Simple Framing Connections to HSS Columns

Dr. Donald R.Sherman

Author

Dr. Donald R. Sherman is professor of civil engineering at the University of Wisconsin in Milwaukee. He received his bachelor of science and master of science degrees from Case Institute of Technology and his Ph.D. from the University of Illinois in 1964. As a registered professional engineer he has worked as a test engineer for the Pittsburgh Testing Laboratories and as a design engineer for Exxon Research & Development Company.

Since joining the University of Wisconsin in 1966, Dr. Sherman has been active in research and consulting in the design of metal structures, including tubular construction, metal buildings and cranes. He has written a large number of publications and reports concerning the design of tubular steel members and connections. He is director of the University of Wisconsin Structural Testing Laboratory which is involved in many areas of testing large-scale structural components.

Dr. Sherman is a member of the American Institute of Steel Construction's Specification Committee and chairs the task committee on Tension and Compression Members and the task committee on Tubular Structures. He is also the current chairman of the Structural Stability Research Council and has been involved in its activities for over 20 years. As a member of the American Society of Civil Engineers, he has been active on several national technical committees, including a term as chairman of the Committee on Metals. He has also held local offices in American Society of Civil Engineers and the Society for Experimental Mechanics.

Summary

There are a variety of simple-shear connections that are used to frame wide-flange beams to columns. Although, designers are familiar with using these connections with wide-flange columns, there are sometimes questions on how they perform with HSS columns. The concerns are whether there is a limit state in the HSS that could govern the connection design or if local distortion of the HSS wall could reduce the column capacity.

This paper summarizes the results of tests on nine types of standard simple connections. Except for the end-plate connection, all connecting elements are welded to the HSS column and bolted to the beam web. The HSS column sizes include both thick and thin walled sections and two beam L/d are used to provide data for both stiff and flexible beams. The only HSS limit state that was observed was a punching shear failure when thick shear tabs were used with thin HSS. A simple criteria for the thickness ratio can be used to prevent this failure mode.

This paper presents information on the local strain levels in the HSS, the eccentricities of the shear reaction relative to the bolt or weld lines and the connection capacities. Even with the most severe local strains in the HSS columns, further data is presented to show that column capacity is not affected for HSS that are not classified as thin-walled. The other major conclusion is that the values for connection capacities give by AISC are equally valid for HSS and wide-flange columns.

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SIMPLE FRAMING CONNECTIONS TO HSS COLUMNS

INTRODUCTION

In recent years, the use of square and rectangular hollow structural sections (HSS) as columns in building construction has become increasingly popular. This is due to the efficiency of the shape as a compression member and the aesthetics when the steel column is exposed. For connecting wide-flange beams, designers have adapted many of the standard simple connections typically used with wide-flange columns, even though little data is available regarding their use with HSS columns. The connection capacities published by AISC are the basis for designs. However, concerns are still raised regarding simple framing connections used to connect wide-flange beams to HSS columns. The concerns are whether there is a limit state in the HSS that could govern the connection design or if local distortion of the HSS wall could reduce the column capacity.

This paper presents an overall discussion of nine different types of simple framing connections used with HSS columns. These are:

- shear tabs
- through-plates
- double angles
- tees with vertical fillet welds
- tees with flare bevel groove welds
- unstiffened seated connections
- single angles with L shaped fillet weld
- single angles with two vertical fillet welds
- web end plates

In all but the web end plate, the connecting elements are welded to the HSS column and bolted to the web of the wide-flange beam, with the exception of the seat angle where the flange bears on the outstanding leg. For the web end plate, the plate is welded to the beam and bolted to the HSS column using a flow-drill process that produces a tapped hole that replaces a nut in blind connections. (See the Appendix for more information on flow-drilled holes.)

The primary focus of the paper is comparisons of the limit states considered in the AISC manual for connections [1] with experimental capacities. Potential limit states in the HSS are discussed and evaluated. Strain measurements indicate the relative degree of distortion in the HSS wall and data is presented to verify that the connection producing the highest strain levels in compact HSS columns does not reduce the axial load capacity. Finally the relative economics of the various types of connections are discussed.

