Designing With Structural Tubing

DONALD R. SHERMAN

INTRODUCTION

Although the use of structural tubing as truss members and columns in building construction continues to increase in the U.S., it has not reached the proportion found in some countries where it approaches half the structural steel tonnage. Many designers still think of structural tubing as a new product, even though round tubes were used in some of the earliest steel structures. Steel design specifications were primarily developed from experience with hot-rolled sections and it was not until the late 1940s that criteria for circular tubes appeared in U.S. design specifications. Technology for efficiently mass producing square and rectangular structural tubes has developed in the past few decades, generating research on member and connection behavior with subsequent development of design criteria.

There are several advantages associated with the tubular section as opposed to shapes with open profiles.

- Since the moment of inertia is the same about any axis for round and square tubes, these sections are the most efficient for columns that have the same end restraints in any direction. For different end restraints about the principle axes, a rectangular tube can be selected with proportions that provide the same column slenderness ratio about the major and minor axes, thereby providing the most efficient use of material. The section modulus can also be optimized for beams in biaxial bending.
- The torsional stiffness of the closed shape and the high weak axis moment of inertia minimize the requirements for lateral bracing of tubular beams. Round and square sections require no lateral bracing and rectangular beams bending about the major would require lateral bracing only for extreme depth to width ratios. The torsional stiffness and strength also make tubes the ideal shape for space frame construction.
- The smooth profile has aesthetic appeal for exposed members and the resistance to fluid flow forces (wind or water) is minimized.
- The profile provides the minimum surface area which minimizes costs for painting and other surface maintenance requirements. The minimum surface is also an advantage for structural members in clean production facilities.

Donald R. Sherman is professor, University of Wisconsin-Milwaukee.

This paper will be restricted to consideration of rectangular tubes (including square tubes) as used in building construction. These tubular products are frequently referred to as HSS sections (Hollow Structural Shapes.) The paper will begin with a discussion of characteristics of HSS that influence structural behavior. This will be followed by a presentation of some design consideration that differentiate the design of HSS structural members from more familiar open sections. The paper will conclude with a presentation of the research that forms the basis for recommendations on the economical design of simple shear connections between wide-flange beams and HSS columns.

HSS PRODUCTS

There are two primary ASTM specifications that refer to HSS sections.

A500: Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes
A501: Hot-Formed Welded and Seamless Carbon Steel Structural Tubing
A618 and A847 are for alloyed hot- and cold-formed tubes that must be obtained by special order from a manufacturer.

From the primary specifications it appears that four types of shaped HSS products are available. However, in the U.S. there is only one type that can realistically be obtained; cold-formed welded. The typical HSS product is A500 Grade B with a yield strength of 46 ksi and an ultimate strength of 58 ksi, although much of it would qualify as Grade C with 50 ksi yield and 62 ksi ultimate. Grade C can be certified by special order from a manufacturer.

In addition to the magnitude of the yield and ultimate strengths, the method of manufacture also influences other characteristics that affect structural behavior.

- Cold-formed A500 HSS have through-thickness residual stresses that are on the order of 80 percent of the yield strength of the material on the inside of the section. The variation of the mean residual stress around the perimeter is not as large, with compression of about 10 percent of the yield stress in the corners. A higher tension residual stress exists in a localized area at the weld.

- The straightness of HSS sections depends on the manufacturer, but in most cases members are well within the tolerance permitted by A500. Common out-of-straightness measurements are less than $L/5000$, which is much better than hot-formed open sections.
- Due to cold-working, there is a variation in the yield strength around the perimeter of the section, with a higher yield in the corners. The specified yield is from the center of one of the walls that does not contain a weld. Consequently, squash loads for stub columns can exceed the yield time the area.
- Thicknesses are very uniform in the sides of the HSS but somewhat greater in the corners.

The topic of thickness merits additional comments. The A500 specification permits the wall thickness to be 10 percent under the nominal value. Plate and strip from which HSS are made are produced to a much smaller thickness tolerance. For several marketing reasons, manufacturers in the U.S. take advantage of this situation and consistently produce HSS near the lower end of the A500 tolerance. Consequently the Steel Tube Institute of North America and AISC have issued a statement concerning the design thickness.

