

PERFORMANCE OF WELD-RETROFIT BEAM-TO-COLUMN SHEAR TAB CONNECTIONS

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ABSTRACT

Single plate shear tab connections are a simple means to connect beams to their supporting members. The tab is fillet-welded to the supporting column in the fabrication shop, and then field bolted to the beam web on the construction site. Situations may arise on site in which the bolt holes do not align. Instead of reaming holes or refabricating the connection, a cost efficient alternative is to use a weld-retrofit connection between the beam web and the shear tab. The AISC recommends that this retrofit approach not be taken due to concerns with the rotational ductility of the connection; however, no test-based performance evaluation was available for the weld-retrofit connection. The intent of the paper is to summarize the laboratory phase of a research project in which various weld-retrofit schemes of shear tab connections were evaluated. The connection performance is compared with nominally identical bolted shear tab connections.

1. INTRODUCTION

Single plate steel shear tab connections are commonly used to connect beams to their supporting column members. The tab is fillet-welded to the column in the fabrication shop; the beam web is then field bolted to the shear tab on the construction site. It is possible that during the steel erection process the bolt holes in the beam web and shear tab do not align (Figure 1) due to detailing or fabrication errors, as well as construction miss-alignment. Instead of reaming holes or refabricating the connection, a cost efficient and expeditious alternative is to use a weld-retrofit connection between the beam web and the shear tab. The American Institute of Steel Construction (AISC) does not recommend that this retrofit approach be applied due to concerns with the rotational ductility of the connection; however, no test-based evidence was available that demonstrated the response to loading of this weld-retrofit connection. Hence, a research project was initiated in which various weld-retrofit schemes of shear tab connections were evaluated by means of full-scale laboratory

testing. The connection performance in terms of overall behaviour, shear resistance and rotation capacity is compared with nominally identical bolted shear tab connections. The scope of the study comprised 13 beam-to-column connection specimens of various size W-shape members, original bolt configurations, and weld-retrofits (Figure 2). This was complemented by 7 matching tests of the original bolted shear tabs (Figure 2). Two of the weld-retrofit specimens consisted of replacement tabs having only two bolt holes to aid in erection; the remaining, contained the full allotment of bolt holes. All beams and columns were of ASTM A992 Grade 50 material, while the tabs were of ASTM A572 Grade 50 steel. The weld retrofit was done in the laboratory after the column and beam had been installed in the test frame; a certified welder with extensive experience in the steel fabrication industry completed the fillet welds of various patterns. Shielded metal arc welding (SMAW) with E49 (E70) stick electrodes was used, as would commonly be done on a construction site. In contrast, E71T flux core electrodes were used for all shop fabricated tab-to-column flange welds with an additional CO₂ shielding gas. This paper describes the design of the retrofit fillet welds and the testing program, as well as the observed and measured performance. Note that this research addresses the expected demands on a shear tab connection under the regular gravity loading scenario; it did not take into account the higher rotational demands on shear tab connections resulting from progressive collapse, i.e. loss of a column, nor the demands that might occur during a maximum considered seismic event for a pin connected structure.



Figure 1: Misaligned bolted shear tab connection requiring retrofit (courtesy of DPHV)

