3rd Specialty Conference on Material Engineering & Applied Mechanics
3e Conférence spécialisée sur le génie des matériaux et mécanique appliquée

Montréal, Québec
May 29 to June 1, 2013 / *29 mai au 1 juin 2013*

EXTENDED SHEAR TAB CONNECTIONS UNDER COMBINED AXIAL AND SHEAR LOADING

Kristin Thomas[1], Robert G. Driver[1], Steven A. Oosterhof[1]
[1] Department of Civil and Environmental Engineering, University of Alberta, Canada

Abstract: Extended shear tabs, or single-plate shear connections, are a cost-effective solution when framing a beam into a column or girder web. By using this type of shear connection, the need for coping the beam is avoided and only bolt installation is required on-site, thereby reducing fabrication and erection costs. However, due to the eccentricity between the bolts at the beam and the weld at the column or girder, the stability of these plates is a potential concern. This issue is compounded when the connection is also subjected to axial load. An investigation into the behaviour of extended shear tab connections was conducted by testing 23 full-scale connections. The specimens were tested by rotating the beam to 0.03 radians, applying a horizontal load, and then applying vertical load until failure. The horizontal loads varied from 500 kN in compression to 200 kN in tension. In addition to varying the magnitude and direction of the applied horizontal load, the specimens differed in four geometric characteristics: number of bolt rows, plate depth, plate thickness, and use of stiffeners. The length of the plate, size of the column, diameter of the bolts, and weld size were kept constant and are representative of those used in typical structures. The failure modes of the connections are described and the influences of the key variables on behaviour and performance are discussed.

1. INTRODUCTION

An extended shear tab (EST) is a type of simple connection commonly used in steel building construction, wherein a plate is welded in the vertical orientation to a column or girder, and bolted to the supported beam. ESTs have the same configuration as conventional shear tabs, but normally frame into the supporting member's web and extend beyond its flanges. This creates a much larger distance between the bolt group and weld, resulting in a load eccentricity that must be accounted for explicitly in design. The recommended limit on eccentricity for conventional shear tabs is typically between 75 mm, according to the Handbook of Steel Construction (CISC, 2010), and 89 mm, according to the Steel Construction Manual (AISC, 2011). ESTs can either be unstiffened, like conventional shear tabs, or stiffened by welding the plate either to perpendicular stiffeners located between the flanges of a column or directly to the flanges of a girder. Examples of unstiffened and stiffened ESTs at a column are shown schematically in Figure 1.

ESTs are advantageous when framing a beam into the web of either a column or girder, as the need for expensive coping of the beam is eliminated and the beam can be easily positioned during construction. Despite their relatively common use, no comprehensive limit states design procedure has been widely accepted because the behaviour of ESTs is not well understood. As such, elastic procedures are commonly adopted for expediency, leading to excessively conservative connections.

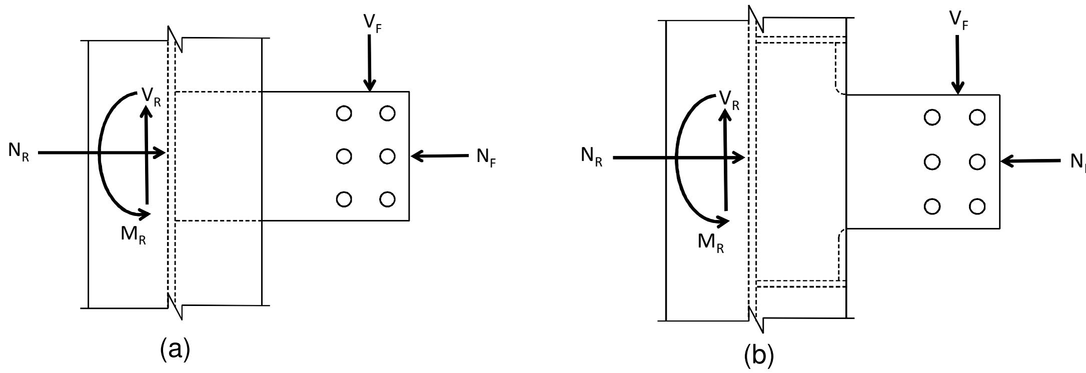


Figure 1: Combined loading on (a) unstiffened and (b) stiffened extended shear tabs

Because the EST plates are slender, one main consideration for design is the stability of the connection. Plate buckling becomes an even larger concern when the connection is loaded under both shear and compression. However, the manner in which shear, axial load, and moment interact on the plate is complex, particularly after significant inelastic action has taken place.

