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TESTING AND DESIGN OF EXTENDED SHEAR TABS - PHASE I

by

Donald R. Sherman and Al Ghorbanpoor

University of Wisconsin-Milwaukee

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Abstract

The single shear plate, or shear tab, is a common type of connection in structural steel framing. It is considered a simply supported connection since only small end moments develop in the beam with this type of connection. A standard shear tab welded to a column flange consists of a single plate with a 3-in. distance between the vertical weld and the bolt line of the connection. In this case, there is no need for expensive coping or flange reduction of framing beams. However, if it is desired to frame the supported beam using a connection to the web of a wide-flange column or the web of a girder it becomes necessary that the beam framing into this type of connection is coped in order to avoid interference with supporting member flanges.

The purpose of this research is to develop a design procedure for a special type of shear tab connection, the extended shear tab. In the extended shear tab, the bolt line on the shear tab extends 3-in. beyond the flanges of the supporting member when the intent is to frame a beam into the web of a wide-flange column or the web of a girder. This type of connection is economically attractive because it eliminates the need for coping or flange reduction of framing beams, which is a time and cost consuming process in structural steel fabrication. In addition, the erection of this type of connection is faster and safer due to the fact that there is no difficult maneuvering of framing beams between supporting member flanges at both ends of the beams.

In order to develop a design procedure for extended shear tabs, a research program that consisted of 17 full-scale tests was performed. The research consisted of two parts. The first part investigated extended shear tabs with no stiffening, by means of horizontal welds, at the top or bottom of the tab. The second part investigated extended shear tabs that were stiffened on either the top, or top and bottom, depending upon whether the connection was made to a support girder or column. The purpose of the research was to identify the eccentricity of the shear force at the end of the beam relative to the end connection bolt line and to determine the critical limit states for the connection.

The results of the research program show that the unstiffened extended shear tabs have lower capacity than standard shear tabs even though they are thicker. However, they are more advantageous to use on beams with relatively light end reactions. Extended shear tabs that were stiffened showed capacity comparable to the standard shear tabs. For all tests, the shear tabs showed considerable shear distortion. For some tests, bolt shear and bolt bearing were evident. Out-of-plane twisting of the extended tabs was observed for most tests, and it was more pronounced for deeper connections made with five-bolts. For unstiffened tests, a web mechanism failure was observed in the support column.

The test results were summarized in a paper included in the Proceedings of the year 2000 North American Steel Construction Conference held in Las Vegas. The paper also contained a recommended design procedure. This report amplifies the discussion in the paper and includes the complete data base in the Appendices. The equation for twisting given in Section 6.2 of this report has been changed from that in the paper and produces results that are more consistent with observed behavior. Also, an initial step has been added to the design procedure for estimating the number of bolts and the length of the tab.

ACKNOWLEDGEMENTS

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

EXTENDED SHEAR TABS - PHASE 1

1. INTRODUCTION

A common type of simple framing connection is the single plate shear connection in which a single plate is shop welded to the support member and field bolted to the supported beam. In the early-to-mid 1980s the single plate shear connections, also known as shear tabs, for the transfer of simply supported beam end reactions became more popular based on the ease of fabrication and the ease of field erection of this type of connection. A shear tab is considered a simply supported connection because only small end moments develop at the connection due to the small distance that exists between weld and bolt lines (typically 3-in.). The plate has prepunched bolt holes. In the field, the supported beam, also with prepunched bolt holes, is moved into position and field bolted to the framing plate. When the objective is to frame a beam into a support column flange, the shear tab can be a very simple and effective framing connection (see Fig. 1.1). The fabrication and field erection are both quick and cost effective.

Figure 1.1: Typical shear tab connection to column flange

However, when the desire is to frame into the web of a support column or web of a support girder, both the fabrication and field erection become more cumbersome and time consuming. This is due to the fact that the supported beam must have either its top flange or both top and bottom flanges coped depending on whether framing into a girder or column. In addition, the drop-and-bolt method used in field erection for connection to a column flange can no longer be employed because the supported beam must be maneuvered between the supporting member flanges for connection to the shear tab. To alleviate these problems, it is possible to use an extended shear tab. With the extended shear tab, the bolt line extends 3-in. beyond the flanges of the supporting member. This type of connection is analogous to the standard shear tab in terms of steel fabrication and field erection which makes the extended shear tab a very economical alternative for simple framing connections.

There are three possible weld configurations for connection of an extended shear tab to a support member. In either columns or girders, it is possible to connect the shear tab with only a pair of vertical fillet welds. For girders, a pair of horizontal welds between the top of the tab and the top flange of the support girder can be used in addition to the vertical welds. For columns, stiffening plates are welded between the column flanges at both the top and bottom of the tab. Horizontal welds between stiffening plates and tabs are placed at the top and bottom of the shear tab in addition to the vertical welds to the column web. Although all three of these weld configurations have been used in practice, there has been no uniform design procedure available until this study.

2. OBJECTIVES

The purpose of this research was to study the behavior of and to develop a design procedure for extended shear tabs welded to the webs of support girders or columns. Specific objectives of this testing and evaluation program were:

1) To evaluate the capacity of extended shear tabs.
2) To determine which limit states, commonly associated with standard shear tabs, may be the critical limit state for unstiffened and stiffened extended shear tabs. These limit states are bolt shear, bolt bearing, shear yield of the tab plate, shear rupture of the tab plate, block shear of the tab plate and weld fracture.
3) To identify any additional limit states (support web failure, twisting) that may result due to the large weld to bolt line distance that is present in an extended shear tab.
4) Determine the location of the shear reaction eccentricity relative to the bolt and weld lines.
5) To recommend a uniform design procedure for extended shear tabs.

This research was separated into two parts:

1) The investigation of "unstiffened" extended shear tabs - This part of the research consisted of conducting a set of four tests: two tests with the extended tab welded to a support girder web and two tests with the extended tab welded to a support column web. The weld configuration for all tests in this part consisted of only a pair of vertical fillet welds between the extended shear tab and support web (see Fig. 2.1). The weld configuration for these tests is the same as used for standard shear tabs.

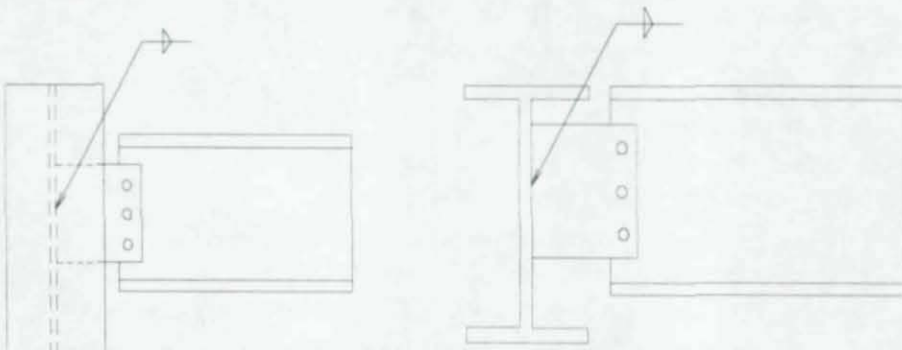


Figure 2.1: Unstiffened extended shear tabs

2) The investigation of "stiffened" extended shear tabs - This part of the research consisted of conducting a set of thirteen tests. Five tests were with extended tabs that were welded to girder webs and eight tests were with extended tabs welded to column webs. For the girder tests, the weld configuration consisted of vertical fillet welds between the tab and girder web and a pair of horizontal fillet welds between the top of the extended shear tab and top flange of the support girder. For the column tests, stiffening plates were welded to the

inside faces of both flanges of the support column and the extended shear tab configuration included vertical welds between the tab and column web and horizontal welds between the tab and stiffening plates both at the top and bottom of the extended tab (see Fig. 2.2).

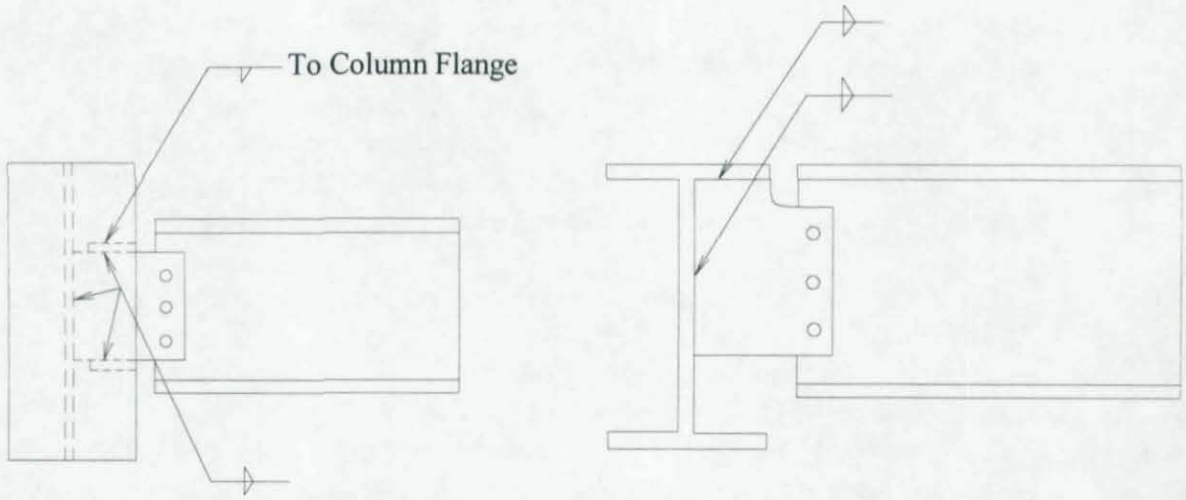


Figure 2.2: Stiffened extended shear tabs

The behavior and capacity of both the unstiffened and stiffened extended shear tabs were studied as a function of the following test parameters:

1) The span-to-depth ratio (L/d) of the supported beam
2) The width-to-thickness ratio (h/t_w) of the support member web
3) The size of the shear tab
4) The number of bolts
5) The type (STD or SSL) of bolt holes, and
6) Lateral bracing of the supported beam

In addition, four other "special" cases were investigated to determine their effect on the behavior and capacity of extended shear tabs. They were:

1) A column test with no vertical welds to connect the tab to the column web. Only horizontal welds between the tab and stiffening plates were made.
2) A column test with a deep (19-in.) shear tab. This would simulate making a shear tab connection to continuity plates (or bearing stiffeners) that were required due to a beam framing into the column flanges.
3) Two column tests where the stiffening plates were also welded to the support web.
4) A girder test with a vertical fillet welds only on one side of the shear tab.

3. BACKGROUND

3.1 Types of Connections

The American Institute of Steel Construction (AISC) categorizes steel frames by the amount of restraint developed by their connections. Based on this amount of restraint, steel frames are divided into one of three types. These types include type FR (fully restrained), PR (partially restrained), and simple framing that are outlined in Section A2.2 of Part 6 of the Load and Resistance Factor Design (LRFD) Specification [1]. Shear tabs are considered as simple framing connections with the following characteristics.

Simple Framing: This type assumes that "for the transmission of gravity loads the ends of the beams and girders are connected for shear only and are free to rotate". For simple framing, the following requirements (LRFD A2.2) apply:

a) The connections and connected members shall be adequate to resist the factored gravity loads as simple beams,
b) The connections and connected members shall be adequate to resist the factored lateral loads, and
c) The connections shall have sufficient inelastic rotation capacity to avoid overload of fasteners or welds under combined factored gravity and lateral loading.

3.2 Load Transfer to Shear Tabs

The shear tab connection is well suited to beams that have a relatively moderate or light end shear. The supported beam reaction or shear load is assumed to be distributed equally among the bolts in the connection. Also, it is assumed that relatively free rotation is allowed to occur between the end of the supported member and the supporting beam or column. The connection achieves this relatively free rotation from: bolt slip if the bolts are not in bearing at the time loading is initiated, bolt hole distortion in the beam web and/or the shear tab, bolt deformation in shear, and the flexibility of the supporting element. However, due to eccentricities of the shear force, these connections can develop some connection moments at the bolt line and weld. The magnitude of these connection moments is dependant on the number and size of the bolts and their arrangement, the span-to-depth ratio of the beam, the type of loading, the flexibility of the supporting element, and the thickness of the connection plate. The bolt tightening force can also have an effect on these connection moments at service loads, but does not appear to be a major factor at ultimate loads. When symmetry is used and a shear tab connection is made on both sides of a supporting member, the flexibility of the connection is reduced due to the stiffening of the support web by the shear tab on either side. This would increase connection moments by increasing the connection flexural restraint. In general, the supported beam shear is transferred to tab in the form of shear and moment.

3.2.1 Shear

As previously discussed, the shear tab is considered a simple framing connection. As a result, the shear tab is proportioned only for shear. The AISC-LRFD Specifications outline the general provisions regarding simple connections as follows:

"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."

The connection must be designed to meet these general AISC provisions.

3.2.2 Moment

The magnitude of the moment relative to the bolt line or weld developed in a simple connection, such as the shear tab, can be approximated by $M = R*e$, where R is the shear reaction and e is the reaction eccentricity. Although simple connections are normally designed exclusively for shear transfer, significant connection moments do affect the proportioning of bolt and weld sizes. The reaction eccentricity is defined as the distance from a reference point to the point of zero moment. That reference point is usually taken as the bolt line (see Fig. 3.1) or the weld line for design purposes. The reaction eccentricity is dependent on a number of factors including the number of bolts, the relative flexibility or rigidity of the supporting member, the thickness and proportions of the shear tab, the extent of bolt tightening, and the amount of rotation at the end of the supported beam (which is dependent on the stiffness or L/d ratio of that beam.)

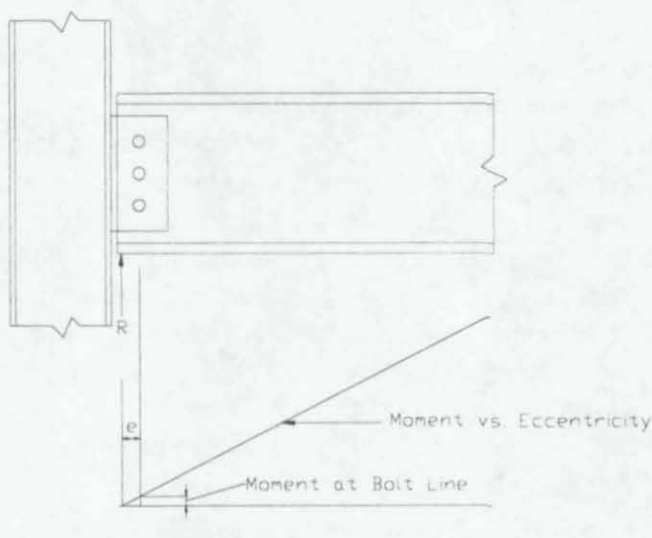


Figure 3.1: Determination of connection moment

3.3 Design of Shear Tabs

In the design of standard shear tabs, various limit states need to be investigated. The relevant limit states are bolt shear, bolt bearing, gross shear yield of the tab, net shear rupture of the tab, block shear of the tab, and weld failure. The number of bolts and weld size is based on ultimate analysis with the effect of the eccentricity included. Three other limit states need to be investigated when the flanges of beams need to be coped in order to make the shear tab connection. The first of these is the block shear of the supported beam web. In addition, copes of the flanges may affect local web buckling, as reported by Cheng and Yura [2], and lateral-torsional buckling, as reported by Cheng, Yura, and Johnson [3]. Based on observed buckled shapes in their experimental study and a theoretical parametric study, Cheng and Yura determined that the buckling capacity of a coped beam is governed not only by the buckling strength of the uncoped length but also by the buckling strength of the coped region. For beams with long and deep copes, the buckling capacity was controlled by the coped region and was nearly independent of the uncoped region. This explained why short beams with coped sections had a significantly reduced buckling capacity.

3.4 AISC Shear Tab Design Procedure

Richard [4] and Astaneh [5] have performed research on standard shear tabs. These studies included investigations of the shear tab welded to the flange of wide-flange columns. Each of the studies developed a design procedure for the connection, but the current AISC procedure is primarily based on Astaneh's procedure. Based on the experimental results, the failure modes to be considered are the following.

Shear failure of bolts
Yield of gross area of plate
Fracture of net area of plate
Fracture of welds, and
Bearing failure of beam web or plate.

The steps taken in the design adopted by AISC for a shear tab are as follows:

1) Calculate the number of bolts needed to resist the combined effects of shear R and moment R^*e_b.

For a rotationally rigid support:	STD holes	$e_b = \lvert(n-1) - a\rvert$
	SSL holes	$e_b = \lvert 2n/3 - a\rvert$
For a rotationally flexible support:	STD holes	$e_b = \lvert(n-1) - a\rvert \geq a$
	SSL holes	$e_b = \lvert 2n/3 - a\rvert \geq a$

2) Calculate the gross area of the plate, dimension the plate, and check the plate for shear yielding of the gross area, fracture of the effective net section and block shear rupture.

3) Check a rotational ductility requirement to insure that the tab is not too stiff

$$t \leq d_b + 1/16\text{-in.}$$

4) Check for buckling of the tab

$$t \leq L/64 \geq 1/4\text{-in.}$$

5) Design the fillet welds for the combined effect of shear and moment.

a) $e_w = (n)$, or
b) $e_w = a$ (whichever is larger)

However, the AISC Manual conservatively uses a weld size that is ¾ of the plate thickness.

6) Check the bearing capacity of the bolt group in the tab and beam web.

3.5 Additional Failure Modes for Extended Shear Tabs

The extended shear tab is a shear tab that extends 3-in. beyond the supporting member flanges. Due to the large distance between the weld line and bolt line for this type of connection there are other possible failure modes that need to be considered. The flexibility of the support web is of particular concern when the extended shear tab is connected only to the web of a support column. Different boundary conditions exist when connecting the shear tab to a girder web or a column web with stiffening plates. For these cases, the flexibility of the support web is not a concern. Sherman and Ales [7] conducted research to develop a design procedure for shear tabs connected to tubular columns. The research investigated the evaluation of the tube wall strength. It identified two failure modes for the tube wall. The tube wall could experience either a bending failure or punching shear failure. The bending failure limit state is determined from the yield line theory and plastic analysis. The bending failure is a result of the tube wall producing plastic moments along yield lines. When a sufficient number of plastic hinges have developed, as a result of these yield lines, the tube wall can turn into a mechanism and fail (see Fig. 3.2). The punching shear limit state takes place when the applied load exceeds the shear resistance of the tube wall around the perimeter of the shear tab. This failure mode can be avoided if the shear tab is proportioned so that the shear tab yields before the tube wall fractures in shear. The column support web is analogous to the tube wall for these two failure modes and hence needs to be investigated for bending failure and punching shear failure in the design of extended shear tabs. Punching shear is also a possibility for thin girder webs.

Another failure mode that needs to be investigated in the design of extended shear tabs is the twisting of the shear tab, especially when deep extended shear tabs are to be used (5 or more bolts). This twisting is caused by the eccentricity of the reaction shear with respect to the centroid of the shear tab. This eccentricity is due to the fact that the supported beam web is connected only on one side of the shear tab. For standard shear tabs, this eccentric loading is not of concern but for extended shear tabs this eccentricity can cause torsional shear stresses in the shear tab as a result of pure torsion of the rectangular shear tab cross section. Failure is predicted to occur when the shear stress produced from this torsion, combined with direct shear stresses, equals or exceeds the shear yield of the material.

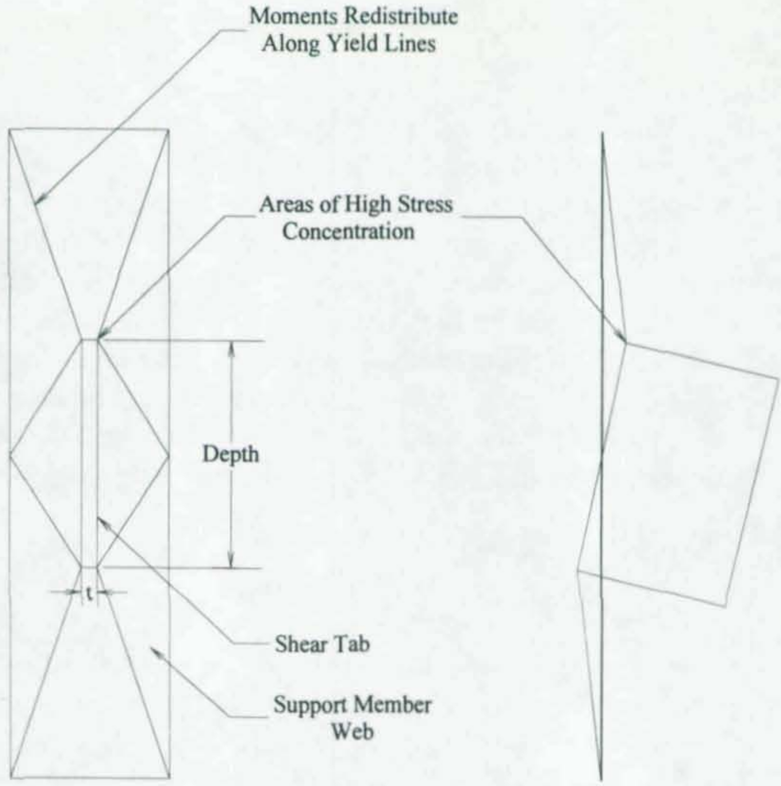


Figure 3.2: Web bending failure

4. EXPERIMENTAL PROGRAM

4.1 Test Beam Selection

In actual building construction, most framed beams are uniformly loaded along their length. The shear tab that connects these beams to girders or columns experiences a realistic combination of reaction shear and end rotation. In an experimental program investigating the behavior and capacity of shear tabs, it is essential that the shear tabs tested be subjected to this realistic shear-rotation loading. Laboratory testing is usually limited to point loading. In order to address this problem, a correlation may be developed between point loaded and uniformly loaded beams so that the location of a point load can be selected in such a manner as to produce the same reaction and rotation at the beam ends. Consider the uniformly loaded beam AB in Figure 4.1. For a simply supported beam, the end reaction R_A and end rotation θ_A are given as:

$$R_A = \frac{w\,(L_u)}{2} \qquad \theta_A = \frac{w\,(L_u)^3}{24EI} \qquad 4.1$$

By solving each of the above equations for w and setting them equal, an expression for θ_A in terms of R_A can be found:

$$\theta_A = \frac{R_A\,(L_u)^2}{12EI} \qquad 4.2$$

Now consider beam CD of Figure 2.2. The beam has a point load applied at a distance a from point C. The end reaction R_C and end rotation θ_C are given as:

$$R_C = \frac{b}{L_p}(P) \qquad \theta_C = \frac{P\,b\,(L_p{}^2 - b^2)}{6\,(L_p)\,EI} \qquad 4.3$$

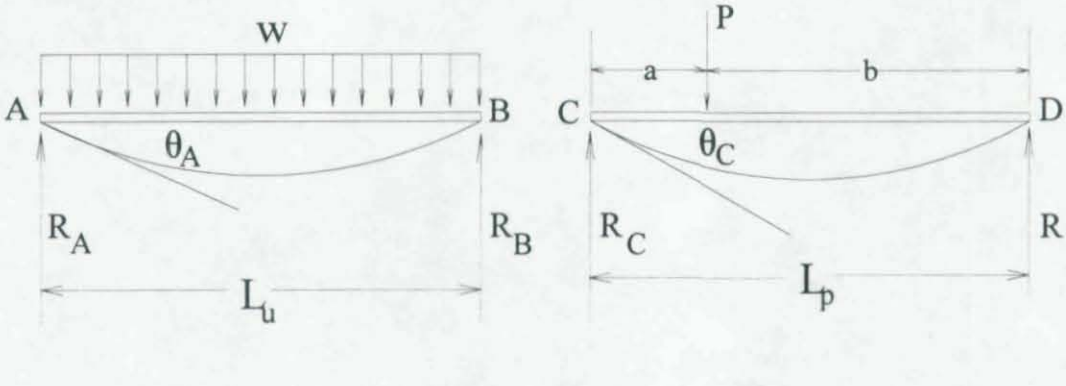


Figure 4.1: Test beam selection variables

Similarly, by solving the two equations for P and setting them equal to each other an expression for θ_C in terms of R_C can be found:

$$\theta_C = \frac{R_C \, (L_p^{\,2} - b^2)}{6\,EI} \qquad 4.4$$

Since θ_A is to be simulated by θ_C, the two terms are set equal to each other:

$$\frac{R_A \, (L_u)^2}{12EI} = \frac{R_C \, (L_p^{\,2} - b^2)}{6EI} \qquad 4.5$$

R_A must equal R_C to complete the simulation, and the modulus of elasticity and moment of inertia for the beam are constant independent of loading, therefore the (R/EI) terms may be cancelled in the equation and a relationship can be developed in terms of the relative lengths of the beams and the location of the concentrated load:

$$L_u = \sqrt{2}\sqrt{(L_p^{\,2} - b^2)} \quad \text{or} \quad L_p = \sqrt{\frac{L_u^{\,2}}{2} + b^2} \qquad 4.6$$

These formulas allow to determine the location of the point load on the beam to provide end reaction and rotation equivalent to those from a uniformly distributed load. They were used to design the test beams in this study.

One objective of the research was to study connection behavior as a function of end rotation. End rotation is a function of the span-to-depth ratio (L/d) of the supported beam. To do this, two beams with different L/d ratios were selected. The first beam was designated as the high rotation beam and was used with the three bolt connections. The L/d ratio for this beam was approximately 24, which is a relatively high span-to-depth ratio for beams commonly used in building construction. The second beam was designated as the low rotation beam and was used with the five bolt connections. The L/d ratio for this beam was approximately 12. The beams that were selected to fit this criteria were a W12X87 (L_u/d ratio of 23) and a W18X71 (L_u/d ratio of 10). Test beam selection calculations are included in Appendix A.

4.2 Test Program

Another objective of the research was to study connection behavior as a function of the h/t_w of the supporting member webs. A range of h/t_w ratios for the supporting member webs was selected. This range was from 22 to 54. There were four combinations of supporting members and supported beams used in the experimental program. Two of these groups used support girders and two used support columns. They are:

Group 1: W14X53 support girder ($h/t_w = 30.8$) with 30-ft. long W12X87 beam
Group 2: W24X55 support girder ($h/t_w = 54.6$) with a 20-ft. long W18X71 beam
Group 3: W8X31 support column ($h/t_w = 22.2$) with a 30-ft. long W12X87 beam
Group 4: W14X90 support column ($h/t_w = 25.9$) with a 20-ft. long W18X71 beam

For all of the tests, the column segments were 8-ft. long and the girder segments were 10-ft. long. See Figure 4.2 for the test setups for all four support member and beam combinations. Calculations for the locations of the point loads are in Appendix A.

4.2.1 Far end beam reaction

The far end beam support consists of a load cell, a roller bar placed directly on top of the load cell, and a support fixture used to stabilize the load cell. The load cell is used to measure the far end reaction. For the column tests, the support fixture was placed on top of two large steel crates and shimmed in order to bring the support fixture and load cell up to a height capable of measuring the far end beam reaction (approximately 4-ft. off of the laboratory's test floor). For the girder tests, the support fixture was placed directly onto the test floor. A schematic of the far end reaction is shown in Figure 4.3 (a photograph of the actual far end reaction can be found in Appendix B, Figure B.1).

