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Numerical study on the behavior of link-to-column connections in eccentrically braced frames

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Abstract:

Geometry of eccentrically braced frames (EBFs) in some cases causes the connection of link beam to the column. The details of such conditions should be studied carefully due to the full plastic rotation in the link beam. In this research, the behavior of link-to-column connection is modeled and the failure modes are considered. Based on the previous researches shear link can exhibit better behavior in such conditions. However, different models are designed and analyzed using finite element method to investigate the influence of short, medium and long link beams on the failure modes of the connections. Two types of connections are applied in designing these models. The results show that models with access hole in connection details are failed before achieving 50% full plastic rotation in the link beams. Besides, in the connections without access hole, the link beams behave as ductile elements and experience up to 90% full plastic rotation that are recommended by the codes.

1. Introduction

Eccentrically braced frames (EBFs) are the lateral resistance systems in which the Link Beam behaves as dissipated part of structure. The design purpose for seismic-resistant steel Eccentrically Braced Frame (EBF) is that inelastic action under strong earthquake motion is restricted primarily to the links.

Some types of EBFs have one end of the link connected to a column. In such EBFs, the integrity of the link-to-column connection is crucial in ductile performance of the link, and therefore in the ductile performance and safety of the EBF. Despite their apparent similarities to moment frame connections, the force and deformation demands of EBF link-to-column connections are substantially different from moment frame beam-to-column connections. These connections are required to transfer large shear and moment developed in a fully plastic and strain hardened link, and accommodate large plastic rotation of the link.

Malley and Popov [1] observed that large cyclic shear force, developed in EBF links, could cause recursive bolt slippage in the welded flange-bolted web connections. The bolt slippage ultimately induced sudden failure of the connection by fracture near the link flange groove weld. Therefore, the use of this type of connection in EBFs is restricted .Engelhardt and Popov [2] tested long link elements attached to the columns, and observed frequent failures in the link-to-column connections due to the fracture of link flange.

It was recommended to avoid EBF systems with long links ($e > 1.6 M_P / V_P$) attached to columns due to the failures typically occurred before developing significant inelastic deformation in the link .

The design, detail and construction of EBF link-to-column connections are usually very similar to those of beam-to-column connections in moment resisting frames. However, the force and failure criteria of EBF link-to-column connections are significantly different from those of beam-to-column connections. Okazaki and Engelhardt [3] conducted an experimental study on the cyclic performance of four types of connections in EBF link-to-column connections (Pre-Northridge, modified weld access hole, free flange and non weld access hole Connections). The Behavior of connections were considered in short (shear), intermediate and long (moment) links. The experimental tests demonstrated that link-to-column connections are susceptible to fracture in the link flange near the groove weld, regardless the link length. Moreover, premature failure of link flange is a concern in the connections with long link ($e > 1.6 M_P / V_P$) to the column, and also in those with short shear links.

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According to Okazaki et al. [3], the connection details that perform well in the moment frame connections may perform inadequately in the link- to-column connections.

Developing the connection details that can sustain these demands without fracture remains an important challenge for researchers and engineers. In this research three types of connections are developed for using in the link-to-column connections. They have been modeled and analyzed by Finite Element Method. Their analytical results and ductility level are also presented in the following.

2. Analytical Models

General configurations of models and materials behavior are shown in Fig.1. These models are analyzed by finite elements method using solid elements. The cyclic loading sequence presented in the Appendix S of 2002 AISC Seismic Provisions [4] (displacement/ plastic rotation control) has been used for all analytical models.

An analytical program is used to investigate the behavior of link-to-column connections. This program consists of analytical models of some connections, tested in the experimental works (Okazaki and Engelhardt [3]) and some other types of connections, proposed after Northridge Earthquake. These analyses have been summarized in this study and compared with experimental results.

The details of models for analysis are based on the experimental models of Okazaki and Engelhardt. Therefore, link beam and column are made up of W18x40 and W12x120, respectively. According to the experimental investigation three types of links are modeled; that is short links ($e.V_P / M_P = 1.1$), intermediate links ($e.V_P / M_P = 2.2$) and long links ($e.V_P / M_P = 3.3$). A link-to-column connection with non-dimensional length of 1.6 was selected ($e.V_P / M_P = 1.6$) to meet the exact conclusion in some types of connections.