2. PAST EXPERIMENTAL WORK

2.1 Unstiffened Extended Shear Tabs

There have been 23 unstiffened EST tests reported in the literature; of these, 21 were beam-to-column connections and two were beam-to-girder. A summary of the geometries, peak loads, and failure modes for the eleven beam-to-column EST connections tested by Moore and Owens (1992), six by Sherman and Ghorbanpoor (2002), and four by Metzger and Murray (2006) is shown in Table 1. Moore and Owens tested beams with a shear tab connection, of either extended or conventional configuration, at each end. If these connections had the same EST geometry and results, only one is reported in the table. Also, in some cases when two different geometries were used, only one of the connections failed. The “a” distance is the horizontal dimension from the centre of the bolt group to the support. None of these tests included axial loading.

The possible failure modes of the unstiffened ESTs are similar to those typically observed in conventional shear tabs and include bolt shear, bolt bearing, shear rupture, shear yield, and weld fracture. As shown in Table 1, Moore and Owens (1992) and Sherman and Ghorbanpoor (2002) identified twisting of the plate as another possible failure mode. Sherman and Ghorbanpoor also identified column web yielding as a failure mode. Column web yielding was observed but not listed as a failure mode by Moore and Owens when the EST was welded to the column web in tests F1/1 and F1/2. Metzger and Murray (2006) did not include column web yielding as a failure mode because, in all tests, the ESTs were welded to the column flange rather than the web. Plate buckling was not reported for any of these tests.

2.2 Stiffened Extended Shear Tabs

Sherman and Ghorbanpoor (2002) also conducted 23 stiffened EST tests. Of these 23 tests, 14 were beam-to-column connections; the remaining were beam-to-girder connections. Another six beam-to-column EST tests were completed by Goodrich and Basu (2005). The beam-to-column EST test results from both testing programs are listed in Table 2. In all cases, the specimens had only one column of bolts. The Goodrich and Basu group of tests included three different configurations and each configuration was tested twice. The two test results for each configuration were reported as being almost identical in all three cases, and only one capacity was given. Therefore, only three test results from this program are listed in the table. The “a” distance for stiffened tests was defined for both testing programs as the distance from the centre of gravity of the weld group to the bolt line.

Table 1: Results from previous unstiffened EST tests

Researcher	Test I.D.	Bolt Columns	Bolt Rows	Thickness (mm)	"a" Distance (mm)	Peak Shear (kN)	Primary Failure Mode*
Moore and Owens (1992)	S3**	1	4	8	180	212***	T
	S4-1	1	3	8	180	275***	T
	S4-2	1	4	8	180	275***	T
	F1/1-1	1	4	8	180	180***	DNF
	F1/2-2	1	4	8	180	100***	DNF
	S7**	1	5	10	190	380***	DNF
	S8**	1	7	10	190	675***	T
	F2-1	1	7	10	190	550***	DNF
Sherman and Ghorbanpoor (2002)	3-U	1	3	9.5	174	244***	WY
	3-UM	1	3	9.5	174	261	WY
	4-U	1	5	12.7	255	439***	T
	6-U	1	6	12.7	255	614	WY, BF
	6-UB	1	6	12.7	255	604***	WY, BF
	8-U	1	8	12.7	255	772	WY
Metzger and Murray (2006)	5b	2	3	12.7	114	391	WR
	6	2	5	12.7	114	890	WR
	7	1	7	9.5	229	431	BM
	8	2	5	12.7	267	431	BM

*WY = Column Web Yielding, BF = Bolt Fracture, WR = Weld Rupture, BM = Beam Failure, T = Twisting
DNF = Did Not Fail
**Same connection on both ends of beam
***Beam not braced during test

The failure modes for the stiffened ESTs are similar to those of the unstiffened ones. However, the stiffeners reduced the potential for column web yielding. As shown in Table 2, Sherman and Ghorbanpoor (2002) noted shear yielding and twisting as the most common failure modes, whereas plate buckling was the primary failure mode in all tests by Goodrich and Basu (2005).

