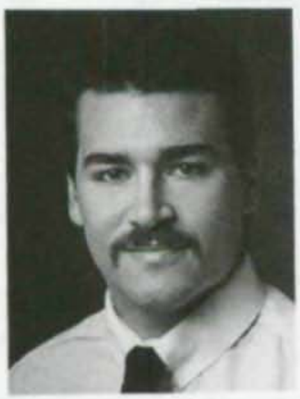
The Design of Shear Tabs With Tubular Columns

Dr. Donald R. Sherman



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The Master's thesis submitted by Mr. Ales was entitled, "The Design of Shear Tabs Welded to Tubular Columns". The research was partially sponsored by the American Institute of Steel Construction and the Society of Iron & Steel Fabricators of Wisconsin. Research involved the testing and analytical evaluation of this type of connection. On the basis of the research, a design procedure was developed.

Summary

This paper presents the results of a study to develop design rules for single plate framing connections (shear tabs) used to connect wide flange beams to tubular columns. The study includes both finite element evaluations and physical tests. The primary parameters in the study are the b/t of the tube wall, shear tab size and beam stiffness. The tests also contained a mix of snug tight and fully tightened bolts. The beams used in the tests were realistically long with a simple support at the far end. They were loaded with a single concentrated load, positioned to produce the same end shear and elastic end rotation as a uniformly loaded beam. These beams were an important factor in restraining the rotation at the shear tab.

Failure modes that were identified were, yielding of the shear tab, bearing and shear failures of the bolts, weld failures and punching shear failures of the tube wall. Yield line distortions occurred in the tube walls, but these were restrained by the beam and did not limit the shear capacity of the connection. End moments were small enough so that the beams could be considered as simply supported. However, local moments in the connection are important in the design of the connecting elements. Except for the case of very stiff tube walls, the eccentricity (point of zero moment) was found to be in the shear tab between the bolts and welds. The paper contains suggestions for design rules for this type of connection.

THE DESIGN OF SHEAR TABS WITH TUBULAR COLUMNS

INTRODUCTION

One of the simplest and most economical simple beam connections is the single plate framing connection, or shear tab. Presently, the shear tab is most extensively used with wide-flange columns or other supporting structures which provide essentially rigid faces at the weld line. Research and testing of shear tabs with wide-flange columns has been conducted by Richard [1] and Astaneh [2]. Although the design procedures developed in both investigations can be used safely, the method outlined by Astaneh has been incorporated into the Ninth Edition of the Manual of Steel Construction [3].

When the shear tab is welded to an HSS (Hollow Structural Section) column, the flexibility of the tube face precludes a rigid face assumption. Astaneh's procedure makes an allowance in the analysis for a "rotationally flexible element", which in the case of a tubular column would be the tube face. This term is rather general, however, and does not take into account possible limit states involving this "rotationally flexible element". This investigation examined the behavior and evaluated the shear capacity of shear tabs welded to HSS columns and developed a method of analysis and design for the connection.

BACKGROUND

Load Transfer to Shear Tabs

The shear tab is considered a simple connection, and as such, is normally proportioned for shear only. General provisions regarding simple connections are given in the AISC-ASD Specifications [3]:

> "Except as otherwise indicated in the design documents, connections of beams, girders or trusses shall be designed as flexible and ordinarily may be proportioned for the reaction shear only. Flexible beam connections shall accommodate end rotations of unrestrained (simple) beams. To accomplish this, inelastic deformation in the connection is permitted."

Sufficiently large moments, however, may develop in the connection which can affect the design of the fastening elements, i.e. bolts and welds. The amount of moment developed in a simple connection can be approximated as M=R*e, where "R" is the reaction shear and "e" is the reaction eccentricity. The reaction eccentricity is defined as the distance from some reference (weld line or bolt line) to the point of zero moment. The reaction eccentricity depends on a number of factors, such as the number of bolts, the dimensions and material of the shear tab, the amount of end rotation in the beam and the relative rigidity or flexibility of the supporting structure, or column. When the shear tab is welded to a rigid support, the rotational restraint in the connection is released through shear deformation of the bolts, distortion in the bolt holes, out-of-plane bending of the plate and bolt slippage. When the shear tab is welded to a flexible support, such as a tube wall, the flexibility provides an additional path through which the rotational restraint can be released.

Bolt Installation

The tightness of the bolts used in shear tab connections may be divided into two categories, snug tight and fully torqued. Snug tight is defined as the tightness that exists when all plies in a joint are in firm contact. This conditioned is attained by tightening the bolts with an ordinary spud wrench and stopping when further bolt turning can only be accomplished by extra effort. Fully torqued bolts are bolts which are tightened until the tension in the bolts reaches 70% of the specified minimum tensile strength of the bolt. Fully torqued bolts rely on friction to carrying the working load shear and have been traditionally used in building construction until recently. Snug tight bolts rely on bearing forces to carry the shear, but require less time and cost to install than fully torqued bolts. Therefore, for non-slip critical connections, snug tight bolts are an attractive alternative.

The Design of Shear Tabs with Wide-Flange Columns

Research on the shear tab has been done by Richard [1] and Astaneh [2]. Each investigation studied shear tabs welded to wide-flange columns and produced a design procedure for this type of connection. The former study concentrated on reaction eccentricity and involved finite element modeling, cantilever testing to obtain moment-rotation characteristics and full scale testing. The study found that connection eccentricity increased with the number of bolts, the size of the bolts and the thickness of the plate. These tend to increase the rotational stiffness of the plate. An increase in the span-to-depth ratio of the beam also increased the connection eccentricity.

