INFLUENCE OF BEAM FLANGE THICKNESS ON SEISMIC PERFORMANCE OF FLANGE PLATE CONNECTIONS

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ABSTRACT

This paper presents numerical studies on the cyclic behavior of flange plate connections. Nonlinear finite-element modeling is carried out in order to evaluate the effect of beam flange thickness on the seismic response of flange plate connections. The connections with thinner beam flange achieved the AISC seismic provision requirements; however the connection with thickest beam flange could not. Based upon the review is conduct in this paper, flange plate connection should not be classified as prequalified, and also it is recommended modifications for the design procedures.

Keywords: Flange plate connection, seismic performance, ductile damage, stiffness, ductility.

1. INTRODUCTION

Steel moment resisting frames were highly regarded for seismic resistant design for many years. This high regard was based on research performed in 1960s and early 1970s, and on the observation that steel frames had performed well in many past earthquakes. Unfortunately, these claims were tarnished after the Northridge earthquake on January 1994 (Roeder and Foutch 1996). After the Northridge earthquake, widespread damages were observed in steel buildings, moreover, the factual nature of these damages were generally attributed to deficiencies in the moment connections (Engelhardt and Sabol 1998; Mao et al. 2001).

In response to these unexpected damages, the SAC Joint Venture, Federal Emergency Management Agency (FEMA) and California Office of Emergency Service investigated the damages to steel moment connections and developed repair techniques and new design approaches to minimize the damages (Kim et al. 2002). Consequently, some new details have been developed, such as strengthening of beam ends by cover plates, flange plates, haunches ribs, etc. to move the plastic hinge region away from the beam end (FEMA 350. 2000).

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One of the fairly practical and inexpensive details has been suggested is flange plate connection (FP). In the FP connection just flange plate is welded to the column flange by a CJP groove weld and beam flange is not connected to the column flange. Extensive studies have been carried out by (Kim et al. 2002) to evaluate the performance of FP connection in seismic zone. As a result, FP connection has been classified as a “pre-qualified welded fully restrained connection” in FEMA 350 since 2000 (Maranian et al. 2003). On the other hand, FEMA 350 has prescribed the following set of requirements about geometry of beam and column in FP connections:

1. Maximum beam depth is limited to almost 95 cm (W36 equivalent).

2. Beam flange thickness less than 2.5 cm is generally accepted for seismic moment frames.

3. Also, just column sections equivalent to W12 and W14 are permitted in FP connections (FEMA 350. 2000).

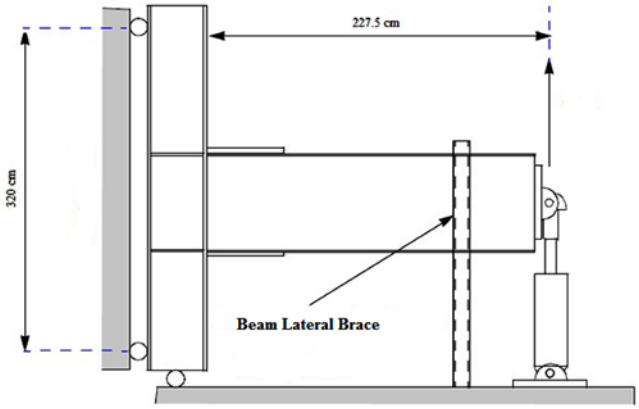
In the present study, four validated finite element models are used to evaluate the effect of beam flange thickness on the seismic performance of FP connection. Moreover, their behavior are compared with criteria has been established by (AISC 341 2005; AISC 360 2005) including strength, stiffness and ductility. In addition, some indices are presented to demonstrate considerable potential for brittle and ductile fracture. These indices were used by (EL-Tawil et al. 1999) for the finite element studies of strength and ductility of fully restrained steel beam column connections.

2. FINITE ELEMENT SIMULATION

2.1. Description of models

Three-dimensional finite element analysis is carried out to represent the actual behavior of FP connections. Entire components in the connections are discretized using 8-node brick incompatible full integration element because of the improved capability of the incompatible mode formulation to model bending of thin plates (ABAQUS. 2008). The groove welds that join the beam to the column, the reinforcing plates to the column, and the continuity plates to the column flanges and the column web are not modeled explicitly. The overall view of the typical test setup is shown in Figure 1.

