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A DETAILED 2D FINITE ELEMENT MODEL FOR THE SEISMIC ASSESSMENT OF STEEL FRAMES WITH TOP-AND-SEAT ANGLE WITH DOUBLE WEB-ANGLE CONNECTIONS

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Abstract. *Early work on predicting the moment-rotation response of semi-rigid connections was conducted through simplified linearized and curve fitting models compared to experimental data. The models were developed using monotonic tests with ambiguity in the loading and boundary conditions used. As a result of such, the models often failed to predict the actual response of the connection particularly when the connection is cyclically loaded. With advancements in computational techniques and power, attention was shifted to developing 3D models which have proven to be capable of capturing the true behavior of the connection. Notwithstanding their effectiveness, 3D models are hard to construct and are computationally expensive, thus their ability to conduct large parametric studies is limited. This paper presents an overview of a new 2D inelastic finite element model (FEM) for predicting the monotonic and cyclic response of semi-rigid connections with top-and-seat angle with double web angle. The model comprises 2D plane strain elements and includes various behavioral features including bolt preload, friction between faying surfaces, connection slip, the effect of bolt-hole ovalaization and hot-rolling residual stresses in the angles. Characterized by its reduced number of nodes and elements, the model is computationally inexpensive and is capable of capturing the true behavior of the connection.*

1 INTRODUCTION

The integrity of fully-welded connections in the beam-to-column joints of steel frames under earthquake loading has come under question as many steel and composite buildings suffered severe damage in connections during the Northridge (1994) and Hyogo-ken Nanbu (1995) earthquakes. Forensic investigations following the earthquake identified failed welded beam–column connections in more than 200 buildings [1]. The failure was attributed to poor connection detailing practices and inadequate weld material properties that were common prior to the earthquake [2]. The backing bars and weld runoff tabs used to make the groove welds connecting the beams to the columns were normally left in place after completion of the weld. The existence of backing bars or welds tabs created a lack-of-fusion defect, which was large enough to originate crack growth which propagated in the heat affected zone of the weld metal and in the column flange. Studies on the weld metal used in construction, E70T-4, revealed that the weld had very low Charpy toughness [3]. Subsequently, there has been a tendency to be overly conservative in the design and detailing of these connections. Figure 1 shows a typical fractured beam-column connection in a steel moment frame during the 1994 Northridge Earthquake.

Figure 1: Fractured connection in a steel moment frame during the 1994 Northridge earthquake (crack propagated at backing bar detail and propagated in the column flange and web)

Uncertainly in the performance of welded connections spiked interest in investigating the use of bolted connections in the construction of steel frames in seismic regions. Bolted partial-strength semi-rigid connections were evaluated as a viable alternative due to their lower construction costs and simple fabrication process. Analytical and experimental studies were carried out to assess the fundamental characteristics of the connection. The cyclic behavior of the connection was evaluated through testing of beam-column subassemblies. The experimental results demonstrated the large energy absorption capabilities of these connections under cyclic loading with stable hysteretic behavior [4, 5].

The experimental results were used to develop simplified analytical and power models aimed at capturing the connection response [6, 7, 8]. The models were however developed using old tests in which high degree of uncertainty exist regarding the degree of bolt pretension and actual material properties. In addition, the tests did not cover a wide range of specimen sizes and focused on small specimens with shallow beams and thin angles. As a consequence, test data for specimens comprising deep beams and thick angles did not show good agreement with the curve-fitting models [9].

Two and Three-dimensional finite element models were developed to capture the complicated behavior of the connection such as slip, friction between surfaces in contact and prying action [10, 11, 12, 13, 14]. The component level experimental and finite element results were used for developing frame models for assessing the seismic performance of steel frames. In these models, line elements are used to represent the beams and columns and rotational springs with idealized moment-rotation relationships, obtained from the experimental results or the finite element models, are used to represent the connections [15, 16, 17]. The drawback of using such approach, in addition to idealizing the moment-rotation relationships, is that the interaction between the beam and column flanges and the angles comprising the connection is not captured. Such interaction is essential as it influences the spread of yielding in the beam. This paper presents an overview of new 2D finite element model of top-and-seat angle with double web angle connections and its use is frame analysis for the seismic evaluation of steel frames with such connections.