4.2.2 Beam loading mechanism

The jacks used to load the test beams had a capacity of 120 kips each. The "high rotation" tests using the W12X87 test beam, required only one jack. For the "low rotation" tests that used the W18X71 test beam, two jacks were needed and were placed side-by-side to apply the load. The jacks were mounted to a cross beam which was secured to the laboratory test floor using 2-in. diameter high strength steel threaded rods (capacity of 150 kips each). Small diameter steel cables were strung from the crossbeam and secured to the test floor to provide lateral bracing for the crossbeam and jacks. These cables also served a dual purpose in that they were the means by which the entire loading fixture was plumbed so that the load application was vertical. The load setups for the two test beams are shown in Figure 4.4 (photographs of the test setups can be found in Appendix B, Figures B.2 and B.3).

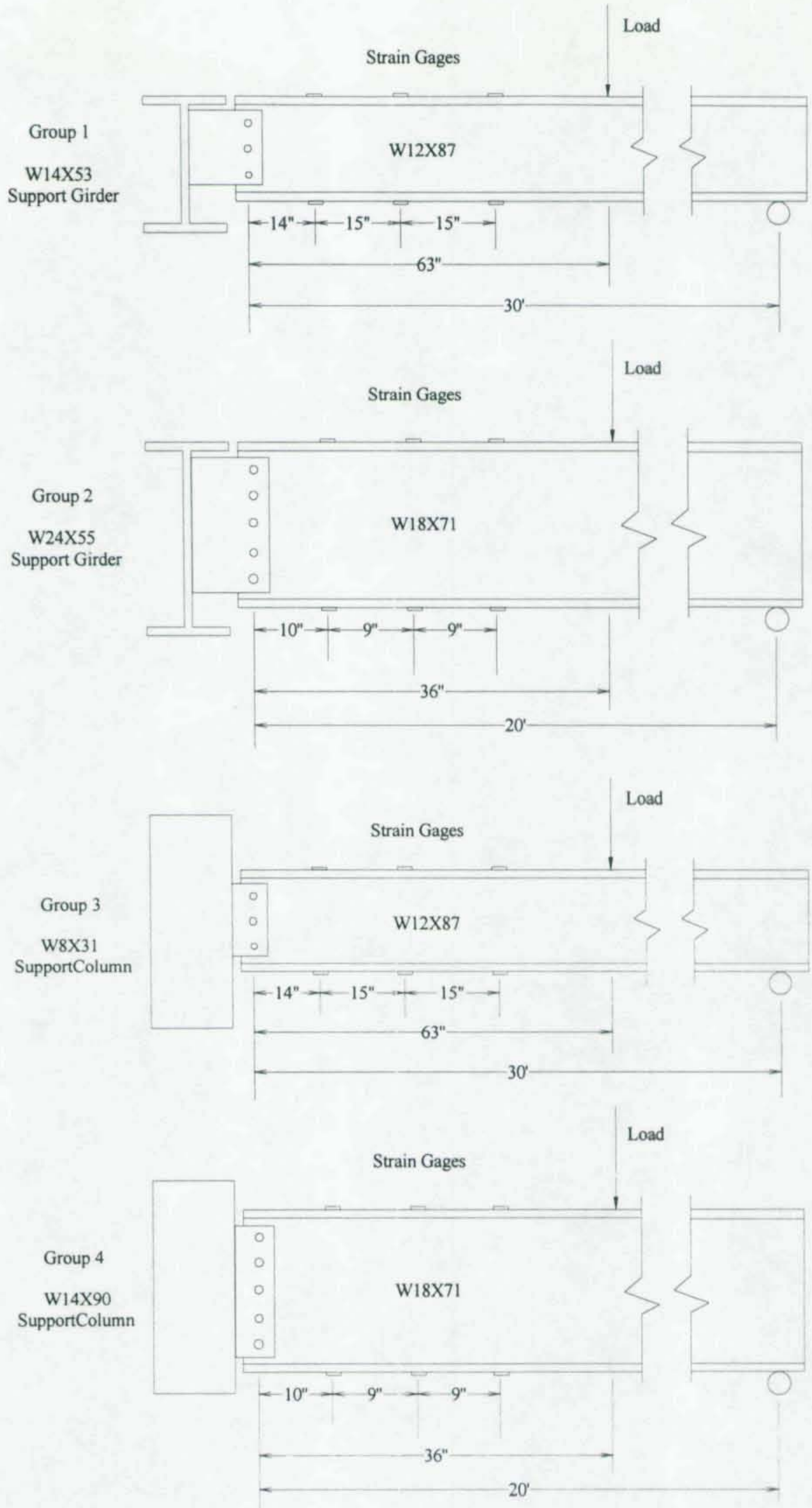


Figure 4.2: Setups for beam and support member combinations

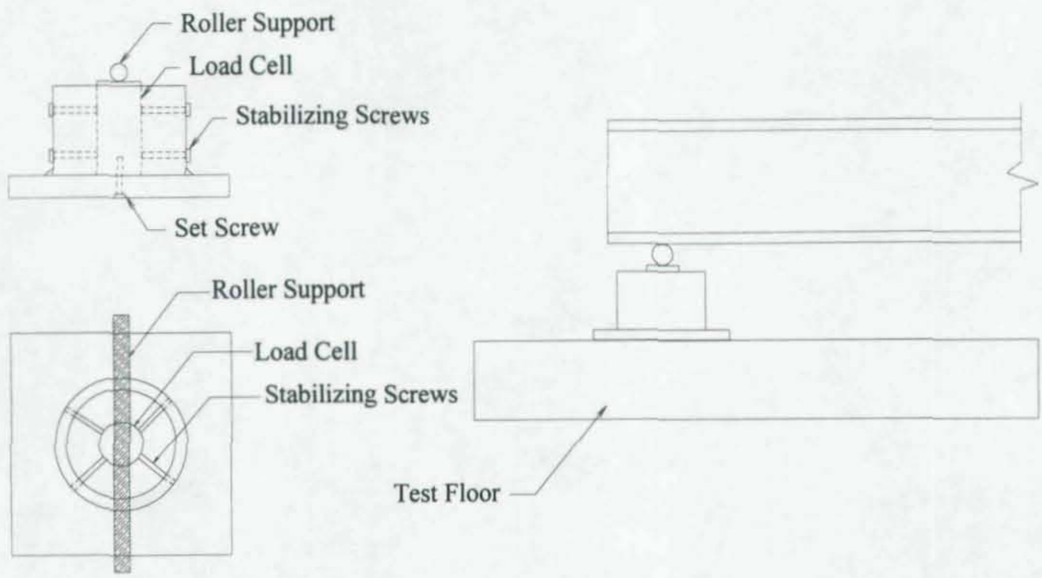


Figure 4.3: Far end reaction

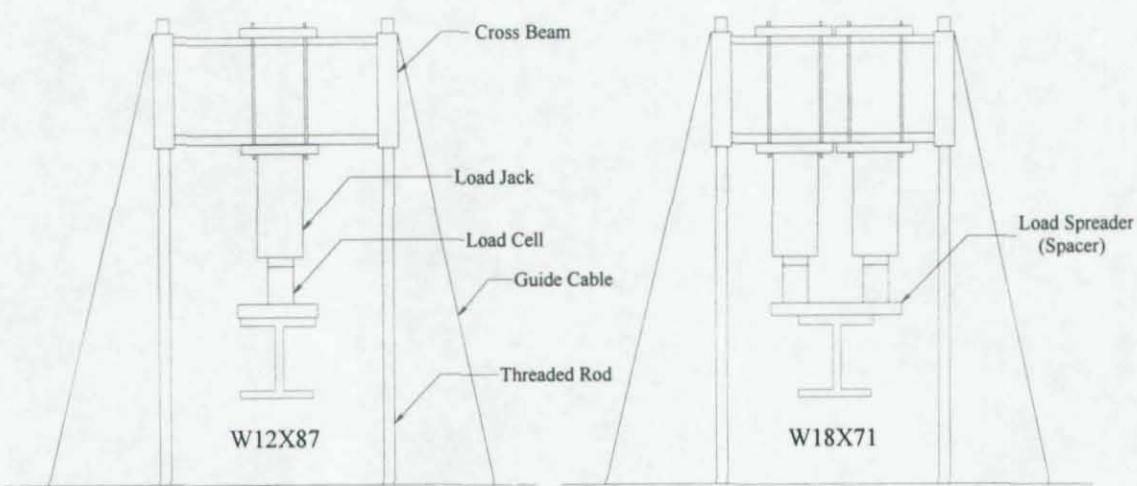


Figure 4.4: Beam loading apparatus

4.2.3 Girder and column supports

As previously mentioned, all girders tested were 10-ft. in length and all columns were 8-ft. in length. Two cross beams held the girders to the laboratory test floor at the ends of the girder. These cross beams were connected to the test floor with 1-in. diameter, high strength steel rods. These cross beams provided an unrestrained length of 9-ft. for the girder. The girder was shimmed on either end at the locations of the cross beams so that the girder did not rest directly on the test floor. The support columns were placed vertically against a reaction wall and tied back to the wall with two crossbeams and 1-in. diameter steel rods. Two ½-in. diameter steel rods were placed behind the columns at the locations of the crossbeams so that the columns were not bearing directly on the reaction wall. Both of these tie down methods allowed the respective supporting member to have realistic rotational capabilities while properly resisting the reaction shear and moment. These girder and column supports can be seen schematically in Figure 4.5 (photographs of actual supports can be seen in Appendix B, Figures B.4 and B.5).

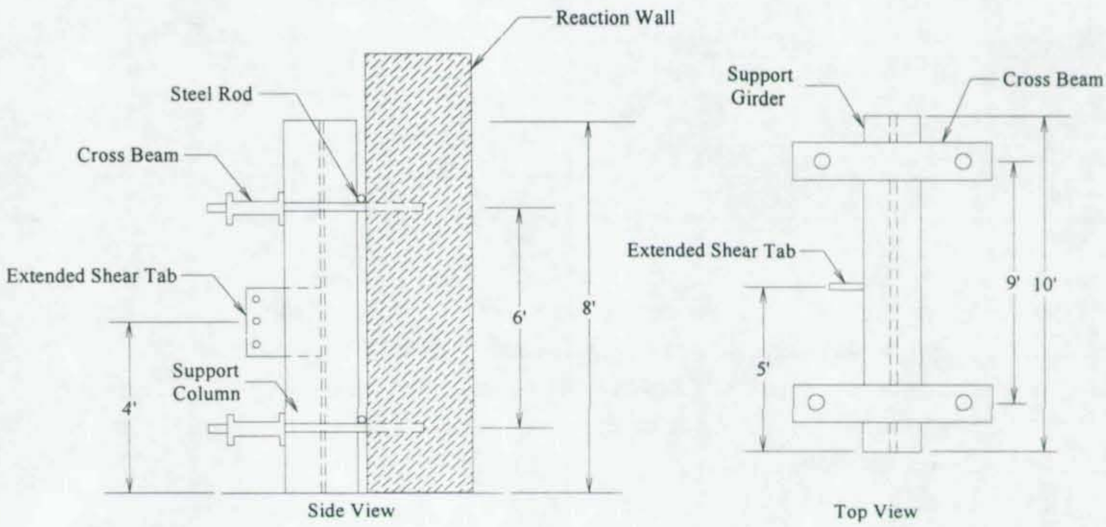


Figure 4.5: Test column and girder supports

4.3 Test variables

The test program was done in two parts. Part 1 investigated the extended shear tabs with no stiffeners and Part 2 examined the extended shear tabs with stiffeners. There were some parameters that were common for all of the tests performed. They are:

1) Group 1 (W14X53 support girder) and Group 3 (W8X31 support column) always used 3-bolt connections and the flexible W12X87 test beam.
2) Group 2 (W24X55 support girder) and Group 4 (W14X90 support column) always used 5-bolt connections and the stiff W18X71 test beam.
3) All extended shear tabs were A36 steel.
4) All support members and both test beams were Grade 50 steel.
5) All bolts used were ¾-in. diameter A325-X.
6) All bolts were snug tight in standard holes and fully tightened in slotted holes.
7) E70 electrodes were used for all welding.
8) All columns were 8-ft long and all girders were 10-ft. long. In both cases, the tabs were welded at mid-length.
9) Fillet welds were used on both sides of the tab (except Test 2-C).
10) Short slotted holes were perpendicular to the length of the tab when used.
11) All tabs had 3-in. pitch and 1½-in. edge distances.

12) There was a 3-in. spacing between flange tips and bolt line.
13) Weld thickness was 5/16-in. on both sides of tab in the unstiffened tests and it was 3/16-in. on both sides of tab in the stiffened tests

Table 4.1 lists the test variables for the unstiffened tests.

Table 4.1 - Test variables for unstiffened tabs

Test	Support Member	Tab t (in.)	Weld Size (in.)	Web h/t_w	# of Bolts	Tab Length (in.)	Weld-Bolt Distance (in.)	Bracing
1-U	W14X53	3/8	5/16	30.8	3	9	6.85	NO
2-U	W24X55	3/8	5/16	54.6	5	15	6.30	NO
3-U	W8X31	3/8	5/16	22.2	3	9	6.86	NO
4-U	W14X90	1/2	5/16	25.9	5	15	10.04	NO

All the unstiffened tests used short slotted holes in the tab. The tab thickness was chosen to meet stability limit states under the anticipated maximum shear. If the extended shear tab is considered a short cantilevered beam with a narrow rectangular cross section and with a concentrated load applied to the centroid of the section at the location of the bolt holes (see Fig. 4.6), the limiting end load is given by [10]:

$$V_{cr} = \frac{0.669\sqrt{EG}\,t^3\,L\,\sqrt{1-0.63\,t/L}}{a^2} \qquad 4.7$$

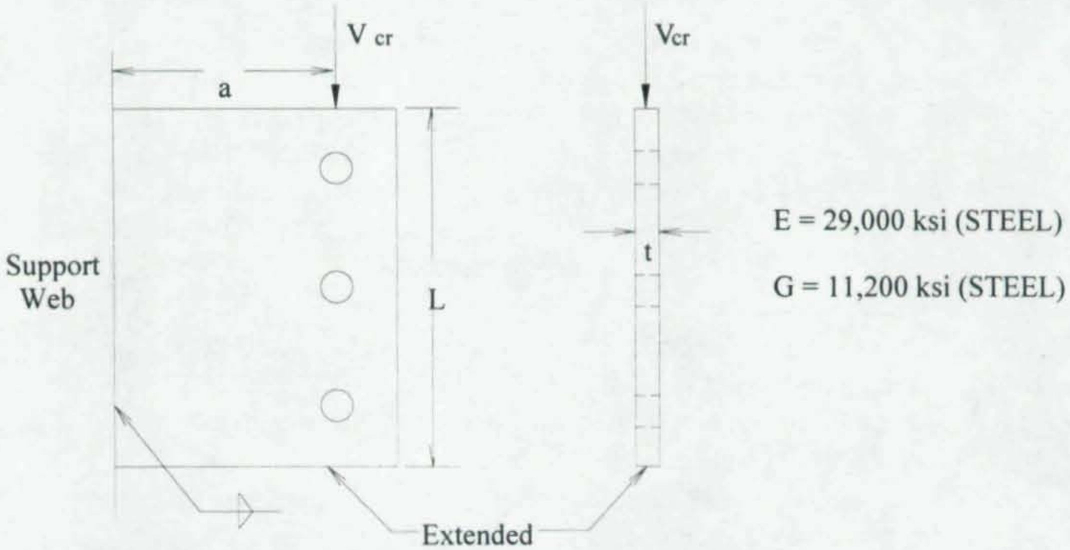


Figure 4.6: Schematic for tab thickness determination for the unstiffened tests

For common proportions of a shear tab, the thickness to length ratio (t/L) found in Equation 4.7 is very small and the second radical term is approximately unity. The equation can be solved approximately for the required thickness when the applied shear is known:

$$t \geq (Va^2/12000L)^{1/3} \qquad 4.8$$

A sample calculation of shear tab thickness determination for unstiffened tabs can be found in Appendix C. In addition, the tab thickness should meet the stability criteria in the AISC Manual [1] for single plate shear connections:

$$t \geq L/64 \geq 1/4\text{-in.} \qquad 4.9$$

AISC recommended weld sizes of approximately ¾ t were used for the unstiffened tabs of this study to ensure that weld failures did not precede shear yielding of the plate's gross section.

Table 4.2 lists the test variables for the stiffened tab tests.

Table 4.2 - Test variables for stiffened tabs

Test	Support Member	# of Bolts	Bolt Hole Type	Tab Length (in.)	Weld Configuration*	Weld c.g. To Bolt Distance (in.)	Bracing
1-A	W14X53	3	STD	9	W-T	6.50	NO
1-B	W14X53	3	SSL	9	W-T	6.50	YES
2-A	W24X55	5	STD	15	W-T	5.98	NO
2-B	W24X55	5	SSL	15	W-T	5.98	YES
2-C	W24X55	5	STD	15	W(3/8-in.,one side)-T	5.98	NO
3-A	W8X31	3	STD	9	W-T-B	5.91	NO
3-B	W8X31	3	SSL	9	W-T-B	5.91	NO
3-C	W8X31	3	STD	9	T-B	5.91	NO
3-D	W8X31	3	STD	9	W-T-B**	5.91	NO
3-E	W8X31	3	STD	19	W-T-B**	6.23	NO
4-A	W14X90	5	STD	15	W-T-B	8.25	NO
4-B	W14X90	5	SSL	15	W-T-B	8.25	YES
4-C	W14X90	5	STD	15	W-T-B	8.25	YES

* Weld Configurations: W = Web; T = Top; B = Bottom

** Stiffening plates welded to column web

For the stiffened tests, the weld to bolt line distance was taken from the center of gravity (c.g.) of the weld to the bolt line. The tab thicknesses for the stiffened tab tests only needed to meet the AISC stability criteria of Equation 4.9 and not the cantilever model criteria of Equation 4.8 because the distance to the bolt line from the end of the horizontal weld was only 3-in. Therefore, all stiffened tabs were 1/4-in. thick and all weld sizes were 3/16-in.

Each group in the stiffened tab tests had two base tests (designated by the A and B suffixes in Table 4.2). Those tests consisted of the standard connection details. Tests "A" were with standard holes and tests "B" were with short-slotted holes. Additional variations on these base tests were performed in tests designated with C, D, and E suffixes in Table 4.2. In Test 2-C, the fillet weld to the column web was doubled in size and placed on only one side of the tab. In Test 3-C, no weld was used between the tab and column web, the tab was welded only to the top and bottom stiffening plates. Tests 3-D and 3-E had the stiffening plates welded to the column web analogous to a case where the continuity plates (or stiffeners) could be used for orthogonal framing. Test 3-E also incorporated an extra long tab (19-in. compared to 9-in.) for the case where the continuity plate spacing is considerably larger than required for the 9-in. tab length.

The unstiffened tab test results indicated that there was some twisting of the extended shear tabs. Therefore, lateral bracing elements at two positions along the length of the supported beams were provided for tests 1-B, 2-B, 4-B, and 4-C, see Figure 4.7. The first bracing element was positioned at 42 inches from the connection bolt line for each beam. The bracing elements were designed to provide lateral restraining for both the top and bottom flanges of the supported beams. This in-effect prevented the twisting of the section at the positions of the bracing elements. The bracing elements did not inhibit the beams from undergoing vertical displacement. It must be noted that in real structures, a lateral restraining effect from the deck is normally present for only the top flange of supported beams. No significant restraining effect against twisting of the section is normally provided by the deck. Photographs of the bracing system used in this study are shown in Appendix B, Figures B.6 and B.7.

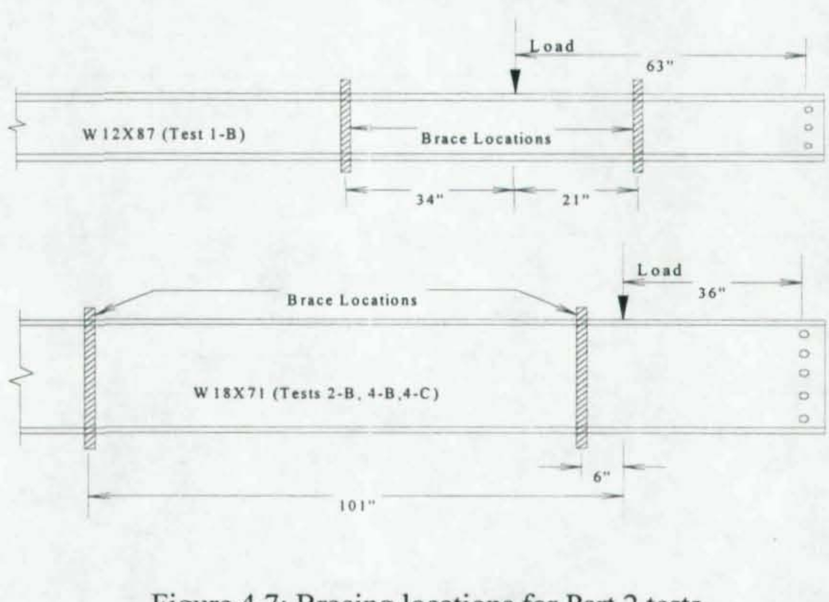


Figure 4.7: Bracing locations for Part 2 tests

4.4 Instrumentation

A variety of instrumentation to measure load, distortions and strains were used.

4.4.1 Load Cells

Load cells were used to measure the applied load and the far end reaction of the supported beam. The difference between the two is the shear force in the connection. Experimental setups for the load cells are shown in Figures 4.3 and 4.4 (a photograph can be found in Appendix B, Figure B.8).

4.4.2 LVDTs

Two LVDT's (Linear Voltage Displacement Transducers) were used to measure the vertical displacement along the edge of the top flange at the end of the supported beam relative to the supporting member. The LVDT measurements were used to also obtain the twist at the end of the supported beam. The average of readings of the two LVDTs at each load increment was used to obtain the connection displacement and the difference between the readings of the two LVDTs divided by the spacing between them gave the connection twist. The LVDTs used for this study were Sensotec model VL7A (AC/AC, long stroke, spring return). A signal conditioner was used in conjunction with the LVDTs to transform the output signal into direct current (DC) for processing by the data acquisition unit. The LVDTs were placed 1-in. off of both edges of the supported beam (see Fig. 4.8 for LVDT locations and Appendix B for photographs, Figures B.9 and B.10).

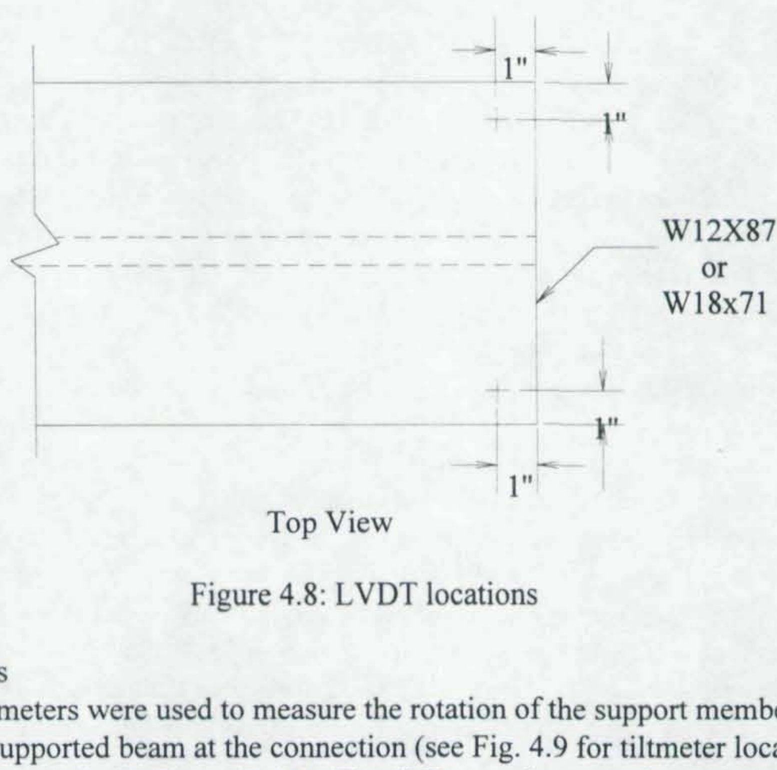


Figure 4.8: LVDT locations

4.4.3 Tiltmeters

Two tiltmeters were used to measure the rotation of the support member and the rotation of the supported beam at the connection (see Fig. 4.9 for tiltmeter locations and photograph in Appendix B, Figure B.11). The difference between the two results in the rotation of the connection. The tiltmeters used were Applied Geomechanics model 801 Uniaxial tiltmeters. For the column tests, one tiltmeter was mounted on the centerline of the column flange in the same vertical position as the tiltmeter mounted on the test beam. For the girder tests, the tiltmeter was mounted onto a steel plate, located in the same plane as the tab, that was tack welded to the back of the girder web. It also was placed on a vertical level with the tiltmeter mounted on the test beam.

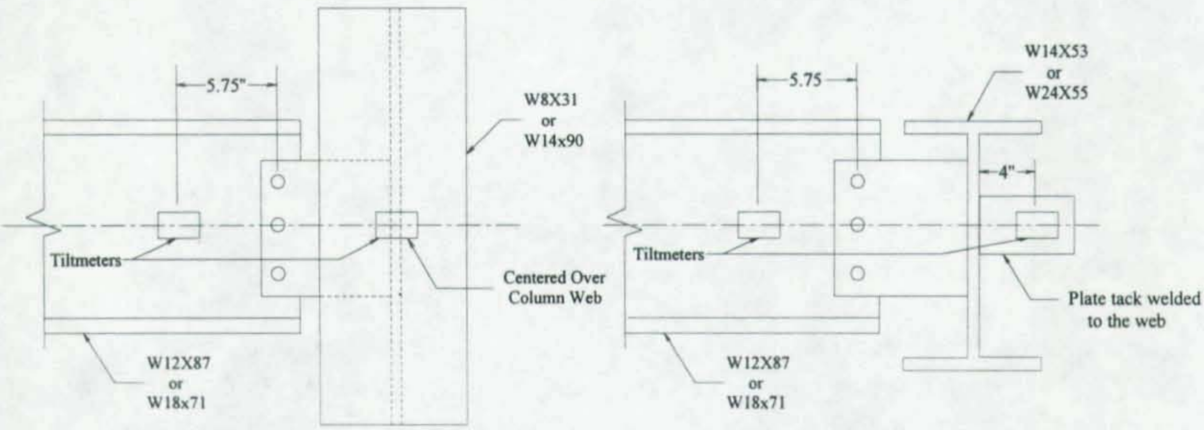


Figure 4.9: Location of tiltmeters

4.4.4 Strain Gages

For all of the tests, strain gages were installed on the test beams, shear tabs, and support members. These strain gages were used to monitor material behavior during testing, determine the onset of yielding of the shear tab and support member web, and to determine the location of the reaction shear. Three pairs of strain gages were mounted on the top and bottom flanges of the supported beams at three locations between the bolt line and the applied load (see Fig. 4.2 for strain gage locations for both test beams). Each pair of gages produce strains proportionate to the bending moment at that location. A linear regression analysis of the data collected from the three pairs of gages establishes the moment gradient and the eccentricity of the shear force from the bolt line or weld center of gravity for each test beam.

Strain gages were also mounted on the shear tab and support member webs. Figure 4.10 shows the locations of the strain gages. In this figure, only the shear tab and support members are shown but not weld configurations because the weld configurations varied for all of the tests. Tab dimensions are shown as variables and some strain gage locations are indicated in terms of these tab dimensions. Strain gages F and G are on the back of the support member web and are horizontal. Table 4.3 identifies gages that were used with each test. The gages designated H, I, J, K are in the same locations as gages A, B, C, D, respectively, but are on the opposite side of the plate. Appendix B (Figure B.13) includes a photograph showing strain gage locations.