3. SCOPE AND OBJECTIVES

A total of 23 full-scale beam-to-column EST connections were tested as part of this research project. The purpose of these tests is to determine the behaviour of ESTs under not only vertical loading alone, but also combined vertical and horizontal loading. To describe connection behaviour, two principal objectives were identified. The first was to determine the failure modes of ESTs under both loading scenarios and the second was to determine how key variables influence the stability, strength, and ductility of the plate. These variables include the use of stiffeners, horizontal load, plate depth, number of bolt rows, and the thickness of the plate.

Table 2: Results from previous stiffened EST tests

Researcher	Test I.D.	Hole Type*	Bolt Rows	Thickness (mm)	“a” Distance (mm)	Peak Shear (kN)	Primary Failure Mode**
Sherman and Ghorbanpoor (2002)	3-A	STD	3	6.35	150	237***	SY, T
	3-B	SSL	3	6.35	150	236***	SY, T
	3-C	STD	3	6.35	150	98***	SY, T
	3-D	STD	3	6.35	150	227***	SY, T
	3-E	STD	3	6.35	158	214***	SY, T
	3-F	SSL	3	6.35	150	304	SY, BB
	3-G	SSL	3	6.35	150	290	SY, BB
	3-H	SSL	3	6.35	150	302	SY, WY
	4-A	STD	5	6.35	210	458***	SY, T
	4-B	SSL	5	6.35	210	476	SY, T
	4-C	STD	5	6.35	210	476	SY, T
	6-B	SSL	6	7.94	220	554	T, BB
	8-A	STD	8	7.94	227	873	BB
	8-B	SSL	8	7.94	227	1012	T
Goodrich and Basu (2005)	1-1	STD	4	9.52	193	400***	PB
	2-1	STD	3	6.35	176	294***	PB
	3-1	STD	3	12.7	185	454***	PB

*STD = Standard Bolt Holes, SSL = Short Slotted Bolt Holes
**SY = Shear Yielding, T = Twisting, BB = Bolt Bearing, WY = Column Web Yielding, PB = Plate Buckling
***Beam not braced during test

4. EXPERIMENTAL DESIGN

4.1 Set-up and Testing Procedure

The test set-up consisted of an interchangeable column stub, a beam, and three actuators (two vertical and one horizontal), as shown in Figure 2. The actuators had pinned ends, allowing any combination of horizontal load and rotation to be applied and maintained while increasing the vertical load. Each EST was welded to the web of the column stub. Before testing, the specimen was bolted to seats at the top and bottom of the column stub and then to the beam. The beam was bolted to the three actuators, and low-friction slide plates were provided as lateral bracing for the beam near the connection. All bolts were pretensioned, other than those connecting the EST to the beam, which were snug-tight.

A load cell and clinometer were mounted on each actuator and were used to calculate the total vertical and horizontal loads applied to the connection. Three additional clinometers were used to monitor beam rotation, connection plate rotation, and column stub rotation. Cable transducers were installed on each actuator as well as on the beam web to allow deformations and moment arms to be determined for each point load. Two linear variable differential transformers were mounted on the column stub: one to measure its global horizontal movement and one to measure the local deformation of the web.

The specimens were tested by first rotating the beam counter-clockwise (Figure 2) to 0.03 radians and then applying the specified horizontal load for that test. This rotation and horizontal load were kept constant as upward vertical load was applied to the connection until failure, which was defined as either a substantial decrease in the applied vertical load from the peak value or bolt fracture.

