Determination of Capacities of Eccentric Stiffeners Part 1: Experimental Studies

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ABSTRACT

Forty column specimens were experimentally tested with an effort to identify the effective stiffener capacities of eccentric stiffeners when used within moment connections of beams connecting to column flanges. Three test methods were performed described as (1) single tension with load pulling away from the column specimen, (2) single compression with load applied toward the column specimen, and (3) double compression with load applied toward the column specimen and with a reaction plate directly opposite from the applied load. The sizes were selected with a range of slenderness ratios for the web and flange and to study multiple concentrated load limit states using AISC *Specification* Section J10 (AISC, 2016b). For each column size and test method, four column specimens were tested: (1) without stiffeners, (2) with stiffeners concentric with the applied load, (3) with stiffeners at a low eccentricity of 2 in. or 3 in., and (4) with stiffeners at a high eccentricity of 4 in. or 6 in.

For column specimens tested without stiffeners and for compression tests, the maximum loads compared favorably to those predicted for web local crippling and were significantly higher than those predicted for web compression buckling (double compression tests) and web local yielding. For single tension tests, the maximum load always exceeded that predicted for the limit states of flange bending and web local yielding.

KEYWORDS: eccentric stiffeners, moment connections, concentrated loads, flange bending, web compression buckling, web local crippling, web local yielding.

INTRODUCTION

A comprehensive research project was performed at Lawrence Technological University (LTU) to evaluate the influence of stiffener eccentricity on the effective resistance of column members when subjected to concentrated loads that develop from moment connections. The study can be extended for the design of stiffeners for other scenarios when wide-flange beams are subjected to concentrated loads. Forty column specimens were experimentally tested under concentrated loads. The concentrated load simulated either an attached component in tension or an attached component in compression. The column specimens were notably smaller than column sizes that are used in practice due to limitations in the maximum loads that could be obtained by hydraulic actuators. However, the results of the experimental investigations were later utilized to calibrate/verify finite element models. Finite element models were then developed for more "practical" column sizes. This paper discusses the experimental investigations only, while the discussion of the finite element models is presented in Part 2: Analytical Studies (Alvarez Rodilla and Kowalkowski, 2021).

When concentrated loads are applied to column flanges at moment connections, capacities for different "limit states" are calculated to determine the resistance of the column section to the concentrated loads. Different limit states are considered if (1) a single compression force is applied to one flange; (2) a single tension force is applied to one flange; and (3) a double compression force is applied to both flanges simultaneously and the force on each side is directly opposite to each other, creating a through force. Tension or compression force refers to whether the component connected, such as a plate or beam flange, is subjected to a tensile or compressive axial force. Columns may also be subjected to a tension force on both flanges simultaneously or subjected to a compression force on one flange and a tension force on the other flange. However, there are no specific limit states associated with these two combined loading conditions, and individual forces on one flange can be treated as either single tension or single compression forces. Therefore, the three conditions analyzed in this research were single tension, single compression, and double compression. The limit states that define resistance of a steel member to concentrated loads and those applicable for the design of transverse stiffeners are summarized as follows (AISC, 2016b):

- Web local yielding—Limit state applies for all loading

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conditions, tension or compression, and at each loaded location on the flanges.

- Flange bending—Limit state applies when a concentrated tension force is applied to the flange and across the width of the flange (usually the case for moment connections).
- Web local crippling—Limit state applies when a single- or double-concentrated compression force is applied to the flange(s).
- Web compression buckling—Limit state applies when a double compression force is applied to both flanges simultaneously, causing a through-compression force.

In the event that the calculated applied force (demand) from the moment connection exceeds the capacity per the preceding limit states, stiffener plates and/or doubler plates are utilized to reinforce the column section. Usually, the use of stiffener plates is preferred, but in some cases, interferences exist that disallow the stiffener plates to be concentric with the applied force. Instead, the stiffeners are detailed and utilized at an eccentricity with respect to the location of the concentrated force.

Limited experimental studies have been performed to evaluate the influence of stiffener eccentricity and the effective resistance of column sections to concentrated loads. This paper evaluates the "effective stiffener capacity," which is defined as the difference between test capacities for column specimens with stiffeners, both concentric and eccentric, and test capacities for the corresponding column specimens without stiffeners and subjected to the same loading condition.

BACKGROUND

When concentrated loads are applied perpendicular to a column flange, several concentrated load limit states are checked to avoid local failure mechanisms per Sections J10.1, J10.2, J10.3, and J10.5 of the AISC *Specification for Structural Steel Buildings* (AISC, 2016b), hereafter referred to as the AISC *Specification*. If the capacity of any of these limit states is less than the computed concentrated force, AISC permits the use of full-depth transverse stiffeners to partially resist the concentrated force. The lowest applicable capacity for the concentrated load limit states is used to define the member capacity. The required stiffener capacity is equal to the applied concentrated load minus the member capacity. In accordance with AISC *Specification* Section J10.8, stiffeners are designed as part of the connections and considered as tension or compression members.

Eccentric stiffeners refer to that fabricated in the column section at an offset from the beam flange or flange plate. Examples of concentric and eccentric stiffener conditions are shown in Figure 1. Figure 1(a) demonstrates a condition where stiffeners are concentric, while Figure 1(b) shows an eccentric stiffener scenario where one beam flange is offset from the stiffener.

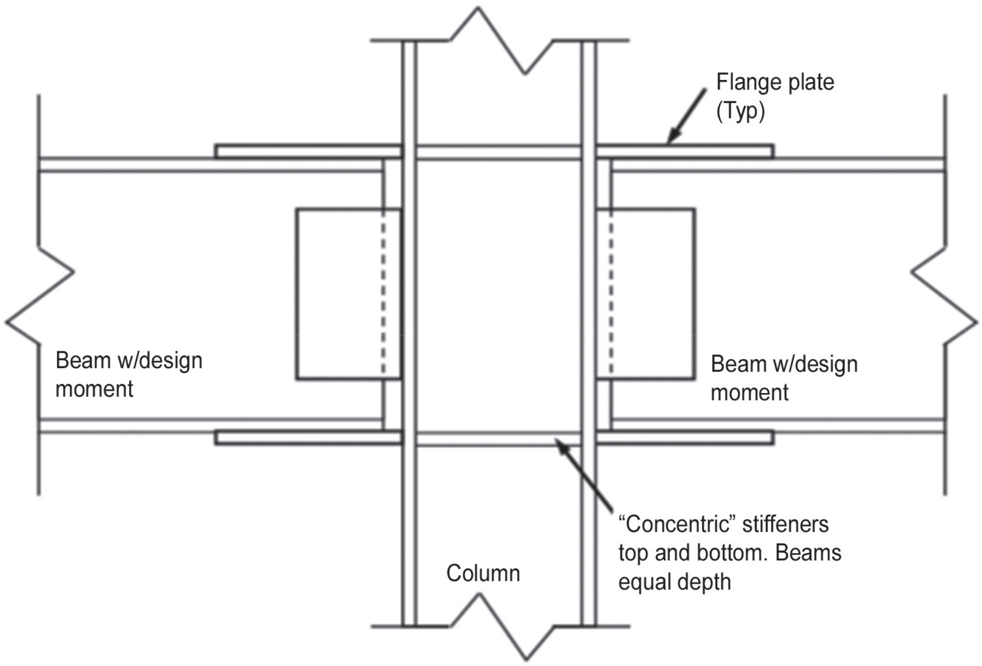
Figure 1 shows one example of the utilization of eccentric stiffeners, but in general, they may be required due to the presence of multiple connections framing into the joint or other cases that would create interferences within the column boundaries. Common cases that occur in practice when multiple plates are theoretically required at slightly different elevations include:

- When beams are moment connected to both column flanges but are of different depths and calculations mandate, stiffeners are required for the concentrated loads being applied to both flanges (as in Figure 1).
- When beams are moment connected to both column flanges but the top of steel elevation is different on both sides and calculations mandate, stiffeners are required for the concentrated loads being applied on both sides.
- When the location of a stiffener for a moment connection to a column flange would interfere with a flange plate designed as part of a connection where a beam frames into a column web.

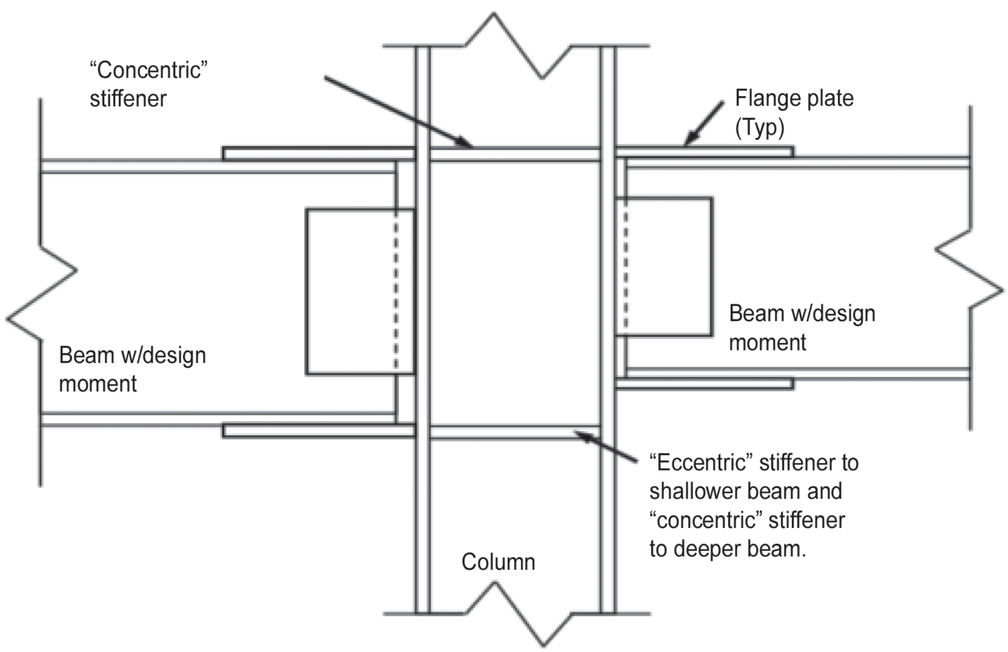
In all three cases, concentric stiffeners may not be feasible due to clearance for welding that must occur at each stiffener. In addition, it may not be necessary to have each stiffener if other stiffeners are close to the proximity of the concentrated load; this "acceptable" eccentricity is one of the key focuses of this research project.