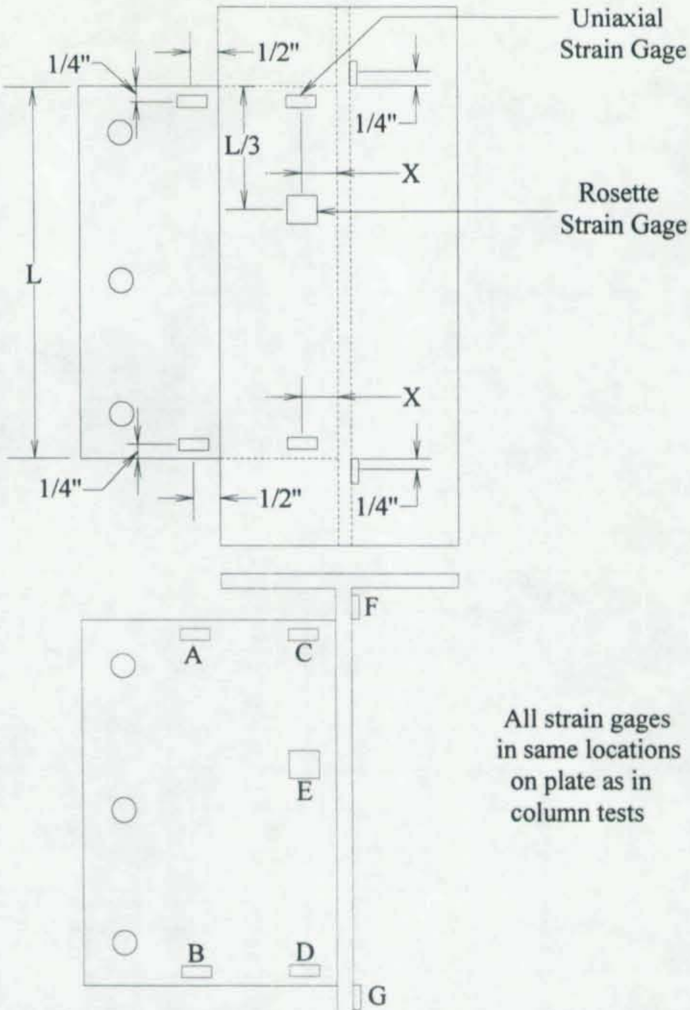


Figure 4.10: Strain gage locations on shear tabs

4.4.5 Whitewash

All of the tests were terminated when there was appreciable yielding of the connection or instrumentation became endangered of being damaged. In order to monitor the onset and spread of yielding during the tests, a whitewash coating (comprised of hydrated lime and water) was applied to the connection, the support member and test beam. Also, the whitewash allowed for visual monitoring of bolt slip and plate distortion. Appendix B includes a photograph of a typical whitewash coating (Fig. B12).

4.4.6 Data Acquisition

The data acquisition unit used for the testing was a Hewlett Packard model 34970A. For each of the four types of sensors used (load cells, strain gages, tiltmeters, LVDTs), a scaling factor was entered into the device so that the equipment could process the input signal from the sensor and display the output in the appropriate engineering units (lb., μE, degree, in.). All of the sensors used in this study, including those with known scaling factors provided by the manufacturer, were calibrated in the laboratory to either verify or arrive at the proper scaling factor.

Table 4.3: Strain gages for all tests

Test	L (in.)	L/3 (in.)	X (in.)	Strain Gages
1-U	9	3	1.5	C,D,J,K,E*,F,G
2-U	15	5	1.5	C,D,J,K,E,F,G
3-U	9	3	1.5	C,D,J,K,E
4-U	15	5	1.5	C,D,J,K,E,F,G
1-A	9	3	1.0	A,B,H,I,K,G
1-B	9	3	1.0	A,B,H,I,K,G
2-A	15	5	1.0	A,B,H,I,K,G
2-B	15	5	1.0	A,B,H,I,K,G
2-C	15	5	1.0	A,B,H,I,K,G
3-A	9	3	1.0	A,B,H,I,J,K,F,G
3-B	9	3	1.0	A,B,H,I,J,K,F,G
3-C	9	3	1.0	A,B,H,I,C,D,F,G
3-D	9	3	1.0	A,B,H,I,J,K,F,G
3-E**	9	3	1.0	A,B,H,I,E*
4-A	15	5	1.0	A,B,H,I
4-B	15	5	1.0	A,B,H,I,E,G
4-C	15	5	1.0	A,B,H,I,E

* Biaxial strain gage used instead of rosette

** Test 3-E was the test with the 19-in. long shear tab so two additional strain gages were installed to monitor the behavior of the plate during testing. One was a rosette strain gage installed near the bottom plate size transition and the other was a biaxial strain gage installed 5 in. from the bottom of the deep shear tab.

The HP data acquisition unit offered a graphical interface in which the output of two of the sensors could be plotted against each other during testing. For instance, load could be plotted against connection displacement. This was a very helpful tool during testing because the onset of yielding of the connection could be monitored in terms of the load-displacement curve. If the curve leveled off considerably in between two successive load increments, the conclusion was that the connection had yielded and the test could be terminated. A permanent record of all test data was made after completion of each test.

4.5 Test Procedure

The following test procedure was used for all 17 tests performed:

1) Three pairs of strain gages were installed on the test beam.
2) Strain gages were installed on the test specimen according to Table 4.3.

3) The support member was properly secured either to the reaction wall or laboratory test floor and the test beam was brought into place, bolted, and leveled.
4) The remaining sensors (LVDTs, tiltmeters, and load cells) were attached to the test setup. This process included making sure the loading apparatus and LVDTs were exactly vertical.
5) All of the sensors were connected to the data acquisition unit and their initial reading was zeroed out. This was done so that beam self weight and other factors, such as bolt tightening, did not influence test results.
6) The test was initiated by activating the data acquisition unit that began to scan, record, and display the sensor measurements on a computer screen. The data acquisition unit scanned all of the sensors approximately every 4 seconds.
7) Loading commenced. Loading was controlled by a hydraulic pump and the rate of loading was approximately 5000 lb./minute.
8) Loading was paused every 5000 lb. for the 3-bolt connection tests and every 10000 lb. for the 5-bolt connection tests. This pause allowed for multiple measurements to be taken at the same load and for visual inspection of the connection.
9) Loading was terminated when the connection could not sustain increasing load or when appreciable yielding occurred.
10) A permanent record of all test data was created.
11) All of the sensors were removed and the test setup was disassembled.
12) The support member, shear tab, and bolts were visually inspected. This included checking for bolt shear, bolt bearing in bolt-holes, flaking of whitewash coating.

5. TEST RESULTS

5.1 Material Properties

Tensile properties of the shear tabs were obtained from plate material of the same stock as the test specimens. Tensile properties of the support member webs were obtained from coupons cut from the member after each test was completed. These tensile properties were determined according to the ASTM 370 specifications and are used in the calculation of limit states. The material properties are summarized in Table 5.1.

Table 5.1: Material properties

Member	Thickness (in.)	Yield Strength (ksi)	Ultimate Strength (ksi)	% Elongation
Unstiffened Tests				
3/8-in. TAB	0.371	42.6	66.5	34
1/2-in. TAB	0.506	40.5	63.6	36
W14X53 WEB	0.370	54.2	70.8	38
W24X55 WEB	0.392	55.1	70.1	38
W8X31 WEB	0.288	55.2	75.3	31
W14X90 WEB	0.468	56.7	71.7	37
Stiffened Tests				
1/4-in. TAB (W8, W24 TESTS)	0.246	44.4	72.3	33
1/4-in. TAB (W14 TESTS)	0.247	45.7	69.8	30
W14X53 WEB	0.363	55.5	73.8	29
W24X55 WEB	0.382	59.2	73.6	30
W8X31 WEB	0.276	55.7	73.6	28
W14X90 WEB	0.473	55.5	70.8	30

For the stiffened tests, the 1/4-in. shear tabs came from two separate stocks. The W8X31 tests and W24X55 tests used the same material and the W14X53 and W14X90 tests used a separate material. Also for the stiffened tests, the values for the material properties reported in the table are an average for all of the tests in that group. Group 1 consisted of two tests, Group 2 of three tests, Group 3 of five tests and Group 4 of three tests. For each group, material properties varied by less than 4% so the average was reported for each group and the average was used for calculation of limit states for each test in each group.

5.2 Shear-Displacement, Shear-Twist, Shear-Rotation Results

For each test, a graph was constructed for shear vs. displacement, shear vs. twist and shear vs. rotation, where shear is defined as the shear force in the connection, taken as the difference between the applied load and the far end reaction of the beam.

5.2.1 Shear-Displacement Results

To identify the point at which connection behavior became nonlinear and to determine ultimate shear forces, a graph of shear vs. displacement was constructed for all tests. In each graph, the vertical axis is the shear force in the connection and the horizontal axis is the vertical displacement of the connection. An example of this type of graph is shown in Figure 5.1 for Test 3-B. The shear-displacement graphs for all tests can be found in Appendix D.

5.2.2 Shear-Twist Results

To identify the point at which connection behavior became nonlinear and to determine ultimate shear force in the connection, a graph of shear vs. twist was constructed for all tests. In each graph, the vertical axis is the shear force in the connection and the horizontal axis is the twist of the connection. An example of this type of graph is shown in Figure 5.2 for Test 3-B. Graphs for all tests can be found in Appendix E.

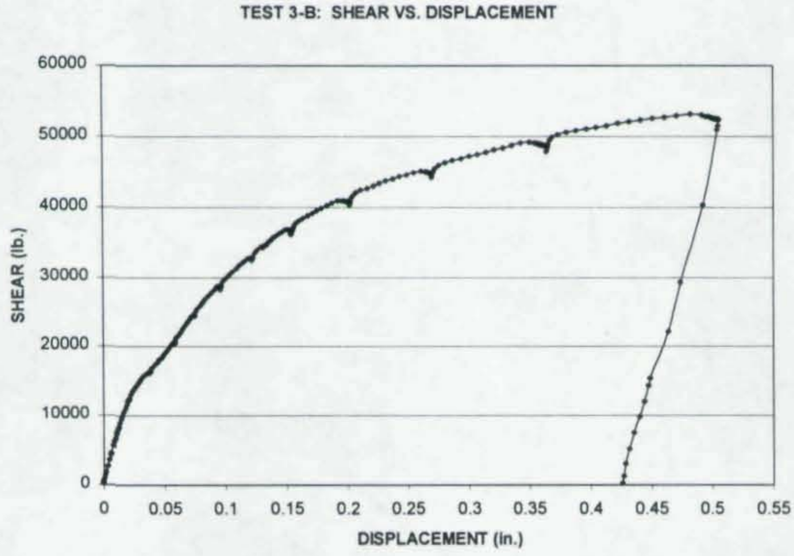


Figure 5.1: Shear vs. Displacement

5.2.3 Shear-Rotation Results

Shear vs. Rotation graphs were also constructed for each test. The rotation was measured directly with the use of tiltmeters, which were mounted on the supported beam near the connection and on the support member. An example of this type of graph is shown in Figure 5.3 for Test 3-B. Graphs for all tests can be found in Appendix F.

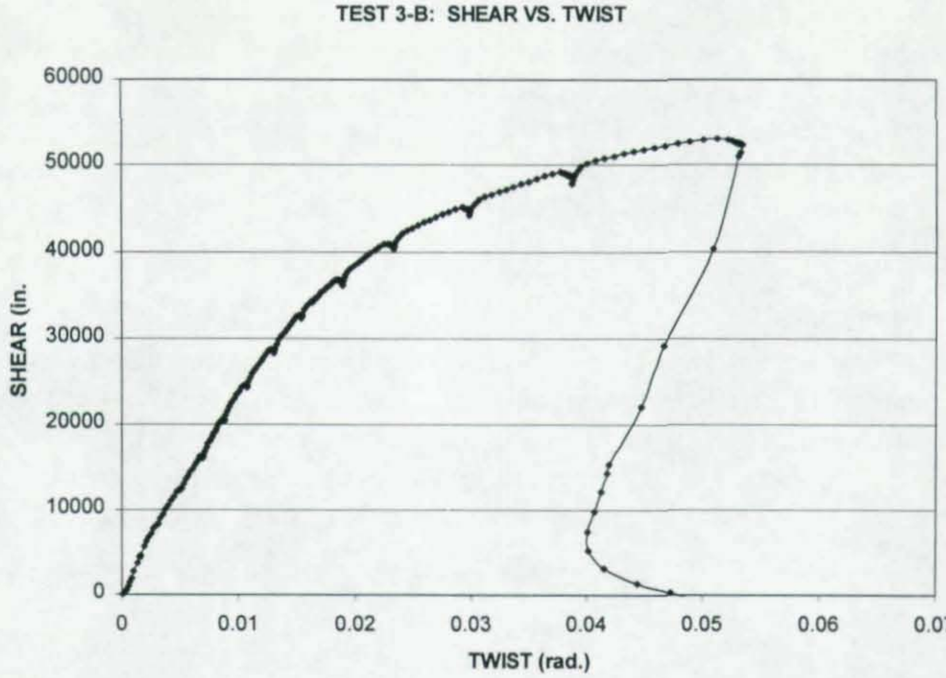


Figure 5.2: Shear vs. Twist

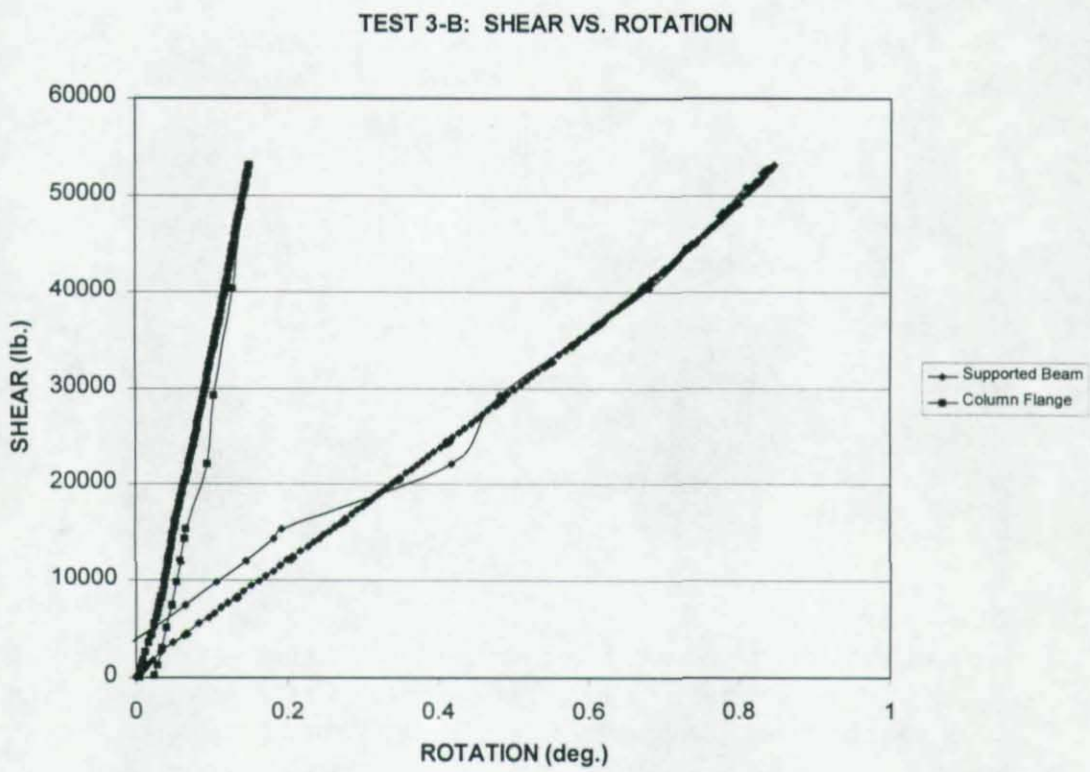


Figure 5.3: Shear vs. Rotation

Graphs of shear vs. displacement, shear vs. twist, and shear vs. rotation were important in determining ultimate shear capacities, point of global connection yielding and failure modes. For all tests, either the shear-deflection or shear-twist curves approached a level condition, indicating that failure was imminent. The shear value at which either the shear-displacement or shear-twist curve approached a level condition was taken as the ultimate shear capacity of the connection. The shear-displacement and shear-twist curves also indicate a point at which the connection behavior became nonlinear. Beyond this point, the connection was still able to resist higher load but the respective curve had begun to level off. In all cases, this point of nonlinear behavior occurred well below the calculated yield load for the connection plate. This point does not represent a point of imminent failure, but rather a system connection phenomenon in which several conditions exist simultaneously to produce the onset of nonlinear behavior. These conditions include shear distortion of the shear tab, twisting of the shear tab and yield line mechanism of the support member web. The shear-displacement, shear-twist, and shear-rotation curves were also important in helping to identify failure modes. For example, twisting of the connection plate was identified as the primary failure mode if a flattening of the shear-twist curve occurred before the shear displacement curve had leveled off. However, if the shear-displacement curve leveled off before the shear-twist curve, shear yield of the shear tab was identified as the primary failure mode. If the shear-rotation curves indicated large values for rotation of the support member web then a yield line mechanism of the support member web identified as the primary failure mode.

5.3 Eccentricity, Ultimate Shear Forces, Failure Modes

The three most important test results were the values of the shear force eccentricity, the ultimate shear force in the connection, and the failure mode of the connection. These items are important in developing a connection design procedure.

5.3.1 Shear Force Eccentricity

Measured eccentricities were determined from the three pairs of strain gages mounted on the supported beam as shown in Figure 5.4.. A linear regression analysis was used to determine the point of zero strain, or zero moment, at each load increment applied to the connection. Although the method used is an extrapolation of collected data, and therefore not precise, the results of the method agree well with observed failure modes. The sign convention used for the shear force eccentricity can be seen schematically in Figure 5.4. The reference line is taken as the bolt line of the connection.

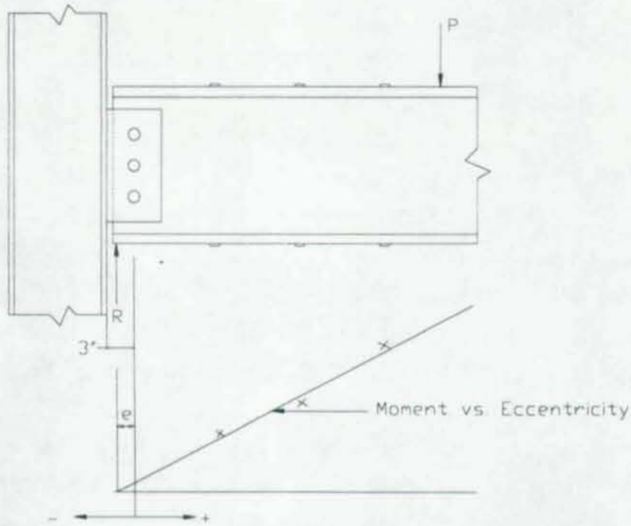


Figure 5.4: Sign convention for shear force eccentricity

A graph for each test was drawn in which connection shear was plotted versus shear force eccentricity. Figure 5.5 shows this graph for Test 1-B.

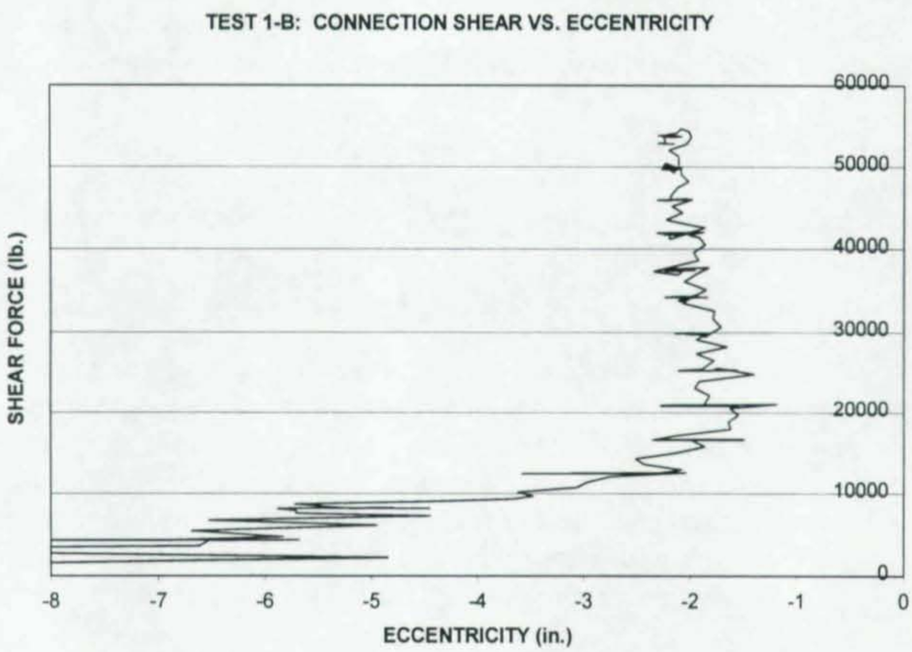


Figure 5.5: Shear Force vs. Eccentricity (All Data)

The graph shows a shift in the values of the eccentricity at different load levels when the loading was paused for test observation. If the shear and eccentricity values are taken only at the points when the loading was first paused, a more obvious correlation between connection shear and eccentricity could be observed. A graph of this (for Test 1-B) is shown in Figure 5.6. Graphs for all of the tests can be found in Appendix G.

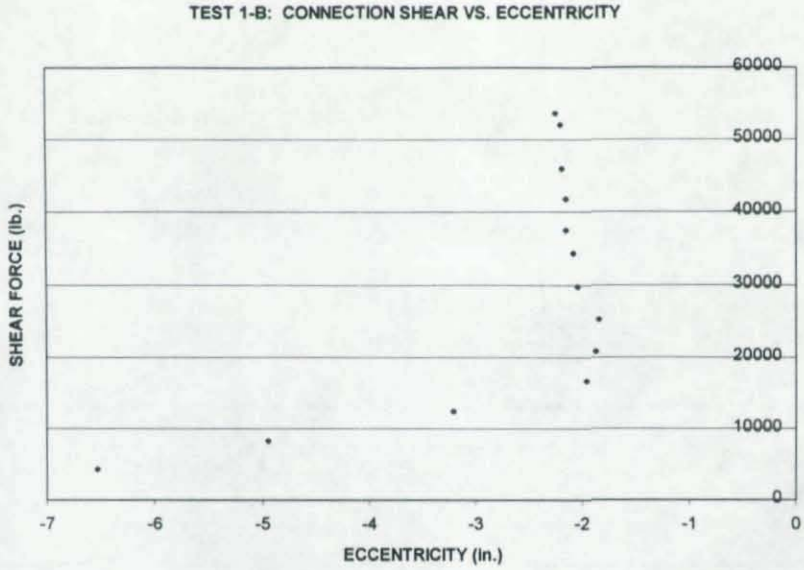


Figure 5.6: Shear Force vs. Eccentricity (5000 lb. load increments)

The bolt line eccentricity, e_b, is taken as the absolute value of the eccentricities from the graph of Figure 5.6. This eccentricity can be found at every shear value. The weld line eccentricity, e_w, is found by taking the distance between the weld center of gravity and the bolt line and subtracting the value of e_b. The moment experienced by either the bolt group or the weld can be found by taking the reaction shear and multiplying it by the respective eccentricity.

Figures 5.5 and 5.6 show that the measured eccentricity varies with shear. The relatively large shift in the eccentricity at lower shear values is due to the expected slip that occurs in the connection. It must be noted that the eccentricity that is important for evaluating limit states is the eccentricity that is approached as the shear reaches its ultimate value. As the figures show, the eccentricity tends towards a constant value as the ultimate shear is reached. This value approached at ultimate shear is the experimental eccentricity, e_{exp}.

The shear eccentricity relative to the bolt line and welds is important in evaluating several of the limit states relevant to shear tab connections. The AISC Manual [1] contains eccentricity equations relative to the bolt line for flexible and rigid supports with standard and short slotted holes. They are:

Rigid - Standard: $e = |(n-1) - a|$ 5.1

Rigid - Slotted: $e = |2n/3 - a|$ 5.2

Flexible - Standard: $e = |(n-1) - 1| \geq a$ 5.3

Flexible - Slotted: $e = |2n/3 - a| \geq a$ 5.4

In these equations: e = eccentricity to the bolt line
n = number of bolts in the connection
a = space between weld line and bolt line

Table 5.2 shows the experimental results for the eccentricity relative to the bolt line. Included in this table, for reference, are the AISC eccentricities found from Equations 5.1 through 5.4. All of the experimental eccentricities were determined from graphs similar to the one shown in Figure 5.6. These graphs are found in Appendix G.

Table 5.2: Shear eccentricities relative to the bolt line

			AISC			
Test	Support Member	e_{exp} (in.)	Rigid-STD (in.)	Rigid-SSL (in.)	Flexible-STD (in.)	Flexible-SSL (in.)
Unstiffened						
1-U (SSL)	W14X53	-3.2		**-4.85**		-6.85
2-U (SSL)	W24X55	-5.8		-2.97		**-6.30**
3-U (SSL)	W8X31	-3.3		**-4.86**		-6.86
4-U (SSL)	W14X90	-6.5		**-6.71**		-10.04
Stiffened						
1-A (STD)	W14X53	-2.8	**-4.50**		-6.50	
1-B (SSL)	W14X53	-2.2		**-4.50**		-6.50
2-A (STD)	W24X55	-4.3	-1.98		**-5.98**	
2-B (SSL)	W24X55	-5.0		-2.65		**-5.98**
2-C (STD)	W24X55	-4.9	-1.98		**-5.98**	
3-A (STD)	W8X31	-0.3	**-3.91**		-5.91	
3-B (SSL)	W8X31	-0.2		**-3.91**		-5.91
3-C (STD)	W8X31	2.6	---		---	
3-D (STD)	W8X31	0.5	**-3.91**		-5.91	
3-E (STD)	W8X31	0.4	**-4.73**		-5.91	
4-A (STD)	W14X90	-1.7	**-4.25**		-8.25	
4-B (SSL)	W14X90	-0.3		**-4.92**		-8.25
4-C (STD)	W14X90	-1.5	**-4.25**		-8.25	

Table 5.2 uses the same sign convention as Figure 5.4. To determine the AISC eccentricities, it is necessary to know the "a" distance in Equations 5.1 to 5.4. For the unstiffened tests, it is taken as the distance from the vertical weld to the bolt line. For the stiffened tests, it is taken as the distance from the weld center of gravity to the bolt line. No AISC eccentricities are given for test 3-C because AISC does not include any design procedures for shear tabs without vertical welds.

It is not the focus of this study to address the design of supporting members in shear tab connections. However, it is apparent that the resulting shear and moment forces from the connections do affect the design of the supporting members and will need to be considered.

5.3.1.1 Unstiffened Tests

The items shown in bold face in Table 5.2 represent the AISC eccentricity that best approximates the measured eccentricity, e_{exp}. As Table 5.2 indicates, neither the AISC rigid support nor AISC flexible support eccentricity equations provides a very good correlation with e_{exp}. The e_{exp} for Tests 1-U, 3-U, and 4-U seem to correlate better with the AISC rigid support eccentricity equation while e_{exp} for Test 2-U correlates better with the AISC flexible support eccentricity equation. This could be due to greater flexibility of the web of the W24X55 girder (h/t_w = 54.6) compared with that of the W14X53 (h/t_w = 30.8). For all of the unstiffened tests, e_{exp} is smaller than the AISC eccentricity values. This would result in a less conservative design for the weld if one were to follow the AISC specifications.