DESCRIPTION OF THE TEST PROGRAM AND SPECIMENS

All of the connection tests used the same basic setup shown in Fig. 1. A short segment of HSS column is held vertical and the connected wide-flange beam that is simply supported at the far end is loaded with a concentrated load at a distance b from the face of the HSS. By equating the relations for shear and end rotation between a uniformly loaded beam and the test beam with concentrated load, the length of a simulated uniformly beam is obtained.

$$L_u = \sqrt{2}\sqrt{L_{beam}^2 - (L_{beam} - b)^2} \tag{1}$$

Two different beam sizes were used to provide information on both stiff and flexible beams. The flexible W12x87 had a length/depth ratio L_u/d of 23 while the W18x71 had L_u/d of 9.8. The former was used with three bolts in the web connection while the latter used either three or five bolts.

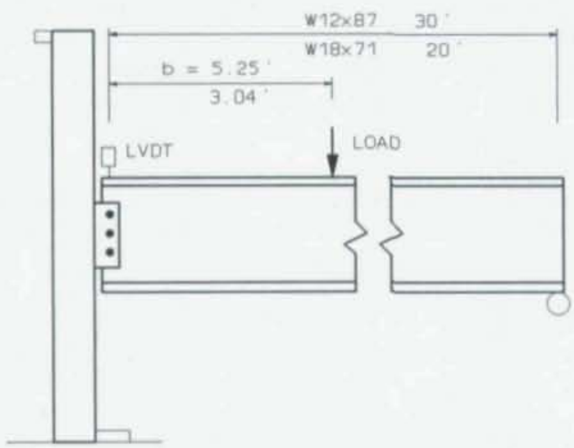

FIG. 1 - CONNECTION TEST SETUP

The thirteen tests of shear tab connections were reported in a previous paper [7]. These tests included HSS with five different flat width/wall thickness ratios (b/t) and both snug and fully tensioned bolts. Since the bolt tightness had little influence on the ultimate capacity, only average results are summarized in this paper.

For the remaining tests, two thicknesses of 10x10" HSS were used, producing b/t ratios of 36 and 16. All tests used 3/4" A325 bolts in standard holes with 3" spacing. Standard connection details in the AISC manual [1] were followed. The variables in the test program are summarized in Table 1. In addition to the TAB tests, fifteen additional tests on different types of connections are indicated. The first digits in the symbols used to designate the specimens are the b/t of the HSS. The following letters indicate the type of connection, then a digit gives the number of bolts and finally the F or S designate a FLEXIBLE or STIFF beam.

The gage distance from the face of the HSS to the bolt line was 2¼" for the Double Angles and 2-3/4" for the Single Angles. All other connection that were bolted to the beam web had gage distances of 3".

TABLE 1 - TEST PROGRAM SUMMARY

TYPE	SYMB	HSS b/t	BEAM	BOLTS	CONNECTING ELEMENT SIZE	CONNECTING ELEMENT LENGTH
DOUBLE L	36DL3F	36	FLEXIBLE	3	L3.5x 4x 1/4	8.5"
"	16DL5S	16	STIFF	5	"	14.5"
TEE, Vert. Welds	36TV3F	36	FLEXIBLE	3	WT 7x 21.5	8.5"
"	36TV3S	36	STIFF	3	"	8.5"
"	16TV3S	16	STIFF	3	"	8.5"
"	16TV5S	16	STIFF	5	"	14.5"
TEE, Flare Welds	36TF3F	36	FLEXIBLE	3	WT 7x 30.5	8.5"
"	16TF5S	16	STIFF	5	"	14.5"
SEAT	36SeF	36	FLEXIBLE		L6x 4x 1/2	6"
"	16SeS	16	STIFF		"	6"
SINGLE L, AISC Weld	36SL3F	36	FLEXIBLE	3	L3x 4x 3/8	8.5"
"	16SL5S	16	STIFF	5	"	14.5"
SINGLE L, Vert. Welds	36SV3F	36	FLEXIBLE	3	"	8.5"
"	16SV5S	16	STIFF	5	"	14.5"
END PLATE	36EP3F	36	FLEXIBLE	6 tot	6x 5/16 PLATE	8.5"
TAB	5TB3F	5	FLEXIBLE	3	5/16" PLATE	9"
"	5TB5S	5	STIFF	5	"	15"
"	10TB3F	10	FLEXIBLE	3	"	9"
"	16TB3F	16	FLEXIBLE	3	"	9"
"	16TB3S	16	STIFF	3	"	9"
"	40TB3F	40	FLEXIBLE	3	"	9"
"	45TB5S	45	STIFF	5	"	15"
"	45TB3F	45	FLEXIBLE	3	"	9"
"	45TB3F*	45	FLEXIBLE	3	1/4" PLATE	9"