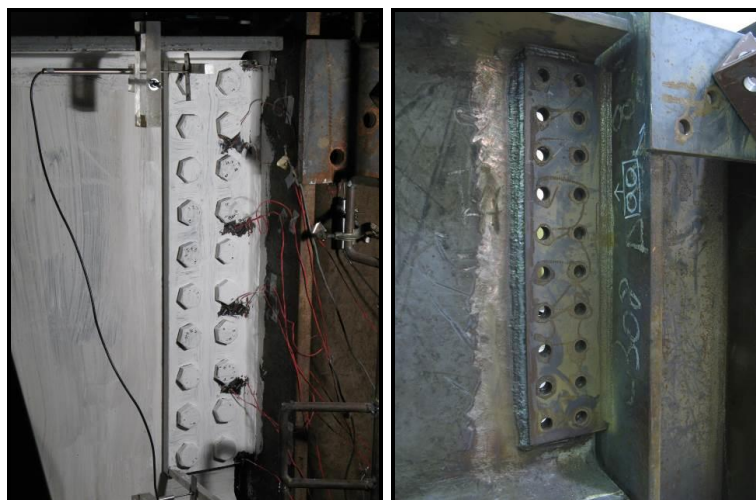


Figure 2: Typical bolted shear tab connection with matching weld-retrofit detail

1.1 Background Information on the North American Shear Tab Design Method

The design procedures for conventional and extended bolted beam-to-column shear tab connections in North America are best documented in the 14th Edition of the AISC Steel Construction Manual (2010) and best described for extended configurations by Muir and Hewitt (2009). Extensive testing of bolted shear tab connections by a variety of researchers dates back to the work of Lipson (1968); a detailed review of the relevant literature is found in the thesis of Creech (2005). Numerical evaluations of shear tab performance have also been carried out by various researchers (Sherman & Ghorbanpoor 2002). Connections are designed for block shear rupture, bolt bearing, bolt shear, shear yielding of the plate and shear rupture of the plate. For extended shear tab connections, the eccentricity of the bolt group is considered in determining the demand on the fasteners. The maximum allowable thickness of the plate is calculated to ensure ductility in the connection. The flexural yielding strength of the plate must also be checked, in addition to plate buckling.

2. WELD-RETROFIT SHEAR TAB TEST PROGRAM

2.1 Overview

The weld-retrofit shear tab test program involved 7 bolted connections and 13 weld-retrofit connections (Table 1, Figure 3). Each of the original bolted shear tab connections was initially designed by Marosi (2011), Marosi et al. (2011), Hertz (2013) and Hertz et al. (2015) following the method documented in the 14th Edition of the AISC Steel Construction Manual (2010), and then tested under gravity loading accounting for shear and rotational demands. The calculated factored resistance of these connections was taken as the starting point for the design of the weld detail for the retrofitted connections. For example, the resistance of the full “C” weld for configuration 1 was designed to have the same factored resistance as the corresponding bolted connection.

Table 1: Summary of weld-retrofit shear tab test specimens

Config.	Beam	Column	Shear Tab Thickness (mm)	Original Bolted Connection			Weld Retrofit	
				Bolt ^{a,b} Config.	Bolt ^a Size (mm)	Weld ^c Size (mm)	Weld ^d Config.	Weld ^d Size (mm)
1	W310x60	W360x196	6	1x3	19	6	Full C	4.8
2	W310x60	W360x196	6	1x3	19	6	Partial C	6.35
3	W310x60	W360x196	10	2x3	19	6	Full C	6.35
4	W310x60	W360x196	10	2x3	19	6	Partial C	7.9
5	W610x140	W360x196	8	1x6	22.2	6	Partial C	7.9
6	W610x140	W360x196	8	1x6	22.2	6	L	7.9
7	W610x140	W360x196	16	2x6	22.2	10	Partial C	11.1
8	W610x140	W360x196	16	2x6	22.2	10	L	14.3
9	W920x223	W360x196	10	1x10	25.4	6	Partial C	7.9
10	W920x223	W360x196	22	2x10	25.4	14	Partial C	15.9
11	W310x74	W360x196	9.5	2x3	19	6.35	Partial C	9.5
12	W610x140	W360x196	8	Temp. ^e	22.2	6	Partial C	6
13	W610x140	W360x196	16	Temp. ^e	22.2	11	Partial C	11

^aConfiguration & bolt size for the original bolted connection; threads excluded. Bolt holes 1/16” (2mm) larger. ^b#vertical rows x #bolts per row. ^cWeld from shear tab to column flange (weld both sides of plate). ^dRetrofit weld shape from beam web to shear tab. ^eHoles were provided only for two temporary installation bolts. ^fThe ‘a’ distance for all configurations (Figure 3) was 76 mm, except #11 (152mm).

