Behavior of Link-to-Column Connections in Eccentrically Braced Frames

Fakhredin Danesh
K.N.Toosi University of Technology, Iran

Mohammadali Faridalam
Arad Tadbir Company, Iran

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SUMMARY:
Geometry of Eccentrically Braced Frames (EBFs) in some cases causes that the link beam is connected to the column. In this condition, because of achieving the full plastic rotation in the link beam, the connection must be detailed carefully. In this paper, the behavior of the link-to-column connection is modeled and the failure modes are considered. Although previous researches showed that the shear link can exhibit better behavior in such condition, in order to investigate the influence of short, medium and long link beams on the failure modes of the connections, different models are designed and analyzed using finite element method. In design of these models, two types of connections are applied. The results show that models using access hole in connection details are failed before achieving 50% full plastic rotation in the link beams, while on the other types of connections without access hole, the link beams behave as a ductile elements and experience up to 90% full plastic rotation that are recommended by the codes.

Keywords: Eccentrically Braced Frames, Link-to-Column Connection, Seismic Design

1. INTRODUCTION

Eccentrically Braced Frames (EBFs) are a type of lateral resistance system in which the Link Beam behaves as dissipated part of structure. The integrity of the link-to-column connection is designed such that to achieve high ductility of the link beam.

Malley and Popov observed that the large cyclic shear force developed in EBF links could cause repetitive bolt slippage in welded flange-bolted web connections. The bolt slippage ultimately induced sudden failure of the connection by fracture near the link flange groove weld. It is therefore, the use of this type of connection in EBFs is restricted. Engelhardt and Popov tested long link elements which are attached to the columns, and observed frequent failures at the link-to-column connections due to fracture of the link flange. Since these failures typically occurred before significant inelastic deformation that was developed in the link, so it was recommended that EBF systems with long links (link length, e, of $e > 1.6 M_P / V_P$) attached to columns should be avoided.

Usually EBF link-to-column connections have been designed, detailed, and constructed very similar to the beam-to-column connections in moment resisting frames. However the force and failure criteria at EBF link-to-column connections are significantly different. Okazaki and Engelhardt conducted an experimental study to investigate the cyclic performance of four types of connection in EBF link-to-column connections (Pre-Northridge Connections, Modified Weld Access Hole Connections, Free Flange Connections, No Weld Access Hole Connections). The Behavior of connections were considered in Short (shear), Intermediate and Long (Moment) Links. The Experimental test was shown that the majority of link-column specimens failed by fracture of the link flanges near the groove weld. They suggested that the premature failure of the link flange is a concern not only for connections of a long link ($e > 1.6 M_P / V_P$) to a column, but also for connections with short shear links.

The aim of this research is to develop two types of connection for using in the link-to-column connection. These types of connections were modeled and analyzed by Finite Element Method. The analytical results and the level of ductility of mentioned connections are also presented in this paper.

2. ANALYTICAL MODELS

A general view of models and materials behavior is shown in Figure.1. The finite elements method using solid elements is used to analyze these models. The cyclic loading sequence provided in Appendix S of the 2002 AISC Seismic Provisions (Displacement / Plastic rotation control) was used for all analytical models.

An analytical Program is used to investigate the behavior of the Link-to-Column Connections. This program is consist of Analytical models of some connections were tested in experimental work (Okazaki and Engelhardt) and some other types of connections were proposed after Northridge Earthquake. In this paper, a summary of these analyses and comparison with experimental results is presented.

The details of models for analysis are based on experimental models of Okazaki and Engelhardt. So link beam is made of W18x40 and Column is made of W12x120. According to the experimental investigation three types of links are modeled (Short Links $e.V_P / M_P = 1.1$, Intermediate Links $e.V_P / M_P = 2.2$ and Long Links $e.V_P / M_P = 3.3$). To achieve an exact conclusion in some type of connections, a link-to-column connection with non-dimensional length equal to 1.6 was selected ($e.V_P / M_P = 1.6$).