Although the AISC *Specification* provides some guidance for the design of transverse stiffeners, there is no assistance associated with tolerances or design with eccentric stiffeners. The only source available from AISC for the design of eccentric column stiffeners or alternate methods to reinforce columns is AISC Design Guide 13, *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications* (Carter, 1999). More specifically, Design Guide 13, Section 5.1, discusses options for column stiffening for beams of differing depth and/or top of steel. The Design Guide shows multiple options if the beam flanges on opposite sides of the columns are at different elevations, such as the use of partial depth stiffeners or sloping stiffeners. A final option discussed in the Design Guide is the use of one transverse stiffener that is aligned with one beam flange and not the other or to provide one transverse stiffener in between the two beam flanges. In either case, this creates a design eccentricity e as shown in Figure 2.

Research performed by Graham et al. (1959) showed that transverse stiffeners with a 2-in. eccentricity provides 65% of the strength of an identical transverse stiffener that is noneccentric. In addition, it was recommended, "…for design purposes, it would probably be advisable to neglect



(a) Concentric stiffener condition



(b) Eccentric stiffener condition

Fig. 1. Example of concentric stiffeners vs. eccentric stiffeners.

the resistance of stiffeners having eccentricity greater than 2 in. The required transverse stiffener area, width and thickness can be established by the same criteria as for concentric stiffeners, provided that the strength is reduced linearly from 100 percent at zero eccentricity to 65 percent at 2 in. eccentricity."

Further research on concentrated loads in wide-flange sections was performed by Sherbourne and Murthy (1978). This research focused on analyzing the stability of column webs in beam-column moment connections. Sherbourne and Murthy carried out analytical tests on wide-flange sections with concentric and eccentric stiffeners. Eccentric stiffeners were placed at 25% and 50% of half the column depth. The results showed that the relative load capacity of eccentric stiffeners decreased dramatically compared to the specimens with concentric stiffeners in any event, regardless of the column's flange or web thicknesses. For the first test series with stiffeners at lower eccentricities, the effectiveness of the stiffeners ranged between 50% and 75% of that for concentric stiffeners. Sherbourne and Murthy also found that the web thickness has a direct influence on the buckling load and behavior.

Norwood (2018) performed analytical research on wide-flange sections similar to the experimental tests carried out by Graham et al. (1959). Similar to previous research, Norwood found that increasing the level of eccentricity between the flange and the transverse stiffener results in a decrease in resistance to concentrated forces. The results showed that eccentric stiffeners were more effective in comparison to the recommendations given by Graham et al.

The limitation of a 2-in. eccentricity has been problematic in design. Further, verbal communication with representatives of the steel industry indicates that fabricators and designers have utilized stiffeners at higher eccentricities, regardless of the lack of design standards for such cases. The results presented in this research project are intended to provide further clarity and expand the recommendations presented in Carter (1999). The project investigates eccentricities higher than 2 in. and, more importantly, evaluates a wider range of column sizes and loading conditions in comparison to the work performed by Graham et al. (1959).

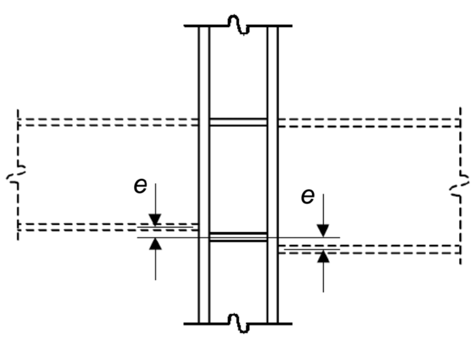


Fig. 2. Stiffener option at eccentricity e *per AISC Design Guide 13 (Carter, 1999).*

EXPERIMENTAL RESEARCH PROJECT

Forty "column" specimens (the test setup is described later) were subjected to concentrated loads at mid-length (center of beam with respect to longitudinal axis). Four different cross-section sizes were tested: W10×19, W10×39, W12×26, and W16×31. For single compression tests, all four sizes were tested. For double compression tests and single tension tests, three of the four sizes were tested (all except W10×19 for single tension and W10×39 for double compression). For each test type and each section size, the following four tests were performed and will be further referred to as the "test group":

1. One test was performed without any stiffeners at mid-length to establish a baseline capacity prior to the addition of any stiffeners.
2. One test was performed with stiffeners directly concentric with the concentrated load and, therefore, with an eccentricity of 0 in.
3. One test was performed with stiffeners at a high eccentricity from the concentrated load. For the W16×31 column specimens, high eccentricity was equal to 6 in.; for other column specimens, the high eccentricity was equal to 4 in.
4. One test was performed with stiffeners at a low eccentricity from the concentrated load. For the W16×31 column specimens, the low eccentricity was equal to 3 in.; for other column specimens, the low eccentricity was equal to 2 in.

It is important to note why the "high eccentricity" and "low eccentricity" magnitudes are different for some column specimens. Initially, an objective was to identify the effective stiffener capacities of eccentric stiffeners as a percentage of the effective stiffener capacity of concentric stiffeners up to a maximum eccentricity of 6 in. Initial tests on W16×31 column specimens demonstrated no difference in the maximum load without stiffeners and when stiffeners were used at a high eccentricity of 6 in. and therefore, the low and high eccentricities were changed to 2 in. and 4 in., respectively, prior to installing stiffeners into other column sections.

TEST MATRIX

Table 1 shows an abbreviated experimental test matrix. Three factors were considered in selecting the column specimen sizes for the experimental test matrix:

Table 1. Test Matrix for Experimental Column Specimens				
Member	Test Method	$b_f/2t_f$	h/t_w	Predicted Failure Mode without Stiffeners Using Nominal Capacities
W16×31	Single compression	6.28	51.6	Web local yielding
W12×26	Single compression	8.54	47.2	Web local yielding
W10×39	Single compression	7.53	25.0	Web local yielding
W10×19	Single compression	5.09	35.4	Web local yielding
W16×31	Double compression	6.28	51.6	Web compression buckling
W12×26	Double compression	8.54	47.2	Web compression buckling
W10×19	Double compression	5.09	35.4	Web compression buckling
W16×31	Single tension	6.28	51.6	Flange bending
W12×26	Single tension	8.54	47.2	Flange bending
W10×39	Single tension	7.53	25.0	Flange bending

1. Member sizes were selected for single compression testing, double compression testing, and single tension testing such that the capacity of the hydraulic actuator would not be exceeded prior to failure, with and without stiffeners.

2. A proportioning study was performed to relate the geometry of smaller shapes to the geometry of larger shapes utilizing the slenderness ratios of the flange and web. Three different ranges were considered for the flange slenderness (defined as the $b_f/2t_f$ ratio), and three different ranges were considered for the web slenderness (defined as the h/t_w ratio). The ranges included approximately the same number of wide-flange members. More information can be found in Kowalkowski and Alvarez Rodilla (2019).

3. Member sizes were selected to have a variation in the predicted limit state that defines capacity under concentrated loads. These limit states include web local yielding, web compression buckling, and flange bending (AISC, 2016b). Nominal capacities were used for this study as shown in Table 1.

The nomenclature used for each column specimen, as shown with the results presented later, includes the column specimen size, the test method, and the stiffener condition. For the test method, DC refers to double compression, SC refers to single compression, and ST refers to single tension. For the stiffener condition, NA indicates a test without stiffeners. E2 refers to a column specimen with stiffeners at an eccentricity of 2 in. A similar designation is utilized for other magnitudes of eccentricity.

TEST SETUP

All experimental testing was performed in the Structural Testing Center at Lawrence Technological University in Southfield, Michigan, using a high-capacity load frame and hydraulic actuator. The hydraulic actuator had a capacity of 220 kips for a downward force (compression) and a capacity of 145 kips for an upward force (tension). Therefore, the capacity for tension was lower than the capacity for compression. This fact was unknown until the experimental testing stage progressed, which led to some tension specimens unexpectedly reaching the actuator capacity without any mode of failure.

Photographs of the experimental test setups are provided in this section. Full schematics are provided with dimensioning, plate, and weld sizes in Kowalkowski and Alvarez Rodilla (2019).

Figure 3 shows a photograph of the test setup used for single compression tests. The column specimens were simply supported at 5 ft, and load was applied using a ¾-in. loading plate fabricated from ASTM A36 steel. Lateral supports were utilized at each end to avoid lateral translation of the top flange and twisting of the column specimen. Figure 3 identifies the location of paint/spackle used for digital image correlation (DIC). This instrumentation will be discussed later in this paper.

The double compression setup was the same as the single compression setup but with an additional ¾-in. reaction plate located directly below the column specimen and in line with the applied load. The reaction plate was part of a fixture, similar to the fixture that is attached to the hydraulic actuator. The reaction plate and fixture were supported on a built-up support frame that was supported by the concrete floor. Figure 4 shows a photograph of the test setup used for the double compression tests.

The test setup for single tension specimens included a specially designed reaction frame that was attached to the existing steel frame as shown in Figure 5. The reaction frame consisted of W10×68 posts, W10×68 spreader beams, HSS bracing, plates and bolts. The test setup was designed to allow force to be applied upward without fabricating new post-tensioning holes in the concrete strong floor. Unlike the compression tests, the loading plate was welded to the top flange of the column specimen. This detail is discussed more in the next section. The loading plate was bolted to two vertical plates simulating double shear in the bolts. The bottom flanges of the column specimens were laterally supported to prevent lateral translation and twisting of the column specimens under loading. Additional details of the test setups are provided in Kowalkowski and Alvarez Rodilla (2019).