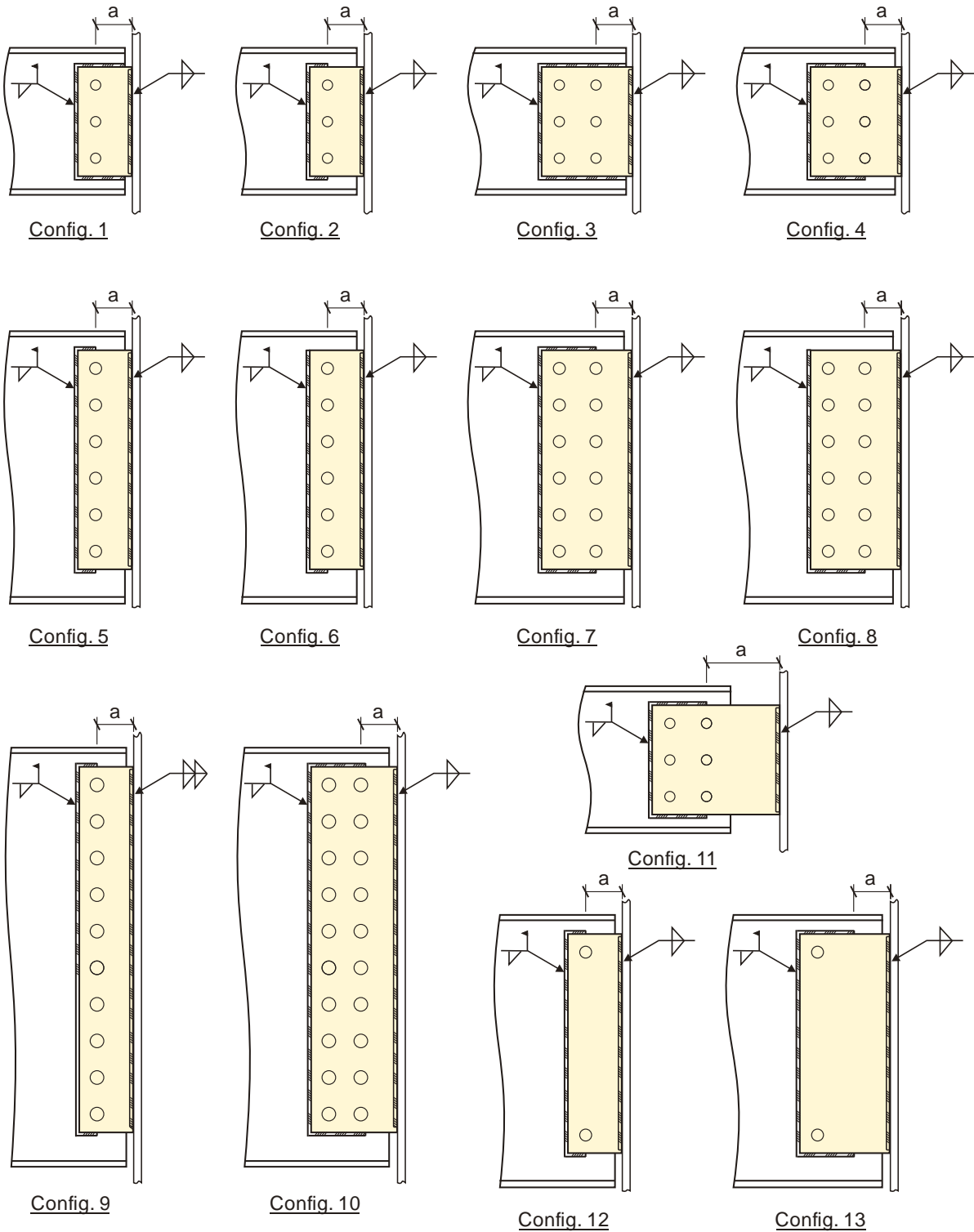


Figure 3: Detail drawings of weld-retrofit shear tab connections

2.2 Weld-Retrofit Design and Fabrication Methodology

The design checks were carried out using the conventional and extended shear tab design procedures, where applicable, of the AISC Steel Construction Manual (2010) to determine predicted resistances for the single and double row shear tab specimens (Marosi 2011, Hertz 2013). The AISC conventional design approach, for example, does not apply to connections when multiple rows of bolts are present. Because the design was done prior to materials testing, overstrength values of $1.1F_y$ and $1.1F_u$ were initially

assumed as the probable yielding and ultimate strengths of the shear tabs when checking to ensure that the shear tabs would fail prior to inelastic deformations taking place in the test beams. ASTM A325 snug tight bolts were used for design.

Practicing structural engineers were consulted concerning the design of the retrofit-weld connections to determine what types of retrofits may be used on construction sites. The weld group shapes included a “Full C”, a “Partial C” and an “L” (Figure 3). The logic behind using the “Full C” shape weld was to utilize the maximum space available for the weld. The “Partial C” shape weld was used because past tests had demonstrated that