> "...a suggested modified wall thickness representing .93 of the nominal wall dimension should be used for calculations involving engineering design properties."

Tables of section properties and load tables for structural members that reflect this policy are being prepared.

MEMBER DESIGN CRITERIA

It is not the intent of this paper to review all the member design provisions for HSS sections. However, there are a few items of concern or differences with more familiar procedures for hot-formed open profiles that will be discussed. The criteria are from the current LRFD Specification[2] issued by AISC.

Axial Compression

There have been a few HSS column testing programs in North America, but most data is from an extensive series of column tests conducted by CIDECT (Comite International pour le Developpement et l'Etude de la Construction Tubulaire) in the 1970s.[8] A distinct difference in the normalized column strengths between hot-formed and cold-formed HSS was observed in the CIDECT programs, causing cold-formed tubes to be assigned to lower column curves in specifications with multiple curves. The high levels of residual stresses is a major factor for the lower normalized strength. In the U.S. where a single column curve is used in the LRFD Specification, much of the cold-formed data falls below the curve, indicating somewhat unconservative design. However, this situation is not as severe as accepted practice with heavily welded open shapes, where normalized test data is even lower than that for A500 HSS.

The apparent unconservative design of cold-formed HSS columns is not as critical as it appears. Much of the CIDECT test data was normalized by the offset yield of the section obtained from stub column tests. This reflects the inherent high yield stress in the corners of the tube resulting from cold working. Since U.S. practice is to determine the yield strength with a coupon taken from the middle of a side of the finished tube, the yield load calculated by the material yield strength times the gross area will be less than the weighted average that includes higher strengths in the corners.

Local buckling of HSS is an important consideration since about half of the standard HSS sizes have at least one pair of sides where the flat-width/thickness ratio exceeds $238/\sqrt{F_y}$ and the section is classified as thin-walled. Therefore, the LRFD column equation in Appendix B is the basis for many HSS designs.

$$P_{cr} = A_g(0.685^{Q\lambda_c^2})QF_y \text{ for } \lambda_c\sqrt{Q} \le 1.5$$

$$P_{cr} = \left[\frac{0.877}{\lambda_c^2}\right]F_y \text{ for } \lambda_c\sqrt{Q} > 1.5 \qquad (1)$$

$$\lambda_c = \frac{Kl}{r\pi}\sqrt{\frac{F_y}{E}}$$

The factor Q accounts for local buckling of HSS and is based on the effective width concept. This concept was theoretically proposed by von Karman and later empirically modified by Winter[4] to account for inelastic action and imperfections. The concept pertains to the force carried by a long plate supported on two edges parallel to an axial force. A uniform stress, which has the same magnitude as the true stress at the edge, acting on the effective width will result in the same post-buckling force using the true stress distribution. The effective width equation for the case when the side supports have the same thickness as the buckled plate is used by AISC for local buckling of a tube wall.

$$b_e/t = 1.91\sqrt{E/F}[1 - 0.381\sqrt{E/f}(b/t)] \le b/t \qquad (2)$$

In this equation, b is the flat width of the side of the tube and f is the average stress based on the total gross area, usually the critical stress for the column. The reduction factor Q is the ratio of the remaining effective area divided by the gross area and Equation 1 is used to determine the column buckling load, which reflects local buckling interaction. Since AISC bases f on the full section properties of the section rather than the effective properties, iteration to determine the critical load is avoided. If the average column stress is sufficiently low so that the effective width is the full flat width, Q is equal to one.