5.3.1.2 Stiffened Tests

The stiffened tests exhibited a similar trend as the unstiffened tests. The e_{exp} for Group 2 tests, with the more flexible support web, all seemed to correlate better with the AISC flexible support eccentricity values while Groups 1,3, and 4 correlated better with the AISC rigid support eccentricity values. Again, the measured eccentricities were smaller than those from the AISC eccentricity equations.

The measured eccentricity values are nearly identical for the two different types of bolt holes, which is evident from the results of the sub-group A and sub-group B tests. For 3-bolt connections (Group 1 and Group 3), AISC specifications predict no difference in eccentricity between STD and SSL holes. For 5-bolt connections, the specifications predict the same eccentricity values for STD and SSL holes for flexible supports. For rigid supports, the specifications predict larger eccentricities for SSL holes than STD holes. Group 2 results seem to support this assumption with a –5.0 in. eccentricity for STD holes and a –4.3 in. eccentricity for STD holes. However, Group 4 results indicate the opposite, with a –0.3 in. eccentricity for SSL holes and a –1.7 in. eccentricity for STD holes.

5.3.2 Ultimate Shear Forces and Failure Modes

The ultimate shear forces for the tests, V_{exp}, are shown in Table 5.3. This shear force corresponds to that experienced by the connection when a test was terminated. The tests were terminated when the shear distortion (either shear-displacement or shear-twist) curves leveled off or when the connection was undergoing appreciable yielding without maintaining additional load. This table also includes the failure modes for each test. These failure modes were identified either visually or with the aid of shear-displacement, shear-twist, and shear-rotation graphs. The experimental failure mode identification process was subjective, however, due to the fact that several conditions usually existed simultaneously. The table lists primary failure modes that are followed by secondary failure modes in parentheses.

Table 4-2 of the AISC Hollow Structural Sections (HSS) Connections Manual [8] lists the standard limit states for various connections, including the single plate connections. For the single plate connections in this study, they are:

1) Bolt shear by ultimate analysis (includes the effects of eccentricity)
2) Bolt bearing (in the tab - based on the bolt shear analysis)

3) Gross shear at yield (of the tab)
4) Net section shear rupture (of the tab through the bolt line)
5) Block shear rupture (of the tab)
6) Weld shear by ultimate analysis (includes the effects of eccentricity)

Table 5.3: Ultimate shear forces, failure modes, and limit states

	Experimental		AISC Critical				AISC Typical
			V_{th}		V_e		V_3
Test	V_{exp} (kips)	Failure* Mode	(AISC e) (kips)	Limit State	(Exp. e) (kips)	Limit State	(3-in. e) (kips)
Unstiffened							
1-U	58.7	B (A,D)	36.1	A	45.6	A	72.6
2-U	89.3	F (A,B,E)	61.7	A	70.2	A	142.0
3-U	54.8	E (A)	31.5	A	41.0	E	72.6
4-U	98.7	F (A,E)	80.9	A	64.3	A	144.0
Stiffened							
1-A	58.3	C (F,B)	34.1	A	49.8	A	61.0
1-B	54.6	C (F,B)	34.1	A	56.7	A	61.0
2-A	89.0	C (F,B)	68.6	A	85.5	A	98.3
2-B	92.6	C (F,B)	68.6	A	77.9	A	98.3
2-C	83.3	C (F,B)	68.6	A	78.9	A	---
3-A	53.2	C,F	38.9	A	59.0	C	59.0
3-B	53.1	C,F	38.9	A	59.0	C	59.0
3-C	22.1	C,F	---	---	---	---	---
3-D	51.1	C,F	38.9	A	59.0	C	59.0
3-E	48.1	C,F	32.4	A	59.0	C	---
4-A	103.0	C,F	86.1	A	102.0	C	98.3
4-B	107.0	C,F	78.7	A	102.0	C	98.3
4-C	107.0	C,F	86.1	A	102.0	C	98.3

* Limit States: A = bolt shear B = bolt bearing C = shear yield
D = shear rupture E = web mechanism F = twist

Table 4-2 of the HSS Manual also provides provisions for beam web limit states. They are not considered here because they generally do not control. Table 4-2 of the HSS Manual also provides, in addition to the strength limit states, two limitations on the tab thickness. They are:

t_{max} $d_b/2 + 1/16$-in. (Ensures Rotational Ductility)
t_{min} $L/64$ $1/4$-in. (Prevents Local Buckling)

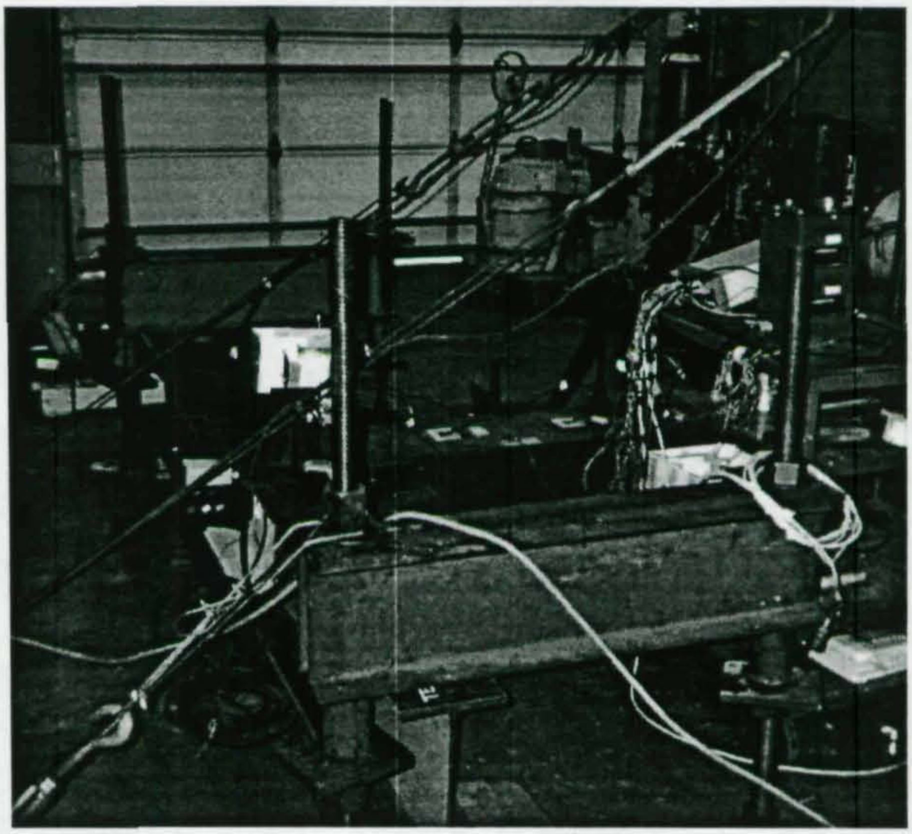


Figure B.5: Typical girder tie-downs (Test 1-B)

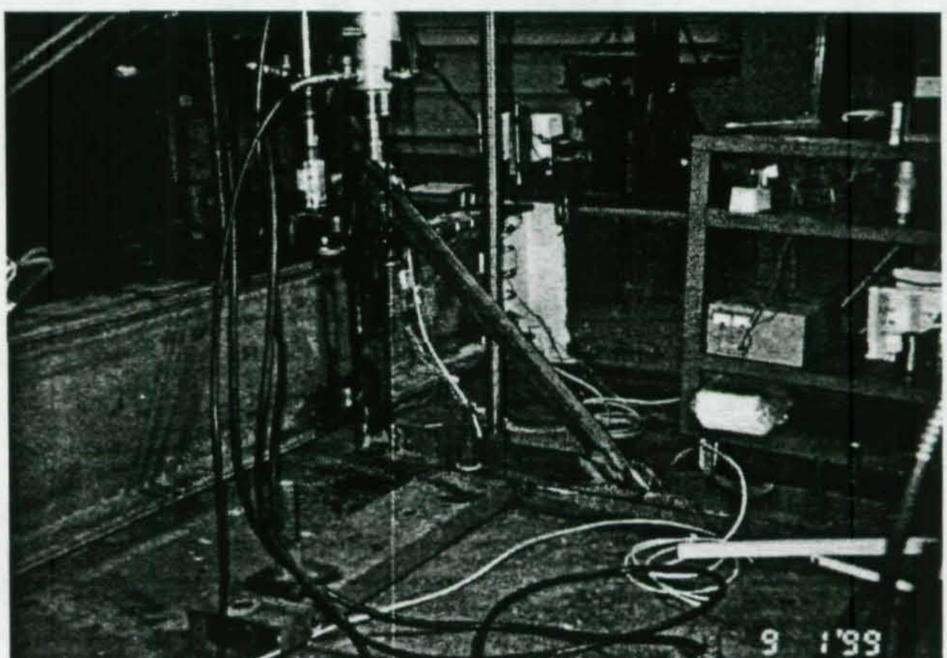


Figure B.6: Bracing near the point of loading (Test 2-B)

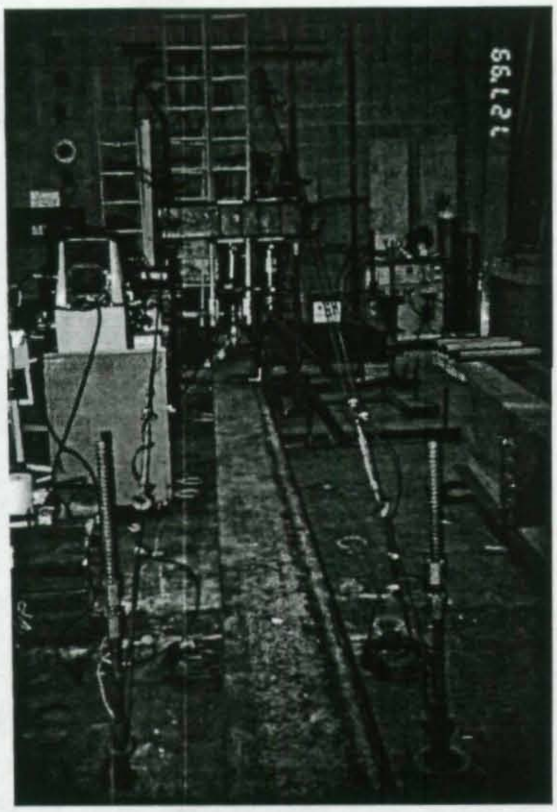


Figure B.3: Typical test setup for girder test (Test 1-B)

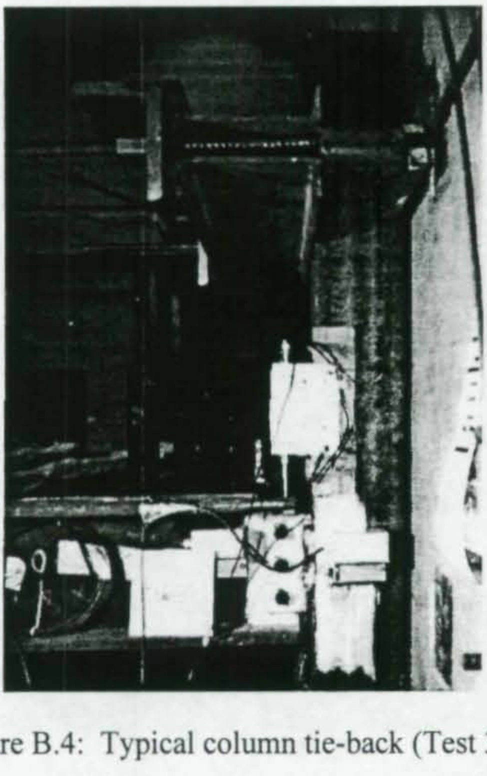


Figure B.4: Typical column tie-back (Test 3-B)

APPENDIX B: PHOTOGRAPHS OF TEST SETUP

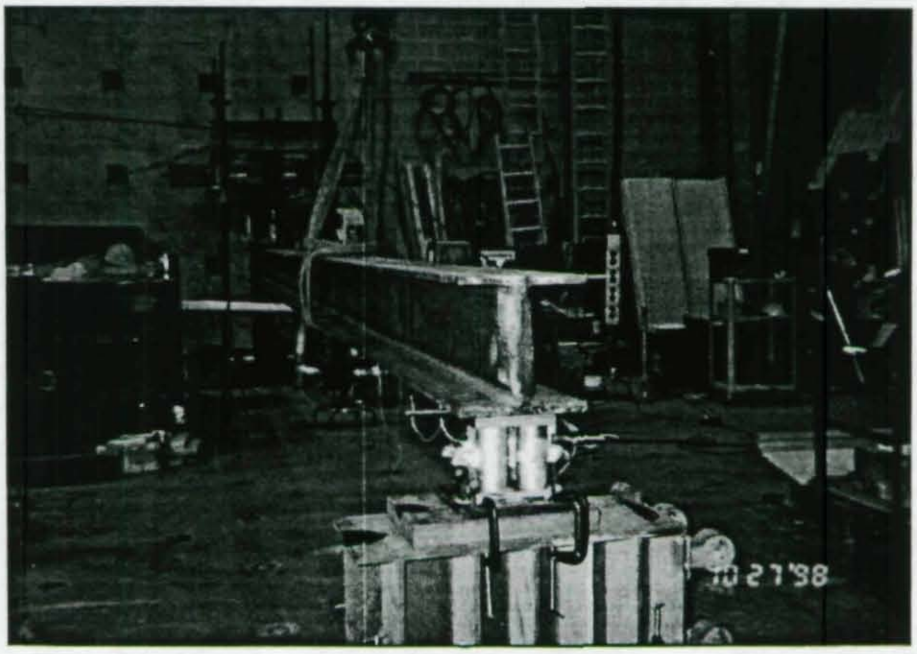


Figure B.1: Typical far end reaction for column test (Test 3-U)

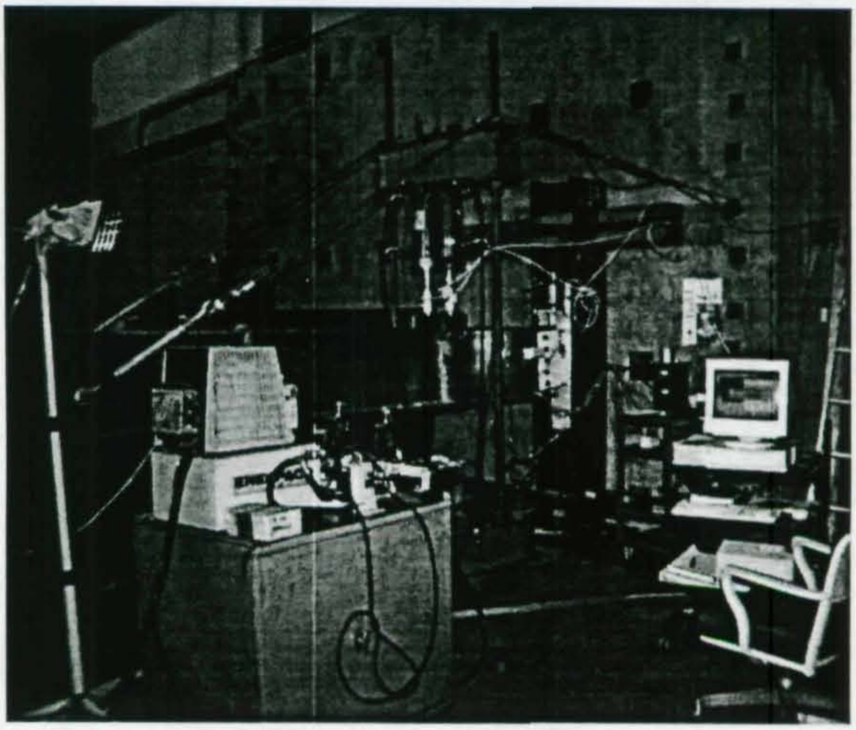


Figure B.2: Typical test setup for column test (Test 4-A)

For a uniformly loaded beam: $R = 1/2wL_u$ or $w = 2R/L_u$

$M_{max} = w(L_u)^2/8$ or $L_u = M_{max}(8)/2R = 381(8)/2(100) = 15.3$ ft

Use $L_u = 15$ ft

Test Beam: $L_p = 20$ ft

$b = (L_p^2 - (L_u^2/2))^{1/2} = (20^2 - (15^2/2))^{1/2} = 16.96$ ft

Use a = 3ft = 36 in.

Summary: The 20-ft. long test beam with a concentrated load placed 36 in. from the shear tab exactly simulates the same reaction shear and end rotation as a 15 ft long uniformly loaded beam.

APPENDIX A: TEST BEAM SELECTION

High Rotation Beam:

Beam Size: W12X87

Properties: $S_x = 118 \text{ in}^3$

$F_y = 36$ ksi

$R_{max} = 60$ kips

Calculations: $M_{max} = F_yS/12 = 36(118)/12 = 354$ ft-k

For a uniformly loaded beam: $R = 1/2wL_u$ or $w = 2R/L_u$

$M_{max} = w(L_u)^2/8$ or $L_u = M_{max}(8)/2R = 354(8)/2(60) = 23.6$ ft

Use $L_u = 24$ ft

Test Beam: $L_p = 30$ ft

$b = (L_p^2 - (L_u^2/2))^{1/2} = (30^2 - (24^2/2))^{1/2} = 24.75$ ft

Use a = 5.25 ft = 63 in.

Summary: The 30-ft. long test beam with a concentrated load placed 63 in. from the shear tab exactly simulates the same reaction shear and end rotation as a 24 ft long uniformly loaded beam.

Low Rotation Beam:

Beam Size: W18X71

Properties: $S_x = 127 \text{ in}^3$

$F_y = 36$ ksi

$R_{max} = 100$ kips

Calculations: $M_{max} = F_yS/12 = 36(127)/12 = 381$ ft-k

Appendices

shear tabs. As of now, it is recommended that the design procedure only be used with anywhere from 2-5 bolts in a single row. More research is recommended if the design procedure can be used for connections with more than 5 bolts and with snug tight bolts in SSL holes.

5) Using AISC criteria (AISC HSS Connections Manual [8]), determine the critical limit state based on:
   - bolt shear
   - bolt bearing in the tab
   - 90% of shear yielding on the gross area- reduce this value an additional 10% for elongated tabs on columns
   - shear rupture of the net section
   - block shear rupture

6) For unstiffened connections to columns webs, also determine the yield line mechanism by Equation 5.5.
7) Use a weld size equal to ¾ of the plate thickness.
8) Examine the beam web for bolt bearing.

Table 7.1 compares the design strengths, Vn, determined by the proposed design procedure with the unstiffened and the average of the A and B stiffened experimental strengths for each Group.

Table 7.1: Comparison of design shear strengths

Group	V_n (kips)	V_{exp} (kips)
Unstiffened		
Group 1	44	59
Group 2	66	89
Group 3	43	55
Group 4	78	99
Stiffened		
Group 1	45	56
Group 2	69	91
Group 3	53	53
Group 4	91	105

The design strengths are conservative with respect to the experimental capacities and are between the capacities calculated using AISC and measured eccentricities in Table 6.3. A sample calculation for the proposed design procedure can be found in Appendix K.

7.3 Conclusions

A design procedure for extended shear tabs has been developed. The design procedure is based on a series of 17 full-scale beam-to-column and beam-to-girder tests in which shear tabs, both stiffened and unstiffened, with various weld configurations to the supporting member web were performed. The design procedure is a modification of the existing AISC criteria for single plate shear connections. A yield line mechanism of the web of a supporting column was identified as a new limit state for unstiffened extended

7. DESIGN PROCEDURE AND CONCLUSIONS

7.1 Limits of Applicability

Since the design procedure is based on the experimental database, extrapolation beyond the test parameters may not be justified. The limits of applicability for the design procedure are as follows:

1) The number of bolts should be from 2 to 5. Since 5-bolt connections were the largest tested, extrapolation to connections with more than 5 bolts is outside the range of the database, and caution is required.
2) A vertical weld to the web of the supporting member is required and all welds should be in pairs on both sides of the tab.
3) The bolt line should be 3-in. beyond the flange tip of the supporting member.
4) Either STD or SSL holes may be used. Bolts should be fully tightened in SSL holes but may be snug tight in STD holes.
5) Stiffeners between column flanges do not have to be welded to the column web, but they may be if the stiffener also serves as a continuity plate.
6) Any elongated length of the tab between the column flanges should not exceed twice the length of the tab at the connection.
7) Rigid body twist of the beam must be prevented by bracing or floor framing.

7.2 Design Procedure

For connections that meet the limits of applicability, the following design procedure may be used:

1) Estimate the number of bolts required for the shear, V, using the shear capacity (or slightly reduced capacity) for the type and size of bolt. Determine the length of the tab using 3-in. for bolt spacing and 1 ½ in. for edge distances.
2) Determine the distance *a* from the centroid of the weld group to the bolt line. The centroid is at the welds for unstiffened connections and Table 8-42 and 8-44 of the AISC Manual [1] are convenient for determining the weld centroid for stiffened connections.
3) Determine the tab thickness for stability. Use the larger thickness from Equations 4.8 and 4.9 for unstiffened tabs, where V is the required shear. For stiffened tabs, only Equation 4.9 applies.
4) Determine the eccentricity e of the shear force relative to the bolt line. It is permissible to use the AISC equations for eccentricity. In lieu of these, a less conservative but still satisfactory design may be obtained using:

   $e = a$ for h/tw of the supporting member > 35
   $e = 0.5a$ for unstiffened connections with $h/tw \leq 35$
   $e = 0.5a$ for stiffened connections to girders with $h/tw \leq 35$
   $e = 0.25a$ for stiffened connections to columns with $h/tw \leq 35$

tests. This would indicate that there is a contribution to the shear stresses from torsion but that the contribution is not as great for the stiffened tests as is it for the unstiffened tests, which is predicted due to the large distance between the weld and bolt line in the unstiffened tests compared to the stiffened tests.

As it is for coped beams, lateral-torsional buckling was considered as a limit state associated with twist [3]. However, torsional bracing of the beam near the connection had no effect on the ultimate experimental shear. For this reason, lateral-torsional buckling was not considered to be a limit state. In support of this assumption, load-deflection and load-twist curves (Figures 5.1 and 5.2) indicate that there was no drop in load as is indicative of a stability failure. However, it must be recognized that the shear tab, especially the extended shear tab, has very little torsional stiffness at high loads and rigid body rotation of the beam at the tab must be prevented. The history of using shear tabs in practice indicates that typical lateral bracing and floor framing near the connection are satisfactory for this purpose.

For the torsional limit state, failure is defined when the maximum shear stress reaches the shear yield of the material. The resulting equation for the limit state is:

$$V_t = 0.3LtF_y \qquad 6.1$$

The derivation of this equation can be found in Appendix J. The torsion limit state equation was derived by using a superposition of the shear stresses obtained from direct shear and from torsion of the shear tab. The torsion limit state, V_t, average termination of load linearity, V_1, and the average ultimate experimental strengths, V_{exp}, are shown in Table 6.2. Average values are given in this table for the stiffened tests because the torsion limit state is the same for all tests in a group and the shears at the termination of linearity and average experimental strengths are very close for all tests in a group.

Table 6.2: Shear at yield for torsion

Test	V_t (kips)	V_1 (kips)	V_{exp} (kips)
Unstiffened			
1-U	42.7	48	58.7
2-U	71.1	65	89.3
3-U	42.7	37	54.8
4-U	92.2	82	98.7
Stiffened			
Group 1	30.4	43	56
Group 2	49.2	65	89
Group 3	29.5	38	52
Group 4	50.8	85	106

6.2.1 Discussion on twisting

There appears to be a reasonable correlation between the termination of linear behavior and the derived torsion limit state for all of the unstiffened tests. Although twisting was listed as the primary failure mode for only the five bolt connections (Tests 2-U and 4-U) in Table 5.3, monitoring of strain gage data indicated that twisting of the tab had begun at small load levels for all of the unstiffened tests. The twisting of the tab was not as visually apparent for the three bolt tests because other primary failure modes, such as web mechanism failure, controlled for those tests and were partially responsible for the termination of nonlinear behavior of the connection. The correlation between the termination of linear behavior and the derived torsion limit state is not as close for the stiffened tests. This is probably due to the fact that the torsion limit state equation was derived from a finite element analysis of an unstiffened test. For the stiffened tests, it appears that the measured load at which nonlinear behavior first occurred more closely correlates with the value for shear yielding through the depth of the tab, as presented in Table 6.1. However, the calculated value for shear yielding through the depth of the tab is larger than the load at which nonlinear behavior first occurred for all of the stiffened

Nonlinear behavior began at approximately the same load level for all stiffened tests in a Group. For all cases, nonlinear behavior was observed well below the calculated load for shear yielding through the depth of the tab and well below when strain gage data indicated the shear yield stress of the material had been reached. Therefore, the onset of nonlinear connection behavior can be considered a global connection phenomenon. The point when nonlinear behavior is reached does not necessarily indicate impending failure of the connection. For most cases, the calculated load for shear yielding through the depth of the tab is larger than the ultimate test load. However, local distortion of the tab and flaking of the whitewash coating on the tab were observed at lower load levels. This observation and the initiation of nonlinear behavior at lower load levels indicates that yielding had occurred in the tab and that the yielding cannot be attributed to direct shear yielding alone but rather a combination of shear yielding from direct shear and shear stresses developed from twisting of the extended shear tab.

6.2 Twisting

Twisting of the shear tab was also observed in a majority of the tests. The twisting was of special concern for the deep tab tests using 5-bolt connections. The twisting of the tab was visually apparent by an out-of-levelness of the supported beam's top flange and by a separation between the shear tab and supported beam's web. This separation usually occurred at the bottom of the tab and was large enough so that a shim could be inserted and pushed into the gap all the way to contact with the bottom bolt. The twisting of the tab was also observed by discrepancies in strain gage data. Strain gages mounted on opposite sides (gages A-D and H-K in Figure 4.10) of the tab were monitored during testing. The magnitude and sign convention of these gages indicated twisting of the tab even at low loads. The twisting of the tab was always in the same direction as the side of the tab that the supported beam web was connected.

An equation was developed for this torsional limit state based on the rectangular cross section of the tab. The torsion applied to the section was taken as the shear force multiplied by a distance equal to $t/6$ where t is the thickness of the extended shear tab connection plate. The $t/6$ distance for the eccentricity of the applied load relative to the centroid of the cross section of the shear tab was determined by a finite element analysis. This analysis involved using the experimental results of Test 1-U. Shear load increments for the test were chosen and were applied to a shear tab at various locations transversely across the thickness of the tab. The results of the finite element model (see Appendix I) were then compared to experimental strain gage data to determine which location best simulated the strains observed on the actual plate. When the shear load was applied at 1/3 the thickness of the plate, or with an eccentricity of $t/6$ relative to the shear tab centroid, experimental strains most closely match strains determined from the finite element analysis.

6. CONNECTION BEHAVIOR

6.1 Nonlinear Behavior

For each test, there was a distinct measured load at which the load-deflection curve became nonlinear. Above this load, the load-deflection curve leveled off and considerable shear distortion was experienced in the shear tab connection. At this point, the connection could maintain only small additional load. This nonlinear behavior can be seen in the load-deflection curve of Figure 5.1. Table 6.1 lists three loads related to this nonlinear behavior.