COLUMN SPECIMENS

Column specimens were 6 ft long and fabricated from ASTM A992 steel. Uniaxial tension tests were performed according to the requirements of ASTM E8/E8M-09 (ASTM, 2009) on coupons fabricated from the column specimens, which confirmed that the yield stress was approximately 55 ksi for all column specimen sizes. All coupons were taken from the web and/or flange of the column specimen, away from mid-length.

Near mid-length and when applicable, stiffeners were located on both sides of the web with a total depth equal to the depth between column flanges minus 1⁄16 in. For W10×39, W12×26, and W16×31 column specimens, all stiffeners were 3⁄8 in. and welded using 1⁄4-in. fillet welds. For W10×19 column specimens, all stiffeners were 1⁄4 in. and welded using 3⁄16-in. fillet welds. The depth of the stiffeners extended to the edges of the flange to the nearest 1⁄16 in. At the fillet radius between web and flange, 1⁄2-in. corner clips were fabricated. For all stiffeners at mid-length, welding was performed using flux-core arc welding (FCAW). The stiffener plates were specified to be A36 steel. However, when delivered, it was revealed that some stiffeners met the specifications for A36 and A572 Gr. 50 and were, therefore, dual certified.

Bearing stiffeners were used for most tests at the supports to ensure concentrated load failure mechanisms were not developed at the support reactions. The stiffeners were of the same dimensions as the mid-length stiffeners. The bearing stiffeners were fillet welded to both flanges and web of the cross-section; however, FCAW was not used. For

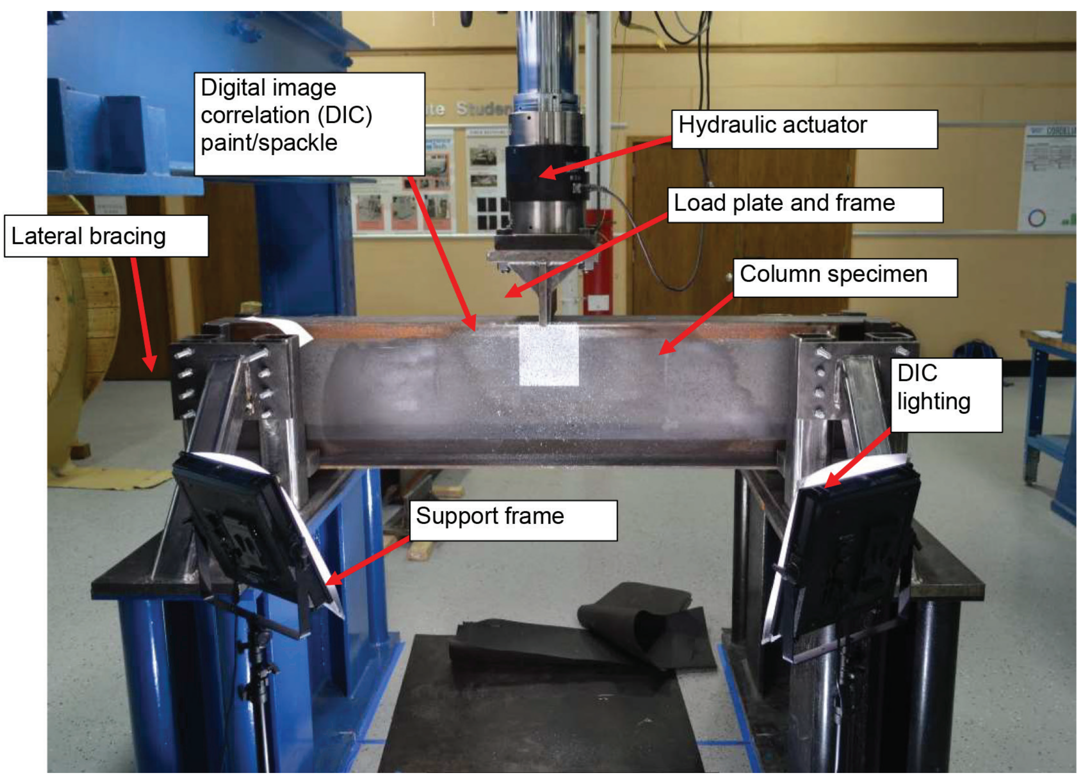


Fig. 3. Single compression test setup.

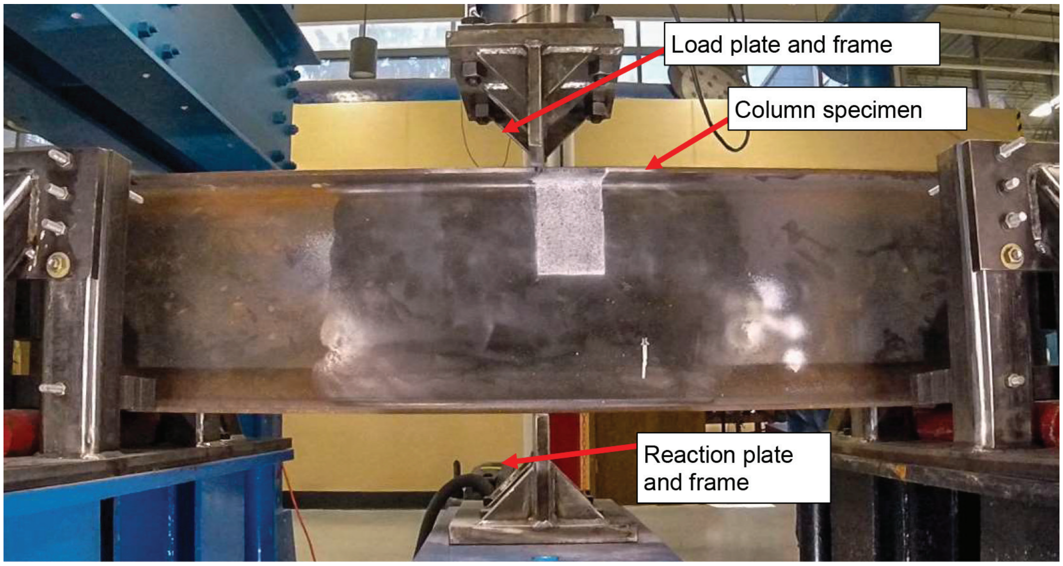


Fig. 4. Double compression test setup.

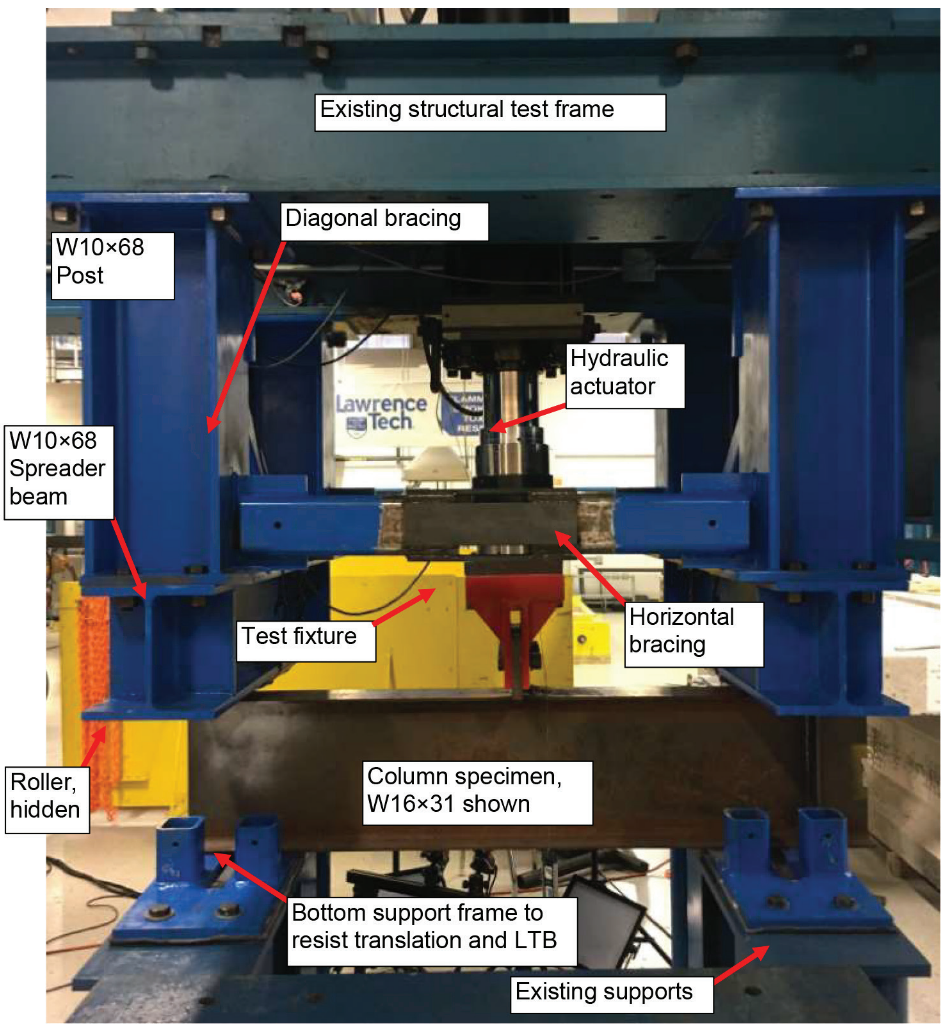


Fig. 5. Single tension test setup.

double compression tests, no bearing stiffeners were used because all the load was assumed to pass through the web to bottom support reaction.

