ADVANCES IN DESIGN OF ECCENTRICALLY BRACED FRAMES

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SYNOPSIS

Eccentrically Braced Frames (EBFs) have attained recognized status as a viable structural steel system for resisting lateral seismic forces. Sustained research at the University of California, Berkeley, since 1977 and numerous field applications provide a good database for their proper design. In this paper the different types of EBF are critically evaluated, and the kinematics of their inelastic deformation are examined with particular reference to the behaviour of isolated short beam segments or links. Desirable link length and web stiffening are recommended. A preliminary design procedure for hand-calculation of EBFs is decribed and some suggestions for brace connection details are advanced.

INTRODUCTION

When moment-resisting frames (MRFs) cannot be economically designed sufficiently stiff for resisting wind forces, concentrically braced frames (CBFs) are generally employed. In some instances in order to accommodate architectural requirements for openings, braces are offset from the columns or do not intersect at the floor beams resulting in an eccentrically connected bracing. This is the prototype for seismic-resistant EBFs. Several alternative bracing arrangements for such framing are shown in Fig. 1; a number of other configurations can be devised. The characteristic feature of this bracing system is that the axial forces induced in the braces are transmitted either to a column or another brace largely through shear and bending in a segment of a floor beam called a link.

For seismic applications the braces are designed such that they do not buckle under extreme loading conditions. This basic requirement can be assured since the ultimate capacity link can be accurately estimated, and an EBF is so proportioned that under severe loadings the major inelastic activity takes place in the link. In this manner links provide the fuses necessary to prevent buckling of the braces.

As to the efficiency of eccentric bracing for augmenting the elastic stiffness of a frame, it is instructive to compare the behaviour of an EBF with a moment-resisting frame and a concentrically braced frame. For this purpose consider the variation in stiffness of the simple EBF shown in Fig. 2 as a function of the link length e . For $e = L$, one has an MRF and the relative frame stiffness is at a minimum. For $e/L > 0.5$ little benefit is gained from the bracing. However, as the length of the link decreases, a rapid increase in elastic frame

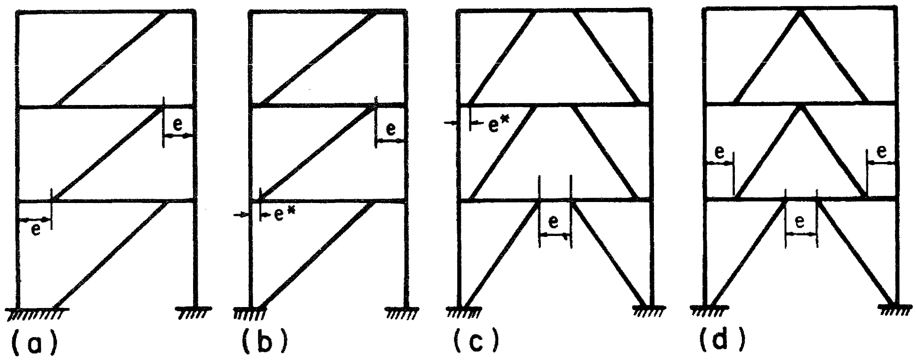

FIG. 1 - SOME ALTERNATIVE BRACING ARRANGEMENTS FOR EBFs

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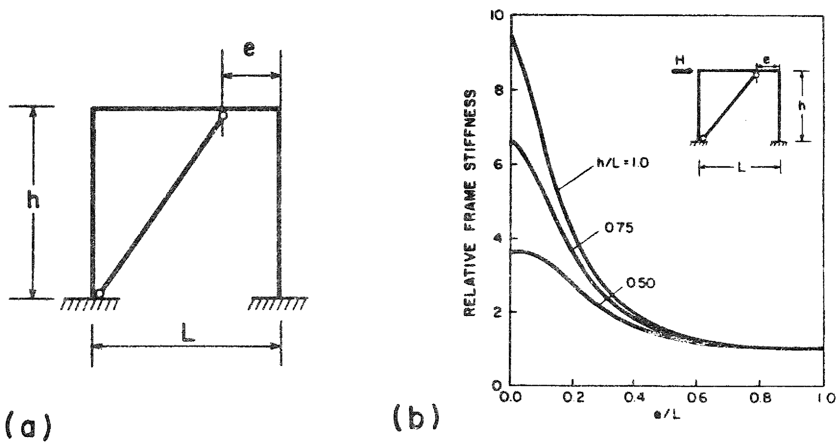

