A Review of Research on Steel Eccentrically Braced Frames

Sina Kazemzadeh Azad, Cem Topkaya*

Department of Civil Engineering, Middle East Technical University, Ankara, Turkey

Abstract: This paper reviews the research conducted on steel eccentrically braced frames (EBFs). Both component level and system level responses for such braced frames are treated and discussed. For the component level response, a thorough review of the investigations on links, which are the primary sources of energy dissipation in EBFs, has been presented. The results of experimental and numerical studies on strength, rotation capacity, and overstrength of links are discussed. Furthermore, studies on the effects of axial force, the presence of a concrete slab, the loading history, compactness, link detailing, and the lateral bracing on link behavior are summarized. Relevant available research on link-to-column connections is revisited. Different approaches for the numerical modeling of links are also given. For the system level response, characteristics of EBF systems are discussed in light of the capacity design approach. Findings of numerical studies on the seismic performance of EBFs are discussed to provide insight into suitable response factors utilized in the design of these systems. Additionally, special topics and emerging applications of EBFs, such as replaceable links, are provided. The impact of research findings on the design of EBF systems is demonstrated considering the AISC Seismic Provisions for Structural Steel Buildings. Finally, future research needs for improvement of EBF design and application are identified and presented.

Keywords: Eccentrically braced frames, links, structural fuses, review of research, experiments, numerical studies, seismic design

*Corresponding author, Tel: 90-312-210-5462 e-mail address: ctopkaya@metu.edu.tr

1. Introduction

The main idea in the design of an eccentrically braced frame (EBF) is to integrate the advantages of both moment resisting frame (MRF) and concentrically braced frame (CBF) lateral load resisting systems into a single structural system. The EBF system originated from Japan in 1970s [1, 2] with the aim of achieving a structure with high elastic stiffness as well as high energy dissipation during severe earthquakes.

There are several configurations for an EBF system, some of which are depicted in Figure 1 along with their expected plastic mechanisms. Larger architectural openings can be used with EBF systems when compared to CBFs. The short segment of the frame generally designated by the length *e* (Figure 1) is called the link. In EBF systems, yielding is concentrated only at link segments and all other members of the frame are proportioned to remain essentially elastic. Therefore, during severe earthquakes, links can be considered as structural fuses which will dissipate the seismic input energy through stable and controlled plastic deformations.

A comprehensive review is provided in this paper on the behavior and design of eccentrically braced frames. The review includes research conducted on links, as they comprise the most critical elements of an EBF. In addition, the research on EBF system response is elaborated. Areas of future research needs are also identified. The comparison of design provisions as presented in various design specifications is out of the scope of this work; however, the AISC Seismic Provisions for Structural Steel Buildings [3] are mentioned to illustrate relationships between research findings and design rules.

2. Characteristics of Links

2.1. Yield Behavior, Shear Capacity and Overstrength

The length of a link segment (e) is one of the key parameters that controls the stiffness, strength, ductility, and behavior of an EBF system. The link length ratio, $\rho = e/(M_P/V_P)$, where M_P and V_P are the plastic moment and plastic shear capacities of the link, provides a convenient measure for the yield behavior. The free-body diagram of an isolated link is shown in Figure 2. Based on equilibrium, considering equal end moments at the ultimate state, no moment-shear interaction, and an elastic-perfectly plastic material, the theoretical dividing link length ratio between shear dominated and flexure dominated behavior is $\rho^{theor} = 2.0$. In short (or shear) links, shear yielding of the web is found to be predominant (Figure 3a). On the other hand, in long (or moment) links, flexural yielding controls the link behavior (Figure 3c). An intermediate link, however, would experience a combination of both shear and flexural yielding (Figure 3b).

There are substantial differences between the behavior of short and long links. Although longer links provide more architectural freedom for openings, early experimental studies by Roeder and Popov [4, 5] and Hjelmstad and Popov [6, 7] showed that the performance of short links is considerably better than that of long links under severe cyclic loadings in terms of strength and ductility. Over the years Popov and his colleagues [6, 8-10] suggested different practical limiting lengths for shear dominated behavior, finally arriving at the limit of $\rho < 1.6$, which is still in use in many design specifications including AISC 341-10 [3].

The first comprehensive study on the behavior of intermediate and long links ($\rho > 1.6$) was conducted by Engelhardt and Popov [11] in 1989. A total of 14 tests were conducted on 12 two-third scale subassemblage specimens with ρ ranging from 1.45 to 4.25. Based on the experimental results it was concluded that a gradual transition from the shear-dominant behavior to the flexure-dominant behavior occurs as ρ is increased from 1.6 up to 3. Despite this, in most of the previous and current specifications (e.g. [3, 12]), links with length ratios of $1.6 < \rho < 2.6$ are classified as intermediate links while links with $\rho > 2.6$ are generally referred to as long links. It is important to note that the presence of high axial force in a link may change this categorization, as discussed in Section 2.3. Engelhardt and Popov [11] also reported that moment-shear interaction has a notable effect on the behavior of intermediate links, while short and long links are generally unaffected.

Terms such as very short and very long links are also used in the literature. Although there are no explicit definitions, links with $\rho > 3 \sim 3.5$ are sometimes referred to as very long links [11, 13] while links with $\rho < 1$ as very short links [14]. In 2012, finite element (FE) analyses conducted by Daneshmand and Hashemi [13] demonstrated that the behavior of very long links can differ notably from that of long links in terms of failure mode and ductility, and thus, dividing the long link range in design codes into two or more sub-regions was suggested. Furthermore, recent studies (e.g. [14]) have proved that there are also other remarkable differences between the characteristics of very short links and short links, which will be discussed later.

The nominal shear capacity (V_n) of a link can be defined as follows [3]:

$$V_n = \text{Min}\ [V_P,\ 2M_P/e] \tag{1}$$

where the plastic capacities (i.e. V_P and M_P) are calculated based on the nominal yield stress, F_y. For links with length ratios less than 2.0 the first term will govern; however, for longer links the second term is dominant. In order to meet the objectives of the capacity design approach, it is necessary to estimate the maximum shear force that can develop in a link during an intense loading, i.e. V_{max}. Consequently, other structural members shall be designed to remain essentially elastic and resist the loads developed by the fully yielded and strain hardened link. Any underestimation of the maximum link force may lead to unfavorable failures in other members. To have a reliable estimate of V_{max} the link overstrength concept is generally utilized as follows:

$$V_{max} = \Omega(R_y V_n) = \Omega V_e \tag{2}$$

where R_y is the ratio of expected to nominal yield stress, considered for each steel grade based on statistical data, V_e is the expected (or actual) shear capacity of the link, and Ω is the overstrength factor due to strain hardening. Early experimental studies by Popov and his colleagues [4, 6, 11, 15, 16] suggested the value of $\Omega = 1.5$ for design purposes, which is still (implicitly) in use in most seismic design specifications (e.g. [3, 17]). The link overstrengths observed in the previous experimental studies on horizontal I-shaped links [6, 11, 14, 15, 18-39] are presented in Figure 4. The traditional line of $\Omega = 1.5$ is also depicted in this figure. It should be mentioned that the value of V_e for every data point is calculated using the measured properties reported in the original reference. Only cyclic test results for links with $F_y > 200$ MPa are included in the figure. A summary of the experiments conducted between 1983 and 2002 on EBF links is also presented by Richards [40] in his dissertation. Most of the links tested by Popov and his colleagues [6, 9, 11, 15, 19] were constructed from A36 steel, while links with higher strength steels such as A709 and A992 were tested later by other researchers such as McDaniel [24], Okazaki and his colleagues [31, 33], Mansour et al. [41], and Dusicka et al. [36]. Shear links constructed from low-yield-strength steels were also tested more recently by Dusicka et al. [36] and Ji et al. [14].

As can be seen in Figure 4, the value of $\Omega = 1.5$ seems like a reasonable upper-bound for links with $\rho > 1$. However, this overstrength value overestimates V_{max} for some intermediate links ($1.6 < \rho < 2.6$) and drastically underestimates V_{max} for very short links ($\rho < 1$). There are several reasons for these discrepancies, which can be summarized as follows.

Using a constant value of 1.5 for the overstrength factor neglects the effect of moment-shear (M-V) interaction. However, the study by Engelhardt and Popov [11] proved that this interaction can be important for intermediate links where Ω values lower than 1.5 were observed (Figure 4).

The nominal shear capacity of short links is generally calculated based on the web area, neglecting the contribution of the flanges [3]. However, as mentioned by McDaniel et al. [24], in short links with relatively thick flanges, there is a significant shear force carried by the flanges. Manheim and Popov [8] and Richards [40] have proposed methods for determining the link plastic shear capacity considering the flange effect. Through nonlinear FE analysis of isolated links, Richards [40] demonstrated that the overstrength factor of 1.5 is a reasonable limit even for very short links, provided that the flange effect is included in determining V_P. However, the same conclusion was not reached when previous test results on short and very short built-up links [21, 22, 24, 42] were examined. The calculated overstrengths for these links decreased slightly by including the flange effect in determining V_P; however, they were still significantly higher than 1.5 for most cases. Richards [40] attributed this discrepancy to the presence of other factors, beside the flange effect, in experimental studies. In 2007, Okazaki and Engelhardt [31] reported the test results for a total of 37 links constructed from ASTM A992 steel. Although not very short, some of the specimens had very high ratios of flange to web area (A_f / A_w). These specimens however did not exhibit overstrengths substantially higher than 1.5. On the contrary, other recent results by Ji et al. [14] indicated that very short links with even lower flange to web area ratios could achieve remarkably high overstrengths, in some cases even over 2.0. A further numerical investigation by Ji et al. [14] revealed that for the tested specimens the shear contribution of flanges could be as much as $0.2V_P$. However, it was concluded that other factors in addition to the flange effect are influential in causing the observed high overstrengths. Thus, it can be deduced that the numerical and experimental studies are inconclusive about the contribution of flanges to the link overstrength.

If axial restraints are present at the link ends, tension can develop during shearing, due to nonlinear geometric effects, especially at high rotation angles [40]. In 2012, this crucial issue was investigated by Della Corte et al. [43] using detailed FE analyses considering both geometric and material nonlinearities. Based on the numerical results it was concluded that the presence of axial restraints can significantly increase the overstrength (up to 15% increase was reported in some models) especially for short links. In a more recent study, Ji et al. [14] also mentioned that restraining axial deformations of very short links by adjacent members can create non-negligible axial forces in these links and affect their behavior significantly.

Another possible cause for the very high overstrengths observed in some tests is the excessive cyclic hardening of steel due to very large plastic strains. This idea was introduced recently by Ji et al. [14] on the basis of an experiment conducted by Kasai et al. [44] on stocky short steel panels in which hardening continued to very large shear angles. Based on this observation, Ji et al. [14] proposed that the very high overstrengths reported for very short links are related to their high rotation capacities. These links experience large plastic rotations (substantially higher than 0.08 rad) and consequently their webs are subjected to excessive shear strains which may lead to excessive cyclic hardening. A refined FE analysis by Ji et al. [14] on very short links revealed that for a link plastic rotation of the order of 0.15 rad the cyclic hardening effect can increase the shear strength by about 70%.

The combined effect of the above-mentioned factors should be considered for each link in order to have a reasonable estimate of its overstrength. Nonetheless, reasons behind some of the unusually high overstrengths reported in the literature still remain unclear. For instance, Dusicka et al. [36] observed overstrengths of about 5.0 in their experiments on very short links constructed from low-yield-strength steel (F_y = 100 MPa) without web stiffeners. Such cases require further investigation.

Based on the numerical and experimental works of Berman and Bruneau [45-49] the use of built-up box (or tubular) links is also permitted by AISC 341-10 [3]. A comparison between the test results reported by Berman and Bruneau [49] with the numerical data presented by Richards [40] revealed that the overstrength factor of built-up box links is typically higher than that of I-shaped links, by about 11%. It is worth noting that the use of hollow structural sections (HSS) as links is prohibited by AISC 341-10 [3] due to their questionable performance in terms of low cycle fatigue life under large strains [50]. In addition, as explained in Berman and Bruneau [47], the longest shear link that can be constructed from a common HSS is about 460 mm, which is rather short and will cause congested details as well as very high rotation demands at the code-specified drift level.

2.2. Link Rotation Demand and Capacity

The expected plastic mechanisms of EBF systems are depicted in Figure 1. The angle between the link and the beam outside of the link is termed as the total link rotation angle (γ) and its inelastic part as the inelastic (or plastic) link rotation angle (γ_P). In general, the inelastic rotation capacity of a link, γ_P^{max}, is defined as the maximum inelastic rotation angle (i.e. excluding the elastic portion which is usually less than 0.01 rad) sustained by the link during a cyclic test, for at least one full cycle of loading, before the shear resisted by the link drops below a predefined limit. This limit can be quantified as 80% of the maximum link shear recorded during the cyclic test ($0.8V_{max}$), the plastic shear capacity (V_P), or the code-specified nominal shear strength of the link (V_n) [14, 30, 41, 51]. Figure 5 shows an example of using the first definition for determining the inelastic rotation capacity (γ_P^{max}) of a specimen tested by Okazaki et al. [29].

Experimental studies have shown that the rotation capacity of links depends significantly upon several factors such as the link length ratio, loading history, compactness and web stiffening. A summary of findings from the experimental studies on horizontal I-shaped links [6, 11, 14, 15, 18-39] is given in Figure 6. The figure includes links with F_y > 200 MPa subjected to various cyclic loading histories. It should be mentioned that a few monotonic tests were also conducted on links [15, 18, 27, 28, 37, 52] which demonstrated rotation capacities between 0.19 ~ 0.42 rad. Short links exhibit higher rotation capacities compared to intermediate and long links. As shown in Figure 6, AISC 341-10 [3] limits the inelastic rotation of short links to 0.08 rad and long links to 0.02 rad. For intermediate links, linear interpolation is utilized. The following issues should be borne in mind when examining the data shown in Figure 6. In a number of tests in which connection yielding or failure was reported (grey triangles in the figure), specifically those located in the range of $1 < \rho < 2$, high rotation capacities in excess of 0.10 rad were observed. It should be noted that in most of these tests the connection inelastic rotation also contributed to the total link rotation, and thus, the reported high γ_P^{max} values are not solely because of the inelastic action of the link. The results denoted by hollow circles indicate tests conducted by Galvez [25] and Ryu [28] and reported by Okazaki and Engelhardt [31], in which the observed rotation capacities fall significantly below the code-specified limit due to the use of a severe loading protocol. On the other hand, most of the γ_P^{max} values reported for very short links (i.e. $\rho < 1$) exceed the AISC limit by a wide margin. These results (particularly the ones reported in [14, 36]) revealed that the rotation capacity of very short links can be substantially higher than that of short links. Note that some of the tested very short links (shown by hollow diamonds) failed prematurely due to the brittle fracture of the link web, which was initiated as a result of the insufficient offset between the stiffener-to-web and the web-to-flange welds, as explained by McDaniel et al. [24]. Considering only the proper test data (grey diamonds in Figure 6), it can be deduced that the current AISC requirements provide a reasonable lower-bound for the inelastic rotation capacity of links with various length ratios.

It is difficult to predict the rotation capacity of all links by means of pure numerical simulation. For the case of long and some intermediate links, in which strength degradation is due to local buckling, γ_P^{max} can be estimated reasonably via nonlinear FE analysis. However, it is troublesome to estimate the rotation capacity of shorter links since strength degradation is generally due to low-cycle fatigue induced fractures, which are not commonly considered in FE simulations. Nonetheless, results of the numerical simulations [13, 51, 53-55] concurred with experimental observations that the inelastic rotation capacity of short links is considerably higher than that of intermediate and long links.

The inelastic link rotation demand must be estimated at the design stage. The most accurate way of determining this quantity is through inelastic dynamic analysis. Alternatively, a rigid plastic mechanism can be used to estimate the inelastic link rotation angle. In this method the inelastic link rotation angle, γ_P, is related to the plastic story drift, Δ_P, via geometrical relationships [3]. Extensive nonlinear time-history analyses conducted by Koboevic et al. [56, 57] on low-, mid-, and high-rise EBFs further confirmed the appropriateness of assuming such a relationship. However, it was noted by Koboevic et al. [56, 57] and other recent studies [58-60] that determining Δ_P based on the results of an elastic analysis using the codified displacement amplification factors (C_d) or the equal-displacement rule may provide unconservative estimates of γ_P. This item is further discussed in Section 5.2.

2.3. Effect of Axial Force

Links can be subjected to axial loads due to the axial restraining effect of adjacent members (which was discussed in Section 2.1) and also the loading scheme and/or geometry of a structural system. For instance, in case of a seismic loading, the EBF configuration shown in Figure 1b will impose higher axial forces on link segments compared to the negligible link axial forces developed in the configuration shown in Figure 1a. A study by Kasai and Popov [10] revealed that the presence of axial force can have deteriorative effects on link behavior. These researchers proposed modified expressions for the plastic moment and shear capacities of links in the presence of axial force. In addition, a modified shear link length limit was defined. The modified capacities are in general related to the axial load ratio, defined by P/P_y, where P is the available axial force and P_y is the nominal axial yield strength. This study revealed that the plastic capacities should be reduced as a function of P/P_y when this ratio exceeds 0.15. These recommendations still form the basis of the AISC 341-10 [3] provisions for links under axial loads.

In 1990, Ghobarah and Ramadan [61] used nonlinear FE analysis to investigate the effect of axial force on the performance of EBF links. Numerical results demonstrated that the presence of axial force not only reduces the link plastic strength but also its plastic rotation capacity and energy dissipation. A maximum decrease of 37% in γ_P^{max} was reported. It was also noted that the effect of axial force is most pronounced in links with length ratios near 1.6.

In a study by Mansour et al. [41] in 2011, the behavior of replaceable shear links was investigated inside a frame system. In these tests the cyclic loading was applied to the floor beam from one end, and thus, half of the applied load was transferred axially through the link. The maximum value of P/P_y was reported to be 0.26 which represented a relatively high axial force level. Mansour et al. [41] observed that the links experienced higher peak shear forces when subjected to tension rather than compression, with a maximum difference of 12%. Mansour [34] also demonstrated this issue numerically in his dissertation and proposed a simple equation for estimating the increase in the shear capacity due to axial tension.

In 2014, Dastmalchi [62] conducted nonlinear time-history analyses on a three-story prototype structure with eccentric braces, similar to the configuration shown in Figure 1b. Results revealed that the peak value of P/P_y recorded for the links exceeded 0.15 by a wide margin under most earthquake records, indicating the high probability of developing large axial forces in shear links with the selected configuration. Dastmalchi [62] also performed nonlinear FE analyses to examine the behavior of shear links under very high levels of axial force ($0.15 < P/P_y < 0.5$). The monotonic and cyclic shearing of short links under constant axial compression confirmed its detrimental effect on the shear strength and ductility of short links, especially when the length of the shear link was increased. It was also found that the shear capacity formula given in AISC 341-10 [3] underestimates this effect for $P/P_y > 0.2$, and therefore, a numerically calibrated modification factor was introduced for this formula.

It is worth noting that, as stated in the AISC 341-10 [3] commentary, the effect of high axial force on the behavior of long and intermediate links has not been investigated adequately. Therefore, the code [3] requires the use of shear links if high axial force is present in a link.

2.4. Effect of Concrete Slab

In 1989, Ricles and Popov [63] conducted research on the effect of a concrete slab, placed over the steel framing as a floor system, on EBF links. These researchers reported that the initial stiffness and strength of the composite links were higher than those of the steel links, however, the composite action deteriorated in later cycles. The maximum shear forces resisted by the composite links were 1 to 13% higher than the corresponding steel links. Nevertheless, the general hysteretic behavior of the composite links resembled that of the bare steel links. The slab damage was localized in the vicinity of the link and no damage was observed in the concrete located away from the link segment. The test results also demonstrated that a concrete slab alone cannot provide sufficient lateral bracing for links. Engelhardt and Popov [11, 64] noted in their experimental study that if a diagonal brace is connected to the bottom flange of a link, the presence of a concrete slab can substantially enhance the stability of the link by restraining the top flange. A similar observation was also reported by Tsai et al. [65] during the tests on large EBF sub-assemblages conducted at the National Taiwan University in the early 1990s.

Mansour et al. [41] also investigated the effect of a concrete slab on link behavior. In the details studied, the replaceable link segment had a smaller depth than the floor beam. Therefore, there was no direct interaction between the link and the composite slab. Nevertheless, it was reported that the specimen with the concrete slab sustained a higher shear force, 14% more when compared to the bare steel specimen, without a notable change in the link rotation capacity. Mansour et al. [41] recommended using more shear studs away from the link region in order to guarantee the slab diaphragm action during major earthquakes.