Column specimens for single tension tests were welded to a ⅞-in. loading plate made from A36 steel, which was bolted to two plates of the test setup as discussed in the previous section. The loading plates were attached to the column specimens using ⅜-in. fillet welds on both sides using the FCAW process. Figure 6 shows a photograph of the W10×39 ST-E2 column specimen prior to testing.

INTRUMENTATION AND EXPERIMENTAL PROCEDURE

The hydraulic actuator was equipped with a load cell that measured the applied load during testing. In addition, the actuator stroke displacement was tracked and recorded. Additional instrumentation to capture strain and displacements under the applied load was used. Digital image correlation (DIC), which is an optical method that employs tracking and image registration techniques for accurate two-dimensional and three-dimensional measurements of changes in images, was used as the primary source of instrumentation. DIC is capable of developing a three-dimensional strain and displacement state and was utilized to evaluate local deformations and stresses, along with the mode of failure. A localized area of the column's web and flange and the interior stiffener face, if applicable, were prepped for DIC as discussed in Kowalkowski and Alvarez Rodilla (2019). Figure 3 identified the localized area on a column specimen without stiffeners that was painted/spackled for DIC.

Figure 7 demonstrates DIC results for the W12×26 SC-E4 column specimen at a load of 78 kips. The load of 78 kips corresponds to the maximum load obtained for the equivalent column section without stiffeners. In this example, strain is shown for the *y*-direction, which corresponds to the direction of the applied load. The blue and green represent areas of high negative (compression) strain. Figure 7 shows that for this specimen, the concentrated load is spread into the column section in a very localized area under the applied load away from the eccentric stiffener. This negligible influence of eccentric stiffeners in resisting concentrated loads was a common trend in specimens with eccentric stiffeners as described in later sections.

All tests, tension or compression, were performed in displacement control with a loading rate of 0.05 in./min. For most compression tests, the experiment was considered complete when a maximum load was obtained and the load started to decrease significantly with an increase in vertical displacement. A decrease in load indicated that a buckling failure mechanism had occurred. Common failure modes in compression tests are best described as web local crippling and stiffener buckling. The test was allowed to continue until a failure mechanism was visibly apparent.

The test procedure for the single tension tests was similar to the test procedure used for the compression tests. The column specimens were attached to the top test fixture using bolts, and the column specimen had to be raised into contact with the top rollers (see Figure 5) prior to initiating the test. The single tension tests were considered complete when one of the two events occurred: (1) the actuator capacity was reached or (2) the weld between the loading plate and column specimen failed. Weld failure generally occurred for column specimens without stiffeners and when eccentric stiffeners were used. During one single tension test, rupture occurred in the web of the column specimen directly beneath the loading plate. For all three column specimen sizes, the capacity of the actuator was reached when concentric stiffeners were used. In addition, for all W10×39 column specimens, the capacity of the hydraulic actuator was reached prior to another failure occurring. Reasons for why the actuator capacity was not sufficient in these tests are described in Kowalkowski and Alvarez Rodilla (2019). This result was not desired, but the DIC results were utilized to evaluate experimental behavior for different stiffener conditions prior to reaching the capacity of the hydraulic actuator.

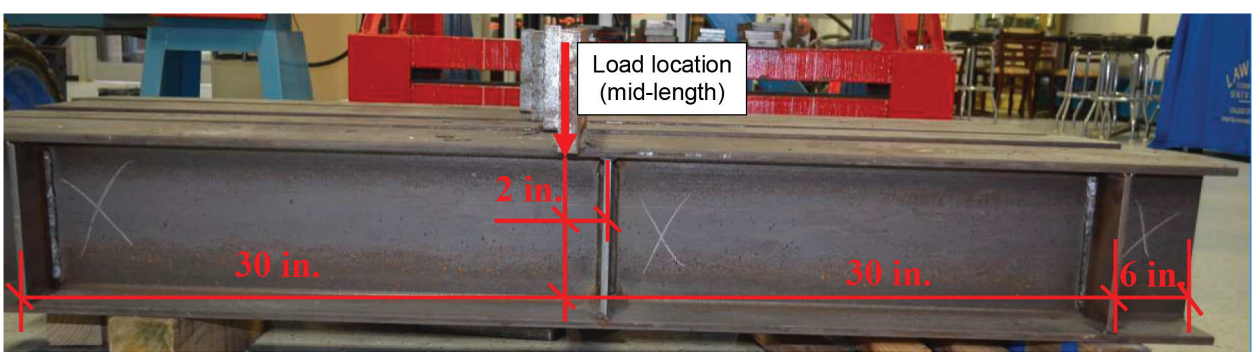


Fig. 6. Photograph of W10×39-ST-E2 (eccentricity = 2 in.) prior to testing.

ANTICIPATED FAILURE MECHANISMS

Table 1 listed the anticipated failure mechanisms or limit states for column specimens when no stiffeners were used near the location of the concentrated loads. In this research, it is crucial to understand how the stiffeners either share loading or stiffen the web, preventing the web's ability to buckle or cripple. Therefore, for each test method, a particular limit state was evaluated, and column sizes were selected to ensure that the particular limit state controlled for the applicable test method. When nominal strengths are calculated (i.e., without ϕ or Ω factors per LRFD or ASD, respectively), the limit state of web local crippling never controls for any standard wide-flange member when load is applied away from the end of the member. As shown later in this report, most columns subjected to either single compression or double compression reached their actual capacity when web local crippling occurred. However, web yielding was identified during the tests. After initial yielding, the load continues to increase until a buckling limit state is reached; after which the load starts to decrease rapidly.

Strengths were calculated using AISC *Specification* Section J10 (AISC, 2016b) using an expected yield strength of 55 ksi from the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2016a). Flange bending is given by Equation 1 and is only applicable in design for single tension tests. Web local yielding is given by Equation 2 and is applicable for all test methods. Web local crippling is given by Equation 3 and is applicable for all compression tests. Web compression buckling is given by Equation 4 and is applicable only for double compression tests. For Equations 1–4, the first equation number corresponds to the respective equation in the AISC *Specification* (AISC, 2016b).

$$R_n = 6.25F_{yf}t_f^2 \qquad \text{(from AISC } \textit{Spec}\text{. Eq. J10-1) (1)}$$

$$R_n = 1.0(5k + l_b)F_y t_w \qquad \text{(from AISC } \textit{Spec}\text{. Eq. J10-2) (2)}$$

$$R_n = 0.80t_w^2\left[1+3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_y t_f}{t_w}} \qquad \text{(from AISC } \textit{Spec}\text{. Eq. J10-4) (3)}$$

$$R_n = \frac{24t_w^3\sqrt{EF_y}}{h} \qquad \text{(from AISC } \textit{Spec}\text{. Eq. J10-8) (4)}$$

where

F_y = expected yield strength = 55 ksi

d = overall depth of section, in.

h = depth of web between fillets = $d - 2k$, in.

l_b = bearing length, in this research is equal to thickness of loading plate, in.

k = depth from face of flange to edge of fillet, in.

t_f = thickness of flange, in.

t_w = thickness of web, in.

EXPERIMENTAL RESULTS

The discussion of the experimental results is separated by test method. Detailed discussion focuses on W12×26 column specimens for compression tests and W16×31 column specimens for the single tension tests. The results of all experimental testing are best described and represented utilizing the results of these column specimen sizes and the corresponding test methods. All experimental results for all column specimens are provided in Kowalkowski and Alvarez Rodilla (2019). For each test, load-displacement relationships were plotted and the four tests of the "test

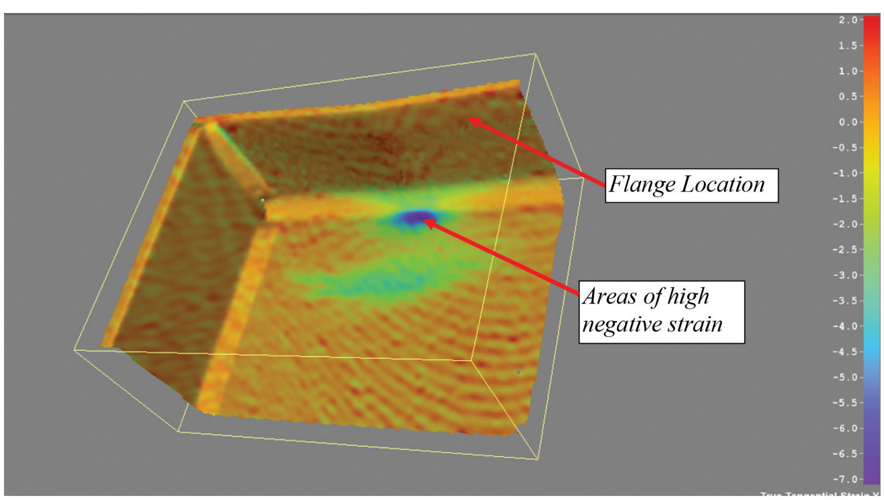


Fig. 7. DIC results; vertical strain for W12×26 SC-E4 column specimen at 78.2 kips.

group" were plotted together (one without stiffeners, one with concentric stiffeners, and two with eccentric stiffeners) to further evaluate the influence of eccentric stiffeners. The hydraulic actuator's load cell measurement for load and the actuator's stoke displacement were used to develop the load-displacement relationships. When applicable, theoretical (or nominal) capacities associated with concentrated load limit states are shown on the figures for the wide-flange section in question. These are represented with horizontal lines labeled FB for flange bending, WLY for web local yielding, WCR for web local crippling, and WCB for web compression buckling. Per the AISC *Specification* (2016b), flange bending only needs to be considered in design when a tension load is applied to the column flange and, therefore, would not to be considered in design for a compression load. However, the calculated limit state capacity is shown on all load-displacement results since significant bending was witnessed during all compression tests, which influenced the nonlinear behavior of the load-displacement results.

