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PANEL ZONE IN THE FLANGE PLATE CONNECTION TO BOX COLUMN

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ABSTRACT

This paper presents analytical and experimental studies on the cyclic behavior of flange plate connection between a steel beam and a welded box column. A full-scale, single-sided specimen with flange plate connection was tested using a standard connection prequalification test protocol. The flange plate connection in the test specimen achieved the AISC seismic provision requirements for special moment frames. The finite element model developed using ABAQUS was validated using the test results. This model was subsequently used to further investigate the behavior of the test specimen and to evaluate the effect of panel zone strength on the response of flange plate connections.

Keywords: Connections; Flange Plate; Box Columns; Experimental program; Finite element

1. INTRODUCTION

Box columns are frequently employed in areas of high seismic risk because they have an excellent capacity to resist biaxial bending. Cold-formed hollow sections are often used for low and medium rise buildings and built-up sections made up of four plates welded together are used for high rise buildings (Nakashima et al. 2000).

Extensive studies have been carried out and several new connection details have been proposed for the connection of I-beams to wide flange columns since the 1994 Northridge earthquake (Shiravand et al. 2010), but limited research for the connection of I-beams to box–columns has been conducted (Chen et al. 2004; Kim et al. 2003).

In the present study, the effect of panel zone strength on the behavior of a welded flange plate connection, shown in Figure 1, has been investigated. This type of connection is mainly fabricated on site. The geometry of flange plates is considered in a manner that site welding in a horizontal position is possible for connecting flange plates to beam and column. (Gholami et al. 2012) investigated the effect of connection details on the response of welded flange plate connection.

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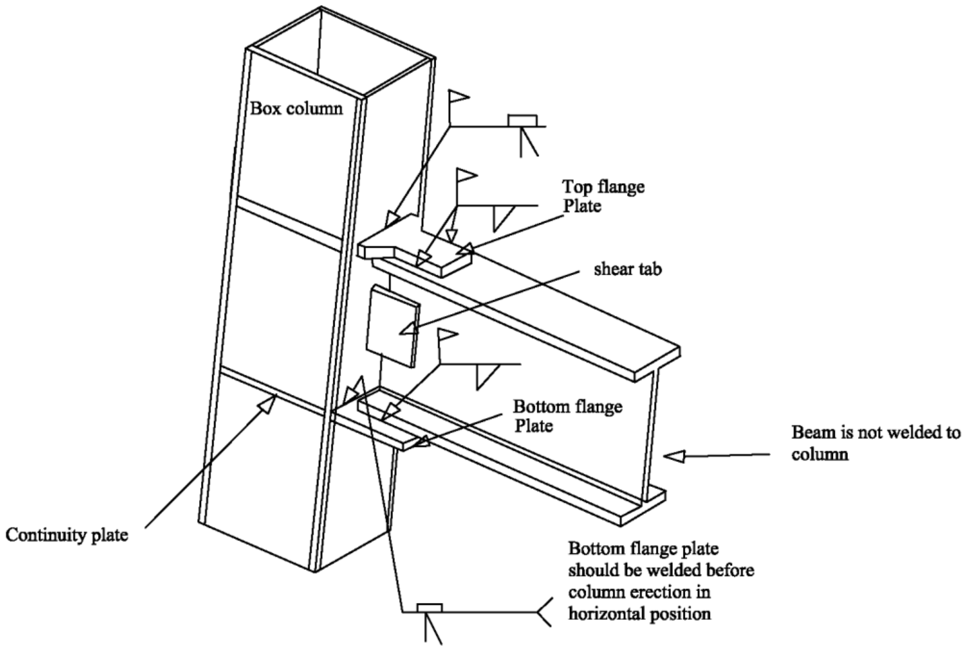


Figure 1: Field welded moment connection.

2. EXPERIMENTAL PROGRAM

The behavior of the moment connections under severe cyclic loading, particularly in regard to the initiation and propagation of fracture, cannot be reliably predicted by analytical means alone. Consequently, the satisfactory performance of connections must be confirmed by laboratory testing (AISC 341, 2005). Therefore, one specimen with flange plate connection was tested to well capture and monitor the connection seismic behavior and related issues. The testing procedure and test results for global and local seismic behavior of the test specimen are discussed in the following sections.