Recently, Ciutina et al. [37] tested a single-story EBF setup with very short links with and without a concrete slab. In one case the shear studs were placed along the entire beam length, while in the other case they were suppressed in the link region. Tests by Ciutina et al. [37] confirmed that omitting shear studs only in the link region does not fully eliminate the composite action, since in both of the above cases the shear strength and stiffness were notably higher than those of the steel link. Therefore, it was recommended that the composite action be considered during the EBF design, even if shear studs are only available on the floor beams and not in the link regions. It is worth noting that link regions are protected zones per AISC 341-10 [3] in which the use of shear studs is prohibited. In addition to link behavior, the presence of a concrete slab also affects the structural response of EBF systems, which is discussed further in Section 5.2.

2.5. Effect of Loading History

Popov and his colleagues (e.g. [6, 18]) noticed that the applied loading history (or protocol) during an experimental study has a major effect on the observed plastic rotation capacity of EBF links. A comprehensive study on this effect was, however, conducted recently by Richards and Uang [66, 67].

Okazaki et al. [29] reported unexpected link web fractures for a number of tested short links prior to reaching the code-specified 0.08 rad rotation limit. Richards and Uang [66, 67] attributed this issue mainly to the utilized loading history and stated that the loading protocol used by Okazaki et al. [29] (based on Appendix S of AISC 341-02 [68]) was significantly more severe than the loading sequences used in the 1980s tests on short links. Nonlinear time-history analyses were conducted by Richards and Uang [66, 67] on three prototype EBF structures subjected to 20 large-magnitude-small-distance Los Angeles ground motions. The obtained cumulative rotation demands were used to come up with a new protocol which was adopted by AISC 341-05 [69] for the cyclic testing of link-to-column connections. The proposed protocol has fewer cycles with large rotations compared to the old protocol.

Okazaki et al. [29] retested the specimens which failed to reach the code-specified plastic rotation limit under the AISC 341-02 [68] old protocol using the new (revised) loading protocol. All of the newly tested specimens exhibited rotation capacities higher than the code-specified limit, with an average increase of 52% in γ_P^{max}. Further studies by Okazaki and his colleagues [31, 33] also confirmed the conclusion that the old protocol is overly demanding compared to the revised protocol for shear links. It is worth noting that other loading protocols, for instance, random loading protocols, loading protocols available in specifications other than the U.S. standards, or more severe loading histories than the AISC 341-02 [68] old protocol have also been used in some studies (e.g. [14, 31]) to demonstrate the sensitivity of the rotation capacity of shear links to the applied loading sequences.

As explained by Richards [40], if the revised protocol is used for testing intermediate or long links, a modest increase in γ_P^{max} for intermediate links and a modest decrease in γ_P^{max} for long links might be observed. Okazaki et al. [33] also noted that the cyclic demand imposed on flexural links by both protocols is somewhat similar. The revised protocol was used in a number of experimental and numerical studies on intermediate and long links (e.g. [13, 31, 70]). Daneshmand and Hashemi [13] demonstrated the sensitivity of the rotation capacity of intermediate links to the employed loading protocol. The value of γ_P^{max} obtained via nonlinear FE analysis for an intermediate link reduced by 18% when the link was loaded based on the old protocol instead of the revised protocol.

3. Detailing of Links

3.1. Flange and Web Compactness

The link flange slenderness limit is needed to prevent severe strength degradation due to flange local buckling during intense loadings. Kasai and Popov [10] calculated the link flange stress at the ultimate shear of $1.5V_P$ and its associated moment for 156 links with four different yield stresses and two length ratios, $\rho = 1.6$ (short) and $\rho = 2$ (intermediate), with and without axial force. The obtained maximum flange stresses were compared to a conservative critical plastic buckling stress determined based on Haaijer's method [71]. Kasai and Popov [10] did not detect flange buckling for the links with $\rho = 1.6$ with no axial force when the flange slenderness (i.e. $b_f / 2t_f$) was limited to $0.38\sqrt{E/F_y}$, where b_f and t_f are the flange width and thickness respectively and E is the elastic modulus of the steel material. However, if these links were subjected to axial force, the stringent flange slenderness limit of $0.3\sqrt{E/F_y}$ was suggested to prevent flange buckling. On the other hand, some of the intermediate links with $\rho = 2$, especially if subjected to axial force, were prone to flange buckling even if the flange slenderness was kept below the more stringent limit of $0.3\sqrt{E/F_y}$. Based on these, it was recommended to limit the flange slenderness of links to $0.3\sqrt{E/F_y}$. This limit was adopted by the early EBF specifications [12, 72, 73] and was in use prior to the 2005 edition of AISC 341 [69].

Adhering to the flange slenderness limit of $0.3\sqrt{E/F_y}$ disqualified several efficient wide-flange rolled sections constructed from A992 steel from being used as links. A992 steel has become the most widely used steel material in the U.S. and has replaced A36 steel after the 1994 Northridge earthquake [51]. Using heavier sections to satisfy this requirement would be the common approach in practice, but this is not fruitful from the capacity design point of view. Richards and Uang [51] conducted a comprehensive numerical study to further investigate this issue. After verifying the FE modeling procedure using the experimental data reported by Arce [23], a total of 112 isolated I-shaped link models were analyzed considering different flange slenderness values. The numerical results demonstrated that the flange slenderness limit can be relaxed from $0.3\sqrt{E/F_y}$ to $0.38\sqrt{E/F_y}$. Although some of the intermediate links could not achieve the code-specified rotation limit, this issue was related to the stiffener requirements and not the flange slenderness limit [51]. In addition to the numerical investigation of Richards and Uang [51], subsequent experimental studies by Okazaki and his colleagues [29, 31] on wide-flange links constructed from A992 steel confirmed that the above relaxation can be safely applied to short links. However, Okazaki et al. [29, 31] observed strength degradation due to flange buckling in some of the intermediate link specimens. Taking into account previous studies [10, 29, 31, 51] it was permitted as per AISC 341-05 [69] to use moderately ductile flanges for I-shaped links with the slenderness limit of $0.38\sqrt{E/F_y}$ only in short links ($\rho < 1.6$). It should be noted that in the numerical study of Richards and Uang [51] and also the recent tests of Okazaki and his colleagues [29, 31] no axial force was imposed on the links. Nevertheless, the above relaxation in the flange slenderness requirements includes all shear yielding links regardless of the level of axial force.

Based on the results obtained from an exhaustive numerical parametric study containing more than 200 analyses as well as a subsequent experimental study, Berman and Bruneau [48, 49] concluded that the flange slenderness of built-up box links should be limited to $0.64\sqrt{E/F_y}$. Until 2005, this limit was the seismically compact limit for walls of rectangular HSS members as per AISC 341 [69]. However, in 2010 the more stringent limit of $0.55\sqrt{E/F_y}$ was adopted by AISC 341-10 [3] for flanges of highly ductile built-up box sections, which is the case for tubular links.

In general, compact webs are used in links to prevent or delay the deteriorative effect of web buckling [3]. Berman and Bruneau [48, 49] suggested that the web slenderness for built-up box shear links should be limited to $1.67\sqrt{E/F_y}$. However, this limit was reduced to $0.64\sqrt{E/F_y}$ for intermediate and long built-up box links in which local buckling of both webs and flanges can cause strength degradation [48, 49].

3.2. Web Stiffeners

The proper use of end and intermediate web stiffeners in links is a major parameter for achieving stable and controlled hysteresis behavior. End stiffeners are usually full-depth stiffeners provided for all link length ratios located on both sides of the web at link ends. In 1977, Roeder and Popov [4] provided the rationale for the necessity of using end stiffeners to ensure local stability at a brace-link-beam connection panel. In the previous and current EBF specifications (e.g. [12, 72-74]) the use of end stiffeners has always been mandatory with an aim of improving the link shear force transfer to reacting elements as well as preventing premature local buckling in links.

In the early experiments by Popov and his colleagues (e.g. [6, 75]) it was observed that in short links tearing of web and severe strength degradation usually occurred shortly after web buckling. Although in some tests (e.g. [6]) a considerable amount of energy was also dissipated by the link in the post-buckling phase, since post-buckling behavior and its subsequent failure are difficult to predict and more hazardous, web buckling is generally considered as the design ultimate state for short links [6, 7, 75]. Popov and his colleagues [6, 18, 75] demonstrated that providing intermediate stiffeners could substantially improve the strength and energy dissipation capacity of links. In 1983, Hjelmstad and Popov [7] proposed the first relation for determining the required intermediate stiffener spacing based on the expected energy dissipation of a link. However, later tests by Kasai and Popov [10] demonstrated that such a relation does not exist and instead, the required intermediate stiffener spacing is dependent upon the expected ultimate link rotation, γ_u. In 1986, Kasai and Popov [75] proposed a conservative spacing formula for intermediate stiffeners using a cyclic plastic theory and the experimental data obtained from tests [15] on short links constructed from A36 steel. The stiffener spacing was expressed as a function of the depth of the I-shaped link and its web thickness for three different ultimate rotations. The maximum spacing allowed for the intermediate stiffeners of short links per AISC 341-10 [3] is based on this proposal with slight modifications. It is worth noting that a number of recent studies [31, 34] have mentioned that the shear link stiffener spacing requirements of the AISC Seismic Provisions [3] are somewhat conservative and might be relaxed if justified by further experimental research.

Malley and Popov [9] investigated the required area and moment of inertia of link intermediate stiffeners using Basler’s theory for plate girders [76] and the approach adopted by Bleich [77]. The requirements proposed by Malley and Popov [9] were not included in AISC 341 [3, 68, 69], and instead, the regular requirements of plate girder web stiffeners were recommended for determining the required moment of inertia of link stiffeners with the addition of a minimum thickness limit. Bruneau et al. [50] further discussed this issue and mentioned that the previous reasonable performances of

EBFs designed as per AISC 341 [3, 68, 69] indicate that the recommendations of Malley and Popov [9] were based on overly conservative assumptions.

Intermediate stiffeners are also required in links with flexure-dominant behavior. The comprehensive experimental study by Engelhardt and Popov [11, 64] on long links revealed that, unlike shear links, local buckling of flanges will not necessarily cause strength degradation in stiffened long links. It was concluded that placing stiffeners at a distance of $1.5b_f$ from each end of the link, while not preventing flange buckling, would limit the strength loss due to flange buckling [64]. The large scale pseudo-dynamic tests by Tsai et al. [65] indicated that such stiffeners may still be beneficial if substantial axial force is also available in the link. Engelhardt and Popov [64] also mentioned the beneficial effect of placing stiffeners outside the link region, in the brace-link-beam connection panel. Furthermore, it was concluded that intermediate links, which will experience both shear and flexural yielding, should have intermediate stiffeners at $1.5b_f$ from the link ends, and also equally spaced additional stiffeners through the link length based on the requirements of short links. These recommendations have generally been adopted by AISC 341-10 [3]. However, the provisions require no intermediate stiffeners when the link length ratio (ρ) is larger than 5.0.

Previous and recent experimental studies [9, 10, 29, 64] have demonstrated that, unlike end stiffeners, intermediate stiffeners can also be one-sided in links with various lengths. The numerical study of Daneshmand and Hashemi [13] revealed that using one-sided stiffeners can reduce the rotation capacity of intermediate and long links, however, this reduction rarely decreases γ_P^{max} below the code-specified rotation capacity. Furthermore, the reduction was reported to be more pronounced for links in the range of $1.8 < \rho < 2.2$. Nevertheless, AISC 341-10 [3] permits the use of one-sided intermediate stiffeners for links with a depth of less than 635 mm. Malley and Popov [9] demonstrated that partial depth intermediate stiffeners can also be used in shear links, provided that a concrete slab properly restrains the top flange. However, as required by AISC 341-10 [3], it is more advisable to use full-depth intermediate stiffeners welded to the web and both flanges since these stiffeners can enhance the stability of the link against flange local buckling as well as against lateral torsional buckling [7, 64, 78].

Recent tests and the probabilistic analysis of Bulic et al. [52] suggested the use of at least two couples of properly designed web stiffeners in short I-shaped links to achieve enough reliability according to Eurocode 0 [79] for the mean recurrence interval of 50 years.

Richards and Uang [51] noticed during their numerical study that some of the intermediate links failed to achieve the rotation capacity predicted by the provisions, with a maximum difference of 11%. Similar observations were also reported by Arce [23]. Richards and Uang [51] attributed the issue to the intermediate stiffener spacing requirements of the AISC Seismic Provisions [3]. As stated earlier, Kasai and Popov [75] proposed their stiffener spacing formula for short links; however, the provisions extended its use to intermediate links, without accounting for the significant moment-shear interaction that is present in the web panels of these links. Thus, it was concluded by Richards and Uang [51] that the direct use of stiffener spacing requirements of short links may be unconservative for intermediate links.

During a series of tests [23, 25, 26, 28] from 2002 to 2005 on A992 links, conducted at the University of Texas at Austin, another important issue was observed. Most of the specimens with $\rho < 1.7$ exhibited web fractures at the ends of the stiffener-to-web welds prior to any notable web buckling (Figure 3a). This was not consistent with the failure modes observed in the early studies of Popov and his colleagues (e.g. [6, 15, 18]) where web fracture occurred only after severe web buckling at locations of large deformations. The pre-buckling web fracture had also been reported previously by McDaniel et al. [24] during tests on large built-up links where it was attributed to the insufficient offset between the stiffener-to-web and the web-to-flange welds. It is also worth noting that in a number of more recent tests this type of failure mode has also been reported [14, 41]. Based on the test results, Okazaki and his colleagues [29, 31] have concluded that altering the applied loading history or the utilized stiffener detailing cannot change the link failure type from the web fracture mode. The test results however revealed that it is fruitful to terminate the stiffener-to-web weld at a distance not less than $5t_w$ from the k-line of the link section. A clear correlation between the reduced material toughness in the k-area and the occurrence of link web fracture was not established [31]. In addition to the above findings, a new stiffener detailing was also reported by Okazaki and Engelhardt [31] which can delay web fracture and provide enhanced cyclic performance. This detail consists of two-sided intermediate stiffeners which are welded only to the flanges that restrain the web by sandwiching it. Additional research regarding this detail was deemed necessary by Okazaki and Engelhardt [31].

In order to further investigate the cause of the recently observed pre-buckling web fracture failures, Chao et al. [80] conducted a detailed numerical study. Numerical results suggested that the web fractures are due to the high triaxial constraints that develop at the ends of the stiffener-to-web weld and the localized high plastic strains at these locations. Beneficial effects of welding stiffeners to both flanges, using two-sided intermediate stiffeners and avoiding large stiffener spacing were also mentioned. Similar observations were also reported in previous experimental studies [25, 31]. Chao et al. [80] also proposed a possible reason for the observed web fractures in the recent tests as opposed to the 1980s experiments. The new straightening process for structural shapes causes higher strength and reduced toughness in the k-area, along the full length of the rolled sections, unlike the old method which induced localized changes in the material properties due to work hardening. As explained by Chao et al. [80], the higher k-area strength in the new shapes prevents yielding in this region, and consequently, high plastic strains are developed in the adjacent web steel. These localized strains coupled with high stress triaxiality at the stiffener-to-web weld ends increase the possibility of ductile fracture initiation at the weld ends. Based on the numerical results, Chao et al. [80] proposed a single horizontal stiffener instead of multiple vertical stiffeners for short links and demonstrated its promising performance numerically. The sandwich stiffener detail proposed by Okazaki and Engelhardt [31] also performed well during the simulations [80].

Dusicka et al. [36] conducted experiments on isolated very short built-up I-shaped links with a different stiffener detailing in which the link web was constructed from low-yield-strength steel (with F_y of 100 MPa or 225 MPa) without any stiffeners. The test results revealed that, unstiffened links with stocky webs constructed from low-yield-strength steel can sustain extremely high cyclic rotation angles, of the order of 0.2 rad. As a result of such detailing, the failure mode was altered from that controlled by fracture at the ends of the stiffener-to-web weld to web tearing at the link end corners accompanied by web out-of-plane deformations in some cases. Bahrampoor and Sabouri-Ghomi [81] also studied the effect of using very low-yield-strength steel with F_y = 90 MPa in EBF links. Comparison between the results of one-story one-bay FE models with links constructed from regular and low strength steels revealed that the energy dissipation characteristics of links can be enhanced using unstiffened stocky webs constructed from very low-yield-strength steel.

The use of diagonal web stiffeners for shear links was studied both experimentally and numerically by Yurisman et al. [82]. Based on the results reported by these researchers it appears that the diagonal web stiffeners may provide an alternative to the commonly used vertical stiffener arrangement for short links; however, further research is essential. Chegeni and Mohebkhah [70] proposed new stiffener details for improvement of the rotation capacity of long links. Two details were suggested: placing an additional one-sided stiffener at a distance of $0.75b_f$ from the link ends; and using small one-sided diagonal stiffeners between the end stiffener and the intermediate stiffener in long links. Results of the parametric study indicated that the latter detail is more effective in improving the rotation capacity and energy dissipation of long links. However, further experiments for validating the suggested details were deemed necessary [70].

Ohsaki and Nakajima [83] investigated the optimization of stiffeners in I-shaped EBF links. These researchers used the heuristic Tabu Search algorithm for optimizing the locations and thicknesses of link stiffeners.

Stiffener requirements for built-up box links were studied by Berman and Bruneau [45-49]. The use of end stiffeners for built-up box links was recommended similar to I-shaped links. However, results reported by Berman and Bruneau [48, 49] revealed that intermediate stiffeners are only required for shear links ($\rho \leq 1.6$) in which the web depth-to-thickness (h/t_w) ratio is greater than or equal to $0.64\sqrt{E/F_y}$. For shear links with lower h/t_w ratios, flange buckling is the controlling failure mode, for which the presence of intermediate stiffeners is not effective. Berman and Bruneau [48, 49] also demonstrated that intermediate stiffeners are not beneficial in intermediate and long links where compressive local buckling of both webs and flanges controls the link performance. The required intermediate stiffener spacing for shear links with $h/t_w \geq 0.64\sqrt{E/F_y}$ has been determined by Berman and Bruneau [45] using a methodology similar to that used by Kasai and Popov [75] for I-shaped links.

In the numerical and experimental investigations of Berman and Bruneau [48, 49] external stiffeners welded to webs and flanges were considered. However, as stated above, these stiffeners are not effective in controlling flange buckling. Thus, stiffeners welded to outsides of the link webs or located inside the box section and welded to the insides of the webs can be considered in practical applications.

During the 2011 earthquakes in Christchurch, New Zealand, the first documented field fractures of EBF short links were recorded at the Christchurch hospital garage [84]. In some of the fractured links, unlike the correct detailing, the end stiffeners were not aligned with the brace flanges. This misalignment was speculated to be a probable reason for the observed fractures in the link flange and connection panel. Kanvinde et al. [84] performed in-depth numerical analyses to investigate the issue. Numerical results revealed that the misalignment had an influential effect in triggering the fractures; however, other factors, such as the imposed ground acceleration which was several times higher than the expected design value, also played a major role. The use of field-welded stiffeners and gusset plates for brace connections was suggested by Kanvinde et al. [84] for better fit-up. Another recent study by Imani and Bruneau [85] further confirmed that the severe stress concentration in the vicinity of the misalignment can initiate fracture and reduce the link rotation capacity significantly. The numerical results also demonstrated that correcting the misalignment by using a different brace section or, more conveniently, by relocating the end stiffeners can mitigate the problem [85].

3.3. Lateral Bracing

Lateral torsional buckling (LTB) can have deteriorative effects on the cyclic performance of links. The lateral bracing requirements are intended to restraint the link against out-of-plane displacement and twist to ensure stable inelastic response [3]. Although lateral bracing was provided in earlier tests [4], the importance of proper link lateral bracing was fully understood during the tests of Manheim [86] on three-story prototypes in 1982, where LTB occurred in some test specimens. As a result, lateral bracing of link ends was suggested by Manheim [86] with moment connections between the lateral braces and the link ends to increase the torsional stiffness of the link. In 1989, Hjelmstad and

Lee [87] conducted an study to investigate the lateral buckling of beams in EBFs. Based on the experimental results of five tests on propped cantilever beams with different lateral bracing schemes at the link ends and a numerical parametric study, Hjelmstad and Lee [87] concluded that providing full rotational restraints at the link ends is essential. Furthermore, it was noted by these researchers that the forces imposed on the lateral braces were much higher than the traditionally used design load of 2% of the flange yield force, P_y^{flange}. During the experimental study of Engelhardt and Popov [11, 64] on long links, strength degradation due to LTB of the link or the beam outside the link was reported for some specimens. These observations further emphasized the importance of providing strong and stiff lateral bracing at link ends. Similar to Hjelmstad and Lee [87], Engelhardt and Popov [11, 64, 88] also mentioned that the demand on link lateral braces is several times higher than the minimum load of 1.5% of P_y^{flange}, considered in the 1980s design codes [12, 72]. A minimum load of 6% of P_y^{flange} was suggested by these researchers for the design of lateral braces of short and long links. Furthermore, it was recommended that the lateral braces should frame into the link ends from only one side in order to prevent imposing excessive in-plane restraint to the link [11].

