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 (2013) 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. Addirionally, 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.

Keywords: Extended Shear Tab, Skewed Connections, Rigid Supports, Twist, Torsional Moment, ABAQUS.

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. (1980) 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. (1982) 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. (1984) 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. (1989) 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. (1993) 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 (2004) 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 (2005) 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. (2007) 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 (2002). 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 (2006) used as a reference to validate these models. Metzger (2006) 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 (2005) 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. Figure 1 and Table 1 show the details of the extended connections tested by Metzger (2006).

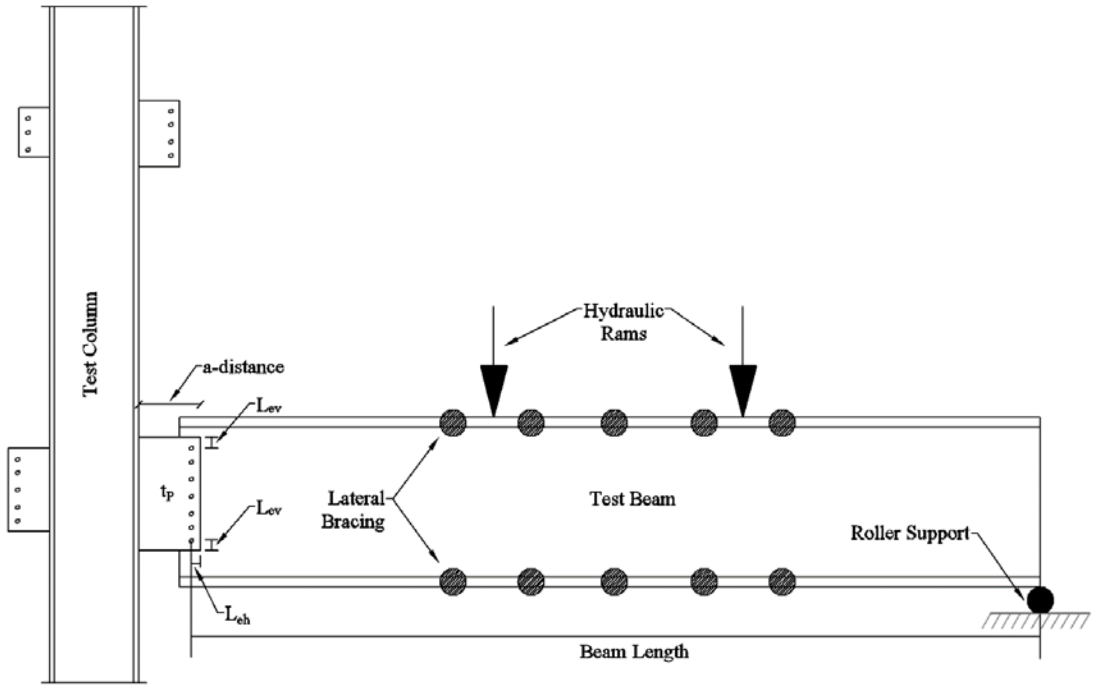

Figure 1. Metzger (2006) test setup.

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	W18x55	5.66	W21x62
10B2C - 4.5 - 1/2	2	5	12.70	114.30	W30x108	7.49	W21x62
7B1C - 9 - 3/8	1	7	9.53	228.60	W24x62	6.97	W21x62
10B2C - 10.5 - 1/2	2	5	12.70	266.70	W24x62	6.97	W21x62

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 radians 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 (2013). 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 (2006) 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.

Table 2. Material Properties.

Member	E, MPa	ν	σ_y, MPa	σ_u, MPa	% Elongation
9.53 mm Tab	200,000	0.3	478	664	20
12.7 mm Tab	200,000	0.3	470	674	22
W18X55	200,000	0.3	406	535	27
W24X62	200,000	0.3	400	532	27
W30X108	200,000	0.3	424	547	31

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 shear forces and failure modes obtained from the

FEA and experiments. This table lists primary failure modes and followed by secondary failure modes in parentheses.

Table 3. Ultimate shear forces and failure modes.

Test	Failure Modes*		V_{exp}				
	Experimental	FEA	Experimental (kN)	FEA (kN)	Error (%)	Average (%)	Standard Deviation
6B2C - 4.5 - 1/2	E	E, (H)	399	409	2.5		
7B1C - 9 - 3/8	G, B, F	G, B, F, (C, A, D)	436	467	7.1	6.3	2.8
10B2C - 10.5 - 1/2	I,C	I, (G, F, H, E)	421	460	9.3		

* Failure modes:

A = bolt shear
B = bolt bearing of the beam web
C = shear yield
D = twist
E = weld
F = plate buckling
G = LTB of the beam flanges at midspan
H = yielding of plate corners
I = local buckling of the beam web at midspan

4.1. Connection's Shear-Beam End Rotation Curves

In the experiments, the beam end rotation was measured using two linear potentiometers, the first one was placed over the center of gravity of the bolt groups, and the second potentiometer was placed 6 in. away from the first one. In the FEA, the vertical displacement was obtained at the same locations as measured and recorded in the experiments to measure the beam end rotation.

Figures 2, 3, and 4 show the FEA and the experimental results for the connection's shear versus beam end rotation curves for Test 5, 7 and 8. As shown in the figures, there is good agreement between the experimental results and FEA results. In general, the results show that the FEA models are stiffer than the actual experimental. This was expected due to the difficulties in applying the lateral bracings along the beam and in controlling the lateral torsional buckling of the beam's top flange during the experiments. Additionally, the FEA shows local buckling in the column's web and flange; note that in the experimental work, all the connections were welded to the same column. The local buckling in the column web and flange affect the connection behavior.

However, the lateral bracing can be more controlled in the FEA, and unlike the experimental study, no initial stress and possible plastic deformations existed in the column. Nevertheless, the error did not exceeded 10% in all the models.