Fine mesh in the vicinity of the beam-to-column connections is used to analyze the performance of these connections more precisely. On the other hand, a coarser solid-element mesh is applied elsewhere. In addition, a minimum of three elements through the flange plates and bottom plates are considered to capture the through-thickness deformation. Hinged boundary conditions were applied to support the column top and bottom, and also the load is applied by imposing incremental vertical displacements at the beam tip according to (AISC 341 2005) (Figure 2). Furthermore, to prevent the lateral-torsional buckling of the beams, lateral support according to (AISC 360 2005) is considered.



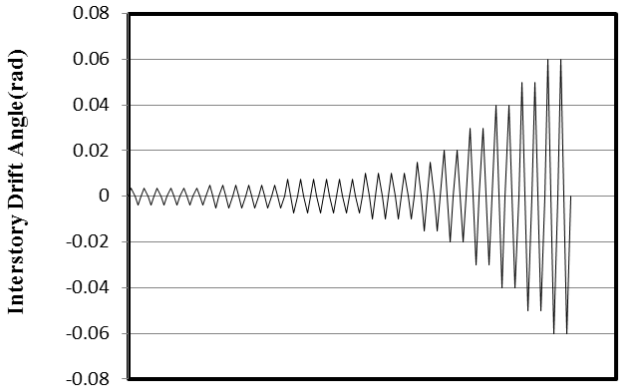


Figure 1: Typical setup of analytical models **Figure 2:AISC 341-05 loading history**

A36 material data is used to establish a bilinear stress-strain relationship for whole parts of connections identified in Table 1 and Figure 3. All models consist of A36 isotropic steel with elastic modulus of $2.1\times10^6 \frac{\text{kg}}{\text{cm}^2}$. The material yield strength is $2500 \frac{\text{kg}}{\text{cm}^2}$ with an ultimate strength of $4000 \frac{\text{kg}}{\text{cm}^2}$ at 20% plastic strain, and likewise the Von Mises yield criterion is employed to account for material nonlinearity.

Table 1: Dimensions of models subassemblies

Model	t_f	t_w	L_t	t_t	L_b	t_b	Strong col/weak beam
BFT0.9	0.9	1.7	50	2.1	50	2.1	5.8
BFT1.2	1.2	1.95	50	2.5	50	2.5	4.6
BFT1.6	1.6	2.45	60	3.4	60	3.4	3.7
BFT2	2	2.8	75	4	75	4.0	2.9

2.2. Validation of Finite Element Models

The experimental data obtained by the Berkeley specimen, PN2, tested during phase I of the SAC Steel Project performed by (Whittaker et al. 1998) is used to validate numerical study contained in this paper. The detail of the experimental specimen is demonstrated in Figure 4, moreover, the displacement history cycles is applied to the beam end corresponds to the incremental displacement is defined by (ATC. 1992).

The cyclic analytical out-come is compared with the cyclic experimental result in Figure5. As shown, the experimental result and finite element out-come are in good agreement. Meanwhile, ultimate load and initial stiffness are reasonably estimated. The subtle differences between finite

element results and experimental results arise from uncertainties in the material model, inevitable residual stress and Bauschinger effect.