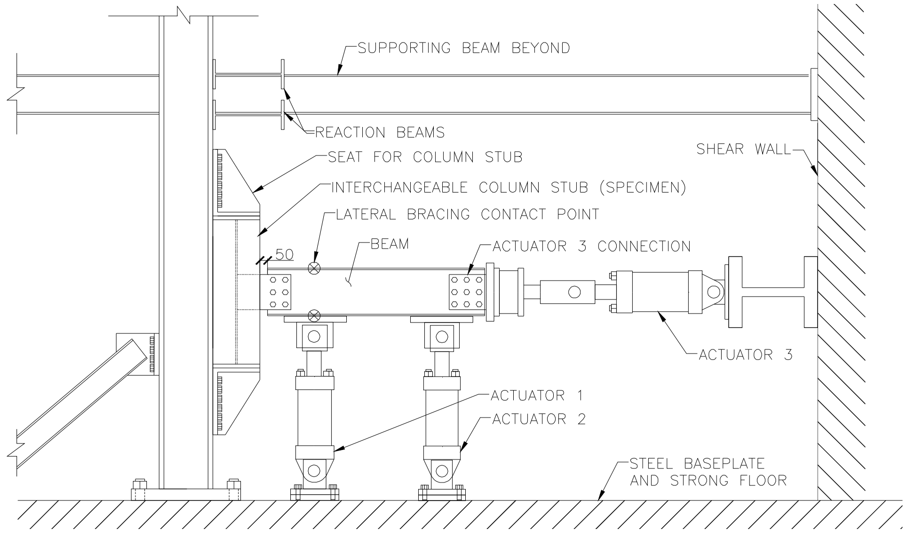


Figure 2: Test set-up

4.2 Specimen Design

The EST specimens were designed based on current procedures used at a local steel fabrication shop. In total, six geometries were designed, as shown in Figure 3.

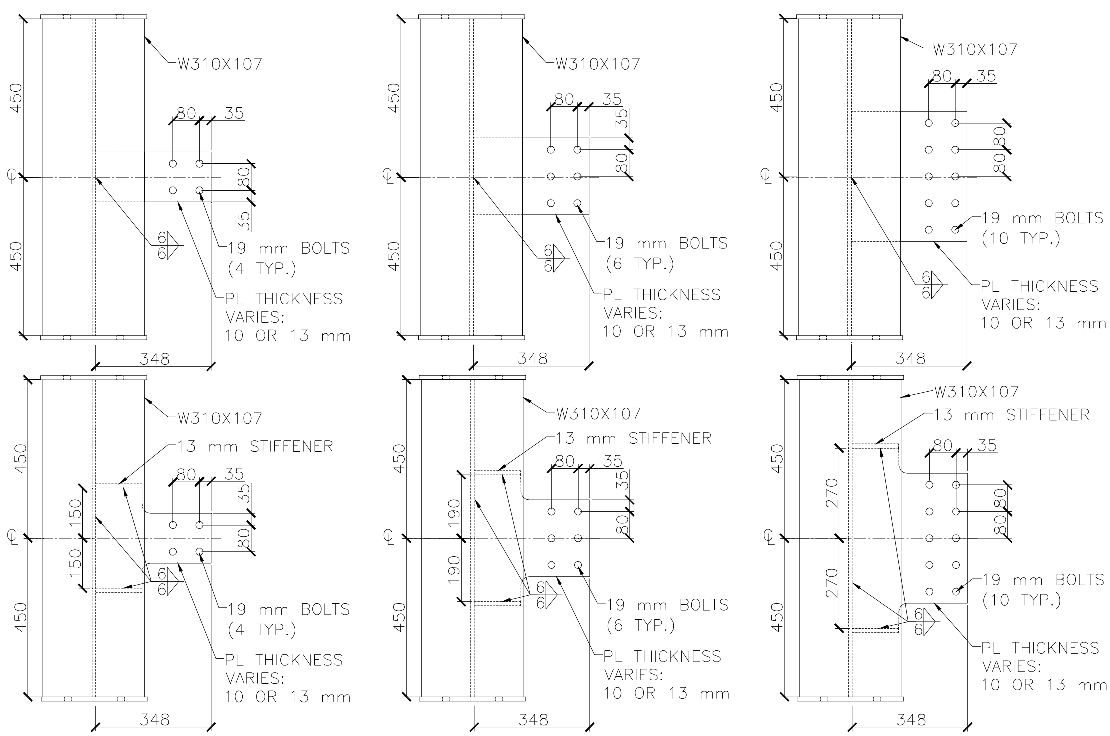


Figure 3: Specimen geometries

The specimens were separated into two main groups: unstiffened and stiffened. They were further subdivided based on the number of bolt rows; ESTs with two, three, and five bolt rows were tested. In addition to the use of stiffeners and number of bolts, the specimens can be described based on the plate's thickness, either 9.5 mm or 12.7 mm, hereafter referred to as 10 mm and 13 mm, respectively, for expediency. Each specimen was also assigned a horizontal load. These loads varied from 200 kN in tension to 500 kN in compression and some specimens were tested with a 0 kN horizontal load. One two-bolt-row, 10 mm, unstiffened specimen was also tested with no beam rotation to confirm that this would not constitute a more critical condition.