most of the deformation in the shear tab occurred over the ‘a’ distance, and hence it was decided to avoid placing a weld in this location to allow similar deformations to occur. The “L” shaped weld was chosen to facilitate the on-site welding procedure, where it was anticipated to be difficult to weld in the confined space between the top of the shear tab and the underside of the upper flange of the beam. Once the weld group shapes had been identified, the predicted factored resistance of the respective bolt group was set equal to the predicted factored resistance of the weld group, to determine the size of the fillet weld. This procedure was enabled for all specimens by using the instantaneous centre of rotation (ICR) method as it is provided in the Canadian Institute of Steel Construction (CISC) Handbook (2010), which is based on the Canadian Standards Association (CSA) S16 Design Standard (2009). The eccentricities used in calculating these weld group resistances were taken as the distance from the face of the column to the centroid of the respective weld group. Typical weld-retrofit specimens and the in-lab fabrication of a retrofit weld are depicted in Figure 4.

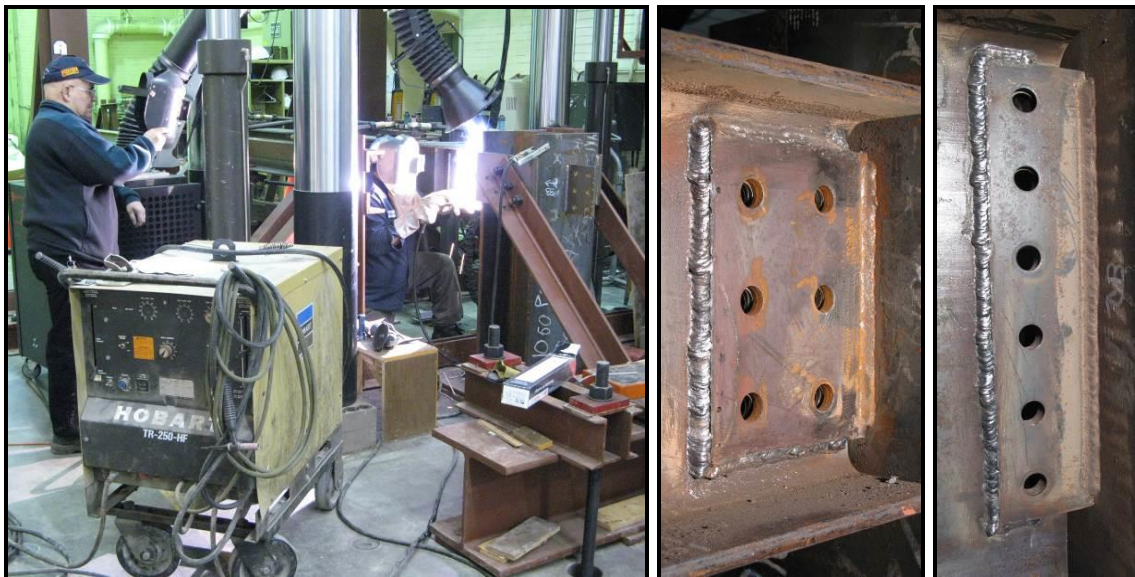


Figure 4: Photographs of representative weld-retrofit shear tab connections prior to testing (Test Configurations 3 & 5)