FIG. 2 - (a) SIMPLE ECCENTRICALLY BRACED FRAME

(b) VARIATIONS OF STIFFNESS FOR DIFFERENT ASPECT RATIOS WITH CONSTANT MEMBER SIZES

stiffness occurs. Maximum stiffness develops when $e = 0$, i.e. when one has a concentric brace. Since the behaviour of such braces under severe cyclic loading is unreliable, this is the very condition that is advisable to avoid. This can be achieved by using eccentrically connected braces. To gain the largest possible frame stiffness the links should be made as short as possible compatible with their ability to sustain severe cyclic deformations. Based on studies at Berkeley (Roeder and Popov 1977, Manheim 1982, Hjelmstad and Popov 1983a, 1983b, 1984, Malley and Popov 1983, 1984, Kasai and Popov 1984, 1986a, 1986b, 1986c) recommendations for the design of such links are given herein following a discussion of inelastic deformations of EBFs. Then a preliminary design procedure for hand-calculation of EBFs is described, and suggestions for brace-connection details are given.

INELASTIC DEFORMATION OF FRAMES

An eccentrically braced framing system is a hybrid deriving its stiffness from the truss action and its ductility by inelastic deformation of the links. Therefore it is useful to have a qualitative comparison of the deformation of a conventional MRF with a pair of typical EBFs. As noted in the introduction, since the braces in EBFs are designed so as not to buckle, the danger of brace buckling does not enter the problem.

Kinematically admissible fields of deformation for a typical MRF and two EBFs assuming rigid-plastic members are shown in Fig. 3 (Hjelmstad and Popov 1984). The story drift of these frames is designated by an angle θ . The frames plastically deformed at hinges give a clear indication of the order of magnitude of the member ductility demands. It is least for the MRF for which joints must sustain a plastic hinge rotation θ . For EBFs the short links experience significantly larger rotations. As will be shown in the next section, it is best to make these links short to have them yield predominantly in shear in order to attain the required large angular rotations. To emphasize this point the links in Figs. 3(b) and (c) (Hjelmstad and Popov 1984) are cross-hatched. These short or shear links are efficient in dissipating energy when forced into an inelastic mode. The moment links such as develop in MRFs or in intermediate and long links of EBFs are less efficient energy dissipators because less material of the members is plastically deformed.

For design purposes the inelastic (plastic) member rotations of EBFs must be quantified. This can be most easily done by constructing Energy Dissipation Mechanisms (EDMs) (in plastic analysis commonly known as collapse mechanisms) as shown in Fig. 3. Two more detailed examples are given in Fig. 4 (Kasai and Popov 1986a). Note that the length of a link should be measured from the face or the edge of a column. The nominal link rotation angle γ_p of a link is the same whether such a link is formed by plastic hinges at the ends or is due to shear deformation. The rotation angles depend entirely on the ultimate story drift and geometry of the structure. For the same story drift, span and link lengths, the ductility demand for a V-brace frame is only half as large as for an eccentric K-brace frame. The formulas given in Fig. 4 for angles of rotation γ_p are useful in design.

LINK LENGTH AND WEB STIFFENER REQUIREMENTS

The critical element of an EBF is the link. At Berkeley an extensive experimental and analytical program was carried out on numerous links and their behaviour was studied under monotonic and severe cyclic loading conditions. In several of the experiments different boundary conditions were employed. Some 25 full-size bare links were tested in a state of equal antisymmetric end moments (Hjelmstad and Popov 1983a, Malley and Popov 1983, 1984). In this series of experiments the number, size and spacing of web stiffeners was varied in order to establish preliminary criteria for link length and web stiffeners.

