Experimental Investigation into the Capacity of Concentrically Loaded Steel Connections with Pretensioned High-Strength Bolts and Longitudinal Fillet Welds in Combination

Christopher D. Waite, Ligang Shen, Mohamed Soliman, and Bruce W. Russell

ABSTRACT

This paper presents the results of an experimental study aiming to investigate the behavior of steel connections that combine pretensioned high-strength bolts and longitudinal fillet welds on a common faying surface. A total of 75 double-shear tension splices were tested under direct tension loading to quantify the effect of various connection variables on the load-deformation behavior of the connection. These variables include the (1) bolt pattern (2×2 and 2×3), (2) bolt size (¾ in. and 1 in.), (3) bolt grade (ASTM F3125 Grade A325, A490, and F1852), (4) bolt pretensioning method (turn-of-nut and tension control bolts), (5) faying surface class (Class A and B), and (6) weld/bolt strength ratio. The variation in the connection characteristics covered a wide range of weld/bolt strength ratios from 0.50 to 2.00. The bolts were installed in oversized holes, and the specimens were assembled in a negative bearing condition to allow for a maximum slip distance. The load-deformation behavior of the combination connections was recorded and compared to that of the bolted- and welded-only control specimens. In all tests, the addition of welds increased the capacity of the connection. The investigation shows that the capacity of the combination connection with pretensioned high-strength bolts and longitudinal fillet welds can be computed by adding the capacities of the individual connecting elements while considering the strain compatibility.

Keywords: pretensioned high-strength bolts, slip-critical connection, fillet weld, combination connection, double-shear, tension splice, steel connection, experimental testing.

INTRODUCTION

Structural steel connections have been traditionally designed and constructed as either bolted or welded. The need to supplement a bolted connection with welds may arise during retrofit and strengthening of existing structures or in an effort to accommodate a change in design loads after fabrication. Although a weld tensile coupon can exhibit significant deformation, welded connections are generally considered to be stiffer than snug-tightened bolted connections. As a result, if snug-tightened mechanical fasteners are combined with welds in a single load sharing system, the welds may reach their ultimate capacity within a very small deformation that is not sufficient for bolts to fully engage in the force transfer. Accordingly, the current American Institute of Steel Construction (AISC) *Specification for Structural Steel Buildings* (2016), herafter referred to as the AISC *Specification*, does not allow snug-tightened bolts to be combined with welds. This situation may be exacerbated if transverse welds are used in combination with snug-tightened bolts given the significant decrease in ductility of connections using this weld orientation. Furthermore, the load-deformation behavior of different connecting elements in the elastic range (i.e., stiffness) may not enable the direct addition of the various capacities (Miller, 2001, 2002).

Experimental investigations to quantify the capacity and load-deformation characteristics of connections utilizing bolts and welds in combination began in the late 1960s. One of the earliest known studies into combination connections is highlighted in the *Guide to Design Criteria of Bolted and Riveted Joints*, 2nd Ed. (Kulak et al., 2001). The authors discuss an experimental study by Steinhardt et al. (1969) into the load-deformation behavior of small tension butt splices with bolts and welds in combination. This early study concluded that the connection capacity can be predicted as the sum of the individual bolted-only slip load and the ultimate load of the welded-only connection.

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Perhaps the largest research body in literature aiming at characterizing the behavior of combination connections can be attributed to the efforts made by Dr. Kulak and his co-workers. Holtz and Kulak (1970) started this endeavor by investigating how connection variables—that is, weld orientation (longitudinal or transverse), bolt pretension, and bolt hole clearance—influenced the connection performance. Their testing program included three different configurations of double-lap connections that vary the aforementioned connection variables. Although the combination connections implementing transverse welds reached higher capacities than connections with longitudinal welds, the researchers advised against the use of transverse welds in combination connections due to the limited ductility of connections utilizing this weld orientation. Longitudinal welds, however, showed a higher deformation capacity when combined with bolts. The connections without bolt hole clearances provided higher factors of safety than the connections with standard bolt hole clearances. However, these direct bearing bolt conditions are not representative of typical steel construction when standard holes are used.

The effect of weld orientation was further studied through an additional experimental investigation by Jarosch and Bowman (1986). The researchers tested tension splices with pretensioned high-strength bolts employed with varying weld orientation (longitudinal and/or transverse). Again, the study recommended that transverse welds should not be combined with pretensioned high-strength bolts due to the limited ductility of this weld orientation. Additionally, they showed that the frictional resistance of the bolts may not contribute to the overall connection capacity when combined with transverse welds. For their tested connections implementing longitudinal welds, the ultimate capacity was conservatively predicted by summing the weld shear strength and the bolt slip force.

Further investigations were continued by Manuel and Kulak (2000) to study the effect of weld orientation, bolt pretension (pretensioned or snug-tight), and bolt bearing condition. The researchers defined two bearing conditions: positive bearing and negative bearing. For negative bearing, the connection is assembled in a way that allows the bolts to slip over a distance equal to twice the hole clearances before the bolts would engage in bearing. Positive bearing bolts would engage in bearing immediately when load is applied. Similar to previous research, the authors recommended that transverse welds should not be combined with pretensioned high-strength bolts due to ductility limitations of the connections with this weld orientation. The frictional resistance of tested connections was noticeable in the experimental data but not clearly understood. For connections with bolts in positive bearing, certain connections displayed a capacity increase that reached 81% compared with the capacities achieved with the negative bearing condition. The following model was proposed to estimate the ultimate capacity of a combination connection:

$$R_{ult} = R_{friction} + R_{bolts} + R_{trans.welds} + R_{long.welds} \tag{1}$$

In this model, when pretensioned bolts are utilized, the frictional contribution, $R_{friction}$, is equal to 25% of the total slip resistance of the bolts. When it is certain that the bolts are in negative bearing or when transverse welds are used, the resistance provided by the bolt shear strength, R_{bolts}, is removed from the equation. When the bolts are in positive bearing or intermediate bearing (middle of the hole), R_{bolts} is 75% or 50% of the bolt shear strength, respectively. Lastly, when both transverse and longitudinal welds are used together, the longitudinal weld shear strength, $R_{long.welds}$, is reduced to 85% of the weld shear strength. $R_{trans.welds}$ is equal to the weld shear strength of the transverse welds.

Additional research by Kulak and Grondin (2003) and Sato (2000) sought to understand how the randomness in the bolt bearing condition influenced the accuracy of the model presented in Equation 1. Their testing program included nominally similar connections with pretensioned high-strength bolts and longitudinal welds. During the connection assembly, the bolt bearing condition was not controlled. To test the effectiveness of the model, the bolt bearing condition was classified as intermediate bearing. The model predicted the connection capacities with an average error of 2.4%.

Shi et al. (2011a, 2011b) investigated the ultimate capacity of combination connections both experimentally and numerically. The researchers studied combination connections with pretensioned high-strength bolts in combination with longitudinal and transverse welds. They concluded that the ultimate capacity may be dependent on the ratio a between the bolt slip capacity, $R_{friction}$, and the longitudinal weld capacity, $R_{long.welds}$. The following stepwise model was developed to predict the capacity:

$$R_{ult} = \begin{cases} R_{long.welds} & \text{for } a < 0.5 \\ 0.75R_{long.welds} + R_{friction} & \text{for } 0.5 \le a < 0.8 \\ 0.9R_{long.welds} + 0.8R_{friction} & \text{for } 0.8 \le a \le 2 \\ R_{long.welds} + 0.75R_{friction} & \text{for } 2 \le a < 3 \\ R_{friction} & \text{for } a \ge 3 \end{cases}, \quad a = \frac{R_{friction}}{R_{long.welds}} \tag{2}$$

More recently, a study by Kim and Lee (2020) sought to understand how the steel grade, bolt bearing condition, and weld orientation influenced the performance of combination connections. They concluded that the steel grade had little effect on load-deformation behavior of the connection. Similar to previous research conducted by Manuel and Kulak (2000), connections with positive bearing bolts resulted in higher capacities. The researchers proposed the following capacity equation for connections with pretensioned high-strength bolts and longitudinal welds:

$$R_{ult} = 0.8R_{bolts} + UR_{long.welds},\ U = \begin{cases} 1.0 & \text{for } L \geq 2W \\ 0.87 & \text{for } 2W > L \geq 1.5W \\ 0.75 & \text{for } 1.5W > L \geq W \end{cases} \quad (3)$$

In this model, R_{bolts} is the ultimate strength of the bearing-type bolted connection, and $R_{long.welds}$ is the ultimate shear strength of the welds. Any frictional resistance provided by the connection was neglected to be conservative. The weld strength contribution is multiplied by the shear lag factor, U. This factor is characterized by the variables L and W, which represent the connection length and plate width, respectively.

AISC *Specification* Section J1.8 currently provides guidelines for connections using bolts and welds in combination. The specification allows combining pretensioned high-strength bolts and longitudinal fillet welds in shear connections with common faying surface. The available strength is permitted to be taken as the sum of the bolt slip capacity and the longitudinal fillet weld strength. The specification imposes limitations on the percentage of force carried by the bolts and welds given the bolt pretensioning method.

The AISC *Specification* nominal slip resistance of the bolts, R_{nb}, is defined as:

$$R_{nb} = \mu D_u h_f T_b n_s$$

(AISC *Spec*. Eq. J3-4)

in which μ is the mean slip coefficient for Class A or B surfaces; D_u is a multiplier that reflects the ratio of the mean installed bolt pretension to specified minimum bolt pretension, taken as 1.13; h_f is a factor for filters; T_b is the minimum fastener pretension force; and n_s is the number of slip planes in the connection.

The AISC *Specification* nominal shear strength of the weld, R_{nw}, is defined as:

$$R_{nw} = F_{nw} A_{we}$$

(AISC *Spec*. Eq. J2-3)

where $F_{nw} = 0.6\ F_{EXX}$ for fillet welds (from Table J2.5) and A_{we} is the effective fillet weld area. F_{EXX} is defined as the filler metal classification strength. The effective fillet weld area, A_{we}, is equal to the effective weld length multiplied by the effective weld throat. The throat is the shortest distance from the weld root to the face of the fillet weld.

Although previous research provides several models for predicting the capacity of connections with bolts and welds in combination, the combined connection behavior is still not fully understood. For instance, the contribution of the bolt slip capacity is not well characterized. While Shi et al. (2011b) recommended using the slip capacity as the bolt contribution, Kim and Lee (2020) advocated against its use, and the model proposed in Manuel and Kulak (2000) uses only 25% of this friction force. It is also apparent in literature that the positive bearing condition led to a higher connection capacity; however, it may not be practical to specify that combination connections must be assembled in a positive bearing condition. Based on this discussion, it seems that the effects of the plate steel grade and weld orientation are well understood. Furthermore, there exist additional connection variables that may also influence the behavior of the combination connection. These include bolt pattern, bolt size, bolt grade, pretensioning method, and faying surface class. Additionally, further experimental work is necessary to fully understand and quantify the influence of the weld/bolt strength ratio. The comprehensive experimental testing program discussed herein investigates these connection variables.

EXPERIMENTAL TESTING PROGRAM

In this paper, the behavior of connections with bolts and welds in combination was studied experimentally through a testing program encompassing 75 double-shear tension splice connections. The connections were loaded in a direct tension test frame that was designed and constructed for this study. This section illustrates the test specimens as well as the experimental methods used for the research program.