Based on the above research, the use of lateral braces at the link ends was required by the early EBF codes [12, 72, 73]. Similarly, AISC 341-10 [3] requires lateral bracing of both top and bottom flanges of I-shaped links at the link ends. The concrete slab may only provide restraint to the top flange [63, 64] and thus, explicit bracing of link ends is generally necessary.

The main advantage of built-up box links compared to I-shaped links is their significant resistance to LTB. Berman and Bruneau [47] demonstrated that lateral bracing at link ends is not necessary for typical built-up box links. This can be fruitful in cases where providing lateral bracing for links is not possible or not desired. For instance, lateral bracing of links adjacent to elevator cores or links used in bridge piers is generally cumbersome. Consequently, built-up box links are also referred to as self-stabilizing links [46].

3.4. Connections

In EBF systems, brace-to-beam and link-to-column connections attracted particular research attention due to the high level of demands on these joints. Gusset plate buckling at the brace-to-beam connection was observed in the full-scale tests of Roeder et al. [89] and Foutch [90]. Engelhardt and Popov [11, 64] proposed and tested modified gusseted and directly welded brace-to-beam connections. Satisfactory results were reported for all of the proposed connection details; however, the directly welded connections were found to be more advantageous in controlling LTB of the beam segment outside the link. Considering that the braces are expected to remain essentially elastic during severe loadings, most of the ductility requirements which are required for braces of special CBFs are not mandatory for EBF braces and their connections, based on the AISC Seismic Provisions [3].

Tests by Hjelmstad and Popov [6, 7], and Malley and Popov [9, 18] in the 1980s suggested that fully welded link-to-column connections can exhibit satisfactory behavior during severe loading scenarios. In these tests short links with different combinations of complete joint penetration (CJP) groove welds and fillet welds for the flanges and the web were considered. In contrast, details with welded flanges and bolted web connections tested by Malley and Popov [9, 18] showed significant bolt slippage which led to premature flange fractures.

Engelhardt and Popov [11, 64] investigated the behavior of link-to-column connections for the case of long links. Details employing CJP groove welds at the flanges and either a fully welded shear tab or a CJP groove weld at the link web (Figure 7a and b) were considered owing to their acceptable performances in previous tests on short links (e.g. [7, 9]). Premature link flange fracture was observed for these specimens at early stages of loading, and thus, modified details (e.g. Figure 7c and d) were proposed and tested by these researchers. Furthermore, a link-to-column connection with all-around fillet welds (Figure 7e) was also studied. Although some of the modified details sustained high rotations prior to failure (especially the cover plate (Figure 7d) and the all-around fillet weld (Figure 7e) details) due to the doubtful performance of the tested specimens as well as the observed brittle failure modes, it was concluded that the use of long links attached to a column should be avoided. Engelhardt and Popov [11, 64] also tested two short links which were connected to the web of an I-shaped column through continuity plates (Figure 7f). A similar detail was previously studied by

Malley and Popov [9, 18] and acceptable results were reported with minor flaws compared to the link-to-column-flange connections. However, the tests of Engelhardt and Popov [11, 64] revealed that the link-to-column-web connection is prone to premature link flange fracture and should be avoided. Based on these studies, the 1992 AISC Seismic Provisions [91] limited the use of link-to-column connections to short links, recommending details such as Figure 7a and b.

In the experiments of Tsai et al. [65] CJP groove welds at the flanges and fillet weld at the web were used to connect short links to box columns. Premature fractures at very low link rotations (sometimes less than 0.005 rad) were observed in the link flange welds. Although a number of modified details performed remarkably better, the observed fractures gave cause for concern.

Ghobarah and Ramadan [20, 92] proposed an extended end-plate link-to-column connection (Figure 7g) in 1994. In this approach the link is shop-welded to the end-plate using all-around fillet welds and the end-plate is then field-bolted to the column flange. Although some of the specimens exhibited bolt or flange weld fractures, it was concluded that properly designed extended end-plate link-to-column connections would remain elastic during severe loadings and demonstrate a similar performance to that of a fully welded connection. In a recent study by Dusicka and Lewis [35] on end-plate link-to-column connections, different stiffening details were investigated to reduce the demand on link flanges in the vicinity of the link-to-end-plate welds, which was found to be a potential region for brittle failure [20, 31, 92]. These details included using an additional pair of stiffeners in the first web panel which were either parallel to the web and connected to the web or within a small distance from it (Figure 7h), or angled from the link end towards the web, or curved stiffeners made from round HSS. Promising results were reported based on FE analyses for the first detail with stiffeners parallel to the web which shifted the failure mode from the link flange fracture to fracturing of the web. The results were further confirmed through a number of experiments on long links. In 2016, Pirmoz et al. [93] numerically studied the behavior of extended end-plate link-to-column connections, with rib stiffeners, in contrast to the suggestion of Ghobarah and Ramadan [20, 92]. Although the study did not address issues such as low-cycle fatigue or material fracture, the promising performance of this link-to-column connection was demonstrated.

An experimental study was undertaken by Tsai et al. [94] on the behavior of link-to-box column connections in 2000. In these tests, short links were connected to box columns using CJP groove welds at the flanges and either a fully welded shear tab or a CJP groove weld at the link web. Although some of the specimens used the improved weld access hole detail suggested by Mao et al. [95], premature fracture was observed in all connections at the link flange in the vicinity of the groove weld. The urgent need for research on this issue was emphasized. Consequently, an extensive study was initiated at the University of Texas at Austin in the early 2000s [26]. These tests along with those performed previously at the National Taiwan University by Tsai et al. [65, 94] were the first experiments on large-scale link-to-column connections with realistic details. Okazaki et al. [30] tested 12 welded link-to-column specimens considering short, intermediate, and long links with four different connection details using the old loading protocol of AISC 341-02 [68]. Pre-Northridge as well as other moment connections which adhered to the recommendations of FEMA 350 [96] were considered. Recently developed welded moment connections with free flange [97] (Figure 7i) and no access hole (Figure 7j) [98] details were also tested. Except one specimen, all of the tested connections failed due to abrupt fracture of the flange near the groove weld prior to developing the required level of plastic rotation. The performance was inferior for the pre-Northridge and FEMA 350 [96] details which only developed about half of the required γ_P. Results proved that the connections which are suitable for MRFs may not necessarily perform well as link-to-column connections. It was concluded that link-to-column connections are prone to brittle failure regardless of the link length ratio and should therefore be avoided until a satisfactory detail is developed.

In a subsequent study, Okazaki et al. [33] tested an additional 12 welded link-to-column specimens to investigate the effect of loading history and to study other details such as the all-around fillet weld detail (Figure 7e) and a new reinforced detail (Figure 7k) referred to as the supplemental web doubler connection. The test results seemed inconclusive as regards the effect of the loading protocol. On the other hand, excellent performance was reported for most of the specimens with the fillet welded and the newly proposed supplemental web doubler details. A summary of the experimental results for these two connections reported by Okazaki et al. [38] indicates their high potential for practical applications. Several design and welding details for the all-around fillet weld connection were also outlined by these researchers. A step-by-step design procedure for the supplemental web doubler connection was developed by Hong et al. [99] through a series of nonlinear FE analyses and substantiated by experiments [38].

Two numerical studies were conducted recently to propose alternatives for reducing the demand in critical regions of welded link-to-column connections, i.e. at the link flanges in the vicinity of groove welds. Prinz and Richards [100] studied reduced web section links while Berman et al. [101] investigated reduced flange section links. The reduced web section links with perforated webs did not perform well in the simulations while the reduced flange section links were found to be a potential solution, pending testing required for further validation.

The AISC Seismic Provisions [3] do not require qualification testing for the link-to-column connection of a short link, provided that the connection is reinforced with haunches or other proper details which prevent yielding in the reinforced segment adjacent to the column (e.g. Figure 7*l*). Although this approach is found to be effective in MRF connections, no research is available at the time which proves the reliable performance of such a reinforced detail specifically in EBF link-to-column connections. The commentary on AISC 341-10 [3] mentions the promising performance of the supplemental web doubler detail proposed by Okazaki et al. [33]; however, it encourages designers to configure EBFs to avoid link-to-column connections entirely. The use of built-up box links in EBFs with link-to-column connections has not been studied explicitly nor addressed by the specifications.

4. Numerical Modeling of Links

General FE techniques which model links using shell or solid elements can reasonably simulate the behavior of these members under monotonic and cyclic loadings (e.g. [13, 43, 48, 51, 53, 55, 61, 80, 84, 85, 93, 99-103]), specifically if the strength degradation related with low-cycle fatigue-induced fractures does not need to be captured. However, due to the complexity and computational burden of these methods they are not typically used for nonlinear time-history analysis, and instead, simplified approaches are utilized which model a link through a combination of line elements, nodal constraints, springs, or plastic hinges. Although the detailed description and evaluation of the latter are out of the scope of this study, a brief outline of each method is provided here for the sake of completeness.

In 1977, Roeder and Popov [4] proposed a sandwich beam model for links where shear was resisted by the web and moment (through uniaxial stresses) by the flanges with bilinear responses. The method was intended to be used for modeling shear yielding links with small end moments. A very simple model was later proposed by Yang [104] in which the shear link behavior was simulated using a truss member with calibrated uniaxial strength. In 1983, a finite element model based on a stress resultant formulation was developed by Hjelmstad and Popov [6] which utilized a moment-shear yield surface. The method was not suitable for link modeling since strain hardening effects were not incorporated. A simplified approach was proposed in 1987 by Whittaker et al. [16] for short links where regular flexural elements with moment hinges were calibrated to exhibit moment capacities corresponding to the nominal shear strength of the link.

Ricles and Popov [105, 106] developed an approach in which a link was represented by a linear elastic beam with a nonlinear zero-length hinge at each end. Although the numerical results obtained using this approach were fairly accurate, the calibration and programing of the method was quite complex [107].

A modeling procedure was developed by Ramadan and Ghobarah [107] on the basis of the theory proposed by Ricles and Popov [105, 106]; however, with simpler end hinges and the ability to be conveniently incorporated into regular analysis programs. The model was calibrated using the results of experiments conducted at the University of California at Berkeley in the 1980s on links constructed from A36 steel (e.g. [6, 15, 19]). The accuracy of the method was demonstrated through a comparison with test results. In 2003, Richards and Uang [67] modified the link model proposed by Ramadan and Ghobarah [107] in order to improve its accuracy in predicting the behavior of links constructed from A992 steel. The proposed element was later used in an extensive parametric study [66, 67].

A simplified approach was recently used by Khandelwal et al. [108] in which the behavior of short links was simulated using a rectangular truss system with vertical rigid bars and elastic horizontal bars and a nonlinear diagonal spring.

A three element link model consisting of a central beam element and concentrated hinges at the ends was introduced by Rossi and Lombardo [109] in 2007 and extensively used in the numerical studies of Bosco and her colleagues [110-113]. Although suitable for initial analysis, the model was not able to properly capture the cyclic response of links. For instance, non-zero stiffness values were provided by this model even at very large deformations [114]. To overcome these deficiencies, an enhanced model of this element was recently developed by Bosco et al. [114] in which the responses of flexural and shear hinges were defined using the uniaxial material model of Zona and Dall'Asta [115]. The model was separately calibrated for short and long links using a large body of test data [6, 11, 25, 26, 32]. The effectiveness of the approach was demonstrated for short, intermediate, and long links by comparing numerical and test results. It is worth noting that in all of the above-mentioned models the effects of axial force and strength degradation on the nonlinear behavior were neglected.

In studies by Malakoutian et al. [116], O'Reilly and Sullivan [60], and Kanvinde et al. [84] the OpenSees [117] analysis platform was utilized where a beam-column element with distributed plasticity and additional independent nonlinear shear springs located at the element ends was used for link modeling. Various different material models were adopted in each of these studies. Moghaddasi B. and Zhang [118] used the beam element of OpenSees [117] with zero-length moment hinges at the ends as well as four parallel translational springs at each end for simulating the behavior of shear links. This method was also used in the numerical study of Dastmalchi [62].

Other more sophisticated elements have been developed for modeling steel members with dominant shear yielding behavior, that are also applicable to EBF links. On the basis of the method proposed by Ricles and Popov [105, 106], Kazemi and Erfani [119, 120] developed a model with a combined shear-flexural inner hinge and two rigid beams on its sides. Numerical results were compared to the test data reported by Kasai and Popov [10] to demonstrate the accuracy of the proposed approach. An improved version of this model with an axial-shear-flexural hinge was recently developed by Kazemi and Hoseinzadeh Asl [121]. A mixed-formulation (or force-based) element was proposed by Saritas and Filippou [122] with independent displacement, stress, and strain fields, where the displacement field was based on Timoshenko's beam theory. As a result, shear locking was avoided and mesh refinement was deemed unnecessary. Since the axial-shear-flexural interaction was captured using material data only, the model did not need further calibration for different loading and boundary conditions, unlike most of the methods with concentrated hinges. Comparison between numerical and experimental results revealed the accuracy and robustness of the method. Papachristidis et al. [123] have also proposed a force-based element; however, considering a three-dimensional state of stress and taking into account the interaction of axial, shear, flexural, and torsional actions. The kinematics of the model were obtained through the natural-mode method. Fairly accurate results were reported using this element when compared with the results of previous tests and also accurate FE analyses.

5. EBF Systems

5.1. Characteristics and Capacity Design Approach

This section provides general insight into the behavior and design philosophy of EBF systems, while the subsequent section is devoted to more in-depth discussion about the research on the seismic performance of these structures. As mentioned by Popov and Engelhardt [78], the use of eccentric bracing for resisting wind loads was well recognized even in the 1930s [124]; however, the use of this system for seismic applications was proposed in the 1970s in Japan [1, 2]. The studies of Roeder and Popov [4, 5, 125] in the late 1970s pioneered the research on EBFs in the United States. The cyclic loading tests of Roeder and Popov [4, 125] and Manheim [86] on a reduced-scale three-story one-bay EBF, the pseudo-dynamic tests on a full-scale six-story two-bay EBF as a part of the U.S.-Japan Cooperative Program in Earthquake Engineering reported by Roeder et al. [89] and Foutch [90], the shaking table tests of Whittaker et al. [16, 126, 127] on a scaled replica of the same six-story EBF, and the pseudo-dynamic tests of Balendra et al. [128] as well as the early analytical studies of Hjelmstad and Popov [129], Ricles and Popov [105], and Popov et al. [130] on EBF systems, all confirmed that this system could be effectively used for seismic applications. The EBF system was utilized in several major applications (e.g. [131-133]) shortly after these studies. Short links in EBF systems are preferred since they will provide higher stiffness, strength, and ductility over intermediate and long links [78, 130]. Nevertheless, Popov and Engelhardt [78] demonstrated that using links that are too short will impose unmanageably high rotation demands on links. It was also noted by these researchers that the stiffness and consequently the fundamental period (T) of EBF systems can be adjusted simply by altering the link length.

The capacity design approach [78, 130] is utilized in the EBF design to ensure concentration of yielding in links while keeping other members essentially elastic. First, the links are sized and then, other members are designed to resist the loads generated by the yielded and strain hardened links. Plastic design methods for EBF systems were initially proposed by Roeder and Popov [4], Manheim [86], and Kasai and Popov [15] in the 1970s and 1980s. An allowable stress design method was also developed by Teal [134] in the late 1970s. In the current practice, an elastic analysis is typically conducted during the design of EBF systems. Design examples for EBF systems can be found, for instance, in Bruneau et al. [50], the AISC Seismic Design Manual [135], Popov et al. [130] and Becker and Ishler [136].

The overstrength of links (Ω) must be considered in the design of members other than the links to estimate the maximum loads that might be imposed on these members by the fully yielded and strain hardened links. Design specifications (e.g. [3, 137]) provide overstrength values which may be different for beams, braces, and columns. The AISC Seismic Provisions [3] suggest a value of Ω = 1.25 for I-shaped links during the capacity design of EBF braces, which is lower than the traditional value of 1.5 (Section 2.1), mainly for considerations of economy. It is worth noting that a recent numerical study by Yiğitsoy et al. [103] demonstrated the adequacy of the current overstrength provisions for braces.

The design of a beam in an EBF system is often problematic since it is generally under high axial force and high bending moment. Increasing the size of the beam will not benefit the design, since the link-induced forces will also increase. Several methods such as using short links and braces with moment connections possessing inclination angles above 40^{o} can reduce the demand on beams [50, 78, 88, 130, 138]. In addition, an EBF configuration proposed by Engelhardt and Popov [88] (Figure 1e) can minimize the beam axial force at the cost of using larger links and reducing the system redundancy. A similar EBF configuration was tested by Yang [104] in the early 1980s. In order to aid this design difficulty, a lower overstrength factor is used in AISC 341-10 [3] for the capacity design of beams when compared to that of braces. This is justified by considering the positive effect of the composite floor on beam performance as well as the fact that limited yielding of a beam will not negatively affect the behavior of an EBF system, as long as the stability of the beam is assured [11, 64, 139]. Based on extensive FE analyses on EBF sub-assemblages, Yiğitsoy et al. [103] suggested that the overstrength value recommended in AISC 341-10 [3] could be reduced even further in the design of an EBF beam with I-shaped links, provided that the demand-to-capacity ratio of the beam is kept below unity. For other cases a maximum unbraced length recommendation was developed. The FE analyses demonstrated that the probable yielding of the beam due to the above relaxation would not be detrimental and would only affect the brace end moment.

Early analytical and experimental studies [16, 105] demonstrated that all of the links above the level of the column under consideration would not develop their maximum shear forces simultaneously. Based on this observation a lower overstrength factor was suggested by AISC 341-10 [3] for the capacity design of columns in EBFs of three or more stories of bracing, when compared with the Ω value used in the design of brace members. Columns also need to be checked for the amplified seismic axial force of $\Omega_o P_{EQ}$, where Ω_o is the structural overstrength factor taken as 2.0 per ASCE 7-10 [140] and P_{EQ} is the column axial force generated by the code-specified earthquake loads. It is important to note that drift-induced flexural forces are generally neglected in the design of columns as permitted by AISC 341-10 [3].

During the design process, it is necessary to estimate link end moments in order to determine the internal force distribution after formation of the expected plastic mechanism. For links located in the middle portion of floor beams (internal links) the end moments will almost be equal throughout a seismic loading. On the other hand, for links connected to columns (external links) the end moments will not be identical in the elastic range. However, early studies of Kasai and Popov [10, 15] proved that for most cases these moments would equalize as the link goes through large plastic rotations. Thus, in both cases, the link end moments can be readily estimated using equilibrium. The only exception reported by Kasai and Popov [10, 15] was the case of external links with $\rho \leq 1.3$, for which the end moments did not equalize at the ultimate state. For such cases, recommendations for moment distribution were developed by these researchers. Kasai and Popov [15] also demonstrated that it is advantageous to avoid EBF configurations with inactive links (e.g. Figure 1f) in which only one of the links located at one end of a brace would dissipate most of the energy (and thus become active) while the other would not contribute notably to the energy dissipation of the system (and thus remain inactive).

It is worth mentioning that early multi-story tests [16, 89, 90] demonstrated that, in some cases, the plastic behavior could be concentrated in the first story links leading to the development of a soft story mechanism. Popov et al. [141] attributed this issue to the incorrect proportioning of the links along the height of the EBFs in the above-mentioned tests. The static pushover and dynamic time-history analyses of Kasai and Popov [15], Ricles and Popov [105], Ricles and Bolin [142, 143], and Popov et al. [141] revealed that in order to achieve a reasonable distribution of link inelastic action throughout an EBF height, all of the links need to have uniform capacity-to-demand ratios.

5.2. Research on Seismic Performance

An enormous amount of research on the seismic performance and design of EBF systems has been conducted in the past decades and important aspects of this research are briefly discussed in this section.

