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Behavior of skewed extended shear tab connections part II: Connection to supporting flange

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ABSTRACT

It has been proven that the torsional behavior of the extended shear tab connections is affected by the connection orientation due to the additional torsional moment when the plate is welded to the supporting member web (flexible support) in Part I of this study. However, these connections may act differently when the plate is welded to the flange of the supporting member (rigid support). The goal of this study is to check the adequacy of this finding for skewed extended shear tab connections when the plate is welded to the supporting member flange (rigid support). The finite element software ABAQUS ABAQUS (2013a) [1] was used to simulate and study the behavior of orthogonal extended shear tab connections studied experimentally by Metzger (2006). The Finite Element Analysis (FEA) of these connections captured the same failure modes as the experiments. Moreover, additional failure modes were observed by the FEA such as shear yield of the plate, bolt shear, plate twist, and local buckling of the supporting member. After validating the models, the shear tab and supported beam in the orthogonal configurations were oriented at different angles to check the effect of the connection orientation on the behavior of these connections. It was observed that the supporting member contributes in resisting the additional torsional moment at the elastic level. However, this contribution significantly reduces with the increase of the connection shear force and becomes neglected at the plastic level. Additionally, the effect of the connection orientation on the torsional and bending behavior of these connections overall is insignificant. Thus, the modifications on the design procedure for the skewed extended shear tab connections with flexible support in Part I do not apply to these connections.

1. Background

Previous studies investigated the behavior of the extended shear tab and skewed connections, experimentally and analytically. Some of these studies are as follows:

Richard et al. [4] studied the behavior of the single plate framing connections with ASTM A325 and ASTM A490 bolts. The authors proposed design procedure for the single plate framing connections with ASTM A325 and ASTM A490 bolts based on the numerical and experimental results obtained from their study.

Richard et al. [5] investigated the behavior of the single plate framing connections with A307 bolts. The authors proposed a detailed procedure based on the results obtained from their experimental study to design single plate framing connections with ASTM A307 bolts.

Cheng et al. [6] performed a theoretical parametric study to investigate the behavior of coped beams with various coping details using the finite element software BASP and ABAQUS. The authors indicated that the buckling capacity is highly affected by the cope length/depth and span length.

Astaneh et al. [7] investigated the single plate shear connections behavior. It was concluded from this study that the limit states associated with single plate connections are: plate yielding, fracture of the net section of plate, bolt fracture, weld fracture, and bearing failure of bolt holes.

Astaneh et al. [8] studied the behavior of steel single plate shear connections, the authors indicated that the shear connections, in addition to the adequate shear capacity, should have sufficient rotational ductility to accommodate simply supported beam end-rotation to prevent development of a significant moment in the connection.

Ashakul [9] investigated the parameters affecting the bolt shear rupture strength of the single shear plate connection using the finite element program ABAQUS. The author proposed a relationship for calculating plate shear yielding strength based on shear stress distribution.

Creech [10] suggested in his study that the AISC design procedure for single-plate shear connections is overly conservative. The author performed ten full-scale tests for rigid and flexible connections. The author found that the magnitude of eccentricity for connections with four bolts or more is not significant, but for two and three bolts connections, the eccentricity should be considered in the design procedure.

Rahman et al. [11] presented a three dimensional model to study the behavior of the unstiffened extended shear tab connections and validated the experimental results performed by Sherman and Ghorbanpoor

