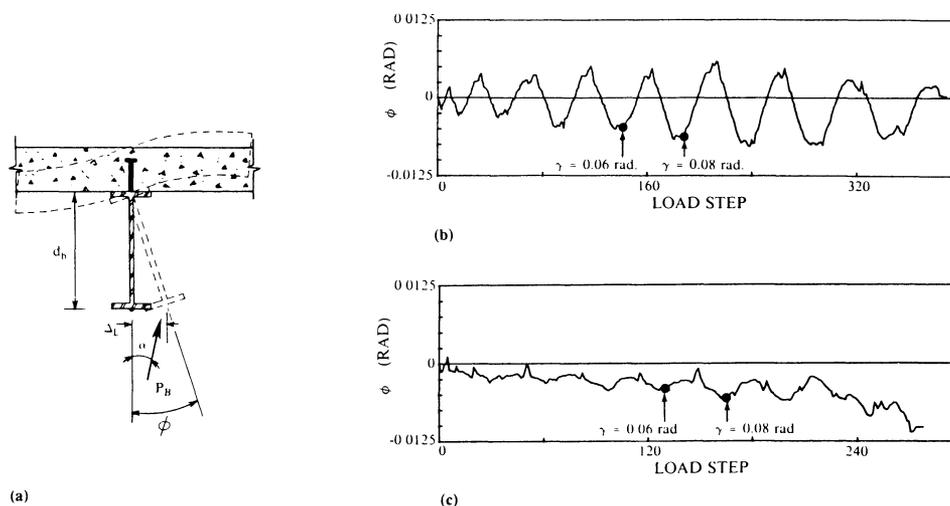


Fig. 5. (a) Lateral-torsional buckling of links (b) cyclic twist of bare link laterally braced at both ends, (c) progressively increasing twist of composite link laterally braced at one end.

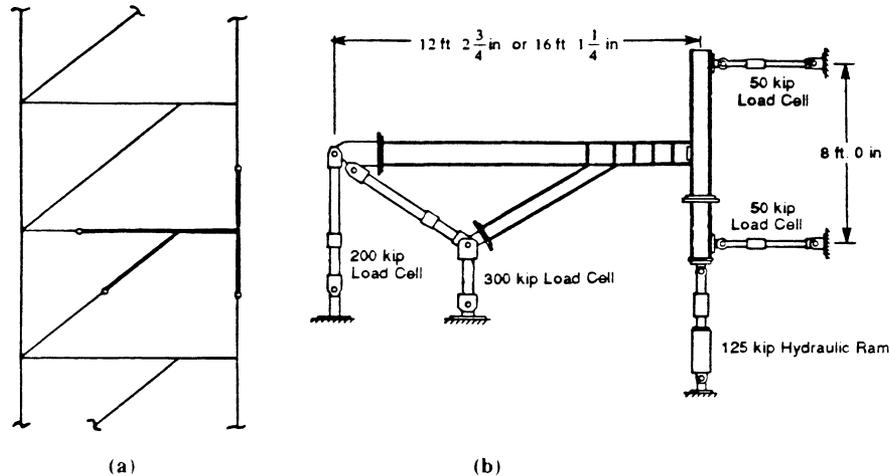


Fig. 6. (a) Location of subassemblage in frame, and (b) experimental set-up for EBF subassemblage.

nology Council, report ATC3-06.¹⁸ In the forthcoming BSSC publication EBF design recommendations are no longer tentative. Based on the same database, these recommendations are comparable to those given by SEAOC and UBC. The American Institute of Steel Construction (AISC), as a supplement to its new Load and Resistance Factor Design (LRFD) Specification¹⁹ is in the process of finalizing provisions for EBFs. These provisions, taking advantage of the latest findings, represent the most up-to-date and comprehensive requirements for EBFs, and although tentative, are followed in the remainder of this paper.

It is to be noted that the SEAOC and UBC provisions, which are essentially identical, are based on the allowable stress design approach, whereas the BSSC and AISC provisions are based on the strength design approach.

EXCERPTS FROM AISC TENTATIVE PROVISIONS FOR EBFs (Reproduced with permission of AISC)

EQ3. DEFINITIONS (Only those definitions directly applicable to EBFs are listed)

Eccentrically Braced Frames (EBF). That form of a diagonally braced frame where at least one end of each brace frames into a beam a short distance from a column-to-girder connection or from another beam-to-brace connection. The EBF shall meet the requirements of Section EQ9.

Link The segment of beam located between the end of a diagonal brace and a column or between the ends of two diagonal braces. The length of the link is defined

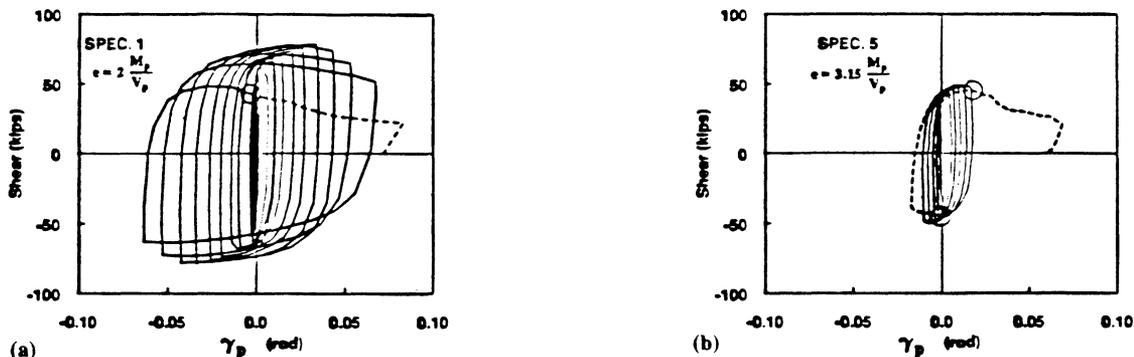


Fig. 7. Hysteretic loops of link shear vs. inelastic (plastic) nominal shear angle for (a) moderately long link, and (b) long link.

as the clear distance between the ends of two diagonal braces or between the diagonal brace and the column face.