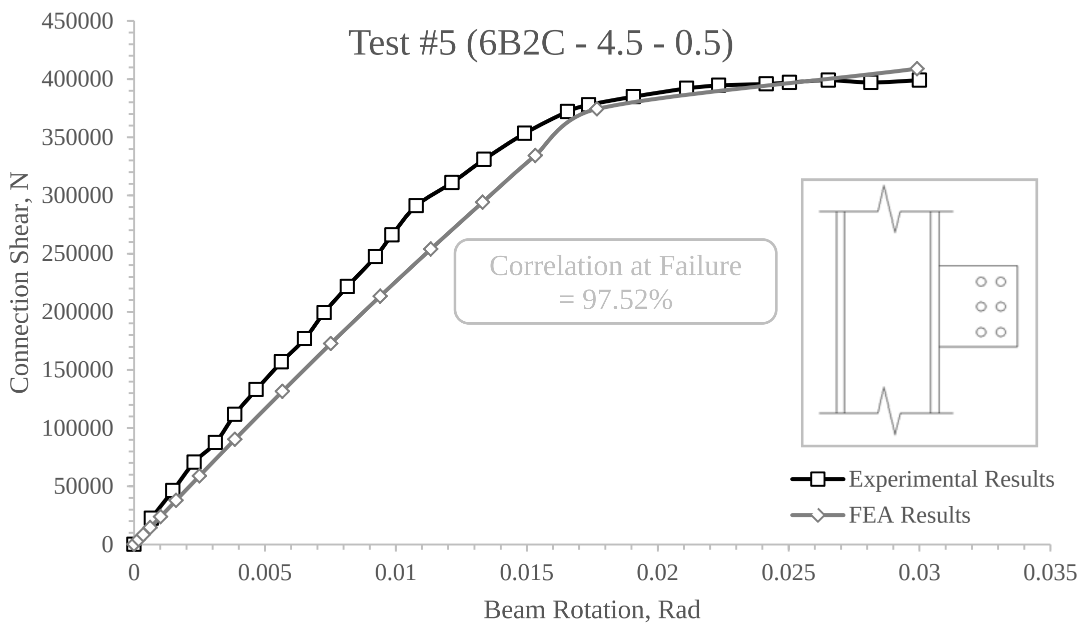

Figure 2. Connection Shear-Beam end rotation curves for test #5.

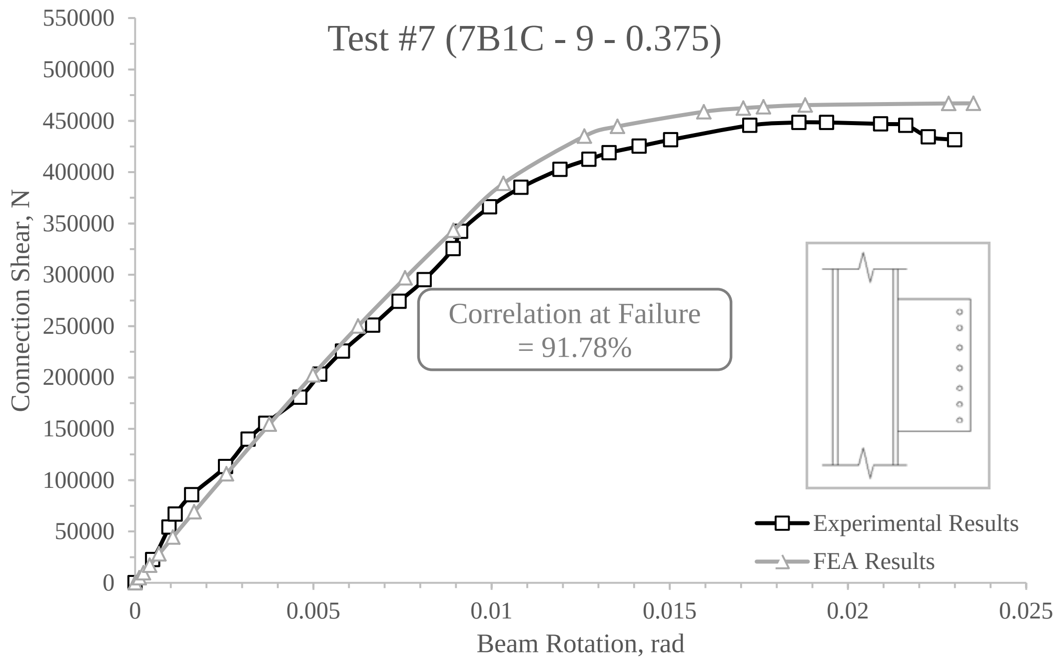

Figure 3. Connection Shear-Beam end rotation curves for test #7.