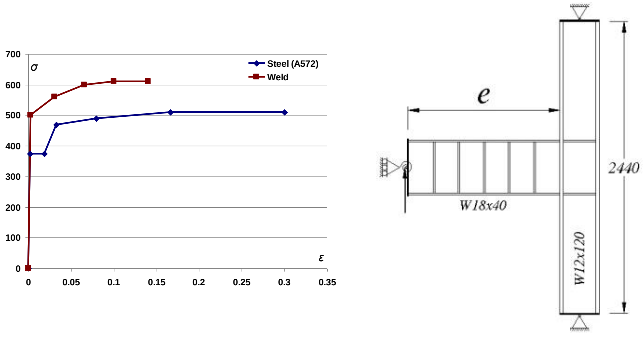


Fig.1: Material behavior and general configuration of models

3. Predicting the Ultimate Capacity

Buckling and rupture are two main modes of failures in steel structures. The modeling of buckling is required to induce an initial imperfection in the finite element program. In this research, the initial imperfection was assumed as 10 percent of plate thickness and its shape in accordance with the shape of first mode of elastic buckling.

A rupture index is computed at different locations of the connection in order to compare the analytical modeling with experimental results and predict the ultimate capacity of the material (before fracture) and fracture criteria [5]. This index (*RI*) is defined as:

$$RI = \frac{\varepsilon_p / \varepsilon_y}{\exp\left(-1.5\dfrac{\sigma_m}{\sigma_e}\right)} \qquad (1)$$

Where, ε_P is effective plastic strain; ε_y is yield strain, σ_m is hydrostatic stress; σ_e is effective stress (also known as the von mises stress). The idea of rupture index was motivated by the research of Hancock and Mackenzie (1976) [5] on the effective plastic rupture strain of steel for different conditions of stress triaxiality. According to a micromechanical Model, rupture in metal material will happen when:

$$\varepsilon_P > \varepsilon_P^{critical} = \alpha.\exp\left(-1.5\frac{\sigma_m}{\sigma_e}\right) \qquad (2)$$

Comparing the Equations (1) and (2), the critical value of rupture index is as below:

$$RI^{critical} = \frac{\alpha}{\varepsilon_y} \qquad (3)$$

Kanvinde and Deierlein [6] suggested the value of 2.6 for α parameter (for steel A-572). Then the Value of $RI^{critical} = 1405$.

4. Behavior of Connections in Eccentrically Braced Frames

4.1 Pre-Northridge Connections

The configurations of these types of connections are as the same of the connections tested by Okazaki and Engelhardt [3]. According to the analytical results of finite element method, the beginning of fracture at the end of weld access hole is happened at the similar plastic rotation obtained in the experimental test (Fig. 2). Fig 3-a shows the moment-rotation hysteresis loop at column face and adjacent to bracing connection. The analytical and experimental results are compared and shown in Fig.3-b. Both results are also compared with code provisions in this figure.

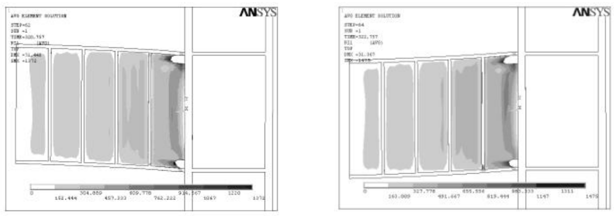


Fig.2: Predicting the location of fracture in short link with PN connection ($\gamma_P = 0.05^{rad}$)

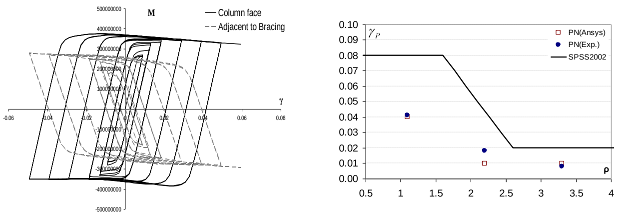


Fig. 3: a) Hysteric loop at column face and adjacent to bracing connection

b) Comparing the analytical and experimental results with code provisions

4.2 Free Flange Connections

These types of connections are modeled according to the experimental details tested by Okazaki and Engelhardt. Fig.4 presents the overall geometry of link-to-column connection details of their study. Based on the results of finite element analysis, the fracture in short link happens at the link web around shear tab (similar to the case of test results) at plastic rotation of 0.07^{rad} (Fig.5). Moreover, the fracture in the long link happens at the shear tab/link web groove weld similar to the location in experimental results. The plastic rotation is 0.02^{rad} (Fig.6). Fig 7-a compares moment-rotation hysteresis loop at the end of shear tab and adjacent to bracing connection in the long link. The analytical and experimental results are compared and shown in Fig.7-b.

