The Development of a New Design Procedure for Conventional Single-Plate Shear Connections

LARRY S. MUIR and WILLIAM A. THORNTON

ABSTRACT

Conventional single-plate shear connections are common and economical connections. The design procedure outlined in the 13th edition AISC *Steel Construction Manual,* relies on the bolt shear values given in the 2005 AISC *Specification for Structural Steel Buildings*. The nominal bolt shear values listed in *Specification* Table J3.2 have historically been 20% lower than the theoretical bolt values. This reduction was provided to account for uneven force distribution among the bolts in end-loaded connections, such as bolted lap splices. The reduction served the secondary function of providing an additional factor of safety for all bolted connections designed in accordance with the *Specification*. The design procedure for conventional single-plate shear connections contained in the 13th edition *Manual* relied on this reduction to justify the practice of neglecting eccentricity in the bolt group for most configurations. The 2010 AISC *Specification* increases the nominal bolt shear values, necessitating a revised design procedure for single-plate shear connections in the 14th edition AISC *Manual*. This paper outlines the revised procedure.

Keywords: single-plate shear connections.

Single-plate shear connections consist of a single plate welded to the supporting beam or column and field bolted to the supported beam. Two different configurations of single-plate shear connections will be recognized in the 14th edition of the AISC *Steel Construction Manual*: the conventional configuration and the extended configuration. The extended configuration is a more general configuration in that it allows greater variation in the distance between the weld and the bolts, the number of bolts, and the plate thicknesses used. The conventional configuration limits the distance between the weld and the bolts to a maximum of 3 in., allows between 2 and 12 bolts in a single vertical line, and limits the ratio of the plate thickness to the bolt diameter.

Conventional single-plate shear connections (Figure 1) are common and economical connections. They provide simple and economical fabrication and erection, and because bolted connections are only used in the connection to the supported member, there is no safety concern over the use of shared bolts through the web of the support during erection.

The design procedure contained in the 14th edition of the *Manual* will be similar to that contained in the 13th edition (AISC, 2005a), but with a few key differences, including revised design eccentricities and further limitation on plate thickness for deeper connections using standard holes.

Larry S. Muir, Structural Steel Consultant, Atlanta, GA (corresponding). E-mail: larrymuir@larrymuir.com

William A. Thornton, Consultant, Cives Steel Company, Roswell, GA. E-mail: bthornton@cives.com

NEED FOR REVISED DESIGN PROCEDURE

The need to reevaluate and revise the design procedure contained in the 13th edition *Manual* arose from an increase in the nominal bolt shear values provided in AISC's 2010 *Specification for Structural Steel Buildings.* The nominal bolt shear values listed in *Specification* Table J3.2 have historically been 20% lower than the theoretical bolt values. This reduction was provided to account for uneven force distribution among the bolts in end-loaded connections, such as bolted lap splices. The reduction served the secondary

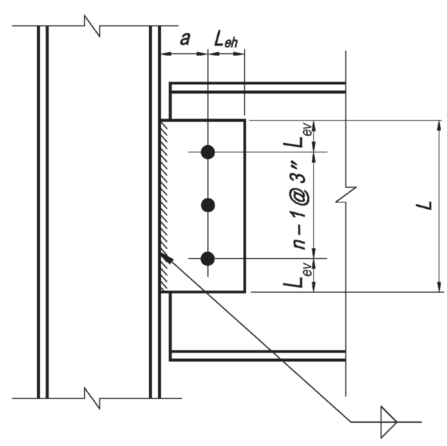

Fig. 1. Single-plate connection.
(Fig. 10-11 in the AISC Manual, 13th ed.)

function of providing an additional factor of safety for all bolted connections designed in accordance with the *Specification*.

The design procedure for conventional single-plate shear connections contained in the 13th edition *Manual* relied on this reduction to justify the practice of neglecting eccentricity in the bolt group for most configurations. Reanalysis has shown that neglecting the eccentricity is no longer appropriate, considering the increased 2010 *Specification* bolt strengths.

14TH EDITION *MANUAL* DESIGN PROCEDURE

Because what follows is largely a discussion of the rationale underlying the design procedure for single-plate shear connections contained in the 14th edition *Manual*, it is appropriate to present the procedure and then present the individual considerations is greater detail.

The procedure presented in this paper has been adopted into the 14th edition *Manual* as a method of designing conventional single-plate shear connections that is applicable over the entire range of support rigidities. This procedure can be used to determine the strength of single-plate shear connections, which meet the dimensional limitations set forth in the procedure.

Shared Provisions

The conventional configuration and the extended configuration of the single-plate shear connection share some attributes and requirements. Although this paper specifically addresses the design of the conventional configuration, the requirements that apply to both configurations are presented together to better reflect the organization as contained in the *Manual*. These shared requirements include:

1. The use of either group A (ASTM A325 or F1852) or group B (ASTM A490 or F2280) bolts is acceptable.
2. The use of snug-tightened, pretensioned, or slip-critical bolts is acceptable.
3. The use of material with either $F_y = 36$ ksi or $F_y = 50$ ksi is acceptable.
4. The weld size shall be $\frac{5}{8}t_p$.

Dimensional Limitations

1. Only a single vertical row of bolts is permitted. The number of bolts in the connection, n, is limited to 2 to 12.
2. The distance from the bolt line to the weld line, a, must be equal to or less than 3½ in.
3. Standard or short-slotted holes are permitted to be used as noted in Table 1.
4. The horizontal distance L_{eh} must be equal to or greater than $2d_b$ for both the plate and the beam web. Note that L_{eh} is measured to the center of the hole or slot.
5. Either the plate or the beam web must satisfy the maximum thickness requirement given in Table 1.

Table 1. Recommended Design Parameters for Conventional Single-Plate Shear Connections

N	Hole Type	e_b	Maximum t_w or t_p
2–5	SSL	$a/2$	None
	STD	$a/2$	$d_b/2 + \frac{1}{16}$ in.
6–12	SSL	$a/2$	$d_b/2 + \frac{1}{16}$ in.
	STD	a	$d_b/2 - \frac{1}{16}$ in.