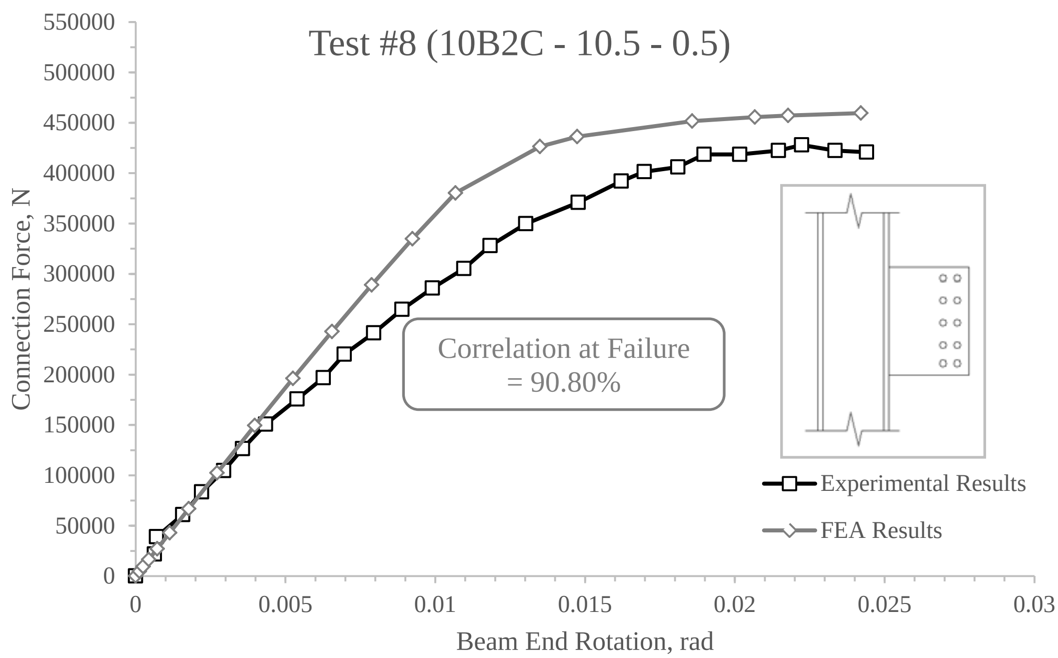

Figure 4. Connection Shear-Beam end rotation curves for test #8.

4.2. Failure Modes

Figures 5 through 8 show the failure modes obtained from the FEA and experiments. As shown in the figures, same failure modes were obtained from the FEA and the experiments. However, the FEA showed additional failure modes that are shown in parentheses in Table 3. Bolt shear failure mode was observed in test (7B1C - 9 - 3/8), this failure mode was not observed in the experiments. Also, plate twisting in tests with a-distance (the distance between the weld line and the bolt line) more than 4.5 in. (114 mm) was significant. The failure modes addressed in the experimental investigation are based on visual inspection of the damaged connections. The failure modes observed in the finite element models are based on the presence of the equivalent plastic strain. It was observed that the equivalent plastic strain for the additional secondary failure modes are small and hard to be observed using visual inspection.

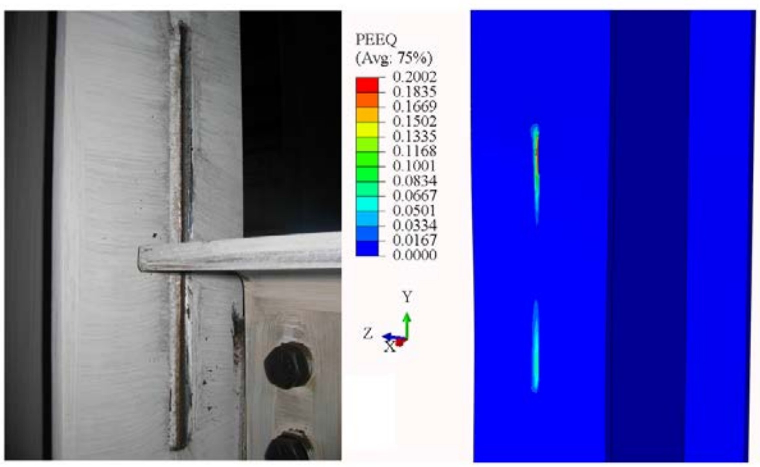

Figure 5. Weld rupture failure mode.

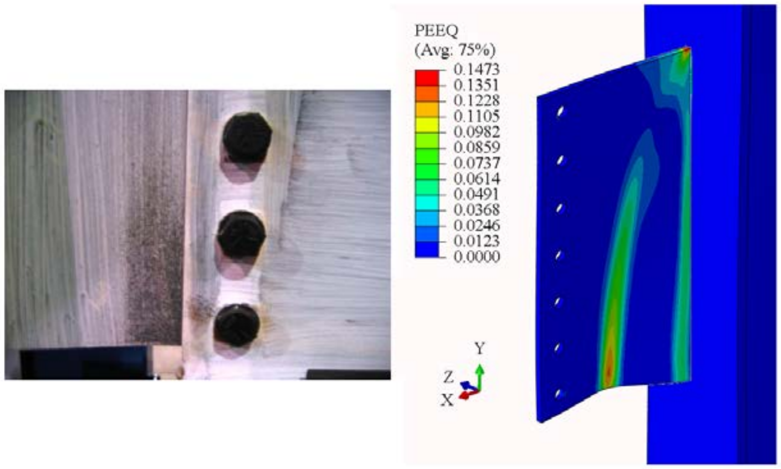

Figure 6. Plate buckling failure mode.

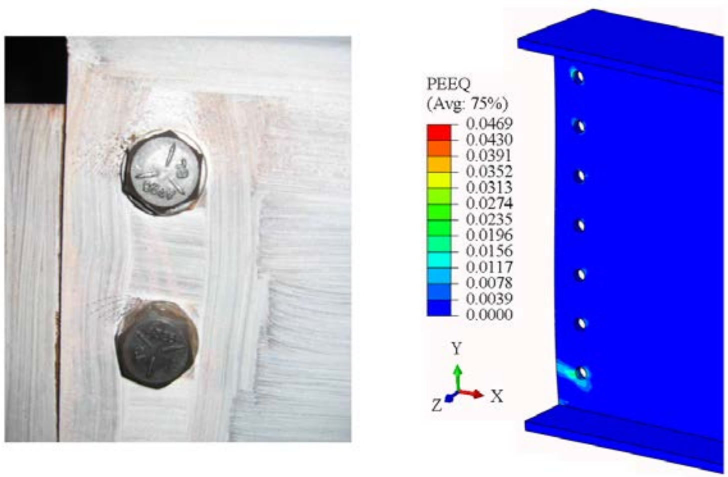

Figure 7. Bolt bearing of the beam web failure mode.