2.3 Laboratory Testing Procedures

Each connection was tested using a cantilever approach, whereby the beam was supported at one end by the shear tab connection to a column. Two hydraulic actuators were operated in displacement control to apply a shear force at the test connection and to simulate rotation by lowering the end of the test beam. Figure 5 shows an overview of the test beam and column, as well as the lateral bracing frames and beam end actuator support frame. Figure 6 contains photographs of a typical specimen in place. The actuator in the foreground provided the end reaction and lowered the beam end as the test

was carried out; the blocking under the beam end was removed prior to testing. Given the anticipated capacity of Configuration 10 in Figure 3, this actuator was replaced with hydraulic jacks (Marosi 2011). Lateral bracing frames were erected along the length of the beam to prevent the occurrence of lateral torsional buckling. A target rotation of the beam (relative to the face of the column) was chosen to be equal to 0.02 rad for the W310 sections and 0.015 rad for the W610 & W920 sections, to be reached at a probable ultimate shear resistance of the shear tab determined using a resistance factor of 1.0, and material properties of $1.1F_y$ and $1.1F_u$ in the calculations. The lower rotation target was applied to the deeper beams because it was anticipated that smaller mid-span deflections would occur in a real single-span loading scenario due to each beam's higher moment of inertia. A full description of the test setup and loading protocols is available in the works of Marosi (2011), Hertz (2014) and D'Aronco (2014).