The research in Astaneh's study concentrated on experimental determination of the ultimate strength of the connection. Five different connections were tested. The failure modes identified from the testing were:

1.) Shear failure of bolts
2.) Yielding of gross area of plate
3.) Fracture of net area of plate
4.) Fracture of welds
5.) Bearing failure of beam web or plate

Each component in the connection, the bolts, welds and plate, must be designed to resist either shear or a combination of shear and moment. The moment developed in the connection depends on the reaction eccentricity. Astaneh found that the eccentricity was proportional to the number of bolts in the connection and developed the following equations for determining eccentricity:

For the bolts: $e_b = (n - 1) - 3$ **(1)**

For the welds: $e_w = n$ **(2)**

The eccentricity is measured in inches, from the bolt line in Equation 1 and from the weld line in Equation 2; "n" is the number of bolts and the "3" is the distance from the bolt line to the weld line. The design procedure developed by Astaneh is simplified and adopted for use in the ASD Manual (Table X); it includes the following modifications to the eccentricity equations:

For the bolts: $e_b = 3$ **(3)**

For the welds: $e_w = MAX (3 \ OR \ n)$ **(4)**

<u>The Evaluation of Tube Wall Strength</u>

The flexibility of the tube wall introduces an additional limit state into the design of shear tabs welded to tubular columns. The tube wall in this type of connection may experience either a bending failure or punching shear failure.

<u>Bending Failure of the Tube Wall</u>

The limit state of tube walls with members branching into them is determined using yield line theory [4] and plastic analysis. For particular configurations of tube walls and branching members, a pattern of yield lines is developed to form a yield mechanism. Bending moments in the tube wall are redistributed along these lines and become the locations for the formation of plastic hinges. When enough hinges form, the member changes into a mechanism and the member collapses or fails. Figure 1 illustrates this concept. This type of failure is dependent on the ability of the tube wall to produce plastic moments along the yield lines. Because the depth of the shear tab is much larger than its thickness ($d_{pl} \gg t$), high strains are likely to develop at the edge of the plate (see Figure 1), producing a localized failure, such as the plate pulling out from or punching into the tube wall; this may occur before a sufficient number of plastic hinges develop to induce a yield line failure.

Punching Shear Failure

In addition to a bending failure, a tube wall may also experience a punching shear failure. Failure is defined as the point at which the applied load exceeds the shear resistance of the tube wall around the perimeter of the branching member; Figure 2 illustrates this method of failure. When using this method of analysis, a failure load for the tube wall is not defined; rather, the thickness of the tube wall must exceed some fraction of the thickness of the branching member, in this case the shear tab, so that the shear tab yields before the tube wall fractures in shear. The equation for the tube wall thickness is written as:

$$t_{tw} \geq [\frac{F_{y_{(pl)}}}{1.2F_{u_{(tw)}}}]\ t_{pl} \qquad (5)$$

where t_{pl} is the thickness of the shear tab
$F_{y(pl)}$ is the yield stress of the shear tab
$F_{u(tw)}$ is the tensile strength of the tube wall

Objectives

The purpose of the research was to develop a design procedure for shear tabs welded to HSS columns. In order to fulfill this purpose, the behavior of the shear tab under static loading was studied as a function of the following parameters:

1.) The span-to-depth ratio (L/d) of the beam.
2.) The width-to-thickness ratio (b/t) of the tube wall.
3.) The tightness of the bolts.
4.) The size of the shear tab and the number of bolts.

The specific objectives of the testing and evaluation were:

1.) Evaluate the capacity of the shear tabs and identify the limit states.
2.) Evaluate the strength of the tube wall.
3.) Determine the effect of the design parameters on the reaction eccentricity.
4.) Determine what affect the bolt tightness has on the eccentricity and the ultimate capacity of the shear tab.

