Modeling of Hysteretic Inelastic Behavior of Beam to Column Moment Connections under Seismic Loads

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INTRODUCTION

Some of the seismic events that have occurred in areas with significant concentration of steel structures, led to widespread failure of a large number of beam-to-column connections in moment-resisting steel frames in turn, leading to unexpectedly high repair costs [1].

One essential approach of seismic provisions in building codes is to make structures behave in a ductile manner under earthquakes without collapse. In the case of steel moment-frame buildings, the nonlinear behavior of beam-column connections significantly affects the dynamic response under earthquakes since connection regions are one of the primary sources of hysteretic damping.

Insofar as moment resisting steel frames are concerned, extensive investigations carried out in the last three decades have built up a satisfactory knowledge of the behavior of beam-to-column joints under cyclic reversal loading [2, 3, 4].

In recent years, extensive investigations of the simulation of hysteretic responses and the prediction of low-cycle fatigue endurance of moment connections and joints under cyclic loading have been conducted experimentally or numerically [5].

Focusing on moment resisting frames (MRF), experimental cyclic tests have been widely used to investigate the dissipative capacities of traditional bolted or welded moment connections.

Nevertheless, more sophisticated hysteresis models need to be developed and implemented in finite element packages. These hysteresis models should be able to deal with (a) stiffness-and-strength degradation; (b) pinching phenomena; (c) asymmetric hysteresic response; (d) curvilinear hysteretic response especially on the unloading path, where piecewise linear hysteresis models may cause remarkable errors in stiffness or energy dissipation [6].
So that, accurate and reliable characterization of the behavior of various connection types is very important both in the seismic design and life-time safety of steel moment-frame buildings [7].

Modeling of the cyclic behavior of connections is very important in evaluation of seismic performances and design of steel moment-frame buildings. After the 1994 Northridge earthquake, many experiments on steel beam-column connections were carried out to improve the seismic performance of welded beam-column connections and to suggest new connection types with improved seismic resistance [8,9].

This work elucidates experimentally and numerically the hysteretic behavior of steel beam-to-column moment connections under seismic loads in tree column system and evaluating of moment frames with rigid, semi-rigid and twosome connections according to Northridge earthquake. Moreover at last part of article the hysteretic behavior of special steel frames with three different of beam connections (Direct connection, cover plate beam connection and widen beam flange connection) were investigated.

**Cyclic Behavior of Steel Beam-Column Connections:**

Since the 1994 Northridge earthquake, extensive research on seismic response and performance of various connection types has been carried out. The large variations in the load-carrying capacity observed in the experiments are likely due to many different yield mechanisms and failure modes.

As such, large variations in strength and ductility can lead to difficulties in modeling of the cyclic behavior. Particularly, plastic engagement of connecting components significantly affects the cyclic behavior of connections [10, 1].

There are four different approaches to model the cyclic behavior of beam-column connections; a. Phenomenological modeling; b. Mechanical modeling; c. Refined three dimensional finite element modeling and d. Neural network (NN) based modeling approach.

The phenomenological models are mainly based on curve-fitting techniques whereby a simple mathematical expression reproduces the experimental data with some curve-fitting constants. The constants are calibrated by the experimental data.

The advantage of the phenomenological model is that once the constants are determined, the moment-rotation relationship can be explicitly expressed and used in ordinary structural analysis for design purposes [11].

The mechanical models are also aimed at predicting the rotational cyclic behavior of connections by assemblages of rigid and deformable elements (spring elements). They are frequently referred to as component-based models in the literature. The advantage of the component-based modeling is that the cyclic behavior of the whole connections can be represented by the uni-axial cyclic behavior of simple deformable elements.

Three-dimensional finite element model is the most accurate approach to predict the cyclic response of beam-column connections [7].

For detailed modeling of components of the connection, modeling techniques such as metal frictional contact, assembly torque, geometric and material nonlinearity are easily employed in complex three-dimensional finite element models.

Since neural network (NN) based material modeling methodology was first proposed by Ghaboussi et al. The main advantages of the NN based material models are that 1) the NN based material models can represent the material behavior properly if they are trained with comprehensive training data and 2) they can represent any complex cyclic material behavior including the post-limit behavior such as local buckling, fracture and tearing of components. The training process of the NN material model is similar to validation with comprehensive experimental observations in conventional phenomenological material models [3, 12].

The hysteretic connection element may be either a translational or rotational spring connecting two nodes with identical coordinates. In view of its direct application to the simulation of moment-rotation responses of steel connections, hereinafter the constitutive relationship of the connection element is presented as a moment-rotation relationship [13].

The cyclic behavior of column-tree moment connection with short, non-prismatic beam and column with H-shaped section:

In this part, the cyclic behavior of column-tree moment with short, non-prismatic beam, with and without using web stiffener at the site of connecting short beam to main beam and column with H cross section (CT-H Model and CT-SH Model) was examined.