To determine how the variables described above influence the behaviour of ESTs, typical sizes and dimensions were chosen for the remaining parameters. These constants include a W310x107 column stub, a 348 mm long plate, 19 mm diameter bolts, two columns of bolts, a 6 mm fillet weld, and a constant stiffener configuration, as shown in Figure 3. A 14 mm beam web thickness was used to preclude web failure.

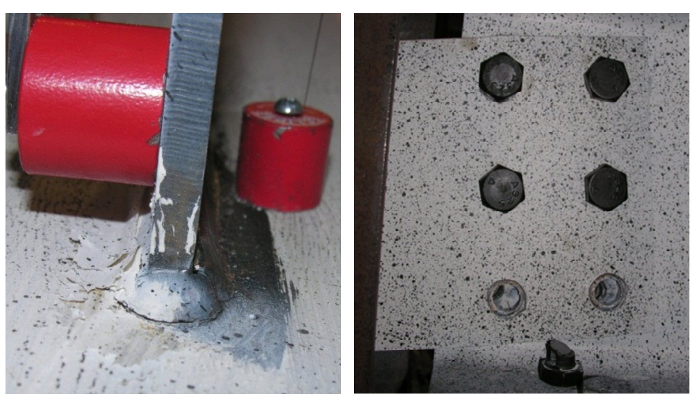
5. RESULTS

The results from this testing program are summarized in Table 3. In this table, details for each of the 23 tests are given based on the specimen's I.D., which consists of the number of bolt rows followed by the plate thickness in millimetres. If the specimen was stiffened, an S is present after the plate thickness. The vertical load reported in the table is the maximum value recorded during testing. Although the capacity of actuator 1 was reached during three of the tests (denoted in Table 3 by ***), the maximum vertical load recorded is still considered representative of the connection capacity. No horizontal load was to be applied in the first specimen 5B-10 test; however, to fail the specimen, tension was applied and the beam rotation permitted to increase until failure occurred at 146 kN. For the two stiffened specimens where the actuator capacity was reached, the vertical load versus deformation curve had reached an extended plateau, suggesting that the capacity of the connection had been approached.

5.1 Typical Failure Modes of Unstiffened Specimens

The unstiffened ESTs failed in essentially the same manner for all but two tests. The vertical load was increased until the tension side of the weld connecting the plate to the column web fractured. At this time, the connection had reached its peak load and the capacity then began to drop until bolt failure occurred. Also, the web of the column stub typically yielded while testing, resulting in significant plastic deformations. The typical failure modes are shown in Figure 4.

There were two tests that did not fail as described above. The first was specimen 3B-10 tested under 200 kN tension. The relatively high tensile force applied to this connection caused the entire weld to fracture, while inducing very little plate or bolt deformation. The second uncharacteristic test was that of specimen 5B-10 tested under 146 kN tension, which did not experience weld fracture. As mentioned previously, the loading regime for this specimen was atypical because the capacity of actuator 1 was reached.



(a) (b)

Figure 4: Typical unstiffened EST failure modes (a) fracture of weld on tension side, and (b) shear of bolts in outer row(s)

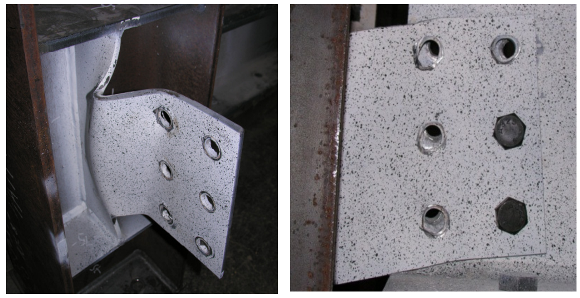
Table 3: Peak loads and failure modes

Specimen I.D.	Axial Load Direction	Horizontal Load (kN)	Peak Vertical Load (kN)	Observed Failure Modes*
2B-10	–	0	188	WY, BB, BF, WR
2B-10**	–	0	197	WY, BB, BF, WR
2B-10	Compression	200	159	WY, BB, BF, WR
2B-13	Compression	200	138	WY, BF, WR
2B-10-S	–	0	317	PR, BB, PB, BF
2B-10-S	Compression	200	258	BB, PB, BF
2B-13-S	Compression	200	323	BB, PB, BF
3B-10	–	0	330	WY, BB, BF, WR
3B-10	Compression	200	339	WY, BB, BF, WR
3B-10	Compression	300	278	WY, BB, BF, WR
3B-10	Tension	200	270	WY, WR
3B-13	Compression	200	263	WY, PR, BF, WR
3B-10-S	–	0	511	PR, BB, PB, BF, T
3B-10-S	Compression	200	382	BB, PB
3B-10-S	Compression	300	279	BB, PB, T
3B-13-S	Compression	200	562	PB, BF, T
5B-10	Tension	146	762***	WY, BB, WT
5B-10	Compression	300	732	WY, BB, PB, BF, WR, T
5B-10	Tension	200	612	WY, BB, BF, WR
5B-13	Compression	300	613	WY, BF, WR, T
5B-10-S	Compression	300	798***	BB, PB, T
5B-10-S	Compression	400	586	BB, PB
5B-13-S	Compression	500	861***	WY, PB, BF