Link Rotation Angle The angle between the beam outside of the link and the link that occurs at a total story drift of E'/E times the elastic drift at the prescribed design forces. The rotation angle is computed assuming the EBF bay as a rigid-ideally plastic mechanism.

Prescribed Design Forces. Forces derived from the nominal earthquake loads per Section EQ2 multiplied by the load factors specified in Section A4.1 of the AISC LRFD Specification.

Link Design Shear Strength. The lesser of ϕV_n or $2\phi M_p/e$, where $V_n = 0.60F_y dt_w$, e is the link length, and $\phi = 0.9$; except as modified in sect. EQ9.2e.

Link Intermediate Web Stiffeners. Vertical web stiffeners placed within the link.

EQ9. REQUIREMENTS FOR ECCENTRICALLY BRACED FRAMES

1. Scope:

Eccentrically braced frames shall be designed so that under earthquake loading, yielding will occur primarily in the links. The diagonal braces, the columns, and the beam segments outside of the links shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded and strain hardened links.

2. Links:

- Links shall comply with the width-thickness ratios of Table A-EQ7-1. The specified minimum yield strength of steel used for links shall not exceed $F_y = 50$ ksi.
- The shear force in the link produced by the prescribed design forces shall not exceed the design shear strength of the link.
- The web of a link shall be single thickness without

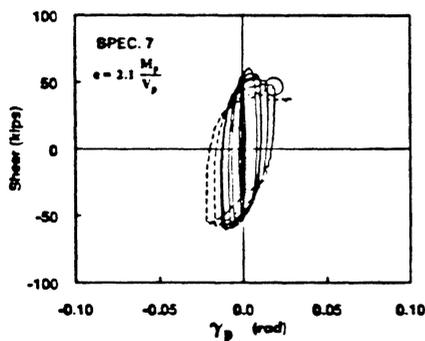


Fig. 8. Hysteretic loops of link shear vs. inelastic rotation for moderately long link in subassembly with brace forming small angle with beam.

- doubler plate reinforcement and without openings.
- If the required axial strength P_u in the link at the prescribed design forces is equal to or less than $0.15P_y$, where $P_y = AF_y$, the effect of axial force on the link design shear strength may be neglected.
- If the required axial strength P_u in the link at the prescribed design forces exceeds $0.15P_y$, the following additional requirements shall be met:
 - The link design shear strength shall be computed as the lesser of ϕV_{na} or $2\phi M_{pa}/e$ where $V_{na} = V_n [1 - (P_u/P_y)^2]^{1/2}$, $M_{pa} = 1.18 M_p [1 - (P_u/P_y)]$, and $\phi = 0.9$.
 - The length of the link shall not exceed:

$$[1.15 - 0.5\rho(A_w/A)] 1.6M_p/V_n \text{ for } \rho(A_w/A) \geq 0.3$$

$$1.6M_p/V_n \text{ for } \rho(A_w/A) < 0.3$$

where $A_w = dt_w$

$\rho = P_u/V_u$

$P_u =$ required axial strength of the link

$V_u =$ required shear strength of the link at the prescribed design forces.

- The link rotation angle shall not exceed the following values (except as noted in Sect. EQ9.2g):
 - 0.08 rad. for links of length $1.6M_p/V_n$ or less
 - 0.02 rad. for links of length $2.6M_p/V_n$ or greater.
 - Interpolation shall be used for links of length between $1.6M_p/V_n$ and $2.6M_p/V_n$.
- Where a link is connected to a column, these additional requirements shall be met:
 - Links connected to columns shall not exceed the length of $1.6M_p/V_n$. This requirement may be waived if it can be demonstrated that the link to column connection is adequate to develop the required inelastic rotation of the link.
 - The link flanges shall have full penetration welds to the column. The connection of the

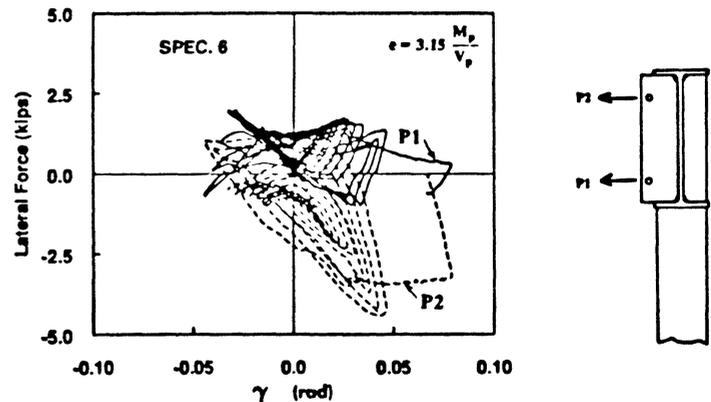


Fig. 9. Forces at link end lateral support in advanced stages of loading of a W12 x 16 link.

link web to the column shall be welded to have a design strength to develop the design strength of the link web.

- (3) Where the link is connected to the column web, the link flanges shall have full penetration welds to the connection plates and the web connection shall be welded to have a design strength to develop the design strength of the link web. The link rotation angle shall not exceed 0.015 rad. for any link length.