Table 6.1: Loads related to linearity

Test	V_1 (kips)	V_2 (kips)	V_y (kips)	V_{exp} (kips)
Unstiffened				
1-U	48	---	85.3	58.7
2-U	65	81	142.0	89.3
3-U	37	$> V_{exp}$	85.3	54.8
4-U	82	$> V_{exp}$	142.0	98.7
Stiffened*				
1-A	42	---	59.0	58.3
1-B	44	---	59.0	54.6
2-A	64	---	98.3	89.0
2-B	69	---	98.3	92.6
2-C	65	---	98.3	83.3
3-A	37	---	59.0	53.2
3-B	38	---	59.0	53.1
3-D	38	---	59.0	51.1
3-E	40	---	59.0	48.1
4-A	88	---	102.0	103.0
4-B	84	93	102.0	107.0
4-C	84	97	102.0	107.0

* Test 3-C is excluded due to the very low failure load

V_1 is the measured load where the load-deflection curve became nonlinear. V_2 is when strain gage data indicates that shear yielding had occurred in the tab. This value is only listed for Tests 2-U, 3-U, 4-U, 4-B, and 4-C because these are the only tests that had rosette strain gages mounted on the tab. When the principal stress from the measured strain equaled the shear yield stress of the material, $0.6F_y$, the shear in the connection was recorded and listed as V_2 in Table 6.1. The value of V_2 for tests 3-U and 4-U are not listed because strain gage data indicated that the test was terminated before shear yield had occurred in the tab. V_y is the calculated load for shear yielding through the depth of the tab. The ultimate experimental load, V_{exp}, is also included for reference.

6) An additional limit state, web mechanism failure, must be considered for columns with high h/t_w. This is evident from Test 3-U in which the web mechanism failure limit state was both the observed and predicted primary failure mode and from Test 4-U in which the web mechanism limit state was observed as a secondary failure mode.
7) For stiffened tab tests, the vertical weld between shear tab and support web is essential. Test 3-C, the test with no vertical weld, failed at less than half the shear capacity as Test 3-A. Using large stiffener plates and stiffener-to-support member welds could slightly increase the capacity of the connection but would not be as time and cost effective as simply using the vertical weld between tab and support member web.
8) Unstiffened tabs can be used for small beam reactions. Compared to the AISC typical 3-in. bolt to weld line spacing, the capacities are greatly reduced for the unstiffened extended shear tabs. The capacity reduction is not as great for the stiffened tests (on the order of only 10%).
9) Using one vertical weld of twice the size on one size of the shear tab did not make much difference in connection capacity compared to using two vertical welds on either side of the shear tab. This is evident from comparing the capacity of Test 2-C to the capacity of Test 2-A.
10) An additional limit state, twisting of the shear tab, was identified as either a primary or secondary failure mode for all of the tests except the 3-bolt unstiffened tests. For the 3-bolt unstiffened tests, differences in strain gage data from strain gages mounted on either side of the tab indicate that some twisting of the tab did occur but other failure modes were more prominent (such as bolt shear and bolt bearing for Test 1-U and web mechanism failure for Test 3-U) before twisting became visually evident.
11) Welding the stiffening plates to the column web does not have a significant affect on connection capacity. This is evident from the capacity of Test 3-D as compared to Test 3-A.

5.3.2.1 Web mechanism failure

For the unstiffened tabs welded to column webs, substantial distortions of the column web were observed. Therefore, an additional limit state of a yield line mechanism (see discussion of this limit state under section 3.5) has been included. The equation for moment capacity of this mechanism is taken from Abolitz and Warner [9] in combination with Equation 4-21 (a) of the HSS Connections Manual:

$$V_{ew} = ((2h/L) + (4L/h) + 4*(3)^{1/2})(F_{yw}t_w^{\,2}/4)(L) \qquad 5.5$$

Where: e_w = shear eccentricity to the weld
h = depth of the column web (h/t_w*t_w)
t_w = column web thickness
F_{yw} = column web yield strength
L = tab length

This yield line mechanism does not apply to the stiffened tests. For the unstiffened girder tests, differing boundary conditions existed with the top of the tab being very near to the top flange of the support girder. This flange acted as a stiffener to the support web. For the unstiffened girder tests, the distortion of the support web was not nearly as severe as the column tests so the yield line mechanism of the web was not considered.

5.4 Observations and Conclusions (refer to Table 5.3)

1) There was no difference in connection capacity between using snug bolts in STD holes or fully tightened bolts in SSL holes. This is evident from the comparison of connection capacities for the A and B stiffened tests.
2) Calculated connection ultimate capacities calculated based on measured eccentricities correlate better with experimental results than capacities calculated using eccentricity values of AISC equations (with the exception of Test 4-U). AISC eccentricities always produced bolt shear as the critical limit state where measured eccentricities often times produced shear yield as the critical limit state which was the observed limit state for many tests.
3) Using current AISC eccentricities for extended shear tabs produce conservative results for extended shear tab design. This is evident from the fact that capacities using the AISC eccentricities are much lower than experimental results whereas capacities using experimental eccentricities correlate much better with experimental results.
4) The change in eccentricity associated with extended shear tabs can change the critical limit state. This is evident from the fact that AISC eccentricities always indicate bolt shear as the governing limit state while experimental eccentricities indicate that bolt shear, shear yield, and web mechanism failure can all be the governing limit state depending on the test configuration. These results more closely represent experimentally observed limit states.
5) Laterally bracing the test beam near the point of load application does not affect the capacity. This is evident from comparing capacities for tests 1A-1B, 2A-2B, 4A-4B-4C.

The 1/4-in. and 3/8-in. shear tabs used in this study satisfy both of these tab thickness requirements. The 1/2-in. tab selected for Test 4U of Part 1 did not satisfy the rotational ductility requirement. The 1/2-in. thickness is slightly larger than the 0.4375-in. limit. This tab thickness was determined from equation 4.8.

Calculations were made for each of the six standard limit states for each test and the critical shear and limit state values are included in Table 5.3. The limit states were calculated for two different eccentricities. The first is the AISC eccentricity highlighted in Table 5.2. The results of these calculations are tabulated as V_{th} (theoretical shear). The second is the experimental eccentricity from Table 5.2. These results are tabulated as V_e (experimental shear). Also included in Table 5.3 is the shear capacity for all of the tests if the standard 3-in. bolt-to-weld line space had been used in the tests, V_3. This value is used merely as a comparison as to the amount of strength reduction when extended shear tabs are used instead of standard shear tabs. For the unstiffened tests, the connection capacities for V_e are much smaller than V_3 (between 47 and 63% depending on which test is considered). However, for the stiffened tests, connection capacities for V_e vary between 79 and 104% of V_3. The stiffening of the tab with horizontal welds to provide a 3-in. distance form a bolt line to end of horizontal weld is the reason that the connection capacity for the stiffened tests approach that of standard shear tabs with a 3-in. bolt line to vertical weld distance. Furthermore, the values of V_e for stiffened girder tests (Groups 1 and 2) are smaller than the values for V_3 whereas the values for V_e for stiffened column tests (Groups 3 and 4) are very close to the V_3 values. This is due to the fact that the girder tests have tabs stiffened only on top and the column tests have tabs stiffened top and bottom which more closely resembles the stiffening of a standard shear tab.

Some comments should be made on Table 5.3:

1) AISC critical capacities are calculated using actual thickness and material properties of tabs and support members. These can be found in Table 5.1.
2) AISC critical capacities are calculated using the bolt to weld line spacing in Tables 4.1 and 4.2. This spacing is taken as the distance from bolt line to the vertical weld for unstiffened tests and from the bolt line to the center of gravity of the weld group for stiffened tests.
3) AISC critical capacities and experimental capacities do not include connection resistance factors. These capacities are nominal capacities and not design capacities.
4) Appendix H includes sample calculations for determining AISC critical capacities and Appendix I includes sample photographs of the various failure modes identified in Table 5.3:

    Figure H.1: Bolt Shear
    Figure H.2: Bolt Bearing
    Figure H.3: Shear Yield
    Figure H.4: Web Mechanism
    Figure H.5: Twist

Shear Rupture was also identified as a failure mode in Table 5.3 for one test, Test 1U, but no photograph is included.

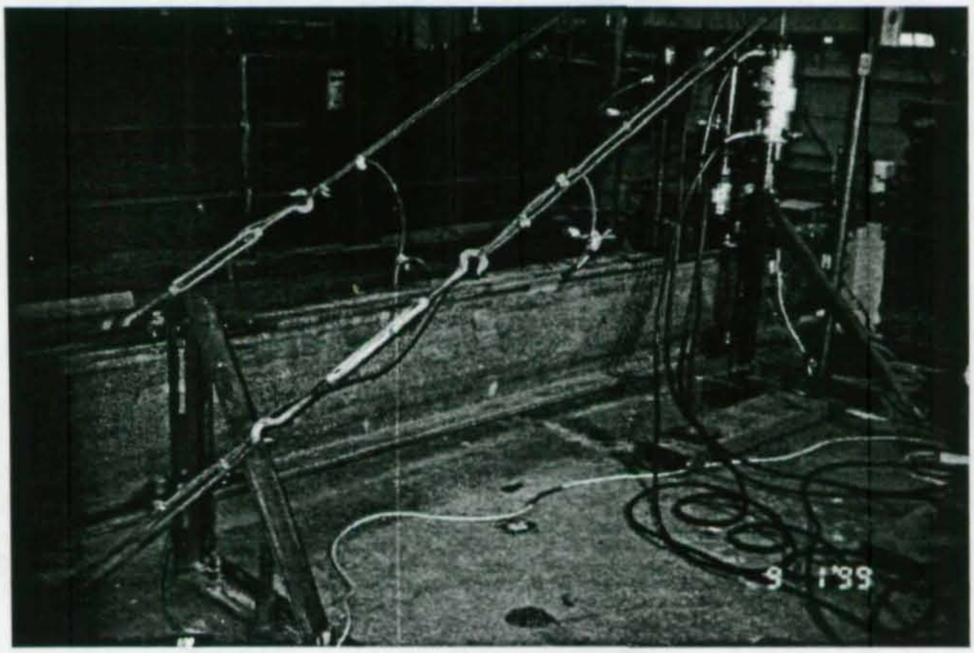


Figure B.7: Bracing locations for girder test (Test 2-B)

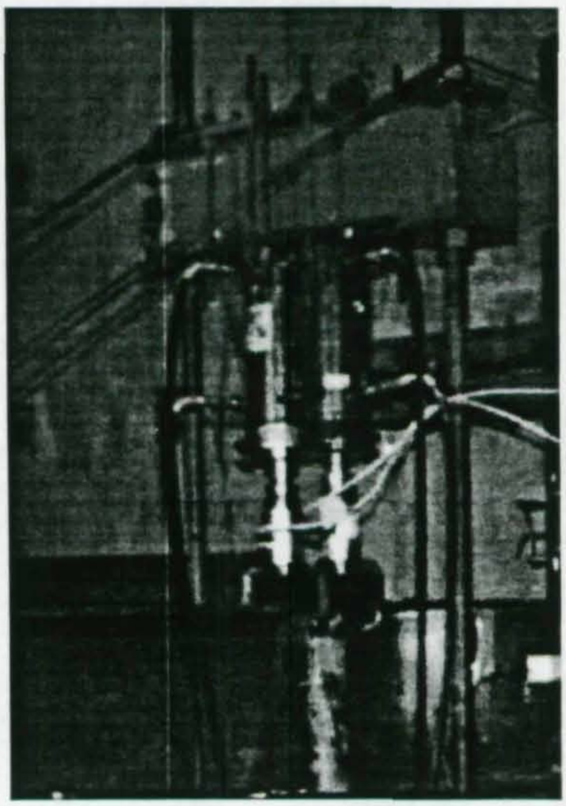


Figure B.8: Load Cells (Test 4-A)

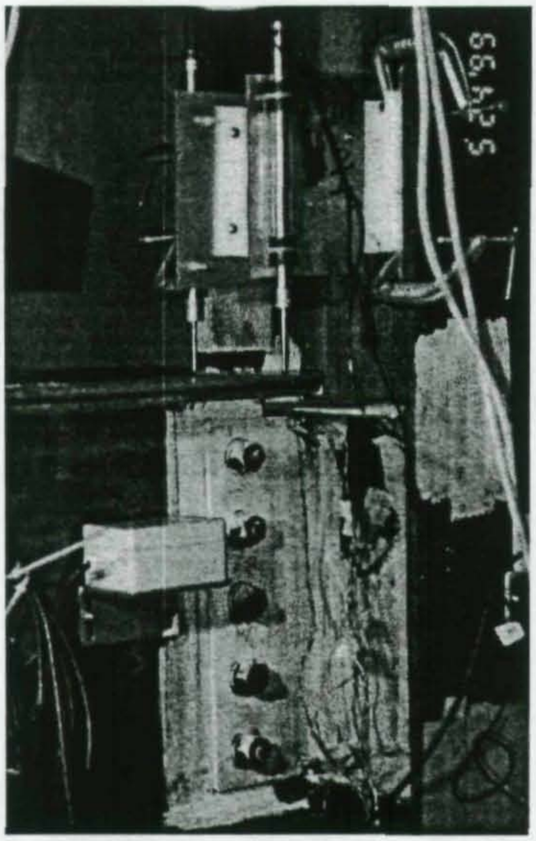


Figure B.9: LVDT setup for column tests (Test 4-C)

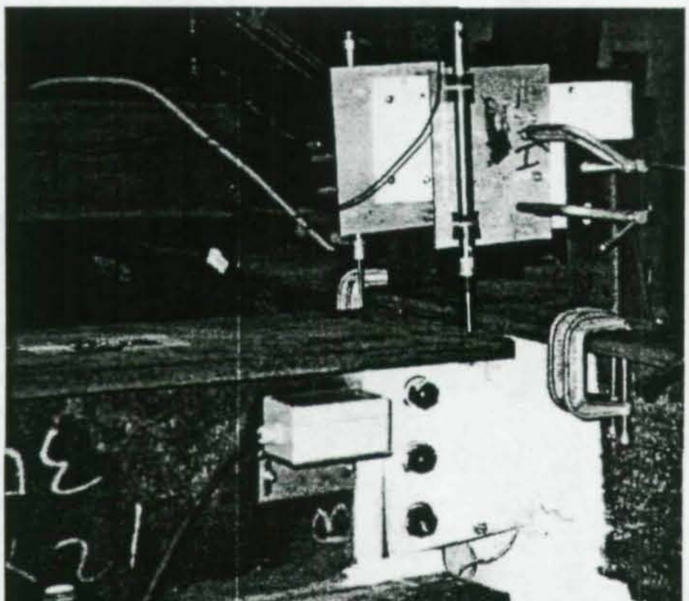


Figure B.10: LVDT setup for girder tests (Test 1-B)

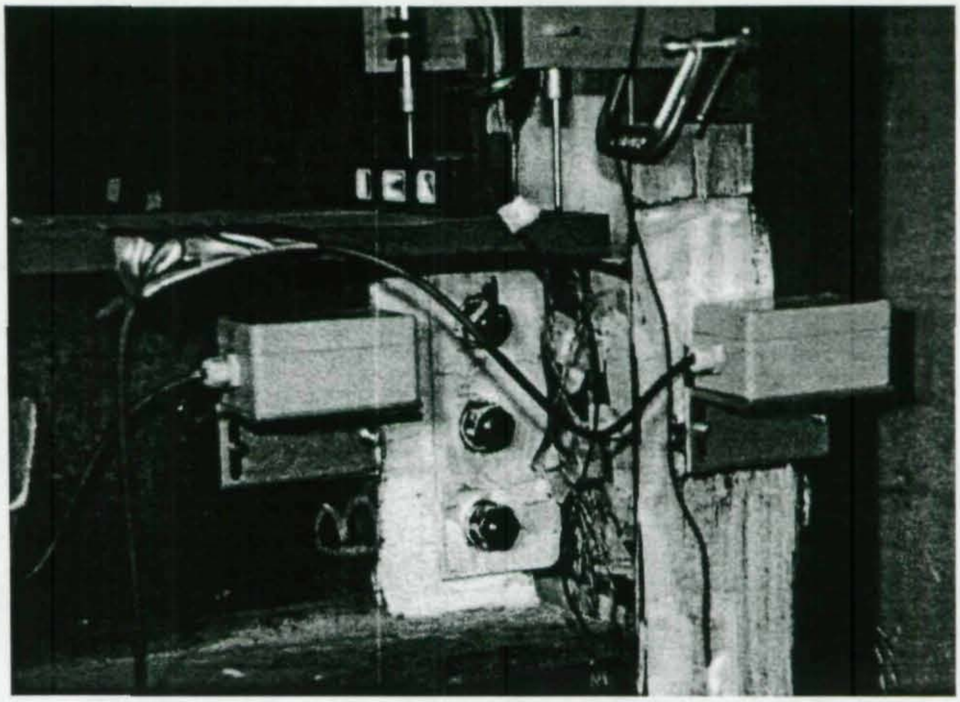


Figure B.11: Location of tiltmeters (Test 3-B)

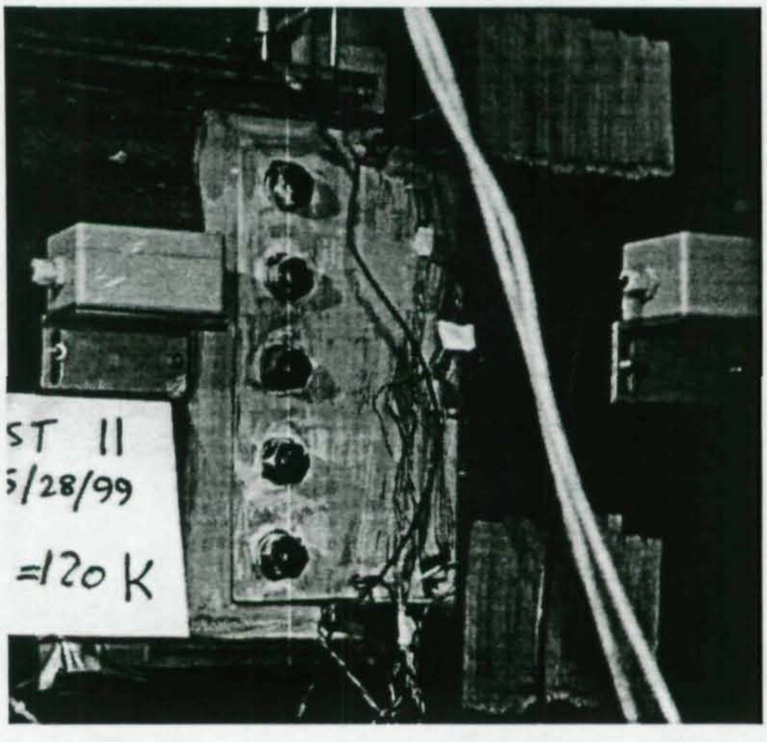


Figure B.12: Typical whitewash coating of connection and support flanges (Test 4-B)

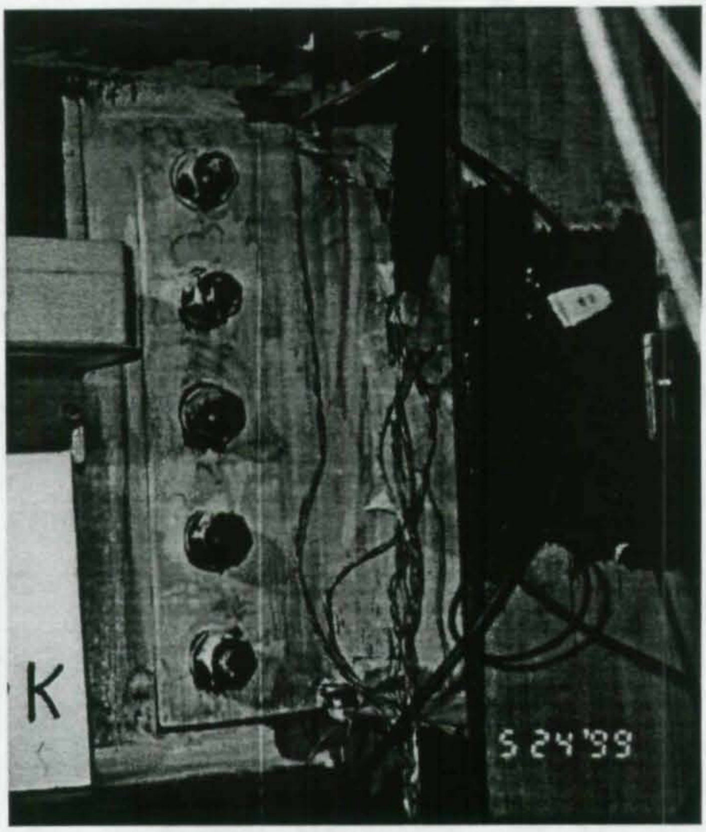


Figure B.13: Stiffened extended shear tab with strain gages and wiring (Test 4-C)

APPENDIX C: TAB THICKNESS DETERMINATION (PART 1)

The tab thickness calculations must ensure shear yield of the gross section before elastic instability of the section:

<u>Example Calculation:</u>

W14X90 Column: $a = 10.04$ in.

$L = 15$ in.

Try 1/4-in. shear tab:

For shear yield: $V_{cr} = 0.6F_ytL = 0.6(36)(.25)(15) = 81$ kips

For elastic stability (Equation 4.8): $V_{cr} = t^3(12000L)/a^2 = .25^3(12000*15)/10.04^2$

$= 28$ kips <u>NO GOOD</u>

Try 3/8-in. shear tab:

For shear yield: $V_{cr} = 0.6F_ytL = 0.6(36)(.375)(15) = 122$ kips

For elastic stability (Equation 4.8): $V_{cr} = t^3(12000L)/a^2 = .375^3(12000*15)/10.04^2$

$= 94$ kips <u>NO GOOD</u>

Try 1/2-in. shear tab:

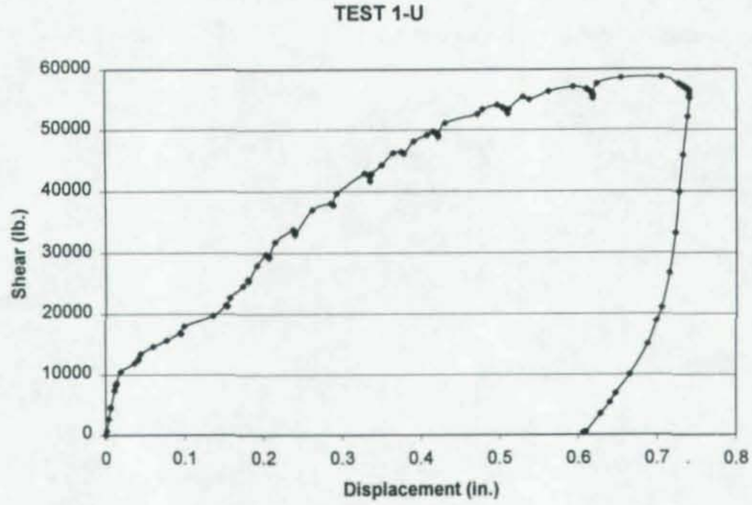
For shear yield: $V_{cr} = 0.6F_ytL = 0.6(36)(.5)(15) = 162$ kips

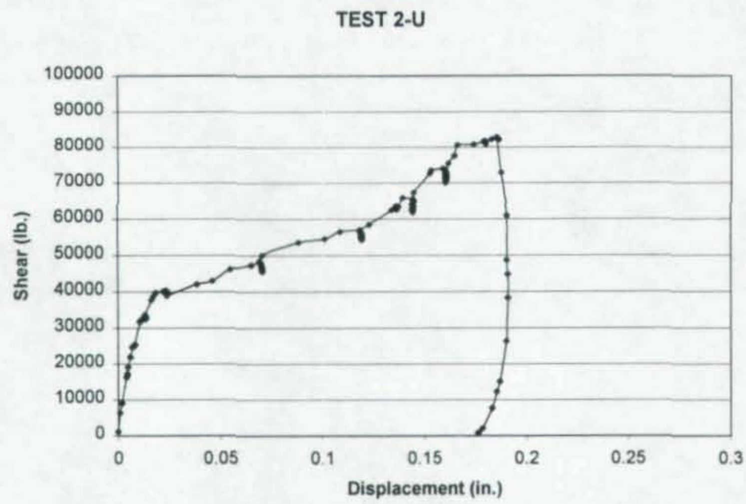
For elastic stability (Equation 4.8): $V_{cr} = t^3(12000L)/a^2 = .5^3(12000*15)/10.04^2$

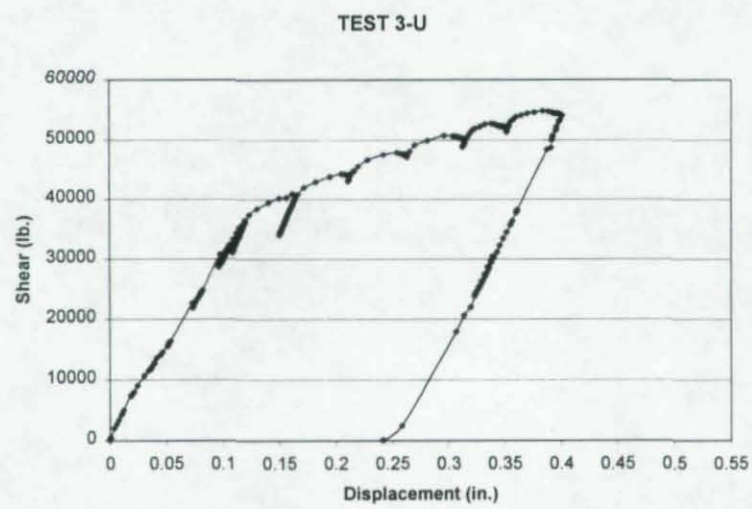
$= 223$ kips <u>O.K.</u>

Use 1/2-in. shear tab thickness for the unstiffened W14X90 column test. A similar procedure was used to arrive at the thicknesses for the other tests in Part 1.