*WY = Column Web Yielding, PR = Plate Rupture, BB = Bolt Bearing, PB = Plate Buckling, BF = Bolt Fracture, WR = Weld Rupture, T = Twisting, WT = Column Web Tearing
**Tested at 0 radians
***Capacity of actuator 1 reached prior to failure

5.2 Typical Failure Modes of Stiffened Specimens

The stiffened specimens had different failure modes from the unstiffened specimens with similar geometry, as shown in Table 3. Significant out-of-plane buckling was observed during all stiffened EST tests, as shown in Figure 5(a) for specimen 2B-10-S under 200 kN of compression. After significant deformation of the plate had occurred, bolt failure often followed, as shown in Figure 5(b).



(a) (b)

Figure 5: Typical stiffened EST failure modes (a) plate buckling, and (b) failure of first column of bolts

6. INFLUENCE OF KEY VARIABLES

6.1 Use of Stiffeners

Using stiffeners increased the strength of the ESTs with geometry that was otherwise identical. For example, the 2B-10-S connection tested without a horizontal load had an increase in peak vertical load of 129 kN, or 69%, when compared to specimen 2B-10. With the addition of stiffeners, the plate-to-column-web connection becomes much more stable and the possibility of column web yielding or weld fracture occurring—two commonly observed failure modes for unstiffened EST tests—is greatly reduced.

The stability of ESTs is strongly related to whether or not stiffeners are used. Significant out-of-plane deformation was not observed during the unstiffened EST tests in this program or those discussed in Section 2. In contrast, plate buckling contributed to the failure of all stiffened specimens during this testing program and that conducted by Goodrich and Basu (2005). Sherman and Ghorbanpoor (2002) also noted out-of-plane deformation but attributed it to shear yielding and twisting. Although the use of stiffeners decreases the effective length of the plate in the axial direction, the increased shear capacity and rotational restraint introduced by the stiffeners led to the development of higher stresses on the compressive side of the plate, which caused plate buckling to become a more prominent failure mode.

6.2 Horizontal Load

Figure 6 shows the results of the 3B-10 and 3B-10-S tests, as well as the results from the 2B-10 and 2B-10-S tests. A negative horizontal load indicates the specimen was subjected to compression. As this figure shows, adding horizontal load tended to reduce the shear capacity of the connections. This reduction is more pronounced when the EST is stiffened and is more severe for the larger, three-bolt-row connections.