Bending

Thin walled HSS in bending are designed with the effective

Table 1. HSS Test Specimen Properties					
Size	b/t	KL/r	F_y ksi	P_y kips	vP_{stub} kips
5×2×1⁄8-in.	36	83.5	46.1	71.0	77.7
5×2×3⁄16-in.	23	86.4	57.0	127.2	161

width concept of Equation 2 for the compression flange. In this case the stress, f, is taken as the yield stress since failure occurs when the yield stress is reached in the corners. Using just the effective width for the compression flange causes a shift of the neutral axis away from the flange, as well as a change in the moment of inertia and the section modulus. The limit moment is determined by setting the bending stress calculated with the effective section modulus equal to the yield stress, or

$$\phi M_n = \phi S_{eff} F_y \qquad (3)$$

Square HSS are not subject to lateral-torsional buckling and, therefore, do not require lateral bracing. Rectangular HSS bending about the major axis could buckle laterally and AISC currently has provisions for the unbraced length. However, for HSS sections, the unbraced lengths are so large that realistic designs would be controlled by deflection or the reduction of the section moment capacity caused by lateral-torsional buckling is negligible. For example an HSS20×4×5⁄16-in., which has one of the largest depth/width ratios of standard HSS, has L_p of 8.7 feet and L_r of 137 feet. An extreme deflection limit might correspond to a length/depth ratio of 24, or a length of 40 feet for this section. Using the linear reduction between the plastic moment and the yield moment for lateral-torsional buckling, the plastic moment is reduced by only 7 percent for the 40-ft. length. In most practical designs where the moment gradient C_b is also a factor, the reduction will be nonexistent or insignificant. The only case where lateral bracing is an important consideration is when a plastic analysis is used for the moment distribution in the structure and some hinges must sustain finite plastic rotations to develop the failure mechanism. The maximum unbraced length from the hinge is

$$L_{pd} = \frac{0.17 + 0.10(M_1 / M_2)}{F_y / E} r_y \geq 0.10 \frac{E}{F_y} r_y \qquad (4)$$

In Equation 4, M_2 is the plastic moment of the section, M_1 is the smaller moment at the end of the unbraced length, and r_y is the radius of gyration about the minor axis.

Cyclic Axial Loading

HSS braces have been known to fracture catastrophically in earthquakes. A pilot program consisting of nine tests of members subject to axial end displacement reversals was conducted to investigate the failure mode. The program consisted of testing two thicknesses of 5 in.×2 in. HSS under axial displacement with ends pinned for column buckling about the weak axis. The properties of the test specimens are summarized in Table 1.

The size, b/t and column slenderness (KL/r) are based on nominal dimensions. The yield stress (F_y) and the measured stub column strength (P_{stub}) were obtained in static tests while the yield load (P_y) is calculated from the static yield stress and the actual HSS dimensions. The fact that the stub column tests are higher than the yield load reflects enhanced yield properties in the corners of the HSS and indicates that local buckling occurred in the strain hardening range.

The AISC Specification defines a thin-walled HSS under uniform compression as having a b/t that exceeds $238/\sqrt{F_y}$, or in this case 35 for the thin HSS. The recent AISC Seismic Provisions[1] limit b/t to $110/\sqrt{F_y}$ or about 15 for both of the two sizes. Therefore, the thicker of the test specimens would have been acceptable under the older code provisions, but neither HSS would be acceptable under the newer seismic provisions.

Both tube sizes were initially tested as columns under very slow monotonic axial loading. The resulting load vs. axial displacement curves are shown in Figure 1. Since the column slenderness is almost identical for the two sizes, overall column buckling occurs at essentially the same axial displacement. Subsequent local buckles, however, develop at less displacement in the thinner HSS. In the cyclic test program, axial displacement limits were at 0.200 in. where only the thin HSS formed a local buckle and at 0.400 in. where both HSS had local buckling.