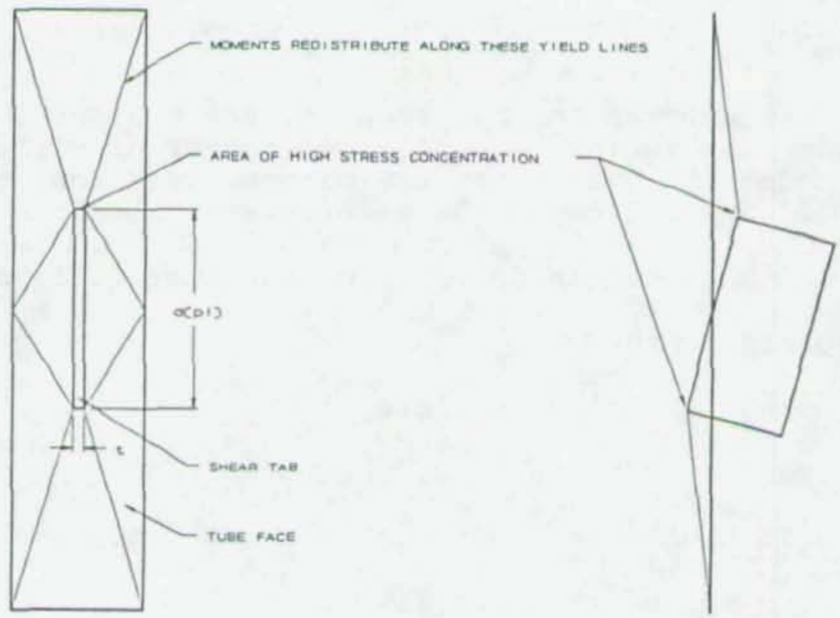


Figure 1 : Yield Lines for Bending

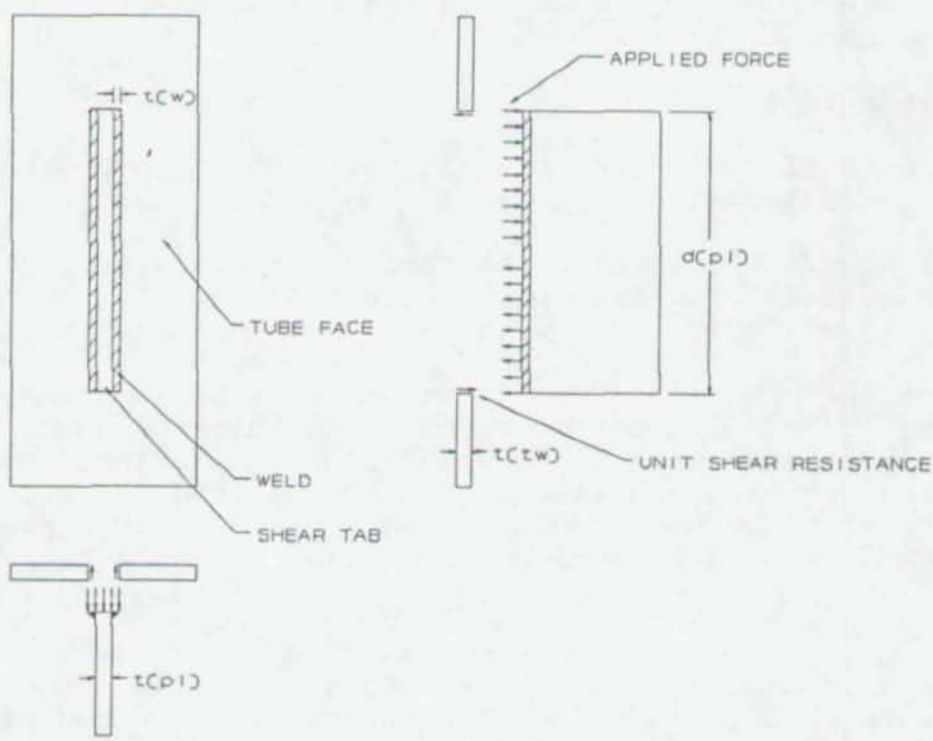


Figure 2 : Punching Shear Failure

TEST PROGRAM

Parameters

The test program for any research on structural steel connections must include a sufficient number of variables so that behavior under the widest variety of configurations can be understood. The following parameters were varied:

1.) width-to-thickness ratio of the tube wall (b/t)

2.) plate depth
plate thickness
number of holes in plate

3.) span-to-depth ratio of the beam (L/d)

4.) bolt tightness (snug tight or fully torqued)

The following parameters were fixed:

1.)	Tube material	: ASTM A500 Grade B
2.)	Shear Tab material	: ASTM A36
3.)	Beam material	: ASTM A36
4.)	Bolt type	: ASTM A325
5.)	Weld material	: E7018 electrodes

Further explanations of the parameters will be given in the following sections, which explain the selection process for each test component.

Shear Tab Selection

Three different sizes of tabs were chosen to be tested. They are as follows:

1.) 9 x 4 1/2 x 5/16 with three 3/4" diameter bolts
2.) 9 x 4 1/2 x 1/4 with three 3/4" diameter bolts
3.) 15 x 4 1/2 x 5/16 with five 3/4" diameter bolts

The three bolt and five bolt configurations are typical of shear tabs used in building construction. The thicknesses of the plates were chosen with regard to design limitations recommended by previous studies and with regard to the load capacity of the loading rams to be used in the tests. The geometric properties of the shear tab test specimens are given in Table 1.

HSS Columns

The HSS columns were selected to give a wide range of width-to-thickness ratios (b/t) for the tube wall connected with the tab. A survey of the standard shapes of structural tubing given in the AISC Manual of Steel Construction [3] showed the flat width-to-thickness ratio (b/t ratio) varied from a minimum of 2 to a maximum of 60. The b/t ratio was calculated as (w-3*t)/t, where "w" is the nominal width of the tube wall and "t" is the thickness of the tube. The extremes for the b/t ratios are impractical for columns used in building construction. The length of the columns used in the tests was 5 feet, with the connection at mid height. The geometric properties of the tubular column test specimens are given in Table 1.

Load Beams

In building construction, shear tabs usually carry the reactions from uniformly loaded beams. In testing situations, however, load application is practically limited to point loading. The purpose of the load beams in this research was to transfer a reaction and rotation to the shear tab connection. Therefore, the point loaded test beam was designed to transfer the same end reaction and rotation as a comparable uniformly loaded beam.

Two uniformly loaded beams with different L/d ratios were simulated so that connection behavior as a function of end rotation could be studied. The end rotation is a function of the L/d ratio, $\theta = (\frac{2}{3})(\frac{f}{E})(\frac{L}{d})$. One beam was designated the "high rotation" beam, which was used with the 3 bolt connections and had an L/d ratio of 23, which is typical of the span-to-depth ratio of beams used in building construction. The second beam was designated the "low rotation" beam and was used with both the 3 bolt connections and the 5 bolt connections. This beam had an L/d ratio of 9.75. The beams selected were a W12X87 and a W18X71. A summary of the properties of the tested and simulated beams is given in Table 2.

Test Schedule

The test schedule is shown graphically in Figure 3 and listed in Table 1. Each test is referred to by an alphanumeric code; the first number is a sequence identifier; the letter, "H" or "L", indicates the rotation group, "high rotation" or "low rotation"; the last number is the b/t ratio of the tube wall.

Test Setup

The basic test setup is shown in Figure 4. A variety of sensors were used to obtain data from the connection tests. These included load cells, strain gages, LVDTs (linear voltage displacement transducer), a slope indicator and white wash coating. The load cells were used to determine the jacking forces and the far end beam reaction. By subtracting the loading force from the far end reaction, the shear load on the connection was determined. The slope indicator was used to measure the beam rotation at the connection bolt line. It was placed on the beam directly over the bolt line. Two LVDTs were used to measure the displacement at the load point and at the shear tab. Both were placed directly on the beam.

Six gages were mounted on the beam between the connection and the point load, 3 on the top flange and 3 directly beneath on the bottom flange. To determine the reaction eccentricity, these strains were averaged, using the formula $\epsilon_{avg} = [\epsilon_b - (-\epsilon_t)]/2$, and a strain gradient was established. The distance from the point of zero strain to the tube wall was defined as the reaction eccentricity. This method of analysis was very sensitive to small variations in strain, so linear regression was used to partially compensate for the sensitivity and all eccentricity values were qualitatively checked by examining the bolt bearing patterns. The connection moments were determined at the bolt line and the weld line by multiplying the reaction shear by the reaction eccentricity.