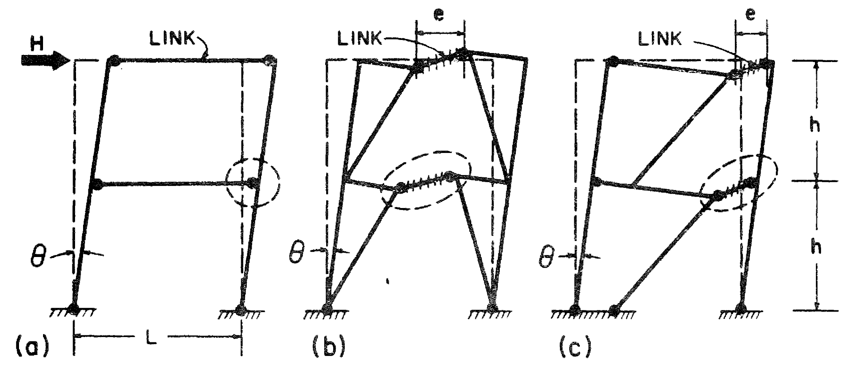

FIG. 3 - KINEMATICALLY ADMISSIBLE FIELDS OF DEFORMATION FOR DIFFERENT FRAMING SCHEMES

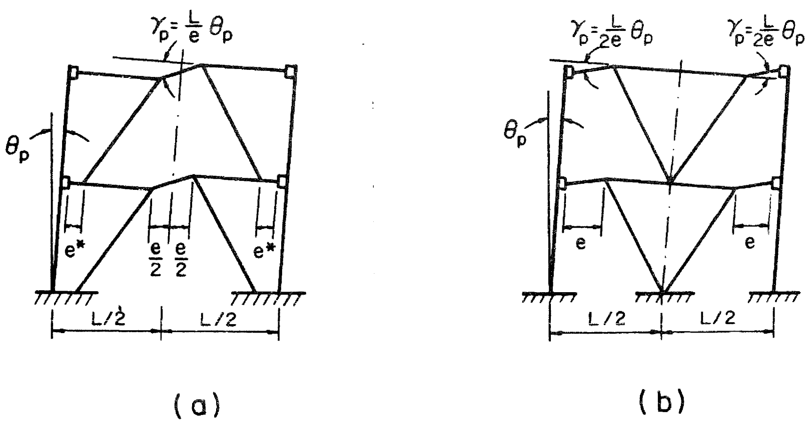

FIG. 4 - ENERGY DISSIPATION MECHANISMS FOR: (a) ECCENTRIC K-BRACED FRAME AND (b) V-BRACE FRAME

Seven additional half-scale bare links were tested to obtain additional data and to determine redistribution of initially unequal end moments in a statically indeterminate situation (Kasai and Popov 1986b). Later cyclic experiments were performed on two bare links at two-thirds scale and on six matching links forming part of a composite deck (Ricles 1985).

In addition to the above component experiments, two major tests were carried out on two complete six-story building models employing eccentric bracing. One of these, a 2-by-2 bay full-size model in Tsukuba, Japan, was tested pseudo-dynamically (Nishiyama 1985). A 0.3-scale replica was tested at Berkeley on a shaking table (Bertero 1986). Prior to these comprehensive experiments six planar three-story EBFs were tested at Berkeley pseudo-statically (Roeder 1977, Manheim 1982).

From the above extensive experimental research a few illustrations follow. To begin with it is useful to demonstrate what can be achieved with a well-designed EBF. Global hysteretic response for one such frame subjected to severe cyclic loading is shown in Fig. 5 (Manheim 1982). From this diagram it can be seen that the test frame successfully sustained very large inelastic cyclic story drift. A drift of over 0.015H from the initial position was achieved without failure. The hysteretic loops prior to this event are remarkably repetitive and show no degradation. The system is stable well into the inelastic range. To obtain such performance a design must be carefully executed. A choice of correct link length and proper stiffening of the web play a crucial role in the design process.