All primary results for single compression, double compression, and single tension tests are summarized in Tables 2, 3, and 4, respectively. The results in the table include the "test capacity," which is the maximum load recorded during the test, and the "effective stiffener capacity." The effective stiffener capacity is equal to the capacity of the test with stiffeners minus the result of the corresponding test without stiffeners. Negative effective stiffener capacities indicate a particular test with eccentric stiffeners that reached a lower capacity than the corresponding test without stiffeners. In these cases, it is assumed that the stiffeners had no influence on the concentrated load capacity of the column specimen, and differences are due to inconsistencies between material properties of multiple column specimens and/or imperfections in the column specimen and loading/boundary conditions when multiple tests are performed.

In Tables 2, 3, and 4, theoretical (nominal) capacities per the concentrated load limit states applicable for the test method are listed. The theoretical stiffener capacity is provided assuming the stiffeners are concentric and that the capacity is controlled by a yielding limit state at the top of the stiffener. The "theoretical capacity" varies for each type of test depending on the stiffener condition. For columns without stiffeners, the theoretical capacity represents the capacity of the lowest computed concentrated load limit state associated with the test method. For columns with concentric stiffeners, the theoretical capacity represents the lowest computed limit state in addition to the concentric stiffener capacity. For column specimens with eccentric stiffeners, the corresponding theoretical capacity is uncertain and related to the fundamental objectives of the research program and, therefore, noted as TBD in the tables.

Single Compression Test Results

Sixteen experimental column specimens were tested in single compression. This includes four wide-flange sizes and four tests per size. A summary of the primary results and theoretical capacities from the single compression tests is provided in Table 2.

Observations from the results presented in Table 2 are as follows:

- For all tests without stiffeners and with concentric stiffeners, the maximum load exceeded the theoretical capacity with one exception, W10×39 SC-E0. As described in Kowalkowski and Alvarez Rodilla (2019), this column specimen, aided with the addition of concentric stiffeners, reached a limit state associated with shear and flexural yielding in lieu of that directly influenced by local deformations from the concentrated load.
- The maximum loads obtained for all specimens with eccentric stiffeners were similar to the results of the corresponding specimens without stiffeners. There were no clear trends in the results. However, the "low eccentricity" condition always resulted in a higher capacity than the "high eccentricity" condition.
- The maximum loads for columns without stiffeners were close to the theoretical values computed for web local crippling with the exception of the W10×19 column specimen. For this case, lateral-torsional buckling was also witnessed during the test as discussed in Kowalkowski and Alvarez Rodilla (2019).
- For column specimens with concentric stiffeners, there was a significant increase in capacity in comparison to the condition without stiffeners. In addition, the effective stiffener capacity results compared well with the theoretical stiffener capacities.

Behavior of Single Compression Tests

Figure 8 shows the load-displacement relationships for the W12×26 SC column specimens and emphasizes differences in experimental behavior for different stiffener conditions. As mentioned, calculated concentrated load limit states (WCR, WLY, FB) are also shown as horizontal lines in Figure 8. Because this represents a single compression test, WLY would be the controlling limit state in design as demonstrated in Table 2.

Figure 8 indicates that the load capacity and elastic stiffness of column specimens with eccentric stiffeners are very similar to the result without stiffeners. Note that the definition of "elastic stiffness" used in this comparison is associated with the load versus actuator displacement and includes flexural, shear, and local deformations that occur in the loading plate as well as deformations that occur in

Table 2. Single Compression Theoretical Capacities and Test Results							
Column Specimen	Eccentricity (in.)	WLY (kips)	WCR (kips)	Stiffener (kips)	Theoretical Capacity (kips)	Test Capacity (kips)	Effective Stiffener Capacity (kips)
W16×31 SC-NA	NA	75.0	103	62.5	75	112	—
W16×31 SC-E6	6				TBD	99.3	–13.1
W16×31 SC-E3	3				TBD	111	–1.2
W16×31 SC-E0	0				138	177	64.4
W12×26 SC-NA	NA	52.5	74.7	76.9	52.5	78.2	—
W12×26 SC-E4	4				TBD	79.2	1.00
W12×26 SC-E2	2				TBD	87.1	8.90
W12×26 SC-E0	0				129	148	70.1
W10×19 SC-NA	NA	58.1	88.2	27.0	58.1	69.6	—
W10×19 SC-E4	4				TBD	57.4	–12.2
W10×19 SC-E2	2				TBD	70.6	1.00
W10×19 SC-E0	0				85.1	103	33.7
W10×39 SC-NA	NA	102	144	97.6	102.2	131	—
W10×39 SC-E4	4				TBD	133	1.90
W10×39 SC-E2	2				TBD	142	11.3
W10×39 SC-E0	0				200	198	67.1

the column specimen near the concentrated load. Because the flexural and shear properties of the cross section do not change, it is assumed that the primary differences in the elastic stiffness are associated with changes in local deformations that occur near the concentrated loads. The maximum load of the column specimen with a low eccentricity of 2 in. increased slightly in comparison to the condition without stiffener. However, for the concentric stiffener condition, the load increased substantially in comparison to the other conditions, and the elastic stiffness of the load-displacement relationship moderately increased. These observations demonstrate how ineffective the eccentric stiffeners were in comparison to concentric stiffeners for these column specimens and other test groups (W16×31 SC, W10×19 SC, and W10×39 SC) revealed similar conclusions.

Even though the load capacity did not increase substantially for columns tested with stiffeners at a low eccentricity, the stiffeners did assist in the post-buckling (or crippling) behavior of the column specimen, which is why a higher load was maintained for higher displacements. For the columns without stiffeners, the load decreased more rapidly after crippling occurred.

The results in Figure 8 show that three of the columns failed at a capacity close to the theoretical capacity for web local crippling (74.7 kips), which was the mode of failure identified from visual inspection of these column specimens. When the results are looked at in more detail, nonlinear behavior does initiate near the capacity associated with the limit state of web local yielding. However, the column specimens reach much higher loads after the calculated value for this limit state, and significant nonlinear behavior is not witnessed in the load-displacement relationship until web crippling initiates.

In addition to the load-displacement results, visual observations revealed that the column specimens with eccentric stiffeners behaved similar to the column specimens without stiffeners. In all three cases, yielding was first seen in the column web beneath the loading plate and continued through the web by means of strain lines radiating from the mentioned point until the column reached its maximum load and suddenly failed by web local crippling. Figure 9 shows the condition of W12×26 SC-NA after testing and provides a good representation of the deformed state of all three aforementioned column specimens after testing. Figure 9 also visually demonstrates that web local crippling was the failure mechanism of the column specimen and identifies the horizontal and vertical strain lines that appeared on the web.

As shown in Figure 8, the W12×26 SC-E0 column specimen (eccentricity = 0 in.) had a slightly higher elastic stiffness and a significantly higher load capacity than the other column specimens. Yielding was first identified at a much higher load as well. Large horizontal and vertical strain lines in the web were identified at a load of 130 kips. Yielding at the bottom flange by means of large strain lines and mill scale peeling off was also identified during the test, suggesting that significant flexural deformations were able to occur with the addition of concentric stiffeners. The total capacity of W12×26 SC-E0 was 148 kips, and the capacity of W12×26 SC-NA was 79 kips, resulting in an effective stiffener capacity of 69 kips. Figure 10 shows a close-up elevation of the column specimen after testing. The front stiffener buckled locally near the top flange and close to the concentrated load. Significant yielding was clearly observed in the stiffener prior to buckling, and therefore, the failure mode is best described as inelastic stiffener buckling. Figure 10 also shows local flange buckling and patterns of strain lines in the web and mill scale peeling off.

Double Compression Test Results

Twelve experimental column specimens were tested in double compression. This includes three wide-flange sizes and four tests per size. A W10×39 column specimen was not considered for double compression tests because preliminary finite element models revealed the capacity with stiffeners would exceed the actuator capacity. For W16×31 column specimens tested in double compression only, stiffeners near the concentrated load were only placed on one side, again due to limitations in capacity of the hydraulic actuator. A summary of the primary results from the double compression tests is provided in Table 3.

For most double compression tests, more local deformations were found near the top flange as opposed to the bottom flange. This suggests that the load was not fully transferred through the web and to the bottom reaction plate as designed. Instead, as load is applied and local deformations occur, the column specimens rotate slightly in-plane at the ends, suggesting that some of the load is transferred to the end supports. This behavior suggests that there are uncertainties associated with comparing the theoretical capacity for web compression buckling to the test capacity in Table 3. However, due to the location of the reaction plate in comparison to the end supports, as well as the condition of the reaction plate after testing, it is interpreted that most of the force was transferred through the column specimen. General observations from the results presented in Table 3 are as follows:

- The concentrated load limit state strengths calculated using AISC *Specification* Section J10 (AISC, 2016b) indicate that web compression buckling should have controlled for all column sizes when stiffeners are not present. In all cases, the maximum load of the column specimen was much higher than the predicted value. In addition, all maximum loads were higher than the theoretical capacity for web local yielding. The maximum loads for specimens without concentric stiffeners related better to the capacity predicted by web local crippling, which was the observed failure mode associated with most tests.

