FULL SCALE TESTING OF EXTENDED BEAM-TO-COLUMN AND BEAM-TO-GIRDER SHEAR TAB CONNECTIONS...

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FULL SCALE TESTING OF EXTENDED BEAM-TO-COLUMN AND BEAM-TO-GIRDER SHEAR TAB CONNECTIONS SUBJECTED TO SHEAR

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Abstract. *This paper discusses the findings of an experimental program that was conducted to investigate the behaviour of extended shear tab beam-to-column and beam-to-girder connections that are commonly used in North America. The laboratory results showed that extended beam-to-column shear tab connections could reach resistance levels consistent with typical predicted failure modes and still maintain an adequate level of plastic rotation capacity. The beam-to-girder connection tests revealed that (a) plate local buckling for full-height connections; and (b) localized deformation of the supporting girder web and flange for partial-height connections should be explicitly considered as part of the design process of such connections.*

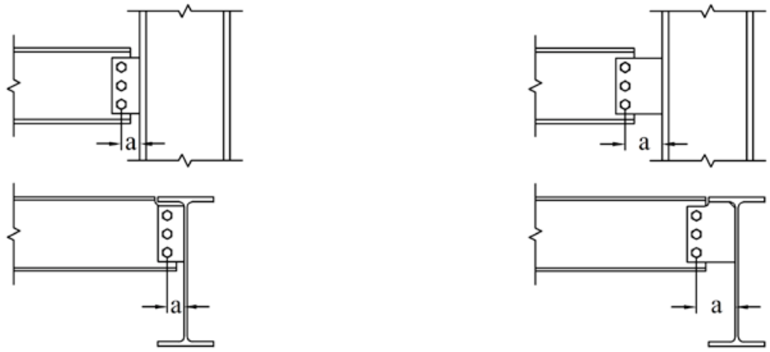
1 INTRODUCTION

Single plate shear connections, commonly referred to as shear tabs, are widely used in steel construction due to low cost, ease of fabrication and ease of installation. Part 10 “Single-Plate Connections” of the American Institute of Steel Construction (AISC) Steel Construction Manual [1], divides shear tab connections into two configurations. Conventional configurations have a single vertical row of two to 12 bolts and an “a” distance less than or equal to 89mm (3½in). The “a” distance is defined as the distance between the support face and the first vertical row of bolts (Figure 1). A shear tab connection is considered to be of an extended configuration if either the “a” distance exceeds 89mm (3½in), or if there are two or more vertical row of bolts. This research concerns itself with “extended” configurations. Figure 1 illustrates the difference in “a” distances between conventional and extended shear tab configurations.

The behaviour of conventional single-row shear tab connections is well understood based on earlier experimental work summarized in [2,3,4]. In particular, the work by Astaneh [4] established the design method seen in “Single-Plate Connections” of the AISC Manual. Shaw and Astaneh [5] found that the design method for single-plate shear tab connections can also be applied to beam-to-girder shear tab connections; this was later refined by Creech [6]. More recent work by [7,8,9] characterized the behaviour of double- and multi-row shear tab connections through full-scale experimental testing. This testing program demonstrated that multi-row shear tab connections can reach resistance levels consistent with typically predicted failure modes and still maintain a comparable plastic deformation capacity with single-row shear tab connections.

The effect of increasing the “a” distance on the behaviour of shear tab connections was first examined by Sherman and Ghorbanpoor [10]. These tests demonstrated that plate buckling should be considered as a limit state for the design of extended shear tab connections. In a more recent experimental program [11] the behaviour of extended shear tab beam-to-girder connections with multi-row of bolts was investigated. The reduction factors for the bolt threads intercepting the shear plane, as well as the uncertainty of the bolt group for bolt shear strength were refined.

This current paper summarizes the findings of an experimental program related to the behaviour of multi-row extended beam-to-column and beam-to-girder shear tab connections with emphasis on the latter. The effect of including stiffeners on the side of the girder opposite to the shear tab was also examined.



(a) Conventional Configuration (a≤89mm) (b) Extended Configuration (a > 89mm)

Figure 1. Conventional and extended shear tab configurations.