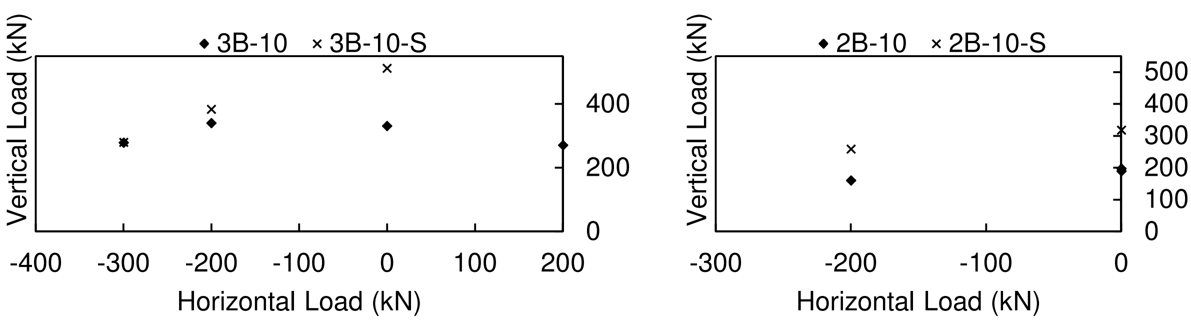


Figure 6: Effect of horizontal load on two- and three-bolt-row tests

The capacities of the unstiffened connections shown in Figure 6 were reduced due to the addition of horizontal load by a maximum of 29 kN (15%) and 60 kN (18%) for the two- and three-bolt-row connections, respectively. In the case of the 3B-10 specimens, the addition of 200 kN of compression actually increased the shear capacity of the connection by 9 kN, but when 300 kN was applied the capacity was reduced. This result indicates that there may be a limiting value of compression below which the shear capacity of the connection is not affected. In contrast, the addition of tension caused the largest decrease in peak vertical load because more demand was placed on the tension side of the weld.

Peak vertical load values reached by the stiffened ESTs shown in Figure 6 decreased significantly with the addition of compression. With the application of 200 kN of compression, the capacity of the two-bolt-row connection decreased by 59 kN (19%), and that of the three-bolt-row connection by 129 kN (25%). When tested with a horizontal compression of 300 kN, the extra strength expected due to the addition of stiffeners was no longer present for the three-bolt-row specimens. Because the stiffened specimens typically exhibited buckling failure, applying axial compression accelerated the out-of-plane movement and decreased the shear capacity of the connections.

6.3 Depth of Plate

As the number of bolt rows and, therefore, the plate depth increased, the strength of the connection also increased, as expected. Figure 7 shows the vertical load versus displacement response of the two- and three-bolt-row connections with the thinner plates tested without horizontal load. The increase in strength is common for both unstiffened and stiffened specimens. However, the deeper stiffened ESTs also showed an increase in ductility.

While buckling, the stiffened EST plate moved away from the beam web, causing the bolts to be loaded in combined shear and tension. As shown in Figure 7, the shear carried by the 2B-10-S connection dropped more quickly than that carried by the 3B-10-S specimen. This is because the bolts failed in the smaller connection, whereas, because more bolts were used, the deeper plate sustained larger deformations after buckling began.

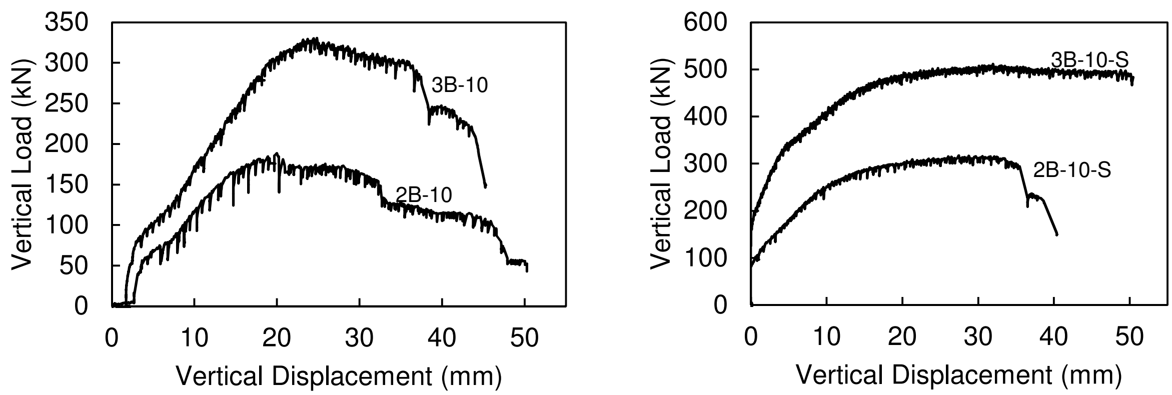


Figure 7: Effect of plate depth on behaviour of connections tested without horizontal load

6.4 Thickness of Plate

The plate thickness affected the behaviour of ESTs differently, depending on whether or not stiffeners were used. For unstiffened connections, increasing the thickness of the plate caused a decrease in the peak vertical load observed during testing, as shown in Figure 8. Although these results may seem counter-intuitive, by thickening the plate the connection's stiffness also increased and the plate deformation was reduced, forcing more deformation into the column web. Because the deformation of the column web contributed to the stress at the tips of the weld, the location of failure for these connections, weld fracture occurred at lower loads when the 13 mm plate was used in lieu of the 10 mm plate.