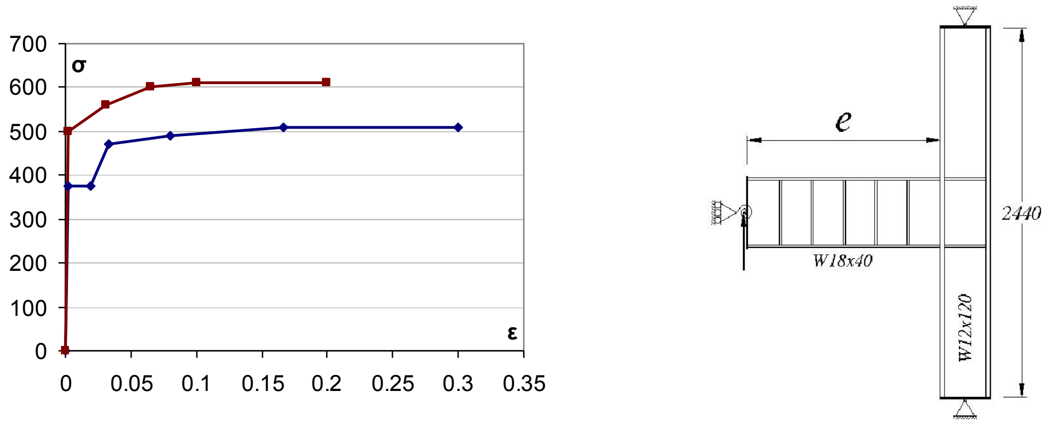

Figure 1. material behavior and general view of models

3. PREDICTION OF 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 Finite Element program. In this investigation, the initial imperfection was assumed equal to 10 percent of plate thickness and the shape of imperfection is accordance with the shape of first mode of elastic buckling.

To compare the analytical modeling with experimental results and to predict the ultimate capacity of the material (before fracture) and fracture criteria, a rupture index is computed at different locations of the connection. The rupture index (*RI*) is defined as:

$$RI = \frac{\varepsilon_p / \varepsilon_y}{\exp\left(-1.5\frac{\sigma_m}{\sigma_{eff}}\right)} \tag{3.1}$$

Where ε_P, ε_y, σ_m and σ_e are effective plastic strain, yield strain, hydrostatic stress, and effective stress (also known as the von Mises stress), respectively. The rupture index was motivated by the research of Hancock and Mackenzie (1976) 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:

$$\varepsilon_P > \varepsilon_P^{critical} = \alpha.\exp\left(-1.5\frac{\sigma_m}{\sigma_e}\right) \tag{3.2}$$

A comparison in equation (3.1) and (3.2) shows the critical value of Rupture index as below:

$$RI^{critical} = \frac{\alpha}{\varepsilon_y} \tag{3.3}$$

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

4. BEHAVIOR OF CONNECTIOS IN ECCENTRACLLY BRACED FRAMES

4.1. Secondary Pre-Northridge Connections

The configurations of these types of connections are as the same connections as were tested by Okazaki and Engelhardt. The analytical results obtained from finite element method shows that the beginning of fracture at the end of weld access hole is happened at similar plastic rotation value indicated in the experimental results. A comparison between analytical and experimental results is shown in figure.3-b. Both results are also compared with code provisions in this figure.

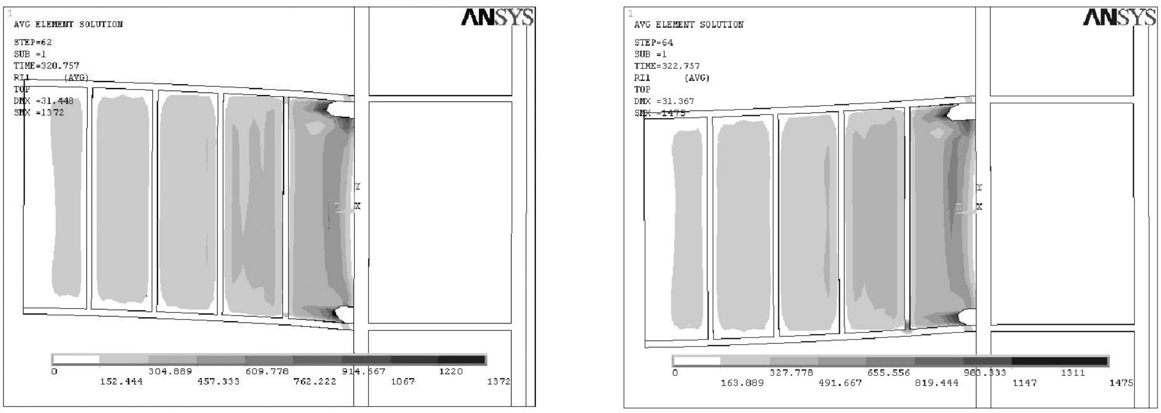

Figure 2. Prediction of location of fracture in short link with PN connection ($\gamma_P = 0.05^{rad}$)