3. Link Stiffeners:

- a. Full depth web stiffeners shall be provided on both sides of the beam web at the diagonal brace ends of the link. These stiffeners shall have a combined width not less than $(b_f - 2t_w)$ and a thickness not less than $0.75 t_w$ nor $\frac{3}{8}$ inches, whichever is larger, where b_f and t_w are the width of the link flange and link web thickness, respectively.
- b. Links shall be provided with intermediate web stiffeners as follows:
 - (1) Links of length $1.6M_p/V_n$ or less shall be provided with intermediate web stiffeners spaced at intervals not exceeding $32t_w - d/5$ for a link rotation angle of 0.08 rad. or $56t_w - d/5$ for link rotation angles of 0.03 rad. or less. Interpolation shall be used for values between 0.03 and 0.08 rad.
 - (2) Links of length greater than $2.6M_p/V_n$ and less than $5M_p/V_n$ shall be provided with intermediate web stiffeners placed at a distance of b_f from each end of the link, where b_f is the link flange width.
 - (3) Links of length between $1.6M_p/V_n$ and $2.6M_p/V_n$ shall be provided with intermediate web stiffeners meeting the requirements of both sections (1) and (2) above.
 - (4) No intermediate web stiffeners are required in links of length greater than $5M_p/V_n$.
 - (5) Intermediate link web stiffeners shall be full depth. For beams less than 24 in. in depth, stiffeners are required on only one side of the beam web. The thickness of one-sided stiffeners shall not be less than t_w nor $\frac{3}{8}$ inch, whichever is larger, and the width shall not be less than $(b_f/2) - t_w$. For beams 24 in. in depth or greater, intermediate stiffeners are required on both sides of the web.
- c. Fillet welds connecting the link web stiffeners to the beam web shall develop a force of at least $A_{st} F_y$, and those connecting the stiffener to the flanges shall develop a force of at least $A_{st} F_y/4$, where $A_{st} = bt$ of stiffener, b = width of stiffener plate, and t = thickness of stiffener plate.
- d. Lateral supports shall be provided at both the top and bottom flanges of the link at the ends of the

links. End lateral supports of links shall have a design strength of at least 4 percent of the link flange nominal strength computed as $F_y b_f t_f$.

4. Diagonal Brace and Beam Outside Link:

- a. The nominal strength of each diagonal brace shall be adequate to resist the forces generated by at least 1.5 times the design shear strength of the link.
- b. The sum of the nominal flexural strength of the diagonal brace and of the beam segment outside of the link shall exceed the link end moment occurring at 1.5 times the design shear strength of the link. The nominal flexural strength of these members shall be determined using interaction equations for combined axial force and bending moment, using the axial force in the member generated by 1.5 times the design shear strength of the link.
- c. Diagonal brace to link connections shall develop the nominal strength of the diagonal brace and transfer this force to the beam. No part of the diagonal brace to link connection shall extend over the link length. If the diagonal brace is assumed to resist a portion of the link end moment for satisfying Sect. EQ9.4b then the diagonal brace to link connection shall be designed as fully restrained (Type FR).
- d. The beam outside of the link shall be provided with sufficient lateral support to maintain the stability of the beam under the forces generated by at least 1.5 times the design shear strength of the link. Lateral supports shall be provided at both the top and bottom flanges of the beam and shall have a strength to resist at least 1.5 percent of the beam flange nominal strength computed as $F_y b_f t_f$.

5. Beam to Column Connections:

Beam to column connections away from links may be designed as partially restrained, Type PR, (simple) connections. The connection shall have a strength to resist rotation about the longitudinal axis of the beam based on two equal and opposite rotation forces of at least $0.015F_y b_f t_f$ acting laterally on the beam flanges.

COMMENTS ON AISC EBF TENTATIVE PROVISIONS DESIGN BASIS AND LOADS

The AISC seismic provisions are intended for use with the AISC LRFD Specification¹⁹ and much of the nomenclature used in the seismic provisions are defined in the LRFD Specification. Unlike SEAOC, UBC or BSSC, neither the LRFD Specification nor the AISC seismic provisions specify design earthquake loads. Rather, these documents specify that the nominal design loads, including earthquake load E , load factors and load combinations be taken from the applicable local code. In the absence of

such a code, loads and load combinations are referred to the American National Standard, *Minimum Design Loads for Buildings and Other Structures*, ANSI A58.1.²⁰ The ANSI load factors and combinations for earthquake loads are: $1.2D + 1.5E + (.5 L \text{ or } .2S)$ and $.9D - (1.3 W \text{ or } 1.5 E)$, where D , L , E , W and S are dead, live, earthquake, wind and snow loads, respectively. ANSI A58.1 earthquake loads are similar to those specified in the 1985 UBC, using the familiar formula for total base shear: $V = ZIKCSW$, where the various factors are as defined in ANSI A58.1. For EBFs, the AISC seismic provisions specify $K = .8$, unless used as part of a dual system, for which $K = .67$. The 1988 UBC uses a completely different formula for design base shear. However, for EBFs, the nominal unfactored earthquake loads from ANSI A58.1 are comparable to those from the 1988 UBC. In the 1988 UBC, the earthquake loads are not factored, and the design is based on the conventional allowable stress design approach. In the AISC seismic provisions, earthquake loads are factored by 1.5 and the design is based on the LRFD strength approach.

EQ3. DEFINITIONS – LINK ROTATION ANGLE

The relationship between link rotation angle γ and story drift θ can be approximated from a rigid plastic mechanism as noted earlier. For example, for the EBF type illustrated in Fig. 1(b), this relationship is $\gamma = (L/e)\theta$. The value of θ used in this expression is computed as $(E'/E)\theta_e$, where θ_e is the elastic story drift occurring under factored loads. The quantity E' is defined as $3ZICSW$, where the factors in this equation are as defined in ANSI A58.1. If E is based on ANSI A58.1, then the values of E'/E are 3.75 for an EBF or 4.5 for a dual EBF-MRF system. In the 1988 UBC, the multiplier on θ_e for computing link rotation is $3R_w/8$, which is again 3.75 for an EBF or 4.5 for a dual EBF-MRF system. However, in the 1988 UBC, θ_e is taken as the elastic story drift occurring at unfactored earthquake loads. Thus, for the same frame, the AISC procedure will result in a larger computed link rotation angle as compared with the 1988 UBC.