In a series of studies by Koboevic and her colleagues [56, 57, 138, 144] in late 1990s and 2010s the seismic behavior of low-, mid-, and high-rise EBF systems was investigated under several earthquake records via nonlinear time-history analysis. Similar to previous findings [11, 64, 139], limited yielding in EBF beams was found acceptable provided that braces were capable of resisting the additional moments. The use of a higher overstrength factor in the capacity design of upper tier columns was recommended, which is the approach used in CSA S16-14 [17]. The importance of drift-induced column flexural forces was also demonstrated in the above studies. The last observation was reported previously by Kasai and Han [145] as well. An iterative design methodology based on selecting appropriate earthquake records and performing time-history analyses was also outlined by these researchers [138, 144]. Although all of the links in the studied EBFs had similar capacity-to-demand ratios, the numerical results [56, 57] revealed that the energy dissipation might be non-uniform along the EBF height and more concentrated in the first and last story links.

The seismic behavior of six-story one-bay EBFs with long links under several earthquake records was studied by Tirca and Gioncu [146] in 1999. It was concluded that long links should be used with caution and avoided as much as possible due to their poor performance as observed in some of the studied cases.

The effect of shear-moment interaction in the plastic design of EBFs was studied by Mastrandrea et al. [147, 148], where a procedure for determining the ultimate link shear force and link end moments was proposed for a given collapse mechanism. In companion studies [149, 150], a design methodology for EBFs was developed which ensures formation of a global mechanism and prevents partial or local collapse mechanisms. The behavior of EBFs designed with this method was compared to that of EBFs designed using a simplified method proposed by Kasai and Han [145] and satisfactory results were reported [149]. Further nonlinear static and dynamic analyses were conducted to demonstrate the effectiveness of the proposed method in distributing the inelastic link action through an EBF height while preventing the occurrence of undesirable failure modes [150]. The method was also verified via incremental dynamic analysis (IDA) in a study by Mastrandrea et al. [151] in 2013. The above-mentioned plastic mechanism control theory was recently implemented in the design of dual EBFs (i.e. systems composed of EBFs and MRFs) by Montuori et al. [152].

In 2006, Köber and Ştefãnescu [153] compared the seismic behavior of EBFs designed on the basis of four different specifications including AISC 341-02 [68] and Eurocode 8-2002 [154] using nonlinear time-history analysis. It was reported that AISC 341-02 [68] required heavier sections for columns, beams, and braces. In a companion study by these researchers [155] the positive and negative aspects of placing webs of I-shaped braces normal or parallel to the plane of EBFs were investigated. Recommendations regarding each configuration were given considering seismic performance and required amounts of steel. In 2009, Köber and Ştefãnescu [156] investigated the effects of using different structural details at the plastic hinge locations near the bottom of the first story columns in EBFs. Numerical results pointed to the advantages of utilizing a detail with reduced column flanges at these locations.

The effects of frame geometry on the seismic behavior and weight of chevron EBFs were investigated in 2008 by Özhendekci and Özhendekci [157]. It was concluded that EBFs with shear links performed better than those with intermediate and moment links. Furthermore, the use of longer shear links and shorter intermediate links was found to be advantageous in terms of seismic performance. However, it was demonstrated that for shear links, the frame weight would also increase along with the link length.

A novel performance-based plastic design (PBPD) methodology for EBFs was proposed by Chao and Goel [158, 159] in 2005. The method, which uses an energy-balance criterion and provides a design base shear for a given hazard level, global yield pattern, and target drift, is a direct method in the sense that it does not require any assessment after the initial design. A procedure for the height-wise distribution of this lateral force was also proposed based on the results of extensive nonlinear dynamic analyses. The results revealed that the EBFs designed using the new approach can satisfy expected performance objectives and perform better than those designed using conventional methods, without any notable increase in material usage. It is worth noting that an application of a performance-based approach for the design of a 97.6 m tall EBF system located in the United States was reported by Sabol and Nishi [160] in 2011.

In 2013, Sullivan [161] developed a direct displacement-based design (DDBD) method for EBFs with the aim of overcoming the deficiencies generally attributed to the force-based design approach. The method replaces a multi-degree-of-freedom (MDOF) system with an equivalent single-degree-of-freedom (SDOF) structure. A flow chart of this iterative design process as well as a design example were also provided. Sullivan [161] and O'Reilly and Sullivan [60] demonstrated the effectiveness of the method through time-history analyses of EBFs designed using DDBD. The studied EBFs exhibited lower-than-expected ductilities ($\mu < 3$), and thus it was noted that the codified response modification factors (i.e. R factors) are generally unconservative. Furthermore, using a unique R factor and a single displacement amplification factor (C_d) factor for all EBFs was found to be inappropriate since the ductility (μ) would tend to reduce as the EBF height increases. The use of longer shear links was recommended by Sullivan [161] similar to Özhendekci and Özhendekci [157].

The effect of height-wise distribution of the demand-to-capacity ratio of links was studied by Rossi and Lombardo [109] in 2007 using IDA. Partial collapse mechanisms, especially in the upper stories, were observed in all of the cases which had scattered height-wise link demand-to-capacity ratios. Furthermore, the *R* factors for mid- and high-rise EBFs as well as EBFs with long links were found to be considerably lower than those proposed by different building codes [68, 162]. A subsequent study was undertaken by Bosco and Rossi [111] in 2009. The results of extensive nonlinear incremental dynamic analyses revealed that the traditional method of designing EBFs considering capacity design principles as well as providing uniform demand-to-capacity ratios for links would not necessarily ensure proper distribution of link inelastic action through the height, especially for mid- and high-rise EBFs. To have a better prediction of the seismic performance of EBFs, a new parameter, called the damage distribution capacity factor, was introduced by Bosco and Rossi [111]. It was demonstrated that, considering both of the demand-to-capacity ratio and the damage distribution capacity factor, it is possible to accurately predict the collapse mechanism of an EBF system.

In 2013, Bosco and Rossi [112] proposed a design procedure for dual EBFs to overcome the deficiencies observed in their previous studies regarding regular EBFs. In this approach, EBF links are the main energy dissipating mechanisms while MRFs provide lateral stiffness during inelastic behavior. Nevertheless, to have a more cost-efficient design, limited yielding was also permitted in MRFs at beam ends, bottom ends of the first story columns, and top ends of upper story columns. In a companion numerical study by Bosco and Rossi [113] it was demonstrated that dual EBF structures designed based on the proposed methodology perform better than those designed according to conventional methods. In addition, an expression for determining the *R* factor for dual EBFs was proposed which was dependent upon the link rotation capacity, γ_P^{max}, and gave *R* factors ranging from 4.0 (for long links) to 7.5 (for short links). A similar formula for the *R* factor of EBFs, ranging from 3.5 (for long links) to 5.0 (for short links), was suggested more recently by Bosco et al. [110]. Furthermore, the use of modal response spectrum analysis instead of equivalent lateral load methods for the design of mid- and high-rise EBFs was emphasized by Bosco et al. [110]. In a very recent study by Bosco et al. [163] the Eurocode 8 [154] EBF design procedure has been thoroughly reviewed and the important drawbacks and discrepancies are highlighted.

The seismic column demands of EBFs were studied by Richards [164] in 2009. The demands from nonlinear dynamic analyses were compared to the amplified seismic axial force ($\Omega_o P_{EQ}$) according to IBC 2006 [165] or ASCE 7-10 [140] considering the amplification factor (or structural overstrength) of 2.0. The numerical results revealed that designing EBF columns solely using the amplified seismic axial force can be quite unconservative for upper tier columns of tall EBFs while overly conservative for columns at the base of these structures. A similar conclusion was reported by Kuşyılmaz and Topkaya [166] in 2013. An average value of 3.25 was reported by these researchers for the structural overstrength factor which is well above the codified value of 2.0. The results reported by both studies may indicate some concern regarding the EBF column design; however, as noted by Kuşyılmaz and Topkaya [166], recalling that the capacity design principles should also be considered in the EBF column design it is anticipated that the use of $\Omega_o = 2$ would not yield substantial underestimations in the EBF column design process.

The appropriateness of using the displacement amplification factor, C_d, equal to 4.0 per ASCE 7-10 [140], which is directly used for estimating the plastic link rotation angle, γ_P, was studied by Richards and Thompson [58] in 2009. Nonlinear dynamic analyses of a large set of EBFs revealed that a factor of $C_d = 4$ can underestimate γ_P for links of low-rise EBFs while overestimating the plastic rotations of links in mid- and high-rise EBFs. Although calibrated C_d factors were proposed, the researchers pointed out that the study was inadequate to recommend factors for general design. To further investigate this design deficiency, a numerical study was undertaken by Kuşyılmaz and Topkaya [59] in 2015 and it was demonstrated that the C_d factor of 4.0 may result in significantly unconservative estimates of γ_P in low-, mid-, and high-rise EBFs. Results of the inelastic time-history analyses were used to develop a nonlinear relation which provides C_d values ranging from 8.0 (in the lower stories) to 5.0 (in the upper stories). In a subsequent study by Kuşyılmaz and Topkaya [167] nonsimulated collapse analyses revealed that EBFs designed in accordance with the U.S. provisions [3, 140], considering an R factor of 8.0, have higher collapse probabilities than expected.

Consequently, two distinct design modifications were proposed to reduce this probability. The first one was a modification to the C_d value based on [59] and the second one was a modification to the R factor, where a value of 4.0 was recommended.

In a recent study by Speicher and Harris [168] the seismic performance of six EBFs designed using IBC 2012 [169] were assessed based on ASCE 41-06 [170] using static and dynamic, linear and nonlinear analyses. The correlation between ASCE 7-10 [140] and ASCE 41-06 [170] in terms of the anticipated performance level was also studied. Numerical results once again indicated the possibility of concentration of inelastic action in a limited number of links in a properly designed EBF system. In addition, it was observed that the linear assessment methods given in ASCE 41-06 [170] are less conservative than the nonlinear assessment procedures. However, it was unclear which of these procedures is more representative of the actual behavior, since the code does not consider the effect of loading history in the assessment process and acceptance criteria. Speicher and Harris [168] mentioned that the responses of links under earthquake loadings were mostly one-sided with a ratcheting approach towards large rotations, and thus, higher rotation capacities might be anticipated for links compared to the codified limits. Consequently, the need for link assessment criteria which are based on cumulative demands was highlighted.

Studies have also been undertaken recently to provide simple relations for estimating the fundamental period as well as the stiffness of EBF systems. In 2010, Richards [171] derived a simple relation for predicting the lateral stiffness of an EBF story based on the design story shear, frame geometry, and beam depth. A comprehensive study was conducted in 2015 by Kuşyılmaz and Topkaya [172] to improve the accuracy of the formula available in ASCE 7-10 [140] for estimating the fundamental period (T) of EBFs, which first appeared in UBC 88 [72] and has not been calibrated since then. A hand-method for estimating T was first formulated and its accuracy was demonstrated by comparing the results with data obtained through an extensive parametric study and also with data available in the literature. A simple period-height relation for EBFs was then developed. The results obtained by this expression were compared to the apparent (measured) periods of actual EBF buildings, reported by Kwon and Kim [173], and acceptable conformity was reported. In a similar study, Young and Adeli [174] studied the effect of building irregularities on the fundamental period of

EBFs. It was demonstrated that the formula given in ASCE 7-10 [140] can yield overly conservative estimates of T for these systems. The results from analyses of 12 properly designed EBFs as well as the data available in the literature were combined to propose a three-variable power expression for predicting T, which included the effects of EBF building irregularities. The study also confirmed that regular EBF buildings tend to have a longer fundamental period compared to EBF buildings with irregularities. The accuracy of the proposed formula was demonstrated using the analytical periods compiled by Tremblay [175] and the apparent periods reported by Kwon and Kim [173].

The effect of the variability of steel material in the seismic performance of EBFs was studied by Badalassi et al. [176] in 2013. The numerical results revealed that the variability of the steel material did not have a major effect on the failure probability of the studied structures. In addition, the capacity design requirements of Eurocode 8 [177] were found to be appropriate. It is worth mentioning that in some of the studied EBFs by Badalassi et al. [176] two floor beams were used in each level of the EBFs to avoid interaction between the floor deck and the link. The coupled beam sustained the gravity loads while the main beam contained the link and carried the seismic loads. This approach is generally attributed to Perretti [178].

The seismic reliability of EBFs was investigated recently by Lin et al. [179] by means of nonlinear dynamic analyses, considering far-fault and near-fault earthquakes, based on the guidelines of FEMA 356 [180]. It was concluded that EBFs (particularly low-rise EBFs) have lower failure probabilities than MRFs. The reduction was more pronounced when far-fault ground motions were considered. The behavior of EBFs under near-fault earthquakes was also studied by Eskandari and Vafaei [181] In 2015. Even though near-fault earthquakes were found to be somewhat more destructive, the probability of low-cycle fatigue-induced premature web fractures were observed to be higher for the case of far-fault ground motions. Nevertheless, it was also concluded that EBFs are suitable systems to be used in near-fault regions owing to the fact that the γ_P values recorded for all links were in the acceptable range based on FEMA 356 [180] requirements for the life safety performance level.

The application of the reduced beam section (RBS) connection in dual EBFs with long external links was investigated in 2011 by Naghipour et al. [182]. Results of nonlinear static (pushover) analyses on four-, seven-, and ten-story dual EBFs revealed that the RBS connection can increase the ductility of the system (by about 10%) and reduce the demand on link-to-column connections by moving the hinge location away from the column face. The RBS connection was however recommended only for long links with shallow sections.

The seismic performance of a special type of EBF, hereafter referred to as HS-EBF, in which links are constructed from conventional steel while other members are made from high strength (HS) steels with F_y in excess of 345 MPa, was investigated by Lian et al. [39] in 2015. Results of a cyclic test on a scaled one-story one-bay chevron HS-EBF conducted by these researchers were used to validate their numerical modeling approach. The results of the subsequent analyses under cyclic loading and three ground excitations revealed a general similarity between the performances of HS-EBFs and equivalent conventional EBFs. However, better load carrying capacity and lower material consumption was reported for the HS-EBFs while slightly higher ductility and energy dissipation was indicated for the EBFs. The inter-story drifts and plastic link rotations were also found to be lower in the studied EBFs. Furthermore, a maximum height limit of sixteen stories was recommended for HS-EBFs in order to achieve better seismic performance.

A number of recent studies have focused on the optimization of EBF systems. In 2013, Gong et al. [183] presented a genetic algorithm based structural optimization technique for EBFs. A successful application of this optimization method was demonstrated for a hypothetical three-story EBF building; however, this came with the cost of an excessive computational burden (of the order of days). In a more recent study, Karami Mohammadi and Sharghi [184] developed an optimum design technique for EBFs based on the concept of uniform deformation theory. These researchers applied the method to three-, five-, and ten-story EBFs and demonstrated that the optimized EBFs have lower weights and better seismic performances compared to those of regularly designed EBFs.

There are also a limited number of studies regarding the effect of concrete slabs on the structural response of EBF systems. In 2013, Danku et al. [185] investigated the issue in three EBFs and three dual EBFs with four, eight, and twelve stories through nonlinear static (pushover) analyses as well as IDA considering seven earthquake records. The models were calibrated using the outcomes of a companion experimental study [37]. The numerical results revealed that composite action can increase the system stiffness and reduce the structural drifts and rotation demands on links and lead to a more optimum EBF design. For the case of dual EBFs, inelastic action was reported to be mainly concentrated in the links. Furthermore, the *R* factor for the steel EBFs and the steel dual EBFs was found to be about 6.0 (similar to Eurocode 8 [137]); however, a lower *R* factor (between 3.5 to 6.0) was reported when composite action was incorporated in the models. Prinz and de Castro-e-Sousa [186] also investigated the effect of concrete slabs on the EBF behavior in 2014. Two three-story EBFs with and without concrete slabs were modeled via FE method. The results of dynamic analyses under two earthquake accelerations confirmed the findings of Danku et al. [185] regarding the effects of concrete slabs on stiffness, inter-story drifts, and rotation demands of EBFs, especially for the case with long links, where a reduction of 35% was reported in the residual drifts because of composite action. Although the link rotation demands were lower in the composite models, Prinz and de Castro-e-Sousa [186] observed that the link damage accumulation was rather independent of the presence of concrete slabs. This increase of plastic demand in the composite models was attributed to the shift in the neutral axis which in turn produced higher strains in the bottom flanges of the links.

6. Special Topics

Beside the topics covered in the previous sections, some special and emerging topics related with EBF systems are also available in the literature, and these are briefly discussed here.

6.1. EBFs with Vertical Links

Although one of the earliest experimental studies on EBFs was conducted on frames with vertical links (also known as inverted Y-braced EBFs; Figure 1d) by Tanabashi et al. [2] in 1974, the use of horizontal links in EBFs has become more popular through the years. Compared to their horizontal counterpart, vertical links have an easier post-earthquake repair process. In addition, they can be conveniently used for the seismic rehabilitation of existing structures. Furthermore, their use can be advantageous in cases where the floor girders are required to remain elastic due to the presence of very large gravity loads. On the other hand, proper lateral bracing of vertical links can be difficult in certain cases. Early static, cyclic, and dynamic loading experiments by Seki et al. [187] and Vetr [188] on single- and multi-story EBFs with vertical shear links revealed that this system can exhibit a very ductile and stable behavior during an intense loading, provided that proper lateral bracing for the link ends is available. The results of a numerical study by Fehling et al. [189] in 1992 further emphasized the importance of lateral bracing for vertical links. A force equal to 1/50 of the link shear force was deemed adequate for the design of these lateral braces. In a subsequent study by Bouwkamp and Vetr [190] a relation for limiting the length ratio of short (or shear) vertical links was proposed which was dependent upon the ratio of the link end moments.

The advantages of hybrid vertical links with low strength web and high strength flanges were demonstrated by Shinabe and Takahashi [191] in 1995. Similarly, the FE analyses of Saedi Daryan et al. [192] revealed that utilizing vertical links constructed from very low-yield-strength steel with F_y of about 100 MPa could increase the energy dissipation of an EBF notably while reducing the probability of local buckling.

The concept of EBFs with double vertical links, originally mentioned by Fehling et al. [189], was recently investigated numerically as well as experimentally by Shayanfar et al. [193, 194] and promising results were reported. This concept can be advantageous in cases where the dimensional limitations of the floor beam in an existing structure do not allow the use of a single large-size vertical link for seismic rehabilitation.

Through extensive nonlinear static and dynamic analyses, Dicleli and Mehta [195, 196] investigated the behavior of EBFs with vertical shear links built from compact HP sections. Results revealed that such systems can combine the advantages of MRFs and CBFs during an earthquake while experiencing less damage and eliminating the negative aspects of each system.

In 2012, Shayanfar et al. [197] tested a composite vertical shear link in which a steel link was partially encased in reinforced concrete. Results revealed that the concrete in these links can delay the web buckling and increase the shear strength and energy dissipation of the specimens significantly. Nevertheless, the behavior of the composite link coincided with that of the bare steel link when the concrete portion was fully damaged at later stages of the loading.

A novel two-stage seismic load resisting system was developed by Zahrai and Vosooq [198] in 2013 which combined an EBF system with vertical shear links with a knee braced frame. During a moderate earthquake, only the vertical links dissipate energy while in the case of a strong ground motion, the plastic deformations in the vertical links are limited to a certain extent using a mechanical stopper device and further frame drift causes yielding in the knee elements.

Shayanfar et al. [199] developed a performance-based plastic design (PBPD) methodology for EBFs with vertical links similar to the design methodology proposed previously by Chao and Goel [158, 159] for EBFs with horizontal links. Based on the approach developed by Mastrandrea et al. [147-150], a rigid-plastic analysis and design approach for EBFs with vertical links was developed by Montuori et al. [200, 201] in 2014 which included moment-shear interaction and aimed to prevent local and partial mechanisms while ensuring formation of a desired global collapse mechanism.

In a recent numerical study by Massah and Dorvar [202], the analysis and design of EBFs with vertical shear links and shape memory alloy (SMA) devices were investigated. The SMA material can recover its original shape even after very large strains through the shape memory effect (which requires heating) and the superelasticity effect (which requires unloading). In this study, SMA devices were mounted on sides of the vertical links to obtain a reversible system with reduced residual deformations.

In 2016, Wang et al. [203] conducted an experimental study on a three-story one-bay by one-bay EBF system with vertical links in which the links were constructed from conventional steel while other members were made from high strength steel, similar to the concept studied for EBFs with horizontal links by Lian et al. [39]. Although excellent cyclic performance was reported, significant out-of-plane deformation was observed at the conjunction of the vertical link and the braces (which was not laterally supported) leading to failure due to fracture in the link-to-beam connection at the first story.

Although the current AISC Seismic Provisions [3] are intended for designing EBFs with horizontal links, the commentary on the code emphasizes the importance of lateral bracing at the intersection of a vertical link and braces, if inverted Y-braced EBFs are utilized.