Design Checks

The bolts and plate must be checked for the required shear with an eccentricity equal to e_b, as given in Table 1.

Plate buckling will not govern for the conventional configuration.

PREVIOUS RESEARCH

A number of research programs have investigated the behavior of single-plate shear connections. A brief summary includes:

- White (1965) conducted tests involving single-plate shear connections to HSS columns.
- Lipson (1968) conducted tests of both single angle and single-plate connections.
- Richard et al. (1980) conducted tests on single-plate shear connections using stub beams.
- Hormby et al. (1984) conducted tests on single-plate connections with Grade 50 steel and composite construction.
- Astaneh et al. (1988) conducted tests on single-plate shear connections with standard holes to rigid supports.
- Astaneh and Porter (1990) conducted tests on single-plate shear connections with short-slotted holes to rigid supports.
- Astaneh and Shaw (1992) conducted tests on single-plate shear connections to support girder webs.
- Sarkar and Wallace (1992) conducted tests on single-plate shear connections to rigid supports.

- Creech (2005) conducted tests on single-plate shear connections to rigid and flexible supports.
- Baldwin Metzger (2006) conducted tests on conventional and extended single-plate shear connections to rigid supports.

The analysis reflected in this paper centered on the tests conducted by Astaneh and colleagues, Sarkar and Wallace, Creech, and Baldwin Metzger. Richard et al. did not test their connections to failure. Therefore, while their tests aid in the understanding of the behavior of single-plate shear connections, they do not provide much information regarding the ultimate strength of these connections. The purpose of White's tests was to determine the effect of connections on hollow structural section (HSS) columns, so though his findings were considered they were not integral to the analysis. The Hormby et al. tests were not included because the design procedure was intended to be used for both composite and noncomposite construction.

ESTABLISHING A DESIGN MODEL

The inherent rigidity of single-plate connections has been a concern for designers worried about considerable, unanticipated moments that could be developed in the connections, which could precipitate a sudden rupture of either the weld or the bolts. Further, Section B3.6a of the AISC *Specification,* requires that simple shear connections have sufficient rotational capacity to accommodate the required beam end rotation. The potential of developing moments in the connection beyond those resulting from the eccentricity between the support and the bolts, the need to accommodate simple beam end rotations, and ductility concerns become primary considerations in the development of a design model for the single plate shear connection. Much of the research and resulting design procedures developed over the years has concentrated on predicting and/or controlling this behavior.

The design model used in the 14th edition *Manual* builds on previous work. First, ductility requirements relating the strength of the plate (or beam web) to the bolt and weld strengths are set. These requirements are based on a combination of theoretically derived ratios and empirical results. The goal of the ductility requirements is to accommodate a target end rotation of 0.03 radian without rupture of any of the elements. The mechanisms used to achieve this ductility are discussed in greater detail later.

Having ensured sufficient ductility, the bolts are then designed to resist the required beam end reaction at an empirically derived effective eccentricity. Finally the plate is assumed to be subjected to loads consistent with the other elements, the beam end reaction applied at the effective eccentricity.

Rotational Demand

Before procedures for ensuring ductility can be derived, a target end rotation to be accommodated must first be defined. Among researchers and AISC committees, 0.03 radian has become a de facto standard; 0.03 radian is roughly equal to the end rotation of a beam whose span is 24 times its depth, is loaded with the maximum uniform design load and is commonly accepted to be a reasonable upper bound for beam end rotation. This level of end rotation is unlikely to occur in many instances, but in order to provide a design procedure that can be applied to a wide range of practical conditions, AISC has adopted 0.03 radian as the target rotation.

Beyond establishing a target magnitude for the rotation, a center of rotation must also be established. If 0.03 radian of rotation occurs at the top bolt of the connection, the movement of the bottom bolt relative to the plate that must be accommodated is about twice that required if the rotation occurs about the center of the bolt group. Research has shown that the center of rotation can vary considerably throughout the loading of the connection. However, when the connection is made to a rigid support, the center of rotation will coincide roughly with the center of the bolt group as the load approaches ultimate. It was observed that "during the test, neutral axis remained very close to the mid-height of the single plate." (Astaneh et al., 1988) Even when the mid-depth of the connection and beam are not coincident, the center of rotation remains near the mid-depth of the connection. (Hormby et al., 1984). No comments are provided in the research regarding the center of rotation of single-plate connections supporting composite beams; however, Hormby et al. (1984) noted, with regard to the use of single-plate connections used with composite beams, that "since full-scale beam tests of off-axis bolt groups resulted in essentially the same moment-rotation and center of rotation response as symmetrical connections, it is concluded that the behavior of the single plate is not affected by its location relative to beam's neutral axis."

The preceding findings relative to the location of the center of rotation are based on the observed behavior of single-plate connections to rigid supports. There is a lack of similar data pertaining to flexible supports; however, because the simple beam end rotation can be accommodated through movement of the flexible support, instead of plowing of the bolts, the location of the center of rotation is of less importance for these conditions.

Theoretically, the single-plate shear connection with standard holes can accommodate the requisite beam end rotations through a combination of plate flexural yielding; bolt deformation; bolt plowing; and, in the case of a connection to a flexible support, through support rotation. Bolt plowing is the local yielding of the plate or beam web, which occurs at the bolt holes causing elongation of the holes. The relatively short distance between the bolts and the welds, which is an integral feature of the conventional single-plate shear connections, allows only a small area over which yielding can occur, so plate yielding is usually discounted as a means of accommodating the simple beam end rotation. Support rotation is not relied on to accommodate simple beam end rotations, because this mechanism cannot be applied to rigid supports and because other effects associated with support rotation can lead to serviceability problems.