EQ3. DEFINITIONS – LINK DESIGN SHEAR STRENGTH

The formulas given in this section provide the shear force required to produce significant yielding of the link. These formulas are based on perfect plasticity and assume no shear-moment interaction, a reasonable assumption for links.^{1,6,22} Significant yielding of short links occurs when the shear force reaches the fully plastic shear capacity of the W section, which has generally been taken as $V_p = .55F_y d t_w$. The value of $V_n = .6F_y d t_w$ was adopted to be consistent with the LRFD Specification. Using the specified resistance factor of $\phi = 0.9$, the value of ϕV_n is nearly identical to the value of V_p . Significant yielding of long

links does not occur until flexural hinges form at both ends of the link, corresponding to a shear force of $2M_p/e$.

EQ9.1

The scope statement represents the overall design philosophy for EBFs. That is, under severe earthquake loading, yielding must be restricted to the links, which are designed and detailed to be the most ductile element of the frame. Accordingly, all other frame members must be “stronger” than the link. This design philosophy is implemented through the application of “Capacity Design” concepts. With this approach, the links of an EBF are sized according to the code specified lateral forces. The braces, columns, and beam segments outside of the links, however, are not designed for code specified lateral forces, but rather for the maximum forces generated by the fully yielded and strain hardened links. That is, all other frame members are designed for the *capacity* of the links to assure that the links are the weakest element of the frame, thereby forcing the yielding to occur in the links and preventing non-ductile failure modes such as brace buckling.

EQ9.2.a

The limiting width-thickness ratios specified in Table A-EQ7-1 of the AISC seismic provisions are taken from Table C-B5.1 of the LRFD Specification, with minor modifications.

EQ9.2.b

Links are sized so that their design shear strength (as defined above) exceeds the shear force produced in the link by the factored earthquake loads. When links are attached to columns, an elastic frame analysis will typically show large bending moments at the column end of the link. The links should not, however, be sized on the basis of these large elastic moments. The initially large elastic moment redistributes to the other end of the link, and significant yielding of the link does not occur until the shear force reaches the design shear strength. Links should be sized on the basis of their design shear strength and not on the basis of elastic bending moments.

EQ9.2.c

Tests on links with doubler plates,² as well as tests on the related problem of column panel zones,²¹ have shown that doubler plates do not carry their proportional share of shear based on plate thickness under large inelastic shear deformations.

EQ9.2.e

Large axial forces in links affect both the link strength and inelastic rotation capacity. Accordingly, EBF framing arrangements that minimize axial forces in the links are preferred. In cases where significant axial forces are trans-

ferred through the links, the reduction in strength can be estimated by the given shear-axial and moment-axial interaction formulas. Further, link length should be limited as indicated in EQ9.2.e(2) to minimize the loss of inelastic rotation capacity. These equations are derived in Ref. 22. Note that these provisions effectively prohibit the use of link lengths exceeding $1.6M_p/V_n$ for links with axial forces exceeding $.15P_y$.

EQ9.2.f

Numerous tests on shear yielding links of length $e \leq 1.6M_p/V_n$ show that with proper stiffening, these links can sustain cyclic inelastic rotations of $\gamma = \pm .10$ rad. The 1988 UBC limits link rotation for $e \leq 1.6M_p/V_p$ to $\gamma = \pm .06$ rad. This limit was intended to provide some design conservatism and to account for uncertainties in predicting actual link rotation demand. In AISC, the rotation limit for short links has been tentatively set at $\gamma = \pm .08$ rad. This limit reflects the difference in the link rotation calculation procedure between UBC and AISC noted earlier. Further, dynamic analyses of an EBF designed according to the draft AISC seismic provisions using an allowable link rotation of $.08$ rad demonstrate that the actual link rotation demands under real earthquake loads are at an acceptable level. These analyses are described later in this paper.

Links of length $e \geq 2.6M_p/V_n$ yield primarily in flexure and provide significantly less inelastic rotation than short shear yielding links. Recently completed tests on long links¹³ indicate that $\gamma = \pm .02$ rad is a reasonable estimate of inelastic rotation capacity of long, flexural yielding links not attached to columns. Within the length range $1.6M_p/V_n \leq e \leq 2.6M_p/V_n$, the rotation limits reflect the transition from shear yielding to flexural yielding as the factor controlling inelastic link behavior.

EQ9.2.g

Recent tests on EBF subassemblages with long links attached to columns¹³ showed failure at relatively low inelastic rotations by fracture of the link flange near the flange to column weld. These premature failures resulted from the very highly concentrated bending strains developed at the ends of long links. To prevent this very undesirable failure mode, the use of links of length $e \geq 1.6M_p/V_n$ attached to columns is prohibited. Further, for any link connected to the column web, the link rotation angle is limited to $\gamma = \pm 0.15$ rad, reflecting the uncertain reliability of link to column web connections.

EQ9.3.b

Equally spaced web stiffeners for links of length $e \leq 1.6M_p/V_n$ are intended to delay the onset of inelastic shear buckling of the web. The specified stiffener spacing is based on Ref. 6, with slight modifications to simplify de-

sign calculations. For links of length $e \geq 2.6M_p/V_n$, inelastic shear buckling of the web is not anticipated and therefore web stiffening is not required. Rather, severe flange buckling and/or lateral torsional buckling are the dominant failure modes. Locating a stiffener at about b_f from each end of the link is intended to delay flange buckling and to torsionally stiffen the link. In the length range $1.6M_p/V_n \leq e \leq 2.6M_p/V_n$, both shear and flexural failure modes are anticipated, and therefore both web and flange stiffening is required. The behavior of very long links of length $e \geq 5M_p/V_n$ is expected to be similar to that of beams in moment resisting frames, and therefore intermediate link stiffeners are not required.