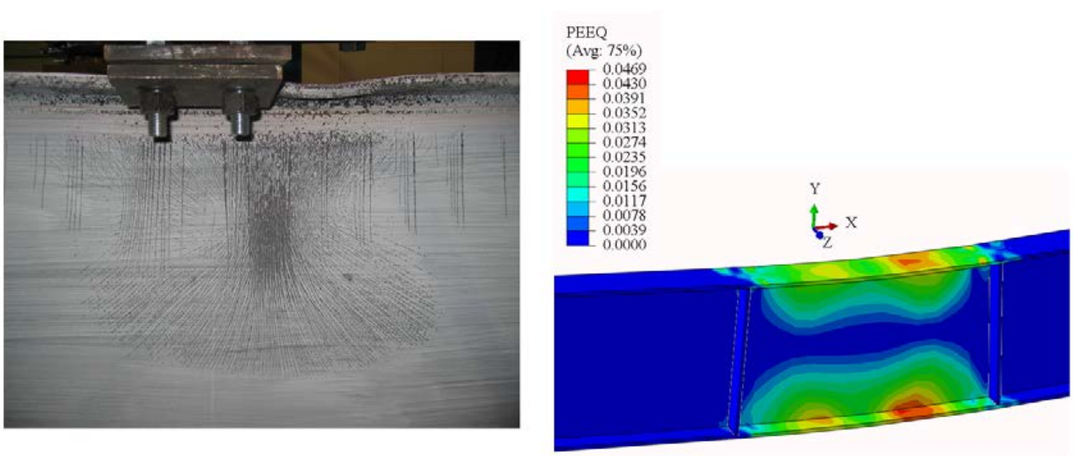

Figure 8. Plastic deformation at the beam midspan failure mode.

Additionally, local web buckling and local flange buckling were observed in the column due to the contribution of the column in resisting the additional torsional and bending moment added to the connection. As indicated, the focus of this study is to investigate the behavior of the connection. However, these additional moments should be considered in designing the supporting member. Figure 9 shows the local buckling of the column for test (7B1C - 9 - 3/8).

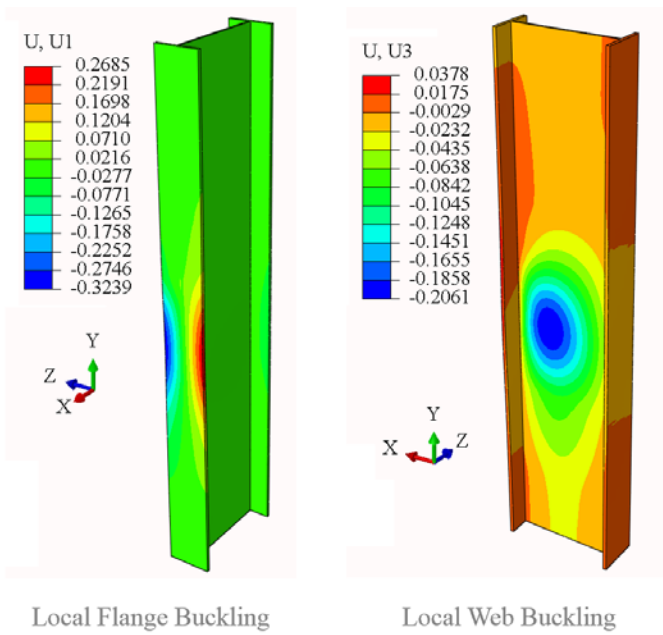

Figure 9. Local buckling of test (7B1C - 9 - 3/8) column (deformation in inches – 1 in = 25.4mm).

5. Skewed Extended Shear Tab Connections Investigation

The extended shear tab connections used to verify the models are in orthogonal configuration where the beam longitudinal axis and column weak axis are parallel. Four skewed connections with different skewed angles (5, 10, 15 and 20 degrees) were generated in order to make a comparison between the orthogonal and skewed configurations. Each orthogonal model was modified by changing the orientation of the supported beam and plate in such a way that the skewed beam's longitudinal axis has an angle (α) with the weak axis of the column. Additionally, the plate geometry was modified by extending the plate to the surface of the column flange to insure full contact between the plate and column flange. Local coordinate systems were assigned to the plate, supported beam and each bolt in order to adjust the plate twist, bolts pre-tensioning forces and far end reactions with the beam orientation. Figure 10 shows the orthogonal and skewed extended shear tab connection with rigid support.

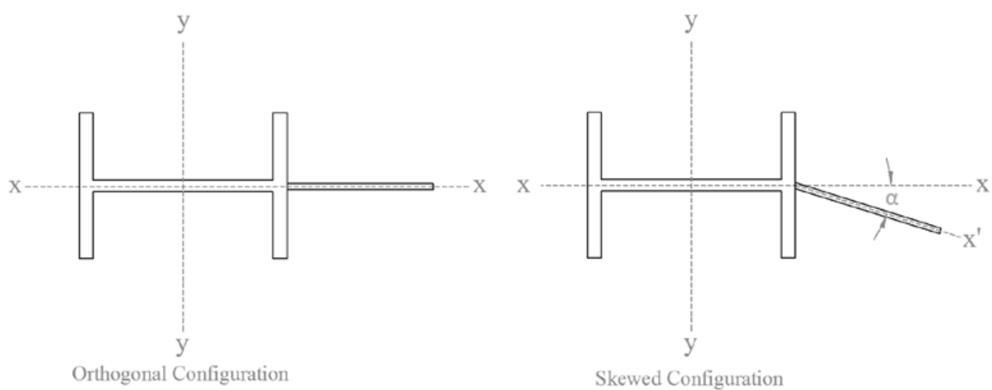

Figure 10. Orthogonal and skewed extended shear tab connections with plate welded to column flange.

6. Results and Discussion

In order to achieve the goal of this study, shear-displacement curves; shear-twist curves; and failure modes for each model at different skewed angles (α) were obtained and investigated.

Connection Shear-vertical displacement curves for orthogonal and skewed configurations of test #5 are shown in Figure 11. The vertical displacement slightly decreases with the increase of the connection orientation. As was explained in Part I, the vertical displacement of the connection depends on the applied moment due to the shear transferred from the beam end to the connection plate. Additionally, the applied moment component ($R \times a \times \cos\alpha$) on the skewed configuration is less than the applied moment ($R \times a$) on the orthogonal configuration (see Figures 12 and 13). In other words, as the connection orientation increases, the applied bending moment on the connection decreases, leading to the reduction of the connection vertical displacement.