2 DESCRIPTION OF THE STRUCTURE

The structure considered is a 2-story, 4-bay (longitudinal) and 2-bay (transverse) steel frame. The height of the first and second story is 15 ft and 13.5 ft, respectively and the bay width is 30 ft. The outer frames are special moment resisting frames (SMRF) designed according to the Structural Seismic Design Manual, Volume 3 [18] while the inner frames are only responsible for carrying their share of gravity load.

The strong-column weak-beam design approach was used for sizing the beams and columns in the SMRFs and resulted in W18 x 40 for the beams and W14 x 159 for the columns. Following the sizing of the beams and columns, the assumed rigid connections in the frame were redesigned to reflect semi rigidity and partial strength. Three different frames are considered with the connections in each frame designed as top-and-seat angles with double web angles according to Eurocode 3 [19]. The sizes of the angles and the bolts were optimized such that the resulting connection capacity in frame 1, 2 and 3 is 30%, 50%, and 70%, respectively, of the plastic moment capacity of the beam. Plan view of the structure, an elevation of a typical SMRF, and a zoom-in of a typical connection topology are shown in Figure 2.

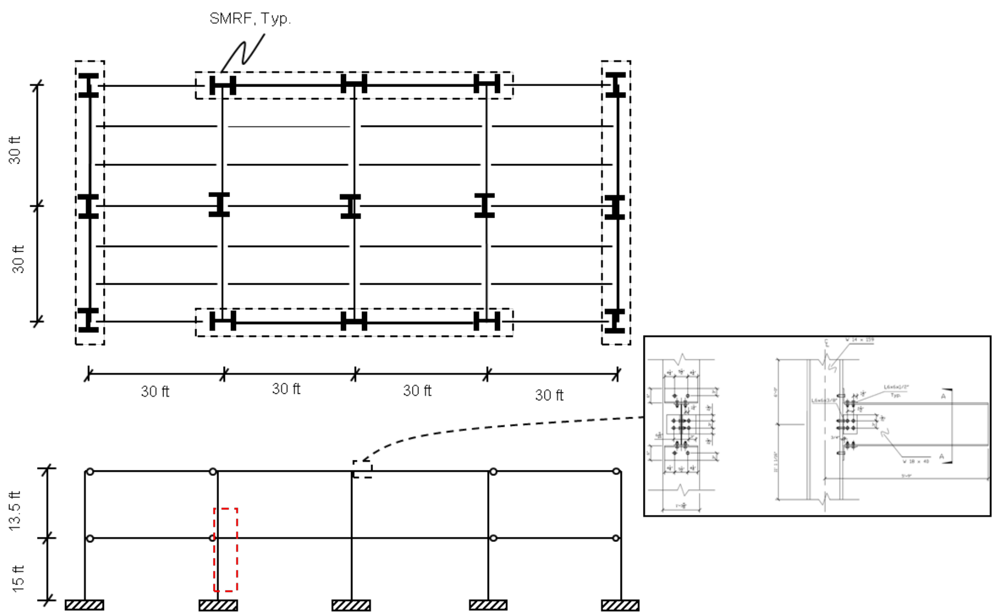

Figure 2: Plan view of the structure, an elevation of the SMRF and a zoom-in of a typical connection topology

3 ANALYTICAL MODEL

3.1 Beam-column connection model

The development of 3D finite element models for connections has been primarily driven by the notion that 2D models cannot properly capture the complex localized behavior of the connection. On the opposing side of the latter argument, 2D models may be much more efficient when compared to 3D models if the problem investigated is planar or could be idealized as planar. In addition, 3D models contain large number of nodes and elements rendering them computationally expensive and rather impractical particularly when used in parametric studies.

An inelastic finite element model is employed in the current investigation. The model comprises a 2D plane stress elements for the beam-to-column connections and 1D beam elements between subsequent connections and is developed using ABAQUS which is a general purpose commercial package [20]. The model, shown in Figure 3, includes various behavioral features namely; 1) bolt preload, 2) friction between faying surfaces, 3) connection slip, 4) the effect of bolt-hole ovalization, and 5) hot-rolling residual stresses in the angles. Figure 4 (a) shows the monotonic moment-rotation relationship (with and without slip) of a beam-column subassembly with a 30% MP_b connection capacity. Details on the model and its features are given in Mahmoud [21].