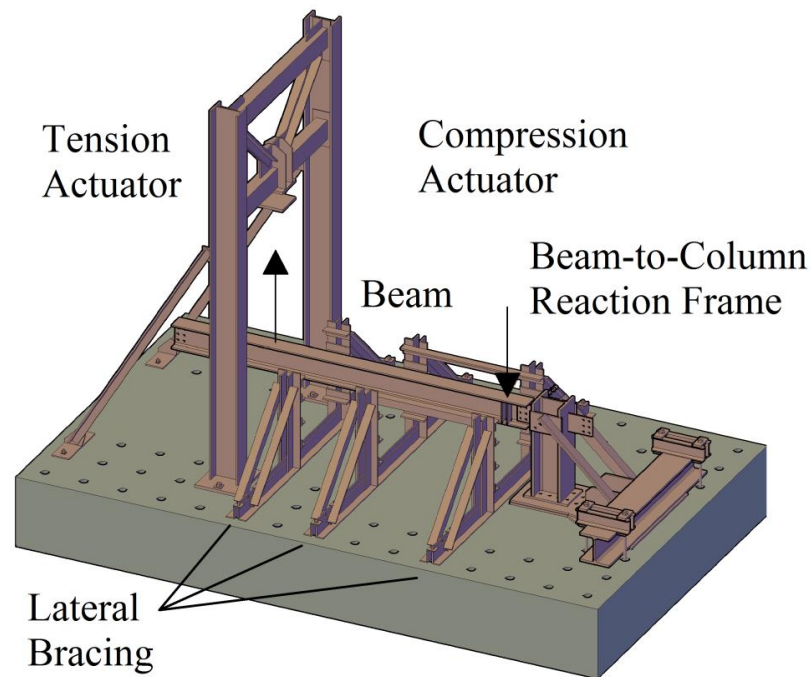


Figure 5: Schematic drawing of beam-to-column shear tab connection test setup

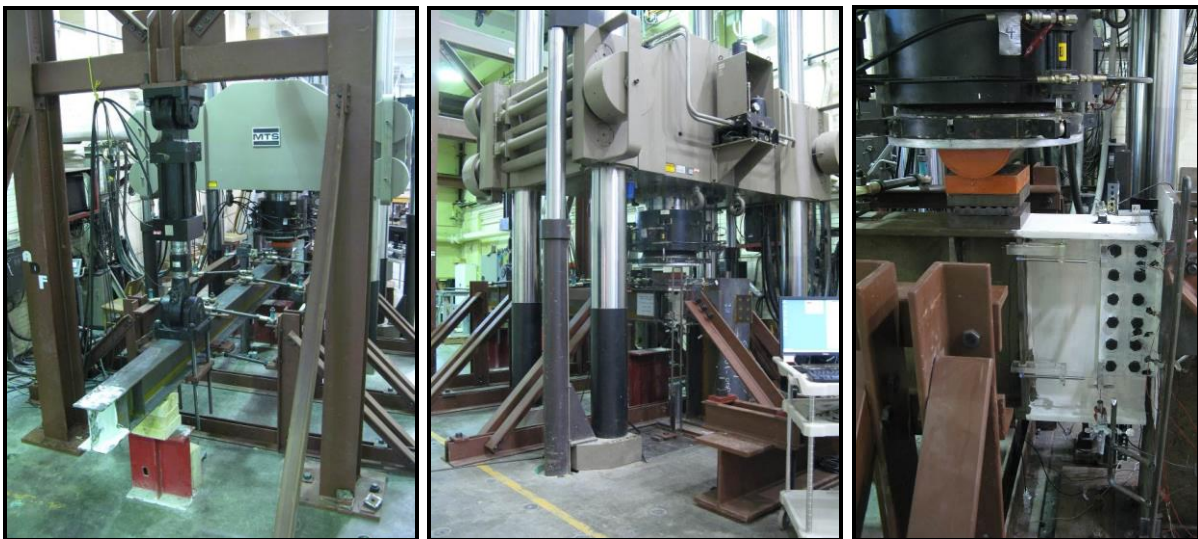


Figure 6: Photographs of beam-to-column shear tab connection test setup

2.4 Discussion of Test Results

Photographs of representative connection Configurations 1 and 7 are provided in Figure 7. The post-test deformations for both the bolted and weld-retrofit connections are shown. Note the similarity in terms of location of inelastic damage / yielding to the shear tab, which was most extensive over the 'a' distance. In the case of Configuration 1 ("Full C" weld) the horizontal welds between the end of the beam web and the centreline of the vertical bolt row fractured, effectively transforming this weld group into a "Partial C" shape. The remaining portion of the retrofit weld was unaffected. Configuration 7 comprised a "Partial C" weld shape, which was not damaged during loading; inelastic deformations were limited to the shear plate. Similar behaviour can also be seen for the other "Partial C" weld shape specimens in Figure 8, including Configuration 11, which possessed a long 'a' distance.