Test Connection Matrix and Specimens

The connections included in this study are separated into groups according to the connection bolt pattern and faying surface class. These connections are highlighted in the test connection matrix depicted in Table 1. Of the total connections tested, 30 were either bolted only or welded only. These tests are classified as ancillary and were utilized to establish experimental material characteristics such as the bolt pretension, slip coefficient, and weld shear stress. The remaining 45 connections in the test matrix combine pretensioned high-strength bolts and longitudinal fillet welds. For these connections, the following test variables were investigated: bolt size, bolt grade, pretensioning method, faying surface class, and, more importantly, the weld/bolt strength ratio. Each test consisted of three connection samples to better characterize the statistical variability of the capacity and how it is affected by the variability of various input parameters. As will be seen later in this paper (e.g., Figure 8), due to this variability, several connections would show a capacity that is equal to or higher than other connections with larger weld size. Accordingly, simply testing one specimen from each configuration would have not provided data that enables understanding the combinational behavior. Further, the proper consideration of this variability allows for predicting the reliability of these connections. The connection samples in the test series were named A, B, and C (e.g., 1A, 1B, 1C.). Additional samples were added to Test 1 and Test 16 to better understand the randomness

	Test No.	Bolt Pattern	Bolt Grade	Bolt Pretensioning Method	Faying Surface Class	Weld Geometry	Weld/Bolt Strength Ratio	Number of Samples
Bolted only	1	2×2	A325	ToN	B	—	—	5
	2	2×2	A325	ToN	A	—	—	3
	3	2×2	A490	ToN	B	—	—	3
	4	2×2	A325	TC	A	—	—	3
	5	2×2	A490	ToN	A	—	—	3
Welded only	6	—	—	—	—	5⁄16 × 3.0	—	3
Bolted and welded	7	2×2	A325	ToN	B	5⁄16 × 5.0	1.50	3
	8	2×2	A325	ToN	B	5⁄16 × 2.25	0.67	3
	9	2×2	A325	ToN	B	5⁄16 × 3.5	1.00	3
	10	2×2	A325	TC	B	5⁄16 × 2.25	0.67	3
	11	2×2	A490	ToN	B	5⁄16 × 2.75	0.67	3
	12	2×2	A325	ToN	A	5⁄16 × 1.25	0.67	3
	13	2×2	A325	ToN	A	5⁄16 × 2.0	1.00	3
	14	2×2	A325	ToN	A	5⁄16 × 3.0	1.50	3
	15*	2×2	A325	ToN	A	5⁄16 × 3.5	1.00	3
Bolted only	16	2×3	A325	ToN	A	—	—	5
Welded only	17-2	—	—	—	—	5⁄16 × 2.0	—	3
	17-4	—	—	—	—	5⁄16 × 4.0	—	2
Bolted and welded	18	2×3	A325	ToN	A	5⁄16 × 2.0	0.67	3
	19	2×3	A325	ToN	A	5⁄16 × 3.0	1.00	3
	20	2×3	A325	ToN	A	5⁄16 × 4.0	1.33	3
	21	2×3	A325	ToN	A	5⁄16 × 6.25	2.00	3
	22	2×3	A325	TC	A	5⁄16 × 3.0	1.00	3
	23	2×3	A490	ToN	A	5⁄16 × 2.0	0.50	3

Table 1. Test Connection Matrix

NOTE: All bolts are ¾ in. diameter (oversized holes) unless noted otherwise.
TC = tension control bolt; ToN = turn-of-nut method
Four fillet weld lines of the specified geometry per connection. Units are inches.
* Bolts are 1-in.-diameter Grade A325 in oversized holes.

of the steel friction coefficient. Test 17 also included additional samples to record the experimental load-deformation behavior of welds with different lengths.

The test matrix in Table 1 outlines the specific variables studied for each connection type. All connections utilized ¾-in.-diameter bolts, except Test 15 where 1 in. bolts were used. For the bolt grade, either A325 or A490 pretensioned high-strength bolts were used, which represent ASTM F3125 Grade A325 and Grade A490 (ASTM, 2021b), respectively. The pretensioning methods included both the turn-of-nut method (ToN) as well as tension control (TC) bolts (i.e., ASTM F3125 Grade F1852). All bolts used in the test program are Type 1 bolts. Class A connections utilize plates with clean mill scale, while Class B connections are classified as SSPC-SP6 commercial blast-cleaning in the AISC *Steel Construction Manual* (2017), hereafter referred to as the AISC *Manual*. The Class B plates were blast-cleaned by the steel fabricator and were free of all visible rust, mill scale, paint, and foreign matter. All welded-only and combination connections include four equal length longitudinal fillet welds, with a 5⁄16 in. weld leg size, using a E70 weld electrode. A ⅛ in. Lincoln Electric E7018 H4R weld electrode was used with the shielded metal arc welding (SMAW) process. Each weld length was specifically designed to achieve a target weld/bolt strength ratio ranging from 0.50 to 2.00, depending on the test. The ratio was computed based on the nominal capacity of connecting bolted-only and welded-only connections.

Each test specimen is composed of three components: the tested connection, the anchorage zone, and the connection grip. Figure 1 shows the typical 2×2 and 2×3 test specimen and highlights the aforementioned components. The tested connection zone indicates the portion of the sample that is to be studied and corresponds to the designated test characteristics highlighted in Table 1. The bolts in this zone are pretensioned and are placed in oversized holes to allow the connection to slip over a longer distance. This leads to a better understanding of the load-deformation behavior of the joint. The bolts in the anchorage zone are placed in standard holes and work in bearing. Finally, a large pin in the connection grip provides an attachment mechanism for the specimen to the load frame to minimize loading eccentricities. The steel used for all specimen plates is ASTM A572 Gr. 50 (ASTM, 2021a).

Experimental Methodology

A direct tension load frame, shown in Figures 2 and 3, was designed and constructed for this study. The frame is self-reacting and is hung off a W30×99 top header beam that is supported on either side with supplemental framing support. The load application occurs on each side of the test specimen through a load column that is attached to the top and bottom header beams. The bottom header beam is made up of two W24×66 sections. Each load column consists of a hydraulic cylinder, load cell, and filler column made of four HSS3×3×⅜ and two end plates. Each hydraulic cylinder was retrofitted with servo valves and a linear variable displacement transducer (LVDT) to be controlled with an MTS FlexTest 60 controller. During the load application, the actuators extend simultaneously at a rate of 0.02-in./min. This rate was adopted to simulate a static loading condition and is similar to the test rate adopted by Holtz and Kulak (1970). This actuator displacement rate also ensured that the average slip rate of the connections was below 0.003-in./min as recommended by the Research Council on Structural Connections (RCSC, 2020). This extension lowers the bottom header beam and applies direct tension to the specimen.

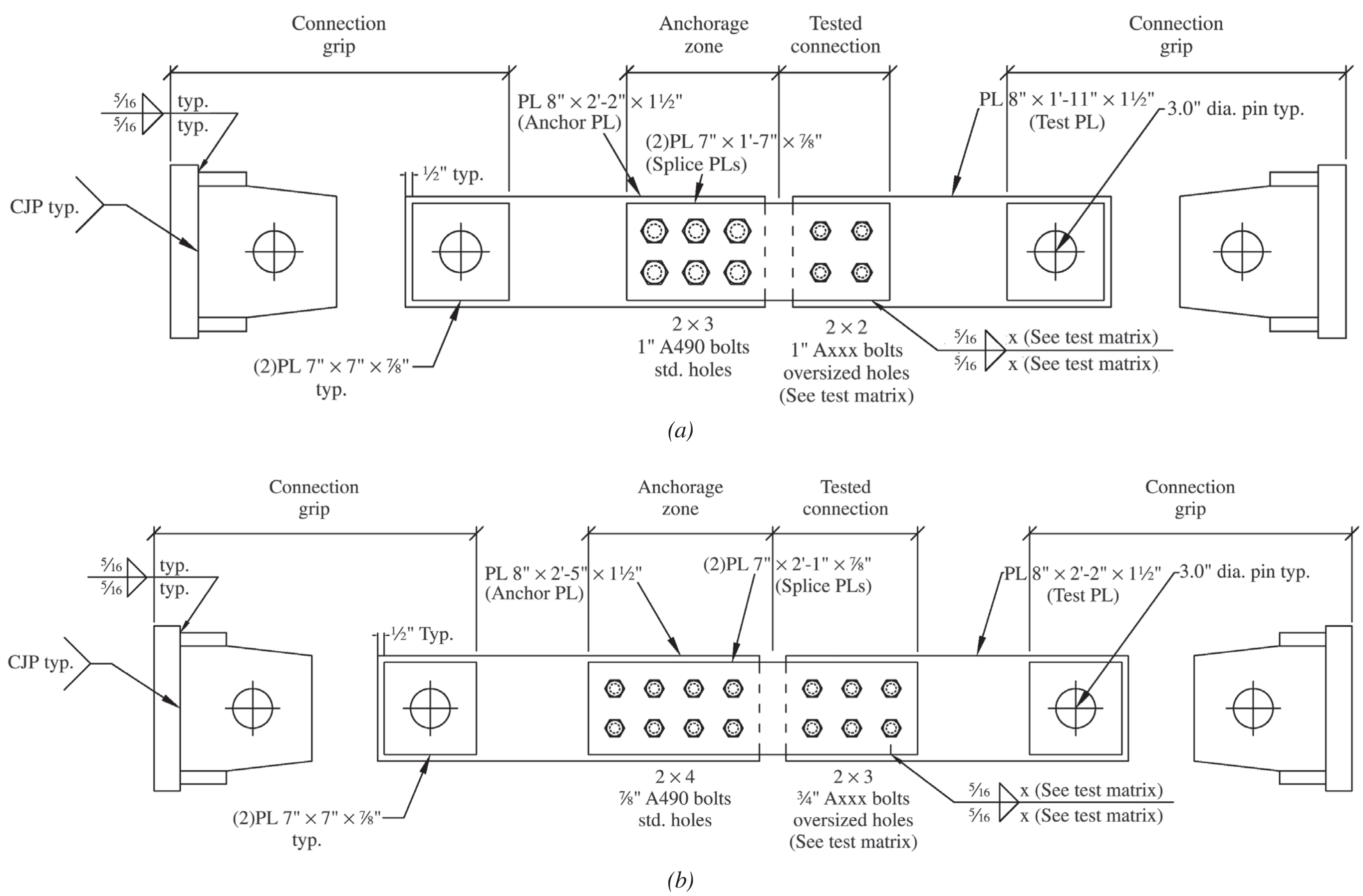


Fig. 1. Details of test specimens.

Fig. 2. Experimental test frame.

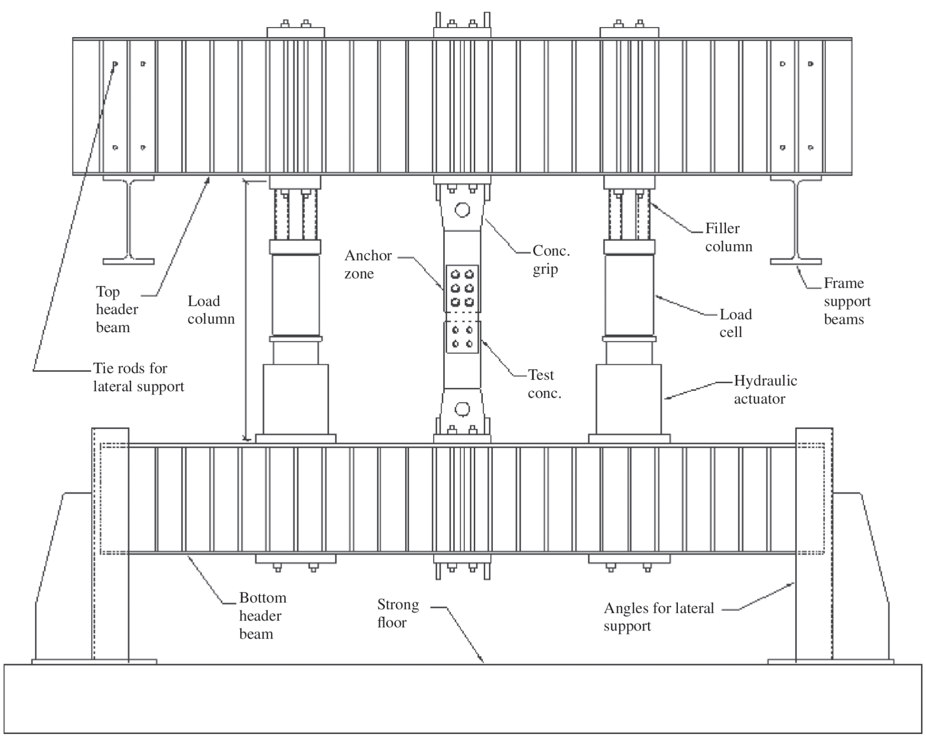


Fig. 3. Experimental test frame details.