2.1. Test specimen

A prototype building was designed following the AISC Seismic Provisions for Structural Steel Buildings (AISC 341, 2005): A 22-story building with a regular bay floor plan; the typical story height is 3m and typical bay dimensions are 5m. The lateral-load resisting system in the prototype building comprises special moment-resisting frames. All beams in these frames are H shape beams. The beam-column connections are welded flange plate connections. One moment connection between a H-530×250×10×15(mm) beam and a built-up box column (B-450 × 380 × 20 × 12 (mm)) was selected as shown in Figure 2. This connection was named BD. The chosen connection feature the largest beam found in the prototype building because smaller sizes of beams would likely deliver greater rotation capacities. Such approach is consistent with the connection prequalification strategy presented in (FEMA 350) and with the trend observed by (Roeder et al. 2002). The column height in the specimen was selected to match the story height in the prototype building. The length of the beam was set equal to half of the span of the corresponding prototype beam. Specimen was designed using the procedures set forth in (FEMA 350). A 15 mm doubler plate was added to the column panel zone to satisfy the panel-zone-strength requirements of FEMA 350.

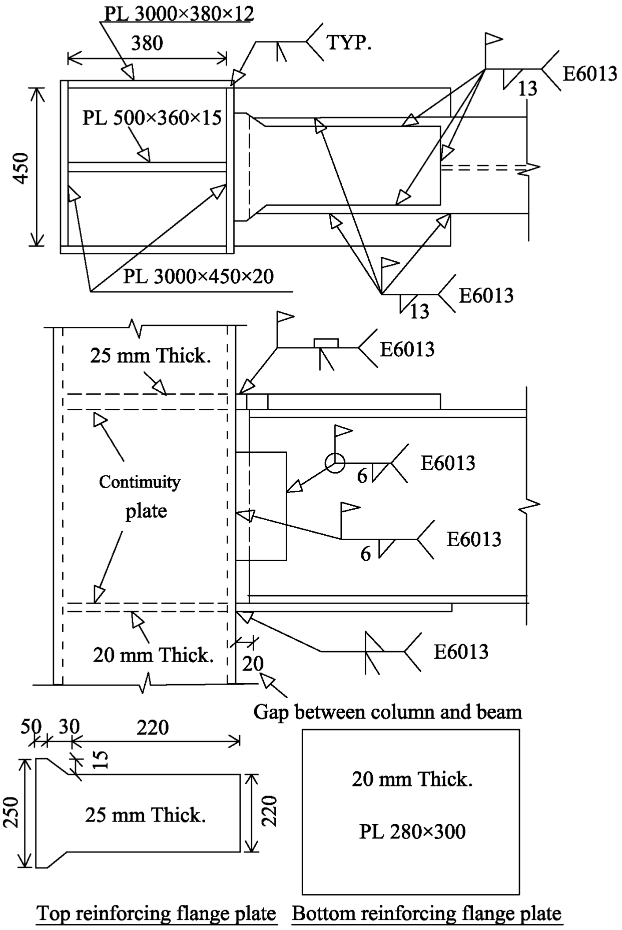


Figure 2: Connection details of specimen BD

2.2. Test setup and instrumentation

According to the shape of the specimen, a test setup was prepared to simulate the boundary conditions of the exterior joint subassembly in a laterally loaded moment frame. The column top and bottom were supported by real hinges. The beam was laterally braced in the vicinity of the plastic hinge and also near the beam end. The general configuration of the test setup and test specimen is shown in Figure 3. The cyclic displacement was applied at the tip of the beam by a hydraulic actuator. The specimen was subjected to the loading sequence proposed by AISC seismic provisions. Cyclic loading history is shown in Figure 4.

2.3. General test observations

As evidenced by the flaking of the whitewash, yielding of the specimen occurred initially in the beam at the nose of flange plate. During the cycles of 2% rad story drift angle, a great amount of the whitewash conspicuously flaked and expanded into the beam flange and the nearby beam web. Local buckling of the beam flanges was noticed in the cycles with 3% rad story drift angle.

In the cycles of 4%rad story drift angle, amplitude of beam local buckling was increased. Tearing was observed at the k-line of the beam top flange during the second cycle of 5% rad story drift angle, as shown Figure 5. At the end of the test, no damage was observed at the groove welds joining flange plates to column flange.