Test Procedure

After the test components were connected and aligned, the sensors were connected to the data acquisition system and testing began. Zero readings were taken for all sensors and the loading began. The loading was done by a hand-controlled electrical hydraulic pump and proceeded at a rate of about 5 kips per minute. Sensor readings were taken every 5 kips and at other loads at which something out of the ordinary happened. The loading was stopped when either the beam reached its elastic limit or the connection continued to deform without an increase in load. The connection was then unloaded.

Finite Element Examination

As a supplement to the test program, a parameter study using finite element models was conducted to determine the location of the point of eccentricity as a function L/d of the beam, b/t of the tube wall and the depth of the shear tab. The finite element program used was ANSYS Revision 4.4 [5]. Symmetry was used to cut the model in half and the tube was unfolded to simplify model generation, with the tube wall continuous over a simple support at the location of the bend.

The model setup for the finite element analysis was similar to the experimental test setup. A point load of 25 kips was placed on the beam to give a reaction of about 20 kips on the connection. The eccentricity was found by calculating the moment caused by the bolt forces, about the weld line, and dividing by the reaction. The analysis was linearly elastic.

Table 1

Properties of Shear Tab Test Specimens

GROUP	TEST	SHEAR TAB DIMENSIONS	TUBE SIZE	b/t	BOLT TIGHTNESS
		in. x in.	in. x in. x in.		
HIGH	1H5	9 X 5/16	4 X 6 X 1/2	5	TORQUED
	2H5	9 X 5/16	4 X 6 X 1/2	5	SNUG
	3H10	9 X 5/16	4 X 6 X 5/16	10	SNUG
	4H16	9 X 5/16	6 X 3 X 5/16	16	TORQUED
	5H16	9 X 5/16	6 X 3 X 5/16	16	SNUG
	6H40	9 X 5/16	8 X 3 X 3/16	40	TORQUED
	7H40	9 X 5/16	8 X 3 X 3/16	40	SNUG
	8H45	9 X 1/4	12 X 8 X 1/4	45	TORQUED
	9H45	9 X 5/16	12 X 8 X 1/4	45	SNUG
LOW	10L5	15 X 5/16	4 X 6 X 1/2	5	TORQUED
	11L5	15 X 5/16	4 X 6 X 1/2	5	SNUG
	12L16	9 X 5/16	6 X 4 X 5/16	16	SNUG
	13L45	15 X 5/16	12 X 8 X 1/4	45	SNUG

All bolts were 3/4" diameter ASTM A325 high strength bolts. The 9" deep tabs had 3 bolts and the 15" deep tabs had 5 bolts; the width of all shear tabs is 4 1/2 inches. The "Group" designation indicates the relative end rotation of the test beam as explained in the section "Load Beams". The tube dimension listed first is the width of the tube wall to which the shear tab is welded; all welds were 1/4" except for test 8H45, which used a 3/16" weld.

Table 2

Properties of the Test Beams

		TEST BEAM		SIMULATED BEAM		
GROUP	BEAM	L	a	L	L/d	θ_{MAX}
HIGH ROTATION	W12X87	30'	5.25'	24'	23	.0193
LOW ROTATION	W18X71	20'	3.04'	15'	~10	.0080

"a" is the distance from the point load to the connection.
θ_{MAX} is the end rotation (radians) at 1.5 times the service load.

