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Methodology for optimum EBF link design

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ABSTRACT: Eccentrically braced frames (EBFs) designed improperly can develop undesirable large inelastic link deformations where energy is dissipated at only a few floors. This unsatisfactory behavior can be caused by ill proportioning of links in an EBF. Cases are discussed drawn from both analytical and experimental studies to illustrate the unacceptable seismic behavior in EBFs with incorrectly proportioned links. A design procedure for avoiding such behavior is explained, illustrating a correct selection of links and corresponding behavior of the EBFs. Primary emphasis is placed on the link strength distribution along the height of an EBF and its desirable effect on inelastic link deformations throughout the structure.

1 INTRODUCTION

Eccentrically braced frames (EBF) have established themselves as a viable seismically resistant frame system. An EBF is a hybrid system, combining the advantages of a moment resisting frame (MRF) and a concentrically braced frame (CBF). Eccentricities in EBFs are deliberately introduced at joints to provide short portions of beams called "links". During frequently occurring minor earthquakes links remain elastic. In the event of a strong earthquake causing structural overloading, links are designed to deform inelastically and thereby prevent buckling of diagonal bracing. It has been recognized that short links that yield primarily in shear provide excellent energy dissipation under severe cyclic loading. Such links are known as "shear links" [Kasai et al., 1986b,c].

EBF design provisions exist in most major design codes in the United States as well as other countries. It has come to the authors' attention that some of the recent EBF construction, as well as EBF research, is based on an incorrect interpretation of EBF link design method. This stems apparently from a misunderstanding of EBF design philosophy associated with proportioning the links, where not enough attention is given to strength distribution of links over the height of an EBF. An incorrect proportioning of links can result in the EBF becoming susceptible to a concentration of excessive inelastic link deformation at particular story levels.

Analytical and experimental evidence of undesirable seismic behavior of EBFs having incorrectly proportioned links is discussed. The authors' original design philosophy for EBF links for avoiding such behavior is illustrated by a method for proportioning links and the corresponding behavior of EBFs. Emphasis is placed on link inelastic deformation developed with respect to the link strength distribution throughout the height of the EBF.

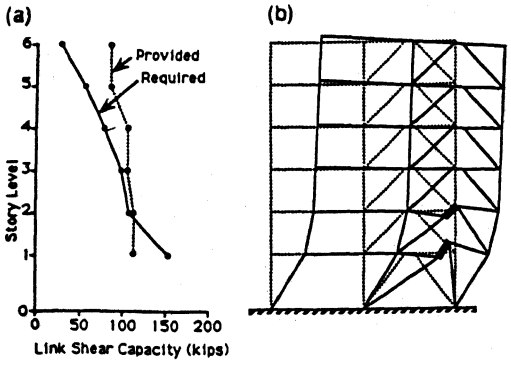

Figure 1. U.S.-Japan EBF Test Structure.

2 PROPORTIONING OF LINKS

Based on studies by the authors [Kasai et al., 1986a; Popov et al., 1989], an approximate link design shear force V_{link} in an EBF subjected to lateral loads is obtained as follows:

$$V_{link} = V_{story}\left(\frac{h}{L}\right) \qquad (1)$$

in which V_{story}, h, and L are the story shear, floor height, and span length, respectively, at the corresponding story level of the EBF. In general, V_{story}, and therefore V_{link}, calculated from typical code forces, varies parabolically over the height of the EBF when the building is of a regular type. In this paper, V_{story} is chosen to be the required nominal story shear at the yield limit state of the EBF. Thus, the link yield capacity must be equal to or greater than V_{link} obtained from Equation 1. Accordingly, if a shear link is used, the design requirement for the link is:

$$\frac{V_p}{V_{link}} \equiv \alpha \geq 1.0 \qquad (2)$$

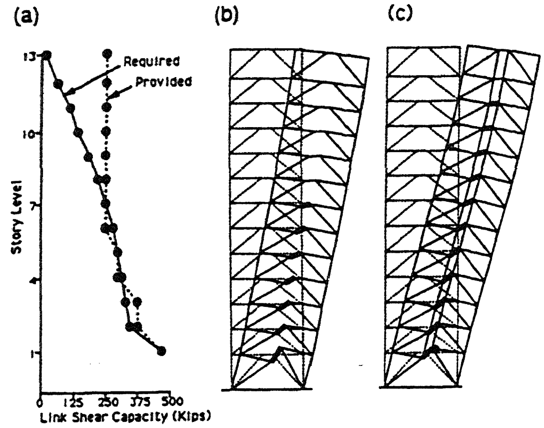

Figure 2. (a) Required Story Shear vs. Designed Capacity, (b) Inelastic Deformation of Untied, and (c) Tied EBF.

where V_p is the nominal shear yield capacity of the shear link, and α is the link strength index [Ricles et al., 1989]. The nominal capacity V_p is calculated as 0.55 times the product of the steel yield stress and web area [AISC, 1992].