Each connection was instrumented with bolt load cells, LVDTs, and strain gauges, as shown in Figure 4. Four high-accuracy AC-LVDTs, with a stroke of 0.20 in., measure the relative displacement between the connection components (i.e., slip) and are located at the bottom corners of each splice plate. The total slip of the connection is measured as the average of the four AC-LVDTs. Two global DC-LVDTs measure the separation of the top and bottom center plates. These devices capture the slip behavior of the connection past the limits of the AC-LVDTs (i.e., 0.20 in. of slip). Each bolt was fitted with a bolt load cell to verify that the minimum pretension was provided and to monitor the bolt pretension during the test. Finally, strain gauges were applied to the tested connection to monitor the strains during testing. Additional gauges were applied in the anchorage zone to detect any load eccentricity. A National Instruments NI-cDAQ-9178 was used in conjunction with LabVIEW NXG (NI, 2018) to record all instrumentation data.

A testing protocol was developed to ensure that all specimens were tested consistently. Before connection assembly, all faying surfaces were cleaned to remove any media that could contaminate the surface. The bolted connection in the anchorage zone was assembled in positive bearing, while the tested connection was assembled in negative bearing. Because literature indicated that connections assembled in positive bearing provide higher capacities (Manuel and Kulak, 2000), the negative bearing condition was chosen for the tested zone to provide a lower bound of the capacity and to allow for the accurate investigation of the bolt frictional contribution to the capacity and load-deformation behavior of the connection.

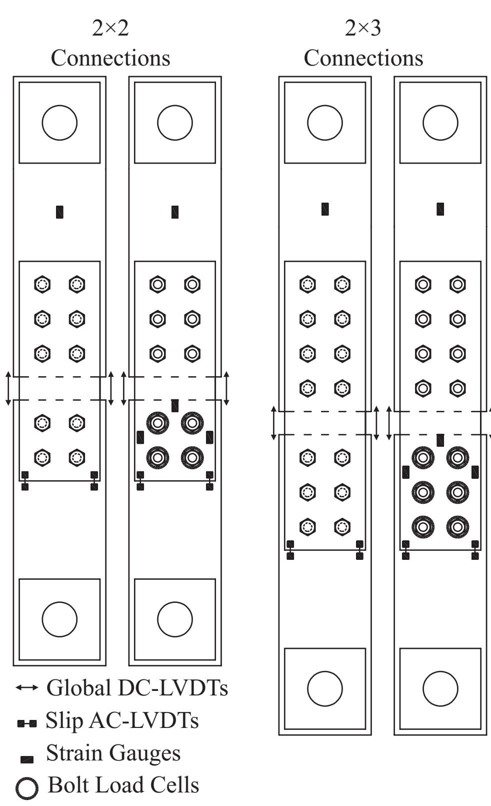


Fig. 4. Specimen instrumentation layout.

Before the test connection bolts were pretensioned, three additional bolts that represent the test bolt group (A325/A490 and ToN/TC) were tested in a bolt tension measurement device, and their pretension data was recorded. These additional tests ensured that the pretensioning equipment was operating properly and provided supplemental bolt pretension data. All ToN bolts were pretensioned with a turn-of-nut wrench, and all TC bolts were pretensioned with a shear wrench. After bolt pretensioning, the connections were welded by a certified welder according to the test matrix, and their lengths and leg dimensions were measured. The leg dimension measurements were taken at three locations along the weld length and were used to estimate the experimental effective throat of the fillet weld. An effective throat computation was adopted from Salmon et al. (2009) and accounted for unequal leg size geometry. Finally, all strain gauges and LVDTs were placed on the connection according to the instrumentation plan in Figure 4.

ANCILLARY TESTING

In order to properly evaluate the capacity of the combination connections, several ancillary tests were completed throughout the research program to establish the following experimental test variables:

T_b = bolt pretension force, kips
μ = slip coefficient of tested plates
τ = weld shear strength

These experimental test variables allow for the proper prediction of the capacity of the connection based on actual material characteristics rather than nominal values.

Pretension Evaluation

Slip-critical bolted connections rely on the frictional forces developed between the faying surfaces for strength. This resistance is both a function of the steel frictional coefficient and the bolt pretension. Throughout the testing program, 201 bolt pretension tests were conducted over a range of bolt styles, grades, and sizes. Before every connection test, three bolts were tested in a bolt tension measurement device. Moreover, the pretension force of the ¾ in. bolts was recorded using washer-type bolt load cells. The load cells were installed on the nut side. The bolt load cell model is Omega LC901-¾-65K. These load cells also come with a conical washer to center the load cell as the nut is being tightened. Both the bolt pretension tests and the bolt load

Table 2. Bolt Pretension Test Probabilistic Measurements				
	¾ in. A325-ToN	¾ in. A325-TC	¾ in. A490-ToN	1 in. A325-ToN
Number of samples	129	27	36	9
AISC minimum pretension* (kips)	28	28	35	51
Mean value (kips)	42.7	38.5	46.8	64.1
Standard deviation (kips)	1.99	2.76	2.30	3.07
NOTE: TC = tension control bolt; ToN = turn-of-nut method * Table J3.1 (AISC, 2016)				

cell measurements provided insight into the experimental bolt pretension that is applied to the connection and ensured that the pretensioning was completed properly.

During the testing program, it was noted that the pretension data from the bolt load cells displayed higher amounts of variability than the bolt pretension test data. For example, the bolt tension measurement device recorded an average pretension of 42.7 kips with a standard deviation of 1.99 kips for the A325 ToN bolt group, whereas the bolt load cells recorded an average pretension of 39.3 kips with a standard deviation of 5.42 kips for this same bolt group. After benchmarking the results of the bolt load cells against those obtained from the bolt pretension test of the bolts from the same lot, it was determined that the higher variability in these measurements can be attributed to the load cell measurement accuracy. Because no probabilistic analysis is performed to account for this variability in this paper, it was decided to use the bolt pretension readings in the capacity prediction models presented herein. The variability in bolt load cell data was properly captured in developing a probabilistic approach to investigate the reliability of these connections in Khandel et al. (2022). Table 2 shows the experimental bolt pretension data from the bolt pretension tests. The mean experimental pretension values were used to evaluate the steel fictional coefficient as well as the predicted capacity of the combination connections.

Bolted-Only Tests and Friction Coefficient Evaluation

The testing program included 22 bolted-only connection samples that were used to evaluate both the steel frictional coefficient used in the study, as well as develop a baseline for bolted-only connection behavior. The test characteristics of the bolted-only connections correspond to Tests 1–5 and Test 16 in the connection matrix shown in Table 1. The RCSC (2020) provides typical load-slip curves for slip-critical bolted connections. These curves are illustrated in Figure 5 and were adopted to determine the slip load of the connections. The slip load corresponds to the maximum load before 0.02 in. of slip for connections following Case a, the load before sudden slip for Case b, and the load at 0.02-in. of slip for Case c.

The bolted-only connections with Class A surfaces displayed load-slip behaviors similar to Case c while Class B surfaces displayed a slip response similar to Case a. However, it should be noted that two of the 2×3 Class A specimens displayed a behavior similar to Case a. The slip load for each bolted-only connection was identified based on these outlined behaviors. Typical bolted-only load-slip curves are presented in Figure 6 for Class A and Class B surfaces (Tests 5 and 3, respectively) and the experimentally obtained connection capacities, denoted by Test R_n in this paper, are shown in Table 3. Note that the deformation levels at which slip occurs are significantly lower than those occurring at the failure of a bearing-type bolted connection. Using the slip load for each test connection, the slip coefficient, k_s, is computed for each sample as:

$$k_s = \frac{slip\ load}{2 \times clamping\ force} \qquad \text{(RCSC Eq. A3.1)}$$

where the *clamping force* is equal to the average bolt pretension pretension force (see Table 2) multiplied by the number of bolts used in the connection. Table 3 presents a summary of this computation for each bolted-only connection. Note that separate computations for the 2×2 Class A and 2×3 Class A surfaces were required due to the different behavior observed during the bolted-only tests for the two groups of plates. The plates used for the 2×2 Class A surfaces displayed a more uniformly textured oxide layer than the plates used for the 2×3 Class A connections. The properties of the oxides layer (e.g., uniformity, chemical composition, adhesion, etc.) may have led to this difference in the load-slip behavior of the connections.

For each faying surface group, the experimental slip coefficient, μ, was identified as the average of all individual slip coefficients, k_s, for connections in the group. The slip coefficients for both the 2×2 and 2×3 Class A surfaces were higher than the AISC *Specification* minimum of 0.3. The 2×2 Class A surface was found to have an average slip coefficient of 0.457. The test data was very consistent with a standard deviation of 0.022 and coefficient of variation of 4.87%. Unlike the 2×2 Class A friction data, the 2×3 Class A data displayed high variability with a mean slip

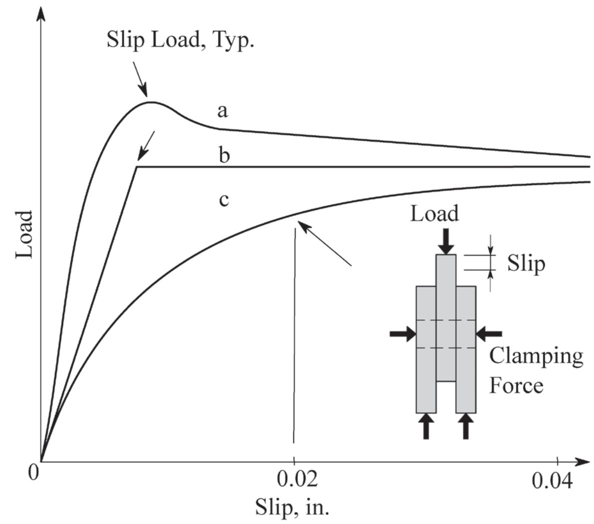


Fig. 5. RCSC load-slip definition (RCSC, 2020).