AISC 2005 procedure is used to evaluate the ductility and resistance of the connection and finite element model of samples were modeled using ANSYS software. Beam and column profile specifications are given in Table 1.

**Table 1: Profile characteristics of connection.**

<table>
<thead>
<tr>
<th>Length (mm)</th>
<th>Profile</th>
<th>Element</th>
</tr>
</thead>
<tbody>
<tr>
<td>3000</td>
<td>H 550x550x40x 45</td>
<td>Column</td>
</tr>
<tr>
<td>2030</td>
<td>H 588x 300x12x 20</td>
<td>Main beam</td>
</tr>
</tbody>
</table>
Steel material is ASTM A572 Gr. 50 and two-linear stress-strain model is used to model its behavior. Steel elasticity coefficient is 200 GPa and yield stress equals to 350 MPa. After the yield point, the curve with slope of second line that equals to 0.04 E, will continue its upward trend.

Sub assemblage finite element model is modeled in ANSYS software. Boundary conditions are considered so that the bottom support of column is roller and the And column up support Act as joint connection. Loading has been done on all nodes of main beam end and in the vertical direction.

Cyclic loading is applied to study the connection behavior. The applied loading is standard loading of SAC 97 that is proposed in Seismic Procedures of AISC 2005 and FEMA 350. This loading is shown in Figure 1.

![Fig. 1: Standard load according to SAC 97.](image1)

Hysteretic curves obtained from cyclic loading are plotted for all models that two of them are shown in Figures 2 and 3.

![Fig. 2: Moment-Drift angel Hysteretic Curve (CT-H Model).](image2)
In hysteretic curves of moment-drift angle, the intent from moment is moment in terms of column and the intent from drift is short end beam displacement ratio to short length beam. The drawn horizontal line is $M_p$ line that is drawn to investigate the moment frame conditions with special ductility according to terms of AISC 2005 regulations and evaluate connection resistance.

In AISC 2005 Seismic codes it is mentioned that in moment frames with special ductility, the moment amount in terms of column in 4% drift angle must not be less than 80% of the beam plastic moment. As can be seen in the models, the moment amount in terms of column in rotation angle of 4% radians is more than 0.8$M_p$. Also, in CT-S-H model no buckling has occurred before reaching radians rotation angle. In case the model CT-H / drift angle (rotation) were buckling with 0.03 radians. It can be concluded that CT-SH model has particular applicability in frames. While, the CT-H model in drift angle (rotation) of 0.04 radians was buckled.

Then we can conclude that CT-S-H model has the capability to be used in specific moment frames. While in CT-H model moment amount in terms of rotation angle of 0.04 radians is 0.8$M_p$. But given that this model was buckled in drift angle (rotation) of 0.03 radians, according to AISC Regulations it has no usability in special moment frames, because one of the terms of especial buckling connections is that studied will not be declined before reaching the rotation angle of 0.04 radians.

According to AISC 2005 Seismic Regulations Terms on intermediate moment frames, connection will not be declined at least until the rotation angle of 0.02 radians. Therefore, we can say that CT-H model has the usability in the middle moment frames.

Moment curves are plotted in Figures 4 and 5 in terms of column relative to rotation angle of panel zone for models in order to investigate the rotation of panel zone. Rotation angle of the panel zone is obtained from the following equation

$$\gamma_{pz} = \frac{\sqrt{(a^2 + b^2)}}{2ab}(\delta_1 - \delta_2)$$

In the above equation, $a$ and $b$ are the basic dimensions of panel zone and $\delta_1$ and $\delta_2$ are changes in the diameter of panel zone.

![Fig. 3: Moment-Drift angle Hysteretic Curve (CT-S-H Model).](image)
As can be seen in the above figure, panel zone torsion is negligible. Because short non-prismatic beam (muscle) in both above connections causes that the place of energy dissipation and connection deformation takes place outside column and within beam and rotational function is completely independent of the partnership of panel zone. Therefore, there is no concern on rigid connections resulted from sudden tearing in the connection place of column web and flange.

**Effect of connections on hysteretic behavior of short Moment Frames:**

In this section, the effect of connections on short moment frame for different forms of connections including rigid, semi-rigid and co-connections of rigid and semi-rigid (dual) has been studied comparing the hysteretic behavior of these frames.

For this purpose, the seismic behavior of three-story frames has been evaluated by non-linear dynamic analysis representing short frames with multiple connections for documented record of the devastating Northridge earthquake, and effects of existing connections in the frame have been evaluated through observing the hysteresis behavior of each story and the total behavior of the entire frame.

Profile specifications are presented in Figure 6 and the samples of studied frame are presented in Figure 7. Two percent rigidity of 0.5 and 0.7 is considered for connections and plastic moment resistance of 0.8 and 1.2 for beam strength is considered.

The beams and columns are, respectively, from steels with 2500 and 3450 kilograms per square centimeter yield limit and the extinction coefficient of frames is considered 5%. Frame response with rigid, semi-rigid dual connections in addition to frames with completely rigid and semi-rigid connections have been analyzed for NORTHRIEGE earthquake by non-linear dynamic analysis and the results of the frames analysis are compared with the rigid mode.
Fig. 7: Studied sample frames.