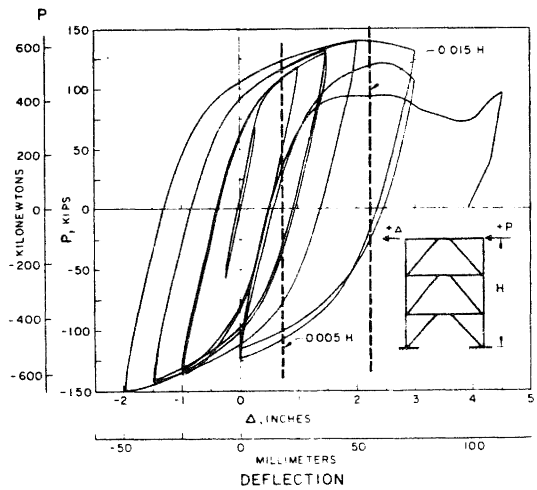

FIG. 5 - HYSTERETIC LOOPS FOR ECCENTRICALLY BRACED TEST FRAME

Because of the efficiency of short links in hysterically dissipating energy most of the work completed at Berkeley thus far has been done on such links. It is recommended that the shear link length e comply with the following criterion (Kasai and Popov 1986b):

$$e \leq 1.6 \frac{M_p}{V_p} \qquad (1)$$

in which M_p is the plastic moment of a W-section and $V_p = 0.55F_y d t_w$, where F_y is the yield strength of steel, d is beam depth, and t_w is web thickness. No doubler plates are permitted within the link.

For the present it is advisable not to exceed the link length given in Eq. 1. In the inelastic range the short or <u>shear</u> links deform appreciably in shear thereby effectively contributing to the energy dissipation by this mode of deformation. Moreover, usually the rotational flange capacity at the end of such links is taxed less severely than in the longer links and fewer problems are encountered with the beam itself outside the link.

Experiments have shown that shear links with well-stiffened webs can sustain without failure a total cyclic angular rotation γ up to ±0.10 rad (Ricles 1985). However, the minimize damage it is recommended by SEAOC (1985) that γ be limited to ±0.06 rad. At this level of link rotation the floor damage is remarkably small (Ricles 1985).

If an analysis shows that γ in a particular design of an EBF is smaller than noted above, fewer web stiffeners need be used. On this basis the following relationships for equal spacing of stiffeners in a link are recommended (Kasai and Popov 1986c):

$$a = 38t_w - \frac{d}{5} \quad \text{for } \gamma = \pm 0.06 \qquad (2)$$

$$a = 56t_w - \frac{d}{5} \quad \text{for } \gamma = \pm 0.03 \text{ or less} \qquad (3)$$

in which a is stiffener spacing, and, as before, t_w is link web thickness, and d is beam depth. For intermediate value of γ it is appropriate to interpolate.

Links designed on the above basis behave well. To demonstrate this it is useful to compare the hysteretic loops from two cyclic experiments shown in Fig. 6 (Hjelmstad and Popov 1984). Both specimens were made from W18 x 40 A36 steel, and their length of 28 in. (0.71 m) satisfied Eq. 1. However Specimen 1 had no web stiffeners, whereas three pairs of 3/8-in (9.5 mm) thick equally spaced stiffeners were used in Specimen 4. A dramatic difference in the capacity of the links to sustain severe cyclic loading is clearly evident from comparison of the two sets of hysteretic loops.

The writers of conventional codes for steel design in the past have not envisioned the severe cyclic shear loadings that may be imposed on links in EBFs. Tentative provisions for the design of EBFs now recognize this problem and accordingly include appropriate safeguards (BSSC 1985, SEAOC 1985).

DESIGN OF EBFs

<u>Basic Concept for EBF Design.</u> The strength and ductility of a properly designed EBF is directly related to the strength and ductility of the links. The required strength and ductility of an EBF can be achieved through the following basic design steps:

(1) Estimate the required shear-resisting capacity of a link and select a beam section to give an appropriate link capacity.