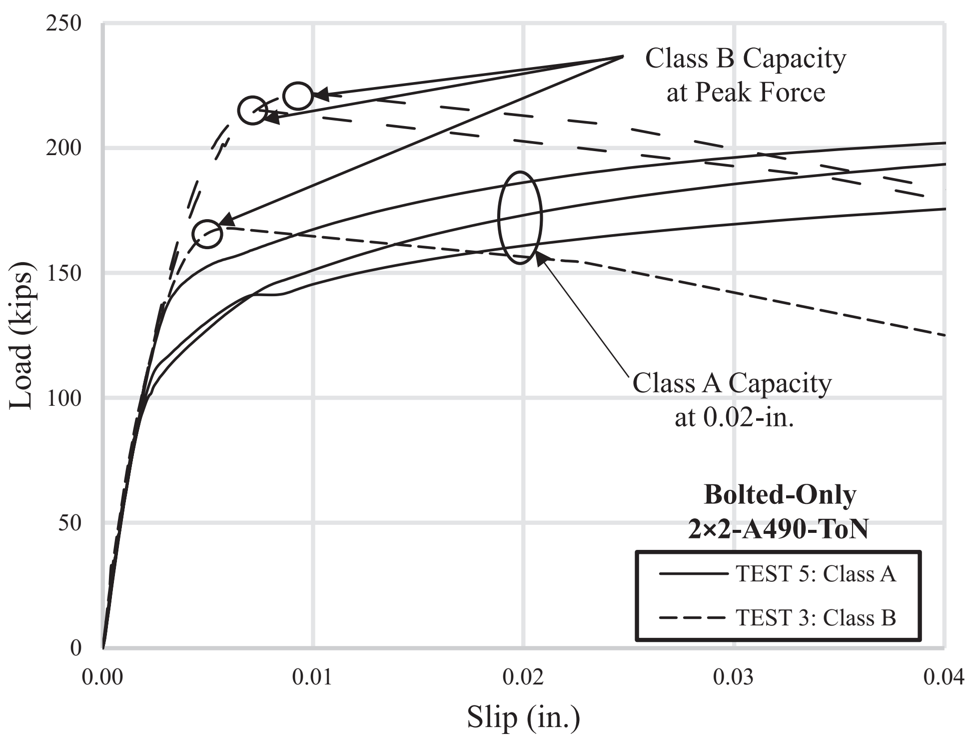


Fig. 6. Bolted-only experimental load-slip curves.

Table 3. Slip Coefficient Evaluation						
Faying Surface	**Bolt Type**	**Test**	**Bolt Pretension (kips)**	**Clamping Force (kips)**	**Test Rn (kips)**	**Slip Coefficient k_s**
2×2 Class A	A325 ToN	2A	42.7	171	153	0.446
		2B			158	0.462
		2C			145	0.424
	A325 TC	4A	38.5	154	143	0.465
		4B			146	0.474
		4C			139	0.452
	A490 ToN	5A	46.8	187	173	0.462
		5B			161	0.430
		5C			186	0.497
						AVG = 0.457 **SD = 0.022** **CV = 4.87%**
2×2 Class B	A325 ToN	1A	42.7	171	189	0.552
		1C			142	0.414
		1D			181	0.530
		1E			217	0.636
	A490 ToN	3A	46.8	187	168	0.448
		3B			222	0.592
		3C			215	0.575
						AVG = 0.535 **SD = 0.079** **CV = 14.8%**
2×3 Class A	A325 ToN	16B	42.7	256	170	0.331
		16C			169	0.329
		16E			183	0.357
NOTE: All bolts are ¾ in. diameter (oversized holes) unless noted otherwise. TC = tension control bolt; ToN = turn-of-nut method AVG = average; SD = standard deviation; CV = coefficient of variation						**AVG = 0.339** **SD = 0.016** **CV = 4.61%**

coefficient of 0.382 and resulted in a standard deviation of 0.079 and a coefficient of variation of 16.3%. To improve confidence in the slip coefficient prediction for the 2×3 Class A surface, a two-tailed Z-test was conducted using slip coefficient data described in Grondin et al. (2007) where a mean value of 0.301 was reported for Class A surfaces. For the Z-test, the null hypothesis was that the mean value of the slip coefficient calculated from the experimental test data was equal to the value reported by Grondin et al. (2007); an alternative hypothesis was that the two values were not equal. Two samples were rejected (16D and 16F) from the 2×3 group with a 90% significance level based on the hypothesis test. A mean slip coefficient of 0.339 was calculated based on the remaining three tests.

Similar to the Class A surfaces, the Class B surface produced an experimental slip coefficient that exceeded the AISC *Specification* minimum of 0.5, but with higher variability. The 2×2 Class B surface displayed a slip coefficient of 0.535 with a standard deviation of 0.079 and coefficient of variation of 14.8%. Note that specimen Test 1B was removed from the study due to a faying surface contamination (hydraulic oil) that would be unlikely to occur under typical construction field conditions. The slip coefficient computed in this ancillary test was found to be very close to the mean value of 0.524 reported by Grondin et al. (2007); however, additional statistical analysis was performed to gain confidence in the slip coefficient prediction. A similar two-tailed Z-test was conducted for the Class B surface data and a p-value was computed as 0.764. Accordingly, the obtained experimental mean was considered to belong to the population distribution. Although these tested Class B bolted-only connections included only four bolts, the slip level at which the first slip event occurred was found to be comparable to experimental results of bolted connection with 32 bolts reported in Borello et al. (2009). These experimental slip coefficients were used to evaluate and predict the slip contribution into the combination connections capacity.

Welded-Only Tests and Weld Shear Strength Evaluation

In addition to the bolted-only connection tests, eight welded-only tests were conducted to evaluate the experimental weld shear strength. These tests include Tests 6 and 17 in the connection test matrix and cover weld lengths of 2, 3, and 4 in. The experimental load-deformation curves for the welded connections are depicted in Figure 7. These curves show similar profiles compared to those reported by Lesik and Kennedy (1990). The ultimate capacity of these connections corresponds to the maximum load sustained during the test.

To compute the experimental weld shear stress, AISC *Specification* Equation J2-3 was adopted and rearranged to solve for the stress. In this equation, R_{nw} corresponds to the test connection ultimate capacity and A_{we} to an effective fillet weld area. This effective fillet weld area is equal to the measured fillet weld length multiplied by the average effective throat. The average effective throat was computed from the weld measurements of all the welded-only and combination tests and was found to be 0.194 in. with a standard deviation of 0.012 in. and a coefficient of variation of 6.13%.

The welded-only connection data highlighted in Table 4 provides the weld shear stress computation for each individual connection. Overall, this data concluded that the experimental weld shear strength is approximately 69.5 ksi with a standard deviation of 3.77 ksi and a coefficient of variation of 5.42%. This weld shear stress is roughly 30% higher than previous experimental work reported in literature for the similar E70 filler metal (Manuel, 1996). The high shear stress computed from the welded-only test data can be attributed to the higher mechanical properties of the weld electrodes used in the study and the use of the pre-fracture measured effective weld area. The higher mechanical properties of the weld electrodes were confirmed by the results of two weld coupons that were fabricated and tested according to AWS B4 (2016). These weld specimens showed a yield stress of 74 ksi and ultimate stress of 83 ksi. As noted in the literature (e.g., Deng et al., 2003), the actual fracture area of the weld is approximately 27% larger than the effective pre-fracture area. A similar value was also observed in this research program. Accordingly, using the actual fracture area would lead to a significantly lower shear stress value. However, since the pre-fracture area is

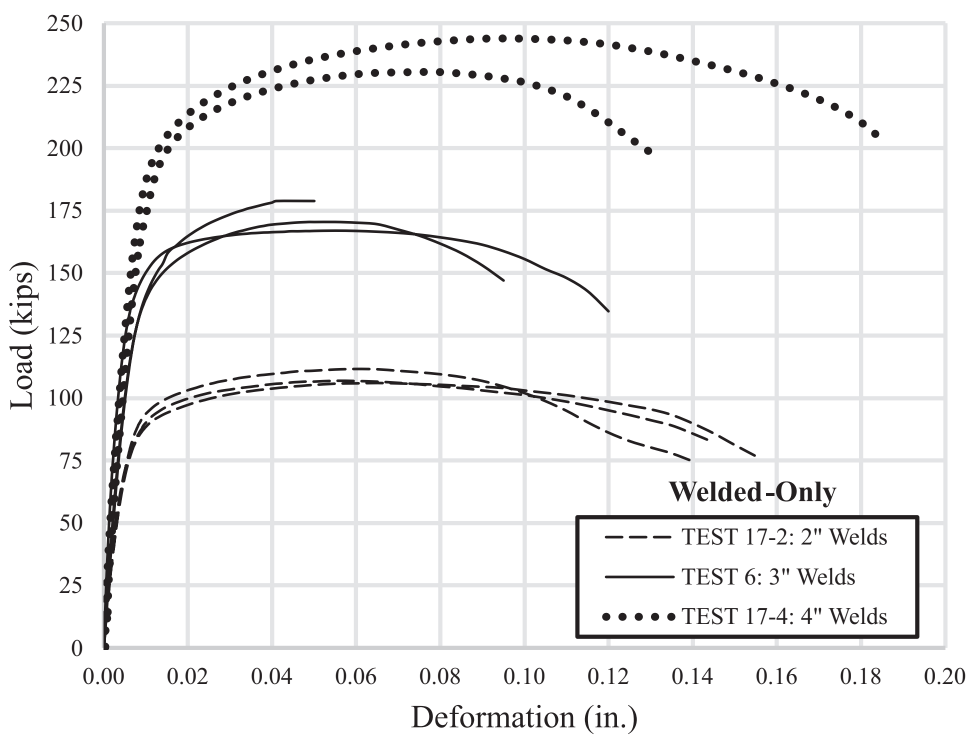


Fig. 7. Welded-only experimental load-deformation curves.

Table 4. Weld Shear Strength Evaluation				
Weld Size	**Test**	**Effective Throat Area (in.^{2})**	**Test Rn (kips)**	**Weld Shear Stress (ksi)**
5⁄16 × 3 in.	6A	2.49	181	72.5
	6B	2.47	170	68.9
	6C	2.36	167	70.8
5⁄16 × 2 in.	17-2A	1.65	107	65.0
	17-2B	1.64	112	68.0
	17-2C	1.65	106	64.2
5⁄16 × 4 in.	17-4D	3.23	231	71.4
	17-4E	3.23	244	75.4
NOTE: Four fillet weld lines of the specified geometry per connection. Units are inches. Average effective weld throat: 0.194 in.				**AVG = 69.5** **SD = 3.77** **CV = 5.42%**

normally reported in the connection design, it was decided to use the 69.5 ksi as the ultimate weld shear stress in conjunction with the pre-fracture area for capacity calculation.

COMBINATION CONNECTIONS TESTING RESULTS AND CAPACITY PREDICTION

A total of 45 connections were tested to investigate the capacity and load-deformation (i.e., load-slip) behavior of connections utilizing bolts and welds in combination. These tests correspond to Tests 7–15 and Tests 18–23 in Table 1. The combination tests are grouped based on their bolt pattern and faying surface as 2×2 Class A, 2×2 Class B, and 2×3 Class A. Using the known properties of the connecting elements (i.e., bolt pretension, slip coefficient, average effective throat, and weld shear stress), a model can be constructed to predict the connection capacity. Figures 8(a)–(c) show, respectively, the load-deformation behavior of the 2×2 Class A (i.e., Tests 12–14), 2×2 Class B (i.e., Tests 7–9), and the 2×3 Class A (i.e., Tests 18–21) connections.

Combination Connection Capacity Prediction

The predicted capacity of the combination tests is highlighted in Table 5 as the As-Built R_n. This value incorporates the results of the ancillary tests with the current AISC *Specification* model for capacity prediction of combination connections. This capacity is computed as:

$$R_n = R_b + R_w \tag{4}$$

where R_b is the slip resistance of the bolted components and R_w is the weld shear strength. AISC *Specification* Equation J3-4 is adopted for the slip resistance, R_b, calculation; however, the term D_u is omitted because the actual bolt pretension from Table 2 is used. The mean slip coefficient, μ, is equal to the values highlighted in Table 3 based on the connections bolt pattern and faying surface class. AISC *Specification* Equation J2-3 is adopted for the weld shear strength computation, R_w, where the weld shear stress, F_{nw}, is equal to 69.5 ksi based on the data in Table 4, and the effective fillet weld area, A_{we}, is equal to the average effective throat multiplied by the measured weld lengths for the individual connection.