2 EXPERIMENTAL PROGRAM

Twelve extended beam-to-column and beam-to-girder shear tab connections were tested at full-scale in the Jamieson Structures Laboratory at McGill University in June 2013 [12]. The test specimens included four beam-to-column connection specimens (W310 beams – 2 vertical rows of bolts, W690 beams – 3 vertical rows of bolts) with “a” distances of 150 mm and 200 mm, as well as eight beam-to-girder connections (W310 to W610 beams; W610 to W760 girders; two vertical rows of bolts; various configurations and stiffening details). These configurations are shown schematically in Figure 2. The “a” distance for each configuration is summarized in Table 1 together with other geometric characteristics of the test specimens. The following steel grades were used; ASTM A992 345 MPa beams, girders and columns, ASTM A325 bolts, E49 electrodes (490 MPa) and ASTM A572 345 MPa plate.

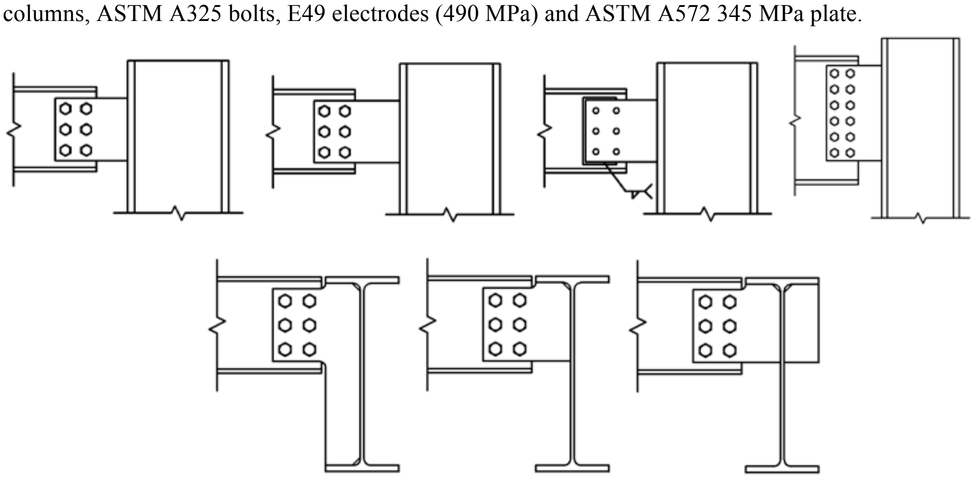


Figure 2. Extended shear tab connection configurations.

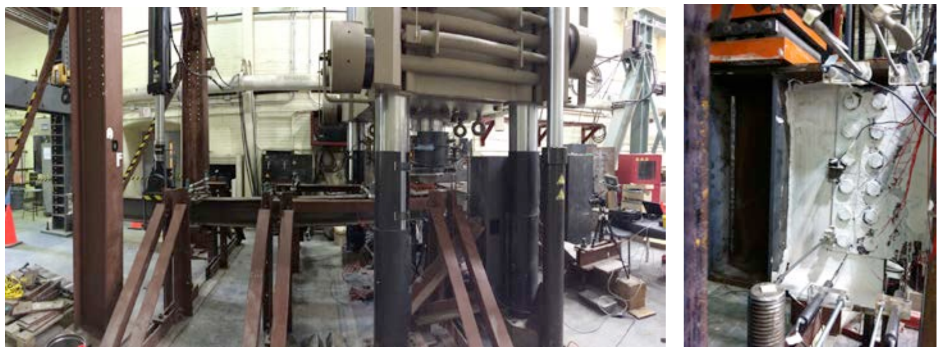
A summary of the test matrix is provided in Table 1 including the main design variables. Note that all 12 configurations had an “a” distance exceeding 89mm; thus they were considered as extended per [1]. Note that configuration 3 had a partial “C” weld replacing the bolts. This configuration was identical with configuration 1 and was included as part of the testing program to simulate the situation in which the shear tab cannot be bolted to the beam on site due to misalignment of bolt holes. This is a typical retrofit case that is preferred by structural engineering practitioners. The extended shear tab connections were designed primarily using Part 10 from the 14th Edition of the 2010 AISC manual [1]. However, for the limit states of block shear, bolt shear and bolt bearing, the design equations per CSA S16-09 [13] were employed for the aforementioned configurations. The supported beams and supporting girders were designed based on [13]. The latter were designed to resist the connection shear and torsion due to the connection shear at the eccentricity of the “a” distance. Detailed information regarding the design of the test specimens can be found in [12].