Both bolt deformation and bolt plowing are dependent on determining not so much the end rotation, but rather the movement of the bolts relative to the plate that occurs as a result of the end rotation. For a given center of rotation, there is a direct relationship between the beam end rotation and the amount of movement the bolt group must accommodate either through deformation in the bolt itself and/or plowing through the joined materials. Assuming a center of rotation at the center of the bolt group, the relative horizontal movement of the bolts and the connected materials can be approximated as:

$$\delta = (0.03\,\text{radian})\frac{(n-1)b}{2} \tag{1}$$

This results in a maximum relative horizontal movement of 0.495 in. for a 12-row connection. Deformation of this magnitude would essentially exhaust the capacity of the bolt—as can be shown using the load-deformation relationship given on page 7-6 of the AISC 13th edition *Manual* (which can be used to calculate the force on a bolt given a deformation, in this instance a deformation of about 0.5 in.):

$$\begin{aligned} R &= R_{ult}\left(1-e^{-10\Delta}\right)^{0.55} = R_{ult}\left(1-e^{-10(0.5)}\right)^{0.55} \\ &= 0.996R_{ult} \end{aligned} \tag{2}$$

To accommodate even the modest deformational demands of a two-bolt connection, nearly 60% of the bolt capacity would be exhausted. Therefore, deformation in the bolt alone cannot be counted on to accommodate the simple beam end rotation.

Eliminating plate flexural yielding and support rotation as a possible means of accommodating the beam end rotations leaves only the combined effects of bolt deformation and bolt plowing. In order for the bolts to plow, however, there must be an upper limit placed on the stiffness and strength of the plate and/or the beam web relative to that of the bolts. Prior to the 13th edition *Manual*, the plate thickness was limited to one-half the diameter of the bolt plus 1⁄16 in. In the 13th edition *Manual,* the possibility that deformations could occur in either the plate or the beam web was formally recognized, and the requirement was changed such that the thickness of *either* the plate or the beam web was limited to one-half the diameter of the bolt plus 1⁄16 in.

Table 2. Results of Bolt Plowing Tests

Bolt	t_p (in.)	Δ (in.)	Plate F_y (ksi)	Plate F_u (ksi)
¾-in. A325-N	¼	0.65	47.5	65.9
¾-in. A325-N	5⁄16	0.6	47.3	65.5
¾-in. A325-N	⅜	0.3	47.6	67.1
¾-in. A490-N	5⁄16	0.7	47.3	65.5
¾-in. A490-N	⅜	0.4	47.6	67.1

To evaluate this requirement, it is instructive to look at the results of tests in which a single bolt was essentially plowed through various thicknesses of plates. Sarkar and Wallace (1992) ran a series of such tests. The results are provided in Table 2.

These results indicate that for a plate thickness equal to one-half the bolt diameter, approximately 0.3 in. of bolt plowing, Δ, can occur. Limiting the plate thickness to approximately half the bolt diameter can, therefore, be expected to accommodate end rotations for up to a seven-row connection. Further limiting the plate thickness to half the bolt diameter minus 1⁄16 in. can accommodate the larger deformations required for deeper connections. Because the tests were run with ¾-in. bolts, it is believed that the results can be safely extrapolated to larger-diameter and -strength bolts. Bolts less than ¾-in. diameter are rarely used in structural connections.

Providing short-slotted holes in the plate can also help to accommodate the simple beam end rotation. The short slots will provide between ¼ in. and 5⁄16 in. of horizontal movement in typical connections, before any deformation must occur in the bolts, the plate or the beam web. This ¼ in. alone is enough to provide the 0.03 radian of rotation for a five-row connection. In other words, when short slots are provided in a single-plate shear connection of five rows or less, the ratio between plate thickness and bolt diameter is immaterial.

As stated previously, 0.03 radian is considered a conservative upper bound for the end rotation. It might be reasonable to relax the plate thickness to bolt diameter requirements when the end rotation is known to be less than 0.03 radian, such as when the beam span is short relative to the beam depth or when the beam is sized based on serviceability rather than strength criteria. Moving from ASTM A325 bolts to A490 bolts might also offer some relief from this requirement. Because all of the tests directly related to bolt plowing utilized A325 bolts, the plowing behavior of A490 bolts has not been established, but it seems reasonable to believe that due to its greater strength, an A490 bolt would be capable of plowing through a greater length or thickness of plate before rupturing.

Edge Distance Requirements

The AISC *Manual* design procedures for single-plate connections had not included a provision requiring that the horizontal edge distance be twice the bolt diameter prior to the 13th edition. This requirement comes from the original Richard et al. (1980) research. It was included in the AISC book *Engineering for Steel Construction* (1984). It was not included in the Astaneh et al (1988, 1989) procedure, which was the basis of the procedure in the 9th edition (ASD), 2nd edition (LRFD), and 3rd edition (LRFD) *Manuals*. Rather than requiring a horizontal edge distance twice the bolt diameter, Astaneh et al.'s procedure recommended a horizontal edge distance 1.5 times the bolt diameter.

The intent of the twice the bolt diameter requirement seems to be to ensure that the bolts will bear without tearing through the edge of the material. However, bolt tear-out never occurred in any of the testing nor was any tearing between the edge of the hole and the edge of the plate observed that might indicate bolt tear-out was imminent. The maximum relative horizontal movement required to develop the simple beam end rotation of 0.03 radian is 0.495 in., as previously discussed. Based on this fact, and considering the fact that it was not required by the Astaneh et al. work (which was the basis for single-plate shear connection design in the United States for 20 years), the edge distance requirement of twice the bolt diameter would seem to be overly conservative and unnecessary.