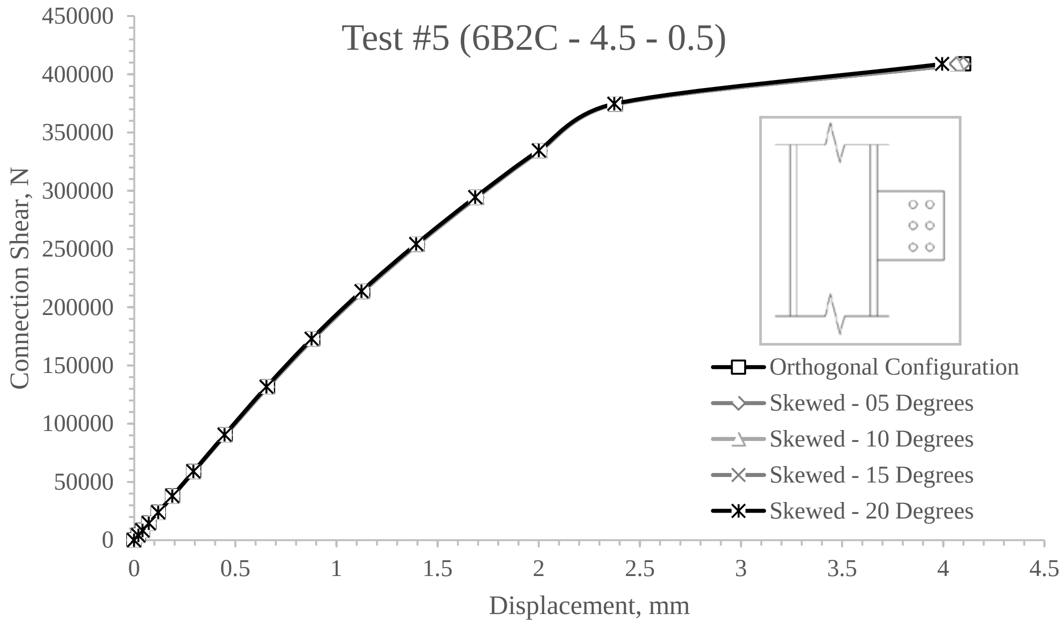

Figure 11. Connection Shear-displacement curve (Test #5).

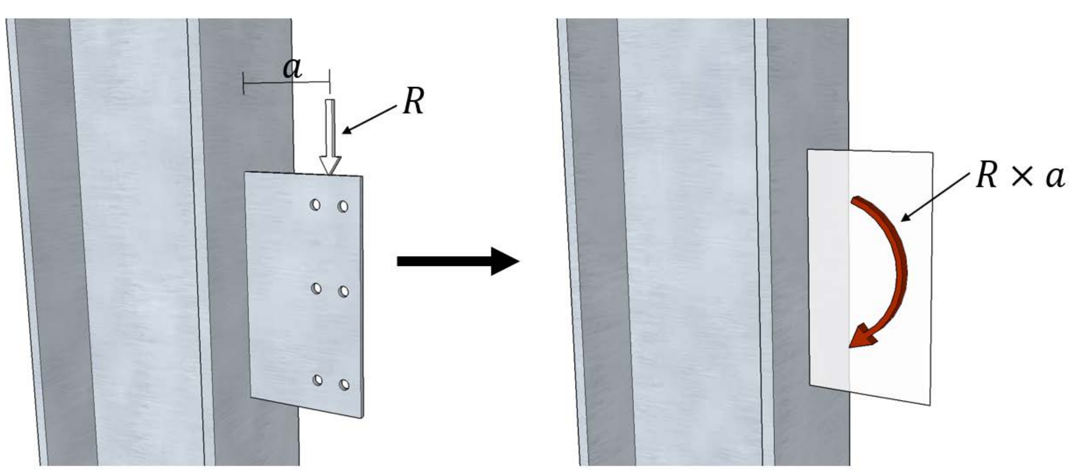

Figure 12. The applied bending moment on the orthogonal configuration.

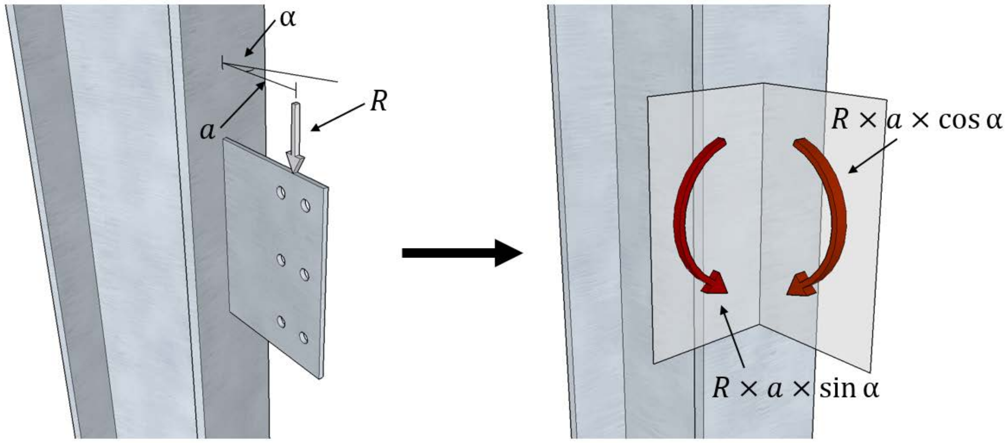

Figure 13. The applied bending moment on the skewed configuration.

Connection Shear-twist curves for test #5 are shown in Figure 14. It was observed that the torsional stiffness increases with the increase of the connection orientation at low shear force. However, the torsional stiffness starts to decrease as the connection orientation increases at a high level of shear force. As was mentioned in Part I, the total torsional moment on the connection is the sum of the torsional moment due to the overlap between the plate and beam web longitudinal axes and the torsional moment due to the connection orientation. The torsional behavior in Figure 14 is related to the contribution of the column in resisting the torsional moment applied to the connection due to the connection orientation. The stress distribution on the plate and column at low, medium and high shear forces are shown in Figures 15, 16 and 17, respectively. In the three cases, the plate stress distribution is the same for the orthogonal and skewed configurations. However, it was observed that the stresses are concentrated on the left side of the column flange and extended away from the connection weld line for the skewed configuration when compared with the orthogonal configuration where the stresses are distributed on both flanges around the connection weld line. This proves that the column contributes significantly in resisting the additional torsional moment due to the connection orientation.