The variables in the cyclic test program were the axial displacement range, the mean axial displacement and the rate of loading as determined by the period for a cycle. A similar

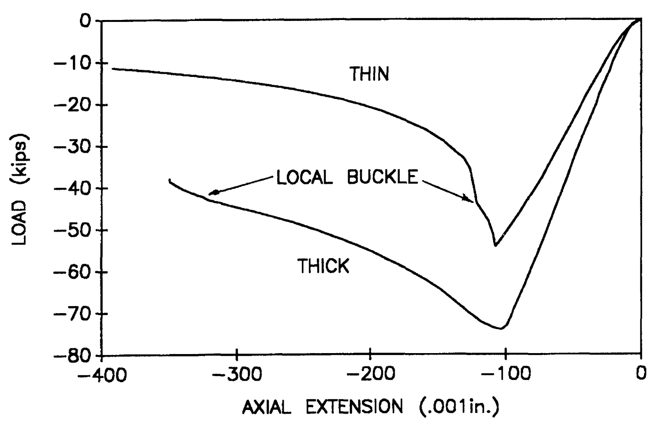

Fig. 1. Axial load-displacement.

Table 2. Cycles to Fracture						
Displacement (in.)	Thick			Thin		
	Test	Period (sec)	Cycles	Test	Period (sec)	Cycles
–.200, +.200				7	480	**32**
	4	16	**500+**	8	16	**32**
–.300, +.300				10	2	**27**
–.400, +.200	2	40	**31**			
	5	5	**34**			
	4a	5	**41**	Preload		
	3	2	**40**	9	2	**18**

pattern of behavior was observed in most of the cyclic tests. Column buckling is followed by a local buckle which leaves "horns" at the corners. After several cycles with tension excursions, cracks initiate at the HSS corners on both horns and propagate through the thickness and away from the corners in subsequent cycles. As section is lost at the cracks resulting in an eccentric load, the lateral deflection reverses during the tension part of the cycle but return to the original direction during compression, producing a snap-through behavior. Eventually the crack pops across the local buckle, resulting in increased lateral deflection that creates a large enough eccentricity to reverse the direction of column buckling in the subsequent compression. Table 2 presents the displacement range, the test identification number, the cycle period and the number of cycles for a full fracture across the width of the section.

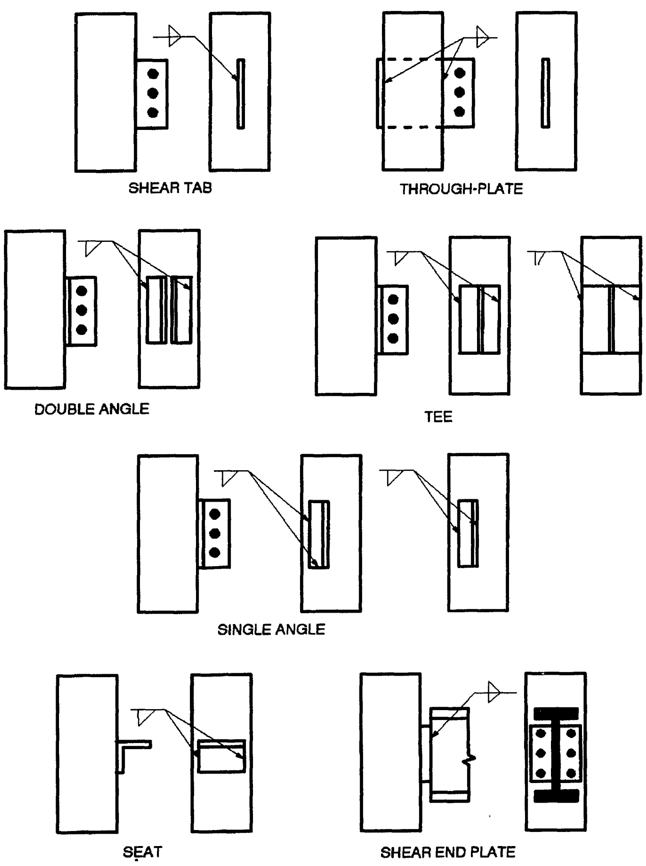

Fig. 2. Types of connections.

The most significant conclusion from the tests is that Test #4, which buckled as a column but did not form a local buckle, sustained over 500 cycles of loading without developing a crack. All other tests where local buckling did occur failed in 41 or fewer cycles.