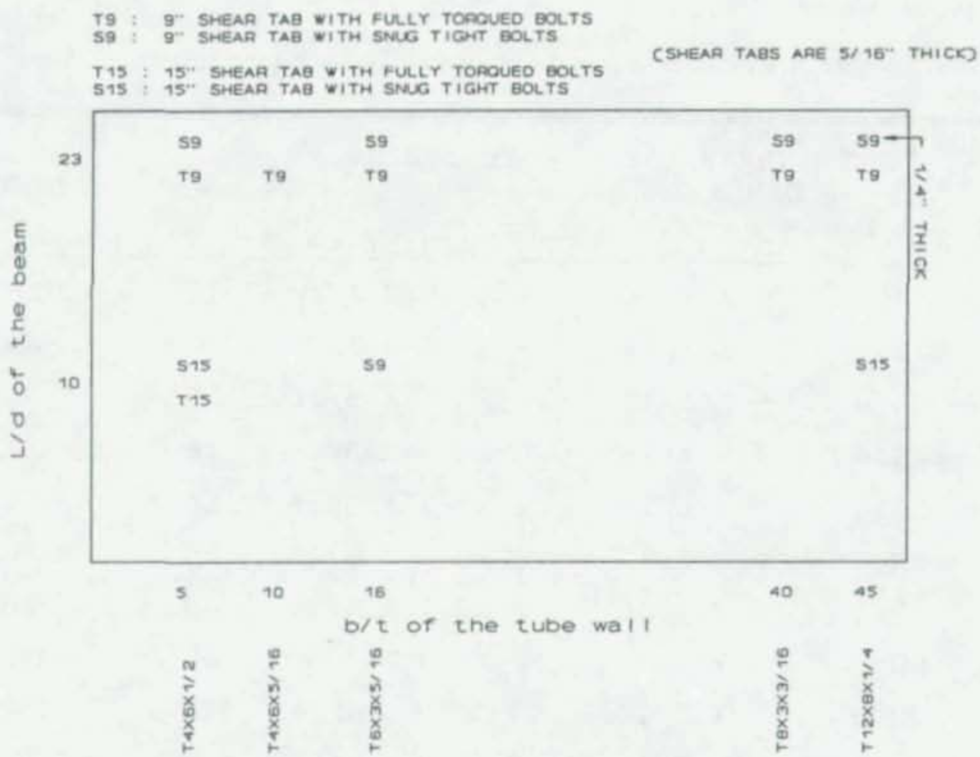


Figure 3 : Test Schedule

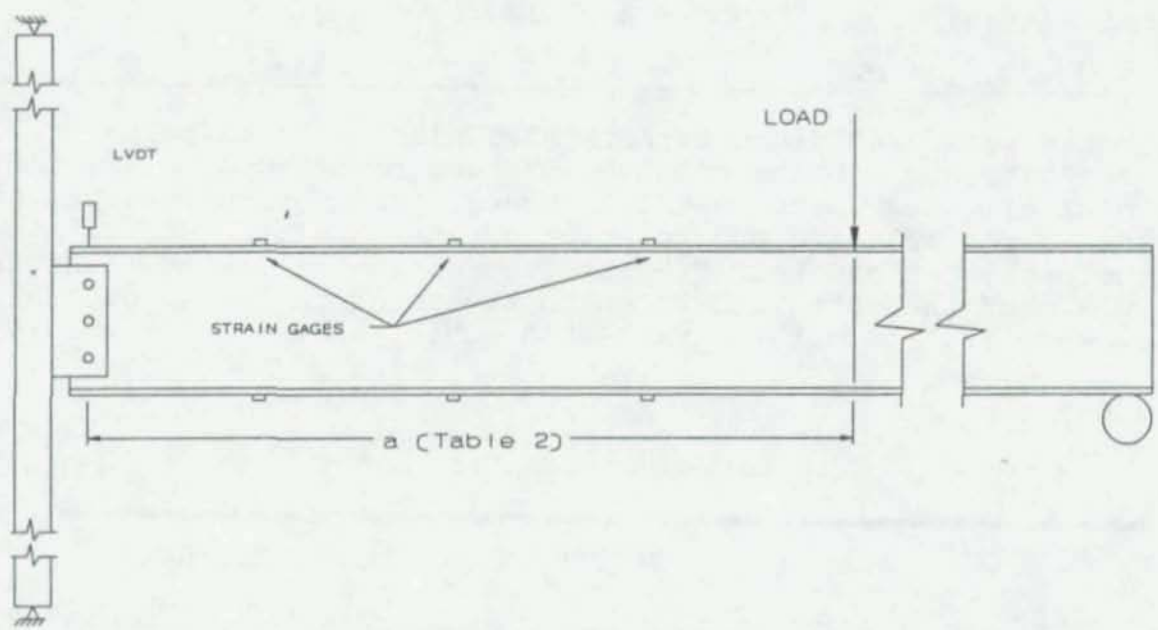


Figure 4 : Test Setup

MATERIAL PROPERTIES

The results from the tension tests of the shear tab coupons are given in Table 3. Two different plate materials in the 5/16 inch thickness were provided by the fabricator. The yield stresses for the tabs varied from 47.7 ksi to 55.5 ksi, well above the nominal yield stress for A36 steel of 36 ksi. The material with the 0.303" thickness had a rounded stress-strain curve, while the curves for the other two tab materials were sharp yielding.

The results from the tension tests of the tube material are also given in Table 3. The yield stresses ranged from 49.6 ksi to 59.5 ksi, above the nominal yield stress of 46 ksi for A500 Grade B tubing. The stress-strain curves for the tube material were rounded, similar in shape to those for the .303 inch shear tab material.

Table 3

Material Properties of the Shear Tabs

Tab Thickness		Yield Stress			
Nominal	Actual	Nominal	Actual	Tensile Strength	Elongation
in.	in.	ksi	ksi	ksi	%
5/16	0.303	36	55.5	69.5	31.3
5/16	0.326	36	47.7	73.0	28.5
1/4	0.239	36	49.3	72.6	27.5

Material Properties of the Tubes

Tube Thickness		Yield Stress			
Nominal	Actual	Nominal	Actual	Tensile Strength	Elongation
in.	in.	ksi	ksi	ksi	%
3/16	0.173	46	53.7	71.2	20.4
1/4	0.236	46	49.6	64.2	28.9
5/16	0.305	46	53.7	63.8	33.1
1/2	0.470	46	59.5	71.8	31.9

CONNECTION BEHAVIOR

Experimental Connection Results

The maximum shear resisted by each connection and the averaged reaction eccentricity is summarized in Table 4. The corresponding theoretical capacities for four failure modes are listed alongside. Failure in each connection was defined as the point at which the shear versus tab displacement curve began to flatten out. The load beams, which had to be reused for each test, were designed to remain elastic at the calculated shear yield of the tabs. The unexpectedly high yield stress of the shear tab material increased the shear capacity of the tabs 150%; at this load, the beams would become inelastic. Therefore, four tests were stopped prematurely to prevent the beam from yielding. The following failure modes were identified from the testing:

1). Yielding of the gross area of the tab
2). Bearing failure of the tab
3). Fracture and yielding of the welds
4). Punching shear failure of the tube wall
5). Surface tearing of tube wall material beneath weld
6). Lateral buckling of the tab
7). Shear yielding and fracture of the bolts

An eccentricity versus shear plot is shown in Figure 5 for three typical curves; the corresponding average eccentricities, given in Table 4, are also plotted. A plot of the eccentricity versus the b/t ratio for each test is shown in Figure 6.

Shear Capacity and Limit States

For those connections which failed, all experienced a shear yielding of the gross area of the tab. This was indicated by the flaking of the white wash and the obvious shear distortion in the tab. The shear tab, however, was able to sustain load and even experience an increase into the strain hardening range. Actual failure was precipitated by some other failure mechanism. In all cases, the maximum shear resisted by the tab was less than the nominal calculated gross area yield and the reduction in load carrying capacity increased as tube wall deflection increased. With the flexible tube walls ($b/t \geq 16$), the yielding seemed confined to the plate area between the top and bottom bolt holes, as indicated by white wash flaking. Therefore, less plate area was available for resistance. In contrast, the shear tabs welded to the stiff tube walls ($b/t \leq 10$) experienced yielding along the entire length of the plate. There was no noticeable distortion in the tube. Figure 7 shows the area of plate resisting the shear for flexible and stiff tube walls. It is suggested that the shear resistance in gross yield be reduced by 20% for those tube walls which can be regarded as "flexible". This situation is defined in step 6 of the design procedure.