It has been established [Kasai et al., 1986a; Ricles et al., 1990] that, if an EBF possesses a uniform value for α throughout the frame height, all links yield almost simultaneously under monotonically increasing lateral load. These EBFs are shown to develop a fairly uniform inelastic deformation, without any concentration of excessive inelastic link deformation at particular story levels. This behavior under static loading is also desirable in an EBF subjected to actual seismic conditions involving earthquake-induced dynamic lateral loads having various histories. A study was conducted which investigated under dynamic loading the: (1) effect of uniform and variable distributions of α throughout the EBF height, and (2) the effect of static V_{story} distribution throughout the EBF height for which the links are proportioned. This investigation and previous studies involving EBFs with ill proportioned links are discussed below.

3 INCORRECT PROPORTIONING OF LINKS

3.1 U.S.-Japan 6-story EBF

A full-scale 6-story EBF building was tested pseudo-dynamically under Phase II of the U.S.-Japan Cooperative Earthquake Research Program Utilizing Large-Size Testing Facilities [Roeder et al., 1987]. The building was originally designed as a concentrically braced frame (CBF) building for Phase I of the test program, in which the concentric braces were placed at the left bay of the frame shown in Figure 1. Upon completion of the Phase I tests, the concentric braces were removed from the building and the structure was repaired. Then, an eccentric bracing system was installed in the right bay of the frame as shown in Figure 1(b).

For economic reasons, it was required that for Phase II testing all the beams and columns designed for Phase I be reused. This restricted the EBF design. The beam sections used for the CBF were found inappropriate for a correct design for the EBF, but had to be used. Fortunately, column sections were found sufficiently large for the EBF. Tube brace sizes were selected in order that the braces remain essentially elastic and not buckle.

Using the existing beam sections, shear links were created by maintaining their lengths sufficiently short. Figure 1(a) compares the required V_{link} of the EBF based on the static design loads with the provided link shear capacity V_p. Note that in general V_{link} varies parabolically throughout the EBF height, since V_{story} varies parabolically as noted previously. The exceptional case is seen at the first floor level, where V_{link} increased due to a larger h as compared with the upper story levels (see Equation 1). In contrast to these, the provided link capacity V_p was almost constant throughout the EBF height, resulting in an extremely nonuniform strength index α which varies from 0.7 at the first floor level to 3.3 at the sixth (roof) level. This indicated that the upper story levels of the EBF were over-strengthen whereas the lower story levels were under-strengthen.

In spite of the problem, the EBF generally performed in a superior manner as compared with the CBF [Roeder et al., 1987]. However, the EBF developed the soft story mechanism illustrated in Figure 1(b). This was expected prior to the experiment in view of the extremely nonuniform α explained above (Figure 1(a)). During the final phase of the test, a failure occurred at a diagonal brace-to-beam connection in the first floor. Therefore improved connection details were proposed [Popov et al., 1989] for avoiding this type of damage in a local region. However, it should be noted that a concentration of such damage could have been reduced significantly if the EBF had much larger links at the lower levels, since the links were under-strengthen at these levels.

3.2 13-Story EBF with and without Tied Links

Some seismic designers do not recognize the importance of having a uniform strength index α to avoid concentrations of large inelastic link deformations at particular floor levels. Such was the case in the design of a 13-story EBF by Martini et al [1990] where the links were incorrectly proportioned in the above sense. Compare the V_{link} and V_p shown in Figure 2. Like the frame discussed in Section 3.1, large overstrength of the EBF at the top six levels is evident due to the oversized links. These incorrectly proportioned links resulted in large link inelastic deformations at the lower levels of the EBFs when subjected to a static lateral load (see Figure 2(b)).