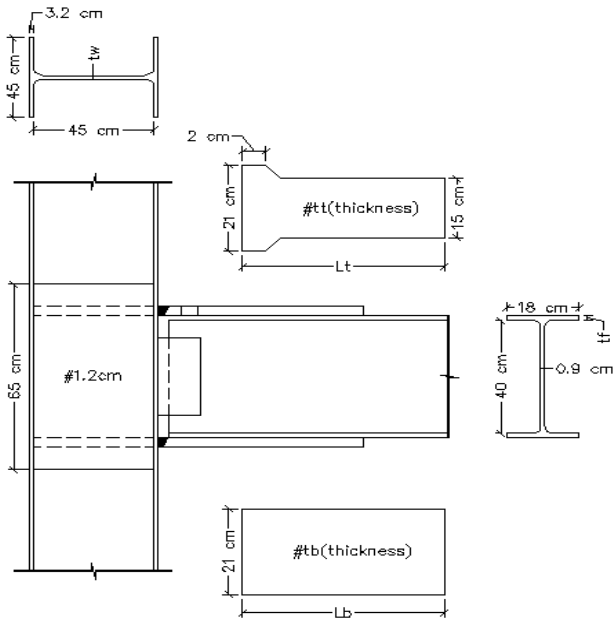


Figure 3: Detail of the models

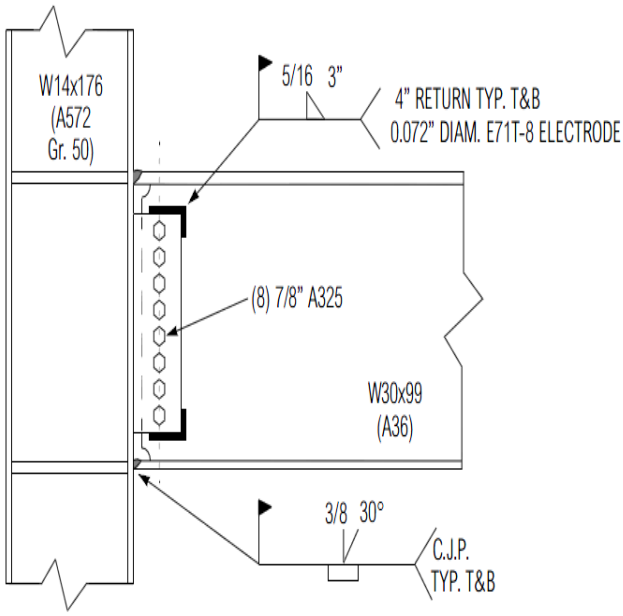


Figure 4: Detail of connection PN2

3. RESPONSE INDICES

In this paper, cracks initiation and crack evolution are not explicitly modeled and therefore some indices are applied to evaluate and compare the behavior of different configurations analyzed in this research. The followings are some of the stress and strain indices employed:

- Triaxiality Index: in the simplest term, triaxiality is the ratio between the hydrostatic stress and the Mises stress ($TI=\frac{\sigma_m}{\sigma_{eff}}$). The values TI between 0.75 and 1.5 can cause large reduction strain of metals and the values greater than 1.5 can make high probability of brittle fracture.
- Rupture Index: this index RI is defined, $RI=\frac{\varepsilon_p/\varepsilon_y}{\exp(-1.5\sigma_m/\sigma_{eff})}$, Where ε_p, ε_y, σ_m and σ_{eff} are the effective plastic strain, yield strain, hydrostatic stress and Von Misses stress, respectively. Rupture Index can be used to compare the fracture potential for two or more configurations (EL-Tawil et al. 1999, Kim et al. 2002).

4. ANALYSIS RESULTS

The moment-rotation characteristics of the models are illustrated in Figure 6. During the analysis, remarkable plastic deformation occurred in beam, while panel zone sustained only elastic rotation.

In models with thicker beam flange, BFT1.6 and BFT2, no deterioration in strength has been observed up to the last cycle, but in models with thinner beam, gradual strength deterioration is

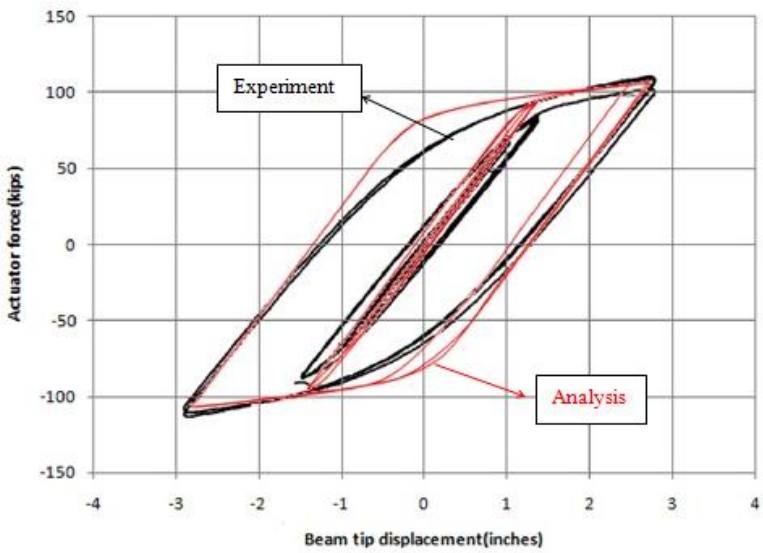


Figure 5: Comparison between experimental and analytical results of PN2 specimen

perceivable that it can be due to strong possibility of local deformation of connections. On the other word, thicker flanges are less susceptible to local buckling than thinner flanges and for this reason the likelihood of strength deterioration increases.

Stiffness, strength and ductility of models have been computed according to (AISC 360 2005) and represented in Figure 8. It can be clearly observed that models BFT0.9, BFT1.2 and BFT1.6 behave as if “fully-restrained connection”, while BFT2 behaves like a “semi-rigid connection”. In addition, the connections with thicker beam flange show more strength and ductility. The connection strength in all models substantially exceeds the fully plastic moment of the beam, therefor the ductility is controlled by the beam and connections can be considered elastic (AISC 360. 2005).