All of the connections exhibited possible multiple failure modes. Most of the failure modes are typical of simple connections, but two should be pointed out for discussion. One of these is the separation of the weld from the tube wall, which may be attributed to either lamellar tearing of the tube wall material or lack of penetration of the weld into the tube wall. The shear tabs used in the first two tests were welded to the tube wall with no preparation of the materials to take the weld. The subsequent failure in each of these two tests was a separation of the weld from the tube wall. For the remaining tests, the mill scale on both the tube wall and the tab, in the area of the weld, was removed and a preheat of approximately 200 degrees was applied to the connection before full welding. The separation problem was practically eliminated, with a few non-critical local exceptions. Therefore, it is recommended that careful preparation of the weld surfaces be undertaken to avoid this type of failure.

Another possible mode of failure is the warping of the tab due to lateral-torsional twisting of the beam end. Neither of the test beams were restrained laterally during the testing (the point load provided for some lateral restraint). It was concluded that the shear tab offers little resistance to this type of force and it is recommended that if the shear tab is at the end of a long, unbraced span a brace point should be established near the shear tab.

Tube Wall Strength

The tube wall experienced a punching shear failure in the two tests with the 3/16 inch thick tube wall (tests 6H40 and 7H40). The shear tab pulled out from the tube wall at the top of the shear tab around the perimeter of the welds. These were the only tests that did not meet the punching shear criteria of Equation 5.

Reaction Eccentricity

In the experimental tests, the reaction eccentricity was most affected by the width-to-thickness ratio of the tube wall and the span-to-depth ratio of the beam. It decreased with b/t of the tube wall and L/d of the beam, as shown in Figure 6. The number of test points was insufficient to develop a mathematical relationship between the eccentricity and the test parameters, so finite element models were developed in place of additional tests. From the results of the study, an empirical equation was derived for calculating the eccentricity:

$$e = 0.08 \left[\frac{\sqrt{t_{tw}}}{(w/t)} \right] (L/d)^{1.35} (d_{pl})^{1.3} \quad (\text{inches}) \qquad (6)$$

The w/t ratio is calculated as the nominal width of the tube over the thickness and the L/d ratio is for a uniformly loaded beam. This equation is plotted in Figure 8. The actual eccentricities from the FEA models and some of the experimental tests are also plotted in Figure 8. There is some scatter in the data and the equation produces overly conservative eccentricity values when the w/t ratio is low. Therefore, rigid reliance on the values calculated from this equation is not justified. Instead, Equation 6 will be used to determine whether the eccentricity is acting 1) outside of the bolt line, or 2) between the bolt line and the weld line. Depending on which case occurs, the eccentricity will be determined by a different method.

If "e" calculated from Equation 6 is greater than or equal to 3, the eccentricity is acting outside of the bolt line and should be calculated using Equations 1 and 2, in accordance with Astaneh's procedure. If "e" is less than 3, the eccentricity is acting between the bolt line and the weld line. The eccentricity can be assumed to act at the midway point between the weld line and the bolt line for this case, giving an eccentricity of 1.5 inches. However, this may be an unconservative estimate if the eccentricity acts nearer to the bolt line or the weld line. Since the scatter of the data can be enclosed by 50% bounds, as shown in Figure 6, the eccentricity of 1.5 inches will be multiplied by 1.5 to give a value of 2.25 inches. This will be the value of the eccentricity for both the bolt group calculation (e_b) and the weld group calculation (e_w).

The moments developed in the connections never exceeded 20% of the fixed end moment, so the beam framing into the shear tab can be considered simply supported.

Bolt Installation

The method of bolt installation had no noticeable effect on the ultimate load. At working loads the connections with the fully torqued bolts was stiffer than the connections with the snug tight bolts. The deflection of the beam at the bolt line was less and the eccentricity was larger. As the connection load increased, the bolts were brought into bearing and the connections exhibited behavior similar to the behavior of the connections with snug tight bolts. The average eccentricities for the two different bolt tightness conditions showed very little difference for the various connections (Figure 6).

Table 4

Experimental Maximum Shear Capacities
and
Corresponding Theoretical Capacities

TEST	t_{pl} in.	d_{pl} in.	e_{ave} in.	V_{TEST} kips	THEORETICAL				
					V_Y kips	V_{FRAC} kips	V_{BV} kips	V_{BB} kips	V_{WELD} kips
1H5T	0.327	9	5.34	74.9^	84	94	66	89	54
2H5S	0.306	9	4.87	74.0^	92	84	73	87	59
3H10S	0.327	9	4.06	73.2^	84	94	82	111	68
4H16T	0.308	9	2.33	70.3	92	84	87	106	97
5H16T	0.326	9	2.55	70.1	84	94	90	121	93
6H40T	0.306	9	1.22	72.2	92	84	74	89	120
7H40S	0.306	9	1.18	69.8	92	84	73	88	121
8H45T	0.244	9	0.98	59.8	65	70	71	71	93
9H45S	0.308	9	1.33	70.3	92	84	75	91	118
10L5T	0.307	15	2.83	142.9	153	140	157	190	185
11L5S	0.307	15	1.97						
12L16S	0.327	9	1.13	83.8	84	94	73	98	122
13L45S	0.308	15	0.32	131.9	154	140	129	157	220

^ Tests 1H5, 2H5 and 3H10 began yielding in shear, but the tests were stopped because the beam had reached yield. Test 11L5 also began yielding in shear, but the test was stopped due to tipping of the test fixtures.

- The capacities for the bolts in shear and bearing (V_{BV} and V_{BB}) and the welds (V_{WELD}) take into account the eccentricity of load and are calculated by the ultimate strength method. The theoretical shear yield capacity is denoted V_Y and the shear fracture of the tab by V_{FRAC}.

- e_{ave} is the average of the eccentricities relative to the weld line. See Figure 5 for a comparison of some typical eccentricity curves compared to the calculated average.