6.2. Tied Braced Frames and EBFs with Zipper Struts

To prevent formation of a soft story mechanism in an EBF [16, 89, 90], Martini et al. [204] proposed a modified EBF configuration in 1990, known as the tied braced frame (TBF), in which the link ends are vertically connected to each other over the entire height of the structure. Although a number of early studies indicated some advantages for using TBFs [143, 204], the nonlinear static and dynamic analyses of Popov et al. [141] in 1992 demonstrated that properly designed EBF systems can have well-distributed inelastic action throughout the building height without the need for ties. In a number of recent studies, however, unsatisfactory seismic behavior was reported, particularly for high-rise EBFs [109, 111]. As a result, design methodologies for TBFs were developed by Ghersi et al. [205] and Rossi [206] on the assumption that these systems are more prone to form a global collapse mechanism. Promising performance was reported by Rossi [206] for TBFs deigned according to the proposed methodology.

In a similar approach, Zahrai et al. [207] proposed and numerically investigated the behavior of EBFs with zipper struts, which vertically connect the mid-points of shear links throughout the height. Results of the extensive pushover, cyclic loading, and time-history analyses revealed that the zipper struts not only help in having coincident yielding of the links but also increase the ductility and energy dissipation of the system.

6.3. Replaceable Links

Although the concept of replaceable links was previously mentioned in a number of studies (e.g. [24, 104, 187]), the first research specifically on EBFs with horizontal links that can be easily dismounted and replaced after an earthquake was conducted by Stratan and Dubina [27, 208] in the early 2000s. A link-to-beam connection with bolted end-plates, which were flush with the floor beam, was studied. Pinched behavior due to the end-plate bending and bolt thread stripping was observed in some cases and it was concluded that the link length ratio (ρ) should be limited to 0.8 to have proper cyclic behavior. Further numerical and experimental studies of Dubina et al. [209, 210] confirmed the applicability of the concept and demonstrated that dual EBFs have a notable re-centering capability which significantly facilitates the post-damage replacement. To further investigate this, a comprehensive full-scale test program, known as the DUAREM project [211, 212], was recently conducted at the European Laboratory for Structural Assessment (ELSA) of the Joint Research Centre (JRC) in Ispra, Italy in which a three-story three-bay to one-bay dual EBF system with end-plated replaceable links was tested pseudo-dynamically under three earthquake levels and link replacement was performed after each ground motion simulation. It was concluded that the dual EBF system with replaceable links is a promising lateral load resisting system that provides the desired seismic performance as well as an economic post-earthquake repair procedure.

Two other replaceable shear link details were recently studied by Mansour et al. [34, 41] through isolated link tests as well as frame tests. The first detail was again a bolted end-plate link-to-beam connection; however, it had a link segment that was smaller than the floor beam, making it possible to use bolts above the top flange and below the bottom flange of the link. Excellent ductility and stable behavior was reported for this detail proving its ability for practical applications. Further numerical studies of Mago [213] in 2013, which are summarized in the HERA R4-145 report, confirmed the reliability of the connection. In a recent study by Dusicka and Lewis [35] on a similar replaceable link connection, the use of an additional pair of stiffeners was proposed to reduce the probability of flange fracture in the vicinity of the link-to-end-plate welds. These additional stiffeners were placed parallel to the link web and located in the first web panel (similar to the detail shown in Figure 7h). In addition to the numerical analyses, the detail was verified experimentally for the case of replaceable long links by Dusicka and Lewis [35].

Another detail developed by Mansour et al. [34, 41] for replaceable shear links (originally proposed by Balut and Gioncu [214] for MRFs) was a bolted web connection where two back-to-back channel sections, considered as the link segment, were bolted to the web of the floor beam. Although larger γ_P^{max} values were obtained compared to the tested end-plate connections, the bolted web connection experienced a pinched behavior due to the repeated cycles of bolt-slip, bolt bearing against the link web, and bolt hole ovalization. The larger rotation capacities of these links were attributed to the inelastic rotation of the connection itself which, in average, was about 16% of the total inelastic link rotation. Modified and reinforced details were proposed to overcome the pinched behavior observed for the web bolted connection. The replaceability of the damaged links was also studied in the frame tests.

There are also a limited number of practical applications of replaceable links in New Zealand which are summarized by Fussel et al. [215], Ramsay et al. [216], and Gardiner et al. [217]. It is worth noting that design guidelines for replaceable links are given in the specifications of New Zealand and Canada [17, 218].

6.4. Use of EBFs or links in Reinforced Concrete Structures

Providing a thorough review of this topic is out of the scope of this study; however, the main applications of EBF configurations in reinforced concrete (RC) structures are mentioned here for the sake of completeness. Steel eccentric braces with or without vertical links can be connected to existing RC frames for seismic rehabilitation. This concept was numerically and experimentally studied by Bouadi and Engelhardt [219], Ghobarah and Abou Elfath [220], Bouwkamp et al. [221], Perera et al. [222], D'Aniello [223], Mazzolani et al. [224], Pina et al. [225], Durucan and Dicleli [226], Özel and Güneyisi [227], Varum et al. [228], and Wang and Yu [229] in the past few decades. Steel links are also utilized as energy dissipating coupling beams between adjacent RC wall piers in hybrid coupled wall (HCW) systems. A comprehensive review of the research on the behavior, analysis, and design of these systems as well as particular topics such as link-to-wall connections can be found in El-Tawil et al. [230].

6.5. Progressive Collapse of EBFs

In 2009, the gravity-induced progressive collapse of EBFs was studied by Khandelwal et al. [108] utilizing validated numerical analyses. To this end, the alternate path method (APM) was used, i.e. critical columns and adjacent braces were instantaneously removed from a ten-story EBF, properly designed based on the U.S. specifications [69, 231] for high seismic risk. The limited numerical results demonstrated that EBFs are less vulnerable to progressive collapse compared to special CBFs, mainly because of their improved system layout. The beneficial effect of locating EBFs at the perimeter of the structure was also noted.

6.6. Fragility Functions for Links

The damage states and fragility functions for EBF links were developed recently by Gulec et al. [232]. Based on a precise evaluation of the test data of 82 links (reported previously in the literature) a total of 9 damage states, such as web yielding, flange local buckling, flange fracture, damage to concrete slab, etc., were indicated. Furthermore, the methods of repair (MoR) were categorized into 4 groups (i.e. cosmetic repair, concrete replacement, heat straightening, and link replacement) and each damage state was related to a certain MoR. Fragility functions were also developed for short and long links based on statistical analyses to specify the probability that a specific MoR will be required as a function of γ_P.

6.7. Other Applications

In a number of studies by Bruneau and his colleagues [233-237], the concept of EBFs with horizontal or vertical links has been used in steel bridge superstructures in order to introduce energy dissipation in end-diaphragms instead of the common approach which relies on the energy dissipation of the substructure. This can be fruitful for instance in cases where stable and ductile behavior of the substructure is doubtful. Results of the extensive numerical and experimental studies as well as design procedures are presented by these researchers.

The applicability of stainless steel in EBFs was studied by DiSarno et al. [238] in 2008. The results of nonlinear pushover and time-history analyses of a sample structure revealed that using members made from stainless steel can enhance the energy dissipation and structural overstrength of the system compared to regular EBFs while reducing the roof drift and the probability of local buckling in members.

In addition to EBFs, HCWs, and bridges, the concept of dissipative link segments has also been utilized in other structural systems. For instance, Moghaddasi B. and Zhang [118] studied the seismic behavior of diagrid structural frames with replaceable shear links, in 2013. In another recent study, the concept of linked column frames (LCF) was proposed and numerically studied by Malakoutian et al. [116]. The use of buckling-restrained braces (BRB) in eccentric configuration was studied by Prinz and Richards [239, 240]. The energy dissipation mechanism in this system is however substantially different from that of EBFs since yielding is concentrated in the braces rather than in the links.

7. Summary and Future Research Needs

A comprehensive review of the research conducted on the behavior and design of eccentrically braced frames has been presented in this study which has covered both component level and system level responses. Experimental and numerical studies that address the main characteristics of links as well as link detailing have been presented. Different numerical techniques for link modeling have also been discussed. Furthermore, studies which focused on the seismic behavior and design of EBF systems have been summarized with an emphasis on the capacity design approach. Finally, the special applications of EBF systems or link segments have been discussed. For the improvement of EBF design and applications, the following items have been identified in the course of this review study as important research needs:

- Further research on the behavior of very short links ($\rho < 1$) is required in order to take advantage of their excessive inelastic rotation capacity in EBF systems. In contrast, considering the doubtful performance of very long links ($\rho > 3 \sim 3.5$) reported in some numerical studies, additional research for identifying their deficiencies and limitations would appear to be beneficial.
- The use of a single link overstrength factor (Ω) for all link length ratios in the EBF design process can lead to significant underestimations of internal forces developed in other members due to the yielded and strain hardened links, especially when very short links are utilized. Thus, further research is needed to develop simple methods for estimating the actual link overstrength factor for different link length ratios considering the effects of moment-shear interaction, link flange shear resistance, axial restraint provided by the adjacent members, and excessive cyclic hardening. A similar concern is also valid for the case of built-up tubular links. Furthermore, the unusually high overstrengths (of the order of 5.0) which have been reported recently for some unstiffened very short links also need further attention.
- There is a substantial research need for investigating the effect of high axial load on the behavior of intermediate and long links. In addition, the effect of tensile axial force on increasing the link overstrength requires further attention, as mentioned in a number of recent studies.

- The increase in the link overstrength due to the presence of a concrete slab has not been adequately investigated. This effect can be hazardous particularly from the capacity design point of view. Methods for estimating this increase are required which can be conveniently applied in practice.
- Additional investigation is required to validate whether the recent relaxation in the flange slenderness requirements of shear links by the AISC Seismic Provisions [3] can also be considered for shear links subject to high axial loads.
- Several issues have recently been reported for the spacing requirements of links per AISC 341-10 [3] which need further consideration. Research is required to investigate the possibility of any relaxation in the stiffener spacing requirements of I-shaped shear links. On the contrary, the observed poor performance of some intermediate links, which is attributed mostly to their stiffener spacing requirements, should be studied carefully to come up with appropriate solutions.
- Many novel web stiffening details for reducing the probability of premature fracture and improving the link rotation capacity including, but not restricted to, the sandwich, horizontal stiffener, and diagonal stiffeners details have been proposed which require further research. In addition, the feasibility of other approaches, such as links without intermediate stiffeners, should be studied. Furthermore, if approved, the development of design guidelines for employing these details is essential.
- The relation between the reduced material toughness in the k-area of I-shaped links and the recently observed link web fractures is still unclear and necessitates additional experiments.
- The deteriorative effect of misalignment between the link end stiffeners and brace flanges in triggering fracture was observed during recent earthquakes, and thus, providing practical methods for reducing the probability of this misalignment in field applications would be fruitful.
- The current intermediate stiffener spacing requirements of built-up box shear links are not dependent upon the required level of γ_P. Further experimental research is needed to validate such a relation.
- Although the crucial importance of link lateral bracing is well understood, there exist only a handful of studies which have explicitly investigated the effect of lateral bracing on link behavior,

considering realistic loading and boundary conditions. Additional research on this topic would be beneficial since the current codified lateral bracing requirements for links are in fact based on studies which focused mainly on the plastic hinge locations of MRFs.

- There is a significant research need in the field of link-to-column connections considering the fact that most of the connections suitable for MRFs exhibit poor performance when used as link-to-column connections. Promising details such as the all-around fillet weld and the supplemental web doubler connections require further experimental verification, especially for long links, with the aim of developing prequalified link-to-column connections. In addition, other less studied link-to-column connections such as the end-plate connection with and without rib stiffeners as well as the connection reinforced by haunches need additional investigation and development of design procedures. There are also details such as the reduced flange section link connection, which have not been studied experimentally at all.
- There is almost no study which specifically investigates the behavior and design of the column panel zones of EBFs with link-to-column connections. Research on this issue is indispensable. The current AISC Seismic Provisions [3] use the requirements of special MRFs for EBF column panel zone design without the necessary research background.
- The amount of reduction in Ω for the capacity design of columns in EBFs is typically limited since there are no methods for predicting the actual number of simultaneously yielded links above the column under consideration except for complicated nonlinear analysis. Thus, developing simple and reliable methods for estimating the reduced column forces can be advantageous, especially for high-rise EBFs, where a potential cost saving might be achieved.
- There is an urgent need for the reevaluation of the response modification factors (R) of EBFs that are in use by most of the design specifications, since they can significantly overestimate the ductility of these structures. In a similar manner, previous studies have proven that the codified displacement amplification factors (C_d) for EBFs are not capable of predicting the actual inter-story drifts and inelastic link rotations based on the results of linear analysis, and thus, reevaluation again appears to be necessary. Although several studies have been undertaken regarding these issues, further research is essential in order to confirm such alterations.

- The problem of the concentration of yielding in a limited number of stories is reported even for some properly designed mid- and high-rise EBFs. New methodologies have been proposed recently to overcome this issue, which require further verification (and in some cases simplification) so as to be considered as reliable and practical methods for the design of EBFs.
- The codified assessment procedures for EBFs typically do not consider the effect of the loading history on the acceptance criteria and link failure detection. As noted in recent studies, this deficiency can be overcome by introducing measures based on cumulative demands. Further monotonic tests or experiments with other more suitable loading histories might be fruitful for developing these cumulative criteria, which can be used for predicting the actual link rotation capacities during an earthquake.
- The research on EBFs with vertical links is limited when compared to their horizontal counterparts. There is a significant need for research on the behavior and design of these structures to address issues such as stability and the detailing of vertical links. There is also a notable gap in the design specifications regarding EBFs with vertical links, which can only be closed with additional studies.
- The promising performance of tied braced frames (TBFs) as well as EBFs upgraded with zipper struts in terms of the proper distribution of yielding over the structure height necessitates further confirmation through extensive numerical simulations as well as experimental studies. Practical procedures for the design of these systems are also required.
- There has been a considerable improvement in the field of replaceable links in the past decade; however, there is still a need for additional research to develop codified design and detailing rules for link-to-beam connections with the final goal of proposing prequalified connection types for replaceable links. Furthermore, studies on intermediate and long replaceable links as well as external replaceable links located between a brace and a column are few in number.
- Research on the progressive collapse of EBFs is very limited. Future research on this topic is essential, particularly for improving the behavior of these structures under probable blast loading scenarios.

- Newly proposed concepts such as composite links, EBFs with shape memory alloy (SMA) devices, EBFs made of high strength or stainless steel, and the use of energy-dissipating steel links in other structural systems such as diagrids and linked column frames (LCF) are also potential areas for future studies.

Acknowledgments

The study was supported by the Scientific and Technological Council of Turkey (TÜBİTAK) through grant number 114M251. The opinions expressed in this paper are those of the authors and do not reflect the views of the sponsor. The help of Professor Okazaki in providing photos and data is greatly acknowledged.

References

[1] M. Fujimoto, T. Aoyagi, K. Ukai, A. Wada, and K. Saito, "Structural Characteristics of Eccentric K-Braced Frames," *Tans. Arch. Inst. Jap.*, Vol. 195, PP. 39-49, 1972. (*in Japanese*)

[2] R. Tanabashi, K. Naneta, and T. Ishida, "On the Rigidity and Ductility of Steel Bracing Assemblage," Proceedings of the *5th World Conference on Earthquake Engineering*, IAEE, Rome, 1974, PP. 834-840.

[3] AISC, *Seismic Provisions for Structural Steel Buildings, ANSI/AISC 341-10*. American Institute of Steel Construction, Chicago, IL, 2010.

[4] C. W. Roeder and E. P. Popov, "Inelastic Behavior of Eccentrically Braced Steel Frames Under Cyclic Loading," University of California, Berkeley, UCB/EERC-77/18, 1977.

[5] C. W. Roeder and E. P. Popov, "Cyclic shear yielding of wide-flange beams," *J. Eng. Mech. Div. ASCE,* Vol. 104, No. 4, PP. 763-780, 1978.

[6] K. D. Hjelmstad and E. P. Popov, "Seismic behavior of active beam link in eccentrically braced frames," Earthquake Engineering Research Center, University of California, Berkeley, CA, Report No. UCB/EERC-83/15, 1983.

[7] K. D. Hjelmstad and E. P. Popov, "Cyclic Behavior and Design of Link Beams," *J. Struct. Eng. ASCE,* Vol. 109, No. 10, PP. 2387-2403, 1983.

[8] D. N. Manheim and E. P. Popov, "Plastic Shear Hinges in Steel Frames," *J. Struct. Eng. ASCE,* Vol. 109, No. 10, PP. 2404-2419, 1983.

[9] J. O. Malley and E. P. Popov, "Shear Links in Eccentrically Braced Frames," *J. Struct. Eng. ASCE,* Vol. 110, No. 9, PP. 2275-2295, 1984.

[10] K. Kasai and E. P. Popov, "General Behavior of WF Steel Shear Link Beams," *J. Struct. Eng. ASCE,* Vol. 112, No. 2, PP. 362-382, 1986.

[11] M. D. Engelhardt and E. P. Popov, "Behavior of long links in eccentrically braced frames," Earthquake Engineering Research Center, University of California, Berkeley, CA, Report No. UCB/EERC-89/01, 1989.

[12] SEAOC, *Recommended Lateral Force Requirements and Tentative Commentary*. Structural Engineers Association of California, Sacramento, CA, 1988.

[13] A. Daneshmand and B. H. Hashemi, "Performance of intermediate and long links in eccentrically braced frames," *J. Constr. Steel Res.*, Vol. 70, PP. 167-176, 2012.

[14] X. Ji, Y. Wang, Q. Ma, and T. Okazaki, "Cyclic Behavior of Very Short Steel Shear Links," *J. Struct. Eng. ASCE,* 2015. (doi:10.1061/(ASCE)ST.1943-541X.0001375)

[15] K. Kasai and E. P. Popov, "A study of seismically resistant eccentrically braced frames," Earthquake Engineering Research Center, University of California, Berkeley, CA, Report No. UCB/EERC-86/01, 1986.

[16] A. S. Whittaker, C. M. Uang, and V. V. Bertero, "Earthquake simulation tests and associated studies of a 0.3-scale model of a six-story eccentrically braced steel structure," University of California, Berkeley, UCB/EERC-87/02, 1987.

[17] CSA, *Design of Steel Structures, CAN/CSA S16-14*. Canadian Standards Association, Mississauga, ON, 2014.

[18] J. O. Malley and E. P. Popov, "Design Considerations for Shear Links in Eccentrically Braced Frames," University of California, Berkeley, Report No. UCB/EERC-83/24, 1983.

[19] J. M. Ricles and E. P. Popov, "Experiments on eccentrically braced frames with composite floors," Earthquake Engineering Research Center, University of California, Berkeley, CA, Report No. UCB/EERC-87/06, 1987.

[20] A. Ghobarah and T. Ramadan, "Bolted link-column joints in eccentrically braced frames," *Eng. Struct.,* Vol. 16, No. 1, PP. 33-41, 1994.

[21] A. M. Itani, "Cyclic behavior of Richmond-San Rafael tower links," Center for Civil Engineering Earthquake Research, University of Nevada, Reno, NV, Report No. CEER 97-4, 1997.

[22] B. Chi and C. M. Uang, "Cyclic testing of steel shear links for the San Francisco Moscone Convention Center expansion project," Department of Structural Engineering, University of California, San Diego, La Jolla, CA, Report No. TR-99/06, 2000.

[23] G. Arce, "Impact of higher strength steels on local buckling and overstrength of links in eccentrically braced frames," M.Sc. Thesis, Department of Civil Engineering, University of Texas at Austin, Austin, TX, 2002.

[24] C. C. McDaniel, C. M. Uang, and F. Seible, "Cyclic testing of built-up steel shear links for the new bay bridge," *J. Struct. Eng. ASCE,* Vol. 129, No. 6, PP. 801-809, 2003.

[25] P. Galvez, "Investigation of Factors Affecting Web Fractures in Shear Links," M.Sc. Thesis, Department of Civil Engineering, University of Texas at Austin, Austin, TX, 2004.

[26] T. Okazaki, "Seismic performance of link-to-column connections in steel eccentrically braced frames," Ph.D. Thesis, Department of Civil Engineering, University of Texas at Austin, Austin, TX, 2004.

[27] A. Stratan and D. Dubina, "Bolted links for eccentrically braced steel frames," Proceedings of the *5th AISC/ECCS International Workshop: Connections in Steel Structures V. Behaviour, Strength and Design,* Delft, The Netherlands, 2004, PP. 223-332.

[28] H. C. Ryu, "Effects of loading history on the behavior of links in seismic resistant eccentrically braced frames," M.Sc. Thesis, Department of Civil Engineering, University of Texas at Austin, Austin, TX, 2005.