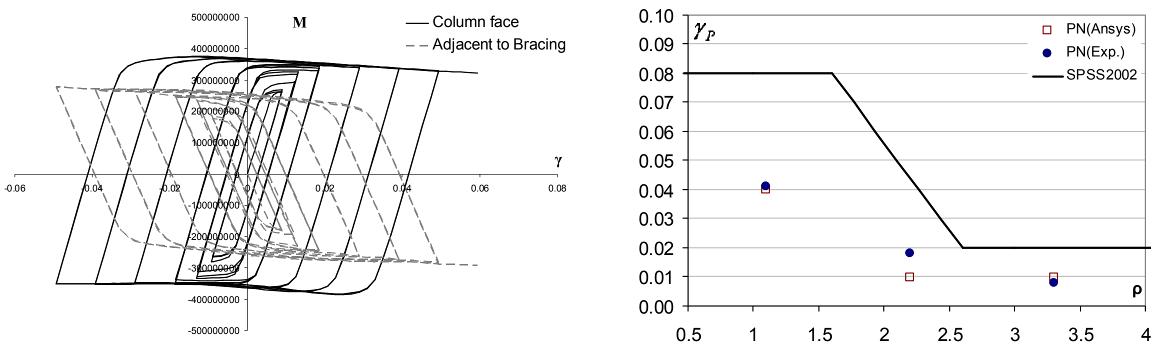

Figure 3. a. Hysteric loop at column face and adjacent to bracing connection
b. Comparison between analytical and experimental results with code provisions

4.2. Free Flange Connections

These types of connections are modeled according to the experimental details tested by Okazaki and Engelhardt. The results from finite element analysis show that the fracture in short link happens at the web of the link, around shear tab (similar to the case of test results) at plastic rotation equal to 0.07 rad (Figure 5). It was also shown that the fracture in the long link happened at the shear tab/link web groove weld similar to the location in experimental results. The plastic rotation was equal to 0.02 rad (Figure 6). A comparison between analytical and experimental investigation is shown in figure 3-b.