As one of the solutions to this problem, Martini et al. proposed a modified EBF connecting the links vertically with ties (see Figure 2(c)). This type of EBF tends to develop a more uniform distribution of link inelastic deformation, as illustrated in Figure 2(c). However, under the static loading condition considered, the ties would have been unnecessary had the links been proportioned by maintaining a more uniform value for α throughout the EBF height (Section 2). Similar

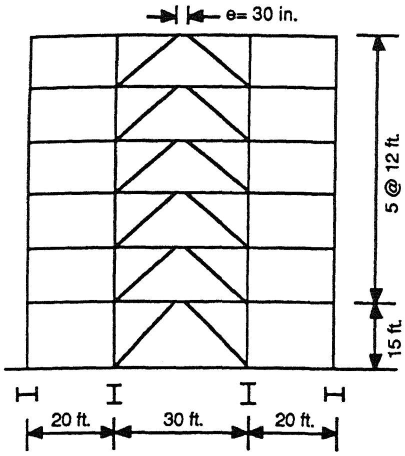

Figure 3. EBF configuration for inelastic time history analysis (6 story EBF shown, 1 ft = 0.305 m, 1 in. = 25.4 mm).

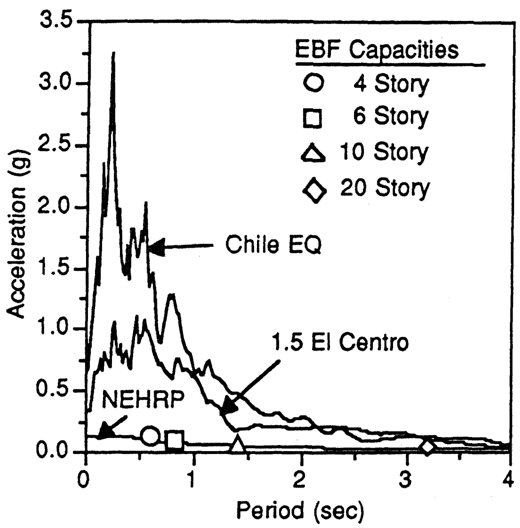

Figure 4. Spectral acceleration corresponding to earthquakes records, NEHRP, and EBFs' base shear.

conclusions can be reached for dynamic loading cases, as will be noted below.

4 EBF DESIGN FOR DYNAMIC ANALYSIS

A variety of EBFs were designed and used in a subsequent inelastic time history analyses. The basic configuration of the frame consisted of a three bay dual system: an MRF with an eccentrically braced interior bay (see Figure 3). The frame was designed to represent a parameter frame in a typical office building in seismic Zone 4 (San Francisco). The outer bays of the frame have a 20 ft (6.1. m) column spacing and the inner bays a 30 ft (9.14 m) column spacing. All links were designed as shear links and had a length of 30 in. (762 mm). The height h of the first floor was 15 ft (4.57 m) with all remaining floors 12 ft (3.66 m) high. To study the effect of the building height, EBFs of 4, 6, 10, and 20 stories were designed and analyzed.

The design of the EBFs was based on the base shear force V_{base} prescribed for a dual EBF system per NEHRP [1988]. V_{base} is plotted as a function of elastic period in Figure 4.

To assess the effect of the distribution of story shear V_{story}, two sets of EBFs were designed and analyzed (Sets A and B). EBFs in Set A included Frames 4A, 6A, 10A, and 20A. The EBFs in Set A were designed for a story shear based on a distribution of V_{base} using the equivalent lateral load distribution per NEHRP, where for floor i the lateral load F_{xi} is:

$$F_{xi} = V_{base} \frac{(w_i h_{xi})^k}{\sum_{i=1}^{roof} w_i h_{xi}} \qquad (3)$$

where w_i, h_{xi}, and k are floor weight, height of floor above ground, and a coefficient that varies from 1.0 to 2.0 as a function of frame height, respectively.

The second set of EBFs, Set B, was designed using the CQC method [A.Der Kiureghian, 1981], accounting for 90% of the effective modal mass, to generate the story design shears. EBFs belonging to Set B included Frames 4B, 6B, 10B, and 20B. The values for V_{story} for EBFs of both sets are shown in Figure 5, where the nominal story shear capacity based on the nominal link capacity V_p for EBFs in Sets A and B are also given, and calculated by:

$$V_{p,\,story} = \frac{L}{h} V_{p,\,link} \qquad (4)$$

As shown in Figure 5, the EBFs in Set A were designed with a greater link overstrength, and not as uniform a distribution of α throughout the frame as those EBFs in Set B. For the design of EBFs in Set B, emphasis was placed on achieving as uniform a value for α as possible. In the 4, 6, and 10 story EBFs of Set B this was achieved by using fictitious links in the model of the frames whose strengths gave a constant value of α =1.25 at each floor level. In the 20 story EBF, available W-Shape sections were used for the links in the model of the frame. A further examination of V_{story} in Figure 5 based on the static and CQC distributions, respectively, of V_{base} indicates that for the shorter EBFs (particularly the 4 and 6 story) the two distributions are nearly identical. As the frame height increases, a larger discrepancy develops between the two distributions, where the CQC prescribes larger shears in the upper floors near the roof and smaller shears in the middle and lower floors compared to the static based distribution.

Two earthquake records were used for the inelastic time history analyses. These included the NS component of the 1940 El Centro earthquake, scaled to 0.5 g maximum ground acceleration, and the N10E component of the 1985 Chile earthquake. The latter had a maximum ground acceleration of 0.67 g. The El Centro accelerogram was selected for its common use in other studies and the cyclic yielding caused by the scaled record, whereas the Chile earthquake was used because

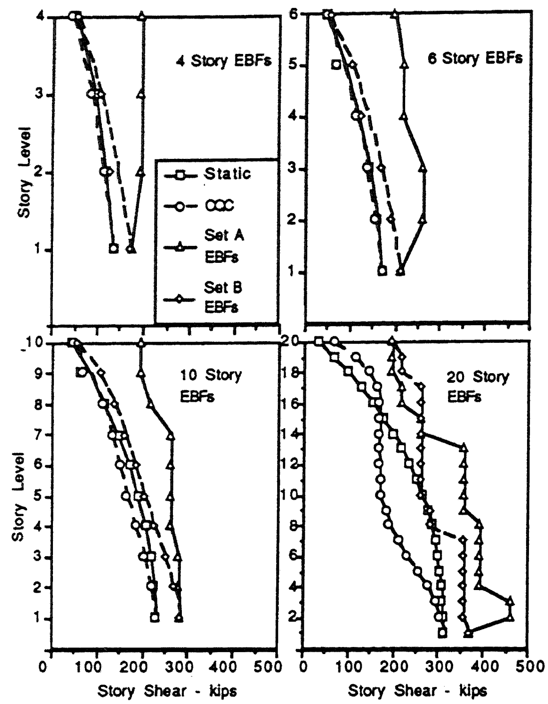

Figure 5. Required and Supplied Story Shear Distributions.

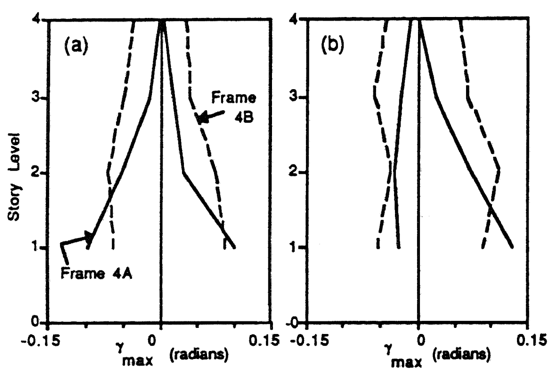

Figure 6. Link Deformation for 4-Story EBFs subjected to (a) 1.5 El Centro and (b) Chile earthquakes.

of the high frequency content in the record and its long duration. The response spectra for these two earthquakes are shown in Figure 4, where they are seen to exceed the NEHRP design base shear and EBFs' strength. Thus, major inelastic response and ductility demand is expected in the time history analysis. The time history analysis was performed using the ANSR-I program [Mondkar, 1975], where the links were modelled by a specially developed link element for random cyclic inelastic loading [Ricles et al., 1987].

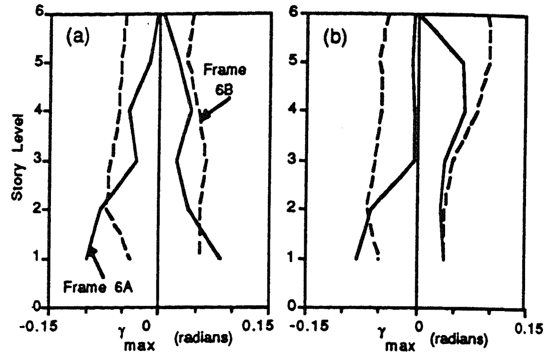

Figure 7. Link Deformation for 6-Story EBFs subjected to (a) 1.5 El Centro and (b) Chile earthquakes.