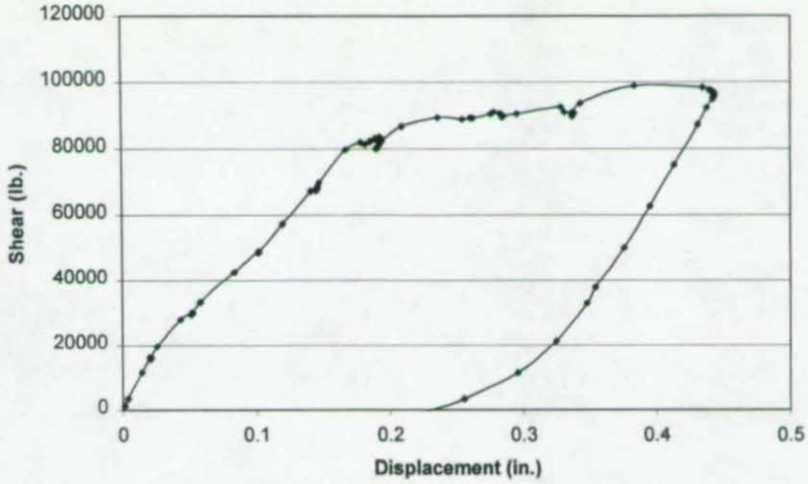
APPENDIX D: SHEAR-DISPLACEMENT GRAPHS



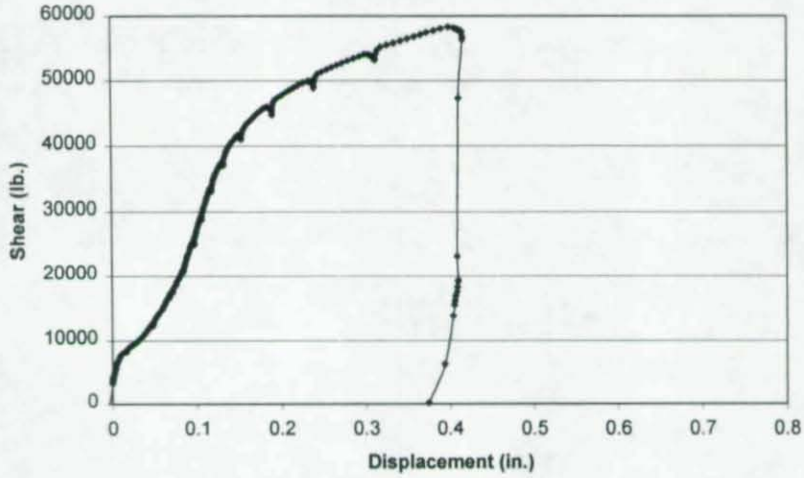




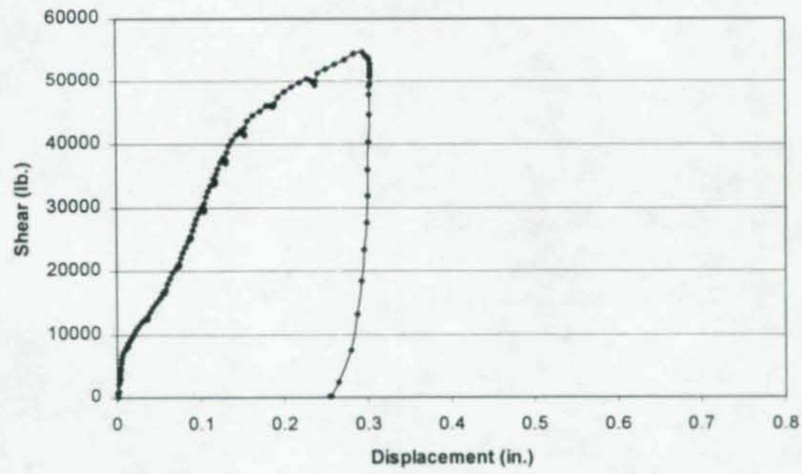
TEST 4-U

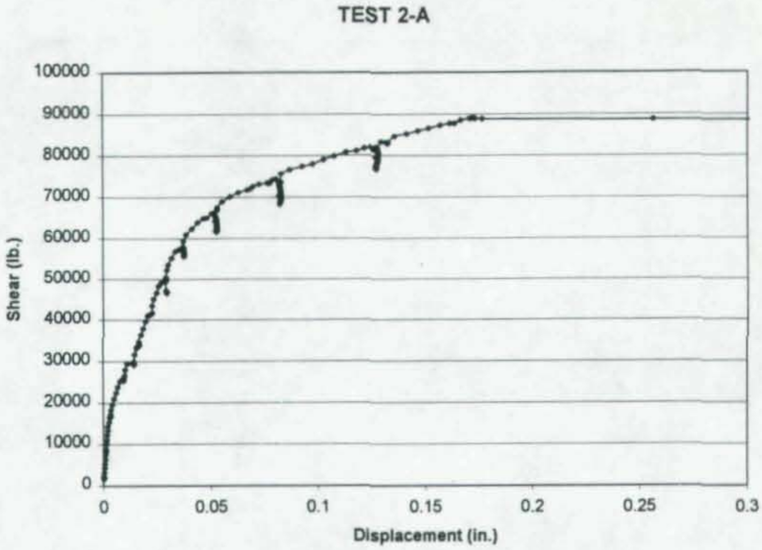


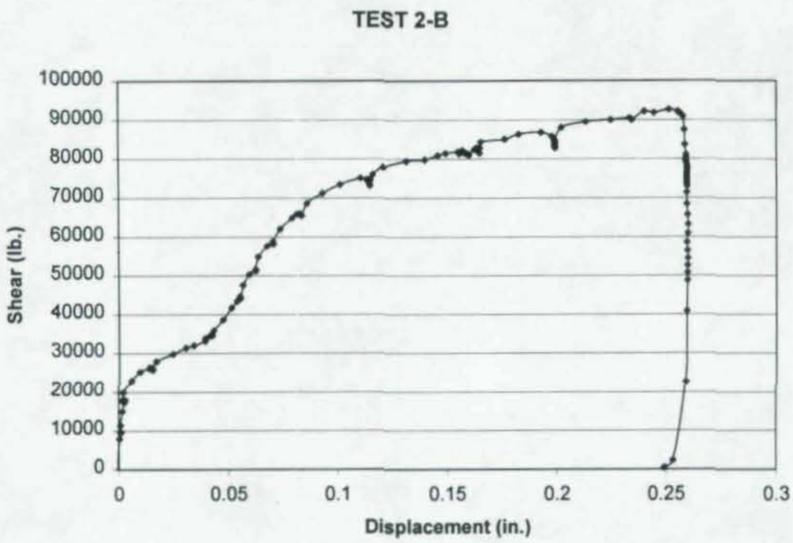
TEST 1-A

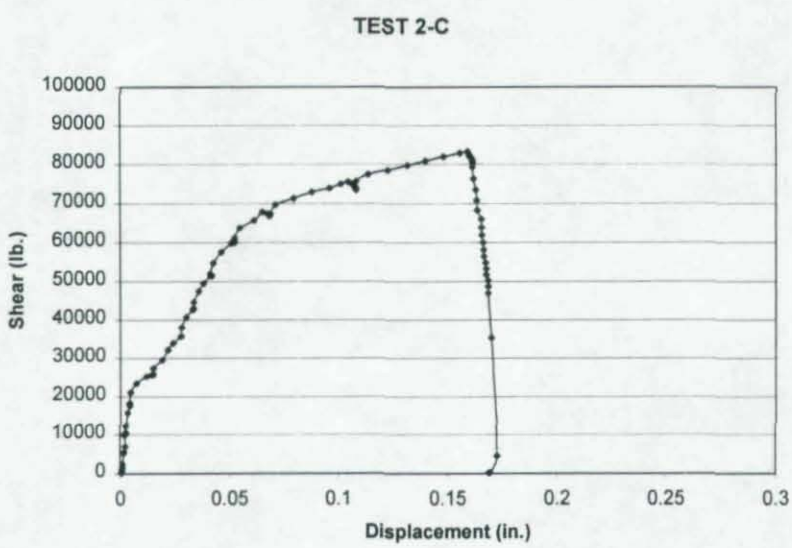


TEST 1-B

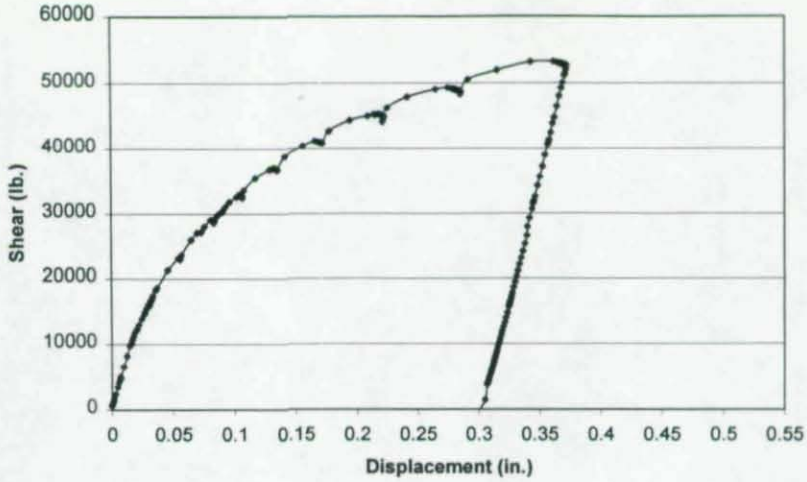



TEST 2-A
Shear (lb.)
Displacement (in.)
100000
90000
80000
70000
60000
50000
40000
30000
20000
10000
0
0
0.05
0.1
0.15
0.2
0.25
0.3

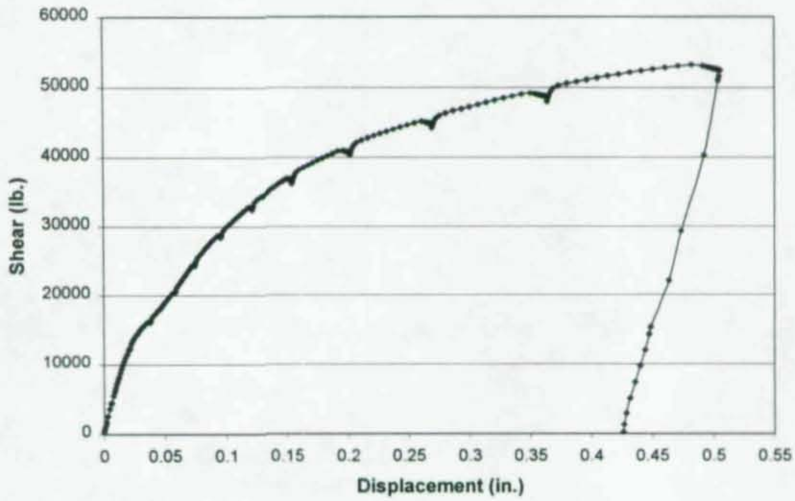

TEST 2-B
Shear (lb.)
Displacement (in.)
100000
90000
80000
70000
60000
50000
40000
30000
20000
10000
0
0
0.05
0.1
0.15
0.2
0.25
0.3


TEST 2-C
Shear (lb.)
Displacement (in.)
100000
90000
80000
70000
60000
50000
40000
30000
20000
10000
0
0
0.05
0.1
0.15
0.2
0.25
0.3