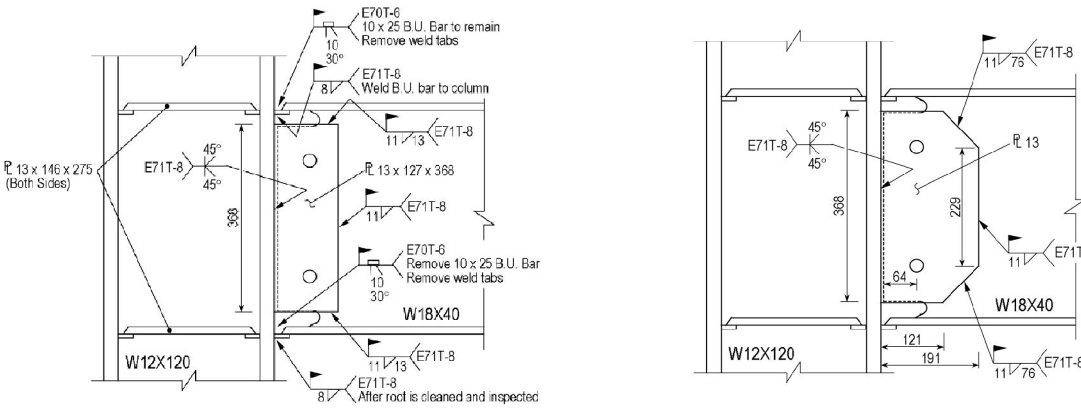

Figure 4. Connection details for short and long link

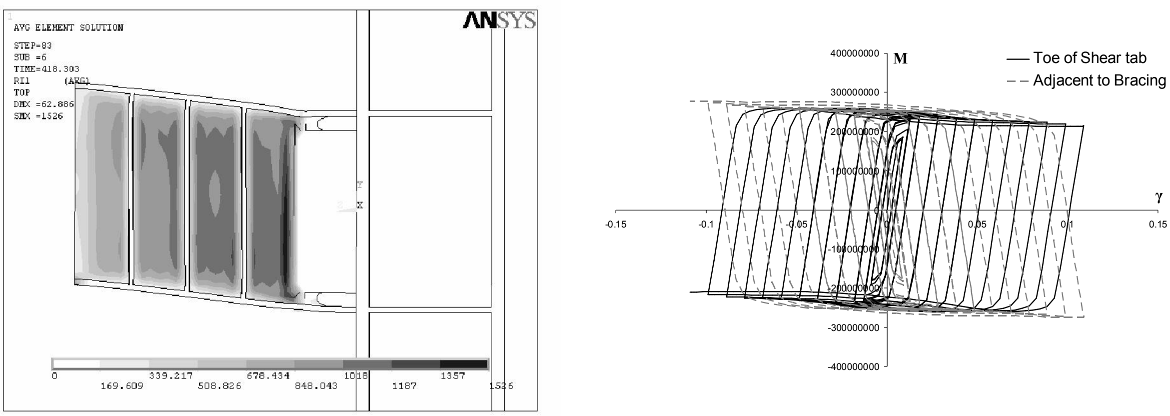

Figure 5. a. Location of fracture in short link (at the web of link, around shear tab) ($\gamma_P = 0.08^{rad}$)
b. Hysteric loop at the end of shear tab and adjacent to bracing connection (short link)

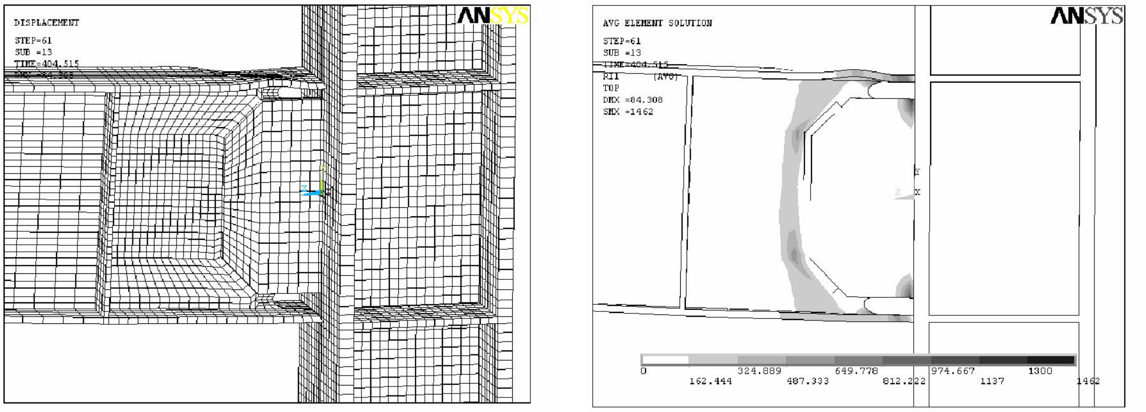

Figure 6. a. Deformed shape of link at connection
b. Prediction of location of fracture in long link with FF connection ($\gamma_P = 0.02^{rad}$)

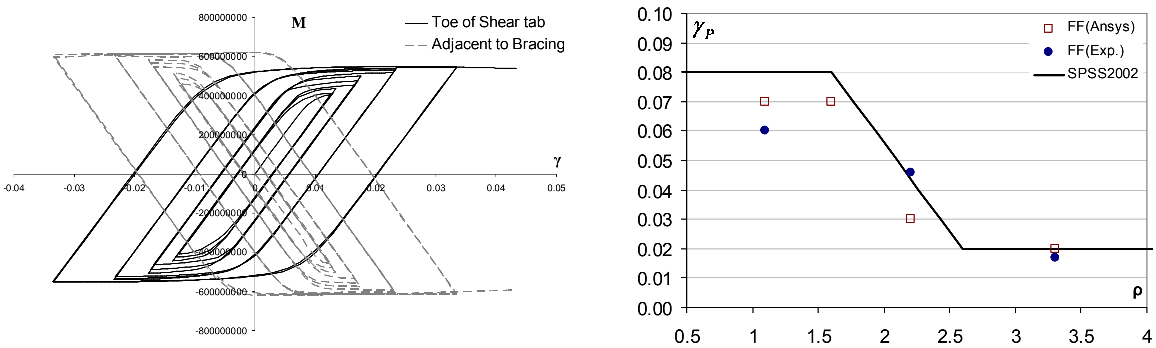

Figure 7. a. Hysteric loop at the end of shear tab and adjacent to bracing connection (long link)
b. Comparison between analytical and experimental results with code provisions

4.3. Connections with reduced beam section (RBS)

The reduced beam section was designed in accordance with AISC358-05. To define the properties of the links (M_P and V_P), the reduced region of beam section is assumed as effective section. So the link length is equal to the distance between the centers of the reduced beam sections. The link with shear behavior (short link) was not investigated.