The connection behavior in the inelastic range can be evaluated through the plastic strain, and then plastic equivalent strain, PEEQ in ABAQUS is utilized to predict higher demand for plastic strain. PEEQ contours in the finite element models are illustrated in Figure 7 and high plastic strain and deformation is clearly visible in the beam at the nose of the flange plates in accordance with experimental results performed by FEMA 355D, hence Triaxiality Index, TI, and Rupture Index, RI, are computed at the nose of flange plate in order to compare the possibility of damage. Plot of TI and RI versus beam flange width of the models are shown in Figure 9. When the beam flange thickness is increased, RI and TI increase slightly; also the RI and TI are higher at the K-line than beam flange edges (BFT0.9, BFT1.2 and BFT1.6). On the contrary, an opposite trend is observed in BFT2 and the indices at the edges of beam flange are more critical than center of beam flange width.

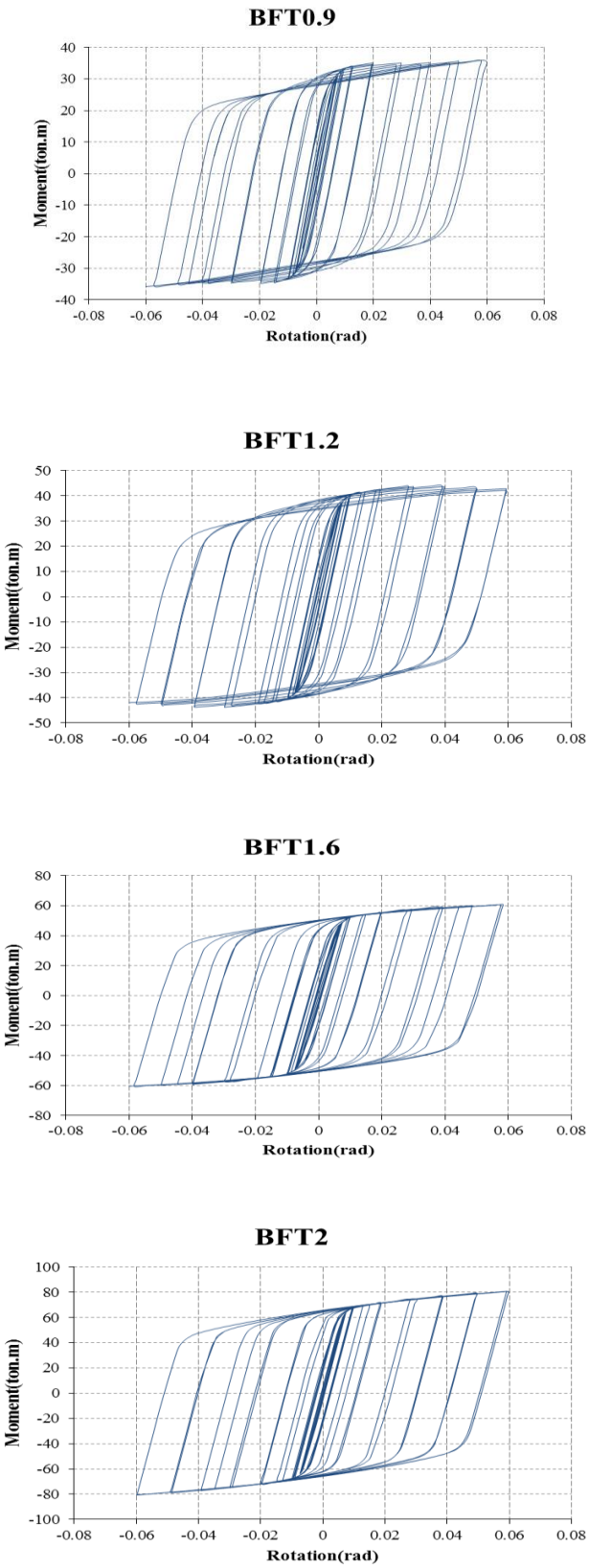


Figure 6: The hystertic curves of models

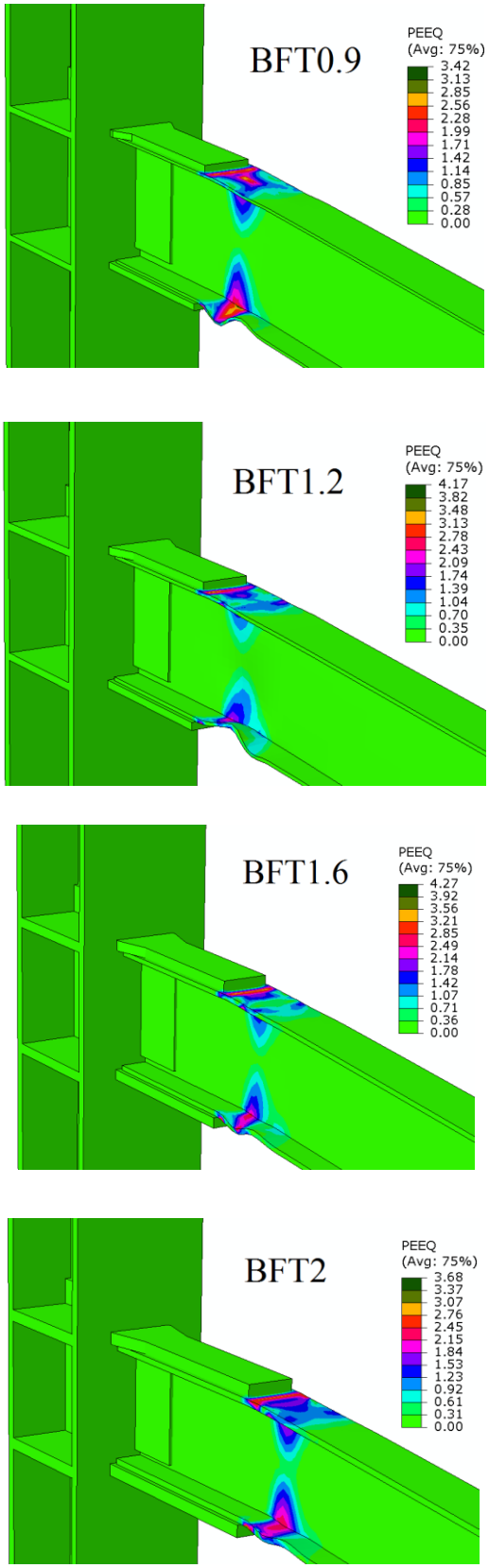


Figure 7: PEEQ distribution

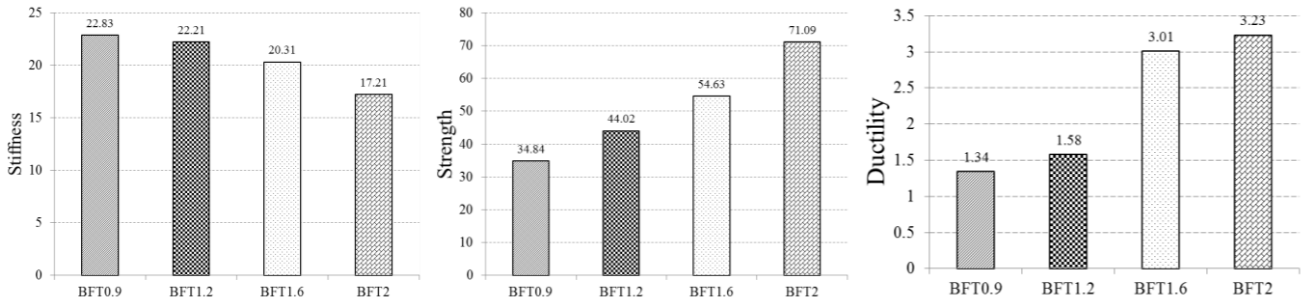


Figure 8: Comparison between stiffness, strength and ductility of the models

By observing the elastic behavior of panel zone in the models during analysis, and also possibility of brittle or ductile damage of thicker beam flanges, the contribution of panel zone in absorbing inelastic deformations should be augmented because a slightly weaker panel zone protects the beam flange from premature fracture.