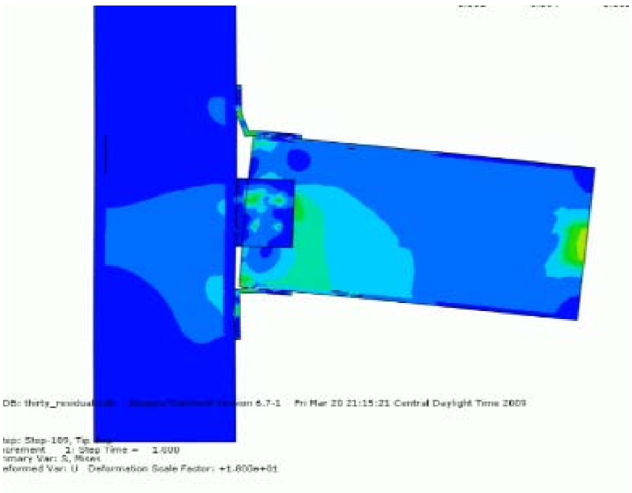

Figure 3: Finite element model of the beam-column subassembly.

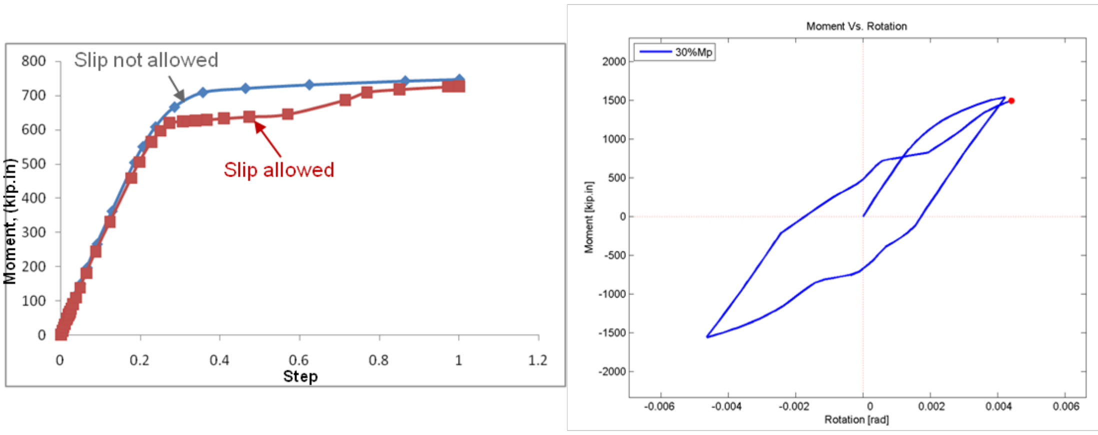

Figure 4: Moment-rotation relationship for a 30% capacity connection with and without slip.

3.2 Frame model

Analytical models of frames have utilized line elements connected with springs representing the load deformation characteristics of the connection. Due to its minimal computational demands, this modeling approach has been viewed as the best alternative for conducting frame analysis. However, the models typically represent idealized behavior and in many cases cannot capture the local response of the various connection components. Furthermore, the deformation and the spread of yielding in the beams or columns are not well represented since the interaction between the beam flanges and the top and seat angles is neither physically modeled nor accounted for. Therefore, the use of 2D or 3D finite element models when conducting time-history analysis can pay significant dividends since the localized connection behavior and its interaction with the beam and column is physically modeled.

In light of the above arguments, a multi-resolution inelastic 2D finite element model is employed for conducting time-history analysis of the steel frames with top-and-seat angles with double web angle connection as shown in Figure 5. The connections in the frame are represented through the detailed 2D finite element formulation of the beam-column subassembly described above while the beams and columns in the frame are represented using line elements.