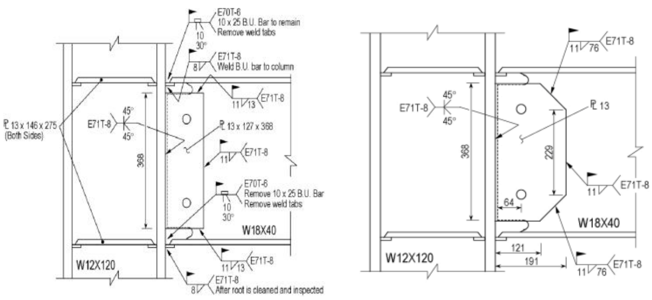


Fig.4: Connection details for short and long links

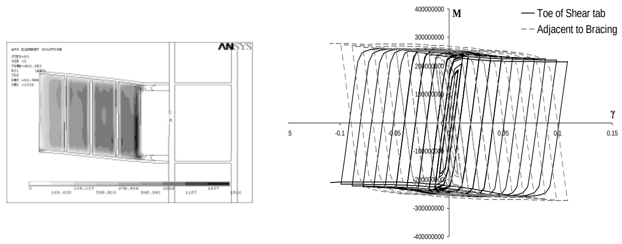


Fig. 5: a) Location of fracture in short link (at link the web, around the shear tab) ($\gamma_P = 0.08^{rad}$)

b) Hysteric loop at the end of shear tab and adjacent to bracing connection (short link)

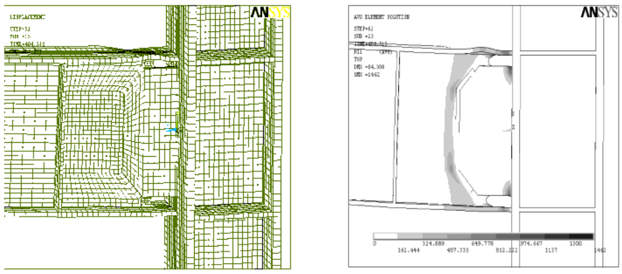


Fig.6: a. The shape of link deformed at the connection

b. Predicting the location of fracture in long link with FF connection ($\gamma_P = 0.02^{rad}$)

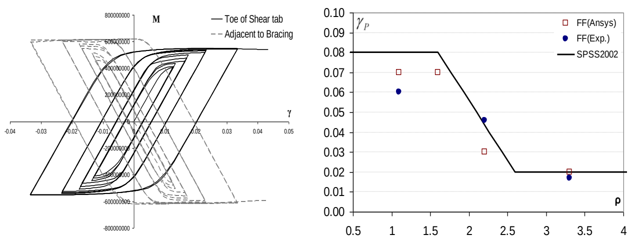


Fig.7: a) Hysteric loop at the end of shear tab and adjacent to bracing connection (long link)

b) Comparing the analytical and experimental results with code provisions

4.3 Connections with reduced beam section (RBS)

The reduced beam section has been designed according to the AISC358-05 [7]. The links properties (M_P and V_P) are defined based on the link beam dimensions in reduced region as the effective section. The link length is assumed equal to the distance between the centers of reduced beam sections. Fig. 8-a shows the detail of reduced section of intermediate link beam. The link with shear behavior (short link) has not been investigated.

The failure mode of links without lateral bracing is lateral torsional buckling (Fig. 8-b). Therefore, lateral bracing around the reduced link section should be provided. The links have been analyzed assuming the lateral bracing at the end of reduced beam section (points C and D, Fig. 8-a). The configuration of lateral bracing is not considered in this study; however, it can be similar to what is used in steel moment-resisting frames with reduced beam sections.

The main form of fracture is the rupture at the end of weld access hole (similar to PN connections) in the lateral braced beams. However, local buckling has been formed in the web of reduced beam section.