DESIGN OF THE BOLT GROUP

It is intuitive to assume that the bolt group in a single-plate shear connection, being offset from the face of the support, will experience some eccentricity. The effective, or design, eccentricity, however, is not necessarily equal to the distance from the weld group to the bolt group, as might be assumed. A significant end moment might develop when a stiff plate connection is attached to a rigid support. In such cases, the inflection point of the beam might be moved considerably into the span, resulting in an effective eccentricity higher than the distance from the weld group to the bolt group. Conversely, the presence of short slots or bolt plowing might reduce the effective eccentricity on the bolt group. Both of these possibilities were reflected in the design procedures used prior to the 13th edition *Manual*. The LRFD 3rd edition *Manual* (AISC, 2001), for instance, calculated the effective eccentricity on the bolt group as:

$$e_b = \left|(n-1) - a\right| \text{ for connections using standard holes} \quad (3)$$

or

$$e_b = \left|\frac{2n}{3} - a\right| \text{ for connections using short-slotted holes} \quad (4)$$

When attached to a flexible support, the effective eccentricity, e_b, could not be less than the distance from the weld group to the bolt group. Assuming a practical range of 2½ in. to 3½ in. for a, the 3rd edition LRFD *Manual* equations would predict an effective eccentricity on the bolt group of between about 5 to 267% of the distance from the weld group to the bolt group. Where the predicted effective eccentricity exceeded the a dimension, this was presumably done to account for potentially large moments occurring at the support, which could result in a larger moment at the bolt group. Though this large effective eccentricity will occur early in the loading history, the tests indicate that the reduction in stiffness due to bolt plowing reduced the eccentricity at ultimate loads as was intended.

Reanalysis of existing data and further testing (Creech, 2005; Baldwin Metzger, 2006) led to a less conservative requirement for the 13th edition *Manual* in which, for most cases, eccentricity was neglected. Though the tests did not indicate that there was no eccentricity on the bolt group, the 20% reduction in the bolt strength inherent in the 2005 *Specification* allowed the conclusion that the eccentricity could safely be neglected.

Because many of the tests were configured such that the bolts governed the capacity of the connection, there is a relative wealth of data on which to base the design procedure for the bolts. Of 31 tests considered here, the bolts governed the strength of 20 of the connections. These 20 connections also contained a good mix of connection depths, hole types and support rigidities.

The approach taken in developing a design methodology for the bolt group followed the historical precedent of determining the effective eccentricity to which the bolt group was subjected. Only the effective eccentricity at ultimate load was considered in developing the design procedure, although effective eccentricities were often reported throughout the loading. There is no evidence in the testing that these larger effective eccentricities applied in conjunction with lesser loads can govern the strength of a single-plate shear connection meeting the dimensional requirements laid out in the procedure. A summary of the test data and the analysis is provided in Table 3.

In Table 3, the predicted bolt group strength without eccentricity values (column 11) were calculated by multiplying the number of bolts in the connection (column 2) by the bolt shear strength (column 4). Where the bolt shear strengths were measured and reported in the available reports, these values were used. Where measured bolt strengths were not reported, the bolt strength was assumed to be 26.5 kips for ASTM A325-N bolts and 33.2 kips for ASTM A325-X and

Table 3. Summary of Test Data

	(1) Test ID	(2) Bolts (All bolts are ¾-in. dia.)	(3) Hole Type	(4) Bolt Strength (kips)	(5) a (in.)	(6) Weld Size (in.)	(7) Plate F_y[b] (ksi)	(8) Plate F_u[b] (ksi)	(9) Tested Strength (kips)	(10) θ (rad.)	(11) Predicted Bolt Group Strength w/o Ecc. (kips)	(12) Test/ Predict	(13) Eff. e = %a	(14) Failure Mode
Astaneh and Colleagues														
1	1	(7)-A325-N	STD	26.5[a]	3	¼	35.3	61	160	0.026	186	0.860	1.02	Bolt
2	2	(5)-A325-N	STD	26.5[a]	3	¼	35.3	61	137	0.054	133	1.03	0	Bolt
3	3	(3)-A325-N	STD	26.5[a]	3	¼	35.3	61	94	0.056	79.5	1.18	0	Bolt
4	4	(5)-A490-N	STD	33.2[a]	2.75	7⁄32	35.3	61	130	0.053	166	0.783	1.07	Bolt
5	5	(3)-A490-N	STD	33.2[a]	2.75	7⁄32	35.3	61	79	0.061	99.6	0.994	0.585	Weld and bolt
Baldwin Metzger														
6	31C-3⅜	(3)-A325-N	STD	27.0	3	3⁄16	68.1	97.5	81	0.032	81	—	—	Beam
7	4B1C-3⅜	(4)-A325-N	STD	27.0	3	3⁄16	68.1	97.5	110	0.027	108	—	—	Beam
8	51C-3⅜	(5)-A325-N	STD	31.0	3	3⁄16	68.1	97.5	146	0.030	155	0.942	0.407	Bolt
9	71C-3⅜	(7)-A325-N	STD	27.0	3	3⁄16	68.1	97.5	173	0.018	189	0.915	0.740	Bolt
Creech														
10	S1-RSS-3-A325-N	(3)-A325-N	SSL	30.3	3	—	39.6	62.1	78.8	0.036	90.9	—	—	Beam
11	S2-RST-3-A325-N	(3)-A325-N	STD	30.3	3	—	39.6	62.1	90.7	0.027	90.9	0.998	0	Bolt
12	S3-FSS-3-A325-N	(3)-A325-N	SSL	30.3	3	—	39.6	62.1	71.8	0.039	90.9	0.790	0.55	Bolt
13	S4-FST-3-A325-N	(3)-A325-N	STD	30.3	3	—	39.6	62.1	61.4	0.023	90.9	0.675	0.82	Bolt
14	S5-FSS-3-A325-N-SR	(3)-A325-N	SSL	30.3	3	—	39.6	62.1	75.6	0.031	90.9	0.832	0.43	Bolt
15	S6-FST-2-A325-N	(2)-A325-N	STD	30.3	3	—	39.6	62.1	44.2	0.012	60.6	0.729	0.44	Bolt
16	S7-FSS-2-A325-N	(2)-A325-N	SSL	30.3	3	—	39.6	62.1	45.5	0.011	60.6	0.751	0.40	Bolt
17	S8-FSS-2-A325-N-SR	(2)-A325-N	SSL	30.3	3	—	39.6	62.1	47.9	0.013	60.6	0.790	0.34	Bolt
18	S9-RSS-7-A325-N	(7)-A325-N	SSL	30.3	3	—	44.4	66.3	166.5	0.028	212	—	—	Beam
19	S10-FSS-7-A325-N-SR	(7)-A325-N	SSL	30.3	3	—	44.4	66.3	202.5	0.027	212	0.955	0.40	Bolt
Sarkar and Wallace														
20	1a-North End	(2)-A325-X	STD	33.2[a]	3.5	—	47.4	—	41.7	0.033	66.6	—	—	Pl yielding
21	1a-South End	(2)-A325-X	STD	33.2[a]	3.5	—	47.4	—	64.3	0.025 (0.043)	66.6	—	—	Pl distortion
22	1b-North End	(2)-A325-N	STD	26.5[a]	3.5	—	47.4	—	60.8	0.028	53	1.15	0	Weld
23	1b-South End	(2)-A325-N	STD	26.5[a]	3.5	—	47.4	—	51.8	0.033	53	0.977	0.032	Weld
24	2a-North End	(4)-A325-N	STD	26.5[a]	3.5	—	47.4	—	93 (66.4)	0.028 (0.32)	106	0.877	0.41	No failure (bolt)
25	2a-South End	(4)-A325-N	STD	26.5[a]	3.5	—	47.4	—	93 (81.6)	0.32 (0.38)	106	0.877	0.41	No failure (bolt)
26	2b-North End	(4)-A325-N	SSL	26.5[a]	3.5	—	47.4	—	129	0.042	106	1.22	0	Bolt
27	2b-South End	(4)-A325-N	SSL	26.5[a]	3.5	—	47.4	—	129	0.042	106	1.22	0	Bolt
28	3a-North End	(6)-A325-N	STD	26.5[a]	3.5	—	47.4	—	119 (102)	0.027 (0.014)	159	0.748	1.14	Bolt
29	3a-South End	(6)-A325-N	STD	26.5[a]	3.5	—	47.4	—	119 (109)	0.027 (0.019)	159	0.748	1.14	Bolt
30	3b-North End	(6)-A325-N	SSL	26.5[a]	3.5	—	47.4	—	168	0.030	159	1.06	0	Bolt
31	3b-South End	(6)-A325-N	SSL	26.5[a]	3.5	—	47.4	—	194	–(0.030)	159	—	—	No failure

