The Behavior of Modified Long Links with Supplemental Double Stiffeners on Eccentrically Braced Frames

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Abstract. Initial failure of long links caused by fracturing and buckling occurs on the flange and web at the end of the link. Local damages are caused by the influence of the dominant bending moments compared to shear forces. The advantage of using long links includes allowing for larger openings in rooms, which makes it popular among architects. Efforts to prevent these specific failures are not covered in the rules and there are few researchers that examine improving the performance of long links. The focus of this study is to provide information on using supplemental double stiffeners at the ends of the link without changing the long link behavior. The behavior of long links is maintained by keeping the flange failures on the flange at the end of the link. The supplemental double stiffeners improve the performance of the long link by extending the inelastic zone and slowing the failure rate of the flange. This experimental study was carried out on four models of the long link consisting of a standard model and a model modified by the addition of supplemental double stiffeners at the flange. Long link models were modified with variable thickness and holes width on the supplemental double stiffeners. The results showed that the addition of the supplemental double stiffeners improved the performance of long links compared to the standard link that is in accordance with the requirements of AISC 341-10. The supplemental double stiffeners are an alternative to improved long link performance, making it more effective in the application of its use in steel construction.

1. Introduction
Eccentrically braced frame system (EBFs) has excellent elastic stiffness under lateral loading and has good ductility when under large earthquake loads [9]. EBFs is an excellent compromise between the moment resisting frame (MRF) and concentrically braced frames (CBF) system so that it is suitable and beneficial to apply structures to make them resistant for earthquake loads [11], [29], [31]. EBF system can be planned in the state of a moderate earthquake load to minimize structural and nonstructural damage. In the event of a severe earthquake, EBF structure must be designed to be able to maintain the inelasticity of the link which in extreme cases can significantly cause permanent deformation. The ability of EBFs to accommodate large room openings make this very popular among building designers.

Based on EBFs method, yielding occurs on a link element due to either shear or moment. Yielding is highly dependent on the length of the link [7], [8]. For long links, shear experiences yield when the moment reaches the end of the plastic moment (Mp). The collapse of a long link is due to bending deformations caused by the lateral torsional buckling and buckling in the flange and web. Enormous strain on both ends of the link can cause a fracture at the end of the weld joint. The shear force acting
on the link has a constant value throughout the link, whereas the moment is not. In addition, the moment is concentrated at both ends of the link and the value decreases towards the center.

Engelhardt and Popov [7] stated that the use of long links on the EBFs structure is not recommended because of its low performance. Subsequently, many experimental and analytical work [4], [9-10], [22], [24-25] discussed the effect of link length on the performance of the EBFs. All of them showed that the performance of long links is less than those of short ones. Therefore, they recommend that links exceeding $e > 1.6 \frac{M_p}{V_p}$ in length not to be used as a link to the column on EBFs.

Several researchers showed some variation in results related to long links indicated that its performance could still be improved. Berman [2], Danesmand [4], Mohebkhah [14], Okazaki [21] and Yurisman [28] suggest that the value of the rotation and strength can be increased which improves the overall performance of the long links. Rotation of the link can still exceed the minimum threshold required, 0.02 rad, and the overstrength can exceed 1.0. Okazaki [17] reported long links with an overstrength value of 1.22 and inelastic rotation ranging from 0.037 to 0.041. Mohebkhah [14] found an average overstrength value of 1.4 with inelastic rotation ranging from 0.04 to 0.09 rad. However, the weakness in the long link is caused by the formation of a plastic hinge on the flange in the welding zone at the end of the link with is then accompanied by failure caused by lateral torsional buckling, and fracture and buckling of the flange [4-5], [8], [14], [16-20], [24], [28]. Some researchers suggested that avoid this weakness by limiting the slenderness of the flange. Richards [24] and Okazaki [17] limited the slenderness to between $0.30 \sqrt{E_s/E_y}$ and $0.38 \sqrt{E_s/E_y}$. It was found that the intermediate links are most susceptible to local buckling, so it does not comply with the prediction of the rotation design. This condition is caused by the influence of the spacing between web stiffeners and limiting the slenderness of the flange to $b_f/2t_f$.

Subsequent research was conducted to improve the performance of the link through numerical and experimental study. Prinz and Richards [23] reduced web section of the link that is connected to the column and found that the value of strain and stress on the flange is reduced, but the strain and the triaxial stress increase on the web at the edges of the hole. To cope with this phenomenon, Naghipour [15] suggested reducing the cross-section of the web that has the capacity to rotate about the same or smaller than the cross-section of the link without a reduced web. Beam-column link model with reducing beam section (RBS) found that the time yielding, and ductility is higher than the model without RBS. Berman [2] reported that the reduction in cross-section of the flange at the end of the link can reduce the strain significantly on all the links, especially plastic strain on the flange at the end of the link where the location of a potential fracture can occur.