[29] T. Okazaki, G. Arce, H. C. Ryu, and M. D. Engelhardt, "Experimental study of local buckling, overstrength, and fracture of links in eccentrically braced frames," *J. Struct. Eng. ASCE,* Vol. 131, No. 10, PP. 1526-1535, 2005.

[30] T. Okazaki, M. D. Engelhardt, M. Nakashima, and K. Suita, "Experimental performance of link-to-column connections in eccentrically braced frames," *J. Struct. Eng. ASCE,* Vol. 132, No. 8, PP. 1201-1211, 2006.

[31] T. Okazaki and M. D. Engelhardt, "Cyclic loading behavior of EBF links constructed of ASTM A992 steel," *J. Constr. Steel Res.,* Vol. 63, No. 6, PP. 751-765, 2007.

[32] A. Drolias, "Experiments on link-to-column connections in steel eccentrically braced frames," M.Sc. Thesis, Department of Civil Engineering, University of Texas at Austin, Austin, TX, 2007.

[33] T. Okazaki, M. D. Engelhardt, A. Drolias, E. Schell, J. K. Hong, and C. M. Uang, "Experimental investigation of link-to-column connections in eccentrically braced frames," *J. Constr. Steel Res.,* Vol. 65, No. 7, PP. 1401-1412, 2009.

[34] N. Mansour, "Eccentrically Braced Frames with Replaceable Shear Links," Ph.D. Thesis, Department of Civil Engineering, University of Toronto, Toronto, Ontario, Canada, 2010.

[35] P. Dusicka and G. Lewis, "Investigation of Replaceable Sacrificial Steel Links," Proceedings of the *9th US National and 10th Canadian Conference on Earthquake Engineering*, Toronto, Canada, 2010.
[36] P. Dusicka, A. M. Itani, and I. G. Buckle, "Cyclic Behavior of Shear Links of Various Grades of Plate Steel," *J. Struct. Eng. ASCE,* Vol. 136, No. 4, PP. 370-378, 2010.
[37] A. Ciutina, D. Dubina, and G. Danku, "Influence of steel-concrete interaction in dissipative zones of frames: I-Experimental study," *Steel Compos. Struct.*, Vol. 15, No. 3, PP. 299-322, 2013.
[38] T. Okazaki, M. D. Engelhardt, J. K. Hong, C. M. Uang, and A. Drolias, "Improved link-to-column connections for steel eccentrically braced frames," *J. Struct. Eng. ASCE,* 2014.
[39] M. Lian, M. Su, and Y. Guo, "Seismic performance of eccentrically braced frames with high strength steel combination," *Steel Compos. Struct.*, Vol. 18, No. 6, PP. 1517-1539, 2015.
[40] P. W. Richards, "Cyclic Stability and Capacity Design of Steel Eccentrically Braced Frames," Ph.D. Thesis, Department of Structural Engineering, University Of California, San Diego, San Diego, CA, 2004.
[41] N. Mansour, C. Christopoulos, and R. Tremblay, "Experimental validation of replaceable shear links for eccentrically braced steel frames," *J. Struct. Eng. ASCE,* Vol. 137, No. 10, PP. 1141-1152, 2011.
[42] P. Dusicka, A. M. Itani, and I. Buckle, "Cyclic behavior of shear links and tower shaft assembly of San Francisco-Oakland Bay Bridge Tower," Center for Civil Engineering Earthquake Research, University of Nevada, Reno, NV, Report No. CCEER-02-06, 2002.
[43] G. Della Corte, M. D'Aniello, and R. Landolfo, "Analytical and numerical study of plastic overstrength of shear links," *J. Constr. Steel Res.*, Vol. 82, PP. 19-32, 2013.
[44] K. Kasai, Y. Ooki, P. Suriyamongkol, and Y. Xu, "Fundamental study on inelastic behavior and low-cycle fatigue of a thick shear panel fabricated without welding," *J. Struct. Constr. Eng.*, Vol. 69, No. 586, PP. 195-202, 2004. (*in Japanese*)
[45] J. W. Berman and M. Bruneau, "Approaches for the Seismic Retrofit of Braced Steel Bridge Piers and Proof-of-Concept Testing of an Eccentrically Braced Frame with Tubular Link," Multidisciplinary Center for Earthquake Engineering Research, State University of New York at Buffalo, Buffalo, NY, Report No. MCEER-05-0004, 2005.
[46] J. W. Berman and M. Bruneau, "Further development of tubular eccentrically braced frame links for the seismic retrofit of braced steel truss bridge piers," Multidisciplinary Center for Earthquake Engineering Research, State University of New York at Buffalo, Buffalo, NY, Report No. MCEER-06-0006, 2006.
[47] J. W. Berman and M. Bruneau, "Experimental and analytical investigation of tubular links for eccentrically braced frames," *Eng. Struct.*, Vol. 29, No. 8, PP. 1929-1938, 2007.
[48] J. W. Berman and M. Bruneau, "Tubular links for eccentrically braced frames. I: Finite element parametric study," *J. Struct. Eng. ASCE,* Vol. 134, No. 5, PP. 692-701, 2008.
[49] J. W. Berman and M. Bruneau, "Tubular links for eccentrically braced frames. II: Experimental verification," *J. Struct. Eng. ASCE,* Vol. 134, No. 5, PP. 702-712, 2008.
[50] M. Bruneau, C. M. Uang, and S. R. Sabelli, *Ductile design of steel structures, 2nd ed.* McGraw Hill Professional, 2011.
[51] P. W. Richards and C. M. Uang, "Effect of flange width-thickness ratio on eccentrically braced frames link cyclic rotation capacity," *J. Struct. Eng. ASCE,* Vol. 131, No. 10, PP. 1546-1552, 2005.
[52] M. Bulic, M. Causevic, and B. Androic, "Reliability of short seismic links in shear," *B. Earthq. Eng.*, Vol. 11, No. 4, PP. 1083-1098, 2013.
[53] T. Ramadan and A. Ghobarah, "Prediction of the ultimate capacity of wide flange link beams under cyclic loading," *Comput. Struct.*, Vol. 40, No. 2, PP. 409-418, 1991.
[54] A. Ghobarah and T. Ramadan, "Seismic analysis of links of various lengths in eccentrically braced frames," *Can. J. Civil. Eng.*, Vol. 18, No. 1, PP. 140-148, 1991.
[55] A. Mohebkhah and B. Chegeni, "Overstrength and rotation capacity for EBF links made of European IPE sections," *Thin Wall. Struct.*, Vol. 74, PP. 255-260, 2014.
[56] S. Koboevic and S. O. David, "Design and seismic behaviour of taller eccentrically braced frames," *Can. J. Civil. Eng.*, Vol. 37, No. 2, PP. 195-208, 2010.

[57] S. Koboevic, J. Rozon, and R. Tremblay, "Seismic Performance of Low-to-Moderate Height Eccentrically Braced Steel Frames Designed for North American Seismic Conditions," *J. Struct. Eng. ASCE,* Vol. 138, No. 12, PP. 1465-1476, 2012.
[58] P. W. Richards and B. Thompson, "Estimating inelastic drifts and link rotation demands in EBFs," *AISC Eng. J.,* No. 3, PP. 123-135, 2009.
[59] A. Kuşyılmaz and C. Topkaya, "Displacement amplification factors for steel eccentrically braced frames," *Earthq. Eng. Struct. D.,* Vol. 44, No. 2, PP. 167-184, 2015.
[60] G. J. O'Reilly and T. J. Sullivan, "Direct Displacement-Based Seismic Design of Eccentrically Braced Steel Frames," *J. Earthq. Eng.,* Vol. 20, No. 2, PP. 243-278, 2016.
[61] A. Ghobarah and T. Ramadan, "Effect of axial forces on the performance of links in eccentrically braced frames," *Eng. Struct.,* Vol. 12, No. 2, PP. 106-113, 1990.
[62] S. Dastmalchi, "Numerical study of axial load effect on the seismic behavior of shear links in eccentrically braced frames," M.Sc. Thesis, Department of Civil and Environmental Engineering, The University of Maryland, College Park, MD, 2014.
[63] J. M. Ricles and E. P. Popov, "Composite Action in Eccentrically Braced Frames," *J. Struct. Eng. ASCE,* Vol. 115, No. 8, PP. 2046-2065, 1989.
[64] M. D. Engelhardt and E. P. Popov, "Experimental performance of long links in eccentrically braced frames," *J. Struct. Eng. ASCE,* Vol. 118, No. 11, PP. 3067-3088, 1992.
[65] K. C. Tsai, Y. F. Yang, and J. L. Lin, "Seismic eccentrically braced frames," *Struct. Des. Tall Spec. Build.,* Vol. 2, No. 1, PP. 53-74, 1993.
[66] P. W. Richards and C. M. Uang, "Testing protocol for short links in eccentrically braced frames," *J. Struct. Eng. ASCE,* Vol. 132, No. 8, PP. 1183-1191, 2006.
[67] P. W. Richards and C. M. Uang, "Development of testing protocol for short links in eccentrically braced frames," Department of Structural Engineering, University of California at San Diego, San Diego, CA, Report No. SSRP–2003/08, 2003.
[68] AISC, *Seismic Provisions for Structural Steel Buildings, ANSI/AISC 341-02*. American Institute of Steel Construction, Chicago, IL, 2002.
[69] AISC, *Seismic Provisions for Structural Steel Buildings, ANSI/AISC 341-05*. American Institute of Steel Construction, Chicago, IL, 2005.
[70] B. Chegeni and A. Mohebkhah, "Rotation capacity improvement of long link beams in eccentrically braced frames," *Sci. Iran. Trans. Civil Eng.,* Vol. 21, No. 3, PP. 516-524, 2014.
[71] G. Haaijer, "Plate buckling in the strain-hardening range," *J. Eng. Mech. Div. ASCE,* Vol. 83, No. EM2, PP. 1-45, 1957.
[72] ICBO, *Uniform Building Code*. International Conference of Building Officials, Whittier, CA, 1988.
[73] AISC, *Load and Resistance Factor Design Specification for Buildings*. American Institute of Steel Construction, Chicago, IL, 1986.
[74] AISC, *Specification for Structural Steel Buildings, ANSI/AISC 360-10*. American Institute of Steel Construction, Chicago, IL, 2010.
[75] K. Kasai and E. P. Popov, "Cyclic Web Buckling Control for Shear Link Beams," *J. Struct. Eng. ASCE,* Vol. 112, No. 3, PP. 505-523, 1986.
[76] K. Basler, "Strength of Plate Girders in Shear," *J. Struct. Eng. ASCE,* Vol. 87, No. ST7, PP. 151-180, 1961.
[77] F. Bleich, *Buckling Strength of Metal Structures*. McGraw-Hill, New York, 1952.
[78] E. P. Popov and M. D. Engelhardt, "Seismic eccentrically braced frames," *J. Constr. Steel Res.,* Vol. 10, PP. 321-354, 1988.
[79] Eurocode 0, *Basis of structural design, EN 1990:2002*. European standard, Comité Européen de Normalisation, Brussels, 2002.
[80] S. Chao, K. Khandelwal, and S. El-Tawil, "Ductile Web Fracture Initiation in Steel Shear Links," *J. Struct. Eng. ASCE,* Vol. 132, No. 8, PP. 1192-1200, 2006.
[81] H. Bahrampoor and S. Sabouri-Ghomi, "Effect of easy-going steel concept on the behavior of diagonal eccentrically braced frames," *Int. J. Civ. Eng.,* Vol. 8, No. 3, PP. 242-255, 2010.
[82] Y. Yurisman, B. Budiono, M. Moestopo, and M. Suarjana, "Behavior of Shear Link of WF Section with Diagonal Web Stiffener of Eccentrically Braced Frame (EBF) of Steel Structure," *ITB J. Eng. Sci.,* Vol. 42, No. 2, PP. 103-128, 2010.

[83] M. Ohsaki and T. Nakajima, "Optimization of link member of eccentrically braced frames for maximum energy dissipation," *J. Constr. Steel Res.*, Vol. 75, PP. 38-44, 2012.
[84] A. Kanvinde, K. Marshall, D. Grilli, and G. Bomba, "Forensic Analysis of Link Fractures in Eccentrically Braced Frames during the February 2011 Christchurch Earthquake: Testing and Simulation," *J. Struct. Eng. ASCE,* Vol. 141, No. 5, 2015.
[85] R. Imani and M. Bruneau, "Effect of Link-Beam Stiffener and Brace Flange Alignment on Inelastic Cyclic Behavior of Eccentrically Braced Frames," *AISC Eng. J.,* Vol. 52, No. 2, PP. 109-124, 2015.
[86] D. N. Manheim, "On the Design of Eccentrically Braced Frames," Ph.D. Thesis, Department of Civil and Environmental Engineering, University of California, Berkeley, Berkeley, CA, 1982.
[87] K. D. Hjelmstad and S. G. Lee, "Lateral buckling of beams in eccentrically-braced frames," *J. Constr. Steel Res.*, Vol. 14, No. 4, PP. 251-272, 1989.
[88] M. D. Engelhardt and E. P. Popov, "On Design of Eccentrically Braced Frames," *Earthq. Spectra EERI,* Vol. 5, No. 3, PP. 495-511, 1989.
[89] C. W. Roeder, D. A. Foutch, and S. C. Goel, "Seismic Testing of Full-Scale Steel Building - Part II," *J. Struct. Eng. ASCE,* Vol. 113, No. 11, PP. 2130-2145, 1987.
[90] D. A. Foutch, "Seismic Behavior of Eccentrically Braced Steel Building," *J. Struct. Eng. ASCE,* Vol. 115, No. 8, PP. 1857-1876, 1989.
[91] AISC, *Seismic Provisions for Structural Steel Buildings*. American Institute of Steel Construction, Chicago, IL, 1992.
[92] T. Ramadan and A. Ghobarah, "Behaviour of bolted link-column joints in eccentrically braced frames," *Can. J. Civil. Eng.,* Vol. 22, No. 4, PP. 745-754, 1995.
[93] A. Pirmoz, P. Ahadi, and V. Farajkhah, "Finite element analysis of extended stiffened end plate link-to-column connections," *Steel Constr.,* Vol. 9, No. 1, PP. 46-57, 2016.
[94] K. C. Tsai, M. D. Engelhardt, and M. Nakashima, "Cyclic performance of link-to-box column connections in steel eccentrically braced frames," Proceedings of the *1st International Conference on Structural Stability and Dynamics,* Taipei, Taiwan, 2000.
[95] C. Mao, J. Ricles, L. W. Lu, and J. Fisher, "Effect of local details on ductility of welded moment connections," *J. Struct. Eng. ASCE,* Vol. 127, No. 9, PP. 1036-1044, 2001.
[96] FEMA, *Recommended seismic design criteria for new steel moment-frame buildings, FEMA-350*. Building Seismic Safety Council for the Federal Emergency Management Agency, Washington, DC, 2000.
[97] J. Choi, B. Stojadinovic, and S. C. Goel, "Development of free flange moment connection," Department of Civil and Environmental Engineering, University of Michigan, Ann Arbor, MI, Report No. UMCEE 00-15, 2000.
[98] K. Suita, T. Tamura, S. Morita, M. Nakashima, and M. D. Engelhardt, "Plastic rotation capacity of steel beam-to-column connections using a reduced beam section and no weld access hole design - Full scale tests for improved steel beam-to-column subassemblages - Part 1," *J. Struct. Constr. Eng.,* Vol. 526, PP. 177-184, 1999. (*in Japanese*)
[99] J. K. Hong, C. M. Uang, T. Okazaki, and M. D. Engelhardt, "Link-to-column connection with supplemental web doublers in eccentrically braced frames," *J. Struct. Eng. ASCE,* Vol. 141, No. 8, 2015.
[100] G. S. Prinz and P. W. Richards, "Eccentrically braced frame links with reduced web sections," *J. Constr. Steel Res.*, Vol. 65, No. 10-11, PP. 1971-1978, 2009.
[101] J. W. Berman, T. Okazaki, and H. O. Hauksdottir, "Reduced link sections for improving the ductility of eccentrically braced frame link-to-column connections," *J. Struct. Eng. ASCE,* Vol. 136, No. 5, PP. 543-553, 2010.
[102] P. Dusicka, A. M. Itani, and I. G. Buckle, "Finite element investigation of steel built-up shear links subjected to inelastic deformations," *Earthq. Eng. Eng. Vib.,* Vol. 3, No. 2, PP. 195-203, 2004.
[103] G. Yiğitsoy, C. Topkaya, and T. Okazaki, "Stability of beams in steel eccentrically braced frames," *J. Constr. Steel Res.*, Vol. 96, PP. 14-25, 2014.

[104] M. S. Yang, "Seismic Behavior of an Eccentrically X-Braced Steel Structure," Earthquake Engineering Research Center, University of California, Berkeley, CA, Report No. UCB/EERC-82/14, 1982.
[105] J. M. Ricles and E. P. Popov, "Dynamic analysis of seismically resistant eccentrically braced frames," Earthquake Engineering Research Center, University of California, Berkeley, CA, Report No. UCB/EERC-87/07, 1987.
[106] J. M. Ricles and E. P. Popov, "Inelastic link element for EBF seismic analysis," *J. Struct. Eng. ASCE*, Vol. 120, No. 2, PP. 441-463, 1994.
[107] T. Ramadan and A. Ghobarah, "Analytical model for shear-link behavior," *J. Struct. Eng. ASCE*, Vol. 121, No. 11, PP. 1574-1580, 1995.
[108] K. Khandelwal, S. El-Tawil, and F. Sadek, "Progressive collapse analysis of seismically designed steel braced frames," *J. Constr. Steel Res.*, Vol. 65, No. 3, PP. 699-708, 2009.
[109] P. P. Rossi and A. Lombardo, "Influence of the link overstrength factor on the seismic behaviour of eccentrically braced frames," *J. Constr. Steel Res.*, Vol. 63, No. 11, PP. 1529-1545, 2007.
[110] M. Bosco, E. M. Marino, and P. P. Rossi, "Proposal of modifications to the design provisions of Eurocode 8 for buildings with split K eccentric braces," *Eng. Struct.*, Vol. 61, PP. 209-223, 2014.
[111] M. Bosco and P. P. Rossi, "Seismic behaviour of eccentrically braced frames," *Eng. Struct.*, Vol. 31, No. 3, PP. 664-674, 2009.
[112] M. Bosco and P. P. Rossi, "A design procedure for dual eccentrically braced systems: Analytical formulation," *J. Constr. Steel Res.*, Vol. 80, PP. 440-452, 2013.
[113] M. Bosco and P. P. Rossi, "A design procedure for dual eccentrically braced systems: Numerical investigation," *J. Constr. Steel Res.*, Vol. 80, PP. 453-464, 2013.
[114] M. Bosco, E. M. Marino, and P. P. Rossi, "Modelling of steel link beams of short, intermediate or long length," *Eng. Struct.*, Vol. 84, PP. 406-418, 2015.
[115] A. Zona and A. Dall'Asta, "Elastoplastic model for steel buckling-restrained braces," *J. Constr. Steel Res.*, Vol. 68, No. 1, PP. 118-125, 2012.
[116] M. Malakoutian, J. W. Berman, and P. Dusicka, "Seismic response evaluation of the linked column frame system," *Earthq. Eng. Struct. D.*, Vol. 42, No. 6, PP. 795-814, 2013.
[117] F. McKenna, G. Fenves, F. C. Filippou, and S. Mazzoni. *Open System for Earthquake Engineering Simulation (OpenSees)* [Computer Software]. University of California, Berkeley, CA.
[118] N. S. Moghaddasi B. and Y. Zhang, "Seismic analysis of diagrid structural frames with shear-link fuse devices," *Earthq. Eng. Eng. Vib.*, Vol. 12, No. 3, PP. 463-472, 2013.
[119] M. T. Kazemi and S. Erfani, "Mixed shear-flexural (VM) hinge element and its applications," *Sci. Iran. Trans. Civil Eng.*, Vol. 14, No. 3, PP. 193-204, 2007.
[120] M. T. Kazemi and S. Erfani, "Special VM link element for modeling of shear–flexural interaction in frames," *Struct. Des. Tall Spec. Build.*, Vol. 18, No. 2, PP. 119-135, 2009.
[121] M. T. Kazemi and M. Hoseinzadeh Asl, "Modeling of inelastic mixed hinge and its application in analysis of the frames with reduced beam section," *Int. J. Steel Struct.*, Vol. 11, No. 1, PP. 51-63, 2011.
[122] A. Saritas and F. C. Filippou, "Frame element for metallic shear-yielding members under cyclic loading," *J. Struct. Eng. ASCE*, Vol. 135, No. 9, PP. 1115-1123, 2009.
[123] A. Papachristidis, M. Fragiadakis, and M. Papadrakakis, "A 3D fibre beam-column element with shear modelling for the inelastic analysis of steel structures," *Comput. Mech.*, Vol. 45, No. 6, PP. 553-572, 2010.
[124] H. V. Spurr, *Wind bracing: the importance of rigidity in high towers*. McGraw-Hill, New York, 1930.
[125] C. W. Roeder and E. P. Popov, "Eccentrically Braced Steel Frames for Earthquakes," *J. Struct. Eng. ASCE*, Vol. 104, No. ST3, PP. 391-411, 1978.
[126] A. S. Whittaker, C. M. Uang, and V. V. Bertero, "Seismic Testing of Eccentrically Braced Dual Steel Frames," *Earthq. Spectra EERI*, Vol. 5, No. 2, PP. 429-449, 1989.
[127] A. S. Whittaker, C.-M. Uang, and V. V. Bertero, "Experimental behavior of dual steel system," *J. Struct. Eng. ASCE*, Vol. 115, No. 1, PP. 183-200, 1989.