MATERIAL PROPERTIES

Tension coupons were removed from all the various types and sizes of connecting elements. For TEE connections, the coupon was taken from the stem that attached to the beam web. The actual thickness, yield strength and ultimate strength are reported in Table 2. The stress-strain curves showed a flat yield region except for TEE with flair bevel welds, which had a rounded curve.

TABLE 2 - MATERIAL PROPERTIES FOR CONNECTING ELEMENTS

TYPE	SPECIMEN	THICKNESS (in.)	F_y (ksi)	F_u (ksi)
DOUBLE L	ALL	.321	43.3	68.1
TEE, Vert. Welds	ALL	.315	49.6	71.1
TEE, Flare Welds	ALL	.387	39.4	66.6
SEAT	ALL	.500	51.8	68.8
SINGLE L	ALL	.368	43.6	64.1
END PLATE	ALL	.319	48.9	70.7
TAB	5TB3F 10TB3F 16TB3F 16TB3S	.326	47.7	73.0
"	5TB3F 5TB5S 16TB3F 40TB3F 45TB3F 45TB5S	.303	55.5	69.5
"	45TB3F*	.239	49.3	72.6

Tension coupons were also taken from the middle of a side of all HSS sections. The results from these tension tests are in Table 3. All of these stress-strain curves were rounded and the reported yield was obtained by a 0.2% offset.

TABLE 3 - MATERIAL PROPERTIES OF HSS

TYPE	HSS	b/t	THICKNESS (in.)	F_y (ksi)	F_u (ksi)
TAB	4x6x1/2	5	.470	59.5	71.8
"	4x6x5/16	10 & 16	.305	53.7	63.8
"	12x8x1/4	45	.236	49.6	64.2
"	8x3x3/16	40	.173	53.7	71.2
ALL OTHERS	10x10x1/2	16	.466	53.2	69.2
"	10x10x1/4	36	.234	53.2	66.5

CONNECTION CAPACITIES

Tables 4-9 summarize the capacities for the various types of connections. In addition to the maximum experimental test load in the last column, the capacities predicted for the various AISC limit states are also given for the bolts, connecting elements and welds as appropriate. The computed capacities are based on the nominal properties of bolts and welds, but on the actual thickness, yield and ultimate stresses for the connecting material. Nominal resistances with no resistance factor are reported. The capacity listed in bold type is the controlling limit state. Each type of connection is discussed separately.

In several cases the connecting material had higher yield strength than expected in the planning stages for the test program. Therefore, most tests were terminated when the yield strength of the beam was reached. The relative vertical displacement between the face of the HSS column and the end of the beam was measured with a displacement transducer. This displacement was essentially the shear and bearing distortion of the connecting element. For test loads reported in the tables that are followed by a + sign, the shear-distortion curves were still essentially linear at the maximum load, indicating that a ductile failure was not imminent. For other loads, the curves had flattened in a manner typical of approaching a ductile ultimate load. It should be noted, however, that all tests showed some sign of distress in the whitewash coating on the connection.

Except for the double angles and end plate, the nominal shear capacities of the bolts are based on no threads in the shear plane. There was no evidence of shear distortion in any bolts after the tests were disassembled. However, there were bearing distortions of the bolt holes connecting material bolted to the beam web.

Double Angles

For the double angles reported in Table 4, no eccentricity was considered in the nominal strength of the bolts or angles and the eccentricity for the welds was the in-plane type proposed by Blodgett [2] and used in the AISC connection manuals for many years. Except for the end plate, the double angle was the strongest connecting element tested. This was due to the combined thickness of the beam web legs of the angles. Theoretically, the welds dictated the connection strength and the test loads approached or exceeded the nominal weld strengths.