Notes:
a. Theoretical bolts strength with no end load reduction, based on nominal strength from the appropriate ASTM standard
b. All plates ⅜ in. thick measured yield and ultimate stresses as shown.
c. Indicates that the data are not available

A490-N bolts. Inherent in these values are the same assumptions made in the 2010 AISC *Specification* that the bolt strengths equal the minimum specified tensile strength given in the ASTM standards, the ratio of bolt shear strength to bolt tensile strength is 0.62 and ratio of effective thread root area to shank area is 0.80.

Once a predicted strength neglecting eccentricity was established, the effect of the eccentricity could be determined. Calculating the ratio of the tested strength (column 9) to the predicted strength (column 11) provided the efficiency of the bolt group in resisting the applied shear (column 12), which is essentially the C-value from the eccentrically loaded bolt group tables in Part 7 of the *Manual*. Using the instantaneous center of rotation method described in Part 7, an effective eccentricity corresponding to the bolt group efficiency could be determined and expressed as a percentage of the *a* dimension (column 13).

It can be seen that seven of the tests (tests 2, 3, 11, 22, 26, 27 and 30) indicate that the strength of the bolt group is best predicted by neglecting the eccentricity. The number of bolts for these tests ranged from two to six installed in both standard and short-slotted holes. There are eight tests (tests 8, 14, 15, 16, 17, 19, 24 and 25) that indicate that the strength of the bolt group is best predicted by assuming an eccentricity equal to 0 to 50% of the *a* distance. The number of bolts for these tests ranged from two to seven installed in both standard and short-slotted holes. Six of the tests (tests 1, 4, 9, 13, 28 and 29) indicate that the strength of the bolt group is best predicted by assuming an eccentricity equal to 74 to 114% of the *a* distance. In every case, where the best predictor of bolt group strength was based on an effective eccentricity exceeding one half the *a* distance, standard holes were used. Four of the tests were either six- or seven-row connections.

The remaining two tests were treated as outliers. One of the tests was a three-row connection in which the support girder was yielded during testing, which was considered unusual. The other outlier was a test in which the applied rotation was 0.053 radian, considerably more than the target. Other connections were subjected to similarly large rotations but did not show an increase in the effective eccentricity. It should be noted that when these two data points are compared to the 14th edition *Manual* design procedure there is still good agreement, even though an eccentricity less than that predicted by the test is used in the 14th edition *Manual* design procedure. This can be explained in part by the 10% reduction in bolt value inherent in the 2010 *Specification*.

In cases where bolt strengths were not reported, it is likely that the actual bolt strengths were greater than the nominal strengths used to calculate the predicted strength of the connection. Underestimating the predicted strength of the connection would lead to an overestimation of the bolt group efficiency and a lower corresponding effective eccentricity. There were 17 tests for which bolt strengths were not reported; of these, the bolt group governed the strengths of 12 tests. Therefore, nearly half of the tests potentially underestimate the effect of the eccentricity. However, in 7 of the 12 cases (tests 2, 3, 24, 25, 26, 27 and 30), the calculated effective eccentricity is more than 20% lower than the recommended eccentricity used in the 14th edition *Manual* design procedure, a considerably larger margin than the reported overstrength of the tested bolts. In 3 of the remaining tests (tests 1, 28 and 29), where the predicted effective eccentricity exceeds the recommended eccentricity used in the 14th edition *Manual* design procedure, the 14th edition *Manual* design procedure limits the plate thickness to less than the tested configuration to increase the ductility of the bolt group. In the final 2 of the 12 tests for which no bolt data were available (tests 4 and 5), the rotational demand on the connection during testing was approximately twice the expected simple beam end rotation.

The use of slip-critical connections should also be addressed. The design procedure contained in the 14th edition *Manual* follows the precedent set by previous editions of the *Manual* in allowing slip-critical connection design values to be used with single-plate shear connections. Because only standard and short-slotted holes are allowed and accommodation of the end rotation is required, the use of slip-critical connections would never be required per the *Specification* for these connections, and AISC discourages the use of slip-critical connections unless required by the *Specifications*. However, the use of slip-critical connection design values was not felt to be detrimental to the performance of the connection, so they have been allowed. Even when designed using slip-critical design values, the bolts in a single-plate shear connection will likely slip into bearing when large end rotations are required.