[128] T. Balendra, K. Y. Lam, C. Y. Liaw, and S. L. Lee, "Behavior of eccentrically braced frame by pseudo-dynamic test," *J. Struct. Eng. ASCE,* Vol. 113, No. 4, PP. 673-688, 1987.

[129] K. D. Hjelmstad and E. P. Popov, "Characteristics of Eccentrically Braced Frames," *J. Struct. Eng. ASCE,* Vol. 110, No. 2, PP. 340-353, 1984.

[130] E. P. Popov, M. D. Engelhardt, and J. M. Ricles, "Eccentrically braced frames: US practice," *Eng. J. AISC,* Vol. 26, No. 2, 1989.

[131] J. R. Libby, "Eccentrically Braced Frame Construction—A Case History," *Eng. J. AISC,* Vol. 18, No. 4, 1981.

[132] A. T. Merovich, J. P. Nicoletti, and E. Hartle, "Eccentric Bracing in Tall Buildings," *J. Struct. Eng. ASCE,* Vol. 108, No. 9, 1982.

[133] X. Qi, K. L. Chang, and K. C. Tsai, "Seismic Design of Eccentrically Braced Space Frame," *J. Struct. Eng. ASCE,* Vol. 123, No. 8, PP. 977-985, 1997.

[134] E. Teal, *Practical Design of Eccentrically Braced Frames to Resist Seismic Forces*. Structural Steel Education Council, El Monte, CA, 1979.

[135] AISC, *Seismic Design Manual 2nd ed., AISC 327-12A*. American Institute of Steel Construction, Chicago, IL, 2012.

[136] R. Becker and M. Ishler, *Seismic design practice for eccentrically braced frames based on the 1994 UBC, Steel Tips*. Structural Steel Education Council, Berkeley, CA, 1996.

[137] Eurocode 8, *Design of Structures for Earthquake Resistance - Part 1: General Rules, Seismic Actions and Rules for Buildings, EN 1998-1:2004*. European standard, Comité Européen de Normalisation, Brussels, 2004.

[138] S. Koboevic and R. Redwood, "Design and seismic response of shear critical eccentrically braced frames," *Can. J. Civil. Eng.,* Vol. 24, No. 5, 1997.

[139] M. D. Engelhardt, K. C. Tsai, and E. P. Popov, "Stability of beams in eccentrically braced frames," In: *Stability and ductility of steel structures*, Y. Fukumoto and G. C. Lee, Eds., 1992, PP. 99-112.

[140] ASCE, *Minimum Design Loads for Buildings and Other Structures, ASCE/SEI-7-10*. Structural Engineering Institute of the American Society of Civil Engineers, Reston, VA, 2010.

[141] E. P. Popov, J. M. Ricles, and K. Kasai, "Methodology for optimum EBF link design," Proceedings of the *10th world conference on earthquake engineering*, Madrid, Spain, 1992, PP. 3983-3988.

[142] J. M. Ricles and S. Bolin, "Energy Dissipation in Eccentrically Braced Frames," Proceedings of the *4th U.S. National Conference on Earthquake Engineering*, Palm Springs, CA, 1990.

[143] J. M. Ricles and S. Bolin, "Seismic performance of eccentrically braced frames," Structural Systems Research Project, Department of Applied Mechanics and Engineering Science, University of California at San Diego, San Diego, CA, Report No. 91-09, 1991.

[144] A. Jain, S. Koboevic, and R. Redwood, "Design and Behaviour of Eccentrically Braced Frames with Flexural Links," In: *Advances in Steel Structures (ICASS '96)*, S. L. Teng and J. G. Chan, Eds. Pergamon, Oxford, 1996, PP. 233-237.

[145] K. Kasai and X. Han, "New EBF Design Method and Application: Redesign and Analysis of US-Japan EBF," Proceedings of the *2nd International Conference on Behaviour of Steel Structures in Seismic Areas (STESSA 1997)*, Kyoto, Japan, 1997, PP. 242-249.

[146] L. Tirca and V. Gioncu, "Ductility demands for MRFs and LL-EBFs for different earthquake types," Proceedings of the *Stability And Ductility of Steel Structures (SDSS'99)*, 1999, PP. 429-438.

[147] L. Mastrandrea, R. Montuori, and V. Piluso, "Shear-moment interaction in plastic design: eccentrically braced frames," Proceedings of the *4th International Conference on Behaviour of Steel Structures in Seismic Areas (STESSA 2003)*, Naples, Italy, 2003, PP. 175-180.

[148] L. Mastrandrea and V. Piluso, "Plastic design of eccentrically braced frames, I: Moment–shear interaction," *J. Constr. Steel Res.,* Vol. 65, No. 5, PP. 1007-1014, 2009.

[149] L. Mastrandrea, R. Montuori, and V. Piluso, "Failrue mode control of seismic resistant EB-frames," Proceedings of the *4th International Conference on Behaviour of Steel Structures in Seismic Areas (STESSA 2003)*, Naples, Italy, 2003, PP. 435-441.

[150] L. Mastrandrea and V. Piluso, "Plastic design of eccentrically braced frames, II: Failure mode control," *J. Constr. Steel Res.*, Vol. 65, No. 5, PP. 1015-1028, 2009.
[151] L. Mastrandrea, E. Nastri, and V. Piluso, "Validation of a design procedure for failure mode control of EB-Frames: Push-over and IDA analyses," *Open Constr. Build. Tech. J.*, Vol. 7, PP. 193-207, 2013.
[152] R. Montuori, E. Nastri, and V. Piluso, "Theory of Plastic Mechanism Control for MRF–EBF dual systems: Closed form solution," *Eng. Struct.*, Vol. 118, PP. 287-306, 2016.
[153] H. Köber and B. Ștefănescu, "A seismic design of eccentrically braced frames," Proceedings of the *International Conference in Metal Structures (ICMS2006)*, Poiana Brasov, Romania, 2006, PP. 537-544.
[154] Eurocode 8, *Design of Structures for Earthquake Resistance - EN 1998:2002*. European standard, Comité Européen de Normalisation, Brussels, 2002.
[155] H. Köber and B. Ștefănescu, ""I" shaped braces in eccentrically braced frames," Proceedings of the *International Conference in Metal Structures (ICMS2006)*, Poiana Brasov, Romania, 2006, PP. 529-535.
[156] H. Köber and B. Ștefănescu, "Potentially plastic zone details in columns in eccentrically braced frames," Proceedings of the *6th International Conference on Behaviour of Steel Structures in Seismic Areas (STESSA 2009)*, Philadelphia, PA, 2009, PP. 629-634.
[157] D. Özhendekci and N. Özhendekci, "Effects of the frame geometry on the weight and inelastic behaviour of eccentrically braced chevron steel frames," *J. Constr. Steel Res.*, Vol. 64, No. 3, PP. 326-343, 2008.
[158] S. H. Chao and S. C. Goel, "Performance-based seismic design of EBF using target drift and yield mechanism as performance criteria," Department of Civil and Environmental Engineering, The University of Michigan, Ann Arbor, MI, Report No. UMCEE 05-05, 2005.
[159] S. H. Chao and S. C. Goel, "Performance-based seismic design of EBF using target drift and yield mechanism as performance criteria," *AISC Eng. J.*, No. 3, PP. 173-200, 2006.
[160] T. A. Sabol and D. E. Nishi, "Application of performance-based design to an eccentrically braced frame structure," *Struct. Des. Tall Spec. Build.*, Vol. 20, PP. 76-84, 2011.
[161] T. J. Sullivan, "Direct displacement-based seismic design of steel eccentrically braced frame structures," *B. Earthq. Eng.*, Vol. 11, No. 6, PP. 2197-2231, 2013.
[162] Eurocode 8, *Design of Structures for Earthquake Resistance - EN 1998:1993*. European standard, Comité Européen de Normalisation, Brussels, 1993.
[163] M. Bosco, E. M. Marino, and P. P. Rossi, "Critical review of the EC8 design provisions for buildings with eccentric braces," *Earthq. Struct.*, Vol. 8, No. 6, PP. 1407-1433, 2015.
[164] P. W. Richards, "Seismic Column Demands in Ductile Braced Frames," *J. Struct. Eng. ASCE*, Vol. 135, No. 1, PP. 33-41, 2009.
[165] ICC, *International Building Code*. International Code Council, Inc., Whittier, CA, 2006.
[166] A. Kuşyılmaz and C. Topkaya, "Design overstrength of steel eccentrically braced frames," *Int. J. Steel Struct.*, Vol. 13, No. 3, PP. 529-545, 2013.
[167] A. Kuşyılmaz and C. Topkaya, "Evaluation of Seismic Response Factors for Eccentrically Braced Frames Using FEMA P695 Methodology," *Earthq. Spectra*, Vol. 32, No. 1, PP. 303-321, 2016.
[168] M. S. Speicher and J. L. Harris III, "Collapse Prevention seismic performance assessment of new eccentrically braced frames using ASCE 41," *Eng. Struct.*, Vol. 117, PP. 344-357, 2016.
[169] ICC, *International Building Code*. International Code Council, Inc., Whittier, CA, 2012.
[170] ASCE, *Seismic rehabilitation of existing buildings, ASCE/SEI 41-06*. Structural Engineering Institute of the American Society of Civil Engineers, Reston, VA, 2006.
[171] P. W. Richards, "Estimating the Stiffness of Eccentrically Braced Frames," *Pract. Period. Struct. Des. Constr.*, Vol. 15, No. 1, PP. 91-95, 2010.
[172] A. Kuşyılmaz and C. Topkaya, "Fundamental periods of steel eccentrically braced frames," *Struct. Des. Tall Spec. Build.*, Vol. 24, No. 2, PP. 123-140, 2015.
[173] O. S. Kwon and E. S. Kim, "Evaluation of building period formulas for seismic design," *Earthq. Eng. Struct. D.*, Vol. 39, No. 14, PP. 1569-1583, 2010.
[174] K. Young and H. Adeli, "Fundamental period of irregular eccentrically braced tall steel frame structures," *J. Constr. Steel Res.*, Vol. 120, PP. 199-205, 2016.

[175] R. Tremblay, "Fundamental Periods of Vibration of Braced Steel Frames for Seismic Design," *Earthq. Spectra*, Vol. 21, No. 3, PP. 833-860, 2005.
[176] M. Badalassi, A. Braconi, S. Caprili, and W. Salvatore, "Influence of steel mechanical properties on EBF seismic behaviour," *B. Earthq. Eng.*, Vol. 11, No. 6, PP. 2249-2285, 2013.
[177] Eurocode 8, *Design of Structures for Earthquake Resistance - EN 1998:2005*. European standard, Comité Européen de Normalisation, Brussels, 2005.
[178] A. Perretti, "Comportamento sismico di telai in acciaio con controventi eccentrici," Ph.D. Thesis, Department of Civil, Building and Enviromental Engineering, University of Naples, Naples, Italy, 1999. (*in Italian*)
[179] K. C. Lin, C. J. Lin, J. Y. Chen, and H. Y. Chang, "Seismic reliability of steel framed buildings," *Struct. Saf.*, Vol. 32, No. 3, PP. 174-182, 2010.
[180] FEMA, *Prestandard and commentary for the seismic rehabilitation of buildings, FEMA-356*. Building Seismic Safety Council for the Federal Emergency Management Agency, Washington, DC, 2000.
[181] R. Eskandari and D. Vafaei, "Effects of near-fault records characteristics on seismic performance of eccentrically braced frames," *Struct. Eng. Mech.*, Vol. 56, No. 5, PP. 855-870, 2015.
[182] M. Naghipour, N. Javadi, and A. Naghipour, "Investigation of RBS Connection Ductility in Eccentrically Braced Frame," Proceedings of the *Twelfth East Asia-Pacific Conference on Structural Engineering and Construction - EASEC12*, Hong Kong, 2011, PP. 743-752.
[183] Y. Gong, Y. Xue, and L. Xu, "Optimal capacity design of eccentrically braced steel frameworks using nonlinear response history analysis," *Eng. Struct.*, Vol. 48, PP. 28-36, 2013.
[184] R. Karami Mohammadi and A. H. Sharghi, "On the optimum performance-based design of eccentrically braced frames," *Steel Compos. Struct.*, Vol. 16, No. 4, PP. 357-374, 2014.
[185] G. Danku, D. Dubina, and A. Ciutina, "Influence of steel-concrete interaction in dissipative zones of frames: II-Numerical study," *Steel Compos. Struct.*, Vol. 15, No. 3, PP. 323-342, 2013.
[186] G. S. Prinz and A. de Castro-e-Sousa, "Effect of Slab Stiffness on EBF Link Rotation Demands and Implications for Link Ultra Low-Cycle Fatigue Susceptibility," Proceedings of the *ASCE Structures Congress 2014*, Boston, MA, 2014, PP. 2664-2674.
[187] M. Seki, H. Katsumata, H. Uchida, and T. Takeda, "Study on earthquake response of two-storied steel frame with y-shaped braces," Proceedings of the *9th World Conference on Earthquake Engineering*, Tokyo-Kyoto, Japan, 1988, PP. 65-70.
[188] M. G. Vetr, "Seismic behavior, analysis and design of eccentrically braced frames with vertical shear links," Ph.D. Thesis, Department of Civil and Environmental Engineering, Technische Universität Darmstadt, Darmstadt, Germany, 1998.
[189] E. Fehling, W. Pauli, and J. G. Bouwkamp, "Use of vertical shear-links in eccentrically braced frames," Proceedings of the *10th World Conference on Earthquake Engineering*, Rotterdam, The Netherlands, 1992, PP. 4475-4480.
[190] J. G. Bouwkamp and M. G. Vetr, "Design of eccentrically braced test frame with vertical shear link," Proceedings of the *2nd International Conference on Earthquake Resistant Construction and Design*, Berlin, Germany, 1994.
[191] Y. Shinabe and Y. Takahashi, "The present state of eccentric brace design in Japan," Proceedings of the *4th Pacific Structural Steel Conference*, Singapore, 1995, PP. 813-820.
[192] A. Saedi Daryan, H. Bahrampoor, M. Ziaei, A. Golafshar, and M. A. Assareh, "Seismic Behavior of Vertical Shear Links made of Easy-Going Steel," *Am. J. Eng. Applied Sci.*, Vol. 1, No. 4, PP. 368-377, 2008.
[193] M. A. Shayanfar, A. R. Rezaeian, and S. Taherkhani, "Assessment of the seismic behavior of eccentrically braced frame with double vertical link (DV-EBF)," Proceedings of the *14th World Conference on Earthquake Engineering*, Beijing, China, 2008.
[194] M. A. Shayanfar and A. R. Rezaeian, "Assessment of seismic behavior of eccentrically braced frame with double vertical link (DV-EBF)," Proceedings of the *3rd International Conference on Advances in Experimental Structural Engineering*, San Francisco, CA, 2009.
[195] M. Dicleli and A. Mehta, "Efficient energy dissipating steel-braced frame to resist seismic loads," *J. Struct. Eng. ASCE*, Vol. 133, No. 7, PP. 969-981, 2007.

[196] M. Dicleli and A. Mehta, "Seismic Performance of a Special Type of Single-Story Eccentrically Braced Steel Frame," *Adv. Struct. Eng.*, Vol. 11, No. 1, PP. 35-51, 2008.
[197] M. A. Shayanfar, M. A. Barkhordari, and A. R. Rezaeian, "Experimental study of cyclic behavior of composite vertical shear link in eccentrically braced frames," *Steel Compos. Struct.*, Vol. 12, No. 1, PP. 13-29, 2012.
[198] S. M. Zahrai and A. K. Vosooq, "Study of an innovative two-stage control system: Chevron knee bracing & shear panel in series connection," *Struct. Eng. Mech.*, Vol. 47, No. 6, PP. 881-898, 2013.
[199] M. A. Shayanfar, A. R. Rezaeian, and A. Zanganeh, "Seismic performance of eccentrically braced frame with vertical link using PBPD method," *Struct. Des. Tall Spec. Build.*, Vol. 23, No. 1, PP. 1-21, 2014.
[200] R. Montuori, E. Nastri, and V. Piluso, "Rigid-plastic analysis and moment–shear interaction for hierarchy criteria of inverted Y EB-Frames," *J. Constr. Steel Res.*, Vol. 95, PP. 71-80, 2014.
[201] R. Montuori, E. Nastri, and V. Piluso, "Theory of plastic mechanism control for eccentrically braced frames with inverted Y-scheme," *J. Constr. Steel Res.*, Vol. 92, PP. 122-135, 2014.
[202] S. R. Massah and H. Dorvar, "Design and analysis of eccentrically braced steel frames with vertical links using shape memory alloys," *Smart Mater. Struct.*, Vol. 23, No. 11, 2014.
[203] F. Wang, M. Su, M. Hong, Y. Guo, and S. Li, "Cyclic behaviour of Y-shaped eccentrically braced frames fabricated with high-strength steel composite," *J. Constr. Steel Res.*, Vol. 120, PP. 176-187, 2016.
[204] K. Martini, N. Amin, P. L. Lee, and D. Bonowitz, "The Potential Role of Non-Linear Analysis in the Seismic Design of Building Structures," Proceedings of the *4th National Conference on Earthquake Engineering*, Palm Springs, CA, 1990, PP. 67-76.
[205] A. Ghersi, S. Pantano, and P. P. Rossi, "On the design of tied braced frames," Proceedings of the *4th International Conference on Behaviour of Steel Structures in Seismic Areas (STESSA 2003)*, Naples, Italy, 2003.
[206] P. P. Rossi, "A design procedure for tied braced frames," *Earthq. Eng. Struct. D.*, Vol. 36, No. 14, PP. 2227-2248, 2007.
[207] S. M. Zahrai, M. Pirdavari, and H. M. Farahani, "Evaluation of hysteretic behavior of eccentrically braced frames with zipper-strut upgrade," *J. Constr. Steel Res.*, Vol. 83, PP. 10-20, 2013.
[208] A. Stratan, D. Dubina, and F. Dinu, "Control of global performance of seismic resistant EBF with removable link," Proceedings of the *4th International Conference on Behaviour of Steel Structures in Seismic Areas (STESSA 2003)*, Naples, Italy, 2003, PP. 175-180.
[209] D. Dubina, A. Stratan, and F. Dinu, "Dual high-strength steel eccentrically braced frames with removable links," *Earthq. Eng. Struct. D.*, Vol. 37, No. 15, PP. 1703-1720, 2008.
[210] D. Dubina, A. Stratan, and F. Dinu, "Re-centring capacity of dual-steel frames," *Steel Constr.*, Vol. 4, No. 2, PP. 73-84, 2011.
[211] G. A. Sabau, M. Poljansek, F. Taucer, P. Pegon, F. J. Molina, D. Tirelli, B. Viaccoz, A. Stratan, A. Ioan Chesoan, and D. Dubina, "Seismic Engineering Research Infrastructures For European Synergies: Full-Scale Experimental Validation of a Dual Eccentrically Braced Frame with Removable Links (DUAREM)," Institute for the Protection and Security of the Citizen, Publications Office of the European Union, Luxembourg, Report EUR 27030, 2014.
[212] A. Ioan, A. Stratan, D. Dubina, M. Poljansek, F. J. Molina, F. Taucer, P. Pegon, and G. Sabau, "Experimental validation of re-centring capability of eccentrically braced frames with removable links," *Eng. Struct.*, Vol. 113, PP. 335-346, 2016.
[213] N. Mago, "Finite Element Analysis of Eccentrically Braced Frames with Replaceable Link," Heavy Engineering Research Association, Manukau, New Zealand, HERA Report R4-145, 2013.
[214] N. Balut and V. Gioncu, "Suggestion for an improved 'dog-bone' solution," Proceedings of the *4th International Conference on Behaviour of Steel Structures in Seismic Areas (STESSA 2003)*, Naples, Italy, 2003.