The below curves show the possibility of comparing the hysteretic curves of rigid, semi-rigid frames with moment rigidity of 0.5 and moment strength of 0.8 for No.5 frame in Northridge earthquake. In below graphs, A represents rigid frame behavior and B represents semi-rigid frame behavior, these curves are selected among different behaviors of frames and to show the importance of examining the hysteretic behavior of frames.

Fig. 8: Hysteretic behavior of first story.
Fig. 9: Hysteretic behavior of second story.
Fig. 10: Hysteretic behavior of studied frame.

As it was shown, comparing hysteresis curves of two frames, number 5 semi-rigid frame is entered plastic area under much smaller shear force than the rigid frame and this issue has influenced structural behavior during later cycles in a way that frame has experienced very different paste modes compared with rigid frame during shake.

This mode has been considered in the final curve of entire frame and relative instability of frame can be stated for this state of rigidity in quite semi-rigid frame 5. Examining the hysteresis cycle of other frames it was observed that this event can be happened for number 4 frame with similar connection properties (moment rigidity of 0.5 and moment strength of 0.8), but other frames have shown good behavior.

Evaluating seismic behavior of beam I -to-box column connections in special moment frames:

In this section, hysteresis behavior of beam I connection to box column is studied. This connection consists of the following three categories: connection with cover plates, direct connection with constant cross section and direct connection with Increment cross section. For this purpose, a 12-story building was considered that one of the side axes was evaluated with respect to the connections’ cross sections. The construct plan is $15 \times 25$ mm, also sheet beams were chosen from box sections and in two directions.

Cover-plate beam, welded connection:

Details of connection with headgear and headrest sheets are shown in Fig 11.

Fig. 11: Schematic representation of the bottom and top beam plate.

In this connection, headgear and headrest plates do not have any contact with column and force transmission is done only through cover plates. In Figure 12, the hysteretic behavior of this connection is shown.
Fig. 12: Hysteretic behavior of cover-plate connection.

*Direct beam connection:*
In this type of connection, beam web and flange are connected directly to the column. Hysteresis behavior of this type of connection is shown in Figure 13.

Fig. 13: Hysteretic behavior of direct connection.

*Increment beam section:*
In this type of connection, with increased width of flange of Plastic hinge location is directed far from connection. Beam flange width increase details is according to AISC2009. The main difference between this type of connection and direct connection is in the absence of using shear plate. Figure 14 shows the hysteretic behavior of this type of connection.

Considering hysteretic curves provided it can be said that the cyclic behavior of these connections is stable and reliable. According to the results, in above connections no resistance loss will be observed until the relative displacement angle of story equals to 0.04 radians.

To connect with a cover plate for displacement with angles of 0.06 radians to 0.2 radians, it has endured the resistance loss of 252 kN. In direct connection and connection with Increment cross section, with increased freight and bearing force of 215 kN, resistance loss equals to 0.15. So all three connections meet AISC1997 regulations for special moment connection in which the minimum 0.03 radians inelastic rotation and the maximum drop of 20 per cent moment strength loss are required.
As a result, these three connections can be used at the moment frame connections. Comparing energy absorption in three connections shows that the coated sheet shows better performance in energy absorption.

**Fig. 14:** Hysteretic behavior of Increment beam section.

**Conclusion:**

Rotational stiffness and connection strength play main role in moment frames behavior in severe earthquakes and effect structural performance by connection features. Incorrect use of these connections to reduce the structure base shear can damage the structural performance and this is easily recognizable by observing the hysteretic behavior of frame.

According to conducted studies, column-tree connections with short, non-prismatic beam with stiffening, and H section column provide AISC 2005 procedure requirements on ductility and moment strength for special moment frames. Using stiffening in the tip of muscles is effective in improving the cyclic behavior of column-tree connections with short, non-prismatic beam.

Procedures do not offer using semi-rigid connection with moment strength less than connection strength, and one reason is unexpected behaviors of the structures that are considered in this paper. Correct combination of rigid and semi-rigid connections can adequately improve structural performance and improve flaws of rigid connection application in structure.

Also, according to the conducted studies on three connections with cover plate, direct connection and connection with increment cross section, it was determined that ductility of connection with cover plate is more than two other connections. According to the results, connection with cover plate as well as two other connections resists 6 percent displacement with drop below 20 percent that can be used in special moment frames according to the procedures. Also connection with cover plate under cyclic loading compared with two other connections will absorb 10% more energy.

**REFERENCES**


[12] Powell, GH., S. Campbell, 1994. DRAIN-3DX element description and user guide for element type 01, type 04, type 05, type 08, type 09, type 15, type 17, and version 1.10. Report No UCB/SEMM-94/08, Department of Civil Engineering, University of California, Berkeley.