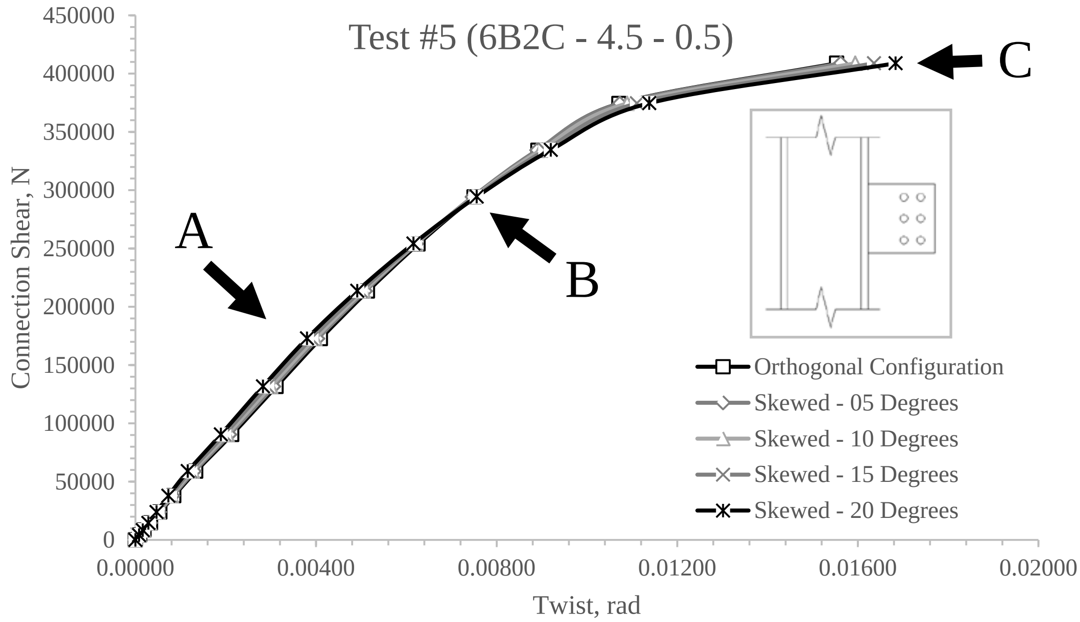

Figure 14. Shear-twist curve (Test #5).

The stress distribution on the plate and column at low shear force applied to the connection are shown in Figure 15 for the orthogonal and skewed configurations. In the orthogonal configuration (the left side of Figure 15), the stresses developed around the weld line on both sides of the column flange. Also the stresses extended from the center of the column flange near the edges of the weld line to the column flange edge due to the contribution of the column flange in resisting the torsional moment caused by the overlap between the plate and beam web longitudinal axes. However, this contribution is insignificant as can be seen in the figure. On the other hand, for the skewed configuration (the right side of Figure 15), the stresses are developed on the column flange side that makes an acute angle with the plate and expanded above and below the weld line due to the contribution of the column flange in resisting the total torsional moment. In other words, at low connection shear force, the contribution of the column flange in resisting the total torsional moment applied at the connection is higher for the skewed configuration when compared with the orthogonal configuration where most of the torsional moment is resisted by the plate itself. This explains the increase of the torsional stiffness as the connection orientation increases at a low level of connection shear force. However, the contribution of the column flange in resisting the total torsional moment in the skewed configuration reduces as the connection shear force increases and becomes insignificant at a high shear force. This explains the reduction of the torsional stiffness as the connection orientation increases at a high level of connection shear force.

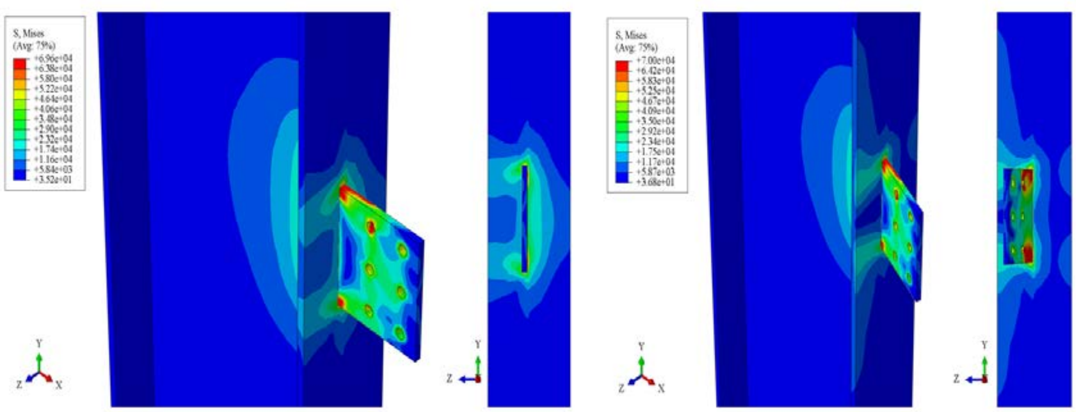

Figure 15. Stress distribution of orthogonal and skewed configurations for point A (Stresses in psi – 1 psi = 6.89 kPa).