These pilot tests demonstrate that the only important parameter in determining whether HSS braces will survive a seismic event is the formation of local buckles. In summary, the *b/t* limits for various limit states appear in Table 3.

SIMPLE FRAMING CONNECTIONS

Connections have been a concern for some designers who consider the use of structural tubing. Research has shown that a variety of familiar simple framing connections can be used to connect wide-flange beams to HSS columns.[9] Since the cost of different simple connections with the same capacity can vary by more than a factor of two, it is important to understand when inexpensive connections such as shear tabs can be used without compromising the strength of the tubular column.

This discussion concerns nine different types of simple framing connections used with HSS columns. These are listed below and shown in Figure 2.

shear tabs
through-plates
double angles

Table 3. Flat-Width/Thickness Limits	
Full yield in axial compression	$238/\sqrt{F_y}$
Plastic bending moment	$190/\sqrt{F_y}$
Yield bending moment with no local buckling	$238/\sqrt{F_y}$
Axial compression for brace in seismic zone	$110/\sqrt{F_y}$

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

In all but the shear 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 beam flange bears on the outstanding leg. For the shear end plate, the plate is welded to the beam web and bolted to the HSS column using blind expansion bolts[5] or a flow-drill process[9] that produces a tapped hole which replaces a nut in blind connections.

There are two categories of weld positions on the HSS for the connections shown in Figure 2. The shear tab, through-plate and single angle with vertical fillet welds have welds at the center of the HSS face, while the others have welds near the edges. Center welds will tend to distort the wall of the HSS more than edge welds, except for the through-plate which provides stiffening of the wall.

The connections are classified as simple, meaning that they produce negligible end moment in the beam. Rotational flexibility is provided by distortion of the connecting elements, particularly the column legs of angles or flanges of tees. Most of the connections are standard shear connections described for use with wide-flange columns in the AISC *Manual of Steel Construction.*[3] Two exceptions are the through-plate, which is unique to hollow members, and the single angle with vertical fillet welds. When a single angle is welded to the flange of a wide-flange column, a vertical weld at the heel would be in line with the web and rotational flexibility would be lost. Therefore, the standard welding pattern is an L-shaped weld with a vertical segment at the toe and a horizontal segment across the bottom. This permits distortion of the column leg of the angle so that the connection can be classified as simple. With an HSS column, however, flexibility is provided by the HSS wall in a manner similar to the shear tab. Therefore, a single angle connection with two vertical welds is considered.

The shear tab is a special connection, even with wide flange columns, due to restricted rotational flexibility. Distortion must come from local yielding of the tab combined with slippage and bearing distortion of the bolts in their holes. Additional flexibility is provided when the tab is used with an HSS column, but some designers fear excessive distortion of the HSS wall. Hence through-plates are sometimes specified to reinforce the wall.

Table 4. Relative Connection Costs	
Type	**Cost**
Single Angle, L-shaped Welds	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 Bevel Welds	2.42

Relative Connection Costs

In order to put the discussion in a good perspective, information on the relative costs of the connections is desirable. Since a number of connection types were being studied and tested at the same time, an excellent opportunity was presented to determine relative costs. Relative costs for 3 bolt connections are listed in Table 4 based on the least expensive (single angle with L shaped fillet 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 blind bolting or flow-drilling the holes are not routine operations at this time. The costs do not reflect shop preparation of the beam or field erection.

The high cost of the Tee with the flare bevel weld is due to labor and consumable electrodes required for the multipass welding. Vertical fillet welds on the Tee are much more economical. For a simple shear connection, there is no behavioral advantage for the flare bevel welds. In a moment connection where horizontal tees are used between beam flanges and the column, flare bevel welds provide a good transfer of the tension and compression forces into the side walls of the HSS and, therefore, may be warranted.