EQ9.4

This section contains the "capacity design" requirements for the diagonal brace and for the beam segment outside of the link. These provisions are intended to permit the full development of link strength and ductility without the occurrence of significant yielding or instability in the beam or brace. Accordingly, these members must be designed for the maximum forces that can be generated by the link, accounting for all possible sources of link overstrength. Link overstrength can be attributed primarily to i) strain hardening, ii) effects of composite floor systems, and iii) the actual yield strength of steel exceeding the nominal specified yield strength. For design purposes, the AISC seismic provisions specify ultimate link strength to be taken as 1.5 times the design strength. Thus, the ultimate shear force is taken as 1.5 times the design shear force as defined earlier. The corresponding ultimate link end moments can then be computed from statics (link length times link shear equals the sum of the link end moments), and by assuming end moments are equal when the link achieves its ultimate strength. For links of length $e \leq 1.3M_p/V_p$ attached to columns, experiments²² have shown that end moment equalization does not occur. For this case, ultimate link end moments can be taken as M_p at the column face and $1.5V_p e - M_p$ at the brace end of the link.²³

The ultimate link shear force will generate a large axial force in the brace. The horizontal component of the brace force, in turn, must be balanced by a large axial force in the beam. For many EBF configurations, very large axial forces, often in excess of $.5P_y$, are generated in the beam when link ultimate strength is developed. Axial force levels in the beam can generally be limited by avoiding very shallow braces, say with a brace-beam angle less than about 35 degrees.

At the brace end of the link, the ultimate link end moment will be transferred to the beam and to the brace. If no significant yielding or instability occurs in the beam or brace, the distribution of link end moment to the beam and brace can be estimated from an elastic analysis. Such an analysis will typically show that the beam carries 80 to

95 percent of the link end moment. However, since the beam also typically carries very large axial forces, yielding of the beam may occur well before link strength is fully developed. This early yielding, due to moment-axial force interaction in the beam, may result in a redistribution of bending moment to the brace.

When checking the requirements of EQ9.4, both the beam and the brace must be treated as beam-columns in strength and stability calculations. Unlike concentrically braced frames, the brace of an EBF may carry significant bending moments, particularly if an inelastic redistribution of moment occurs between the beam and the brace. These moments must be considered when checking EQ9.4.a. For the beam, adequate lateral bracing must be provided to maintain its stability under the combination of large axial force and large bending moment, as required in EQ9.4.d. For certain EBF configurations, limited yielding of the beam outside of the link may be unavoidable. However, the combined flexural strength of the beam and brace must exceed the ultimate link end moment, to assure that the link end moment can be fully developed. This is the requirement stated in EQ9.4.b. Further, per EQ9.4.c, if the brace is considered to carry a portion of the link end moment, then a rigid brace to beam connection is required.

For EBF geometries involving very flat braces and/or long links, satisfying EQ9.4 may require very heavy braces or cover plates on the beams. Such measures can often be avoided through careful choice of frame geometry at the preliminary design stages. EBFs with relatively steep braces, e.g., brace-beam angles in the range of 40 to 50 degrees, combined with short links are preferable for avoiding design problems with the beam and brace.

COLUMNS IN EBFs

Following the capacity design principles noted in the Scope statement of EQ9.1., the strength and stability of the columns must be maintained under the maximum forces generated by the links. One approach for satisfying this requirement is to assume that all links above a column achieve their ultimate strengths (1.5 times the design strength) simultaneously. This may be reasonable procedure for low to medium rise buildings, but is probably too conservative for taller buildings, since all of the links are unlikely to yield simultaneously or uniformly over the height of the frame. Additional research, particularly additional dynamic analyses of EBFs subject to real earthquake records, is needed before detailed capacity design procedures can be established for columns in EBFs. A useful model for capacity design of columns is the procedure presented in the commentary to the New Zealand Standard on the Design of Concrete Structures²⁴ for columns in ductile concrete frames. Development of a similar procedure is needed for columns in EBFs.

Even through the AISC seismic provisions do not in-

clude specific requirements for EBF columns, there is a provision covering columns in all types of steel frames. With certain limitations and exclusions, this provision states that columns must satisfy the following equations:

$$1.2D + .5L + 1.0E' \leq P_n \text{ for axial compression}$$

$$.9D + 1.0E' \leq P_n \text{ for axial tension}$$

where D and L are dead and live load, E' is the amplified earthquake load defined earlier, and P_n is the nominal axial strength of the column in tension or compression as defined in the LRFD Specification. These equations recognize that columns will experience larger forces than those produced by the basic code specified lateral loads. However, sufficient analyses have not yet been conducted to determine if satisfying these equations will assure frame stability under real earthquake loads.

For EBFs, a more rational sizing of columns can likely be achieved by applying capacity design principles, combined with judgement on the probable distribution of link yielding over the height of the frame. Judgement in column design in EBFs can be guided by the available dynamic analyses of EBFs^{11,14} as well as by studying the New Zealand approach for concrete frames noted above.

An additional concern in column design is when a single column forms part of two lateral resisting frames. These "orthogonal effects" are covered by both the 1988 UBC and by ANSI A58.1 by requiring that such columns be designed for 100 percent of the specified earthquake forces in one direction plus 30 percent of the specified earthquake forces in the orthogonal direction.

SEISMIC DESIGN OF 6-STORY EBF

A small six-story office building is considered as a design example using AISC tentative seismic provisions for EBFs. The elevation of one of the two end frames for this rectangular building is shown in Fig. 10. It is assumed that three 24 ft. moment resisting bays are connected to each of the columns of the end EBFs. Therefore, using column centerline dimensions, the building is 72 by 70 ft in plan. The webs of the corner columns are in the plane of the EBFs. The webs of the inside columns are perpendicular to the plane of the end EBFs.