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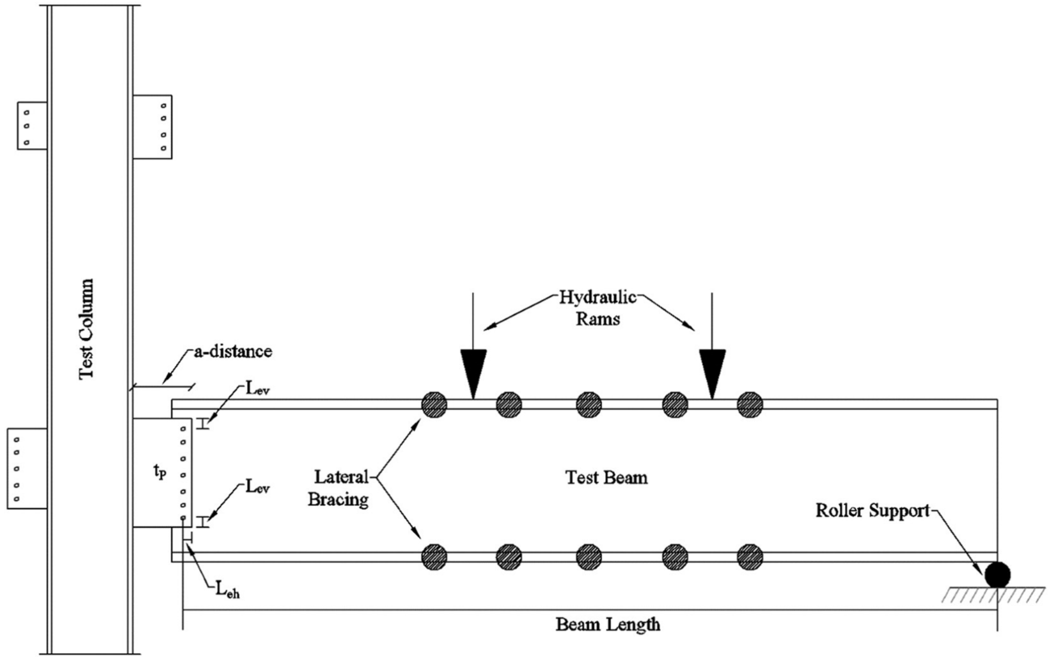

Fig. 1. Metzger [3] test setup.

[12]. The authors concluded that the presented model in their study is a powerful tool in addressing the failure of the unstiffened extended shear tab connection in the plastic region.

2. Source of data (experimental research)

The methodology used in this research is finite element analysis. In order to check the adequacy of the proposed finite element models, orthogonal extended shear tab connections with the plate welded to the supporting column flange investigated experimentally by Metzger [3] used as a reference to validate these models. Metzger [3] performed eight experiments, four conventional plate connections and four extended plate connections. Since the purpose of this paper is to investigate the behavior of the extended shear tab connections, only the four extended connections were studied and modeled. The requirements of the AISC [13] specification and design procedure in the AISC 13th edition manual (2005) were used to design these connections. All bolt holes were standard holes with 1.25 in. (31.8 mm) and 1.5 in. (38.1 mm) vertical edge distance (L_{ev}) and horizontal edge distance (L_{eh}), respectively. ASTM A325-X bolts with 3/4 in. (19 mm) diameter were used for all tests. In order to prevent brittle failure of the connections, the plates were designed to a moment capacity less than the moment capacity of the bolt group. Additionally, the weld size used in these connections was equal to one half times the thickness of the plate. Fig. 1 and Table 1 show the details of the extended connections tested by Metzger [3].

Each test consisted of an extended plate welded from one side to the column flange in such a way that the plate's longitudinal axis and column's weak axis align. The other side of the plate was bolted to the supported beam. The supported beam was supported on the far end by a simple roller support. Two hydraulic rams were placed on top of the beam flange to control the shear and rotation imposed on the connection. Additionally, braces were placed along the test beam to prevent lateral torsional buckling by using angles bolted to the beam web and extend between the beam flanges. Moreover, the four extended plates were welded to the same column, two on each side. In order to provide sufficient bracing for the column, a channel was bolted to the testing frame columns and to the test column. The goal of the investigation was to load the connections up to failure and to reach a beam end rotation of 0.03 rad at the same time by imposing a combination of shear and rotation on the connections.

The previous details for the four extended connections were used to perform FEA using ABAQUS [2]. Additionally, the results from the experimental investigation were used to validate the results obtained from the FEA.

3. Non-linear finite element modeling

The generation of the finite element models was explained in details previously in Part I. The material properties for the plates and members obtained by Metzger [3] were used in modeling the skewed extended shear tab connections with the plate welded to the supporting column flange. The material properties for the plates and members are shown in Table 2.

4. Models validation

The FEA results were verified in two ways: by comparing the failure modes and by comparing the connection's shear versus the beam end rotation curves. Table 3 shows a comparison between the ultimate

Table 1
Metzger tests configurations and geometries.

Test	Bolt columns	Bolts rows	Plate thickness (mm)	a-Distance (mm)	Beam section	Beam length (m)	Column section
6B2C - 4.5 - 1/2	2	3	12.70	114.30	W18 × 55	5.66	W21 × 62
10B2C - 4.5 - 1/2	2	5	12.70	114.30	W30 × 108	7.49	W21 × 62
7B1C - 9 - 3/8	1	7	9.53	228.60	W24 × 62	6.97	W21 × 62
10B2C - 10.5 - 1/2	2	5	12.70	266.70	W24 × 62	6.97	W21 × 62