Table 1. Extended shear tab test specimen matrix

Beam Size	Column or Girder	“a” distance (mm)	# Bolts	Bolt Diameter (mm)	Comments
Extended beam-to-column shear tab connections					
W310x74	W360x196	152	2 x 3	19	-
W310x74	W360x196	203	2 x 3	19	-
W310x74	W360x196	152	-	-	Partial "C" weld
W610x140	W360x196	152	2 x 6	22	-
Extended beam-to-girder shear tab connections					
W310x60	W610x125	165	2 x 3	19	Full height
W310x60	W610x125	165	2 x 3	19	Partial height
W310x60	W610x125	165	2 x 3	19	Partial height/stiffener
W310x60	W610x125	171	1 x 3	19	Side plate
W310x60	W760x257	241	2 x 3	19	Partial height
W310x60	W760x257	241	2 x 3	19	Partial height/stiffener
W610x140	W760x257	241	2 x 6	22	Full height
W610x140	W760x125	241	3 x 7	25	Full height

The test setup that was employed as part of the experimental program discussed above is shown in figure 3a. This setup consisted of a beam-to-column reaction frame (or beam-to-girder reaction frame) in which the respective shear tab was welded. The loading beam was then attached to the shear tab with “snug-tight” bolts. For configuration 3 the loaded beam was welded on the shear tab at the site to mimic the situation that such retrofit would take place at a construction site. The shear tab connection from configuration 1 is shown in figure 3b after the installation completion. The loaded beam is supported laterally at a maximum spacing of 1500mm with a lateral bracing system as shown in Figure 3a to prevent any out-of-plane deformations of the loaded beam.

Two actuators (Figure 3a) were employed in order to control the loading regime such that the expected yielding shear for the shear tab was reached at a connection rotation of 0.015rad. Afterwards, the connections were expected to undergo plastic deformation. The actuator near the extended shear tab connection was in compression. The actuator at the loaded beam tip was in tension. The connection shear force referred herein is the summation of the two-actuator forces. In addition, the connection rotation is the relative rotation of the loaded beam with respect to the respective column face or girder. The loading regime was based on prior experimental work [4,8]. Extensive details regarding the instrumentation as well as the experimental setup for full-scale testing of extended shear tab connections can be found in [12].



(a) experimental setup (b) extended beam-to-column shear tab

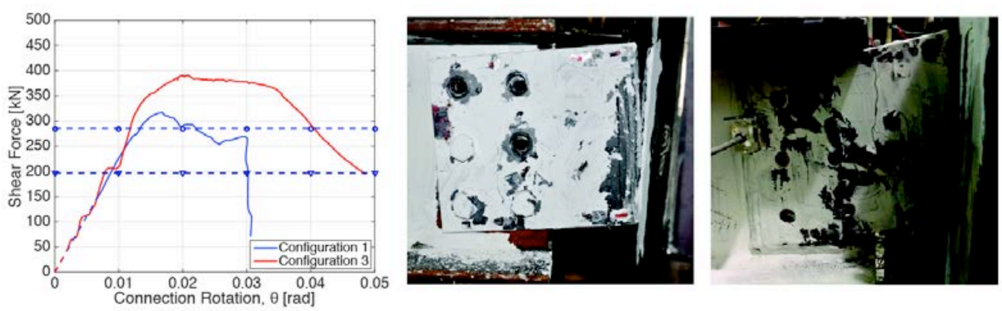
Figure 3. Experimental setup for full-scale testing of extended shear tab connections.

3 SUMMARY OF EXPERIMENTAL RESULTS AND DISCUSSION

This section discusses the major findings from the comprehensive experimental program discussed in Section 2 of this paper. Due to brevity part of the findings are summarized herein. However, a more thorough evaluation of the experimental results is presented by Hertz [12].

3.1 Extended beam-to-column shear tab connections

Figure 4a shows the connection shear resistance versus the connection rotation for Configurations 1 (i.e., beam-to-column bolted shear tab connection) and 3 (i.e., beam-to-column partially "C" welded shear tab connection). Note that the beam and column sizes in this case were nominally identical as shown in table 1. The observed behaviour was characterized by flexural yielding of the shear tab that was initiated at a connection shear resistance of 215kN and 135kN for the bolted and welded connection, respectively. Flexural yielding occurred earlier for the bolted connection due to the partial "C" weld having greater rigidity than the corresponding bolt group.