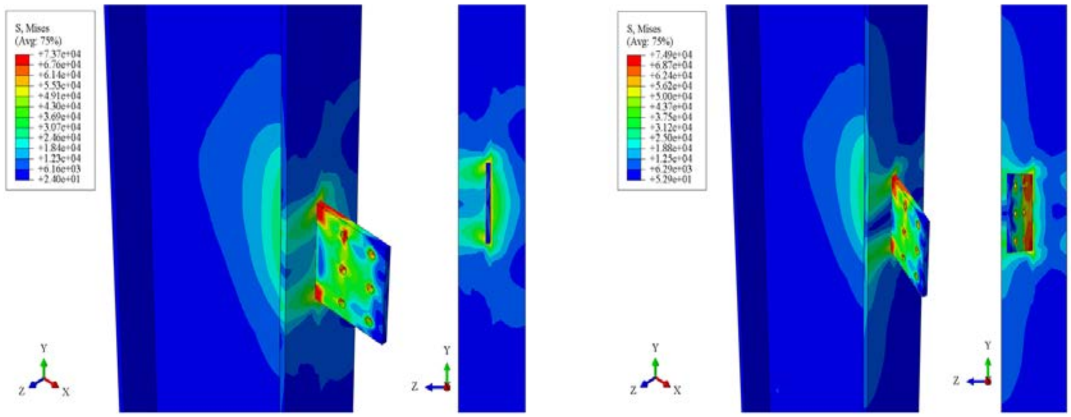

Figure 16. Stress distribution of orthogonal and skewed configurations for point B (Stresses in psi – 1 psi = 6.89 kPa).

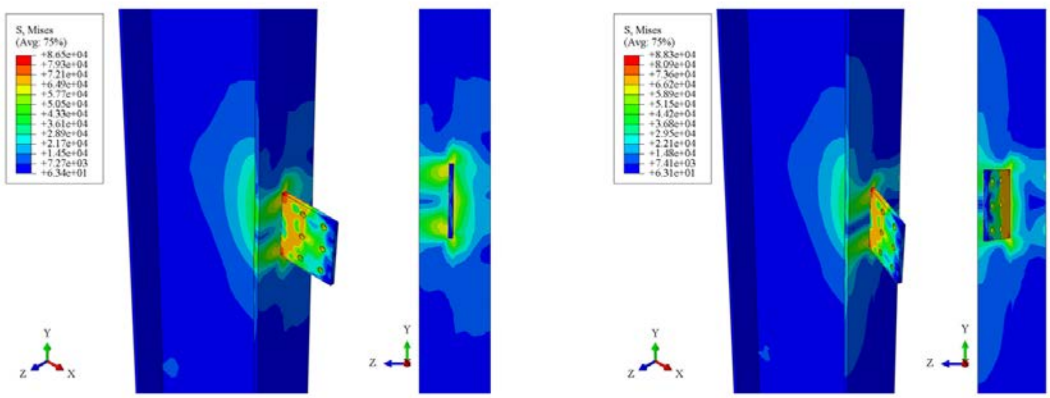

Figure 17. Stress distribution of orthogonal and skewed configurations for point C (Stresses in psi – 1 psi = 6.89 kPa).

The ultimate shear, failure modes, maximum vertical displacement, and maximum twist for connections with the plate welded to the supporting member flange are shown in Table 4. The same failure modes occurred at the orthogonal and skewed configurations at different connection orientations. However, the maximum plate twist increases with the increase of the connection orientation. Additionally, since the maximum vertical displacement slightly decreases with the increase of the connection orientation for the majority of the connections, the effect of the connection orientation on the ultimate vertical displacement of the connection can be ignored.

Table 4. FEA results for orthogonal and skewed configurations.

Test	α	V_{FEA} (kN)	Failure Modes	Displacement (mm)	Twist (rad)
6B2C - 4.5 - 1/2	0	408.93	E, (H)	4.09	0.01553
	5	408.93		4.09	0.01562
	10	408.88		4.06	0.01595
	15	408.88		4.04	0.01636
	20	408.88		3.99	0.01684
7B2C - 9 - 3/8	0	466.97	G, B, F, (C, A, D)	4.32	0.04695
	5	466.40		4.34	0.04857
	10	465.77		4.01	0.04652
	15	465.28		4.17	0.05006
	20	464.79		4.32	0.05377
10B2C - 10.5 - 1/2	0	459.59	I, (G, F, H, E)	7.11	0.04881
	5	459.63		7.06	0.04981
	10	459.68		7.06	0.05169
	15	459.63		7.09	0.05459
	20	459.72		7.42	0.0613

* Failure modes:

A = bolt shear
B = bolt bearing of the beam web
C = shear yield
D = twist
E = weld
F = plate buckling
G = LTB of the beam flanges at midspan
H = yielding of plate corners
I = local buckling of the beam web at midspan

Since the increase of the plate twist is a function of the skewed angle (α), the normalized plate twist (plate twist for skewed configuration/plate twist for orthogonal configuration) was obtained in order to find the relationship between the plate twist of orthogonal and skewed configurations; these values are shown in Figure 18.