For determining the connection capacity based on the experimentally obtained load-deformation profiles (i.e., Test R_n reported in Table 5), it was decided to follow the RCSC (2020) curves depicted in Figure 5. Because these connections may fundamentally be slip-critical bolted connections in need of retrofit, it is essential to limit the slip in the connections to prescribed RCSC limitations. The deformation level at which the slip capacity occurs in a slip-critical bolted connection varies widely depending on the faying surface condition. For Class A surface, some bolted-only connection continued to carry force at displacement levels well beyond 0.02-in. However, connections with other Class A surfaces (i.e., 2×3 plates) and Class B surfaces slipped at very low displacement levels, and the ultimate capacity occurred at less than 0.02 in. of slip. Accordingly, it may be difficult to provide a reliable prediction of the force carried by the bolted connection at slip levels higher than 0.02 in.

Accordingly, the ultimate capacity, Test R_n, is taken as the maximum force carried by the connection at or before 0.02 in. of slip. For Class A connections, this capacity represents, on average, 87% of the maximum load carried by the connection based on the load-slip curves. The ultimate capacity of connections with Class B faying surfaces occurs on average at 0.017 in. of slip. For each combination connection test, the strength ratio, ρ, of the AISC prediction model, Test R_n/As-Built R_n, was computed, both for the

Table 5. AISC As-Built Capacity Prediction						
	Test	**Connection Variables**	**Average As-Built R_n (kips)**	**Average Test R_n (kips)**	**Average Strength Ratio ρ**	**Group Strength Ratio ρ**
2×2 Class A	Test 12	A325 ToN ratio: 0.67	235	241	1.02	**AVG = 0.977 SD = 0.054 CV = 5.55%**
	Test 13	A325 ToN ratio: 1.00	278	262	0.94	
	Test 14	A325 ToN ratio: 1.50	321	309	0.96	
	Test 15*	A325 ToN ratio: 1.00	439	352	0.80	—
2×2 Class B	Test 7	A325 ToN ratio: 1.50	460	470	1.02	**AVG = 1.07 SD = 0.106 CV = 9.94%**
	Test 8	A325 ToN ratio: 0.67	321	348	1.09	
	Test 9	A325 ToN ratio: 1.00	376	391	1.04	
	Test 10	A325 TC ratio: 0.67	289	331	1.15	
	Test 11	A490 ToN ratio: 0.67	355	377	1.06	
2×3 Class A	Test 18	A325 ToN ratio: 0.67	292	266	0.91	**AVG = 0.958 SD = 0.069 CV = 7.25%**
	Test 19	A325 ToN ratio: 1.00	335	323	0.97	
	Test 20	A325 ToN ratio: 1.33	394	355	0.90	
	Test 21	A325 ToN ratio: 2.00	518	490	0.95	
	Test 22	A325 TC ratio: 1.00	327	321	0.98	
	Test 23	A490 ToN ratio: 0.50	306	319	1.04	
NOTE: *Test 15 uses 1-in.-diameter bolts and is not included in the 2×2 Class A group statistics.						

individual test series and connection group, to evaluate the efficacy of the current model.

2×2 Class A Connections

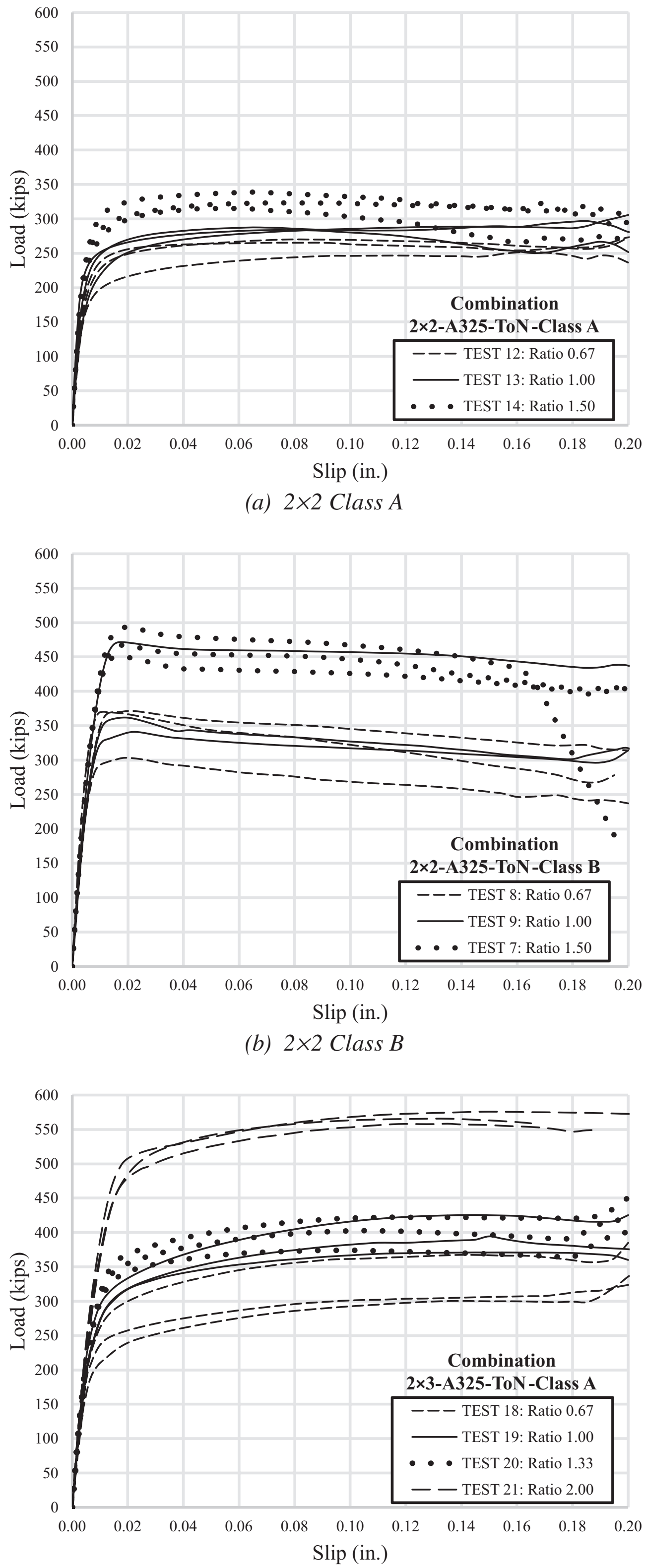
In the 2×2 Class A connection group, 12 specimens were tested. These correspond to Tests 12–15 shown in Table 1. The 2×2 Class A specimens isolate the weld/bolt strength ratio and aim to understand its effects on the connection behavior. The ratios studied in this test group are 0.67, 1.00, and 1.50 for Tests 12, 13, and 14, respectively. These connections use A325 bolts and are pretensioned with the ToN method. To complement this specimen group, Test 15 includes bolts that are 1 in. in diameter to provide insight into the effect of larger bolts.

Figure 8(a) displays the load-deformation curves for Tests 12–14. The behavior of these connections follows the profile outlined by Case c in Figure 5. Therefore, the experimental capacity, Test R_n, for these connections corresponds to the sustained load at 0.02-in. of slip. Test 15 performed in a similar manner, and the Test R_n was also recorded at this slip level. The Test R_n for these connections is highlighted in Table 5. The overall strength ratio ρ for the connections utilizing ¾ in. bolts, Tests 12–14, was 0.977 with a standard deviation of 0.054 and a coefficient of variation of 5.55%. Meaning, on average, the AISC model slightly overpredicted the capacity of the connection. The AISC model also overpredicted the capacity of the connections with 1 in. bolts, Test 15, but at a much higher margin. These connections display a strength ratio, ρ, of 0.80 with a standard deviation of 0.082 and a coefficient of variation of 10.2%. It should be noted, however, that none of the connections failed under the AISC *Specification* predicted capacity using nominal material properties.

2×2 Class B Connections

A total of 15 specimens were tested in the 2×2 Class B combination connection group. These connections correspond to Tests 7–11. Similar to samples in the 2×2 Class A connection group, Tests 7–9 specifically study the effect of the weld/bolt strength ratio. The connections are constructed with ratios of 0.67, 1.00, and 1.50, respectively, and utilize A325 bolts that are pretensioned with the ToN method. The specimens in Test 10 study the effect of the bolt pretensioning method by using TC bolts. Test 11 includes connections with A490 bolts that are pretensioned with the ToN method to understand the effect of higher bolt grades. Both Tests 10 and 11 are constructed with weld/bolt strength ratios of 0.67.

The load-deformation curves for Tests 7–9 are plotted in Figure 8(b). The connections display behaviors similar to that of Case a in Figure 5. Tests 10 and 11 also showed a similar behavior. Therefore, the Test R_n for this connection group is reported in Table 5 as the maximum sustained load before 0.02 in. of slip. The overall strength ratio ρ for



(c) 2×3 Class A

Fig. 8. Combination connection load-slip curves.

this connection group was 1.07 with a standard deviation of 0.106 and a coefficient of variation of 9.94%.

2×3 Class A Connections

The 2×3 Class A connection group is the largest of the combination groups and contains 18 samples. These connections correspond to Tests 18–23 in Table 1. Tests 18–21 vary the weld/bolt strength ratio similar to the other connection groups but include 1.33 and 2.00 ratios. These specimens are constructed with A325 bolts that are pretensioned with the ToN method. The last two test series, Tests 22 and 23, respectively, study the effect of the bolt pretensioning method by using TC bolts, and higher bolt grades with A490 bolts. Test 22 is constructed with a weld/bolt strength ratio of 1.00 and Test 23 with a ratio of 0.50.

The load-deformation curves for Tests 18–21 are depicted in Figure 8(c). Similar to the 2×2 Class A connections, the 2×3 Class A connections show a behavior that closely follows Case c in Figure 5. The Test R_n for these connections is thus considered as the sustained load at 0.02 in. of slip. These values can be found in Table 5. Tests 22 and 23 also followed this behavior, and the same methodology was used to report the Test R_n. The overall strength ratio ρ for the connection group was 0.958 with a standard deviation of 0.069 and a coefficient of variation of 7.25%.

AISC Model Efficacy

Based on the experimental behavior and the analysis of the prediction results of the combination connections, it seems that the current AISC model may overpredict the available strength of certain connection groups. Although the average overprediction is minor (e.g., 4.2% for the 2×3 Class A connections), certain test series even exhibited a 10% overprediction (e.g., Test 20). This overprediction may be attributed to the lack of consideration of strain compatibility within the prediction model. Figure 9(a) further illustrates this observation by analyzing the independent load-deformation curves (bolted only and welded only) that make up the combination connections of Test 14. For this connection, the average of the following experimentally obtained curves are plotted: (A) bolted only, (B) welded only, and (C) combination. The figure also includes a profile that shows the arithmetic summation of curves A and B. As shown, the average experimental behavior of the combination connection given by Curve C can be reasonably approximated by the summation of the individual contributions of the connecting elements (i.e., Curve A + Curve B). This is especially true at lower slip levels. However, for computing the combination connection capacity, the current prediction model adds the friction capacity of bolts, computed at 0.02 in. of slip, to the ultimate weld capacity, which occurs at higher deformation levels; accordingly, strain compatibility may not be properly accounted for.