The lateral force acting on each of the end frames is calculated using the ANSI A58.1 formula for the base shear V defining the LRFD earthquake load E , i.e.,

$$E = V = ZIKCSW$$

It is assumed that this building is located in seismic zone 4, and the EBFs are a part of a dual system. Hence $Z = 1$, and $K = 0.67$. In the above equation the response factor $C = 1/(15\sqrt{T}) = 0.0996$, where $T = 0.05h_n/\sqrt{D_s} = 0.448$ sec., h_n is the height of the frame, and D_s is its width. Further, assume the importance factor $I = 1$, and the soil factor $S = 1.2$, and the half-weight of the building $W = 2,270$

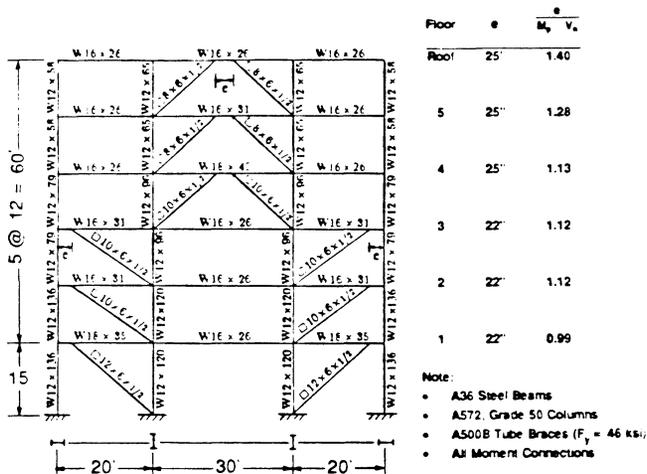


Fig. 10. Elevation of six story EBF

kips. Hence $E = V = 182$ kips. Therefore, since according to AISC LRFD code the load factor for earthquake loads is 1.5, the base shear for each of the two EBFs is $1.5 \times 182 = 273$ kips.

For the preliminary design the above base shear was distributed in a triangular manner as shown in Fig. 11. For the dual system considered here, Fig. 10, it was assumed that in the upper three stories 80% of the lateral load is carried by the braced bay, and 20% by the two adjoining moment resisting bays. All lateral load was assigned to the outside two braced bays in the lower three stories.

Using the above lateral force distribution for the braced frames, the link forces were determined by statics applying the procedure shown in Fig. 11. In this approach the member moments at the cut sections are neglected. The links were then selected to satisfy the code requirements for V_n given in EQ3. Preliminary link lengths were then assigned bearing in mind the link rotation angle criteria given in EQ9.2.f. As noted previously the use of short

links is preferable both for performance and economy. In this design rotation angles $\gamma = .08$ were desired.

All remaining members were sized using the "capacity approach". This means that since the links strain-harden under cyclic loading, the braces are designed for 1.5 times the force that is generated by the link shear ϕV_n . The beams outside the links developing axial forces and bending moments associated with the increased brace forces were checked for code compliance. If it is found that a beam outside the link yields, the beam-brace joint must be moment resisting. Moreover, enough capacity must be developed by the joint such that the brace would not yield. Often this problem can be avoided with short links and steep braces. No such difficulties arose in this design.

The columns for the preliminary design were sized including dead, live, and earthquake loads; applying $1.5\phi V_n$ shear from the links, and including the 30% orthogonality effect in corner columns. Therefore, these columns were designed for bi-axial bending caused by the links and beams in the moment frames.

An elastic analysis for factored loads was then performed giving the elastic story drift θ_e . According to AISC tentative provisions this story drift must be multiplied by the ratio E'/E , where $E' = 3ZICSW$, for determining γ assuming an EBF bay as a rigid ideally-plastic mechanism. After carrying out this check the initially assumed link lengths were increased by 10%, and the necessary adjustments in member sizes were made.

The revised frame conformed to the AISC LRFD specifications including the limitation on the maximum link rotations of 0.08 rad. Inelastic time-history responses were obtained for this frame for several well known recorded earthquakes. The selected records were for the 1940 El Centro, 1966 Parkfield, 1952 Taft, 1978 Miyagi-Ken-Oki, and the 1985 Mexico City earthquakes. Of these quakes, the El Centro and the Taft earthquakes were also magnified to 0.50 g for additional computer runs. This was also done with the Miyagi earthquake after magnifying the available record by a factor of 1.5.

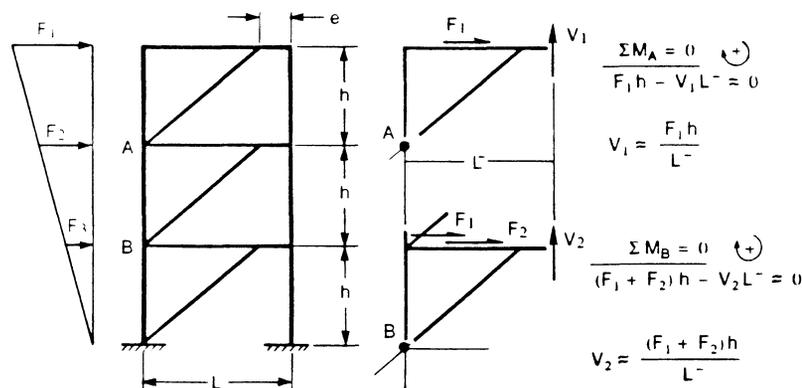


Fig. 11. Simplified Static Analysis of an EBF

Table 1. Maximum Link Deformation During EBF Response to Selected Earthquake Records.

Earthquake Record								
Floor	El Centro (0.33g)	El Centro (0.50g)	Parkfield (0.50g)	Miyagi (0.26g)	Miyagi (0.39g)	Taft (0.30g)	Taft (0.50g)	Mexico EW (0.17g)
[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]
1	0.048	0.081	0.105	0.062	0.133	0.029	0.077	0.014
2	0.035	0.070	0.129	0.060	0.107	0.024	0.069	0.011
3	0.015	0.043	0.105	0.046	0.069	0.016	0.042	0.007
4	0.010	0.013	0.031	0.012	0.015	0.003	0.009	0.002
5	0.012	0.028	0.034	0.031	0.048	0.005	0.022	0.002
R	0.002	0.003	0.002	0.002	0.004	0.002	0.003	0.001

The principal results of the inelastic dynamic analyses are assembled in the two tables. The maximum link deformations during EBF responses to the above earthquakes are displayed in Table 1. It can be observed from this table that the simple static procedure prescribed by the code based on rigid-ideally plastic mechanisms is remarkably good for the design of links for a large variety of severe earthquakes. Only for a near fault location of a building, exemplified by the Parkfield earthquake, do the rotations of the links exceed the desired limit, but since this occurs only during a single pulse, this condition is tolerable. The Miyagi earthquake per se presented no problem for link rotations. The behavior of a frame for a magnified Miyagi quake, however, indicates some difficulties. Therefore, for Japanese designs a somewhat more conservative criteria may be appropriate than in the U.S. The frame was taxed extremely little by the 1985 Mexico City earthquake designed to the AISC code. This may, however, be attributed to the three to four fold larger dominant period of the earthquake to that of the analysed frame.