DESIGN OF THE WELD GROUP

Just as the ratio between the bolt diameter and plate thickness is intended to allow ductile redistribution of moments and accommodation of the simple beam end rotation, the weld is also sized to promote ductile behavior. The *Manual* design procedure requires that the weld size be equal to ⅝ of the plate thickness. A derivation of the weld requirement has been provided by Muir and Hewitt (2009), so only a brief discussion will be provided here. The derivation assumes that the plate must yield prior to weld rupture to ensure ductile behavior. Though most single-plate connections tested had a weld size equal to at least ¾ of the plate thickness, Baldwin Metzger (2006) ran several single-plate connection tests, both extended and conventional configurations, with welds sized to one-half the plate thickness, which confirmed the suitability of the current ⅝ of the plate thickness requirement.

DESIGN OF THE PLATE

Only two of the tests listed in Table 3 were governed by the strength of the plate, and the governing limit states were listed as shear yielding and shear distortion (Sarkar and Wallace, 1992). However, it is clear that the plate must have sufficient strength to resist the required design loads, as required by the *Specification*. The applicable limit states from Section J4 of the *Specification* are shear yielding (Equation J4-3) shear rupture (Equation J4-4) and flexural yielding. Flexural yielding should be checked using the plastic section modulus (Mohr and Murray, 2008). Block shear may also be a governing limit state if the horizontal edge distance does not exceed the vertical edge distance. Buckling will not occur in the plate, because the distance from the weld to the bolt group cannot exceed 3½ in. This can be proven as follows (Muir and Thornton, 2004):

Assuming a = 3.5 in. (the maximum permissible dimension), L = 36 in. for a 12-row connection, F_y = 50 ksi, and t_p = 0.25 in.:

$$\lambda = \frac{L\sqrt{F_y}}{t_p\sqrt{47{,}000+112{,}000\left(\frac{L}{2a}\right)^2}}$$

$$= \frac{36\sqrt{50}}{0.25\sqrt{47{,}500+112{,}000\left(\frac{36}{2(3.5)}\right)^2}}$$

$$= 0.587 < 0.7$$

Therefore, buckling will not govern.

COMPARISON TO TEST RESULTS

A comparison of the 14th edition *Manual* design procedure to the test results is given in Figures 2 and 3. Figure 2 compares the 14th edition *Manual* design procedure to test run with an a dimension equal to 3 in. Figure 3 makes the comparison to test run with an a dimension equal to 3½ in. There appears to be good agreement, and there is only one data point for which the 14th edition *Manual* procedure appears slightly nonconservative.

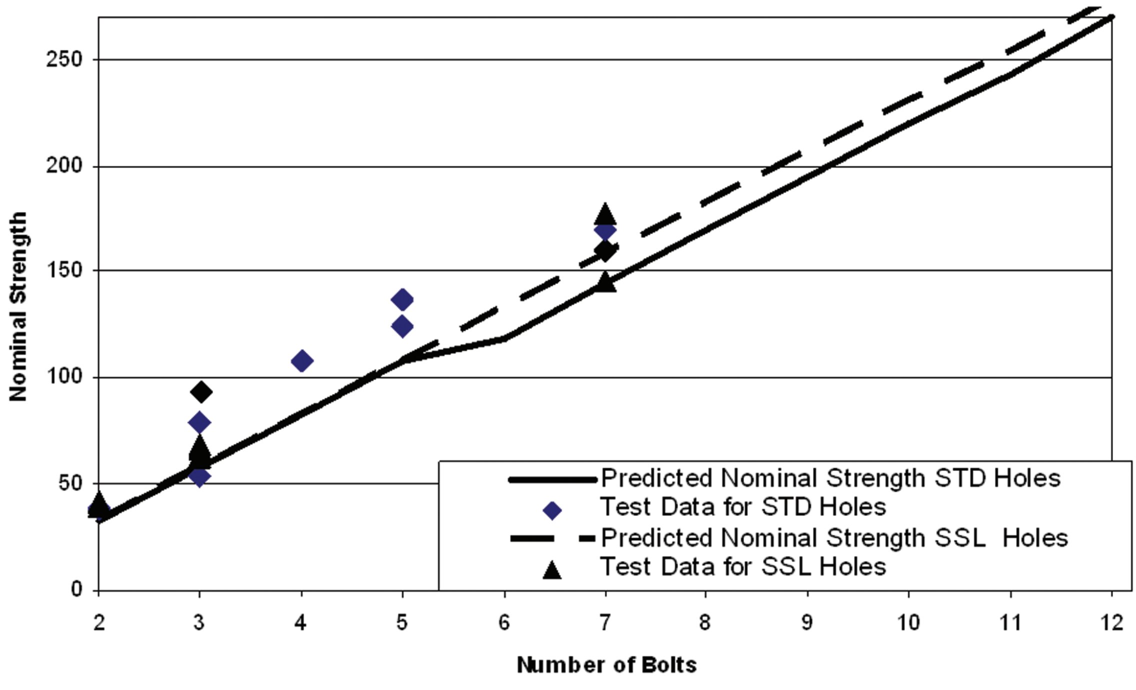

Fig. 2. Comparison of design procedure to test results for a = 3.0 in.

DESIGN EXAMPLE

Given:

Beam: W24×76 (A572 Grade 50); $t_w = 0.44$ in.
Bolts: Six ⅞-in. A325-N (STD holes)
Plate: A572 Grade 50; $t_p = 0.375$ in.; $d_p = 18$ in.