A similar behavior to that reported in Figure 9(a) was observed for most of the tested connection series. However, for some connections—mostly belonging to the 2×3 Class A group—the summation of the individual element contributions led to an unconservative prediction of the combined connection behavior. An example of these connections is shown in Figure 9(b). As shown, the summation (i.e., Curve A + Curve B) yielded a higher capacity than the average test results (i.e., Curve C). This mainly occurred for Test 18 and 20 and can be attributed to the high variability associated with the friction coefficient of the 2×3 Class A faying surfaces.

Proposed Capacity Model

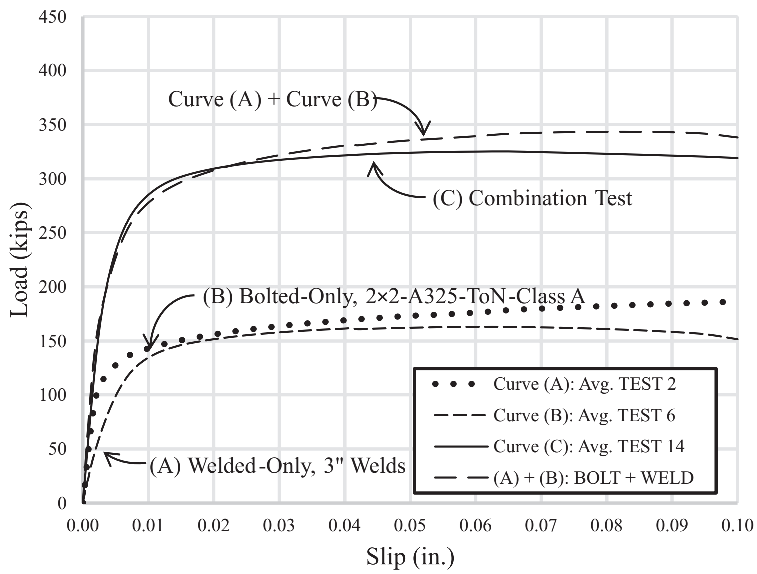
A prediction model that accounts for the strain compatibility between the connection elements can be achieved by identifying the weld shear stress at 0.02 in. of slip. This will allow for a better prediction of the combination connection capacity. For the welded-only connections tests, the average weld shear stress associated to this deformation level was found to be 64.0 ksi. Furthermore, the ratio of the weld shear stress at 0.02 in. of deformation over the ultimate weld shear stress was computed to be 0.92 for the 5⁄16 in. welds used in the testing program. A prediction model accounting for this strain compatibility between the bolt slip resistance and the weld shear strength can then be expressed as:

$$R_n = R_b + C_w R_w \tag{5}$$

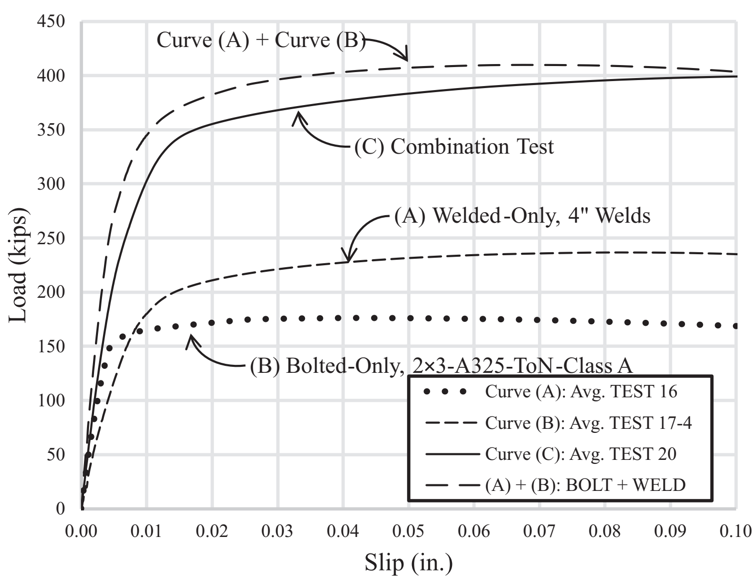
where C_w is the ratio of the weld shear stress at 0.02 in. of slip to the ultimate weld shear strength.

Based on the experimental results reported in Table 4, the C_w factor in Equation 5 is equal to 0.92 for the studied combination connection using 5⁄16 in. welds. The efficacy of the proposed capacity prediction model is shown in Figure 10. For all connections plotted, the model provides an average strength ratio ρ of 1.04. For each connection group, the model shows appropriate levels of conservatism with strength ratio ρ of 1.01, 1.11, and 0.998, for 2×2 Class A, 2×2 Class B, and 2×3 Class A connections, respectively. This data is highlighted in Table 6. The coefficient of variation is 5.29%, 9.73%, and 7.01% for the three respective connection groups considered herein.

It is noted that the capacity prediction model presented in Equation 5, specifically with $C_w = 0.92$, only applies to the tested connections given the weld dimensions measured during the experimental analysis. The tested welds were completed in an ideal environment by a highly trained certified welder. Furthermore, research reported by Lesik and Kennedy (1990) suggests that the load-deformation behavior of welded connections can depend on the weld leg size. The load-deformation prediction model in Lesik and Kennedy captures this effect and is adopted in AISC *Manual* Part 8 to compute the capacity of welds using the instantaneous



(a) 2×2 Class A



(b) 2×3 Class A

Fig. 9. Combination prediction load-slip curves.

Table 6. Proposed Model Capacity Prediction						
	Test	**Connection Variables**	**Average Model R_n (kips)**	**Average Test R_n (kips)**	**Average Strength Ratio ρ**	**Group Strength Ratio ρ**
2×2 Class A	Test 12	A325 ToN ratio: 0.67	229	241	1.05	**AVG = 1.01 SD = 0.053 CV = 5.29%**
	Test 13	A325 ToN ratio: 1.00	268	262	0.98	
	Test 14	A325 ToN ratio: 1.50	308	309	1.01	
	Test 15*	A325 ToN ratio: 1.00	423	352	0.83	**—**
2×2 Class B	Test 7	A325 ToN ratio: 1.50	438	470	1.07	**AVG = 1.11 SD = 0.108 CV = 9.73%**
	Test 8	A325 ToN ratio: 0.67	310	348	1.12	
	Test 9	A325 ToN ratio: 1.00	361	391	1.08	
	Test 10	A325 TC ratio: 0.67	279	331	1.19	
	Test 11	A490 ToN ratio: 0.67	343	377	1.10	
2×3 Class A	Test 18	A325 ToN ratio: 0.67	282	266	0.94	**AVG = 0.998 SD = 0.070 CV = 7.01%**
	Test 19	A325 ToN ratio: 1.00	322	323	1.00	
	Test 20	A325 ToN ratio: 1.33	376	355	0.94	
	Test 21	A325 ToN ratio: 2.00	491	490	1.00	
	Test 22	A325 TC ratio: 1.00	313	321	1.02	
	Test 23	A490 ToN ratio: 0.50	297	319	1.07	

NOTE: * Test 15 uses 1-in.-diameter bolts and is not included in the 2×2 Class A group statistics.

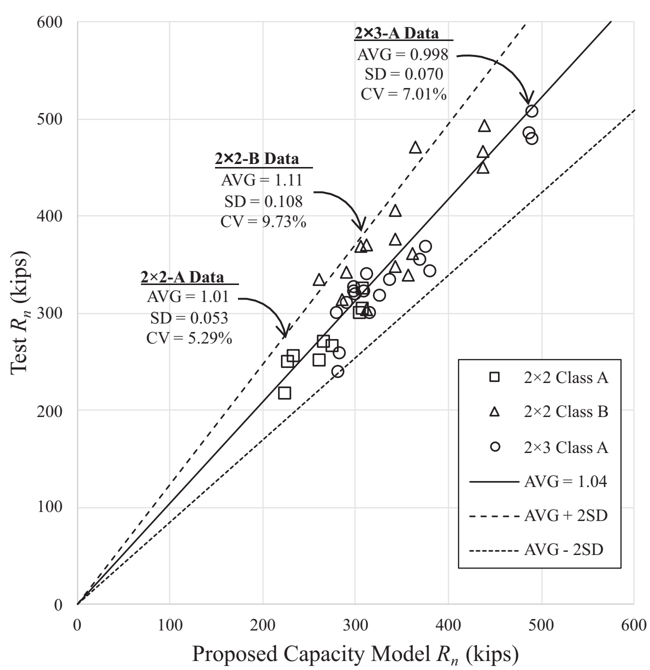


Fig. 10. Proposed capacity model prediction efficacy.

Table 7. C_w Factor (using Equation 8)	
Weld Size (in.)	**C_w Factor**
⅛	0.995
3⁄16	0.949
¼	0.899
5⁄16	0.855
⅜	0.818
7⁄16	0.787
½	0.761

center of rotation method. This model can be used to establish a relationship that represents the ratio between the weld strength at 0.02 in. of slip to the ultimate strength as a function of the fillet weld leg size. The deformation of a weld element at the ultimate strength Δ_u is (Lesik and Kennedy, 1990; AISC, 2017):

$$\Delta_u = 0.209(\theta + 2)^{-0.32}\, w \tag{6}$$

where θ is the weld orientation and w is the weld leg size. Using AISC *Manual* Equation 8-3, the ratio of the weld strength at a specific deformation, Δ, to the ultimate strength can be computed as:

$$f(\Delta) = \left[\frac{\Delta}{\Delta_u}\left(1.9 - 0.9\frac{\Delta}{\Delta_u}\right)\right]^{0.3} \tag{7}$$

By setting Δ equal to 0.02 in., the value of C_w can be found as a function of the weld leg size as:

$$C_w = f(0.02\text{ in.}) = \left(\frac{0.227w - 0.013}{w^2}\right)^{0.3} \tag{8}$$

Adopting Equation 8 for the 5⁄16 in. weld leg size used in the study would result in a C_w equal to 0.855. This value seems lower than the experimentally obtained value of 0.92; however, with the average measured weld leg size of 0.275 in. in the experimental program, the value of C_w obtained by Equation 8 is 0.880, which is within a 4% difference from the experimental value. Accounting for the effect of weld leg size becomes more important as it increases. For connections with ½ in. welds, the C_w factor is further reduced to 0.761 according to Equation 8. However, limited experimental data exists to validate this result for larger fillet welds. Experimental data on welds mostly report the ultimate capacity rather than the full load-deformation behavior. Accordingly, more experimental work is needed to properly characterize the load-deformation of large fillet welds. Until more data is available, it is recommended to compute a C_w factor using the formulation provided by Equation 8. Table 7 presents the C_w factor for various typical weld leg sizes ranging from ⅛ in. to ½ in.

THE INFLUENCE OF DIFFERENT CONNECTION VARIABLES

The results reported in Figure 8 and Table 6 highlight the ultimate capacity and load-deformation behavior of combination connections. The following discussion provides insight into how critical connection variables influence the connection performance. The variables considered herein are the bolt pattern, bolt size, bolt grade, bolt pretensioning method, faying surface class, and weld/bolt strength ratio.