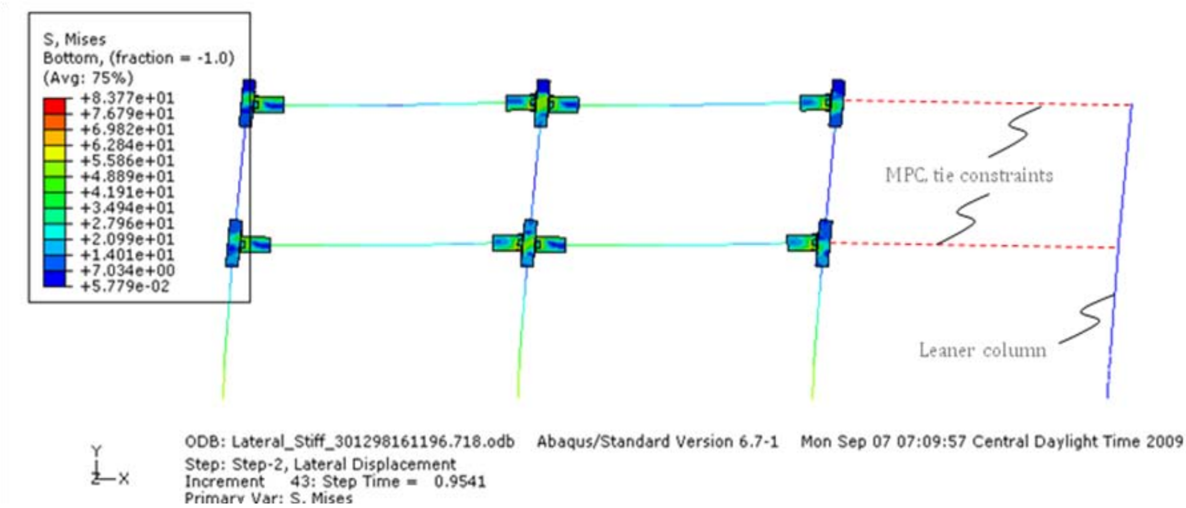

Figure 5: Frame model including a leaner column representing the gravity system.

Prior to starting the simulations, gravity loads were applied to the system using the loading combination of:

$$1.0DL + 10\,psf + 0.25LL \qquad (2)$$

Where the DL indicates dead load, the 10 pfs is used for partition walls, and LL indicates live load. The resulting distributed load is listed in Table 2. Gravity loads carried by the core gravity frames were considered through the application of point loads on a leaning column modeled as a pinned rigid element and connected to the frame through tie multi point constraints.

Level	1.0 DL + Partitions (kip/in)	0.25 LL (kip/in)
Roof	0.0863	0.0013
1st	0.095	0.05

Table 1: Distributed gravity loads applied to the frames during the simulations.

4 PRELINMINARY RESULTS

4.1 Selected record and scaling

Non-linear dynamic time-history analysis was conducted to evaluate the seismic response of the frame. The structure was excited using the Northridge earthquake of 17 January 1994 with M_w of 6.69. The station used is the Arleta - Nordhoff Fire Station which is located 8.66 km away from the epicenter and measured a PGA of 0.332. To speed up the analysis, the original record was truncated where only the first 20 seconds of the record were used in the analysis since the main earthquake activities are concentrated in the first 20 seconds. In addition, to truncating the record, a scaling factor of 1.989 was applied to the record to represent a maximum considered earthquake (MCE) as per ASCE 07 [22]. The original and modified records are shown in Figure 6 (a) and (b) respectively.

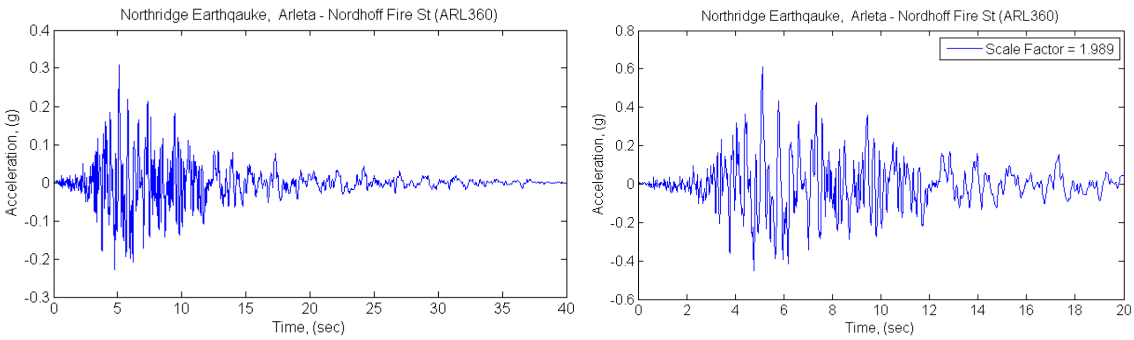

Figure 6: The Northridge earthquake used in the simulation. (a) original record. (b) modified record.