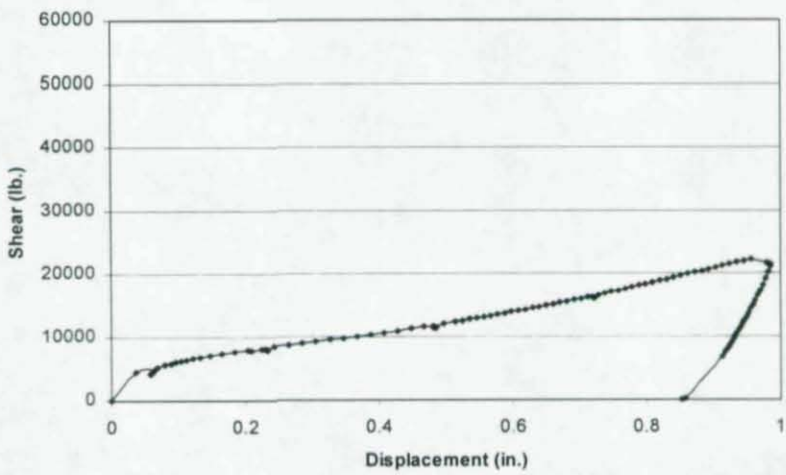
TEST 3-A



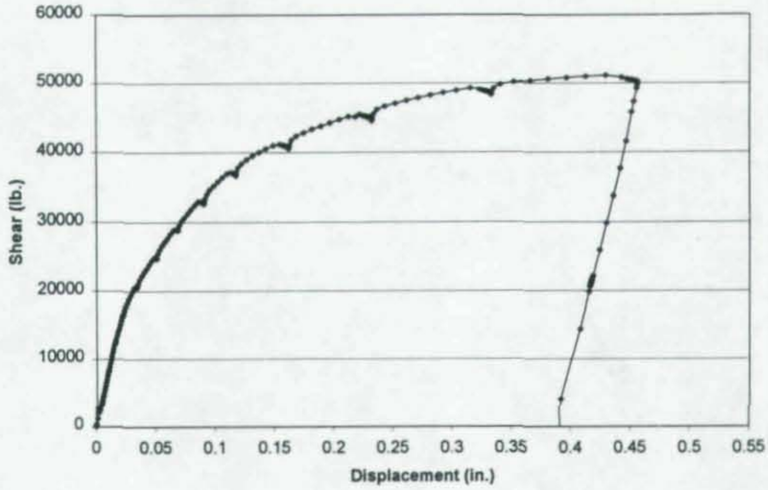
TEST 3-B



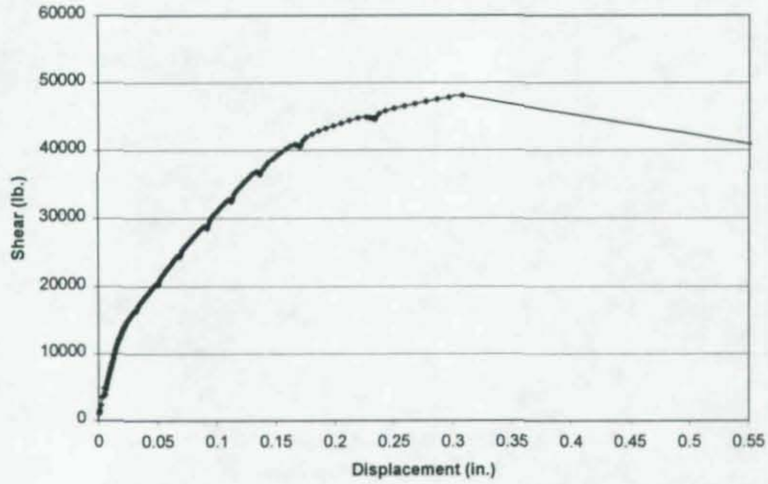
TEST 3-C



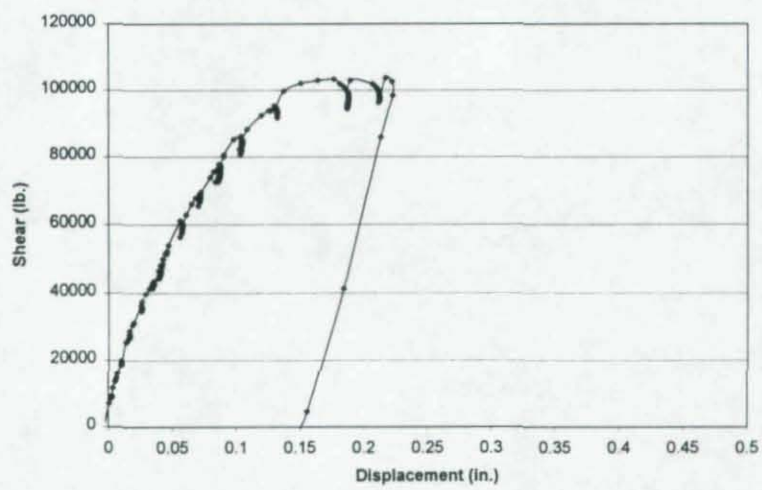
TEST 3-D



TEST 3-E

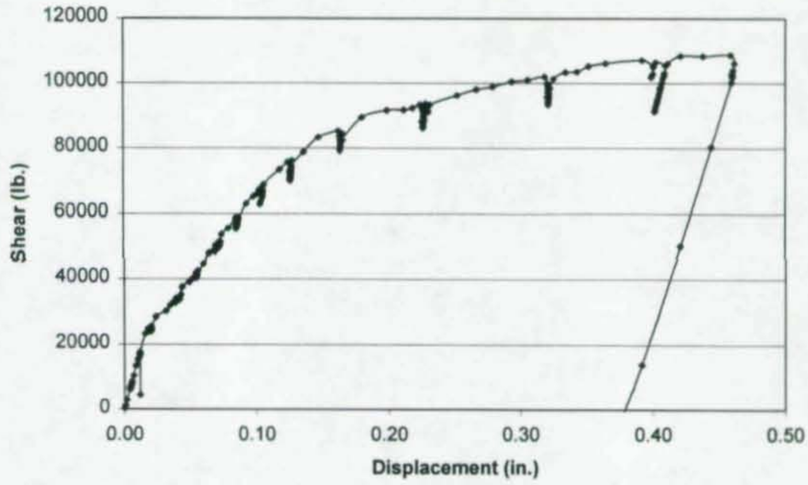


TEST 4-A

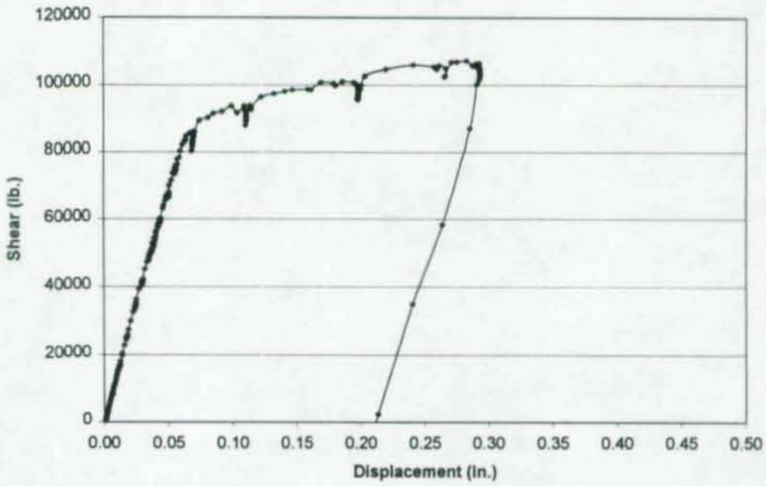


00418

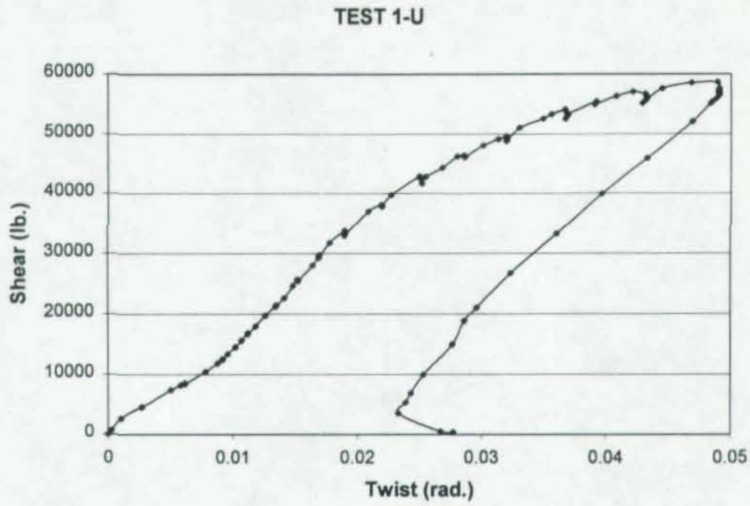
TEST 4-B

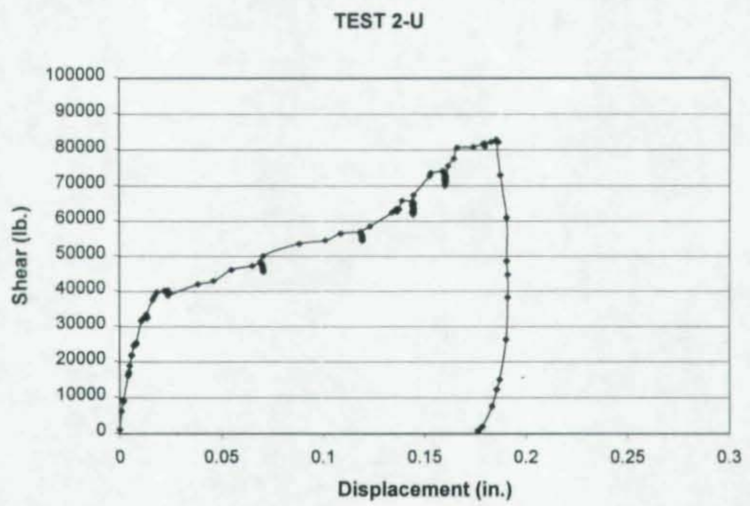


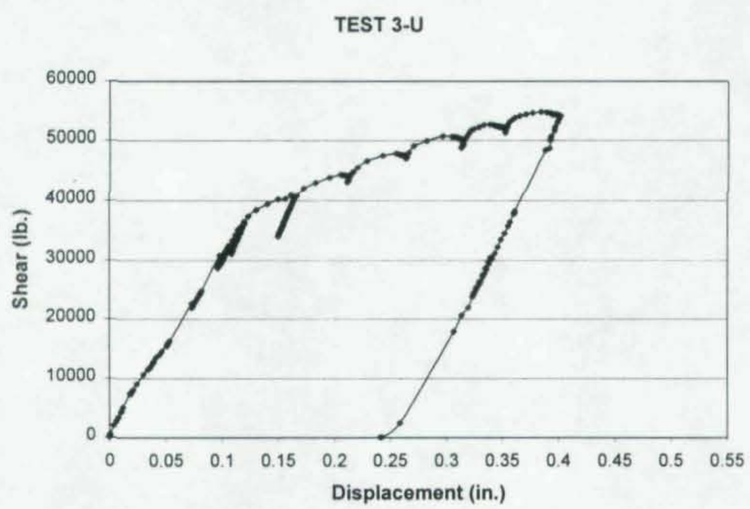
TEST 4-C



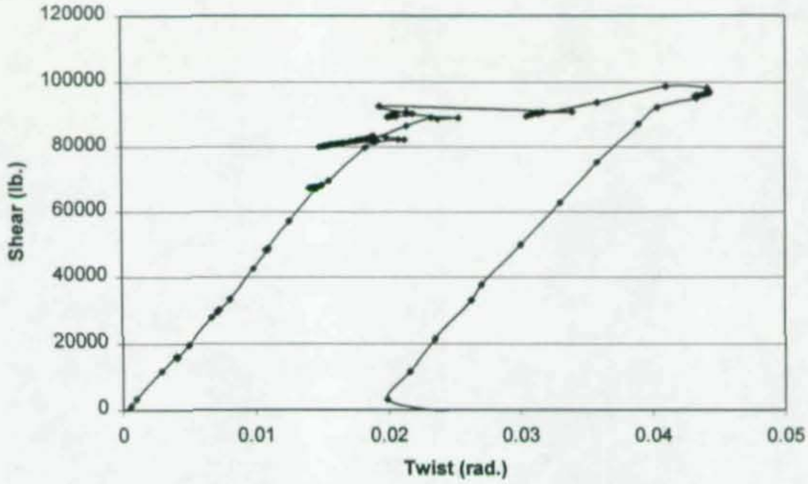
APPENDIX E: SHEAR-TWIST GRAPHS



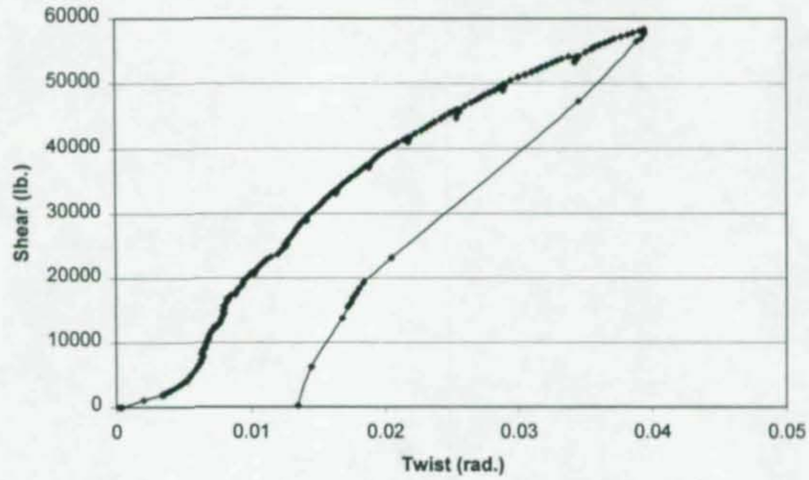




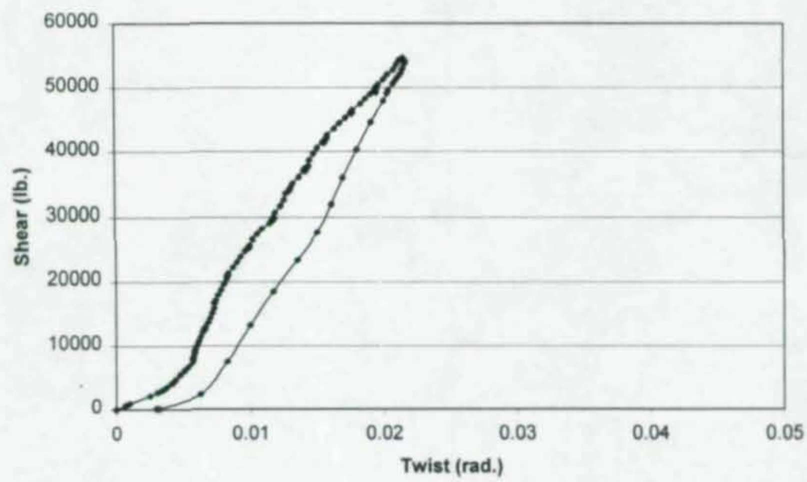
TEST 4-U



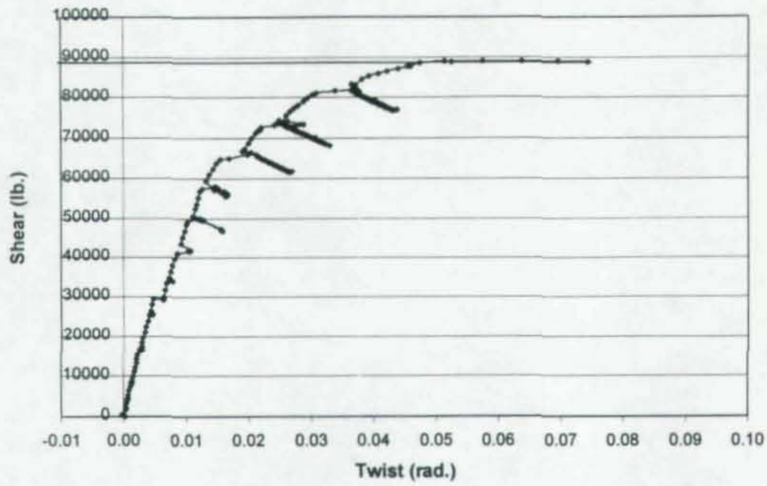
TEST 1-A



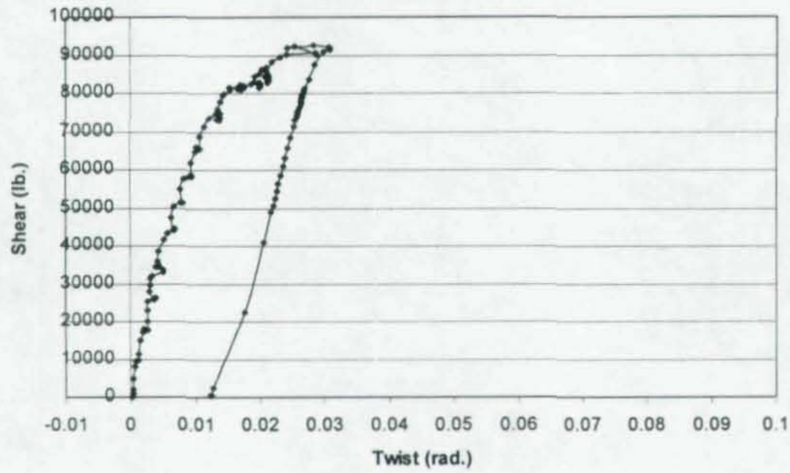
TEST 1-B



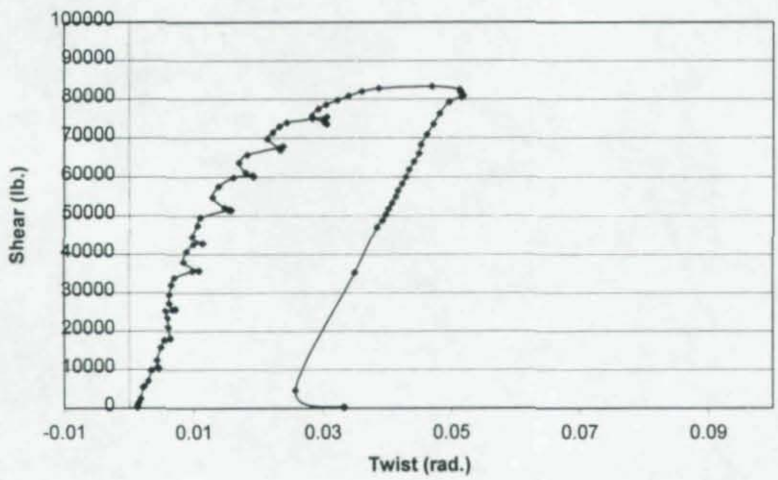
TEST 2-A



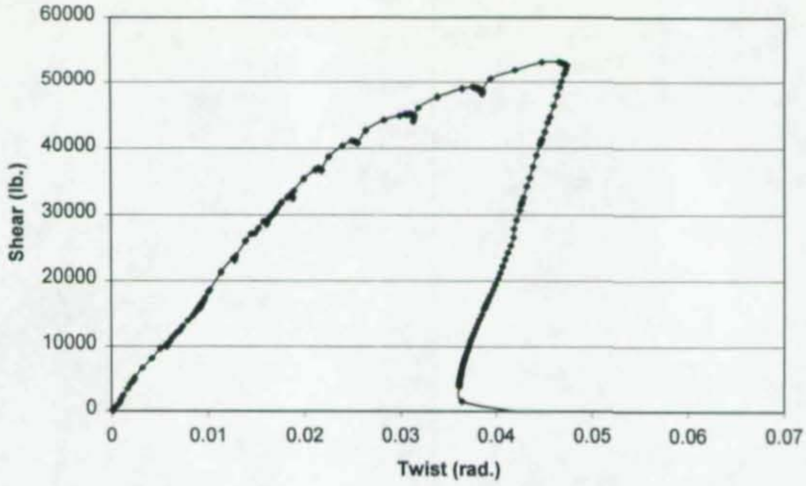
TEST 2-B



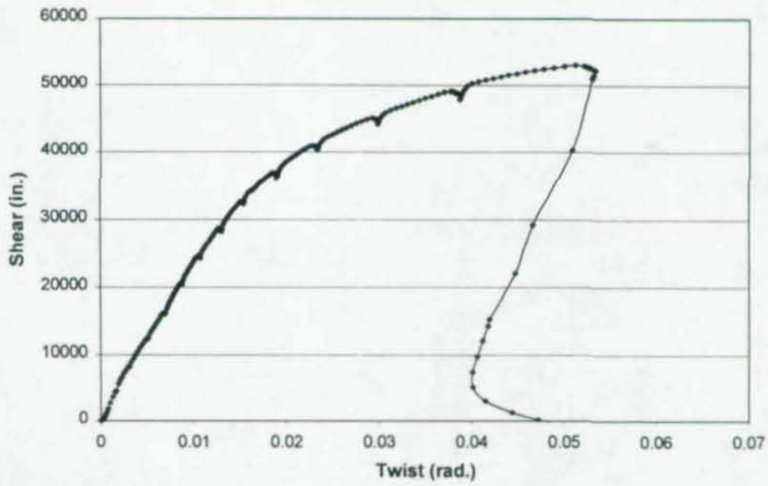
TEST 2-C



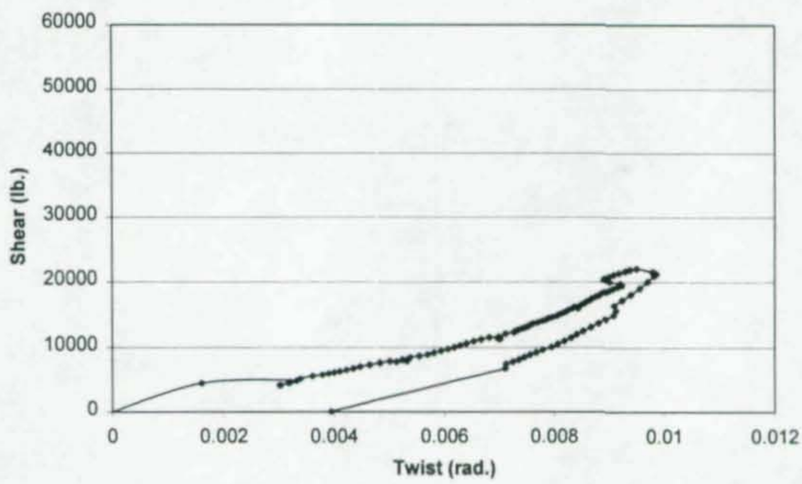
TEST 3-A



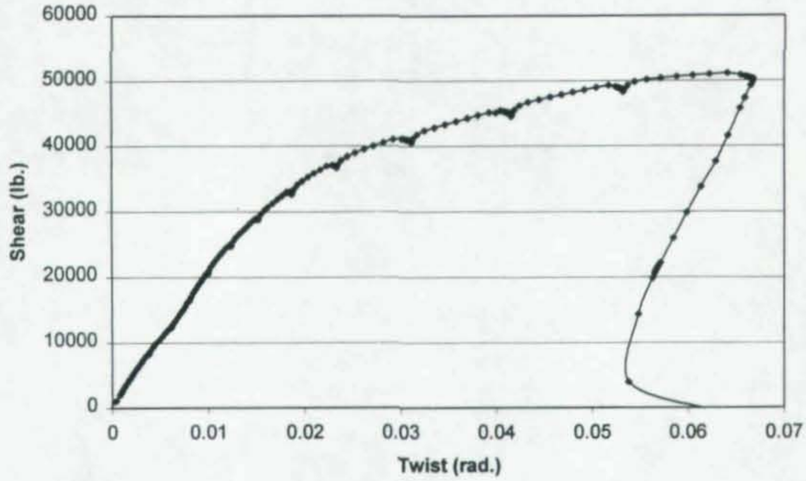
TEST 3-B



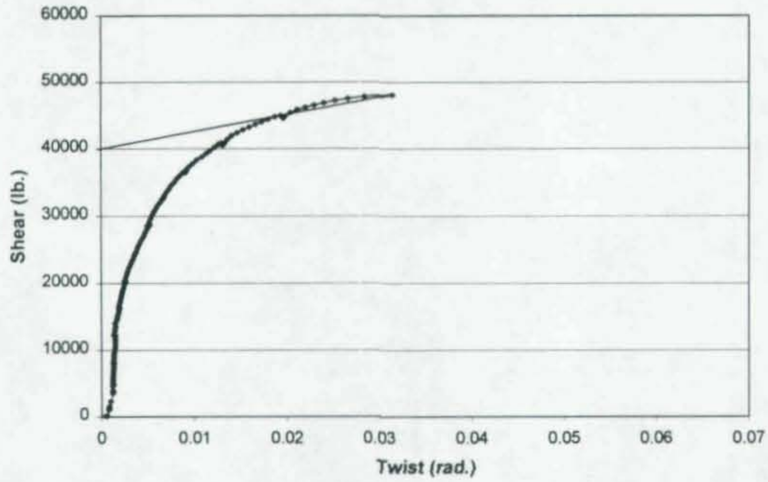
TEST 3-C



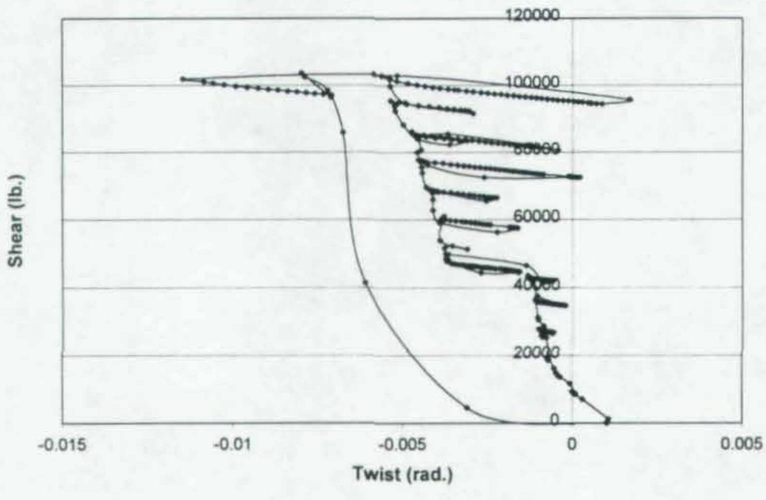
TEST 3-D



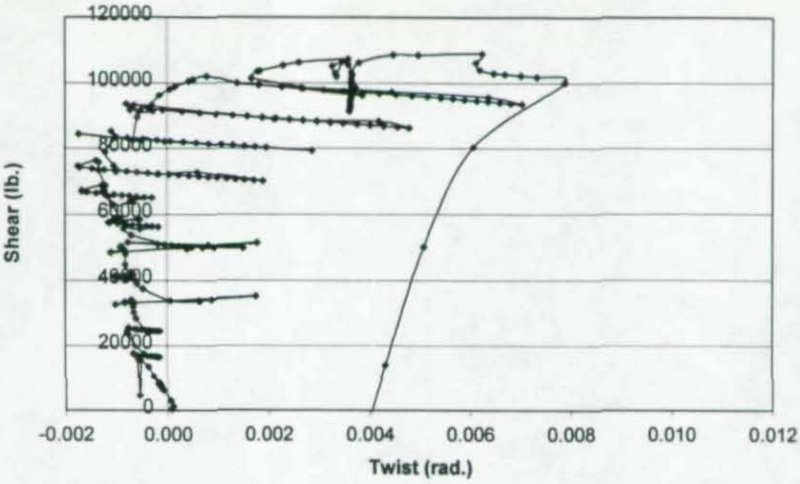
TEST 3-E



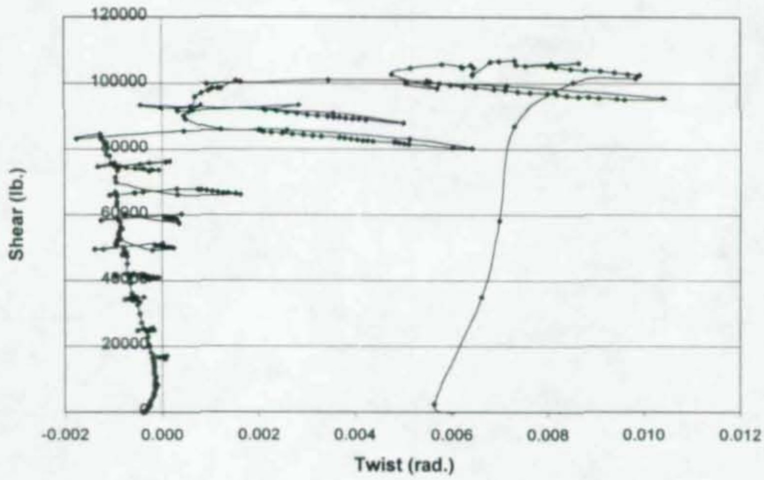
TEST 4-A



TEST 4-B

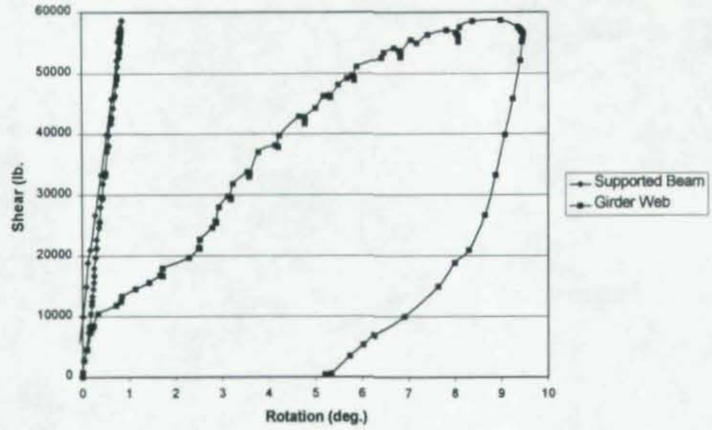


TEST 4-C

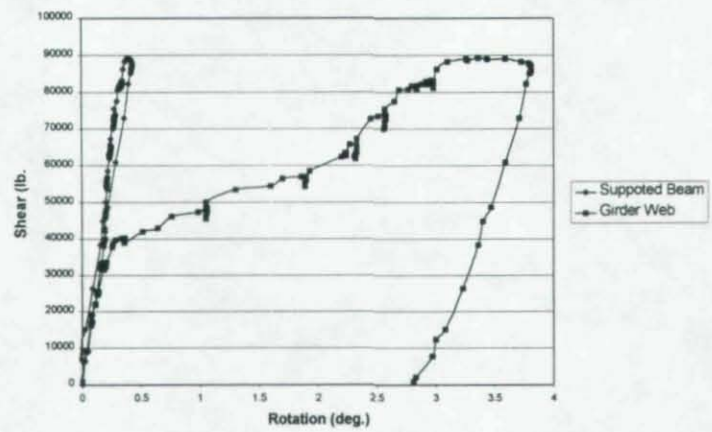


APPENDIX F: SHEAR-ROTATION GRAPHS

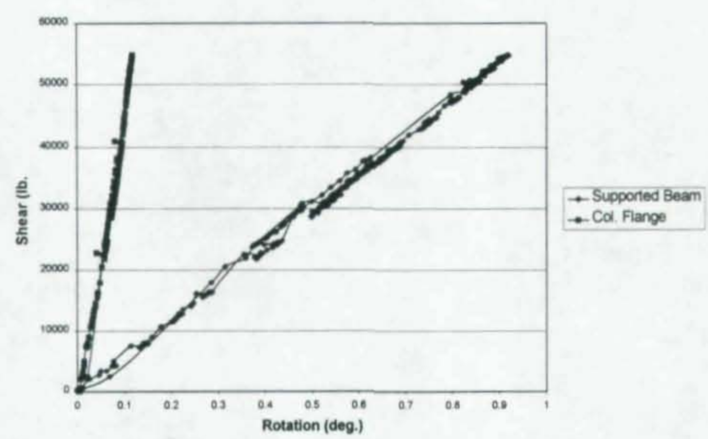
TEST 1-U



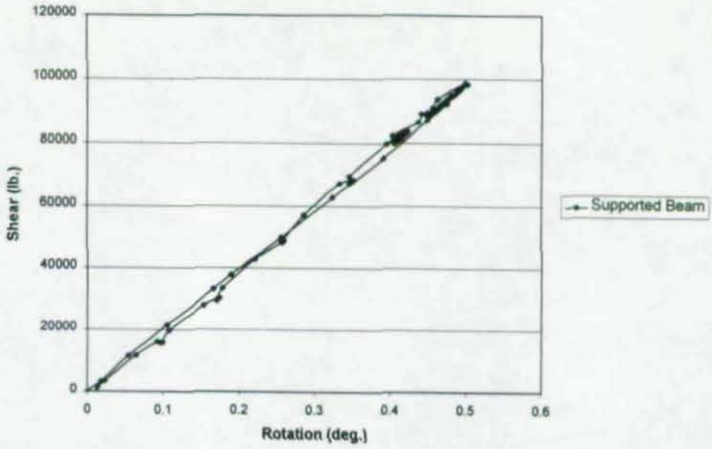
TEST 2-U



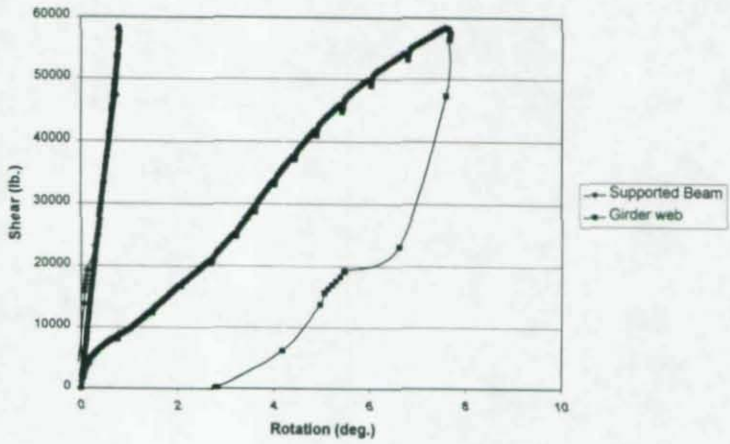
TEST 3-U



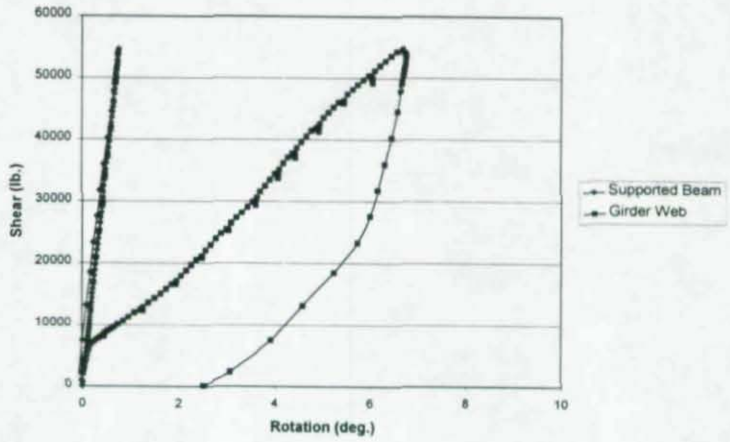
TEST 4-U



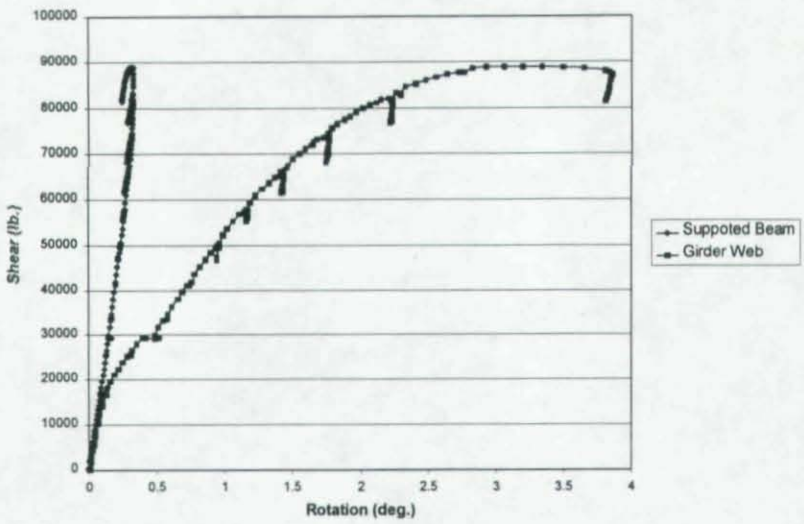
TEST 1-A



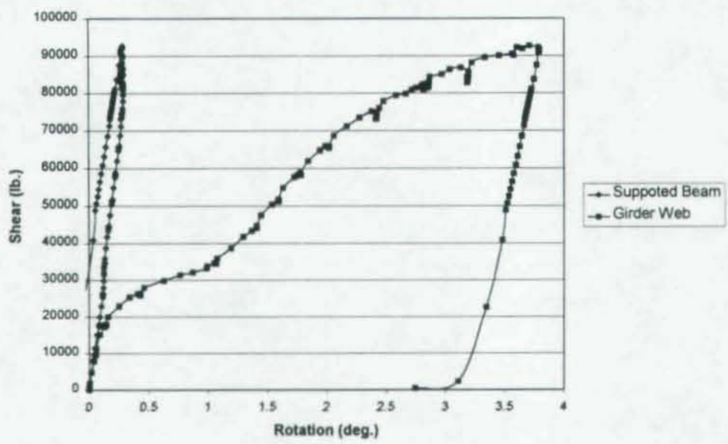
TEST 1-B



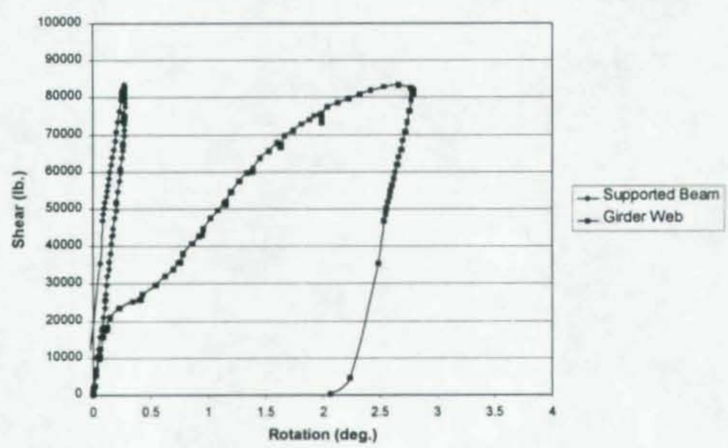
TEST 2-A

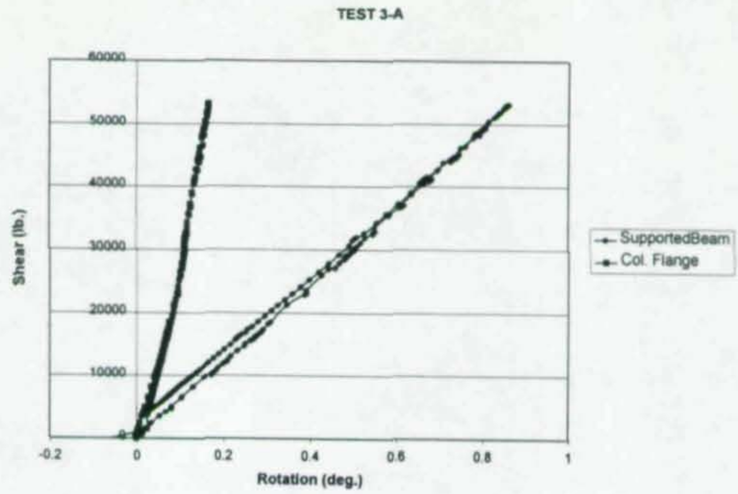


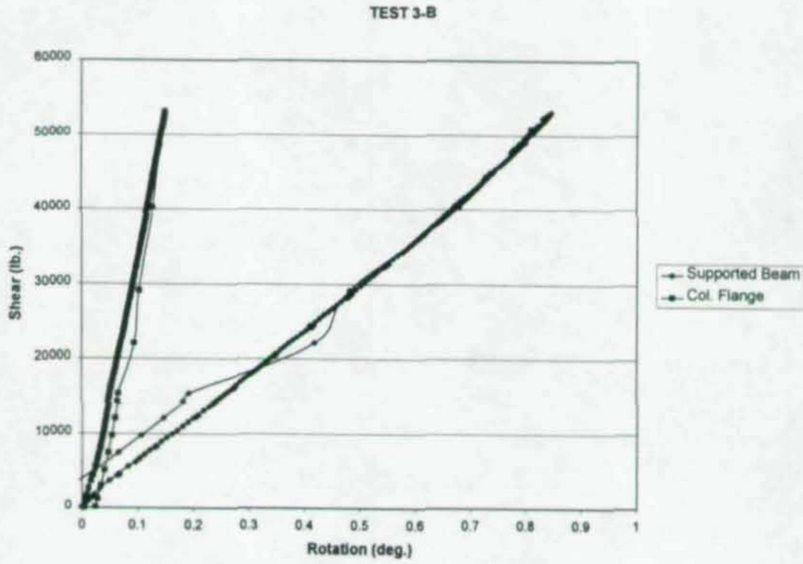
TEST 2-B

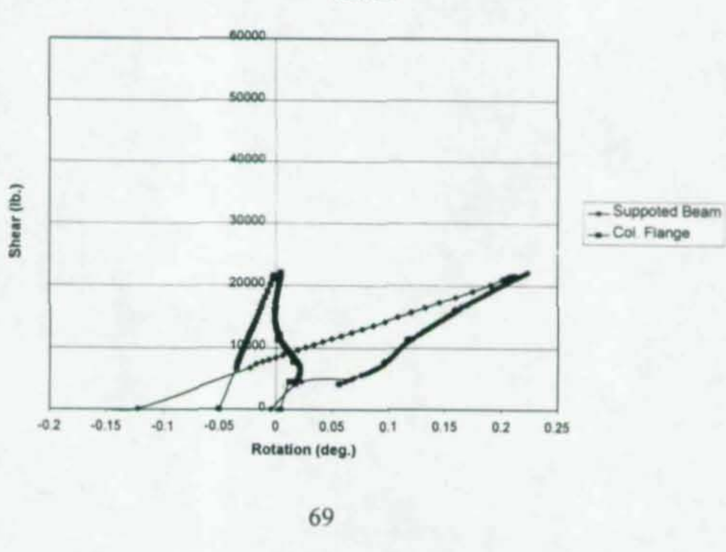


TEST 2-C

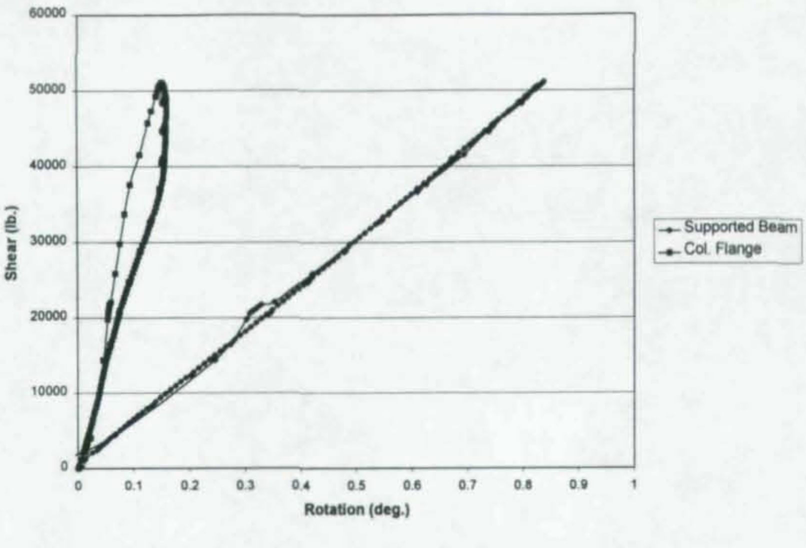



TEST 3-A
Shear (lb.)
Rotation (deg.)
SupportedBeam
Col. Flange
60000
50000
40000
30000
20000
10000
0
-0.2
0
0.2
0.4
0.6
0.8
1

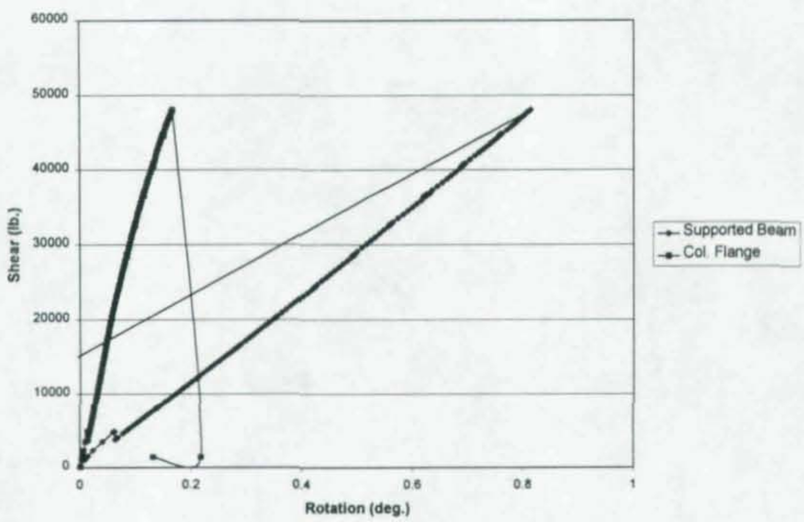

TEST 3-B
Shear (lb.)
Rotation (deg.)
Supported Beam
Col. Flange
60000
50000
40000
30000
20000
10000
0
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1