It is instructive to examine the normalized link shear forces given in Table 2 for the same earthquake input data

as for Table 1. These results show that the braces must indeed be designed for at least 1.5 times the nominal shear strength of the links. For this reason, it may be prudent to be conservative in selecting the braces in the lower two of three stories of a building.

It is interesting to note that using the triangular lateral load distribution both tables show that the upper stories are overdesigned. For buildings of the height considered a uniform lateral load distribution may lead to a more balanced design. Further studies of the type presented in this paper are needed for taller buildings to achieve more economical design of EBFs.

EBF DETAILS

In any structure designed to resist seismic forces it is imperative to have joints with good ductile connections. For EBFs there are three kinds of joints that require special detailing. One of these is for connecting the braces to beams, another for connecting the links to a column, and the third, connecting the braces and beams to a column at a joint. All three joints are illustrated.

Table 2. Maximum Normalized Link Shear Force, V_{max}/V_p , During Response to Selected Earthquake Records.

Earthquake Record								
Floor	El Centro (0.33g)	El Centro (0.50g)	Parkfield (0.50g)	Miyagi (0.26g)	Miyagi (0.39g)	Taft (0.30g)	Taft (0.50g)	Mexico EW (0.17g)
[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]
1	1.51	1.55	1.58	1.55	1.64	1.53	1.51	1.40
2	1.45	1.52	1.59	1.53	1.59	1.39	1.53	1.21
3	1.33	1.45	1.52	1.40	1.49	1.31	1.47	1.11
4	1.16	1.24	1.40	1.20	1.28	1.03	1.21	0.94
5	1.24	1.39	1.42	1.37	1.49	1.07	1.34	0.98
R	0.82	1.02	0.98	0.84	1.04	0.74	1.03	0.53

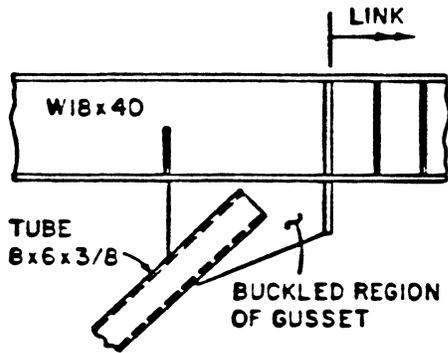


Fig. 12. Detail of failed brace connection.

The detail shown in Fig. 12 and used in Tsukuba full-size tests¹⁰ proved to be not entirely satisfactory. At extreme cyclic loads beyond the initially planned test, the gusset plate attaching the brace to the beam at the second floor buckled. In extreme cases, a negative bending moment reaching as much as $1.5M_p$ may develop at link ends because of strain-hardening. Gusset buckling can be avoided by employing the detail shown in Fig. 13. Here two edges of the gusset plate are stiffened. Note also that the tubular brace comes close to the beam flange. Experiments have shown that such connections behave well under cyclic loads and the tube gusset connection can develop a substantial moment capacity.¹³

Two other types of satisfactory brace-to-beam connections are shown in Figs. 14 and 15. For field erection both of these details require overhead welds. Horizontal stiffeners placed at beam edges spanning between the vertical stiffeners are required for the detail shown in Fig. 15. Both connections behaved well under severe cyclic loading.¹³

The link-to-column connection in Fig. 16 is similar to the conventional detail used in MRFs. The shear tab, however, must be welded to the web of the beam. The bolts shown in the figure are erection bolts. Conventional link-to-column web connections, Fig. 17, should be avoided as experiments have repeatedly demonstrated that such connections sustain only very limited cyclic loads.

Nominally concentric joints for beam-column-brace connections are shown in Figs. 18 and 19. The connections shown in Fig. 18 involve field welding, and may be easily modified to make use of a welded detail in connecting the shear tab to the beam web. The detail shown in Fig. 19 is for field bolting. Note that in either detail the centerlines for the three members do not need to intersect at a point permitting design of a compact joint.

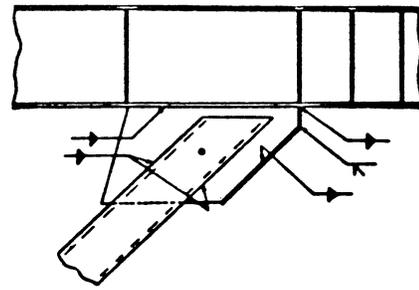


Fig. 13. Detail of improved tube brace-gusset plate connection.

Analyses have shown that very little penalty on cyclic frame performance is paid by assuming the connection of the type shown in Fig. 19 as pinned.¹¹

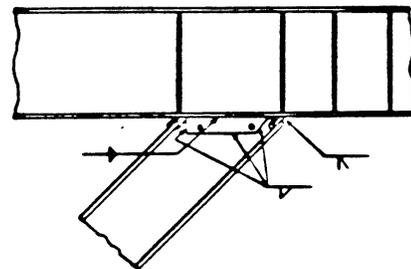


Fig. 14. Detail of WF brace-beam connection

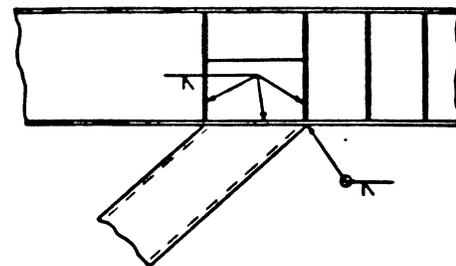


Fig. 15. Detail of tube brace-beam connection.