Bolt Pattern

Two types of connection sizes were included in the test matrix to study the influence of the bolt pattern, 2×2 and 2×3. The results of both configurations can be seen in Figures 8(a) and 8(c). Both connection sets use A325 bolts pretensioned with the ToN method and are constructed with Class A faying surfaces. Although the plates for both groups have the same steel grade, they came from different heats, which affected their surface friction characteristics. As seen in Figures 8(a) and 8(c), the two groups displayed similar load-deformation behaviors. After the elastic region, the 2×2 plates reached their ultimate capacity at an average slip of 0.090 in., whereas the 2×3 plates reached their ultimate capacity at 0.137 in. In comparing the proposed capacity prediction model for both bolt patterns, as reported in Table 6, the group prediction results show similar strength ratios, ρ. As seen from Figures 8(a) and 8(c), at a weld/bolt strength ratio of 0.67, two specimens in Test 12 (2×2) showed similar capacities to those of Test 18 (2×3), even though the connection had two fewer bolts. This is due to the higher slip coefficient of the 2×2 Class A plates versus the 2×3 plates reported in Table 3. Overall, the test data shows that the bolt pattern has a negligible effect on the accuracy of the capacity prediction model for the tested configurations. For these combination connections, load sharing between the frictional resistance of the plates and the longitudinal weld elements was found to occur effectively given the strain compatibility (at low slip levels), between the load-deformation behavior of the frictional resistance and weld shear forces. For connections with different bolt patterns, if typical AISC *Specifications* detailing practices are followed with respect to maximum/minimum edge distances and bolt spacing, loading sharing between the frictional resistance and the weld shear forces is expected to occur. With reliable load sharing, the connection capacity can be computed using the proposed capacity prediction equation.

Bolt Size

Nearly all connections studied in the research program are constructed with ¾ in. bolts, except Test 15, which utilized 1 in. bolts. Figure 11 compares the load-deformation behaviors of Tests 13 and 15 at the same weld/bolt strength ratio. The load-deformation behavior of the 1 in. bolts follows other 2×2 Class A combination connections depicted in Figure 8(a). Accordingly, the Test R_n for the Test 15 specimens is reported at 0.02 in. of slip in Table 6. The test data shows that the AISC model overpredicts the capacity of connections utilizing 1 in. bolts by roughly 20% and the proposed model, as shown in Table 6, overpredicts the connection capacity by 17%. Both models show the strength ratio, ρ, that is significantly lower than the values obtained for connections made with ¾ in. bolts. This drop in capacity may be attributed to the loss of pretension force arising from the localized yielding that was observed around the bolt holes. Significant deformations around bolt holes were observed in the 1 in. tests, while no similar deformation occurred with the ¾ in. bolts. Figure 12 shows the deformations around the bolt holes in one of the splice plates of Test 15. Allan and Fisher (1968) reported a 15% drop in the pretension force for 1 in. bolts when oversized holes are used. A similar reduction in the slip capacity of bolted connections when larger bolts are used has been reported in Shoukry and Haisch (1970) and Heistermann et al. (2013). Accordingly, it is believed that the drop in capacity for these connections is primarily attributed to the lower contribution of the bolt slip load.

Bolt Grade

The connections in Tests 23 and 11 are utilized to study the influence that bolt grade has on the combination connection performance. These specimens utilize A490 bolts, while comparative tests—Tests 18 and 11—use A325 bolts. Test 23 is constructed with the same weld length of Test 18, and Test 11 is constructed with the same weld/bolt strength ratio as Test 8. The load-deformation behaviors shown in Figures 13(a) and 13(b) indicate that the bolt grade does not alter the slip behavior but leads to a change in the connection capacity that is comparable to the increase in the pretension force introduced by the higher grade.

Bolt Pretensioning Method

Two connection groups were added to the test program to investigate the effect of the bolt pretensioning method on the capacity: Tests 22 and 10. These connections are

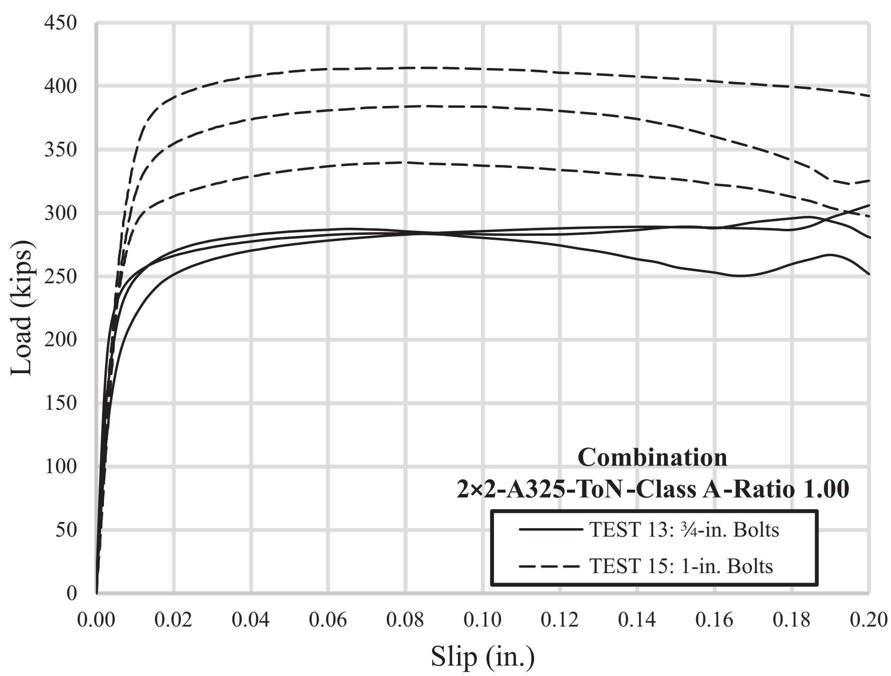


Fig. 11. Bolt size load-slip curve comparison: ¾ in. diameter vs. 1 in. diameter.

constructed with TC bolts while all other connections in the study are pretensioned with the turn-of-nut (ToN) method. The performance of these two connection groups can be analyzed against similar connections pretensioned with the ToN method—Tests 19 and 8, respectively. The load-deformation curves plotted in Figures 14(a) and 14(b) show that the pretensioning method does not alter the general connection behavior. The Class A connections in Test 19 (ToN) show an average slip load of 323 kips, while Test 22 (TC) slipped at an average load of 321 kips, as shown in Table 6. This comparison shows a less than 1% drop in the capacity of the Class A connections using TC bolts. In comparing Tests 8 and 10, a 5% drop in capacity is observed in the TC bolted connection made with Class B plates. This drop in capacity is expected based on the bolt pretension data reported in Table 2, where the ToN method achieved higher pretension than the TC bolts on average. However, it should be noted that the sample size for the TC bolts was significantly smaller than that of the ToN bolts. Furthermore, the TC bolts achieved, on average, a pretension level that is 37% higher than the minimum required pretension force for this bolt style.

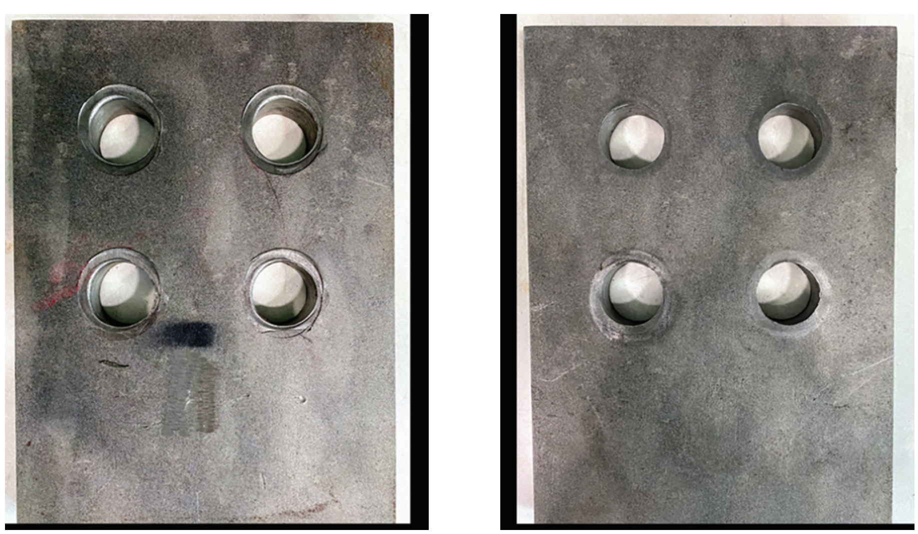
Faying Surface Class

The faying surface was found to have a significant influence on the load-deformation behavior of a combination connection. The load-deformation behavior highlighted in Figures 8(a) and 8(c) for Class A connections shows a stiffness in the elastic region that is comparable to Class B connections. However, as the displacement increases, the connection begins to soften, and a reduction in stiffness occurs. The slip gradually increases until the ultimate load is reached, and the welds start to show fractures at the ends of the weld lines. Further loading leads to additional crack propagation and a gradual drop in the capacity until welds completely fracture or bolt bearing is achieved. Overall, the connections with Class A faying surfaces displayed a highly ductile behavior and were able to sustain loads over large deformations as the bolts slipped into bearing.

The Class B load-deformation behavior depicted in Figure 8(b) is also very stiff in the elastic region of the connection. At approximately 0.017 in. of slip, the connection softens, and the load drops continuously as the deformation increases. These connections rely on the mechanical interlock established between the blast-cleaned surfaces to provide friction resistance. As the interlock between the steel surfaces is disturbed, the friction resistance is diminished. Overall, the addition of weld to a connection with a Class B faying surface significantly increased the ductility of the connection and improved its behavior. Instead of the sudden slip occurring with the Class B bolted-only connections as they reach their slip load, the combination connections were able to sustain loads over a longer slip distance.

Weld/Bolt Strength Ratio

In this test program, weld/bolt ratios ranging from 0.50 to 2.00 were studied across different connection groups. In all tested connections, an increase in the average slip



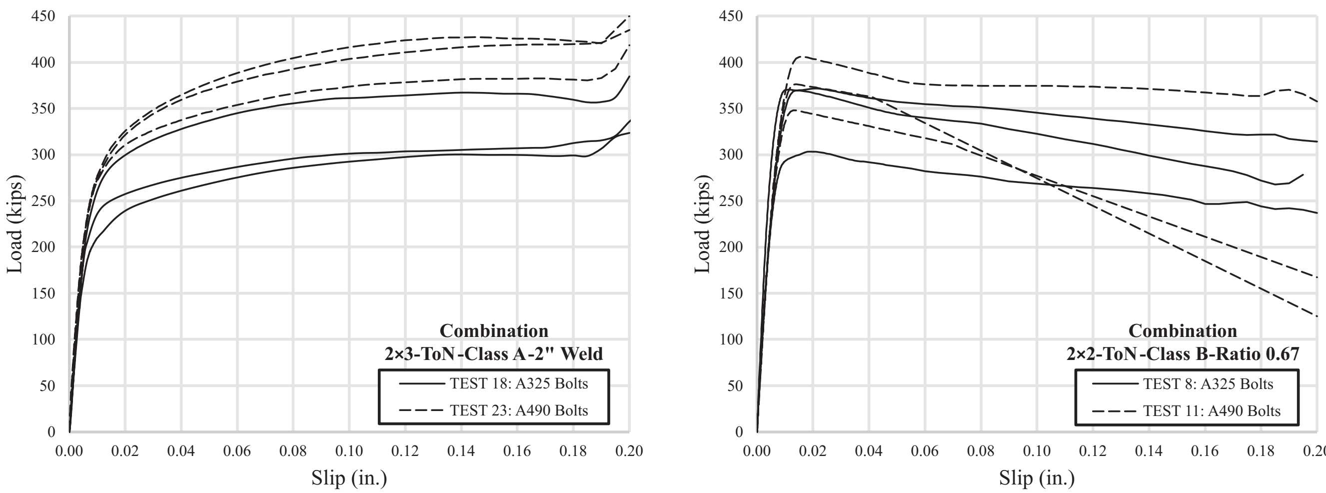
(a) Bolt head side *(b) nut side with washers*

Fig. 12. Plate dents from 1 in. bolt specimens.

load occurs as the weld/bolt ratio increases. This is to be expected given the additional weld length utilized with higher weld/bolt strength ratios. In examining the strength ratio, ρ, reported for each independent test series in a connection group, it is apparent that the weld/bolt ratio also has a negligible effect on the capacity prediction accuracy. As shown in Table 6, ρ does not display a consistent trend with respect to the weld/bolt ratio. Finally, Figure 8 shows that the weld/bolt ratio also does not influence the general load-deformation behavior the combination connection.