It may also be noted in Table 4 that the through-plate connection is more than twice as expensive as the shear tab. This is due to the labor involved in laying out and slotting the HSS to insert the plate. In addition, there are interference problems if connections for perpendicular beams are re-

Table 5.
Limit States for the Connections

Connection Type	A	B	C	D&E	F	G	H	I
BOLTS								
Shear with no eccentricity			X		X	X		X
Shear by ultimate analysis	X	X		X				
CONNECTOR MATERIAL								
Bolt bearing, $L_{ev} \geq 1.5d$	X	X	X	X	X	X		X
Gross shear at yield	X	X	X	X	X	X	X	X
Net section shear fracture	X	X	X	X	X	X		
Flexural yield				X			X	
Flexural rupture				X				
Block shear	X	X	X	X	X	X		
WELDS								
Shear with no eccentricity				X				X
Shear by vector analysis			X				X	
Shear by ultimate analysis	X	X			X	X		
TUBE WALL								
Shear at weld	X	X	X	X	X	X	X	
Bolt bearing								X
Punching Shear	X					X		

A—shear tabs
B—through-plates
C—double angles
D—tee with vertical fillet welds
E—tee with flare bevel welds
F—single angle welded at toe and bottom
G—single angle welded at toe and heel
H—unstiffened seat
I—shear end plate

quired. Consequently, considerable research has been conducted to justify the use of economical shear tabs.

Connection Limit States

Connection limit states were studied in a series of test programs involving 24 tests of simple connection to HSS columns.[9] The connection strength is governed by limit states associated with the bolts to the beam web, connector material, welds and the HSS. Possible limit states are listed in Table 5 with an indication of which apply for various types of connection according the AISC Manual.[3] After applying the appropriate resistance factor, the lowest value governs the strength of the connection, or the criteria can be used to establish a size limit so that a particular limit state will not control. The eccentricities are the result of the small distance between the bolts and welds and do not imply that a significant end moment exists in the beam. Since the criteria for various connections were developed from different research programs that may have been separated by several years or decades, there are inconsistencies in the present state-of-the-art. For example, weld eccentricities are evaluated by elastic vector analysis in some cases and by an inelastic ultimate analysis in others.

Connection design for HSS columns is somewhat simplified since it is unlikely that beams would be coped at the top flange. Therefore, the bolt edge distance limits in the connecting material can be met and no bearing reductions are required for less than minimum edge distance.

Table 5 indicates three limit states associated with the HSS column. Bolt bearing applies only for the shear end plate which requires bolting to the HSS. When the connector is welded to the HSS, shear in the wall adjacent to the weld may control the capacity of the weldment. One way to consider this is to determine the maximum throat dimension of the weld for which the weld material will govern.

$$(throat)_{\max} = \frac{\phi 0.6F_{u(HSS)}}{\phi_w 0.6F_{u(WELD)}} t_{HSS} \tag{5}$$

where

F_u = the ultimate strength of the material

For fillet welds where the throat is 0.707 of the weld size and the two resistance factors are the same according the AISC Specification,[2] the maximum effective weld size is

$$a_{eff} = \frac{\sqrt{2}F_{u(HSS)}}{F_{u(WELD)}} t_{HSS} \tag{6}$$

When the actual weld size is less than a_{eff}, the weld dictates the capacity while for larger welds, the effective weld size controls.

The other limit state associated with the HSS in Table 5 is punching shear. This is a tearing through the thickness of the HSS wall adjacent to the weld. This can occur in shear tab and single angle connections with vertical welds where tension in the material resulting from eccentricity pulls directly at the upper part of the weld. It can be prevented by a simple criteria that keeps the maximum pull as determined by the yield strength in a unit length of the connector material being less than the shear fracture capacity through the two sections of the HSS wall on either side of the weld or pair of welds.

$$F_{y(tab)}\, t_{tab} < 2(0.6F_{u(HSS)})t_{HSS} \tag{7}$$

or

$$t_{tab} < 1.2\frac{F_{u(HSS)}}{F_{y\,(tab)}} t_{HSS} \tag{8}$$

Punching shear will not occur in through-plate connections where the HSS wall is reinforced or in other connections where the pull is transferred to a perpendicular element of the connector, such as the column leg of an angle or flange of a Tee.