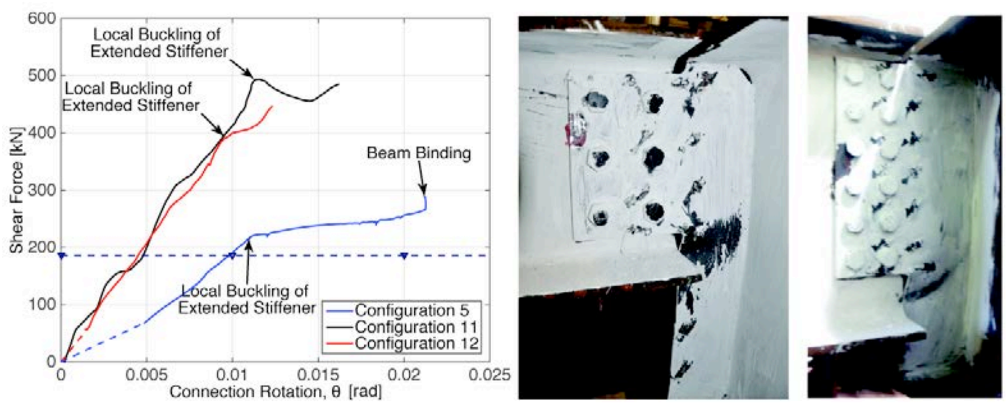
(a) Connection shear force - rotation (b) Configuration 1 (c) Configuration 3

Figure 4. Performance of beam-to-column shear tab connections; configurations 1 and 3.

The primary failure mode for both cases was weld tearing as shown in figures 4b and c for Configurations 1 and 3, respectively. The rapid connection shear force deterioration observed in the bolted connection at about 3% rad connection rotation was due to bolt shear. However, it should be noted that the shear plane intercepted the bolt threads; as such, if the bolts had consisted of a longer shank section this sudden failure could likely have been delayed. In addition, the bolt group eccentricity was assumed by design to be 190mm. However, the real eccentricity was calculated as 128mm using the measured bolt shear resistance. Furthermore, the AISC extended shear tab design method [1] specifies that the bolt shear resistance be calculated under the assumption that the connection rotation occurs with respect to the column face. However, the support does provide a partial moment restraint. This indicates that a more careful evaluation of the moment gradient within such connections should be conducted. This issue is currently under investigation by the authors. Based on figure 4a, the welded connection performance was fairly stable up to about 4% rads. Prior to weld tearing the shear tab combined shear and flexural yielding dominated the connection performance.

3.2 Extended beam-to-girder shear tab connections

Figure 5a illustrates the behaviour of full-height beam-to-girder shear tab connections in terms of connection shear force versus connection rotation. The primary failure mode in such connections was consistent regardless of the configuration type (i.e., deep versus shallow loading beam). In particular, local buckling of the extended connection stiffener dominated the connection behaviour. This can be seen in figures 5b and c for Configurations 5 and 11, respectively. The onset of local buckling is indicated in figure 5a per configuration type. Configuration 12 buckled at a lesser load than Configuration 11 even though the expected buckling resistance was thought to be higher. This could be attributed to the fact that the buckling of full height beam-to-girder shear tab connections is due to a combination of (a) vertical compressive stresses from transfer of shear into the girder; and (b) horizontal compressive stresses from the flexural action of the loading beam. Configuration 12 employed a W690 beam, which was deeper than that for configuration 11 (W610). Therefore, the horizontal stresses would be larger for the same shear load and thus buckling of the stiffeners would occur at a lower connection shear force. Another issue that should be explicitly considered as part of the design process of full-height beam-to-girder connections with girder stiffeners is the actual slenderness ratio (i.e., *b/t*) for the respective stiffener. In particular, it is recommended that this ratio meet the slenderness ratio requirements for plates per [1,13]. This issue is one of the objectives of a more recently completed experimental program by the last two authors.



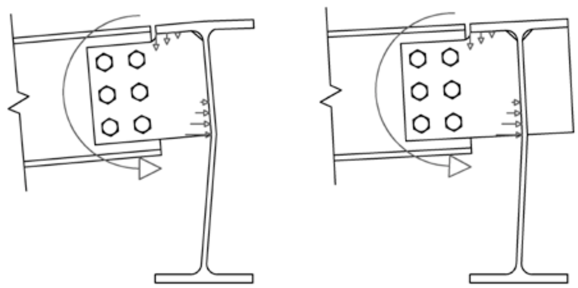
(a) Connection shear force - connection rotation (b) Configuration 5 (c) Configuration 11

Figure 5. Performance of full-height beam-to-girder shear tab connections

Despite the fact that local buckling occurred in the extended stiffener plate, the overall connection behaviour was fairly stable without any strength deterioration (see figure 5a). Eventually, the bottom flange of the loading beam began to bind on the stiffener portion of the shear tab regardless of the tested configuration. Beam binding is typically accompanied by an increase in the connection shear resistance and stiffness and could potentially lead to undesirable connection failure modes [14,15]. Therefore, a

25mm distance between the bottom flange of the loading beam and the stiffener is recommended for the design of such connections.