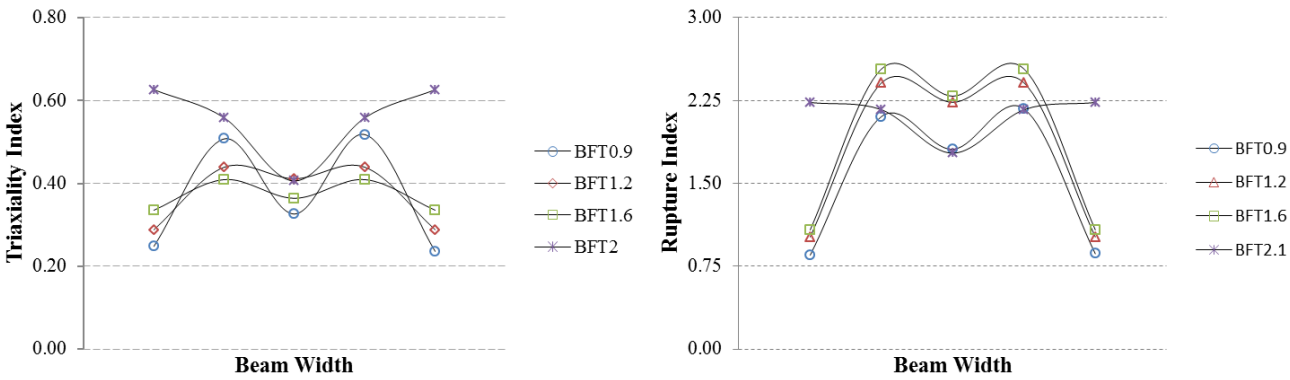


Figure 9: Distribution of Triaxiality and Rupture indices at 4% rad story drift

5. CONCLUSION

Four finite-element models are analyzed to evaluate the effect of beam flange thickness on seismic performance of flange plate connections. All models are absolutely identical in details except beam flange thickness that vary from 9 mm to 20 mm. The models with thinner beam flange satisfy the requirements for special moment frames according to AISC and FEMA. These fallowing conclusions drawn from numerical studies:

1. The connections with thinner beam flange behave like fully-restrained connection, whereas the connection with thicker beam flange act likes a semi-rigid connection.

2. The comprehensive assessments reveal that there is a close correlation between beam flange thickness and strength and ductility of the connections. The connections with thicker beam flange show more strength and rotation capacity.

3. The RI and TI indices are increasing when beam flange became thicker. In the models with thinner beam flange the possibility of ductile damage in the center of beam flange is more than its edge, whereas in the models with thicker beam flange an opposite trend can be observed.

4. In the connections with low ratio of beam flange width to thickness, the design codes should provide some additional requirements to increase the contribution of panel zone in absorbing more inelastic deformations to shield the beam flange against the excessive plastic deformation.

REFERENCES

ABAQUS (2008). User's Manual-version 6.8. Habbit, Karlsson and Sorenson, Inc.

AISC 341 (2005). Seismic Provisions for Structural Steel Buildings No. 341. Chicago, Illinois, American Institude of Steel Construction.

AISC 360 (2005). Specification for Structural Steel Buildings No. 360. Chicago, Illinois, American Institude of Steel Construction.

ATC (1992). Guidelines for Cyclic Seismic Testing of Components of Steel Structures. Redwood City, California, Applied Technology Council.

EL-Tawil, S., E. Vidarsson, T. Mikesell and S. K. Kunnath (1999). "Inelastic Behavior And Design of Stell Panel Zones." Journal of Structural Engineering 125(2): 183-193.

Engelhardt, M. and T. Sabol (1998). "Reinforcing of Steel Moment Connections with Cover Plates: Benefits and Limitations." Journal of Engineering Structures 20(4-6): 510-520.

FEMA 350 (2000). Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings. Washington, D.C, Federal Emergency Managment Agency.

FEMA 355D (2000). State of The Art Report on Connection Performance. Washington, D.C, Federal Emergency Managment Agency.

Kim, T., A. Whittaker, A. Gilani, V. Bertero and S. Takhirov (2002). "Cover-Plate and Flange-Plate Steel Moment-Resisting Connections." Journal of Structural Engineering 128(4): 474-482.

Mao, C., J. Ricles, L. Lu and J. Fisher (2001). "Effect of Local Details on Ductility of Welded Moment Connections." Journal of Structural Engineering 127(9): 1036-1044.

Maranian, P., R. Kern, R. Lyons and G. Brandow (2003). Interim Review on Welded Flange Plate (WFP) Steel Moment Frame Connections. Third Forensic Engineering Congress. San Diego, California, United States, American Society of Civil Engineers: 115-126.

Roeder, W. and A. Foutch (1996). "Experimental Results For Seismic Resistant Steel Moment Frame Connections." Journal of Structural Engineering 122(6): 581-588.

Whittaker, A., A. Gilani and V. Bertero (1998). "Evaluation of Pre-Northridge Steel Moment-Resisting Frame Joints." The Structural Design of Tall Buildings 7(3): 263-283.