One limit state for the HSS that is not shown in Table 5 is that associated with a yield line mechanism. In all the tests that were conducted with the beam simply supported at both ends, there was never enough distortion of the face of the HSS to develop a yield line mechanism. Therefore, the limit states associated with the HSS can be prevented from controlling by determining a maximum effective weld size and by limiting the thickness of the projecting connection material when it is directly welded to the HSS wall.

The experimental strengths reported in Ref. 9 generally match or exceed the strengths predicted by the limit states criteria. Distortion due to gross yielding was usually observed at loads less than the corresponding limit state, but this did not represent a loss of load capacity in the connection. Actual failure modes do not always match the theoretical critical limit state. However, the designs were well balanced so that several limit states have nearly the same capacity, making it uncertain to clearly discern the failure mode in the tests. The conclusion is that the AISC tables for connection strength[3] can be conservatively used for HSS columns provided that the weld does not exceed the effective weld size determined from the HSS thickness and that the punching shear criteria is applied for shear tabs.

The economically attractive shear tab connection was tested to a greater extent than the others. It was determined[7] that the shear eccentricities were generally between the weld and bolt line and less than those used in the AISC tables,[3] except for combinations of HSS with very low width/thickness ratios and flexible beams. However, in the latter cases the experimental eccentricities reasonably matched those used in the AISC Manual. Since a smaller eccentricity leads to greater capacity in the bolts and welds, it is conservative to use the AISC Tables for shear tabs.

HSS Wall Distortion and Column Strength

In order to determine the effect of the connection types on local distortion of the HSS columns in the 24 connection tests, strain gages were mounted at the center of the wall one inch below the connecting element. The transverse strains measured or extrapolated at a common 50 kips shear that are shown in Table 6 form the basis for comparison. Positive transverse strains in Table 6 result from Poisson's ratio and indicate no wall distortion.

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 even at service loads. An exception to this is the through-plate that inherently reinforces the center of the wall and the transverse strains are negligible. 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.

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

Table 6. Transverse Strain in HSS at 50 kips Shear (μin/in)			
Type	**3 Bolt**		**5 Bolt**
Tab	–2100 to –3900		–900 & –1200
Single Angle, Vert. Welds	–2100		–1050
Single Angle, L Weld	–1380		–450
Tee, Vert. Welds	–750 to –1100		–380
Tee, Flare Welds	–1100		20
Double Angle	–975		–600
End Plate		–300	
Seat		40 & 60	
Through-Plate	55 to 700		

Table 7. Column Strengths for Tabs vs. Through-Plate Tests			
		P_{ult} / P_y	
b/t	**Connection**	**Two Sides**	**One Side**
15	Through-Plate, Tight	0.53	
	Shear Tab, Tight	0.51	
	Through-Plate, Snug	0.50	
	Shear Tab, Snug	0.49	
29	Through-Plate	0.63	0.42
	Shear Tab	0.61	0.46
40	Through-Plate	0.58	0.42
	Shear Tab	0.45	0.42

through-plate connections. However, this conclusion was based on only four tests using HSS with a *b/t* ratio of 16. More recently similar column tests were conducted using HSS with *b/t* ratios of 29 and 40.[9] 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 3. In these tests, the beams were loaded to about 70 percent of the connection capacity and then a load was applied to the top