5 DYNAMIC ANALYSIS RESULTS

5.1 4-Story EBFs

Figure 6 illustrates the distribution of peak link inelastic deformation, γ_{max}, of Frames 4A and 4B subjected to scaled El Centro earthquake (see Figure 6(a)) and the Chile earthquake (see Figure 6(b)). For both earthquakes, Frame 4A, having a significantly larger α at the upper story levels, developed a soft story mechanism in the lower story levels where a large concentration of γ_{max} developed. In contrast, Frame 4B, having a more uniform α developed a much more uniform distribution of γ_{max} throughout the height of the EBF.

Research has shown that the "relative link deformation" γ_{rel} measured as the sum of the absolute values of positive and negative γ_{max} is a key parameter to measure the ductility demand on the links. It was found that a well-stiffened link can sustain a γ_{rel} up to 0.18 radians [Kasai et al., 1986b]. In Frame 4A, γ_{rel} at the first floor level is 0.20 radians, exceeding the acceptable limit under the scaled El Centro earthquake. In Frame 4B, γ_{rel} at all story levels are well within the acceptable limit for link deformation under both earthquakes.

5.2 6-Story EBFs

Figure 7 illustrates the distribution of γ_{max} for Frames 6A and 6B subjected to the two named earthquakes. Like Frame 4A discussed above, Frame 6A, having a large value for α at the upper story levels, developed a soft story mechanism with a large γ_{max} concentrated at the lower story levels. In contrast, Frame 6B indicates a much more uniform distribution of γ_{max} throughout the height of the EBF.

In Frame 6A γ_{rel} at the first floor level slightly exceeded the acceptable limit under the scaled El Centro earthquake. In Frame 6B, γ_{rel} at all story levels are well within the acceptable limits under both earthquakes.

Note that γ_{rel} at the first floor level of Frame 6B is 52% of that in Frame 6A under the scaled El Centro earthquake, and 71% under the Chile earthquake. This indicates a considerable reduction of the link deformation

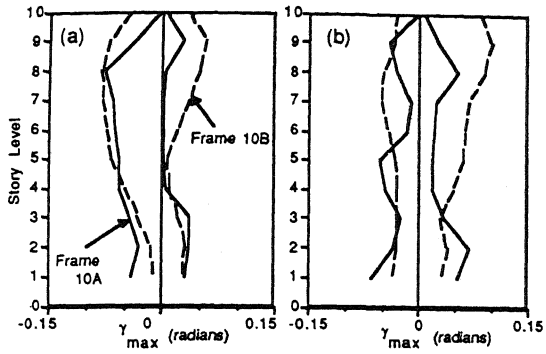

Figure 8. Link Deformation for 10-Story EBFs subjected to (a) 1.5 El Centro and (b) Chile earthquakes.

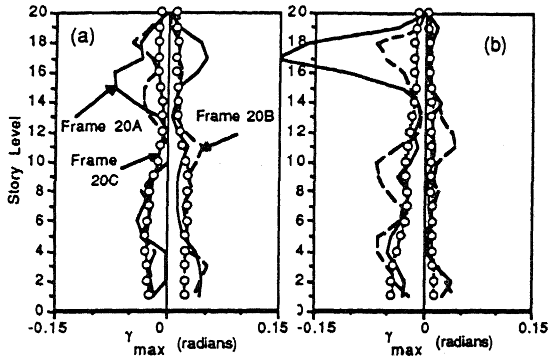

Figure 9. Link Deformation for 20-Story EBFs subjected to (a) 1.5 El Centro and (b) Chile earthquakes.

demand at the lower level by maintaining a more uniform value for α throughout the height of the EBF.

5.3 10-Story EBFs

Figure 8 illustrates the distribution of γ_{max} for Frames 10A and 10B subjected to the two earthquakes. In both frames, the magnitudes of γ_{max} are generally smaller as compared with the 4-story and 6-story frames discussed above. Unlike Frames 4A and 6A, Frame 10A, having a large α at the upper story levels, did not develop a soft story mechanism at the lower story levels under the scaled El Centro earthquake. It did however develop such a mechanism under the Chile earthquake, but the γ_{rel} at the first floor level was well within the acceptable limit.