Based on the results from the previous studies, failure of long links occurs at the end of the link. Some research has been done to avoid failure due to bending dominant force by adding stiffeners to the web and flange. Yurisman [28] conducted a study on short links that were added diagonal stiffeners on the web. Danesh [15] reported utilizing diagonal stiffeners on the web and supplemental stiffeners on the flange. Furthermore, Okazaki [21] and Hong [12] reported using supplemental double stiffeners in connecting a beam to a column and found excellent performances on the link. Stephens and Dusicka [26] reported that the use of continuous stiffeners on the short link could improve its performance, exceeding the standard shear link performance according to the requirements of AISC 341-10. Furthermore, Yurisman [28] and Danesh [5] reported that retrofitting the link delayed failure at the end of the link. Reinforcement at the ends of long links prevents an early failure that is preceded by the formation of the plastic hinge on the flange.

2. Material and Method

The numerical analysis of long link modified with supplemental double stiffeners was conducted by ANSYS 16.0 software in accordance with the requirements specified by AISC 341-10. The results of the numerical analysis showed that the failure of long links occurs under three conditions. First, a failure occurs in a zone close to the ends of links without the extension of the plastification zone.
Second, the failure occurs in the zone near the confluence with the extension plastification. Third, failure shifts towards the middle of the link without plastification at both ends. The experimental study showed that the long link modifications maintained the failures in accordance with the requirements of AISC 341-10.

2.1 Improved Links Proposed in this Study

This study used supplemental double stiffeners that are connected between each edge of the link and the intermediate stiffeners. The supplemental double stiffeners are used to get the best performance from the long link. It does not intend to change the performance of the long link to be similar to that of a short link or an intermediate link. This can be done by controlling the capacity of the overall modulus section. In addition, the best performance of the long link can be measured by using a preliminary numerical analysis where the length of the plastification zone, the model failure position, and the model failure mechanism are observed. These variables can be determined by changing the thickness and width of the holes of the supplemental double stiffeners. The main objective of using double stiffeners is to increase the capacity of the modulus section in resisting flexural force as well as to increase the capacity of the long link to withstand shear forces. Because the effects of shear forces on the long links are not dominant, the modulus section is decreased by providing slide open holes on the outer sides of the double stiffeners, as shown in Figure 1. Limitations used in this study for the supplemental double stiffeners are as follows:

\[
r = \frac{1}{2} B_{ht}; \quad 0.1 H \leq T_{ht} \leq 0.25 H; \quad 0.2 b_f \leq B_h \leq 0.4 b_f
\]

where \(b_f\) = beam flange width and \(H = \) beam depth.

The supplemental double stiffeners on the profile WF increases the capacity of moments and shear forces, thus preventing early failures due to buckling and fracture that occurs on the flange at the end of the links. The supplemental double stiffeners could be an alternative to increase the long links’ performance, thereby becoming more effective in their use in steel construction. The supplemental double stiffeners can additionally increase link capacity in restraining flexural force concentrated at the ends of the long links so the failure caused by buckling and fracture of the flange can be slowed.

Fig. 1 Supplemental double stiffeners limitations

2.2 Material

The steel profile used for the numerical and experimental study was WF 200.100.5,5.8, which met the requirements for slenderness and on the capacity of the power tools used in laboratory testing. Profile Wide flange (WF) was more commonly used in the structure of the EBF. The long link model follows the specifications set by AIS 341-10. The vertical stiffeners were placed on both sides of the web. The width of the vertical stiffeners should be more than \(b_f - 2t_w\), and the thickness should be the greater value between 0.75 \(t_w\) and 10 mm. The maximum spacing between vertical stiffeners was 1.5 \(b_f\). Link length ratio (\(\rho\)) analyzed was 3.03 of 100 cm, complying with the requirements of AISC 341-10. Table I shows the of yield stress (\(F_y\)), ultimate stress (\(F_u\)) and elongation (%) of steel taken from each element of the link and stiffeners that was subjected to a tensile test.
TABLE I
RESULTS OF TENSILE TEST

| Model       | Fy (Mpa) | Fu (Mpa) | Elongation (%) |
|-------------|----------|----------|----------------|
| Web         | 501.99   | 595.61   | 16.05          |
| Flange      | 401.45   | 540.08   | 24.55          |
| Plate 6 mm  | 303.24   | 376.18   | 27.25          |
| Plate 10 mm | 359.81   | 519.43   | 27.60          |

2.3 A subsection
Selection of the experimental model was based on the results of the numerical analysis. The model was selected based on performance criteria which represented the best model to resist loading. The standard link and the link modified with the supplemental double stiffeners are shown in Figure 2. Thickness and hole width in the supplemental double stiffeners served as the main parameter to control and view the position and length of plastification failure. Configurations for the supplemental double stiffeners are shown in Figure 3. This type of failure could be arranged by both parameters. Modulus of cross-section values of the models, which was based on the thickness and hole width, are shown in Table 2.