Figure 3: Test setup configuration

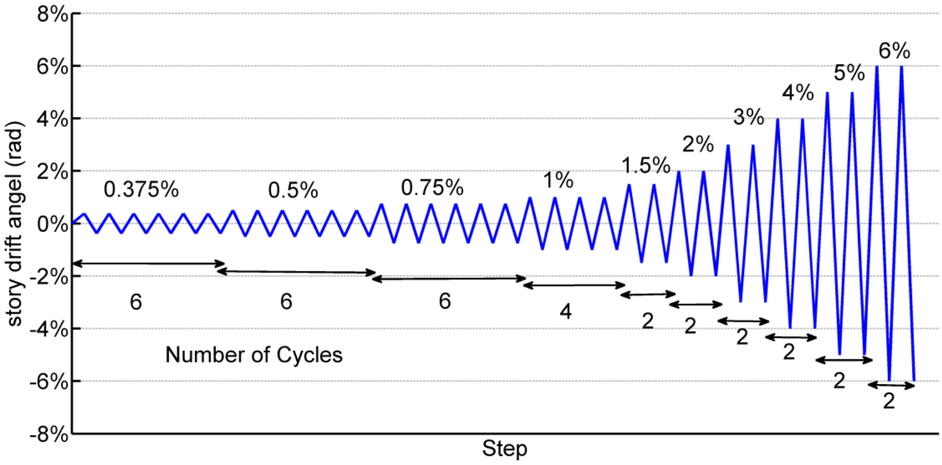


Figure 4: Cyclic loading history.

Figure 5: Fracture at the groove weld joining the beam web to the beam flange in the BD

2.4. General evaluation of the connection behavior

The moment at the column face versus story drift angle (θ) relationship for the test specimen BD is shown in Figure 6. This Figure indicates that moment resistance of the specimen was more than 80% plastic moment of beam at 4% total story drift. Therefore flange plate connection in the specimen BD achieved the AISC seismic provision requirements for special moment frames. It should be noted that the strength degradation of the specimen resulted from ductile local and global buckles during the cyclic loading.

3. NONLINEAR FINITE-ELEMENT ANALYSIS

Finite-element analysis can provide considerable insight into behavior of connections.

3.1. Finite element modeling

ABAQUS model of BD was prepared. As shown in Figure 7, groove welds and fillet welds were modeled. The beam, column, plates, CJP groove welds and fillet welds in the model were discretized using three-dimensional solid (brick) elements. The size of the finite-element mesh varied over the length and height of the model as can be seen in Figure 7. A fine- mesh was used near the connection of the beam to the column and the beam flange to the reinforcing plate. A coarser mesh was used elsewhere. Most of the solid elements were right-angle prisms. Hinged boundary conditions were used to support the column top and bottom. The load was applied by imposing incremental vertical displacements at the beam tip during the analysis.
Data from tests of coupons extracted from the beam and column of specimen were used to establish the stress-strain relationships for the beam and column elements. The weld material was modeled

using the test data of (Kaufmann et al. 1997). To account for material nonlinearities, the von mises yield criterion was employed.