Verify that the plate satisfies the requirements for design as a conventional single-plate shear connection:

Verify number of bolts: $2 \leq n = 6 \leq 12$
Verify distance between the bolt and the weld: $a \leq 3.5$ in.
Verify plate or web thickness: $t_p = 0.375 \text{ in.} \leq d_b/2 - \tfrac{1}{16} \text{ in.} = (0.875 \text{ in.})/2 - \tfrac{1}{16} \text{ in.} = 0.375 \text{ in.}$
Verify horizontal edge distance: $L_{eh} = 1.75 \text{ in.} \geq 2d_b = 2(0.875 \text{ in.}) = 1.75 \text{ in.}$

Determine shear strength of a single bolt:

$$\phi r_b = \phi \frac{d_b^2}{4} \pi F_{nv} = 0.75 \left[\frac{(0.875 \text{ in.})^2}{4} \right] \pi \left(54 \text{ ksi} \right) = 24.4 \text{ kips}$$

Determine bolt bearing strength of plate per bolt:

First determine clear distance, L_c, to edge of plate:

$$L_c = L_e - \frac{d_h}{2} = 1.5 \text{ in.} - \frac{0.9375 \text{ in.}}{2} = 1.03 \text{ in.}$$

$$\begin{aligned} \phi r_{brg} &= \min\left(\phi 2.4\, d_p t_p F_u,\ \phi 1.2 L_c t_p F_u \right) \\ &= \min\left(0.75(2.4)(18 \text{ in.})(0.375 \text{ in.})(58 \text{ ksi}),\ 0.75(1.2)(1.03 \text{ in.})(0.375 \text{ in.})(58 \text{ ksi}) \right) \\ &= \min\left(38.4 \text{ kips},\ 22.6 \text{ kips} \right) \end{aligned}$$

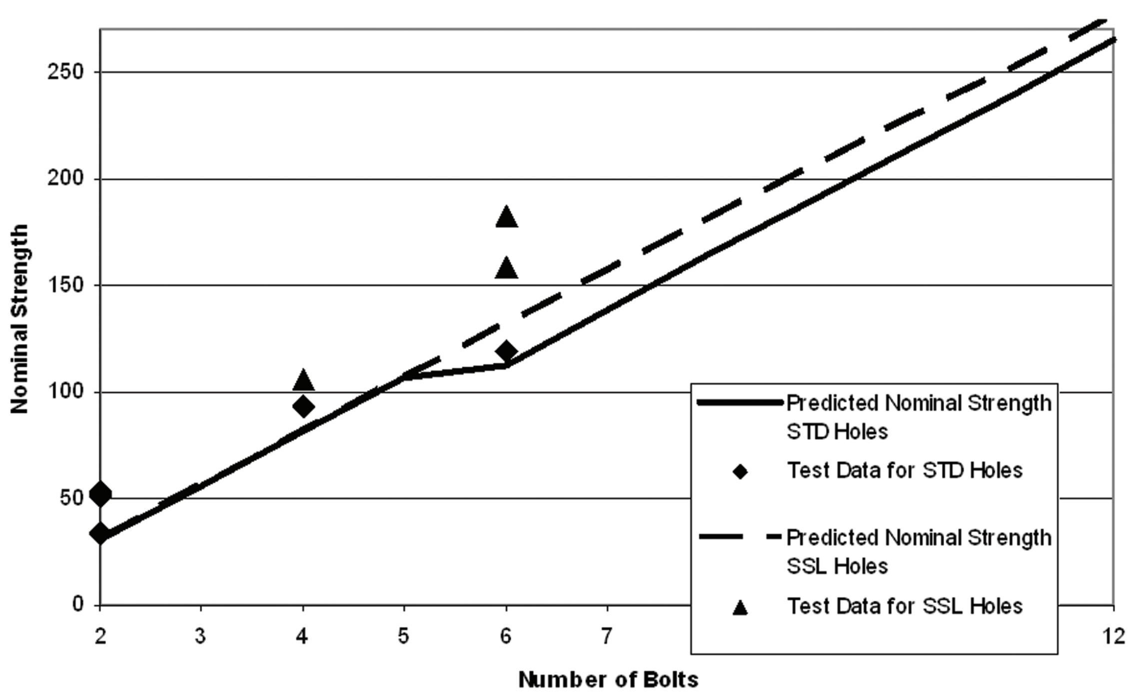

Fig. 3. Comparison of design procedure to test results for a = 3.5 in.

It should be noted that calculating the bearing strength based on lesser of the horizontal and vertical edge distances is conservative and other more exact methods are also acceptable. Because the thickness of the beam web is greater than the thickness of the plate, bearing on the plate will govern.

Determine strength of bolt group:

Because there are six rows of bolts in standard holes, the eccentricity shall be taken as the full distance between the bolts and the weld, 3 in.

From Table 7-7, $C = 4.98$

$$\begin{aligned}
\phi R_b &= \min\left(\phi r_b, \phi r_{brg}\right) C \\
&= 22.6 \text{ kips}(4.98) \\
&= 113 \text{ kips} > 100 \text{ kips} \qquad \textbf{o.k.}
\end{aligned}$$

Determine the shear yielding strength of the plate:

$$\begin{aligned}
\phi R_{vy} &= \phi 0.6 F_y d_p t_p \\
&= 1.0(0.6)(50 \text{ ksi})(18 \text{ in.})(0.375 \text{ in.}) \\
&= 203 \text{ kips} > 100 \text{ kips} \qquad \textbf{o.k.}
\end{aligned}$$

Determine the shear rupture strength of the plate:

$$\begin{aligned}
A_n &= t_p\left[d_p - n\left(d_b + 0.125 \text{ in.}\right)\right] \\
&= 0.375 \text{ in.}\left[18 \text{ in.} - 6\left(0.875 \text{ in.} + 0.125 \text{ in.}\right)\right] \\
&= 4.5 \text{ in.}^2
\end{aligned}$$

$$\begin{aligned}
\phi R_{vr} &= \phi 0.6 F_u A_{net} \\
&= 0.75(0.6)(65 \text{ ksi})(4.5 \text{ in.}^2) \\
&= 132 \text{ kips} > 100 \text{ kips} \qquad \textbf{o.k.}
\end{aligned}$$

Determine flexural strength of the plate:

As noted earlier, the check should be performed using the gross plastic section modulus and buckling of the plate will not govern.

$$Z_g = \frac{t_p d_p^2}{4} = \frac{(0.375 \text{ in.})(18 \text{ in.})^2}{4} = 30.4 \text{ in.}^3$$

$$\begin{aligned}
\phi R_f &= \frac{\phi F_y Z_g}{e} \\
&= \frac{0.9(50 \text{ ksi})(30.4 \text{ in.}^3)}{3 \text{ in.}} \\
&= 456 \text{ kips}
\end{aligned}$$

Size the weld:

$w = \frac{5}{8} t_p = \frac{5}{8}(0.375 \text{ in.}) = 0.234 \text{ in.}$

Therefore, use a ¼-in. weld each side.