(2) Design other members so that ultimate link forces can be developed.

(3) Estimate the ductility demand on the structure and the links. Detail the link so that the required ductility is achieved.

The links should be designed for code or other specified earthquake forces. All other frame members must be designed for the forces generated by the fully yielded and strain-hardened links in order to ensure the desired energy dissipation mechanisms accompanied by the plastic activity of the link. If the design philosophy is followed, the maximum strength and ductility of the

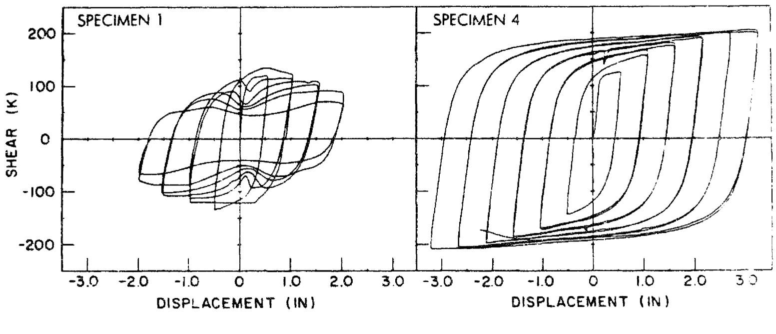

FIG. 6 - HYSTERETIC LOOPS FOR: (a) UNSTIFFENED SHEAR LINK (SPECIMEN 1) AND (b) LINK WITH THREE PAIR OF STIFFENERS (SPECIMEN 4)

EBF are achieved.

Method for Estimating Member Forces. In order to predict member forces in an EBF at the preliminary design stage, the use of plastic analysis technique is the most rational approach, since by such an approach the required strength and desired energy dissipation mechanisms for a frame can best be achieved. The technique was explored in the past at Berkeley (Roeder and Popov 1877, Popov and Roeder 1978, Manheim 1982), and significantly improved more recently (Kasai and Popov 1984, 1986a). The new approach is straightforward and has been found very accurate. In the following sections the application of this procedure is briefly discussed (Kasai and Popov 1986a).

Selection of a Link Beam. For most EBF configurations, the shear force in the link V_{link} can be related to V_{cum}, the static design story shear accumulated from the top to the corresponding level of a structure. The basic relationship can be expressed as:

$$V_{link} = \frac{h}{L} V_{cum}\ , \tag{4}$$

where h and L are the story height and span length, respectively. Figure 7 illustrates this approach for an eccentric K-brace framing. Equation 4 can be derived by writing a moment equilibrium relationship around the column base point A, and neglecting the significance of moments acting at the upper and lower ends of the EBF panel (Kasai and Popov 1986a). For a single diagonal type framing, such as that shown in Fig. 8, the moment of the column next to the link is large. However, through algebraic manipulations, it can be shown that Eq. 4 remains reasonably accurate (Ksai and Popov 1986a).

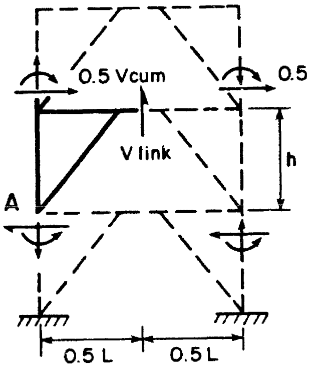

FIG. 7 - APPROXIMATE FREE-BODY DIAGRAM OF ECCENTRIC K-BRACE FRAME

Link Ultimate State. The ultimate shear force V_{ult} of the shear link selected to satisfy Eqs. 1 and 4 should be taken to be at least

$$V_{ult} = 1.5V_p \tag{5}$$

Based on experimental results (Ricles 1985,

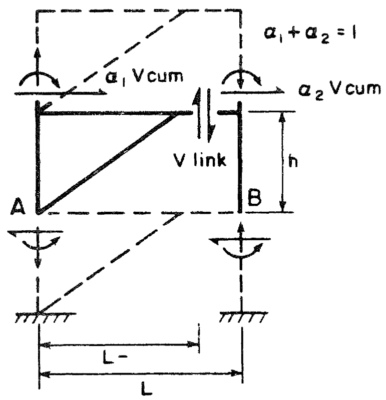

FIG. 8 - APPROXIMATE FREE-BODY DIAGRAM OF ECCENTRIC SINGLE DIAGONAL BRACE FRAME

Kasai and Popov 1986b), the following estimates for the link end moments at the ultimate state may be assumed:

$$\text{If}\quad 1.3\frac{M_p}{V_p} \le e \le 1.6\,\frac{M_p}{V_p}\ ,\ \text{then}\ M_A = M_B = \frac{V_{ult}e}{2} \tag{6}$$

$$\text{If}\quad e \le 1.3\frac{M_p}{V_p}\ ,\ \text{then}\ M_A = M_p,\ M_B = V_{ult}e - M_p \tag{7}$$

where M_A and M_B are the link moments at the column face and at the opposite end of a link, respectively. If an EBF of symmetric configuration such as that shown in Fig. 7 is used, Eq. 6 is appropriate for a link of any length. Further details on this approach will be elaborated upon in a forthcoming paper.

Design of Other Members. The design forces for the beam segment outside the link and the brace at the link ultimate state can be estimated by considering the statics of the brace-beam-link subassemblage. In most EBF configurations, large axial forces are generated not only in the braces and columns, but also in the beam segment outside the link. These segments must therefore be designed as beam-columns, and adequate lateral bracing must be provided to assure stability, particularly if a composite deck is not present.

For certain EBF configurations, plastic hinges may form outside the link prior to the development of the link ultimate state. The rotation capacity of such a hinge may be quite limited due to the presence of a large axial force. Two approaches to this problem are possible. Although both are indicated below, only the first has been fully developed.

The suggested approach consists of maintaining the beam segment in the elastic state to resist the forces that develop in the link ultimate state. To achieve this in design one can reduce the link length as much as possible or one can choose a beam section which has a larger moment and axial force capacity and yet possess an appropriate shear-resisting capacity as referred to in step (1) above. In extreme cases flange cover plates on the beam segment can be applied. If the span length of an EBF is sufficiently large, a scheme in which two braces per bay are employed, such as that shown in Figs. 1(d) or 4(b), may be adopted. For such cases the axial force in the beam segments tends to be smaller due to steeper brace angles. In preliminary design the braces can be considered pinned at the ends. After selecting the members, the designer must check brace end moments using more realistic boundary conditions. Columns of EBFs should be designed for the forces generated by strain-hardened links (Eqs. 5, 6, and 7) as well as for the appropriate gravity load contributions.

For certain configurations, it may be difficult to ensure that the beam segment outside the link remains elastic at the link ultimate state. For such cases, an alternative design approach is to permit limited moment hinging in the beam outside the link, but to require that the combined strength of the brace and beam is sufficient to resist the ultimate link force.

Although sizing of the link can also be accomplished through an elastic analysis (SEAOC 1985) the elements supporting the link must be checked to determine whether they are capable of developing the link ultimate forces as described above.

Link Detailing for Ductility. After the above steps have been completed, the designer must check the ductility demand on the links. From the EBF kinematics discussed earlier, the required plastic rotation of the link can be computed from the ultimate story drift. SEAOC conservatively recommends limiting the link rotation to ±0.07 rad. Future research and experience may indicate that a larger value for rotation can be allowed since some critical experiments have shown a link rotation capacity up to ±0.10 rad (Ricles 1985). After the required plastic rotation has been computed, stiffer spacing is determined using Eqs. 2 and 3. Lateral bracing must be provided at the link ends to permit the full ductility of the link to be developed. To maximize the ductility of a link, the EBF configuration should be chosen so as to minimize axial forces in the links.

TYPICAL CONNECTION DETAILS

Proper detailing is critical to developing the full strength and ductility of any earthquake-resistant structure, including EBFs. This section illustrates what are believed to be both safe and practical details for selected connections in EBFs.