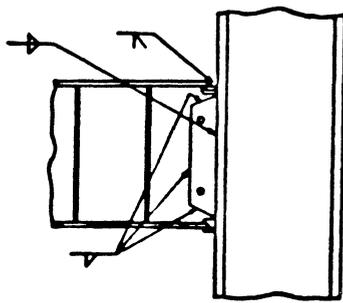


Fig. 16. Typical link-to-column flange connection detail.

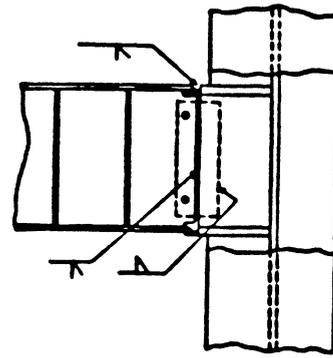


Fig. 17. Link-to-column web connection detail shown has limited cyclic capacity at severe loads.

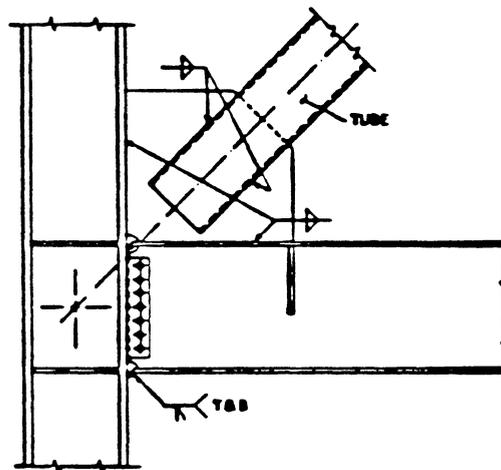


Fig. 18. Detail for brace at beam-column moment connection.

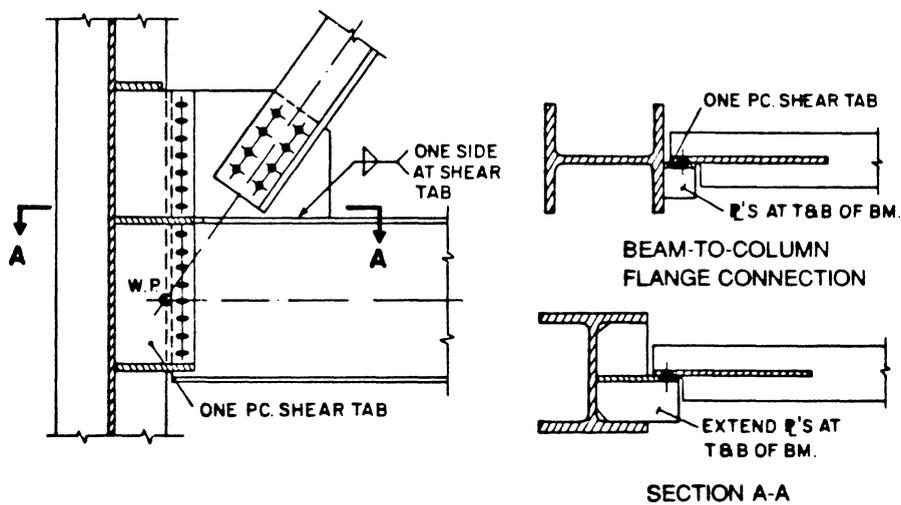


Fig. 19. Bolted detail for beam-column-brace connection.

CONCLUSIONS

Eccentrically braced frames have become a well established structural steel system for seismic resistant construction. Sustained research since the mid-1970's combined with experience from numerous field applications has provided a database for the proper design of EBFs and has led to the development of code provisions for EBF design and detailing. Selected research findings have been highlighted and the development of code provisions for EBFs in the U.S. has been briefly discussed in this paper.

The most recent EBF code provisions are contained in the tentative AISC seismic provisions for steel buildings. This document represents the most up-to-date and comprehensive code requirements for EBFs currently available in the U.S. Excerpts from the tentative AISC seismic provisions pertaining to EBFs have been reproduced in this paper and comments have been provided on the background of selected provisions.

A six story steel building with EBFs was designed using the tentative AISC provisions. A number of inelastic dynamic time history analyses were performed for this building, using several severe ground motion records. The behavior of the EBFs in these analyses was satisfactory, with acceptable levels of link inelastic rotation demands.

Proper detailing is critical for fully developing the ductility of an EBF. Many of the detailing requirements for EBFs, including link stiffener requirements, are provided in the AISC code provisions. To supplement these provisions, a number of suggested connection details for EBFs have been illustrated in this paper.

Based on research and design experiences with EBFs to date, some of the most important design issues are summarized below.

1. Some desirable EBF bracing arrangements are shown in Fig. 1. The EBF type shown in Fig. 1(b) is particularly advantageous because any potential problems with link to column connections are avoided.
2. The use of short, shear yielding links is preferred, as these provide for the maximum strength, stiffness and energy dissipation capacity of the frame. Links in the length range of about M_p/V_p to $1.2M_p/V_p$ appear to be particularly effective.
3. Link lengths exceeding about $1.6 M_p/V_p$ should only be used when links are not attached to columns, as in Fig. 1(b).
4. The use of link to column weak axis connections should be avoided.
5. The use of very flat braces should be avoided in order to limit axial force levels in the beam segment outside of the link. Design problems with the beam and brace can often be avoided by using relatively steep braces, say with a brace-beam angle of 40° to 50° , together with very short links. The use of short links limits the bending moment transferred to the

beam and brace from the link.

6. Strong and stiff lateral bracing should be provided at both ends of the link.
7. The braces, columns, and beam segments outside of the links should be designed for the forces generated by the fully yielded and strain hardened links. That is, "capacity design" principles should be used as an overall design basis for these frame elements.
8. Both the braces and beam segments outside of the link should be treated as beam-columns, considering both axial force and bending moment in strength and stability computations.
9. If the brace is considered to carry a portion of the ultimate link end moment, a rigid brace to beam connection should be used.

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