The fracture of intermediate link has been happened at the end of weld access hole (similar to PN connection) at plastic rotation of 0.02^{rad} (Fig. 9). Therefore, the ultimate plastic rotation without any degradation in cyclic loop and fracture can be predicted at 0.01^{rad}. Long link connection can bear a complete cycle with plastic rotation of 0.02^{rad} without any degradation in strength and fracture (Fig. 10).

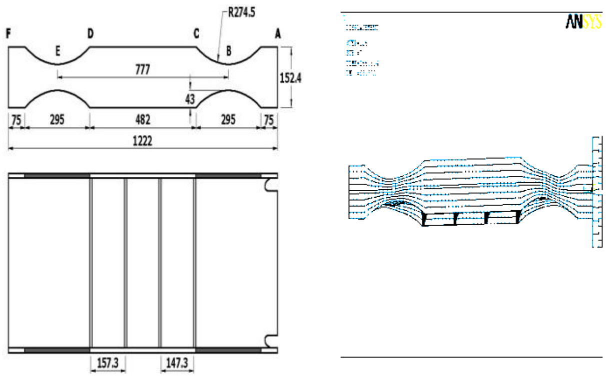


Fig. 8: a) The properties of reduced section for intermediate link b) Lateral torsion buckling in the intermediate beam without lateral bracing

4.4 Welded Top and Bottom Haunch Connections

In this type of connections, the failure mode is concentrated at the toe of haunch (Figs. 12- 13). This sort of failure is the main failure mode that can be happened in the links, if the buckling mode is prevented. Comparing the hysteric.

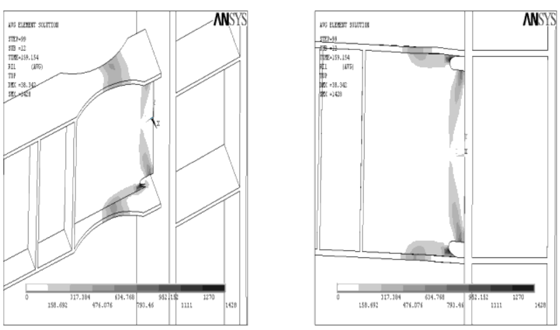


Fig. 9: Location of fracture in the intermediate link with reduced beam section and lateral bracing ($\gamma_P = 0.02^{rad}$)

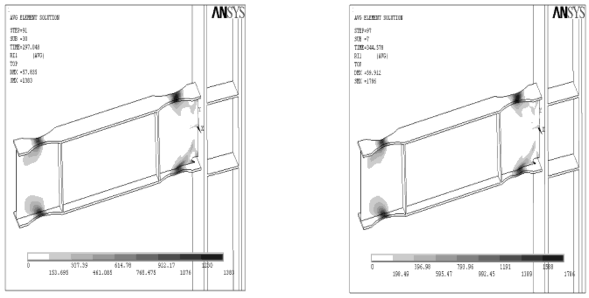


Fig. 10: Location of fracture in the intermediate link with reduced beam section and lateral bracing ($\gamma_P = 0.03^{rad}$)

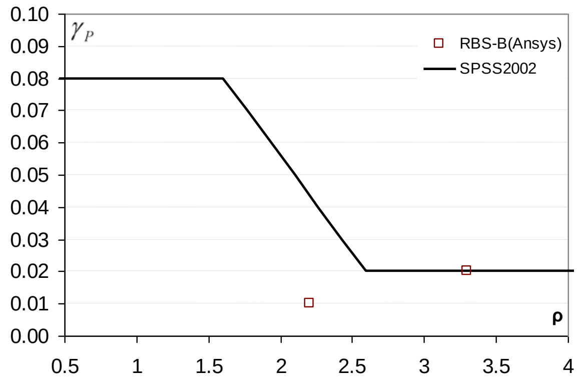


Fig. 11: comparing the analytical results and code provisions

loop at the toe of haunch and adjacent to bracing connection in the long link shows that moment capacity values are virtually equal at both end of the link (Fig. 14-a). Additionally, a gradual strength degradation of link beam is happened due to the local buckling of flange. The short link could not meet the value suggested in code provision. However, in other links (intermediate and long links), the ultimate capacity of the connections is very close to the code provision value (Fig. 14-b).