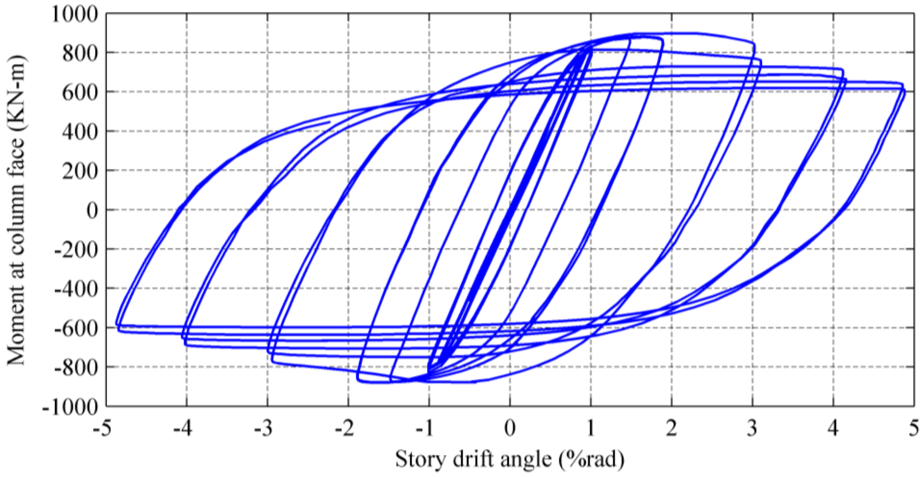


Figure 6: The hysteretic curves for test specimen BD53

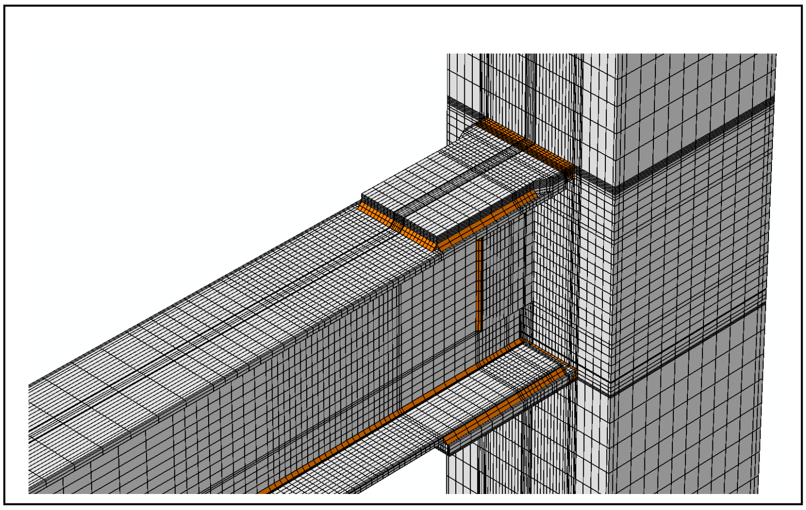


Figure 7: Finite element model

3.2. Rupture index

To compare between the behavior of the different configurations analyzed in this research, a rupture index was used and computed for different cases; this same methodology and approachwas used by others (Chen et al. 2004; Kim et al. 2003). The rupture index (RI) is defined as:

$$RI = \frac{\mathrm{PEEQ}}{\exp(-1.5\frac{\sigma_m}{\bar{\sigma}})} \quad (1)$$

Where PEEQ, σ_m and $\bar{\sigma}$ are, respectively, the equivalent plastic strain, hydrostatic stress, and von mises stress.Where , and are, respectively, the equivalent plastic strain, hydrostatic stress, and von mises stress. Locations in a connection with higher values of RI have a greater potential for ductile fracture.

3.3. Model validation

The finite element analysis of specimen BD was performed by imposing a cyclic displacement to the column tip similar to the loading history that is shown in Figure 4. The cyclic response of finite element model is compared with the cyclic experimental result in Figure 8. As shown in Figure 8, the experimental and finite element results are in good agreement.

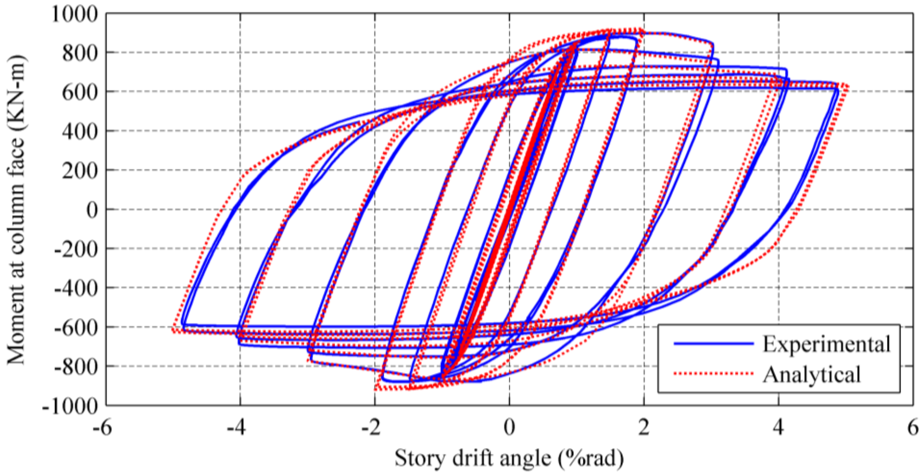


Figure 8: Combined plot of experimental and analytical results for specimen BD

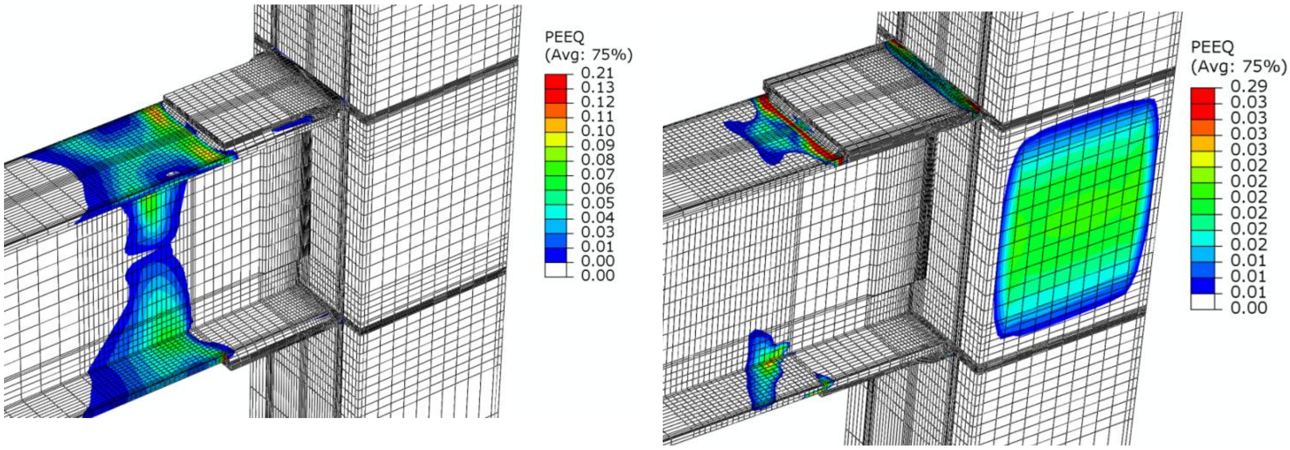
3.4. Influence of panel zone strength on the response of connection