Stiffened ESTs obtained higher peak loads when the thicker plate was used, as shown in Figure 8. The higher load was achieved because the thicker connection was less susceptible to buckling—a common failure mode observed during the stiffened EST tests. Not only did these connections support higher loads, they also had more brittle failures. Because more force was required to deform the thick plates, the out-of-plane movement exposed the bolts to higher tensile forces and caused a more sudden failure.

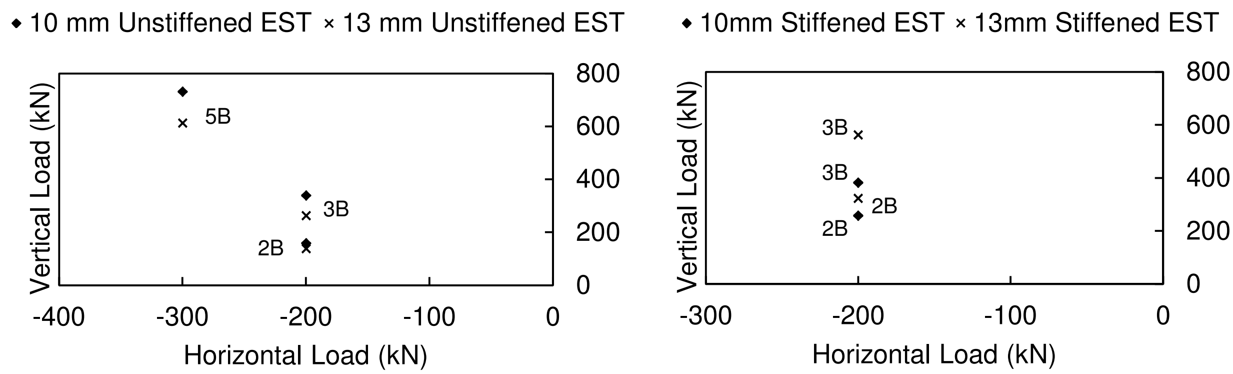


Figure 8: Effect of plate thickness on behaviour of connections tested under compression

7. SUMMARY AND CONCLUSIONS

Extended shear tabs are a practical solution for framing a beam into the web of a column or girder; however, no widely-accepted economical design procedure for ESTs currently exists. To help better understand their behaviour under a variety of loading conditions, 23 full-scale EST connection tests were carried out, with 13 on unstiffened ESTs and 10 on stiffened ESTs.

The influences of several key variables on the behaviour of ESTs have been investigated and the following preliminary conclusions can be drawn. The capacity of an EST connection can, in general, be increased by using stiffeners. If stiffeners are not used, the capacity can be increased by using a deeper plate with more bolts. However, increasing the plate thickness does not necessarily strengthen unstiffened ESTs. When stiffeners are used, additional shear capacity can be added by increasing the plate depth and number of bolts, or the plate thickness. The reduction in shear strength of stiffened ESTs was observed to be more rapid with the addition of horizontal load when compared to the unstiffened configuration, whose capacity may not decrease at all under small horizontal compressive loads.

8. ACKNOWLEDGEMENTS

The authors thank Waiward Steel Fabricators for donating the test specimens and Logan Callele for valuable contributions in the development of the testing program. Financial support for the research was provided by the Natural Sciences and Engineering Research Council, the Steel Structures Education Foundation, and the University of Alberta.

REFERENCES

American Institute for Steel Construction. 2011. *Steel Construction Manual.* 13th ed., AISC, Chicago, Il, USA.

Canadian Institute of Steel Construction. 2010. *Handbook of Steel Construction*. 10th ed., CISC, Markham, ON, Canada.

Goodrich, W. and Basu, P.K. 2005. *Behavior of Extended Shear Tabs in Stiffened Beam-to-Column Web Connections*. Master's Thesis, Vanderbilt University, Nashville, TN, USA.

Metzger, K.A.B. and Murray, T.M. 2006. *Experimental Verification of a New Single Plate Shear Connection Design Model*. Master's Thesis, Virginia Polytechnic Institute and State University, Blacksburg, VA, USA.

Moore D.B. and Owens, G.W. 1992. Verification of Design Methods for Finplate Connections. *The Structural Engineer*, 70(3):46-53

Sherman, D. and Ghorbanpoor, A. 2002. Design of Extended Shear Tabs. American Institute of Steel Construction Final Report, University of Wisconsin-Milwaukee, Milwaukee, WI.