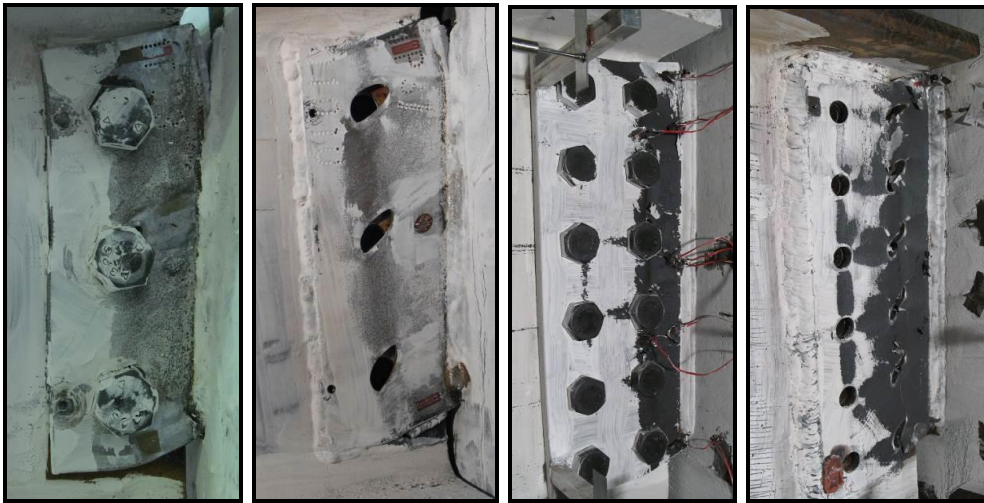


Figure 7: Photographs of post-test deformations for representative bolted and welded connections (Test Configurations 1 & 7)



Figure 8: Photographs of post-test deformations for welded connections "Partial C" weld (Test Configurations 2,4,9 & 11)

Pre and post-test photographs of the two "L" shape weld-retrofit specimens are provided in Figure 9. The response was similar to that observed for the "Partial C" connections. Note however, the greater extent of inelastic damage in Configuration 8 compared with Configuration 7 (Figure 7), whereby the plate yielding extended

past the 'a' distance. The retrofit welds for these specimens were not damaged. The welder involved in fabricating these two specimens, in addition to those having a horizontal weld along the top of the shear tab, commented that this top weld presented no difficulties during the in-lab fabrication.



Figure 9: Photographs of post-test deformations for welded connections “L” shape weld (Test Configurations 6 & 8)



Figure 10: Photographs of post-test deformations for welded connections “Partial C” weld on replacement shear tab plate (Test Configurations 12 & 13)

The final two weld-retrofit configurations, for which the original shear tab was removed and replaced with a plate that only contained two bolt holes to aid in installation, are depicted pre and post-testing in Figure 10. Although the retrofit welds remained largely undamaged for these specimens, the overall inelastic action of the shear tab was less compared with the other specimens that contained the original bolt holes. Nonetheless, these specimens were able to reach the target rotation demand, as did all other specimens, without any sudden failure. Given that the failure section of the shear tab was greater in area than the connections with the original bolt holes, the ultimate resistance exceeded that of other similarly sized weld-retrofit test specimens.