Finite element model B was also prepared. Model B was identical to BD except that the doubler plate in the column panel zone was omitted. Figure 9 presents the PEEQ contours in the finite element models BD and B. In the model B, the panel zone yielded prior to the beam. Only after substantial strain hardening in the panel zone do the beam yield beyond the nose of the flange plates. In contrast, the panel zone of model BD remained elastic throughout the analysis.

Figure 10 presents the RI contours in the models BD and B at 4% rad story drift angle. In model BD, the maximum value of the Rupture Index is recorded in the beam at the nose of the flange plates and values of RI in flange plate at the column face were negligible. Such a result is desirable because the objective of the flange plate connection is to limit or eliminate plastic straining at the column face by forcing yielding into the beam beyond the nose of flange. For this reason, no crack was observed at flange plate-column interface of test specimen BD. For model B, the maximum value of the Rupture Index is recorded in the flange plate at the face of the column. The plastic deformation of panel zone in this model increases PEEQ value at the flange plate-column interface and consequently increases potential for fracture at this location.

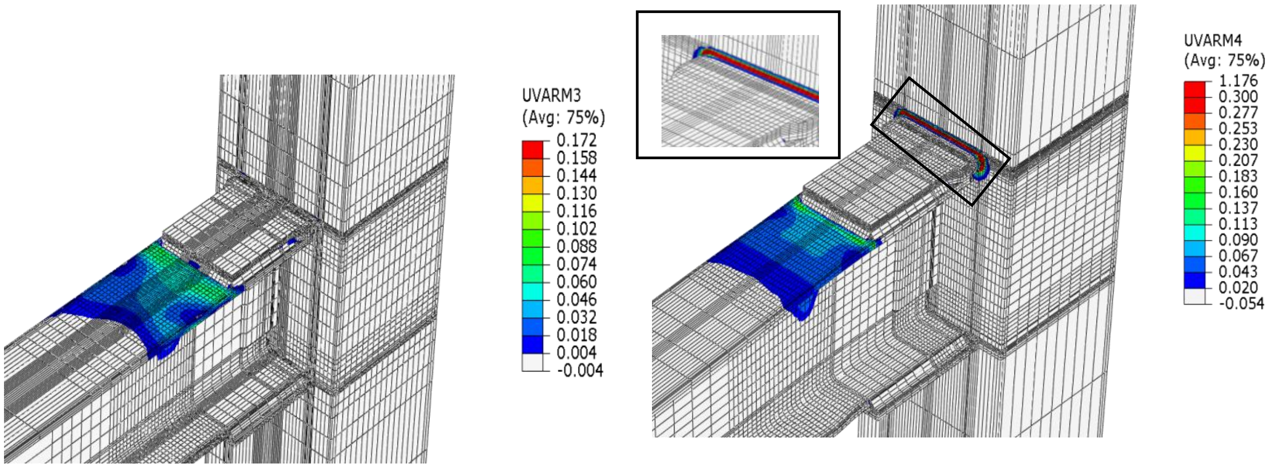
4. CONCLUSIONS

One full-size specimen with flange plat connection was tested using a standard connection requalification test protocol. The specimen composed of a H-shaped steel beam with the dimensions of H-530×250×10×15(mm) connected to a box column with the measurements of B-450 × 380 × 20 × 12 (mm). Specimen was designed using the procedures set forth in FEMA 350. A 15 mm doubler plate was added to the column panel zone to satisfy the panel-zone strength requirements of FEMA 350. The flange plate connection of test specimen achieved the AISC seismic provision requirements for special moment frames. Then, a validated finite element model was used to investigate the effect of panel zoon strength on the response of flange plate connection. The finite element results showed that a weaker panel zone increases PEEQ value at the flange plate-column interface and consequently increases potential for fracture at this location.



a) b)

Figure 9: PEEQ contours in the finite element models a) BD and b) B



a) b)

Figure 10: RI contours in the finite element models a) BD and b) B

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