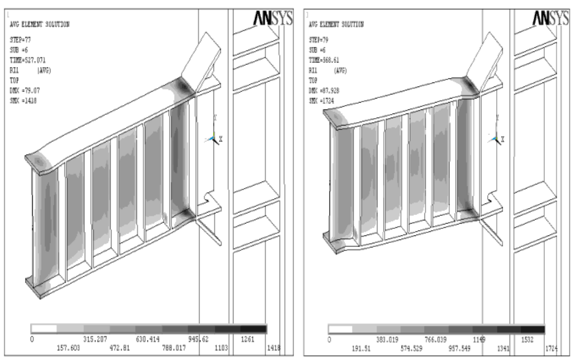


Fig. 12: Location of fracture in short link ($e = 1.6\, M_P / V_P$)

($\gamma_P = 0.08^{rad}$)

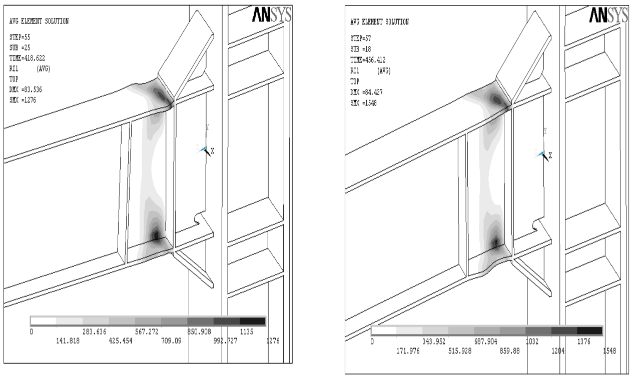


Fig. 13: Location of fracture in long link ($\gamma_P = 0.03^{rad}$)

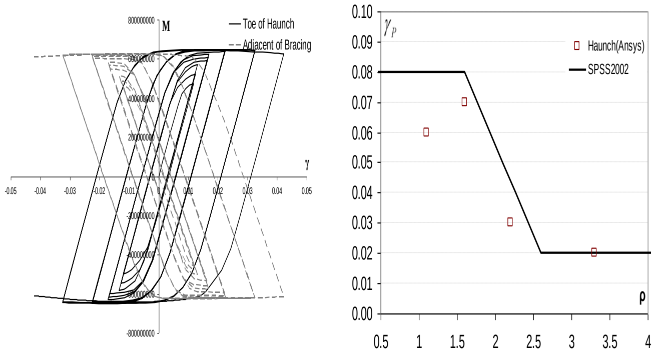


Fig. 14: a) Hysteric loop at the toe of haunch and adjacent to bracing connection (long link)
b) Comparing the analytical results and code provisions

4.5 Rib-Reinforced Connections

This Type of connection is generally used to enhance the seismic performance of welded steel moment connections by increasing the moment of inertia near the column face and reducing the tensile stress in the groove weld. In this study, rib plates are designed base on the method suggested by Lee [9].

The results of analysis are indicated that short link could achieve one complete cycle at 0.09^{rad}. The failure mode of this link is the rupture occurred simultaneously on the flange (at the toe of rib plate) and the web of link beam (Fig. 15). Furthermore, long link surpassed the implied minimum of $\gamma_P = 0.02^{rad}$ and reached to 0.03^{rad} of plastic rotation. Fig.16 shows the location of fracture in long link is on the flange of link beam at the toe of rib plate. According to Fig. 17, the hysteric loop of link beam slightly decreases due to the flange local buckling at the toe of rib plate. Link beams with the normalized lengths of 1.6 and 3.0 could achieve one complete cycle at 0.07 and 0.03^{rad} of total plastic rotation, respectively. Failure mode of all links has been rupture of flange at the toe of rib plate excluding that of short link.