Configurations 6, 7, 9 and 10 were designed with partial height shear tabs (see table 1 and figure 2). Configurations 7 and 10 had additional stiffeners on the opposite side of the girder web. However, all four configurations were dominated by a girder web mechanism as shown in figure 6. The stress distribution on the shear tab of such connection is shown schematically in the same figure. The experimental program showed that when a partial height stiffener is welded on the other side of the shear tab, this generally decreases the deformations in the girder; however, a more effective solution would be to extend such stiffener throughout the web depth of the girder (i.e., bridge girder stiffeners).



(a) without stiffener (configuration 6), (b) with stiffener (configuration 7)

Figure 6. Girder web mechanism for partial height beam-to-girder shear tab connections.

Table 2 provides a summary of the measured maximum connection shear resistance and achieved connection rotation per test. In the same table a summary of the predicted resistances and failure modes per [1,13] is also provided. Note that the predicted resistances are based on the measured material properties from tensile coupon tests [12]. For beam-to-column shear tab connections (i.e., test IDs 1 to 4) there is a good agreement between the predicted and measured resistances.

Table 2. Summary of measured and predicted connection resistances and failure modes

Test ID	Maximum Connection Shear [kN]	Maximum Connection Rotation [rad]	Primary and Secondary Failure Mode*	Predicted Resistance [kN]	Predicted Failure Mode*
1	317	0.031	WT (BS)	197	BS
2	240	0.065	PB	161	BS
3	390	0.055	WT	285	CT
4	1040	0.033	SR (PB, FS)	922	FS
5	266	0.021	FB	186	BS
6	108	0.009	GY	186	BS
7	445	0.127	BB	186	BS
8	410	0.036	BB	178	BB
9	433	0.024	-	142	BS
10	501	0.024	-	142	BS
11	455	0.014	FB	732	FS
12	415	0.011	FB	933	FS

*Failure Mode Definitions: BS: bolt shear; CT: weld tearing (partial "C" weld); SR: shear rupture; BB: bolt bearing; FS: flexural and shear yielding; WT: weld tearing; PB: plate buckling; SR: shear rupture; FB: full height buckling; GY: girder yielding; BB: bolt bearing.

Table 2 also provides a summary of the observed primary failure modes per test (see column 3). In parenthesis, the secondary shear tab connection failure modes are tabulated. All of the beam-to-column shear tab tests were characterized by weld tearing of the shear tab plate-to-column welds. This was due to under-sizing of the welds. In particular, the design method per [1] specifies sizing the welds $5/8^{th}$ of the respective shear tab plate thickness. However, this ratio is based on prior experimental observations from [4]. In order to prevent or delay weld tearing it is suggested that this ratio be replaced by the Muir and Hewitt [16] design equation. In this case, the plate yield stress should be taken as 110% of the specified nominal value.

4 CONCLUSION

This paper summarizes an extensive experimental program that was conducted at full-scale to characterize the behaviour of extended beam-to-column and beam-to-girder shear tab connections subjected to direct shear. The connections were designed according to the extended shear tab design method per [1,13]. The main findings of the experimental program are summarized as follows:

- The beam-to-column tests showed good agreement between the predicted and measured resistances.
- The AISC (2010) design method specifies sizing the welds 5/8ths of the plate thickness. This ratio is based on experimental observation [4] as well as theory [16]. It is recommended that this ratio be replaced by the Muir and Hewitt design equation [16] with the plate yield stress taken as 110% of the nominal.
- The full-height beam-to-girder tests were characterized by plastic buckling of the stiffener portion of the shear tab. A design check is proposed taking into account the vertical stresses due to the connection shear and the horizontal stresses due to flexural action of the beam.
- The partial-height beam-to-girder shear tab tests revealed that girder web and flange deformation is significant when the top flange of the supporting girder is unrestrained. In these cases, including a stiffener opposite to the shear tab for flexible connections can reduce this deformation.

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