TABLE 4 - DOUBLE ANGLE CONNECTION STRENGTHS (kips)

SPECIMEN	BOLT		CONN. ELEM.			WELD	TEST
	SHEAR	BEAR.	YIELD	RUPT.	BLOCK		
36DL3F	127	236	142	154	181	**71**	80+
16DL5S	212	393	242	266	293	**163**	147+

Slight cracking was observed in the whitewash at the ends of the welds in both tests. For the 3 bolt connection, the crack was on the weld, but for the 5 bolt connection, it was at the toe of the weld on the HSS face. In both tests, the most extensive yielding was on the HSS legs of the angles. In the 3 bolt connection, the top of the angles had separated about 3/16" from the HSS, while the separation was on the order of 1/8" for the 5 bolt connection. No gross distortion or other indication of failure in the HSS was observed.

TEE Connections

The nominal capacity of all the TEE connections was determined by the shear strength of the bolts. These connections with welding at the edges of the TEE flanges are considered flexible in the AISC Manual and eccentricity is considered in the bolts and direct shear in the welds. The test loads exceeded the nominal bolt capacities and were at the direct shear yield or shear fracture capacities of the stem. This corresponded with the extensive pattern of whitewash cracking observed on the stems. In tests 36TV3S and 16TV5S, a crack was observed at the bottom of the stem in line with the bolts as well as at the bottoms of the bolt holes, indicating the beginning of a shear rupture failure. Major cracks were also observed in the bolt holes of specimen 36TV3S.

TABLE 5 - TEE CONNECTION STRENGTHS (kips)

SPECIMEN	BOLT		CONN. ELEM.					WELD	TEST
	SHEAR	BEAR.	YIELD	RUPT.	BLOCK	FLEX. YIELD	FLEX. RUPT.		
36TV3F	46	70	79	78	127	187	186	126	62
36TV3S	46	66	74	73	119	174	174	126	77
16TV3S	46	66	74	73	119	174	174	126	79
16TV5S	103	157	136	136	185	547	542	215	124
36TF3F	46	81	78	91	139	185	215	126	76
16TF5S	103	181	133	157	205	537	623	269	136

The TEE connections were tested with the most variation in the HSS columns and beam stiffness. However, neither of these factors affected either the nominal strengths nor the condition of the HSS at the maximum test load. No distress was observed in any of the HSS faces. Separation from the HSS at the top of TEE was on the order of 1/16", but there was more pushing in of the HSS wall at the bottoms of the TEE. Flare bevel welds did not shown any difference in behavior from the TEE connection with vertical welds on the HSS face. However, slight whitewash flaking at the ends of welds were observed in 36TV3S and 36TF3F with the thinner HSS.

Seat Angles

The nominal weld resistance for the seat angle connections in Table 6 considered the eccentricity based on a ½" setback that was actually used in the tests. Following the procedure by Garrett and Brockenbrough [4] as recommended by AISC, the bearing length (N) was less than 2.5 times the distance from flange face of the beam to the toe of the web fillet (k) so that the reaction was assumed to act at (N + 2.5k)/4 from the end of the beam (Case II in the paper.) Since the procedure resulted in a negative value of N, N was taken as zero in the flexural yield calculation.

TABLE 6 - SEAT ANGLE CONNECTION STRENGTHS (kips)

SPECIMEN	CONN. ELEM.		WELD	TEST
	SHEAR YIELD	FLEX. YIELD		
36SeF	124	64	81	50+
16SeS	124	63	81	72

Both tests were terminated when yielding in the beam web was observed. At this load, some yielding was observed in the vertical leg of the seat, but there was no evidence in distress in the HSS or the welds. The outstanding leg was bent so that the observed bearing length to the beam was less than 1¼", which was the position of the erection bolts through the flange.

Single Angles

The nominal resistances for the single angles in Table 7 ignore any eccentricity for the bolts. For the L shaped AISC weld, the in-plane eccentricity was included. Since the connections with vertical welds are not an AISC standard, the weld capacities were based on an out-of-plane eccentricity to the bolt line and half the value from the AISC ultimate strength eccentric load table is reported. Using half the value conservatively assumes that only the weld at the heel of the angle is effective, since it is in line with the outstanding leg.