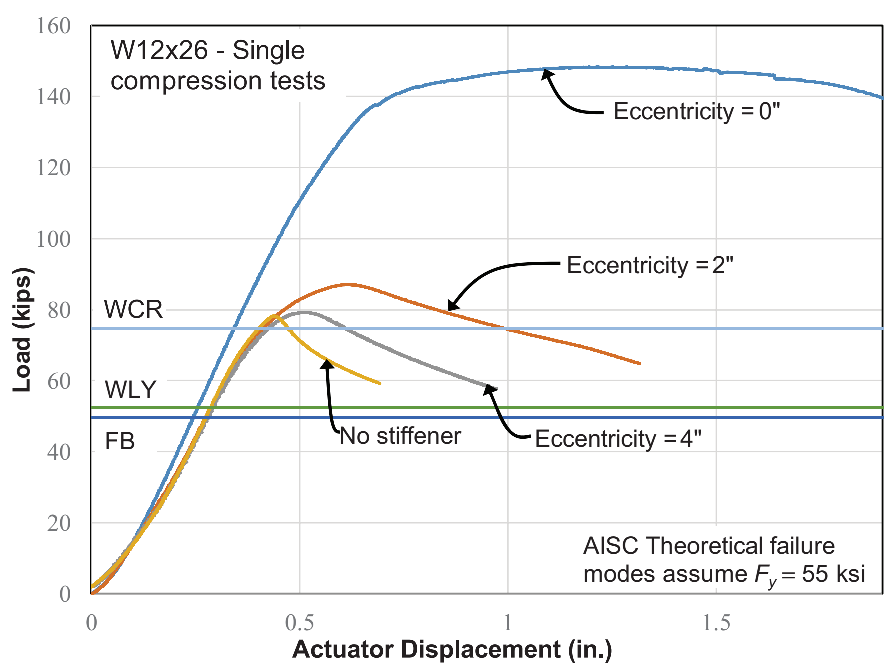


Fig. 8. Load-displacement results of W12×26 SC column specimens.

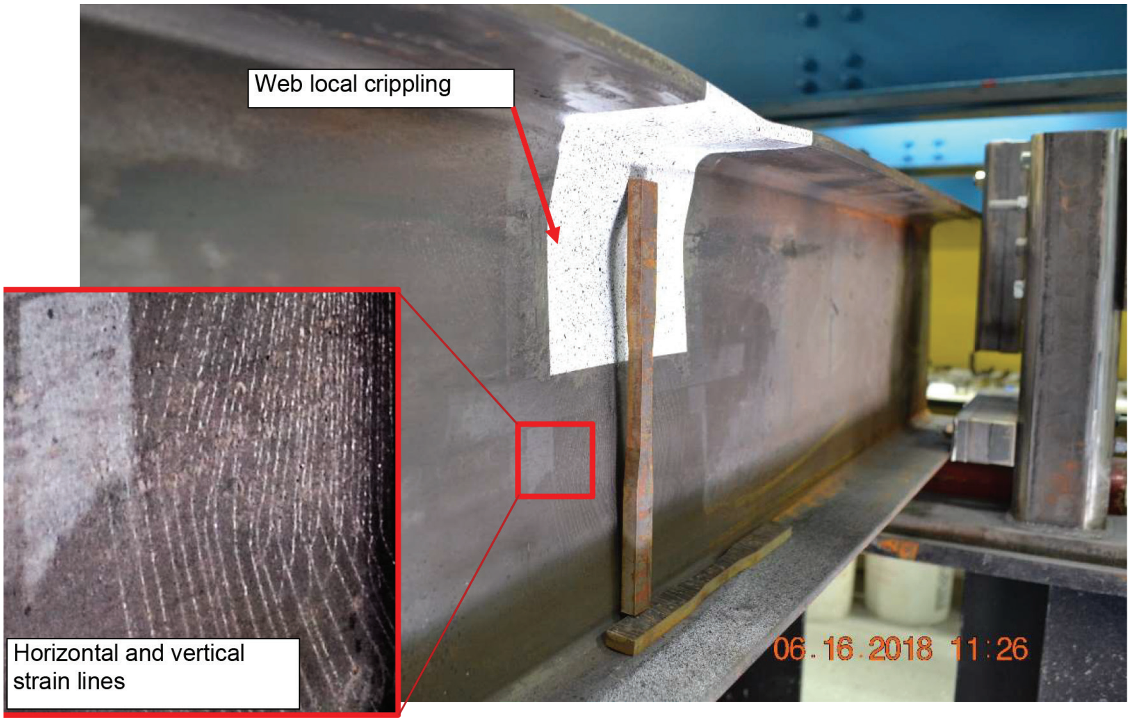


Fig. 9. W12×26 SC-NA after testing emphasizing web crippling and strain lines.

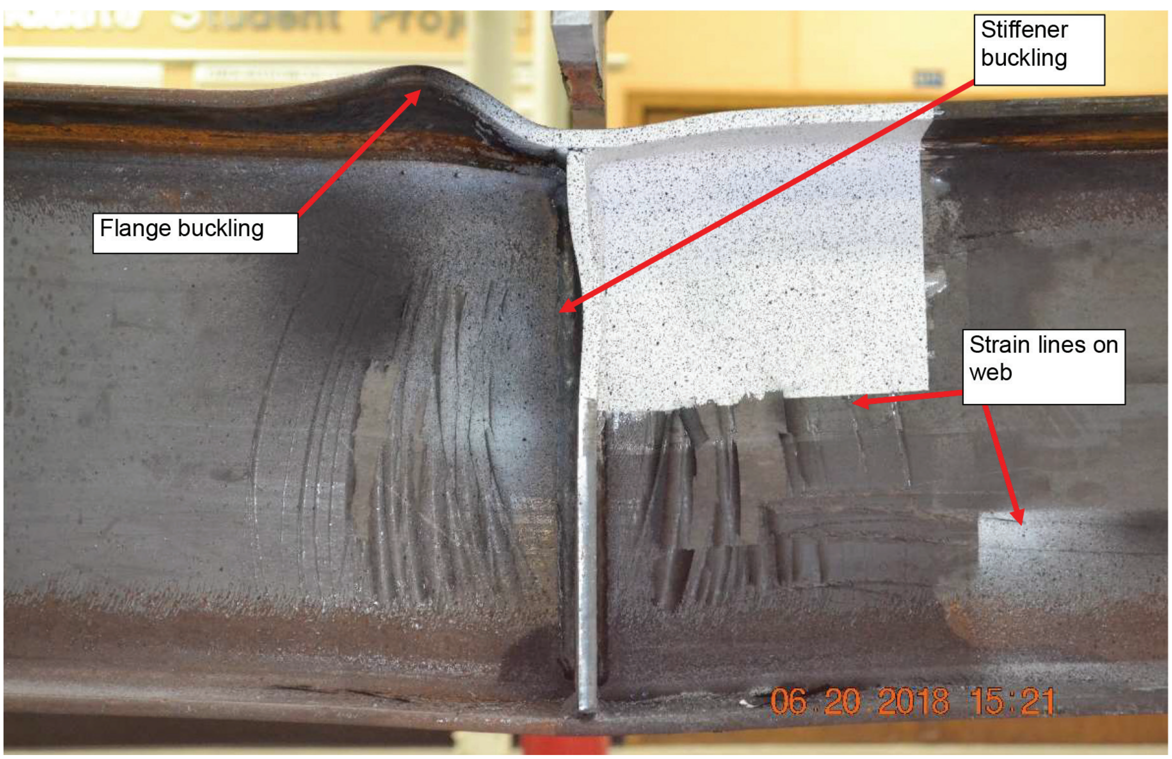


Fig. 10. Close-up view of front side of W12×26 SC-E0 after testing.

Table 3. Double Compression Theoretical Capacities and Test Results								
Column Specimen	Eccentricity (in.)	WLY (kips)	WCB (kips)	WCR (kips)	Stiffener (kips)	Theoretical Capacity (kips)	Test Capacity (kips)	Effective Stiffener Capacity (kips)
W16×31 DC-NA	NA	75.0	44.3	103	31.1	44.3	120	—
W16×31 DC-E6	6					TBD	117	–2.30
W16×31 DC-E3	3					TBD	121	1.40
W16×31 DC-E0	0					75.4	163	43.5
W12×26 DC-NA	NA	52.5	34	74.7	76.9	34	86.4	—
W12×26 DC-E4	4					TBD	90.6	4.20
W12×26 DC-E2	2					TBD	111	24.5
W12×26 DC-E0	0					111	220*	+134
W10×19 DC-NA	NA	58.1	53.7	88.2	27.0	53.7	78.5	—
W10×19 DC-E4	4					TBD	84.4	5.90
W10×19 DC-E2	2					TBD	88.0	9.50
W10×19 DC-E0	0					80.7	180	102

* Reached actuator capacity of 220 kips without failure

- The maximum loads obtained for specimens with eccentric stiffeners were usually close to the maximum loads obtained for the corresponding specimens without stiffeners. The low eccentricity condition always resulted in a higher capacity than the high eccentricity condition. For the W12×26 test set, there was a significant increase in maximum load when the low eccentricity condition was tested. However, the effective stiffener capacity of 24.5 kips was still much lower than the effective stiffener capacity for the column specimen with concentric stiffeners, which is unknown (actuator capacity was reached) but at least 134 kips. The 24.5 kips is also significantly lower than the calculated effective stiffener capacity of 76.9 kips from Table 3. For column specimens with concentric stiffeners, there was a significant increase in capacity in comparison to the condition without stiffeners. The increase in capacity was more pronounced in comparison to the single compression tests. In addition, the effective stiffener capacities were much higher than the computed theoretical values. The double compression tests primarily fail by the concentrated load effect, and the stiffeners have a greater relative influence in bracing the web against buckling.
- For the W16×31 column specimen with one concentric stiffener, the theoretical effective stiffener capacity was noticeably less than the actual effective stiffener capacity obtained from testing. When two concentric stiffeners were used with the W12×26 and W10×19 column specimens, the theoretical effective stiffener capacity was more substantially less than the actual effective stiffener capacity obtained from testing.

Behavior of Double Compression Tests

Figure 11 shows the load-displacement relationships for the W12×26 DC column specimens and emphasizes differences in experimental behavior for different stiffener conditions. As mentioned, calculated concentrated load limit states (WCR, WCB, WLY, FB) are also shown as horizontal lines in Figure 11. Because this represents a double compression test, WCB would be the controlling limit state in design as demonstrated in Table 3.