Because the horizontal edge distance is greater than the vertical edge distance block shear will not govern.

CONCLUSION

The design procedure for conventional single-plate shear connections contained in the 14th edition of the AISC *Manual* has been revised to accommodate a change in the nominal bolt strength presented in the AISC *Specification*. The new procedure represents a safe and economical approach to these connections based on rational design methods and confirmed by testing.

ACKNOWLEDGMENTS

The authors would like to acknowledge the participation of the AISC Committee on Manuals and Textbooks in developing the design procedure presented in this paper with special thanks to Charlie Carter, Patrick Fortney, William Lindley and Clifford Schwinger, who, along with the authors, comprised the task group that worked to distill the available data into a clear and user-friendly procedure.

SYMBOLS

F_y Minimum specified yield stress, ksi

L_{eh} Horizontal edge distance, in.

L_{ev} Vertical edge distance, in.

R Simple beam end reaction, kips

R_{ult} Ultimate shear strength of the bolt, kips

a Distance from the face of the support to the vertical line of bolts

b Spacing between rows of bolts

d Plate depth

d_b Bolt diameter

e_b Effective (design) eccentricity of the bolt group

n Number of rows

t_p Plate thickness, in.

t_w Beam web thickness, in.

w Weld leg size, in.

δ Relative horizontal movement of the bolts and the connected materials, in.

Δ Deformation of the bolt, in.

λ Slenderness parameter, dimensionless

REFERENCES

AISC (1984), *Engineering for Steel Construction*, American Institute of Steel Construction, Chicago, IL.

AISC (2001), *Manual of Steel Construction: Load and Resistance Factor Design*, 3rd ed., American Institute of Steel Construction, Chicago, IL.

AISC (2005a), *Steel Construction Manual*, 13th ed., American Institute of Steel Construction, Chicago, IL.

AISC (2005b), *Specification for Structural Steel Buildings*, ANSI/AISC 360-05, American Institute of Steel Construction, Chicago, IL.

AISC (2010), *Specification for Structural Steel Buildings*, ANSI/AISC 360-10, American Institute of Steel Construction, Chicago, IL.

Astaneh, A., Call, S.M. and McMullin, K.M. (1988), "Design of Single Plate Framing Connections," Report No. UCB/SEMM-88/12, Department of Civil Engineering, University of California, Berkeley.

Astaneh, A., Call, S.M. and McMullin, K.M. (1989), "Design of Single Plate Shear Connections," *Engineering Journal*, AISC, Vol. 26, No. 1.

Astaneh, A., McMullin, K.M. and Call, S.M. (1993), "Behavior and Design of Steel Single Plate Shear Connections," *Journal of Structural Engineering*, ASCE, Vol. 119, No. 8, pp. 2421–2440.

Astaneh, A. and Porter, K.A. (1990), "Design of Single Plate Connections with Snug-Tight Bolts in Short Slotted Holes," Report No. UCB/SEMM-90/23, Department of Civil Engineering, University of California, Berkeley.

Astaneh, A. and Shaw, A.L. (1992), "Experimental Study of Single-Plate Steel Beam-to-Girder Connections," Report No. UCB/SEMM-92/13, Department of Civil Engineering, University of California, Berkeley.

Baldwin Metzger, K.A. (2006). "Experimental Verification of a New Single Plate Shear Connection Design Model," Master's Thesis, Virginia Tech, Blacksburg, VA.

Creech, D. (2005), "Behavior of Single Plate Shear Connections with Rigid and Flexible Supports,: Master's Thesis, North Carolina State University, Raleigh, NC.

Hormby, D.E., Richard, R.M. and Kreigh, J.D. (1984), "Single-Plate Framing Connections with Grade-50 Steel and Composite Construction," *Engineering Journal*, AISC, Vol. 21, No. 3.

Lipson, S.L. (1968), "Single-Angle and Single-Plate Beam Framing Connections," Canadian Structural Engineering Conference, Toronto, ON, Canada, pp. 141–162.

Mohr, B.A., and Murray, T.M. (2008), "Bending Strength of Steel Bracket and Splice Plates," *Engineering Journal*, AISC, Vol. 45, No. 2.

Muir, L.S. and Hewitt, C. (2009), “Design of Unstiffened Extended Single-Plate Shear Connections,” *Engineering Journal*, AISC, Vol. 46, No. 2.

Muir, L.S. and Thornton, W.S. (2004). “A Direct Method for Obtaining the Plate Buckling Coefficient for Double Coped Beams,” *Engineering Journal*, AISC, Vol. 41, No. 3.

Richard, R.M., Gillet, P.E., Kreigh, J.D. and Lewis, B.A. (1980), “The Analysis and Design of Single Plate Framing Connections,” *Engineering Journal*, AISC, Vol. 17, No. 2.

Richard, R.M., Kreigh, J.D. and Hormby, D.E. (1982), “Design of Single Plate Framing Connections with A307 Bolts,” *Engineering Journal*, AISC, Vol. 19, No. 4.

Sarkar, D. and Wallace, B. (1992), “Design of Single Plate Framing Connections,” Draft Research Report, Report No. FSEL/AISC 92-01, University of Oklahoma, Norman, OK.

White, R.N. (1965), “Framing Connections for Square and Rectangular Structural Tubing,” *Engineering Journal*, AISC, Vol. 2, No. 3.