TABLE 7 - SINGLE ANGLE CONNECTION STRENGTHS (kips)

SPECIMEN	BOLT		CONN. ELEM.			WELD	TEST
	SHEAR	BEAR.	YIELD	RUPT.	BLOCK		
36SL3F	80	127	82	83	92	**74**	58
16SL5S	133	212	140	143	152	**126**	134
36SV3F	80	127	82	83	92	**51**	75
16SV5S	133	212	140	143	152	**103**	133

The two connections with the AISC shaped weld had extensive yielding in both the outstanding and the HSS legs of the angle and small cracks were found at the bottom of the lower two bolt holes. The 3 bolt connection had slight whitewash cracking at the ends of the welds, but the 5 bolt connection indicated yielding along most of the length of the vertical weld. The vertical distortion of the connections was 3/8" and the separation of the top of the angle from the HSS was 3/16" for the 3 bolts connection and 1/4" for the 5 bolt connection. Separation of the heel of the angle from the HSS was evident at the very early stages of loading. Flexural yielding of the HSS leg is a limit state that should be considered and probably controlled the capacity.

The connections with vertical welds also had extensive yielding in both legs and the 5 bolt connection had an initiation of shear rupture at the bottom of the bolt line. Both specimens had small cracks at the bottom of the bolt holes. The 5 bolt connection had whitewash cracking at the top end of the center weld, while the 3 bolt connection had slight whitewash cracking in the HSS wall at the ends of both welds. Vertical distortions were again on the order of 3/8" but separation was restricted by the weld.

End Plate

The end plate connection had the highest nominal strength. The bolt and connecting element strengths were similar to the double angle but the weld strength was higher due to the lack of eccentricity.

TABLE 8 - END PLATE CONNECTION STRENGTH (kips)

SPECIMEN	BOLT		CONN. ELEM.			WELD	TEST
	SHEAR	BEAR.	YIELD	RUPT.	BLOCK		
36EP3F	127	244	159	159	192	**119**	79+

The test was terminated due to the beam capacity, although shear yield lines were extensive, especially in the lines of the bolts.

Shear Tabs

The nominal bolt resistances in Table 9 are based on the eccentricities for the rigid (rather than flexible) support in the AISC Manual, since these gave results closer to the test values. Tests 5TB3F and 16TB3F are reported twice since the specimens for snug and fully tensioned bolts had different materials for the tabs.

TABLE 9 - SHEAR TAB CONNECTION STRENGTH (kips)

SPECIMEN	BOLT		CONN. ELEM.			WELD	TEST	
	SHEAR	BEAR.	YIELD	RUPT.	BLOCK		LOAD	MODE
5TB3F	**69**	111	84	91	99	134	75+	
5TB3F	**69**	99	91	81	92	134	74+	
5TB5S	**125**	178	151	134	146	223	143	A
10TB3F	**69**	111	84	91	99	134	73+	
16TB3F	**69**	111	84	91	99	134	70	A
16TB3F	**69**	99	91	81	92	134	70	A
16TB3S	**69**	111	84	91	99	134	84	D
40TB3F	**69**	99	91	81	92	134	71	A,C
45TB5S	**125**	178	151	134	146	223	132	A
45TB3F	**69**	99	91	81	92	134	70	A,D
45TB3F*	69	81	**64**	66	73	134	60	A,B

MODES: A - Significant bearing distortion
B - Shear fracture crack started
C - Punching shear in HSS wall
D - Weld crack or surface tear of HSS beneith the weld

In addition to the potential failure modes observed in the table, all of the specimens had considerable gross yielding of the shear tab. The punching shear failure of the HSS wall for 40TB3F was actually in two specimens, one with snug bolts and an identical test with fully tensioned bolts.

Strength Limit States

With the wide range of connection types variables reported in this paper, only one limit state was identified in the tube of the HSS. This was a punching shear failure when a thick shear tab was used with a thin HSS. A simple criteria to prevent this type of failure can be derived from an inequality that the yield force in a unit depth of shear tab does not exceed the through-thickness shear rupture strength on two planes of a unit length of the HSS wall.

$$F_{y(tab)}\, t_{tab} < 2\,(0.6 F_{u(HSS)}\, t_{HSS} \qquad (2)$$

or

$$t_{tab} < 1.2 \frac{F_{u(HSS)}}{F_{y(tab)}}\, t_{HSS} \qquad (3)$$

Yield line distortion of the face of the HSS was never a limit state. For a simply supported beam, there are limiting end slopes that prevent unrestrained distortion of the HSS. Upon careful examination or measurement, pushing in of the HSS wall at the bottom and pulling out at the top of the connections could be observed. However, gross distortions of all the typical simple connections tested were much more evident. The limit states that controlled were those associated with connecting elements, welds or bolts.