It is indicated that the failure mode links without lateral bracing was lateral torsion buckling. It is therefore, the links were analyzed with lateral bracing at the end of reduced beam section. Although in lateral braced beams the main form of fracture was the rupture at the end of weld access hole (similar to PN connections), but local buckling in the web of reduced beam section were observed.

The fracture in intermediate link was happened at the end of weld access hole (similar to PN connection) at plastic rotation equal to 0.02 rad, so the ultimate plastic rotation without any degradation in cyclic loop and fracture can be predicted at 0.01 rad. The long link can bear a complete cycle with plastic rotation equal to 0.02 rad without any degradation in strength and fracture.

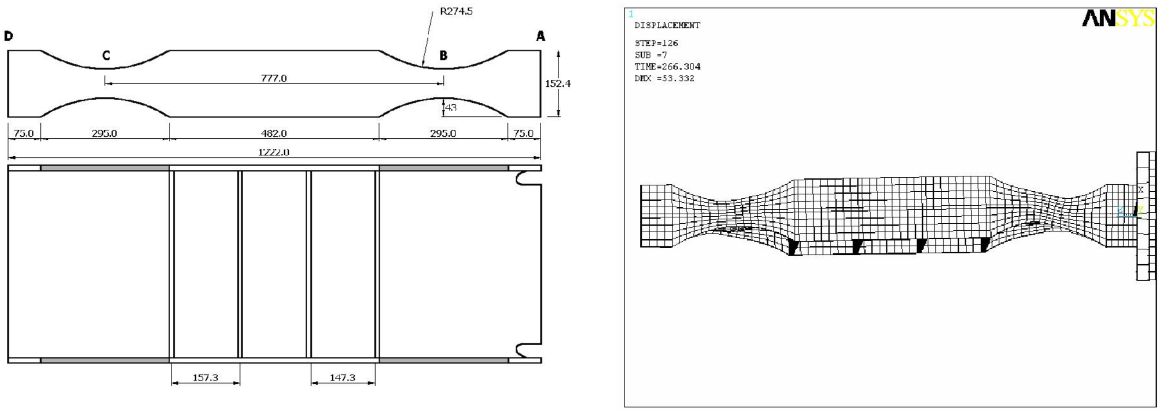

Figure 8. a. reduced section properties for intermediate link
b. Lateral torsion buckling in intermediate beam without lateral bracing

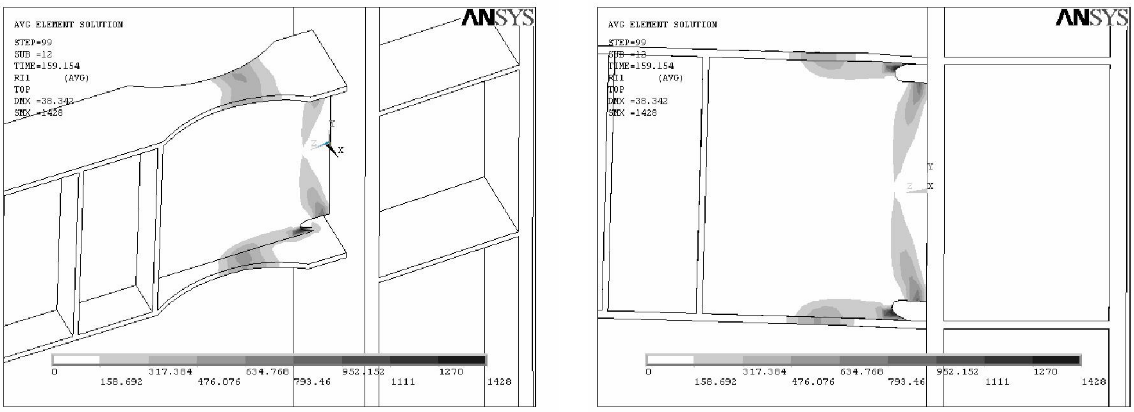

Figure 9. Location of fracture in intermediate link with reduced beam section and lateral bracing ($\gamma_P = 0.02^{rad}$)