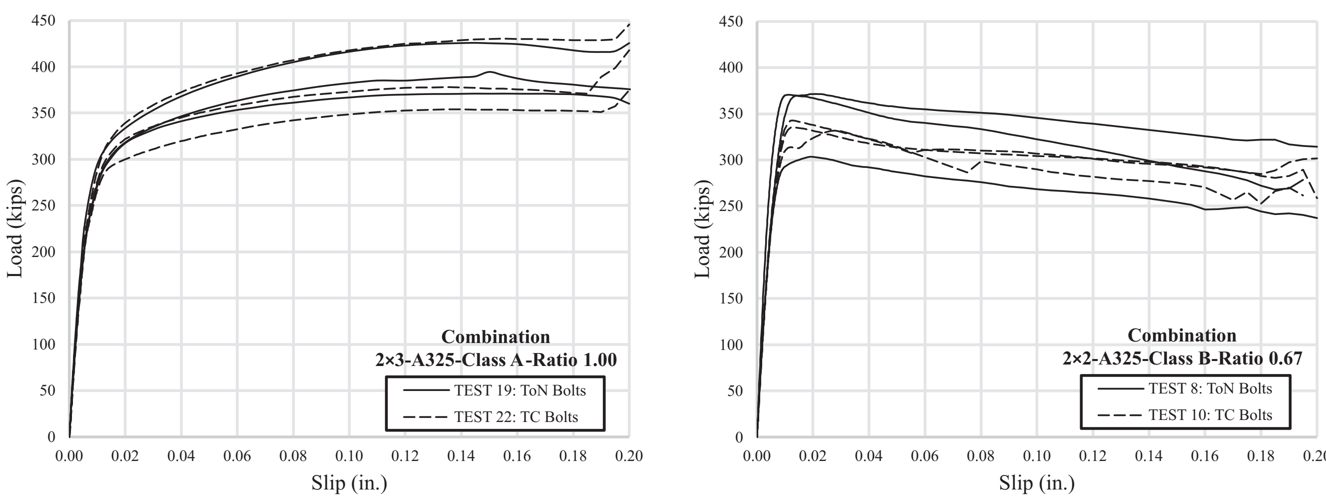
SUMMARY AND CONCLUSIONS

This paper presented the results of an experimental investigation into the load-deformation behavior of double-shear tension splice connections made with pretensioned high-strength bolts and longitudinal fillet welds in combination. The tested connections varied the bolt pattern, bolt size, bolt grade, bolt pretensioning method, faying surface class, and weld/bolt strength ratio. An assessment of the AISC prediction model for a combination connection was made, and a prediction model that maintains strain compatibility



(a) Class A *(b) Class B*

Fig. 13. Bolt grade load-slip curve comparison: A325 vs. A490.



(a) Class A *(b) Class B*

Fig. 14. Bolt tensioning method load-slip curve comparison: ToN vs. TC.

was presented. Based on the observations made during the research program, the following conclusions and recommendations can be made:

1. The addition of longitudinal fillet welds to concentrically loaded slip-critical bolted connections leads to an increase of the connection capacity as well as an improvement in the stiffness of the connection. Connections made with Class B faying surfaces also exhibit an improvement in the ductility when welds are used.
2. Bolted-only connections with Class A (clean mill scale) faying surfaces are ductile and display a hardening behavior. For the tested configurations, combination connections made with these surfaces reach their ultimate capacity at a slip displacement ranging from 0.055 in. to 0.165 in. However, to limit slip of the connection, it is recommended that the connection capacity be limited to the load sustained at 0.02 in. of slip. This load level accounts for 76% to 96% of the capacity.
3. Bolted-only connections with Class B surfaces (SSPC-SP6 commercial blast-cleaning) may slip suddenly before 0.02 in. of deformation. Combination connections with these surfaces reach their ultimate capacity, on average, at approximately 0.017 in. of slip but are more ductile than their bolted-only counterparts. It is recommended that the connection capacity be limited to the maximum sustained load before 0.02 in. of slip.
4. The proposed model, given by Equation 5, which considers strain compatibility of the weld at 0.02 in. of slip, can predict the combination connection capacity with strength ratio value, ρ, of 1.04. This model evaluates the welded component contribution to the combination connection capacity based on the fillet weld load-deformation prediction model provided in AISC *Manual* Part 8. It is recommended that this equation be adopted when determining the ultimate strength of connections utilizing pretensioned high-strength bolts and longitudinal welds in combination.
5. Other connection variables such as the bolt pattern, bolt grade, bolt pretensioning method, and weld/bolt strength ratio show a negligible effect on the general load-deformation behavior of the tested combination connection or accuracy of the prediction model.
6. Future experimental investigations of connections utilizing larger bolts—for example, 1 in. diameter and greater—are recommended to quantify how localized yielding at the bolt holes may affect the bolt pretension, frictional resistance, and ultimate capacity of the connection. These studies may also include connections using ASTM F3125 Grade A490, F1852, and F2280 bolts as well as bolt assemblies using ASTM F959 DTI washers to understand the sensitivity between the pretensioning method and the localized yielding at the bolt hole for large-bolt diameters.

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REFERENCES

AISC (2016), *Specification for Structural Steel Buildings*, ANSI/AISC 360-16, American Institute of Steel Construction, Chicago, Ill.

AISC (2017), *Steel Construction Manual,* 15th Ed., American Institute of Steel Construction, Chicago, Ill.

Allan, R.N. and Fisher, J.W. (1968), “Bolted Joints with Oversize or Slotted Holes,” *Journal of the Structural Division*, ASCE, Vol. 94, No. 9, pp. 2061–2080.

ASTM (2021a), *Standard Specification for High Strength Low-Alloy Columbium-Vanadium Structural Steel*, ASTM A572/A572M, ASTM International, West Conshocken, Pa.

ASTM (2021b), *Standard Specification for High Strength Structural Bolts and Assemblies, Steel and Alloy Steel, Heat Treated, In. Dimensions 120 ksi and 150 ksi Minimum Tensile Strength, and Metric Dimensions 830 MPa and 1040 MPa Minimum Tensile Strength*, ASTM F3125, ASTM International, West Conshocken, Pa.

AWS (2016), *Standard Method for Mechanical Testing of Welds*, B4.0:2016, American Welding Society, Miami, Fla.

Borello, D.J., Denavit, M.D., and Hajjar, J.F. (2009), *Behavior of Bolted Steel Slip-Critical Connections with Fillers*, Newmark Structural Engineering Laboratory, University of Illinois at Urbana-Champaign, Champaign, Ill.

Deng, K., Grondin, G.Y., and Driver, R.G. (2003), *Effect of Loading Angle on the Behavior of Fillet Welds,* Structural Engineering Report 251, Department of Civil and Environmental Engineering, University of Alberta, Edmonton, Alta, Canada.

Grondin, G., Jin, M., and Josi, G. (2007), *Slip Critical Bolted Connections—A Reliability Analysis for Design at the Ultimate Limit State*, Structural Engineering Report No. 274, University of Alberta, Edmonton, Alberta, Canada.

Heistermann, C., Veljkovic, M., Simões, R., Rebelo, C., and Da Silva, L.S. (2013), "Design of Slip Resistant Lap Joints with Long Open Slotted Holes," *Journal of Constructional Steel Research*, Vol. 82, pp. 223–233.

Holtz, N.M. and Kulak, G.L. (1970), *High-Strength Bolts and Welds in Load-Sharing Systems,* Structural Engineering Report No. 8, Department of Civil Engineering, Nova Scotia Technical College, Halifax, Nova Scotia.

Jarosch, K.H. and Bowman, M.D. (1986), "Tension Butt Joints with Bolts and Welds," *Engineering Journal*, AISC, Vol. 23, No. 1, pp. 25–34.

Khandel, O., Tamimi, M.F., Soliman, M., Russell B.W., and Waite, C.D. (2022), "Reliability Assessment of Connections with Slip-Critical Bolts and Fillet Welds in Combination," *Journal of Constructional Steel Research*, Vol. 188, 107036.

Kim, D.K. and Lee, C.H. (2020), "Experimental and Analytical Study of Combined Bolted-Welded Lap Joints Including High-Strength Steel," *Journal of Constructional Steel Research*, Vol. 168, 105995.

Kulak, G.L., Fisher, J.W., and Struik, J.H.A. (2001), *Guide to Design Criteria for Bolted and Riveted Joints*, 2nd Ed., American Institute of Steel Construction, Chicago, Ill.

Kulak, G.L. and Grondin, G.Y. (2003), "Strength of Joints that Combine Bolts and Welds," *Engineering Journal*, Vol. 40, pp. 89–98.

Lesik, D.F. and Kennedy, D.L. (1990), "Ultimate Strength of Fillet Welded Connections Loaded in Plane," *Canadian Journal of Civil Engineering,* Vol. 17, pp. 55–67.

Manuel, T.J. (1996), *Strength of Joints That Combine Bolts and Welds*, M.Sc. Thesis, Department of Civil Engineering, University of Alberta, Edmonton, Alberta, Canada.

Manuel, T.J. and Kulak, G.L. (2000), "Strength of Joints That Combine Bolts and Welds," *Journal of Structural Engineering*, Vol. 126, pp. 279–287.

Miller, D.K. (2001), "Mixing Welds and Bolts, Part 1," *Welding Innovation*, Vol. 18, No. 2.

Miller, D.K. (2002), "Mixing Welds and Bolts, Part 2," *Welding Innovation*, Vol. 19, No. 2.

NI (2018), *LabVIEW NXG 3.0 User Manual*, National Instruments, Austin, Tex., https://www.ni.com/documentation/en/labview/3.0/manual/manual-overview/, accessed December 10, 2018.

RCSC (2020), *Specification for Structural Joints Using High-Strength Bolts*, Research Council on Structural Connections, AISC, Chicago, Ill.

Salmon, C.G., Johnson, J.E., and Malhas, F.A. (2009), *Steel Structures: Design and Behavior: Emphasizing Load and Resistance Factor Design*, Pearson Education, Upper Saddle River, N.J.

Sato, T.L. (2000), *Strength of Joints That Combine High Strength Bolts and Longitudinal Welds*, M.Eng. Report, Department of Civil and Environmental Engineering, University of Alberta, Edmonton, Alberta, Canada.

Shi, Y., Wang, L., Wang, Y., Ma, J., and Bai, R. (2011a), "Finite Element Analysis of the Combined Connection with Bolts and Welds," *Applied Mechanics and Materials*, Vol. 94–96, pp. 316–321.

Shi, Y., Wang, L., Wang, Y., Ma, J., and Bai, R. (2011b), "Proposed Design Method of Combined Connections with Bolts and Longitudinal Welds," *Applied Mechanics and Materials*, Vol. 94–96, pp. 923–928.

Shoukry, Z. and Haisch, W.T. (1970), "Bolted Connections with Varied Hole Diameters," *Journal of the Structural Division*, Vol. 96, No. 6, pp. 1,105–1,118.

Steinhardt, O., Möhler, K., and Valtinat, G. (1969), *Experiments on the Use of Pretressed Screws in Steel Construction, Part IV*, Reports of the German Committee for Steel Construction, 25.