STRAIN IN HSS WALL

In order to determine the effect of the connection types on local distortion of the HSS column, strain gages were mounted at the center of the wall one inch below the connecting element. The transverse strains measured or extrapolated at 50 kips shear in the connection are summarized in Table 10. Previous unreported data from tests on shear tab and through-plate connections are included. The information is listed in descending order of the magnitude of the compressive strain. Positive values in the table are the result of Poisson's effect combined with compressive longitudinal strains.

Connections such as tabs and single angles that have load transfer through a weld at the center of the HSS have the highest transverse strains. These will typically exceed yield. An exception to this in the through-plate that inherently reinforces the center of the wall. Connections with welds near the sides of the HSS have significantly less transverse strain at the center of the wall. The end plate and seat angle connections produce little transverse strain. Longer connections with five bolts produce less transverse strain than 3 bolt connections and HSS with thinner walls or higher b/t tend to have larger strains. These trends are probably evident, but the data in the table quantify the conclusions.

EFFECT OF HSS DISTORTION ON COLUMN STRENGTH

In order to address the question of whether local distortion of the HSS has a detrimental effect on the column capacity, a series of tests were conducted to compare the influence of shear tab and through-plate connections. These types of connections represent the extremes of inducing transverse strain into the HSS wall. A previous paper [6] presented test results leading to a conclusion that there was no significant column strength reduction between shear tab connections and through-plate connections. However, this conclusion was based on only four tests using HSS with a b/t ratio of 16. Recently similar column tests were conducted with b/t ratios of 29 and 40 [5]. This study with eight tests included symmetric connections on both sides of the HSS and unsymmetric connections on just one side. Both snug and tight bolts were included in the original four tests, but only snug tightened bolts were used in the eight later tests.

The test setup for all the column tests is shown in Figure 2. In these tests, the beams were loaded to about 70% of the connection capacity and then a load was applied to the top of the column until a column buckling failure occurred in the lower portion.

Table 11 presents the column strengths as ratios of the maximum experimental load divided by the yield load given by area times the static yield strength from a tension coupon taken from the wall of the HSS. The nondimensional wall slenderness of the HSS is defined as

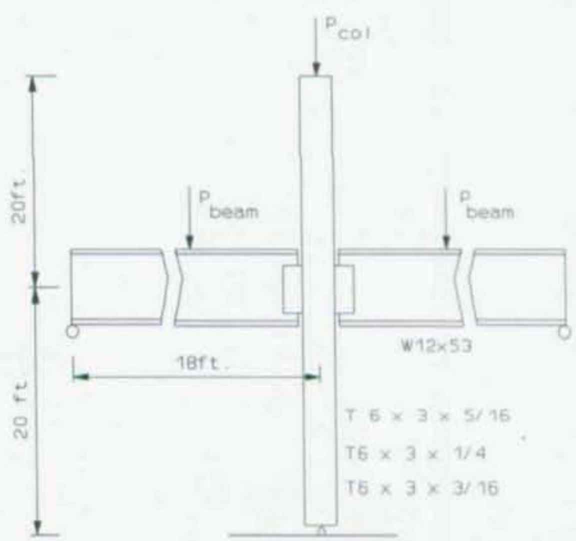

FIG. 2 - COLUMN TEST SETUP

$$\alpha = \frac{\sqrt{E/F_y}}{b/t} \tag{4}$$

In the U.S., a thin-walled tube is defined as one having α less than 0.67.