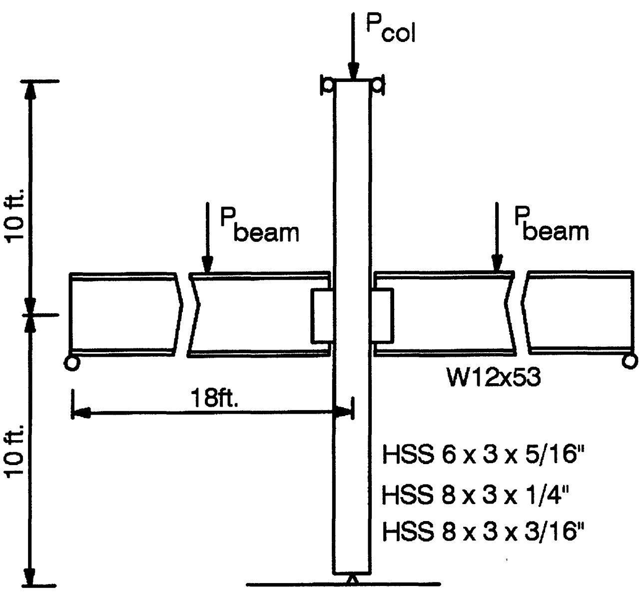

Fig. 3. Column test setup.

of the column until a column buckling failure occurred in the lower portion.

Table 7 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 tests with connection on two sides failed with sudden buckles while the unsymmetric tests failed gradually in bending.

The conclusion from Table 7 is that shear tab connections used with HSS columns 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 on 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 conclusions hold for other types of simple connections that have smaller transverse strains.

SUMMARY AND CONCLUSIONS

There are a few characteristics of square and rectangular HSS that cause some member design consideration to differ from those of open profile sections. First it must be recognized that only cold-formed welded HSS are readily available in the U.S. These sections have good structural properties, although the thicknesses will usually be less than the nominal value. It should be recognized that many of the sections are thin-walled and require appropriate design criteria for columns and beams that reflect local buckling. Design criteria must also prevent local buckling when the HSS are used as braces in seismic applications. Except in an unusual situation or when plastic analysis is used, HSS beams do not require lateral bracing.

The connection 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[3] can be applied when HSS columns are used. The only additional limit states that must be considered are a simple thickness criteria for punching shear of the HSS wall when shear tab connections are used and a limit on maximum effective weld size based on the HSS thickness.

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 specified in a design, since the connection cost can vary by a factor of 2½.

ACKNOWLEDGEMENTS

The connection and column tests programs were supported by the Steel Tube Institute of North America and additional funds for the shear tab investigations were provided by the Society of Iron & Steel Fabricators of Wisconsin and AISC. The HSS material was provided by the Welded Tube Company of America. Special thanks is due to Dave Mathews of Ace Iron & Steel Company of Milwaukee who fabricated the connection material and provided the cost estimates for fabrication. The work was conducted over several years by four graduate students; Steve Herlasche, Joe Ales, Chris Haslam and Homyan Boloorchi.

REFERENCES

1. American Institute of Steel Construction, *Seismic Provisions for Structural Steel Buildings*, Chicago IL, 1992.
2. American Institute of Steel Construction, *Load and Resistance Design Specification for Structural Steel Buildings*, Chicago IL, 1993.
3. American Institute of Steel Construction, *Manual of Steel Construction, Load and Resistance Factor Design*, 2nd Ed., Vol. 2, Chicago IL, 1994.
4. American Iron and Steel Institute, *Commentary on the Specification for the Design of Cold-Formed Steel Members*, Washington, D.C., 1968.
5. Korol, R. M., Ghobarah, A., and Mourad, S, "Blind Bolting W-Shape Beams to HSS Columns," ASCE *J. of Structural Engineering*, 119(12), 1993, pp. 3463–3481.
6. Herlasche, S. M., and Sherman, D. R., "Beam Connections to Rectangular Tubular Columns." AISC *Proceedings of the National Steel Construction Conference*, 1989, pp. 1-1 to 1-22.
7. Sherman, D. R., and Ales, J. M., "The Design of Shear Tabs With Tubular Columns." AISC *Proc. National Steel Construction Conference*, 1991, pp. 23-1 to 23-14.
8. Sherman, D. R., "Tubular Members," *Constructional Steel Design-An International Guide* Eds. P. J. Dowling, J. H. Harding and R. Bjorhovde, London: Elsevier Applied Science, 1992, Chap. 2.4, pp. 91–104.
9. Sherman, D. R., "Simple Framing Connections to HSS Columns," AISC *Proc. National Steel Construction Conference*, 1995, pp. 30-1 to 30-16.