![Fig. 2 Models](image-url)

A groove weld was placed at the end of the outer side of the flange and on the outer supplemental double stiffeners, with the angle of the flange cut by 45°. The purpose of the weld was to avoid early failure due to fracture at the end of the link. However, a fillet weld was performed on the connection zone between the vertical web stiffeners and the inside of the flange, as well as on both sides of the web and at both ends of the link. Initial failure due to fracture is predominantly caused by concentrated moment force. It is predicted that there would be no early failures in the welding zone, so the
connection will remain secure until the model fails. The type of weld connecting a standard link and a modified link with supplemental double stiffeners are shown in Figure 4.

![Fig. 3 Configurations of the model with supplemental double stiffeners (a) MPVT02, (b) MPVT03, and (c) MPVL0](image)

**TABLE II**

| Model     | Supplemental double stiffeners | Modulus of cross-section |
|-----------|--------------------------------|---------------------------|
|           | Thickness ($t_i$) (mm) | Hole width ($t_h$) (mm) | $Z_p$ (mm$^3$) | Enhancement (%) |
| Standard  | -                             | -                         | 200152        | 0.00%           |
| MPVT02    | 6                             | 30                        | 255592        | 27.70%          |
| MPVT03    | 8                             | 30                        | 274072        | 36.93%          |
| MPVL03    | 8                             | 40                        | 292312        | 46.05%          |
2.4 Loading Pattern

Loading patterns are arranged using AISC 341-10 as shown in Figure 5, which followed the loading pattern based on test results from Richards and Uang [25] as well as the proven performance based on the results of experimental studies comparing several types of loading patterns by Okazaki [16]. Although the AISC 341-10 guidelines contain the loading tests for a beam to column connection in a moment–frame system, this loading pattern can be applied to test the link to columns connection on an EBFs structure. Daneshmand [4] reviewed the effect of using different loading patterns: AISC 341-02, AISC 341-05 and ACT24 to a link with similar properties. The results showed that the use of loading pattern AISC 341-05 demonstrated a rotational capacity greater than the results of the other patterns. This fact confirms that loading patterns affect rotational capacity. Many recent studies were based on the AISC 341-10 rules regarding the use of EBF loading patterns.

Fig. 5 Cyclic loading protocol using AISC 341-10 guideline

Selecting the correct electoral boundary conditions and loading pattern would reflect the real conditions of the EBF structure. Richards and Uang [18] proposed a boundary conditions model of the link in which the load is applied by imposing transverse displacements on the right end nodes. Left end nodes were permitted to translate horizontally but were constrained so that all have the same horizontal translation. This loading method...
will produce a constant shear force along the link with the same end moments and without any axial force. This boundary conditions model was applied by later researchers, including Mohebkhah [14], Daneshmand [4] and Nikuokalam [30].

Fig. 6 Boundary conditions with restraints $\Delta x = 0$ and without restraints the axial force $\Delta x \neq 0$

2.5 Loading Pattern

Dusicka [6], Corte [3] and Stephens [26-27] applied the boundary conditions where the degree of freedom to translate and rotate in all axial direction on one side of the link is restrained. The other end of the link could translate in the load direction and in the direction parallel to its length as shown in Figure 6. Corte [3] studied a combination of axial force parameters, the ratio of the flange to the web, and the ratio of the link length and the high-profile cross-section used.

Fig. 7 The experimental setup

The results of the experimental and theoretical studies indicated that there is a strong value over different ranges due to the presence or absence of an axial force on the link. Corte [3] studied the occurrence of buckling on a model IPE 600. The model with the restraints on the axial force showed a hysteretic curve and a positively sloped trend caused by the onset of the dragging axial force which produced a supplemental influence and when adding transverse force. In the case of other models, hysteretic curves with and without restraints axial force showed stable behavior suggesting that different loads in both boundary conditions can be ignored. In this study, the boundary conditions
adopted were similar to the work conducted by McDaniel [13], Yurisman [28], Dusicka [6] and Corte [3] through restraining the deformation due to an axial force. The experimental setup is shown in Figure 7.

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