TABLE 10 - TRANSVERSE STRAIN IN HSS AT 50 kips SHEAR (μin/in)

TYPE	BOLTS	HSS		STRAIN
		t (in.)	b/t	
TAB	3	5/16	19	-3900*
"	3	3/16	40	-3870*
"	3	1/4	29	-3800*
SINGLE ANGLE Vert. Welds	3	1/4	36	-2100
TAB	3	1/4	45	-2100
SINGLE ANGLE AISC Weld	3	1/4	36	-1380
TAB	5	1/4	45	-1200
TEE Vert.	3	1/4	36	-1100*
TEE Flare	3	1/4	36	-1100
SINGLE ANGLE Vert. Welds	5	1/2	16	-1050
DOUBLE ANGLE	3	1/4	36	-975
TAB	5	1/2	5	-900
TEE Vert.	3	1/2	16	-750
DOUBLE ANGLE	5	1/2	16	-600
SINGLE ANGLE AISC Weld	5	1/2	16	-450
TEE Vert.	5	1/2	16	-380
END PLATE		1/2	36	-300
TEE Flare	5	1/2	16	20
SEAT		1/4	36	40
THRU-PLATE	3	5/16	19	55
SEAT		1/2	16	60
THRU-PLATE	3	3/16	40	475
"		1/4	29	700

* average of more that one tests

TABLE 11 - COLUMN STRENGTHS FOR TABS vs. THROUGH-PLATE TESTS

b/t	α	CONNECTION	P_{ult}/P_y	
			TWO SIDES	ONE SIDE
15	1.39	Thru-Plate, Tight Shear Tab, Tight Thru-Plate, Snug Shear Tab, Snug	0.53 0.51 0.50 0.49	
29	0.89	Thru-Plate Shear Tab	0.63 0.61	0.42 0.46
40	0.60	Thru-Plate Shear Tab	0.58 0.45	0.42 0.42

The tests with connection on two sides failed with sudden buckles while the unsymmetric tests failed gradually in bending.

The conclusion from Table 11 is that shear tab connections used with HSS column that are not thin-walled will develop essentially the same column strength as those where the wall is reinforced with a through-plate. With thin-walled HSS, shear tabs may have a detrimental effect of the axial column capacity. For connections on only one side of the HSS column, there is no strength reduction for using shear tabs. It is safe to assume that these conclusion hold for other types of simple connections that have smaller transverse strains.

CONNECTION COSTS

Since a number of connection types were being studied at the same time, an excellent opportunity was presented to determine relative costs. Relative costs for the 3 bolt connections used in the study are listed in Table 12 based on the least expensive (single angle with AISC weld) being given a value of unity. The costs are for the connecting material and the labor to fabricate the connection, including welding to the HSS or to the beam web in the case of the end plate. The cost of the end plate is somewhat uncertain since flow-drilling the holes is not a routine shop operation at this time. The costs do not reflect shop preparation of the beam or field erection.

TABLE 12 - RELATIVE CONNECTION COSTS

TYPE	COST
SINGLE ANGLE, AISC Weld	1.00
SHEAR TAB	1.05
SINGLE ANGLE, Vert. Welds	1.17
SEAT ANGLE	1.36
DOUBLE ANGLES	1.50
TEE, Vert. Welds	1.62
END PLATE	2.15
THROUGH-PLATE	2.25
TEE, Flare Welds	2.42

CONCLUSIONS

The test programs have shown that the variety of simple framing connections typically used in steel construction can confidently be used with HSS columns that are not classified as thin-walled. The tabulated connections capacities and criteria for evaluating connections that appear in the AISC Manual [1] can be applied when HSS columns are used. The only additional limit state that must be considered is a simple thickness criteria for punching shear of the HSS wall when shear tab connections are used.

Connections that involve welding at the center of an unreinforced HSS wall will produce local strains that exceed yield. However, the resulting wall distortions are barely noticeable and not nearly as great as the distortions of the connecting elements. The local distortion in the HSS wall has negligible influence on the column capacity as long as the HSS is not classified as thin-walled. This applies to connections on one side of the HSS or symmetric on both sides.

Careful consideration should be given to the type of connection since the connection cost can vary by a factor of 2½.

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APPENDIX - FLOW-DRILLED HOLES

Flow-drilling is a process to produce a threaded hole in a steel plate to permit blind bolting when one side of the plate is inaccessible. An evaluation of the shear and tension performance of bolts in flow-drilled holes has been made by Sherman [8]. FLOWDRILLING [3] describes the process where a hole is forced through a flat plate by a carbide conical tool rotating at sufficient speed to produce high rapid heating which softens the material in a local area. The material that is displaced as the tool is forced through the plate forms a truncated hollow cone (bushing) on the inner surface and a small upset on the outer surface. Tools can be obtained with a milling collar so that the material on the outer surface is removed, producing a flat surface allowing parts to be brought in close contact. A cold-formed tap is then used to roll a thread into the hole without any chips or removal of material.