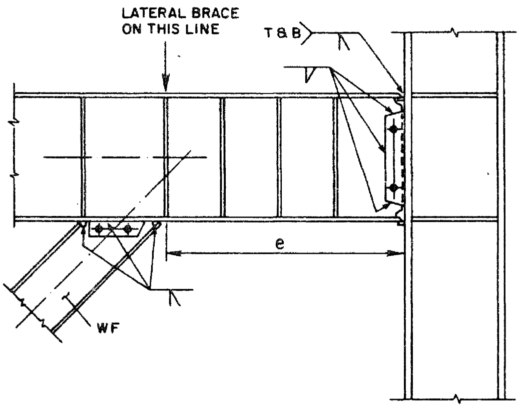

FIG. 9 - LINK AND BRACE DETAIL ADJACENT TO COLUMN

Figure 9 illustrates a link next to a column. A lateral brace must be provided at the link end at the location shown in the figure. Strong and stiff lateral bracing at the link ends is critical if the stability of the link and the brace is to be maintained. The link-to-column connection must be a fully welded moment-resisting connection, with full penetration flange welds and a web connection capable of developing the shear capacity of the link. Either fillet welding the web to a shear tab, as shown, or providing a full penetration weld between the beam web and column flange is acceptable. The welding sequence should be chosen to minimize locked-in stresses due to restraint. For the severe service intended for links, bolted web connections show inadequate ductility due to bolt slippage (Malley and Popov 1983, 1984)

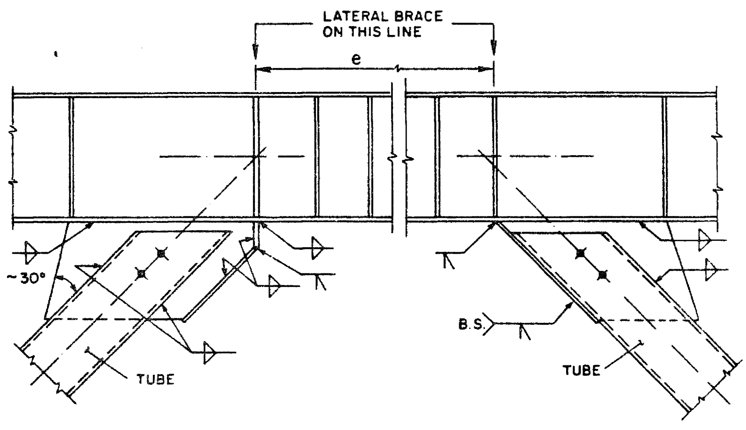

FIG. 10 - TYPICAL DETAILS OF TUBE BRACE AT LINK

and should not be used. Connection of links to column webs should be avoided. The reliability of connections to the column webs has not been firmly established experimentally, and design recommendations cannot as yet be provided.

A recommended detail for connection of a WF brace at a link is also illustrated in Fig. 9. This detail is particularly effective for longer links where large end moments are developed in the brace. Typical gusseted connections for rectangular and square tube braces at a link are illustrated in Fig. 10. As a rule, the brace end should be cut parallel to the beam and located as close to the beam as practical. This minimizes the possibility of buckling of the gusset between the brace end and the beam. Further, the free edge of the gusset nearest the link should be stiffened as shown. Bending moments in the beam cause large compressive stresses along this edge of the gusset, and stiffening is required to prevent buckling.

Nominally, the brace centreline should intersect the beam centreline at the end of the link, as shown on the left side of Fig. 10. However, analytical studies have shown that it is acceptable for the brace and beam centrelines to intersect somewhat inside the link, as shown in Fig. 9 and on the right side of Fig. 10. This will, in some cases, permit a more compact brace connection. The centrelines should not, however, intersect outside the link.

Typical stiffening for shear links is illustrated in Figs. 9 and 10. Two-sided, full-depth stiffeners must be provided at the link end. Equally spaced stiffeners inside the link may be single sided for beam depths less than 24 inches, but should be two sided for deeper beams. In general, stiffeners inside the link should be full depth, welded to the web and to both flanges. Full-depth stiffeners, though more costly, provide restraint against both web buckling and flange buckling. Providing full depth stiffeners outside the link above the brace, as shown in Figs. 9 and 10, is also recommended.