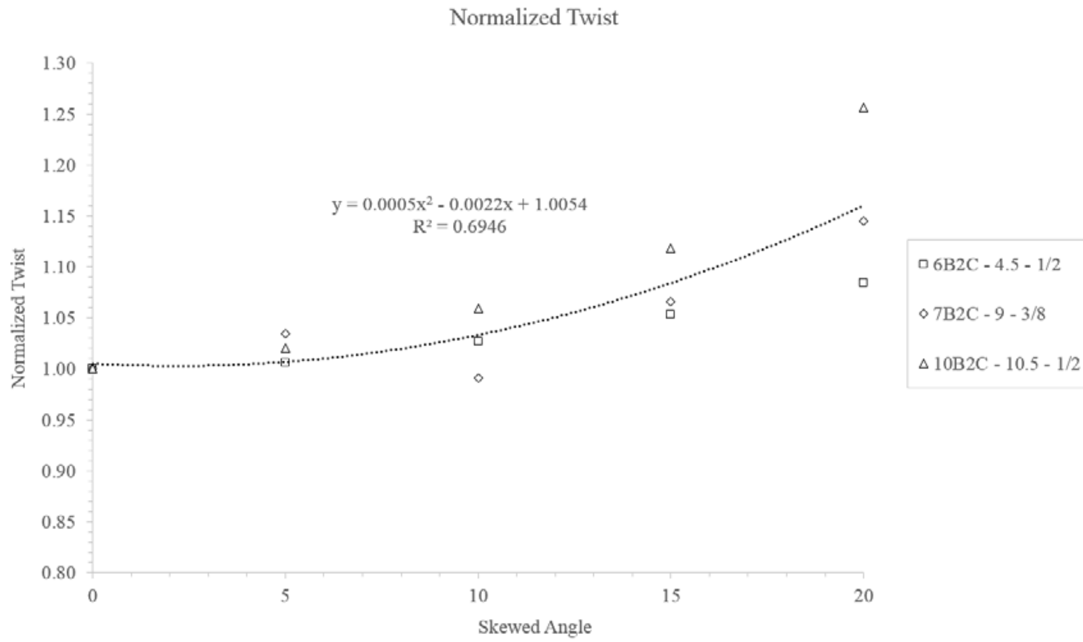

Figure 18. Normalized plate twist of the skewed configuration.

A polynomial relationship of the second degree was used to represent the relationship between the connection orientation and normalized plate twist. This relationship is represented as follows:

$$\frac{T_\alpha}{T_o} = 0.0005 \times x^2 - 0.0022 \times x + 1.0054 \qquad (1)$$

$$T_\alpha = (0.0005 \times x^2 - 0.0022 \times x + 1.0054) \times T_o \qquad (2)$$

Where:

T_α = The twist of the plate along the bolt line for the skewed configuration.

T_o = The twist of the plate along the bolt line for the orthogonal configuration.

α = The skewed angle.

In order to verify Equation (1), the normalized plate twist was calculated using the proposed equation and compared with the values obtained from FEA; the maximum error was (7.83%) at skewed angle $\alpha = 20^0$. The error of using Equation (1) to predict the normalized twist for the skewed extended shear tab connections with rigid support is shown in Table 5.

Table 5. Errors of using the proposed equation to predict the normalized twist of the extended shear tab connections with rigid support.

α	Test	Proposed Equation	FEA	Error (%)	Min (%)	Max (%)	Average	Standard Deviation
0	6B2C - 4.5 - 1/2	1	1.01	-0.54	0.54	0.54	-0.54	0.0
	7B2C - 9 - 3/8	1	1.01	-0.54				
	10B2C - 10.5 - 1/2	1	1.01	-0.54				
5	6B2C - 4.5 - 1/2	1.01	1.01	0.31	0.31	2.24	1.28	0.8
	7B2C - 9 - 3/8	1.03	1.01	2.24				
	10B2C - 10.5 - 1/2	1.02	1.01	1.28				
10	6B2C - 4.5 - 1/2	1.03	1.03	-0.33	0.33	4.38	-0.73	2.8
	7B2C - 9 - 3/8	0.99	1.03	-4.38				
	10B2C - 10.5 - 1/2	1.06	1.03	2.51				
15	6B2C - 4.5 - 1/2	1.05	1.08	-3.32	1.39	3.32	-0.53	2.7
	7B2C - 9 - 3/8	1.07	1.08	-1.39				
	10B2C - 10.5 - 1/2	1.12	1.08	3.13				
20	6B2C - 4.5 - 1/2	1.08	1.16	-7.54	0.99	7.83	-0.23	6.3
	7B2C - 9 - 3/8	1.15	1.16	-0.99				
	10B2C - 10.5 - 1/2	1.26	1.16	7.83				

It was proven from Equation (1) that the plate twist is not only a function of the overlap between the plate and beam web longitudinal axes, but it is also a function of the connection orientation and the rigidity of the supporting member. The torsional stiffness of the skewed configuration is higher than the torsional stiffness of the orthogonal configuration due to the contribution of the supporting member in resisting the additional torsional moment at the elastic level. Additionally, at the inelastic level, the contribution of the supporting member in resisting the additional torsional moment becomes insignificant and can be neglected; the torsional stiffness of the skewed configuration becomes less than the torsional stiffness of the orthogonal configuration. However, the change in the torsional behavior of the connection due to the connection orientation is insignificant and can be neglected. In conclusion, the modifications on the design procedure for the skewed extended shear tab connections with flexible support (Part I) do not apply for these connections when the plate is welded to the supporting member flange (rigid support).

7. Conclusion and current work

The purpose of this study is to investigate the behavior of skewed extended shear tab connections with the plate welded to the supporting column flange numerically. A detailed nonlinear finite element model was developed to predict the behavior and performance of these connections. The models have been verified against data from experimental tests done by Metzger (2006) for orthogonal extended shear tab connections. Each orthogonal configuration was skewed at different angles. The following observations and conclusions can be made from this investigation based on the finite element results. It should be noted that Metzger (2006) experimental study results were used in this paper only for validation purposes:

1. The proposed finite element model is a good tool in predicting the failure mechanism of the orthogonal and skewed extended shear tab connections with rigid support.
2. The FEA models were able to capture the same failure modes as the experiments. Additionally, it was able to detect additional failure modes that were not obtained by the experimental investigation. These failure models have a low plastic strain values which is hard to be observed using visual inspection in the experimental testing.

3. For the skewed connections with rigid support, the connection orientation slightly affects the ultimate vertical displacement of the connection. This effect can be ignored.
4. The column flange contributes significantly in resisting the additional torsional force due to the connection orientation at a low level of shear force. However, this contribution reduces with the increase of the connection's shear force and becomes insignificant at a high level of shear force.
5. The ultimate plate twist increases with the increase of the connection orientation due to the additional torsional moment.
6. The modifications on the design procedure in AISC (2010) for the skewed extended shear tab connections with flexible support (Part I) do not apply for these connections when the plate is welded to the supporting member flange (rigid support).

The additional torsional moment due to the connection orientation might overstress the weld group between the plate and the supporting member. The effect of the connection orientation on the weld group and modifications on the current provisions are being investigated for skewed extended shear tab connections.

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