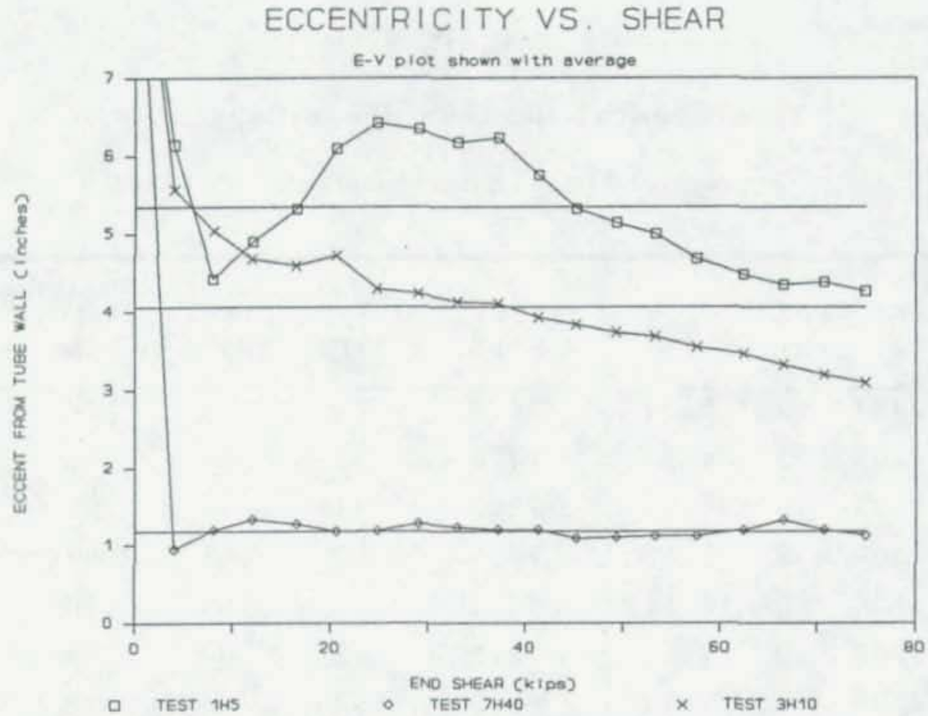


Figure 5 : Eccentricity versus Shear Plot

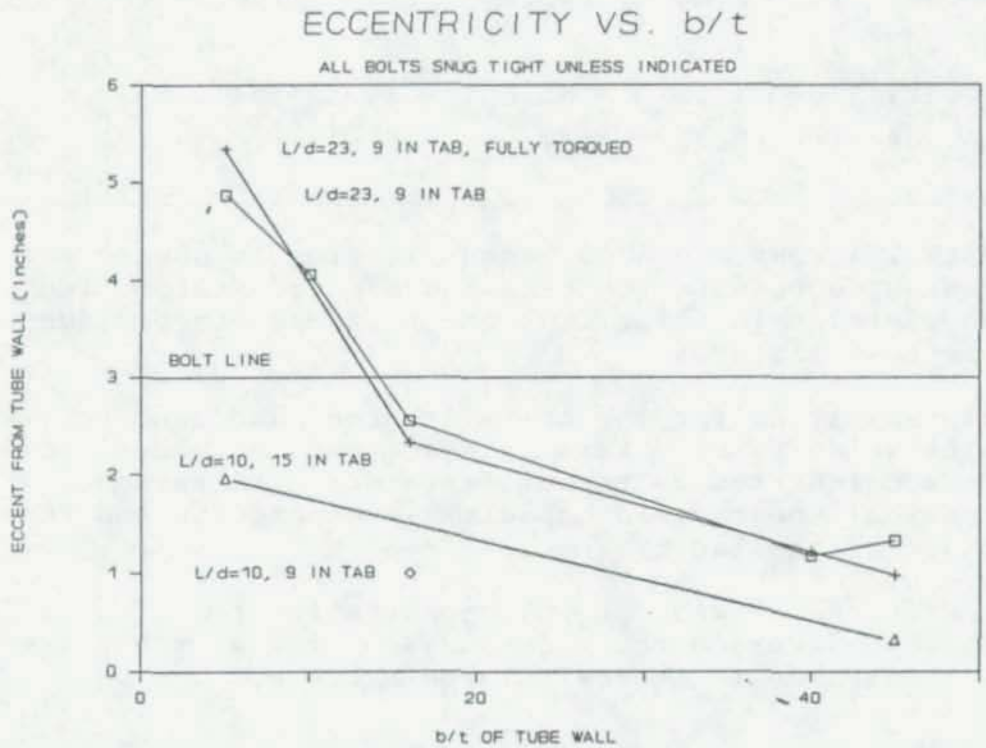
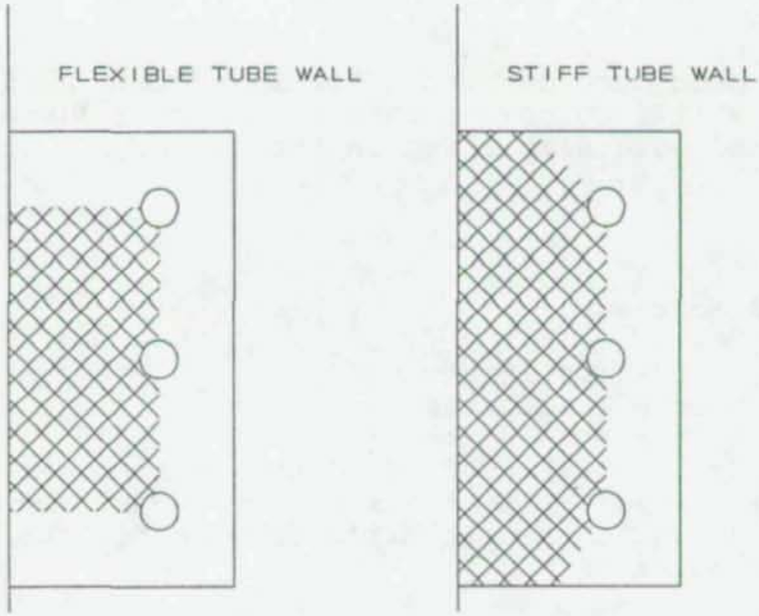


Figure 6 : Eccentricity versus b/t Ratio



THE HATCHING INDICATES THE EFFECTIVE AREA RESISTING THE SHEAR FORCE, AS INDICATED BY THE FLAKING OF THE WHITE WASH DURING THE TEST.