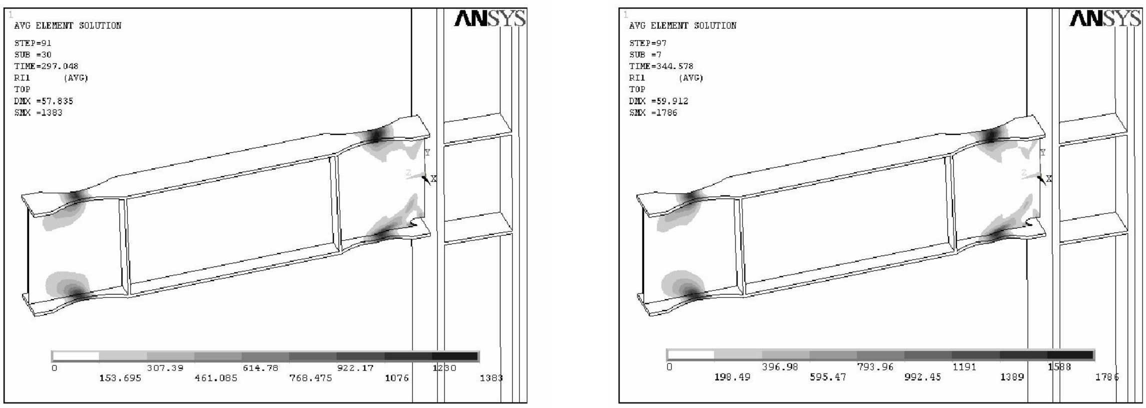

Figure 10. Location of fracture in intermediate link with reduced beam section and lateral bracing
($\gamma_P = 0.03^{rad}$)

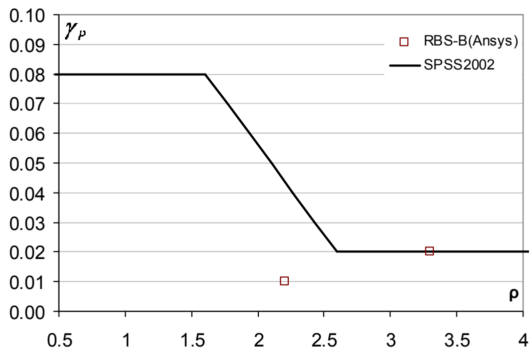

Figure 11. comparison between analytical result and code provisions

4.4. Welded top and bottom haunch connections

These types of connections were designed based on procedure of welded bottom haunch connections which were suggested by Yu and Uang. Their method was based on design of top and bottom haunch (double haunch).

In this type of connections, the failure mode concentrated at the toe of haunch. This sort of failure is the main failure mode that can be happened in the links (if the buckling mode is prevented). Although the short link could not achieve the value suggested in code provision, but in other links (intermediate and long links), the ultimate capacity of the connections is very close to the code provisions value.

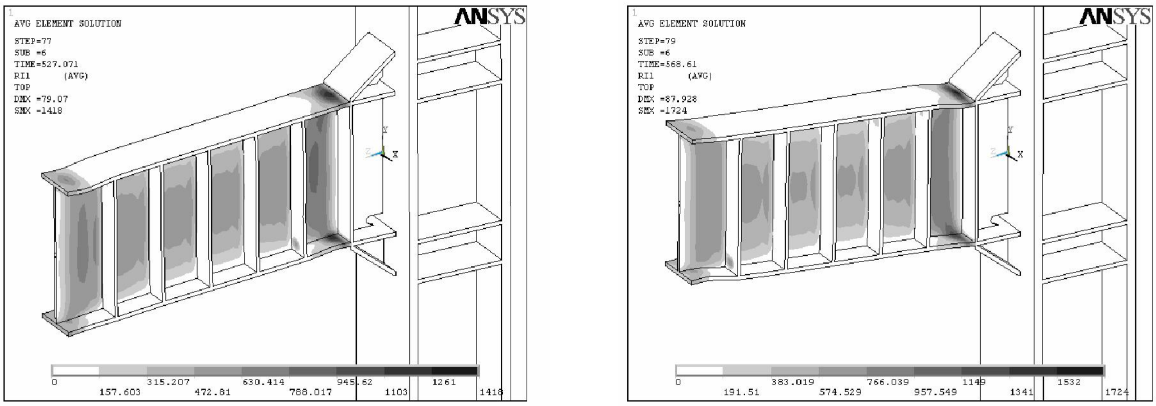

Figure 12. Location of fracture in short link ($e = 1.6 M_P / V_P$) ($\gamma_P = 0.08^{rad}$)