It is recommended that flow-drilled holes can be produced in plate thicknesses up to half the hole diameter. Hence the range of common bolt sizes and HSS thicknesses are compatible. In these thicknesses, the resulting threads in a flow-drilled hole is approximately equal to the depth of heavy hex nuts. In addition, the flow-drilling produces a local hardness of the plate material that results in properties similar to a nut. The combinations of HSS thicknesses and bolt diameters that were evaluated are indicated in Table 13.

TABLE 13 - MATERIAL THICKNESS AND BOLT DIAMETER COMBINATIONS

MATERIAL	BOLT DIAMETER (in.)				
t (in.)	1/2	5/8	3/4	7/8	1
3/16	X	X			
1/4	X	X	X		
5/16		X	X	X	
3/8			X	X	X
1/2					X

The bolts used for the shear and tension evaluation were purchased in the U.S. using customary units for size and the tools were supplied in metric sizes. In addition, there is some uncertainty in predicting the final hole size since cooling takes place after the tool is forced the through the material. This presented a problem in the 7/8" diameter bolt tests where the hole was oversized and the final threads were poorly developed. In other sizes, threads comparable to typical nuts where produced. The threaded holes were produced in all size combinations with virtually no permanent distortion of the HSS walls.

In the shear tests, bolts were tested in both the snug tight and fully tensioned conditions while in the tension tests, all bolts were fully tightened by the turn-of-the-nut method (actually turn of the bolt head.) Three specimens of each type and size combination were tested. The bolts were high strength A325 that have a specified proof stress of 92 psi and a tensile strength of 120 psi. The results of the tests are shown in Fig. 3.

In the tension tests, there were three failures by fracture of the bolts and all other failures were the result of stripping the threads in the hole. However, except for the 7/8" bolts that had the poorly formed threads due to the oversize holes, all test loads exceeded the specified tension of the fastener.

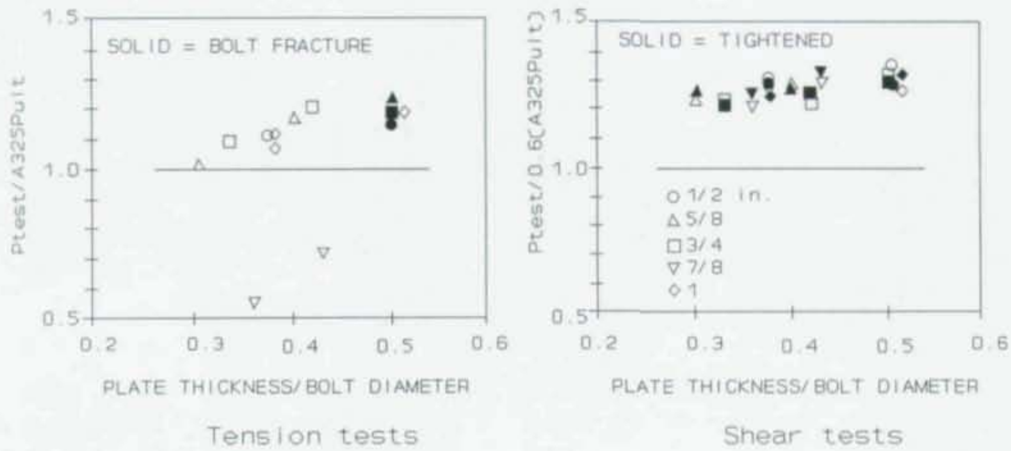

FIG. 3 - TEST RESULTS

The shear strength data has been normalized by 0.6 times the specified bolt tensile strength. At the time of the evaluation, this was the ratio between shear and tension capacities when threads are in the shear plane, which will always be the case with flow-drilled holes. (The current ratio is 0.533, which improves the tests results even further.) In all cases, including the 7/8" bolts with poorly threaded holes, the tests strength exceeded the required capacity.

Based on these tests, flow-drilling has the potential for use in blind bolting to HSS columns. However, there are practical considerations in that the fabricator must have drilling equipment with rotational speeds, torque and thrust required for forming the holes.