[215] A. J. Fussell, K. A. Cowie, G. C. Clifton, and N. Mago, "Development and research of eccentrically braced frames with replaceable active links," Proceedings of the *2014 NZSEE Conference*, Auckland, New Zealand, 2014.

[216] J. J. Ramsay, A. Fussell, and R. G. Wilkinson, "Design of Replaceable-Link Eccentric Braced Frames in Post-Earthquake Christchurch," Proceedings of the *Steel Innovations Conference 2013*, Christchurch, New Zealand, 2013.

[217] G. Gardiner, C. Clifton, and G. A. MacRae, "Performance, damage assessment and repair of a multistorey eccentrically braced frame building following the Christchurch earthquake series, steel innovations conference 2013," Proceedings of the *Steel Innovations Conference 2013*, Christchurch, New Zealand, 2013.

[218] HERA, *R4-76 Seismic Design Procedures for Steel Structures and P4001 Seismic design of eccentrically braced frames*. Heavy Engineering Research Association, Manukau, New Zealand, 2013.

[219] A. Bouadi and M. D. Engelhardt, "On Design of EBFs for Seismic Retrofit of Reinforced Concrete Frames," Proceedings of the *11th World Conference on Earthquake Engineering*, Acapulco, Mexico, 1996.

[220] A. Ghobarah and H. Abou Elfath, "Rehabilitation of a reinforced concrete frame using eccentric steel bracing," *Eng. Struct.*, Vol. 23, No. 7, PP. 745-755, 2001.

[221] J. G. Bouwkamp, S. Gómez, A. V. Pinto, J. Molina, and H. Varum, "Cyclic tests on RC frame retrofitted with K-bracing and shear-link dissipater," The European Laboratory for Structural Assessment (ELSA), Joint Research Centre (JRC), Ispra, Italy, EUR Report No. 20136 EN, 2001.

[222] R. Perera, S. Gómez, and E. Alarcón, "Experimental and analytical study of masonry infill reinforced concrete frames retrofitted with steel braces," *J. Struct. Eng. ASCE*, Vol. 130, No. 12, PP. 2032-2039, 2004.

[223] M. D'Aniello, "Seismic upgrading of RC structure by steel Eccentric Bracing: an experimental and numerical study," *Pollack Periodica*, Vol. 1, No. 2, PP. 17-32, 2006.

[224] F. M. Mazzolani, G. D. Corte, and M. D'Aniello, "Experimental analysis of steel dissipative bracing systems for seismic upgrading," *J. Civ. Eng. Manag*, Vol. 15, No. 1, PP. 7-19, 2009.

[225] F. Pina, G. W. White, G. Taylor, and C. Ventura, "Conventional steel constructions for the performance-based earthquake retrofit of low-rise school buildings," In: *Behaviour of Steel Structures in Seismic Areas*. CRC Press, 2009.

[226] C. Durucan and M. Dicleli, "Analytical study on seismic retrofitting of reinforced concrete buildings using steel braces with shear link," *Eng. Struct.*, Vol. 32, No. 10, PP. 2995-3010, 2010.

[227] A. E. Özel and E. M. Güneyisi, "Effects of eccentric steel bracing systems on seismic fragility curves of mid-rise R/C buildings: A case study," *Struct. Saf.*, Vol. 33, No. 1, PP. 82-95, 2011.

[228] H. Varum, F. Teixeira Dias, P. Marques, A. V. Pinto, and A. Q. Bhatti, "Performance evaluation of retrofitting strategies for non-seismically designed RC buildings using steel braces," *B. Earthq. Eng.*, Vol. 11, No. 4, PP. 1129-1156, 2013.

[229] D. Wang and A. Yu, "Shear Failure Behavior of Y-Eccentrically Brace in RC Frame Structures under Earthquake Action," *Adv. Mat. Res.*, Vol. 639-640, PP. 866-869, 2013.

[230] S. El-Tawil, K. A. Harries, P. J. Fortney, B. M. Shahrooz, and Y. Kurama, "Seismic design of hybrid coupled wall systems: state of the art," *J. Struct. Eng. ASCE*, Vol. 136, No. 7, PP. 755-769, 2010.

[231] ASCE, *Minimum Design Loads for Buildings and Other Structures, ASCE/SEI-7-05*. Structural Engineering Institute of the American Society of Civil Engineers, Reston, VA, 2005.

[232] C. K. Gulec, B. Gibbons, A. Chen, and A. S. Whittaker, "Damage states and fragility functions for link beams in eccentrically braced frames," *J. Constr. Steel Res.*, Vol. 67, No. 9, PP. 1299-1309, 2011.

[233] M. Sarraf and M. Bruneau, "Ductile seismic retrofit of steel deck-truss bridges. I: Strategy and modeling," *J. Struct. Eng. ASCE*, Vol. 124, No. 11, PP. 1253-1262, 1998.

[234] M. Sarraf and M. Bruneau, "Ductile seismic retrofit of steel deck-truss bridges. II: Design applications," *J. Struct. Eng. ASCE*, Vol. 124, No. 11, PP. 1263-1271, 1998.

[235] S. M. Zahrai and M. Bruneau, "Cyclic testing of ductile end diaphragms for slab-on-girder steel bridges," *J. Struct. Eng. ASCE,* Vol. 125, No. 9, PP. 987-996, 1999.

[236] S. M. Zahrai and M. Bruneau, "Ductile end-diaphragms for seismic retrofit of slab-on-girder steel bridges," *J. Struct. Eng. ASCE,* Vol. 125, No. 1, PP. 71-80, 1999.

[237] M. Bruneau, M. Sarraf, S. M. Zahrai, and F. Alfawakhiri, "Displacement-based energy dissipation systems for steel bridges diaphragms," *J. Constr. Steel Res.,* Vol. 58, No. 5–8, PP. 801-817, 2002.

[238] L. DiSarno, A. S. Elnashai, and D. A. Nethercot, "Seismic response of stainless steel braced frames," *J. Constr. Steel Res.,* Vol. 64, No. 7–8, PP. 914-925, 2008.

[239] G. S. Prinz, "Using Buckling-Restrained Braces in Eccentric Configurations," Ph.D. Thesis, Department of Civil and Environmental Engineering, Brigham Young University, Provo, UT, 2010.

[240] G. S. Prinz and P. W. Richards, "Seismic performance of buckling-restrained braced frames with eccentric configurations," *J. Struct. Eng. ASCE,* Vol. 138, No. 3, PP. 345-353, 2012.

List of Figures

Figures

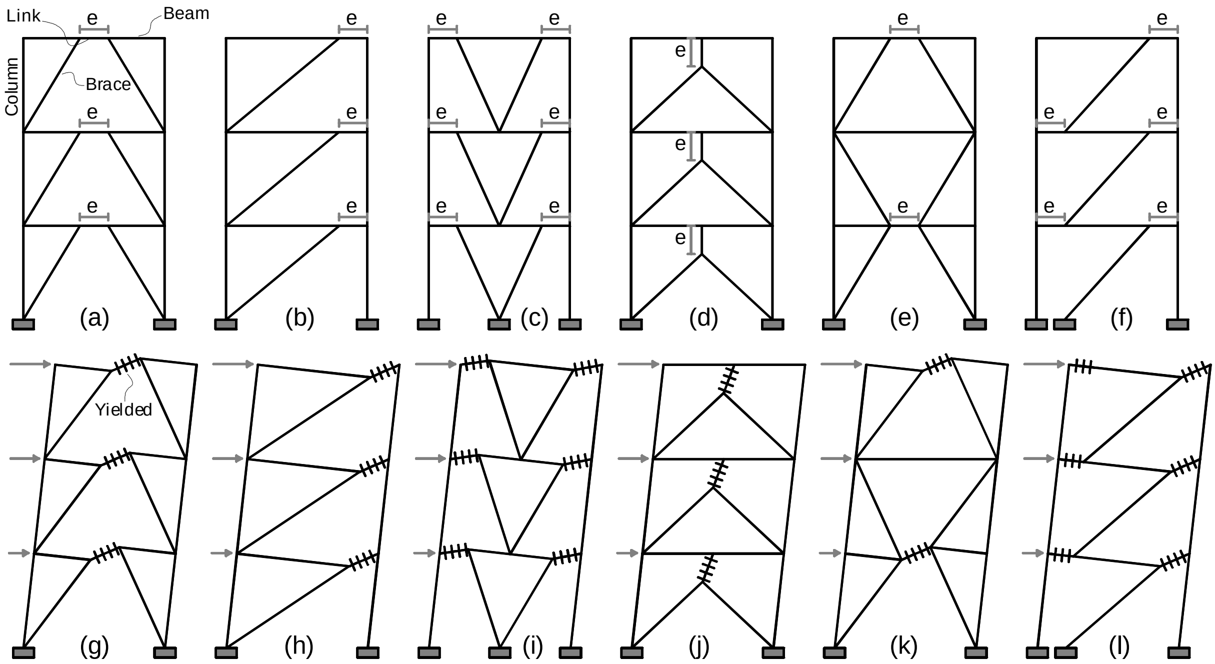

Figure 1: EBF configurations and their corresponding plastic mechanisms

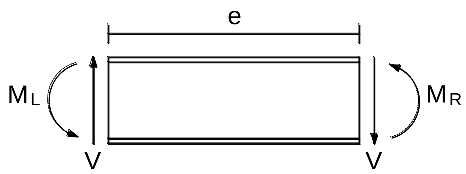

Figure 2: Free-body diagram of an isolated link segment

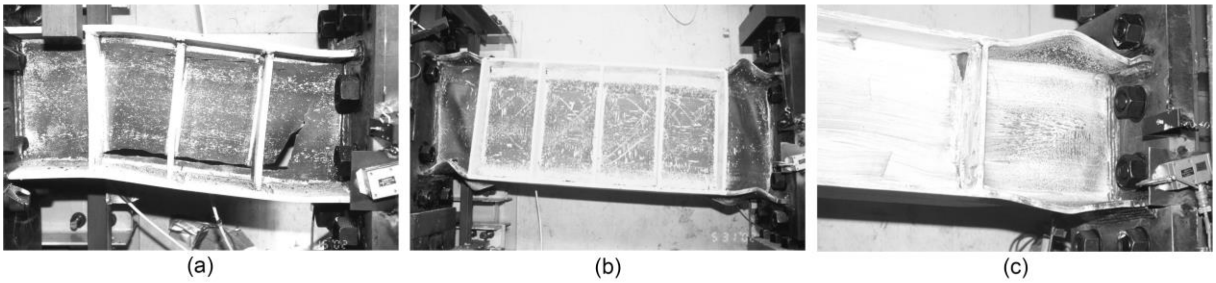

Figure 3: Failure mechanisms of (a) short, (b) intermediate, and (c) long links (photo courtesy of T. Okazaki)

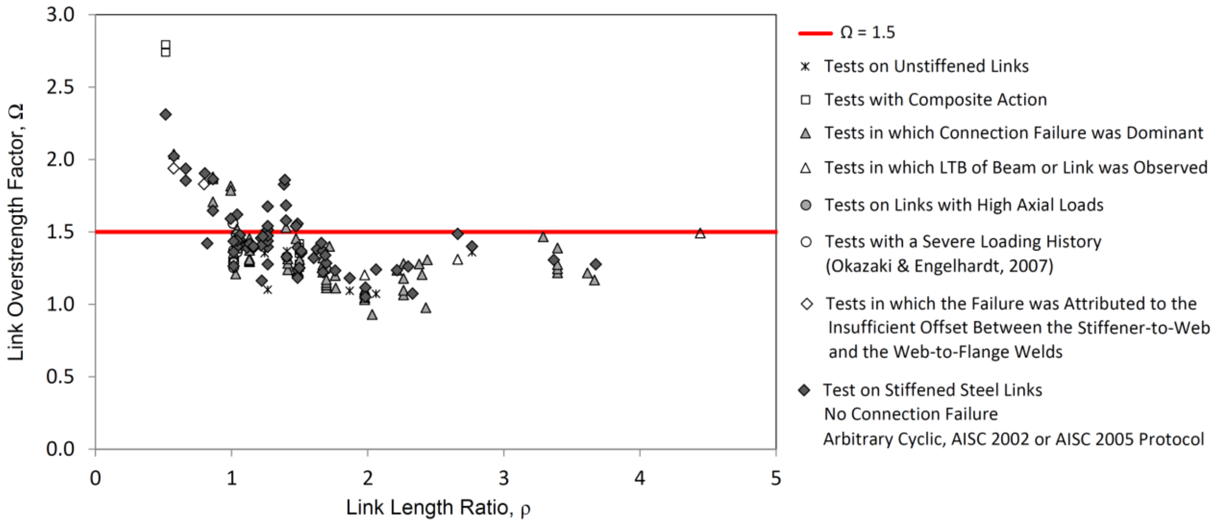

Figure 4: Link overstrength factors reported in different experimental studies

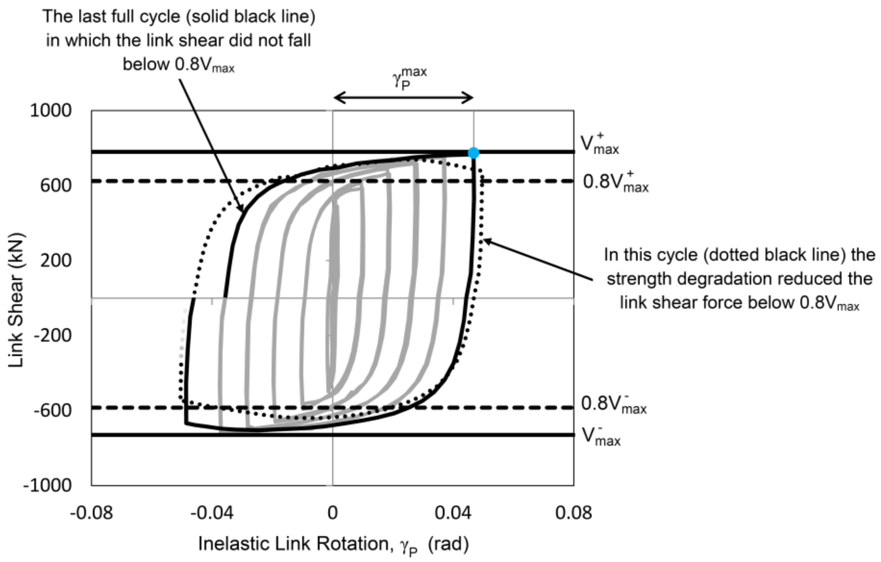

Figure 5: Inelastic link rotation capacity for the UTA 9 specimen (data courtesy of T. Okazaki)

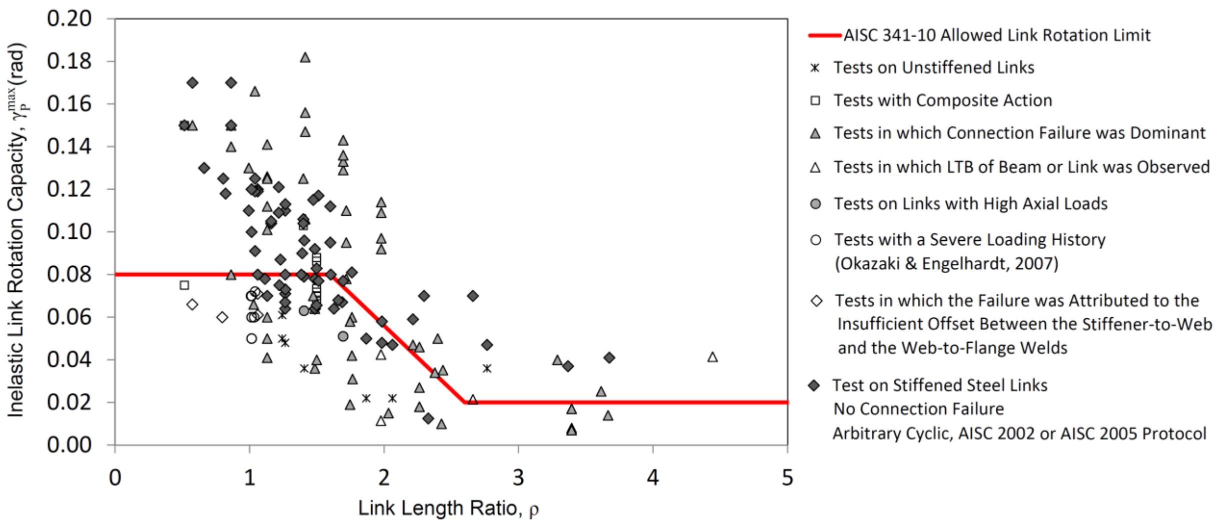

Figure 6: Inelastic link rotation capacities reported in different experimental studies

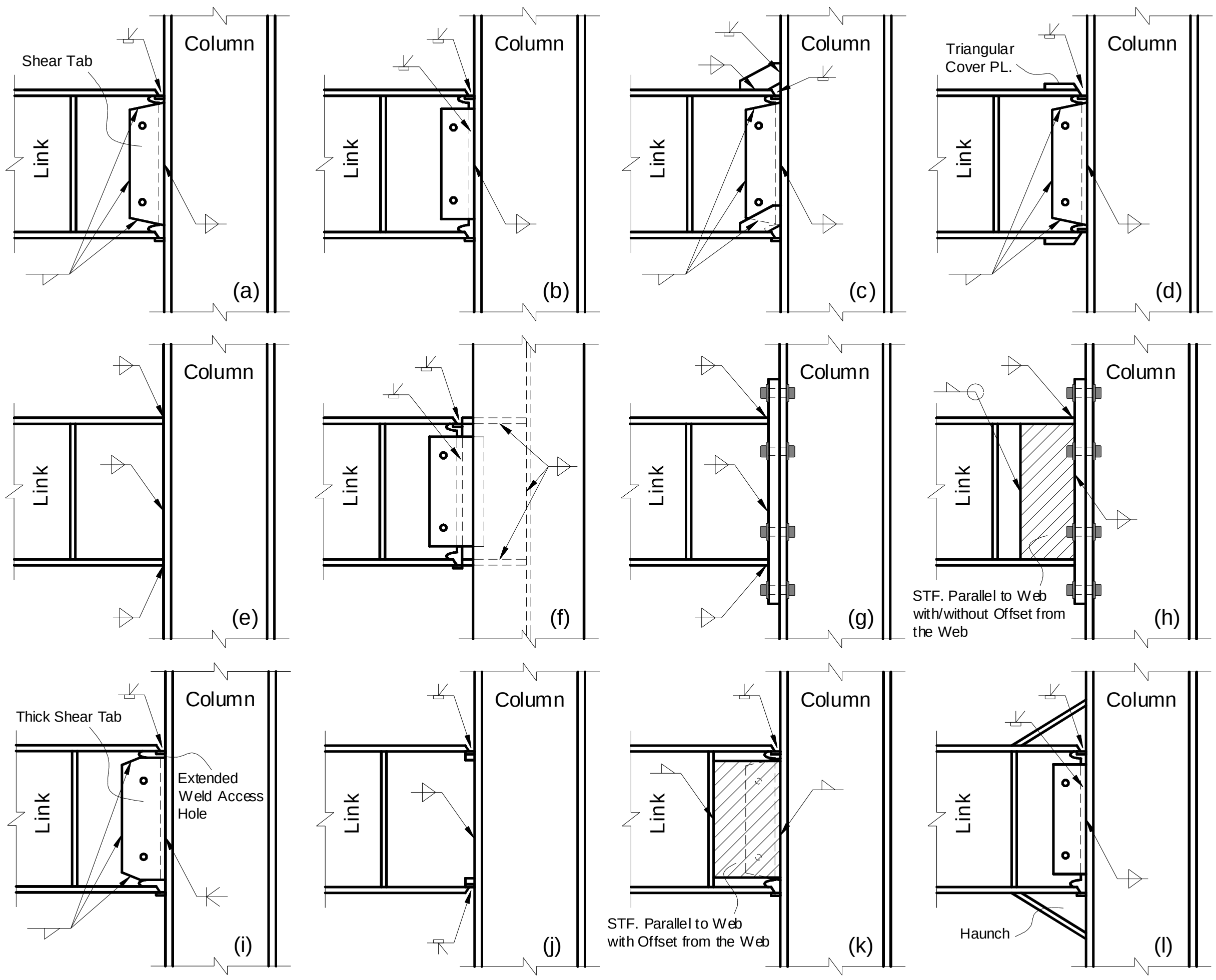

Figure 7: EBF link-to-column connections