Figure 7 : Load Transfer in the Shear Tab

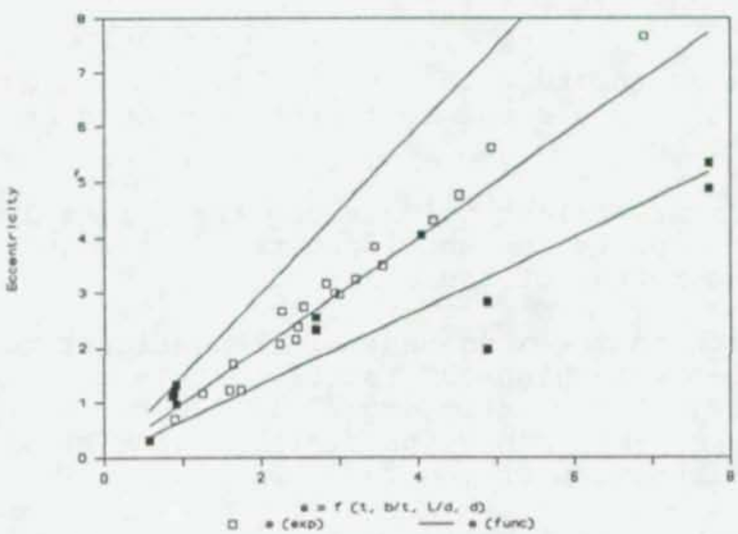


Figure 8 : Eccentricities as a Function of Test Parameters

The filled squares represent data from the experimental testing and the hollow squares represent data from the FEA models. The two outside lines are the 50% bounds of Equation 6.

DESIGN PROCEDURE

The design procedure developed for shear tabs welded to tubular columns will incorporate some of the procedures developed by Astaneh [2] and some simplified in the AISC-ASD Code [3] as Table X.

A.) Assumptions

Shear Tab Dimensions:

- Width = 4.5 inches
- Edge Distance = 1.5 inches
- Bolt Pitch = 3 inches
- Distance from Bolt Line to Weld Line = 3 inches
- $t_{pl} \leq d_b/2 + 1/16$ inches (ductility requirement)
- $d_{pl} \geq 1/2\ T$ (T is the web depth between fillets)
- Number of Bolts: $2 \leq n \leq 7$

Material Properties:

- Shear tab material is A36 steel.
- Welds are made with E60XX or E70XX electrodes.

B.) General Requirements

- If the shear tab is at the end of a long unbraced length, a brace point should be established near the shear tab.

- The area of the tube wall to receive the weld should be free from mill scale and some preheat should be applied before welding.

- The welds are fillet welds along the entire length of both sides of the plate and should be terminated just short of the top and bottom of the shear tab.

- It is left to the judgement of the designer as to whether to use tubes with high b/t ratios. It is recommended that the b/t ratio not greatly exceed 16 due to the uncertainties regarding possible reduction in the column strength caused by local distortion of the tube wall.

- The design is applicable for both fully torqued and snug tight bolts.

C.) Design Procedure

1.) Calculate the number of bolts required to resist the shear force. Assume the force acts at the bolt line:

$$n = \frac{R}{r_v}$$

2.) Calculate the maximum thickness of the shear tab allowed to insure yielding of the shear tab before punching shear failure of the tube wall:

$$t_{pl} \leq t_{tw} \frac{1.2F_{u_{tw}}}{F_{y_{pl}}}$$

3.) Calculate the length (or thickness) of the shear tab needed to resist the shear force :

$$d_{pl} \geq \frac{R}{0.4\ F_{y_{pl}}\ t_{pl}}$$

4.) Calculate the reaction eccentricity:

a.) Determine "e":

$$e = 0.08 \left[\frac{\sqrt{t_{tw}}}{(w/t)} \right] \left(\frac{L}{d} \right)^{1.35} (d_{pl})^{1.3}$$

b.) If e ≤ 3, the point of eccentricity is between the bolt line and the weld line and:

$$e_b = e_w' = 2.25$$

c.) If e > 3, the point of eccentricity is outside of the bolt line and:

$$e_b = (n - 1) - 3$$

$$e_w = n$$

5.) Recalculate the capacity of the bolt group using Table XI of the AISC-ASD Manual:

$$R_b = C (r_v)$$

where L = e_b.

6.) Recalculate the length (or thickness) of the shear tab using the reduction factor ξ, which takes into account the reduction in shear capacity due to the distortion in the tube wall:

$$d_{pl} \geq \frac{R}{\xi \; (\; 0.4 \; F_{y_{pl}} \; t_{pl} \;)}$$

where ξ = 0.80 if e ≤ 3 and b/t ≥ 15
= 1.00 for all other cases.

7.) Check for fracture along the net section:

$$R_{ns} = [\; d_{pl} - n \; (\; d_b + (1/16) \;] \; (\; t_{pl} \;) \; (\; 0.3F_{u_{pl}} \;)$$

8.) Calculate the weld size, in sixteenths, to develop the shear capacity the tab, using Table XIX from the AISC-ASD Manual:

$$D = \frac{R_{pl}}{C \; (\; d_{pl} \;)}$$

where aL = e_w. The weld size need not exceed $0.75t_{pl}$.

9.) Check the bearing capacity of the bolt group:

$$P_b = (\; 1.2F_{u_{pl}} \; t_{pl} \; d_b \;) \; n$$

10.) Check for a block shear failure if the beam is coped.

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