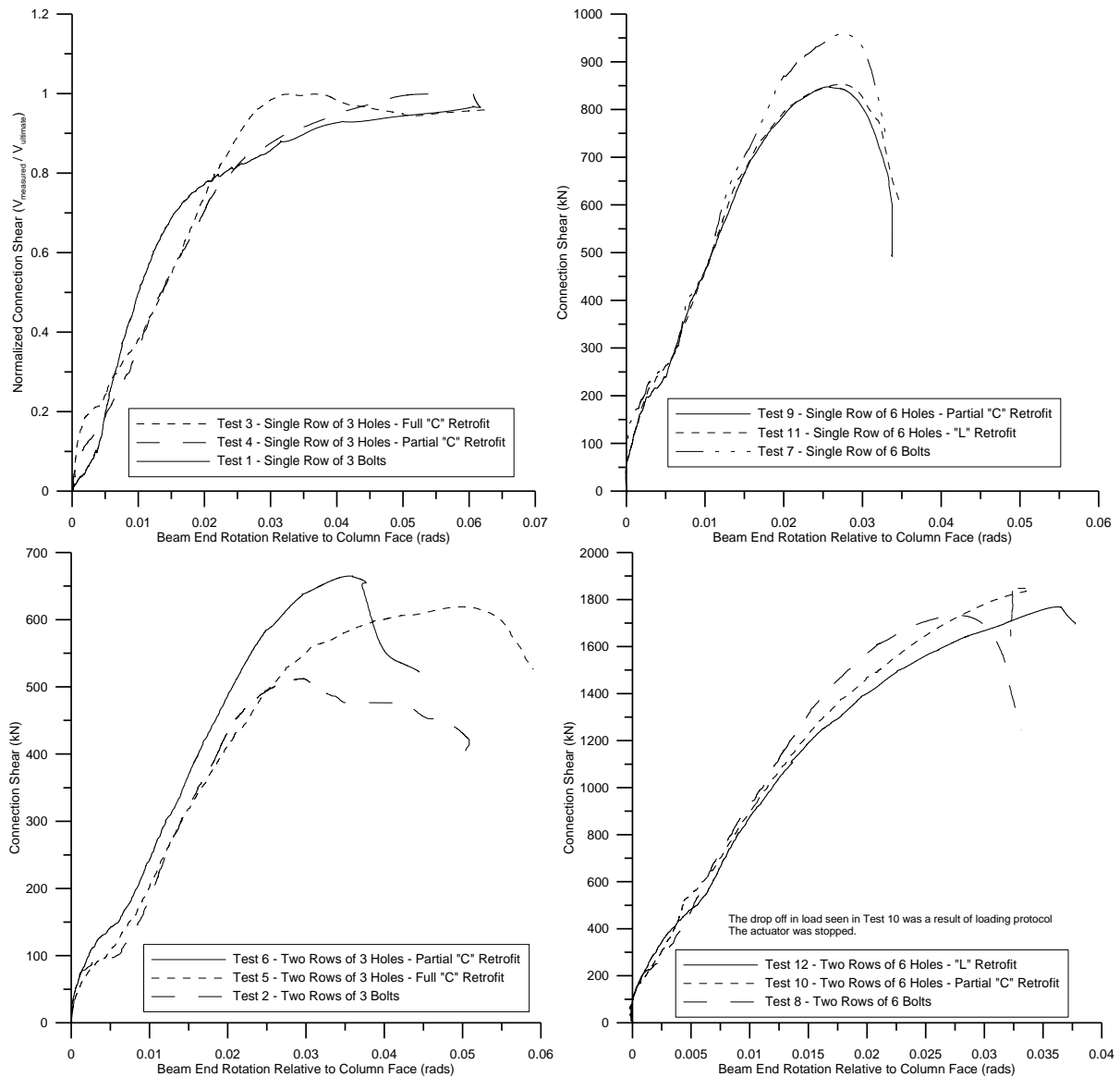


Figure 11: Graphs of connection shear versus rotation for representative bolted and weld-retrofit shear tab connection tests

Representative graphs showing the relationship between the measured connection shear resistance and the rotation between the beam end and column face are provided in Figure 11. Each graph includes the results for the original bolted connection, as well as the two matching weld-retrofit connections. Note: Tests 1, 3 & 4 are Configurations 1 & 2; Tests 7, 9 & 11 are Configurations 5 & 6; Tests 2, 5 & 6 are Configurations 3 & 4; Tests 8, 10 & 12 are Configurations 7 & 8. In all cases, the weld-retrofit specimens were able to reach the same rotation capacity as their matching bolted shear tab connection counterparts. Furthermore, the ultimate resistance levels attained by the weld-retrofit connections are similar to their bolted shear tab connection counterparts. Moreover, the connections all reached the force level associated with the AISC predictions.

3. CONCLUSIONS

The laboratory results demonstrated that these weld-retrofit connections can reach resistance and rotation levels consistent with equivalent bolted shear tab con-

nections. There is no advantage to using the “Full C” shape weld group; in contrast, the authors recommend the use of the “Partial C” shape weld since it does not restrict the deformation of the shear tab over the ‘a’ distance. The “L” shape weld does provide for adequate performance; however, given that the welder for this study did not find the installation of the top horizontal section of a retrofit weld to be difficult, one is not obliged to specify this shape. Given that the scenario presented herein is a retrofit solution to lack-of-fit on the construction site, using the original shear tab with all of its bolt holes is advised. The weakened section is advantageous in terms of maintaining the ductility of the shear tab connection. Using replacement shear tabs without bolt holes does raise the connection resistance.

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