Typical nominally concentric bracing connections for the end of the brace opposite the link are illustrated in Figs. 11 and 12.

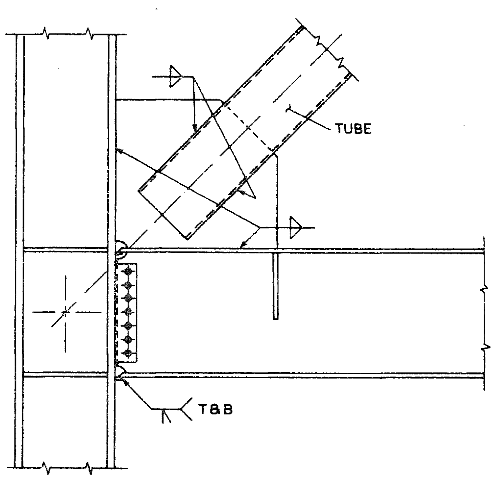

FIG. 11 - TYPICAL DETAIL FOR BRACE AT MOMENT BEAM-COLUMN CONNECTION

The case of the brace framing into a moment-resisting beam column connection is shown in Fig. 11. The use of moment-resisting beam column connections is not essential in EBFs except at the links. Their use, however, provided redundancy in the frame and also results in a stronger concentric brace connection.

If simple framing is used, a suggested detail is shown in Fig. 12. A problem observed in tests of simple beam column connections in EBFs (Manheim 1982) is the out-of-plane twisting of the beam at the connection. The connection must provide restraint against such twisting. A single plate shear tab may not be adequate by itself. For the detail shown in Fig. 12, additional restraint against twisting of the beam is provided by the extended plates at the top and bottom of the beam. The most secure restraint against twisting is provided, of course, by a moment-resisting connection. The intersection of brace and beam centrelines located at the column face is also shown in Fig. 12. Offsetting the work point arbitrarily from the column centreline to, for example, the column face

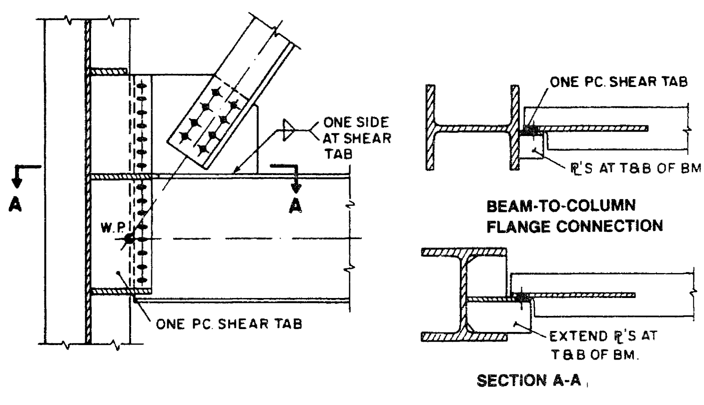

FIG. 12 - TYPICAL DETAIL FOR BRACE AT NON-MOMENT BEAM-COLUMN CONNECTION

often permits a more compact connection. The same concept may also be used to advantage for the type of connection illustrated in Fig. 11. The additional moment produced by an offset in the work point should be included in the column design.

CONCLUDING REMARKS

The eccentrically braced frame has proven to be a viable system and has been adopted for numerous projects in California. Inasmuch as conventional methods of fabrication and erection can be used, little difficulty has been experienced. However EBFs must be carefully implemented at the design stage. At this writing the most critical problems relate to the design of beam segments adjoining the link which, during a severe earthquake, are called upon to carry large forces. Lateral bracing of link ends is also essential.

It would appear that EBFs have a wide range of application. On a number of projects the designers have found it advantageous to combine EBFs with an MRF along the narrow direction of a building. In such usage story drift is very effectively controlled.

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