As shown in Figure 11, the experimental results of W12×26 DC-NA and W12×26 DC-E4 were very similar. Thus, for the high-eccentricity condition, the stiffeners have a minimal influence in resisting the concentrated loads. For both of these column specimens, yielding first began in the column web beneath the top loading plate. Strain lines in horizontal and vertical patterns were seen on the web under the load. Yielding continued through the web until the column reached its maximum load and suddenly failed by web crippling near the top loading plate. From visual observations, the experimental behavior of W12×26 DC-E2 was similar. However, the stiffeners were more engaged in sharing the concentrated load, and the column specimen was able to reach a much higher displacement prior to a noticeable decrease in load. After significant yielding occurred in the web and flanges, eventually the column web failed by crippling. At the conclusion of the test, substantial flange bending in both the top and bottom flanges was noticed as shown in Figure 12.

The experimental behavior of the W12×26 DC-E0 column specimen was significantly different from the behavior of the other column specimens in the test group. As shown in Figure 11, the test reached the maximum capacity of the hydraulic actuator of 220 kips, which was already much higher than that anticipated from theoretical calculations using the AISC *Specification* (AISC, 2016b). Yielding was identified in the stiffeners during the test by means of mill scale peeling off. Strain lines were then identified in the web and in both the top and bottom flanges. The authors believe from visual observation, from the DIC results, and the load displacement results, that the column specimen was nearing the actual load capacity. Figure 13 shows the column specimen after testing.

Single Tension Test Results

Twelve experimental column specimens were tested in single tension. This includes three wide-flange sizes and four tests per size. A summary of all the primary results from the single tension tests is provided in Table 4.

All four W10×39 column specimens reached the capacity of the hydraulic actuator. All column specimens with concentric stiffeners reached the maximum capacity of the hydraulic actuator. With the exception of W12×26 ST-E2 (discussed later), all other column specimens failed at the weld from the loading plate to the top flange of the column specimens. The failure of the weld was due to excessive flange bending causing nonuniform and, therefore, localized stresses in the weld. Observations from the results presented in Table 4 and from additional evidence of experimental behavior are as follows:

- The concentrated load limit state strengths determined using AISC *Specification* Section J10 (AISC, 2016b) are significantly lower than the maximum loads obtained for the column specimens. As discussed more in Part 2: Analytical Studies (Alvarez Rodilla and Kowalkowski, 2021), this conclusion is influenced by the weld size of ⅜ in. from the loading plate to the top flange of the column specimens, which is relatively thick for these column specimens. Part 2: Analytical Studies demonstrates how the thickness of the weld influences the flange bending capacity results in more detail.
- Although the maximum loads always exceeded the calculated capacity for flange bending, bending of the flange creates nonuniform stresses in the weld. When concentric stiffeners are used, this negates bending of the flange, and the welds are capable of resisting a much higher load because more uniform stresses are present.
- From the W16×31 ST test set, eccentric stiffeners have a minimal influence on the concentrated load capacity of the column specimen. However, the low eccentricity condition resulted in a slightly higher capacity than the high eccentricity condition.

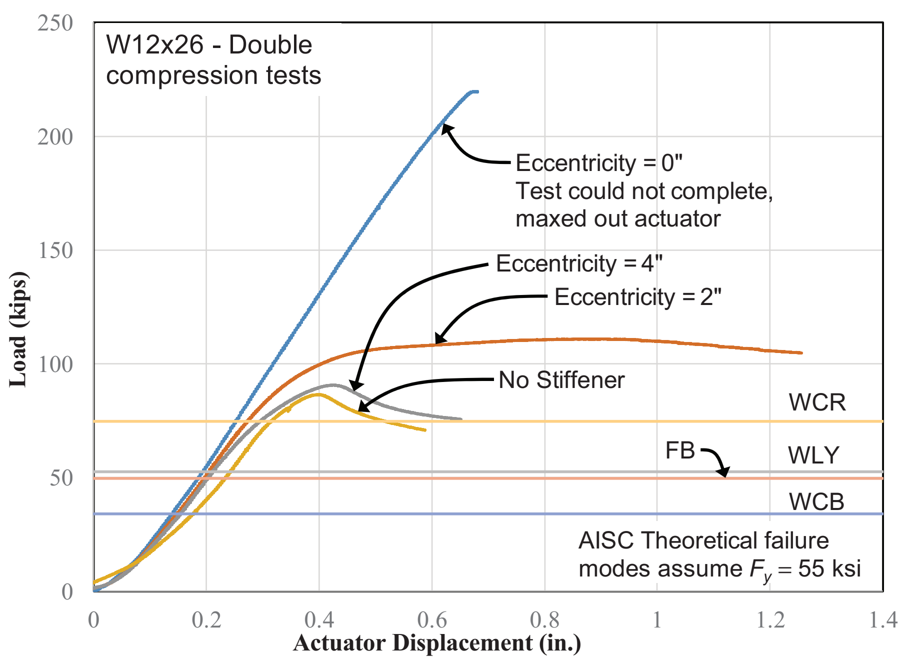


Fig. 11. Load-displacement results of W12×26 DC column specimens.

- A low-tone "bang" was heard while testing W12×26 ST-NA. The intensity of this bang was not nearly as high as expected after performing the tests on W16×31 ST members and further tests on W12×26 ST specimens. From the behavior witnessed, it appeared this test failed prematurely. For the purposes of this work, it is alleged that the capacity of W12×26 ST-NA would have been very similar to W12×26 ST-E4 if this did not occur. However, there is a notable increase in capacity when the low eccentricity condition was tested, which is similar to the comparisons for the double compression tests performed on this column size.
- For column specimens with concentric stiffeners, there is a significant increase in capacity in comparison to the condition without stiffeners, and it is assumed that if the specimens were tested to their true capacity, similar relationships would be obtained as found for the compression studies.

Behavior of Single Tension Tests

For column specimen W12×26 ST-E2, yielding began in the web below the applied load. Strain lines parallel to the direction of the applied load were identified in the web. The column specimen reached its capacity when failure occurred in the weld between the top flange and the inside face of the stiffener. Then, the crack propagated from the weld into the web and below the concentrated load. A few tension tests performed by Graham et al. (1959) also failed by web fracture just below the fillet, but it appeared

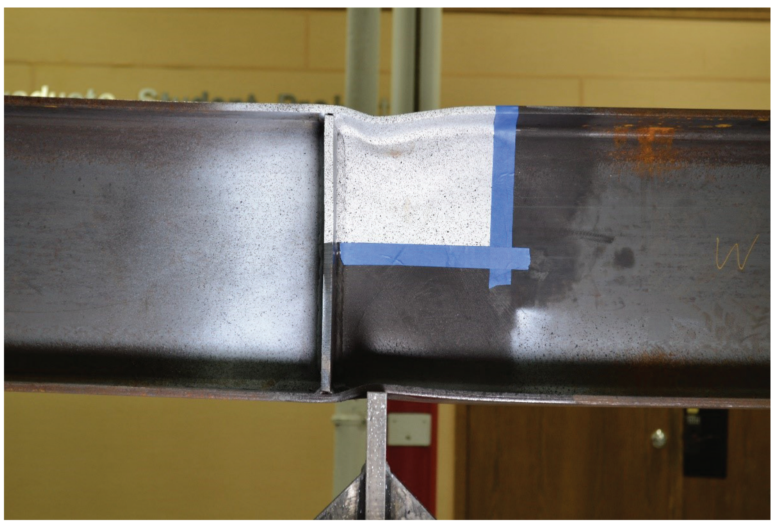


Fig. 12. Elevation view of W12×26 DC-E2 after testing.

Fig. 13. Elevation view of W12×26 DC-E0 after testing.

Table 4. Single Tension Theoretical Capacities and Test Results							
Column Specimen	**Eccentricity (in.)**	**WLY (kips)**	**FB (kips)**	**Stiffener (kips)**	**Theoretical Capacity (kips)**	**Test Capacity (kips)**	**Effective Stiffener Capacity (kips)**
W16×31 ST-NA	NA	76.9	66.6	62.5	66.6	127	—
W16×31 ST-E6	6				TBD	126	–1.50
W16×31 ST-E3	3				TBD	129	2.00
W163×1 ST-E0	0				160	144*	+17.0
W12×26 ST-NA	NA	54.1	49.6	76.9	49.6	79.2	±
W12×26 ST-E4	4				TBD	115	35.9
W12×26 ST-E2	2				TBD	134	54.5
W12×26 ST-E0	0				123	144*	+65.0
W10×39 ST-NA	NA	104	96.6	97.6	96.6	144*	—
W10×39 ST-E4	4				TBD	144*	NA
W10×39 ST-E2	2				TBD	144*	NA
W10×39 ST-E0	0				194	144*	NA

* Reached actuator capacity

from visual observations that failure initiated in the stiffener weld, and therefore, the behavior is different. Fracture through the web of column specimen W12×26 ST-E2 is shown in Figure 14.

Figure 15 shows the load-displacement relationships for the W16×31 ST column specimens and emphasizes differences in experimental behavior for different stiffener conditions. As mentioned, calculated concentrated load limit states (WLY, FB) are also shown as horizontal lines in Figure 15. For this column specimen and for all column specimens subjected to single tension, FB would be the controlling limit state in design as demonstrated in Table 4. Three of the four column specimens reached a maximum load between 125 and 129 kips. The maximum load for the column specimen with stiffeners at a high eccentricity was slightly less than for the column specimen without stiffeners. The test capacity for the column specimen with stiffeners at a low eccentricity was slightly higher than for the column specimen without stiffeners. However, all three capacities were very similar, and in general, eccentric stiffeners have negligible influence on the maximum loads for this column size and these magnitudes of eccentricity.

Figure 15 indicates that all four column specimens had a similar elastic stiffness. The test with concentric stiffeners had the highest elastic stiffness and maintained elastic behavior until the actuator capacity was reached. For specimens without concentric stiffeners, nonlinear behavior does initiate near the limit state of web local yielding (WLY). However, the column specimens reach a much higher capacity than that computed for this limit state and for flange bending (FB).