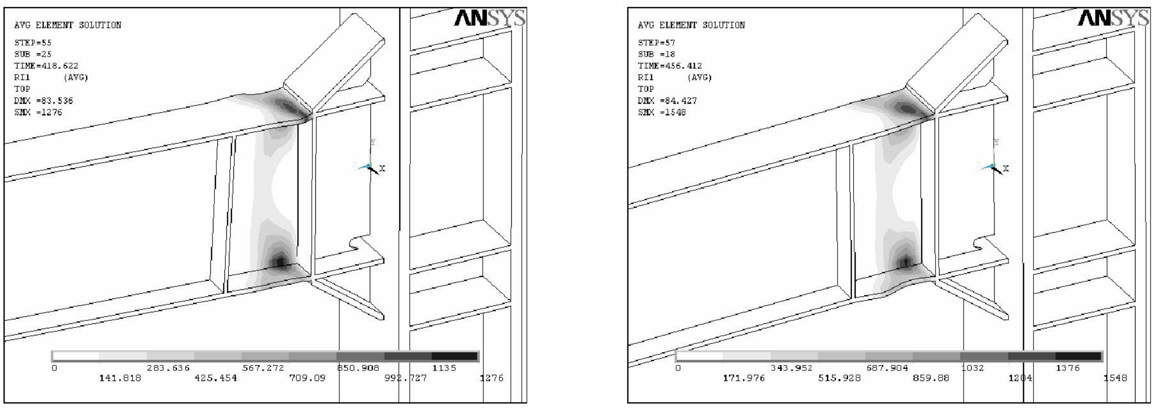

Figure 13. Location of fracture in long link ($\gamma_P = 0.03^{rad}$)

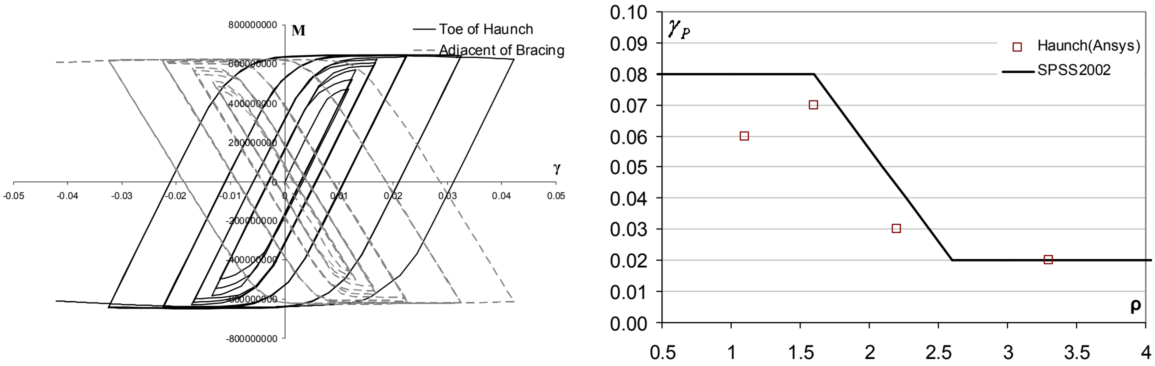

Figure 14. a. Hysteric loop at the toe of haunch and adjacent to bracing connection (long link) b. comparison between analytical results and code provisions

5. CONCLUSION

In this paper, the behavior of four types of link-to-column connection is considered. All connections were modeled by finite element method. These connections details are the same as were tested by an experimental works done by researches. For each of the models, the failure mode (buckling or rupture) was considered and the ultimate capacity was investigated.

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

Using Reduced Beam Section (RBS) in long link changes the location of the failure mode and the plastic mechanism occurred in the reduced section (far from WAH). This behavior was not shown in intermediate link. So RBS connections can be useful in long links.

The link-to-column connections those were stiffened by welded top an bottom haunch were shown that these types of connections can not exactly endure the ultimate capacity of the link plastic rotation (similar to Free Flange connections), but using these types of connections prevent the failure mechanism in welded access hole. So these types of connections can be a suitable choice for design of link-to-column connections.

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