4.2 Roof displacement and Interstory drift ratios

The analysis was conducted on frames with varying connection capacity, yield strength of the connecting angles, slip in the connection and friction coefficient between the faying surfaces. Specifically, the frames were characterized by connection capacities of 30% MP_b, 50% MP_b and 70% MP_b with yield strength of 36 ksi and 50 ksi for the angles and beam elements, respectively, a friction coefficient of 0.33 and 1/16” bolt hole (i.e., a slip 1/32” on both sides of the bolt hole). The roof displacement and the interstory drift ratio of all three frames are shown in Figure 7. The maximum roof displacement experienced by the frames was 4.06 in, 5.82 in and 6.54 in, for the 30% MP_b, 30% MP_b and 30% MP_b, respectively. The resulting first floor interstory drift ratio was calculated to be 2.51%, 3.59% and 4.04% for the 30% MP_b, 30% MP_b and 30% MP_b, respectively. The roof interstory drift ratio was calculated to be 1.53%, 2.78% and 3.15% for the 30% MP_b, 30% MP_b and 30% MP_b, respectively. The calculated interstory drift ratios for all frames are below the 5% limit specified by FEMA 356 [23]

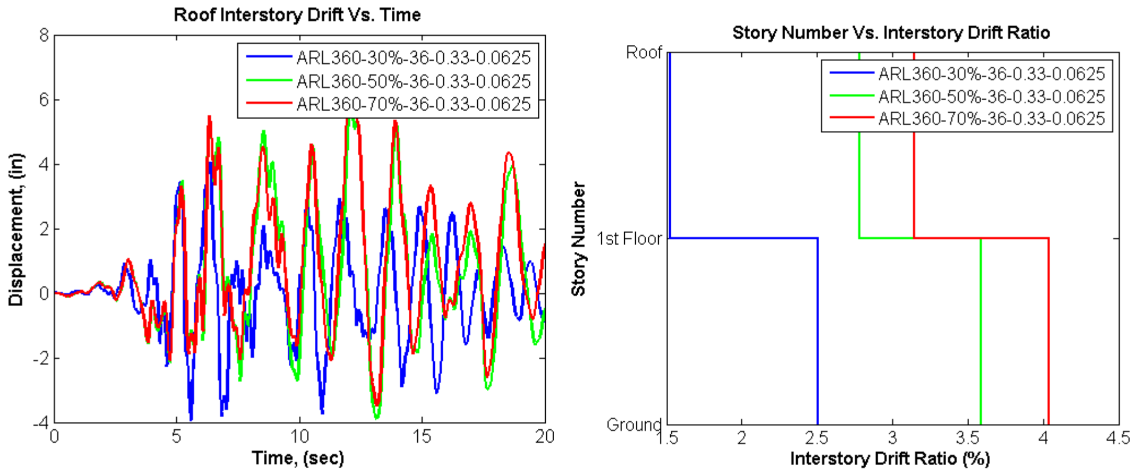

Figure 7: Roof displacement and intersotry drift ratios for all three frames

5 CONCLUSIONS

- A new beam-column connection model for top-and-seat angle with double web angle connections has been developed.
- Many inelastic features are represented in the model including hot-rolling residual stresses in the angles, bolt preload, friction between faying surfaces, connection slip and the effect of bolt-hole ovalization,
- The beam-column model was integrated in a frame model
- Non-linear dynamic time-history analysis was conducted on three different frames with varying connection capacities (30% MP_b, 50% MP_b and 70% Mp_b)
- The analysis results showed that the 30% MPb frame experienced the least roof displacement and interstory drift ratio followed by the 50% MPb frame and the 70% MPb frame respectively.
- The interstory dirft ratio for all three frames did not exceed the 5% limit defined by FEMA for MCE.

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