All three of the column specimens without stiffeners eventually failed by rupture of the welds between the loading plate and the top flange of the column. The weld failure occurred as a loud and sudden bang with limited warning. Weld failure was influenced by excessive flange bending, which creates higher stresses in localized areas of the weld. The outside tips of the flange move more vertically with the loading plate because the localized stiffness of the flange is smaller near the tips. At the center of the weld, the flange is less prone to move with the plate because the web on the backside of the flange resists it. Therefore, higher stresses develop near the center of the welds along the length.

Visual inspection of the tests and the load-displacement behavior show that eccentric stiffeners do not effectively resist flange bending, and therefore, uneven stresses still develop in the welds. A photograph of W16×31 ST-E6 that shows the weld failure and emphasizes the large, localized deformations that occur is provided in Figure 16.

Figure 17 shows an elevation of W16×31 ST-E6 after testing and identifies inelastic deformations that occurred in the web and flange. Figure 17 also identifies the approximate "width" of flange bending (FB width). Yielding occurs in the flange within this width where significant out-of-plane deformations occur in the flange. For this column specimen, the eccentric stiffeners were not found within this effective width and, therefore, did not resist the flange bending effect.

The column specimen W16×31 ST-E0 with concentric stiffeners reached the maximum capacity of the actuator at 144 kips. There were no signs of yielding or other significant deformations when the actuator capacity was reached, and it is likely that the load capacity comparisons in single tension would be similar to the results in single compression. The primary difference in the concentric stiffener condition is that load is transferred directly to the stiffeners, which stiffen the flange from bending. The entire weld will therefore develop more uniform deformations across the width of the flange upon loading. The maximum capacity of the connection would likely be closer to the nominal capacity of the welds themselves, which is computed as 185 kips using AISC *Specification* Section J2 (AISC, 2016b).

CONCLUSIONS

The following are the primary conclusions from all of the experimental investigations considering single compression, double compression, and single tension test results. The conclusions are based on the smaller column sizes that were tested experimentally. Note that the research is further evaluated in Part 2: Analytical Studies (Alvarez Rodilla and Kowalkowski, 2021), which provides more in-depth analysis of larger column specimens.

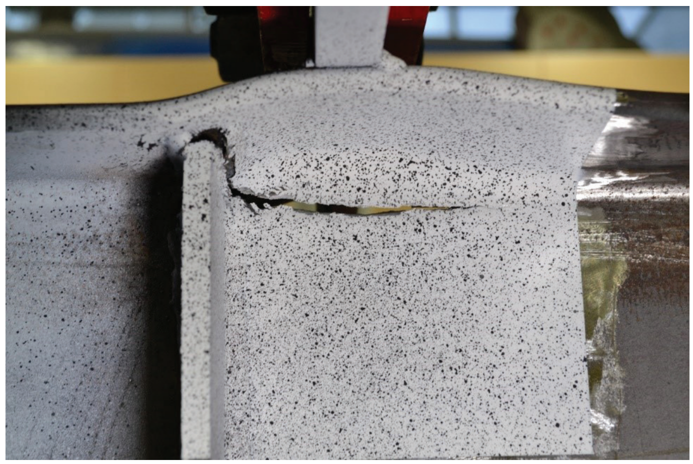


Fig. 14. W12×26 ST-E2 after testing with weld and web fracture.

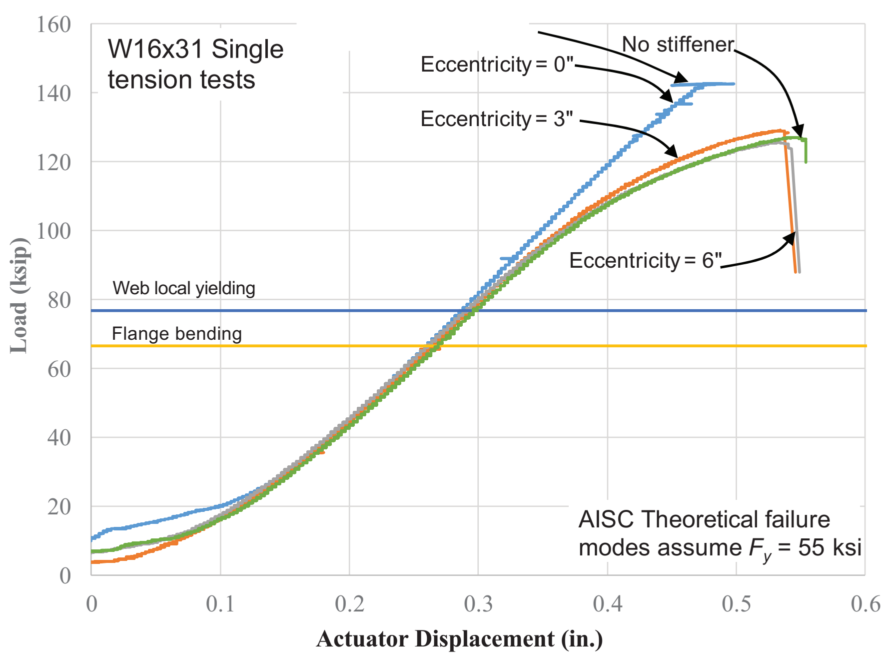


Fig. 15. Load-displacement results of W16×31 ST column specimens.

- All single and double compression tests without concentric stiffeners reached a maximum load when web local crippling occurred, causing a sudden decrease in load. The maximum load generally compared favorably to that calculated for this limit state using the AISC *Specification* (AISC, 2016b) (the theoretically governing limit state was computed as web local yielding for single compression and web compression buckling for double compression).
- For compression tests with concentric stiffeners, the maximum load was generally obtained when inelastic stiffener buckling occurred. The maximum load for single compression results was also influenced by inelastic flexural and shear stresses in the cross section from moment and shear, respectively.

Fig. 16. Weld fracture of W16×31 ST-E6 after testing (side 1).

- For single tension tests, when the actuator capacity was not reached, failure generally occurred in the weld from the loading plate to the top flange of the column specimen due to nonuniform weld stresses between the loading plate and top flange.
- For all test methods, when column specimens are tested without stiffeners, the maximum load is much higher than that predicted by the limit state of web local yielding. However, from visual inspection and load-displacement results, yielding of the web does occur prior to reaching the load carrying capacity.
- For single tension tests without stiffeners, the maximum loads obtained significantly exceeded the capacity calculated for the limit state of flange bending.
- For double compression tests without stiffener, the maximum loads obtained significantly exceeded the capacity calculated for the limit state of web compression buckling.
- In all tests, the effective stiffener capacity for column specimens with eccentric stiffeners was between 0% and 18% (not using negative effective stiffener capacities) of the effective stiffener capacity for column specimens with concentric stiffeners.
- Information in AISC Design Guide 13 (Carter, 1999), which suggests that the capacity of eccentric stiffeners is 65% of that of concentric stiffeners at an eccentricity of 2 in., is not adequate for these smaller column sizes.

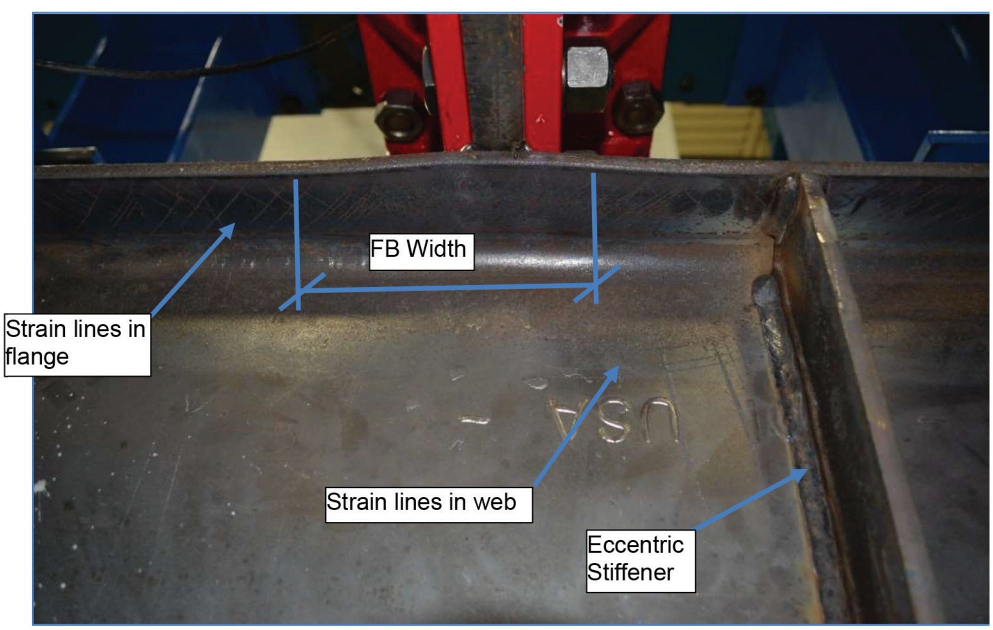


Fig. 17. Elevation view of W16×31 ST-E6 after testing.

LIMITATIONS

A limitation of the experimental studies presented in this paper is that only small wide-flange sections were considered due to limitations in capacity of the hydraulic actuator. Limited valuable test results were obtained for the single tension loading condition since one weld failed prematurely and several other tests reached the capacity of the hydraulic actuator. Larger column sizes were modeled analytically and provide further clarity on the influence of eccentric stiffeners as presented in Kowalkowski and Alvarez Rodilla (2019) and in Part 2: Analytical Studies (Alvarez Rodilla and Kowalkowski, 2021). Testing of larger column specimens was necessary to complete the scope of the project and to come up with recommendations that can be applied in design. All final recommendations are presented in Part 2: Analytical Studies.

Comparisons between the experimental results and theoretical capacities were made assuming a yield stress of 55 ksi for the experimental beam specimens. Tension tests were performed on tension coupons with the yield stress ranging from 52 to 59 ksi. However, an accurate yield stress could not be designated for every beam specimen.

ACKNOWLEDGMENTS

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