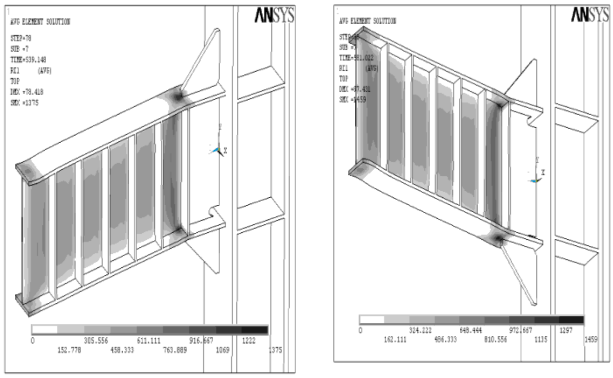


Fig. 15: Location of fracture in short link ($e = 1.6\, M_P / V_P$)

($\gamma_P = 0.08^{rad}$)

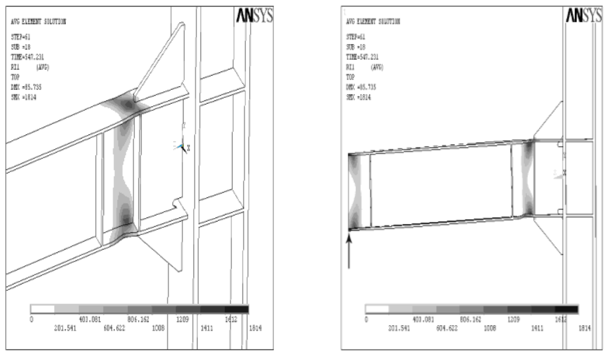


Fig. 16: Location of fracture in long link ($\gamma_P = 0.03^{rad}$)

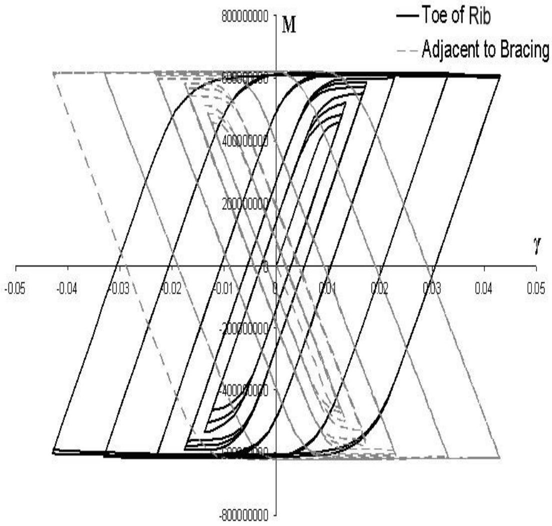
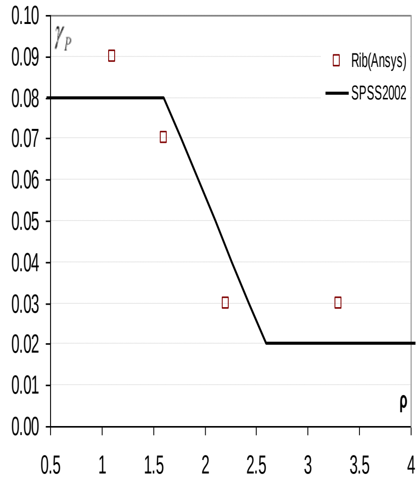


Fig. 17: a) Hysteric loop at the toe of Rib and adjacent to bracing connection (long link)



b) Comparing the analytical results and code provisions

5. Conclusion

This research studies the behavior of four types of link-to-column connections. These connections details are as the same as experimental studies, conducted by former researchers. All connections have been modeled by finite element method. Therefore, the numerical results should be verified by experimental studies. The failure mode (buckling or rupture) and ultimate capacity have been investigated in each models.

The analytical results show that the Weld Access Hole (WAH) is the main factor for premature fracture in link-to-column connections. In the connections with WAH, the failure mode is appeared before achieving 50% of full plastic rotation in the link beams. Strengthening the connection with shear tab (Free Flange Connections) can improve the link capacity; however, the failure mode of connection is not improved significantly.

Using Reduced Beam Section (RBS) in long link will change the location of failure mode and the plastic mechanism occurred in the reduced section (far from WAH). This behavior is not observed in the intermediate link. Therefore, RBS connections are effective in the long links.

Link-to-column connections, strengthened by welded top an bottom haunches, show that these types of connections can not exactly endure the ultimate capacity of the link plastic rotation (like Free Flange connections). However, using these types of connections prevents the failure mechanism in welded access hole.

Rib-reinforced connections could surpass specification requirements in short and long links. The intermediate links have not satisfied minimum requirements of provisions. However, they could achieve rotation capacity close to the code requirements. Therefore, these types of connections can be a proper choice for designing link-to-column connections.

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