Like Frames 4B and 6B, Frame 10B, with more uniform values for α throughout the EBF's height, resulted in a considerable reduction of the link deformation demand at the lower level under both earthquakes. Its γ_{rel} at the first floor level is less than 60% of that of the Frame 10A under both earthquakes. It should be noted, however, that Frame 10B developed much larger γ_{max} at the upper story levels as compared with its lower story levels under the two earthquakes. Although the γ_{rel} at the upper levels are still well within the acceptable limit, this suggests the need for further investigation regarding the optimum value for α in order to achieve a uniform γ_{max} throughout the EBF's height. It can be concluded, however, that the EBF with a uniform α throughout its height, as compared with that with larger values of α at the upper story levels, can distribute link inelastic activity well throughout the frame height, thus avoiding excessive inelastic deformation demand at the lower floor levels.

5.4 20-Story EBFs

Figure 9 illustrates the distribution of γ_{max} for Frames 20A and 20B subjected to the two earthquakes. In both frames, the magnitudes of γ_{max} are relatively small at the lower floors compared with the shorter frames discussed earlier.

Frame 20A has links proportioned to the NEHRP static design force by maintaining an approximately constant value for α. Under the simulation of the Chile earthquake, Frame 20A showed an extremely large γ_{max} of -0.2 radians at the 17th story level, and the γ_{rel} exceeded the acceptable limit of 0.18 radians. Similar behavior was observed under the scaled El Centro earthquake, but the magnitudes of γ_{max} were smaller than those under the Chile earthquake. The frame also showed very moderate link deformation below the 13th story level for both earthquakes.

Frame 20B has the links proportioned to the CQC story shear force, maintaining approximately a constant value for α. Unlike Frame 20A, Frame 20B behaved in an excellent manner showing very moderate and fairly uniform link deformation throughout the frame height under both earthquakes.

The significant difference on performance between Frames 20A and 20B indicates an excessively large γ_{max} at the upper level of tall EBFs that are proportioned to the NEHRP static force. The result also suggests that this problem can be avoided if the CQC story shear force that considers the higher modes of vibration is used for proportioning the links. Although the links in Frame 20A have fairly uniform overstrength with respect to a V_{link} calculated from the NEHRP static force, there is a significant non-uniform distribution of overstrength of Frame 20A's link capacity compared to V_{link} based on the CQC story shear forces. The strength index α for this frame is significantly lower at the 1st, 16th, and 17th levels compared with the other floor levels. This is why a large γ_{max} developed in the vicinity of the 17th story of Frame 20A.

Figure 9 additionally shows the link deformation in a tied EBF, namely Frame 20C, explained earlier in Section 3.2. The purpose for the use of ties is to distribute link inelastic deformation uniformly throughout the EBF story height. As expected, Frame 20C exhibited fairly uniform γ_{max}, but the EBF's performance is no superior to that of Frame 20B. This indicates that even without ties the EBF can be designed to have well distributed link inelastic action throughout the frame height. Other studies by the authors [Ricles et al., 1991], however, indicate some performance advantages in using relatively short tied EBFs.

6 CONCLUSIONS

The following conclusions are given:

(1) An EBF design that typically overstrengthens the links at higher story levels results in a concentration of link deformation at the lower levels, developing large story drift at these floors leading to soft story mechanism.

(2) If the links are proportioned closely to the required link shear force using current static code design forces, an EBF up to 10 stories in height develops inelastic link deformation that is distributed reasonably well throughout the frame height during earthquakes.

(3) If elastic dynamic vibration modes are considered to calibrate the static design force, seismic inelastic performance of taller EBFs can be further improved. The key is to include the contribution of second and third modes of vibration in determining the static story shear design forces.

(4) Link deformation demand on taller EBFs is generally less compared to that of shorter EBFs.

(5) It was observed in this study that the effect of very severe earthquakes on column axial forces in 4 and 6 story EBFs exceeded the commonly specified magnification factor of 1.25 [AISC, 1992]. It was smaller for the higher frames [Ricles et al., 1991].

Short links, a preferred type, were used in the EBF designs. The design can be modified if longer links are needed. These analyses pertain only to seismic loading. In the event that wind loading governs the design, it is still necessary to check the link size for compliance with seismic requirements. It is clear from this study that an EBF having unnecessarily large links at the higher floor levels develops very small link energy dissipation at these levels. Considering this, and since larger links require more expensive detailing as well as large braces and columns, such EBFs are uneconomical. Undoubtedly, the EBF having links of uniform α throughout the height of the structure is a more economical solution.

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