TEST 3-C
Shear (lb.)
Rotation (deg.)
Suppoted Beam
Col. Flange
60000
50000
40000
30000
20000
10000
0
-0.2
-0.15
-0.1
-0.05
0
0.05
0.1
0.15
0.2
0.25

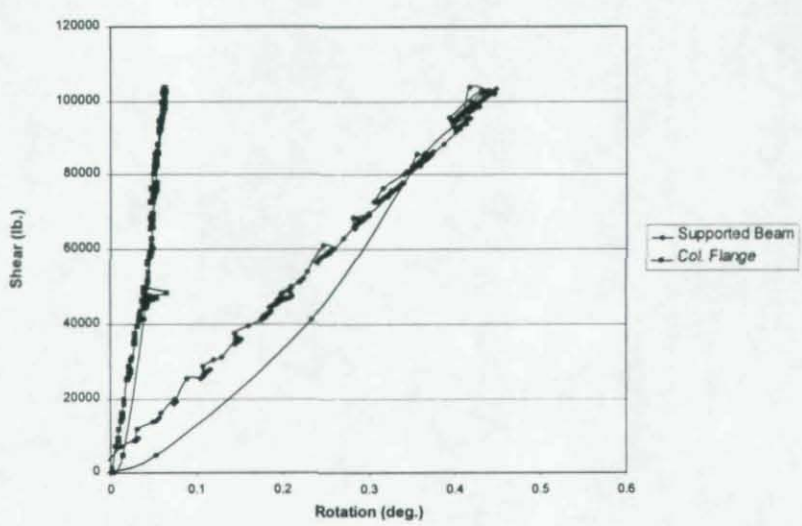
TEST 3-D

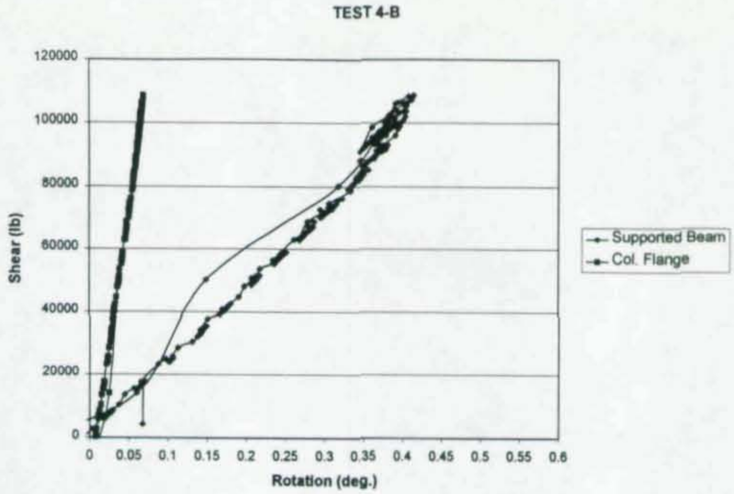


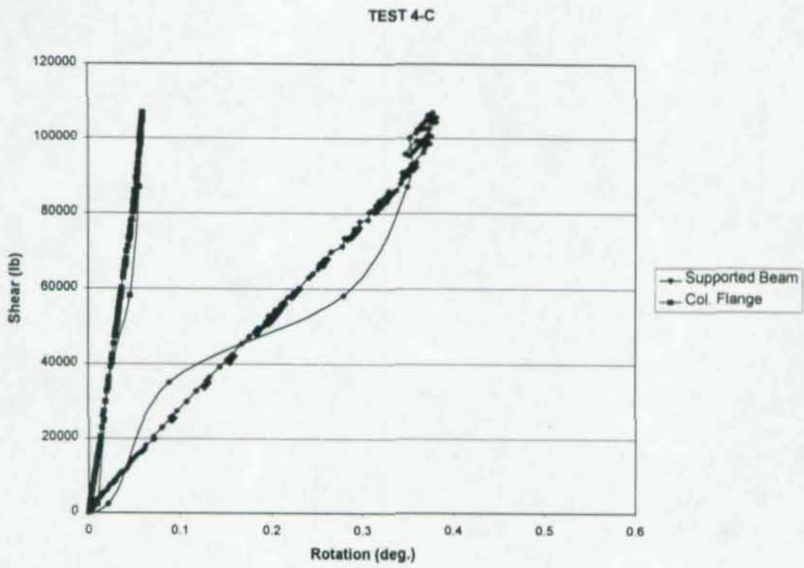
TEST 3-E



TEST 4-A

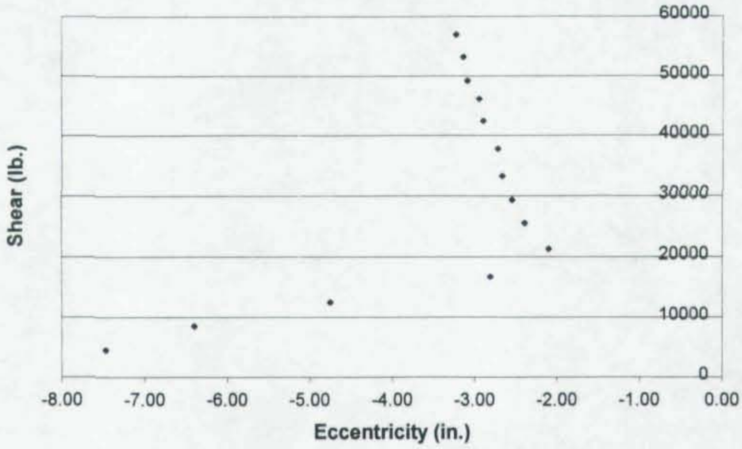



TEST 4-B
120000
100000
80000
60000
40000
20000
0
Shear (lb)
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
0.5
0.55
0.6
Rotation (deg.)
Supported Beam
Col. Flange

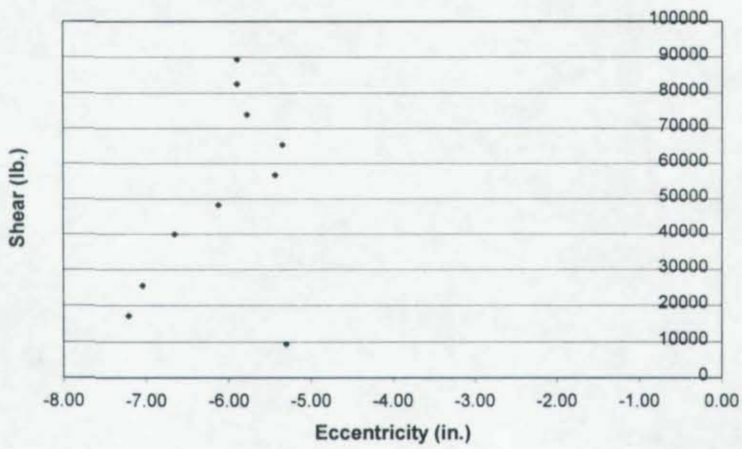

TEST 4-C
120000
100000
80000
60000
40000
20000
0
Shear (lb)
0
0.1
0.2
0.3
0.4
0.5
0.6
Rotation (deg.)
Supported Beam
Col. Flange

APPENDIX G: SHEAR-ECCENTRICITY GRAPHS

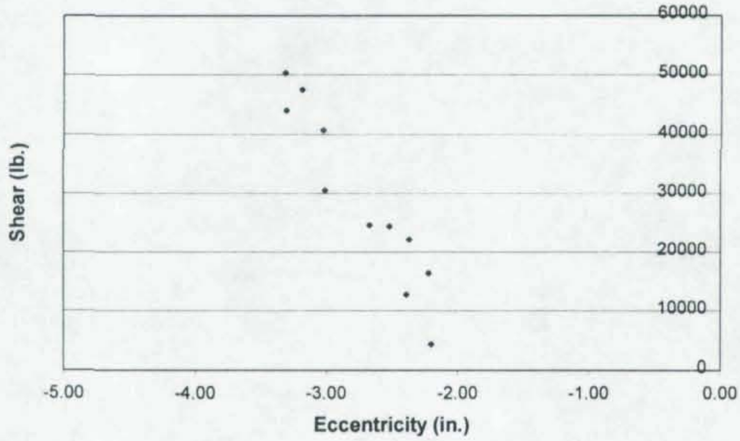
TEST 1-U



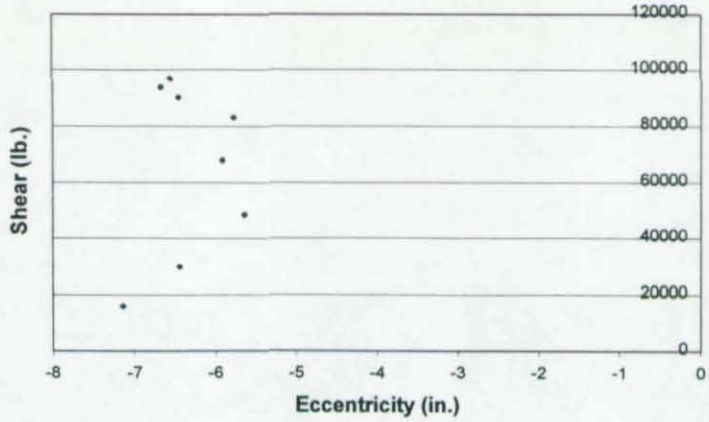
TEST 2-U



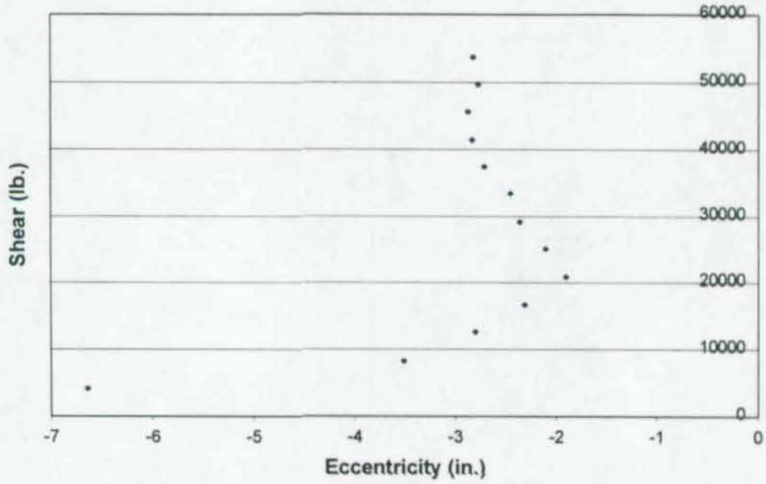
TEST 3-U



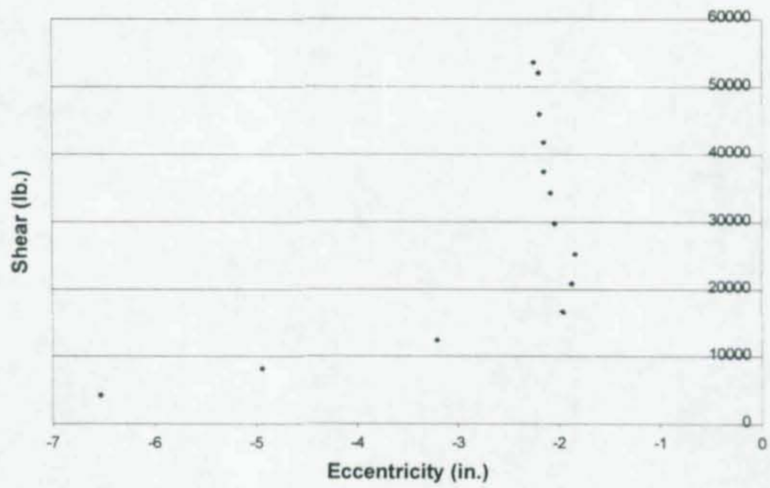
TEST 4-U



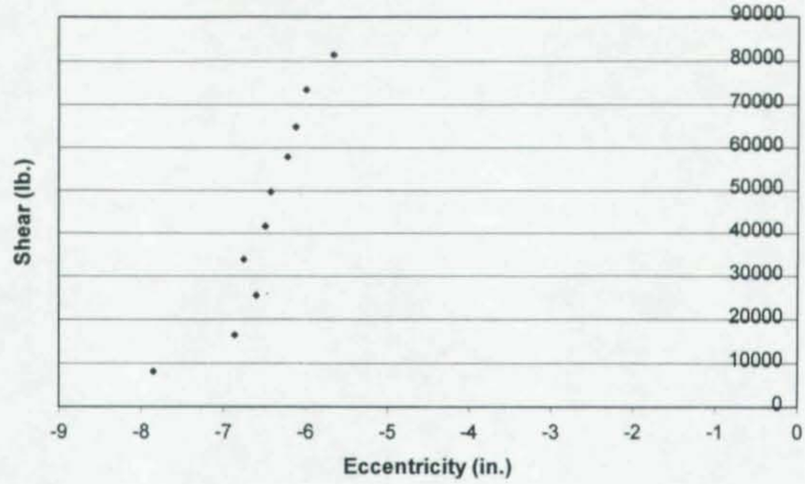
TEST 1-A



TEST 1-B



TEST 2-A



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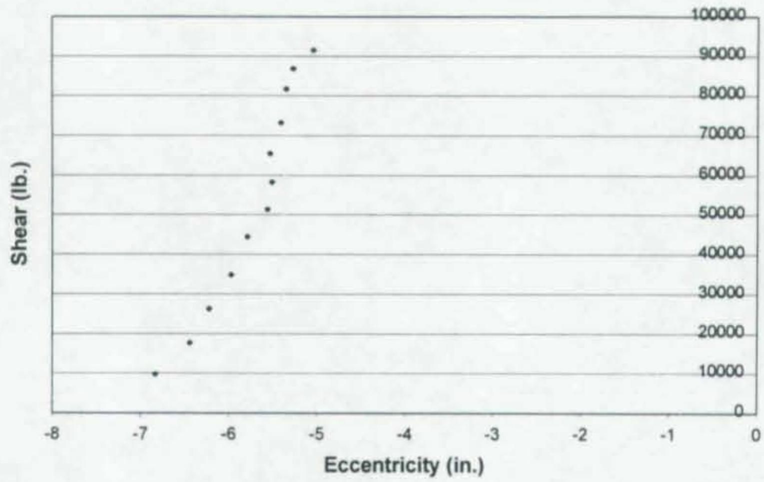
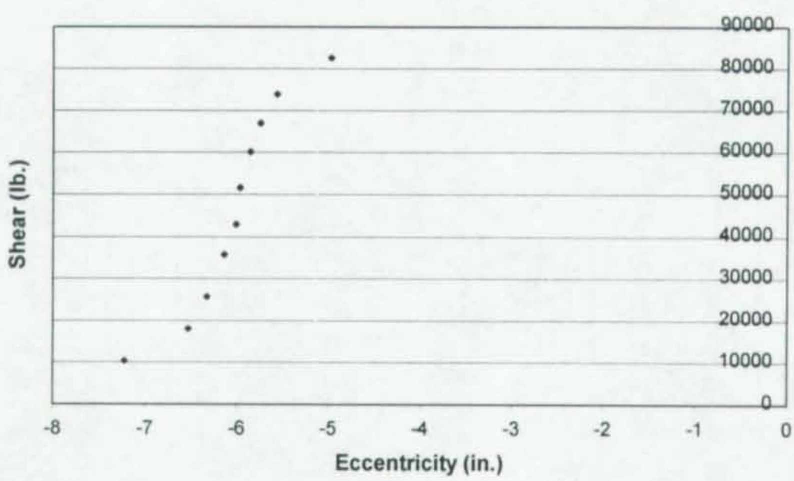
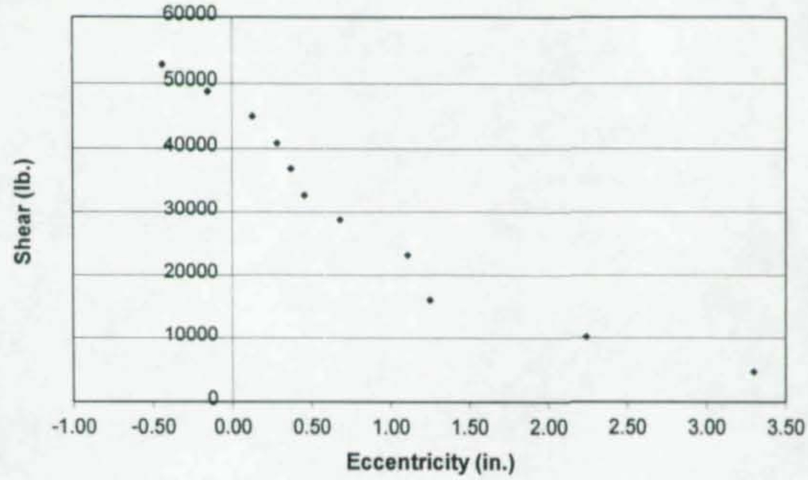


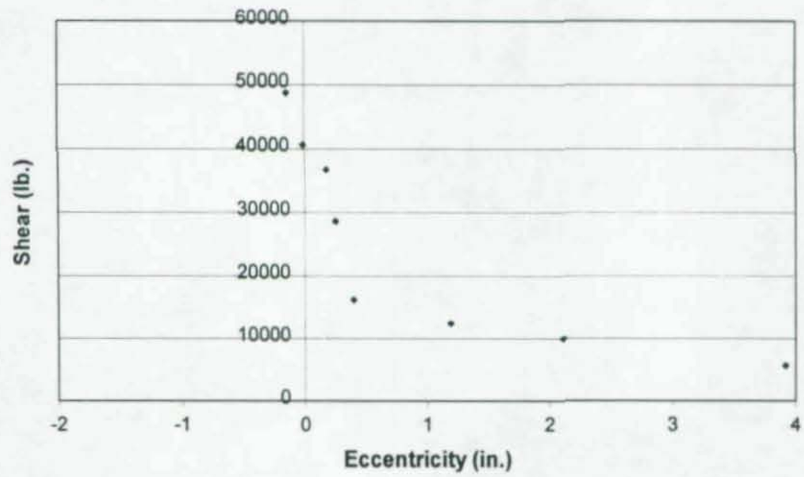
TEST 2-C



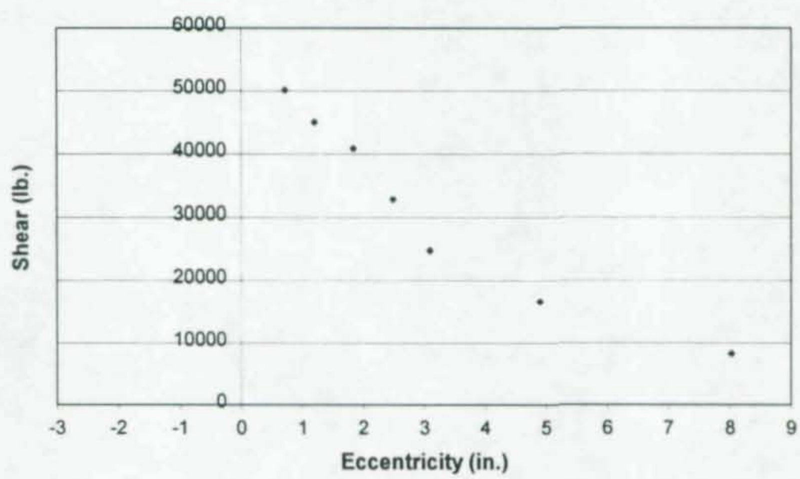
TEST 3-A



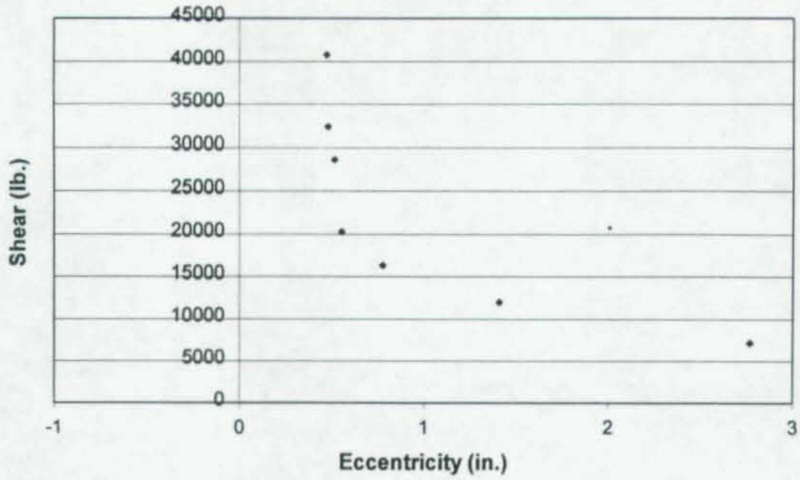
TEST 3-B



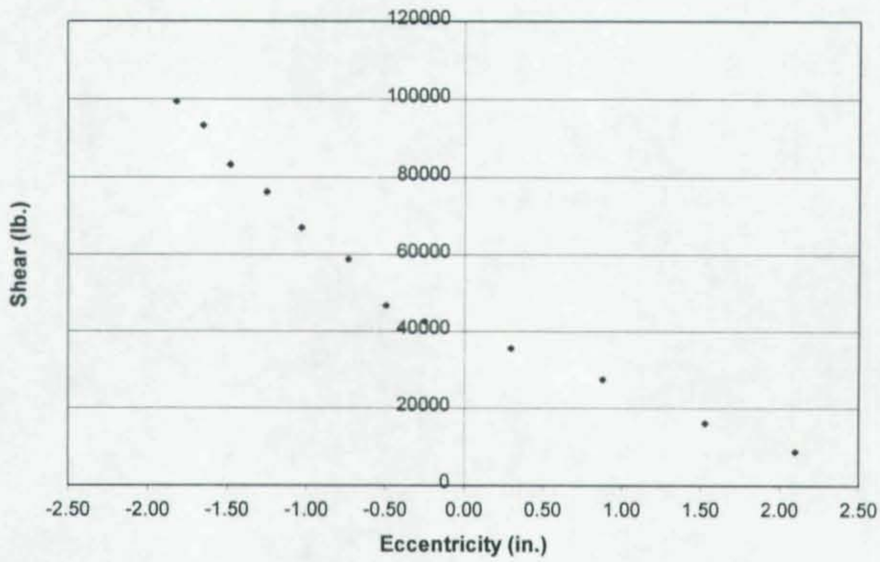
TEST 3-D



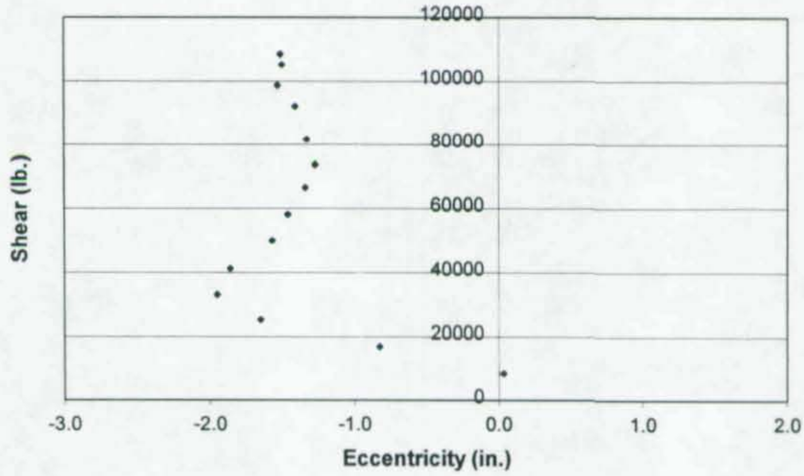
TEST 3-E



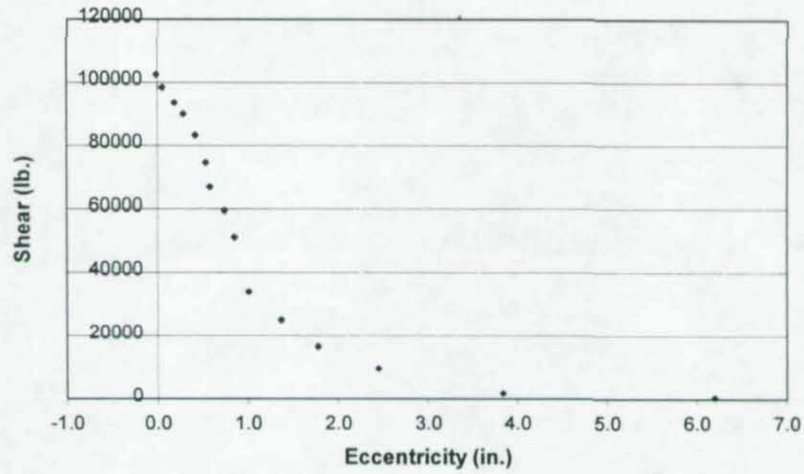
TEST 4-A



TEST 4-B



TEST 4-C



No shear-eccentricity graph is included for Test 3-C due to the very low failure load.

Also, the test failed before the eccentricity could approach a constant value.

APPENDIX H: AISC CRITICAL CAPACITIES

The following is an example of how the AISC critical capacities in Table 5.3 were determined. The process involves checking every limit state in Table 4-2 of the AISC HSS Connections Manual [8] for single plate connections. In addition, the web mechanism limit state of Equation 5.5 must be checked.

Example Calculation:

Test 1-U: AISC critical shear, V_{th}, using AISC eccentricities

The value reported in Table 5.3: **V_{th} = 31.5 kips**

1.) Bolt Shear by ultimate analysis: $R_n = C*A_b*F_v$ {HSS Equation 4-2}

$A_b = .4418$ in.^{2} (Table 8-11 of AISC Manual)

$F_v = 60$ ksi (Table 8-11 of AISC Manual)

$e_x = e_b = 4.86$ in. (Table 5.2)

$C = 1.19$ (Interpolation in Table 8-18 of AISC Manual with $e_x = 4.86$ and n = 3 bolts)

$R_n = C*A_b*F_v$

$= 1.19*.4418*60 = 31.5$ kips

R_n = 31.5 kips

2.) Bolt Bearing: $R_n = C(2.4*d_b*t_c*F_{uc})$ {HSS Equation 4-4}

$d_b = 0.75$ in.

$t_c = 0.371$ in. (Table 5.1)

$F_{uc} = 66.5$ ksi (Table 5.1)

$R_n = C(2.4*d_b*t_c*F_{uc})$

$= 1.19(2.4*0.75*0.371*66.5)$

$\mathbf{R_n = 52.8\ kips}$

3.) Gross Section Shear Yield: $R_n = L(k*t_c)(0.6*F_{yc})$ {HSS Equation 4-5}

$L = 9$ in.

$F_{yc} = 42.6$ ksi (Table 5.1)

$k = 1.0$ (for single plate connections)

$R_n = L(k*t_c)(0.6*F_{yc})$

$= 9(1.0*0.371)(0.6*42.6)$

$\mathbf{R_n = 85.3\ kips}$

4.) Net Section Shear Rupture: $R_n = [L - n(d_h + 1/16)](k*t_c)(0.6*F_{uc})$

$n = 3$ (number of bolts) {HSS Equation 4-6}

$d_h = 0.8125$ in. (d_b + 1/16-in.)

$R_n = [L - n(d_h + 1/16)](k*t_c)(0.6*F_{uc})$

$= [9 - 3(0.875)](1.0*0.371)(0.6*66.5)$

$\mathbf{R_n = 94.4\ kips}$

5.) Block Shear Rupture: $R_n = (k*t_c)[0.6*F_{uc}*L_s + F_{yc}*L_{eh}]$ For $0.6*L_s \geq L_t$

L_s = shear rupture length {HSS Equation 4-10}

$= 9 - 1.5 - 2.5(0.75 + .0625 + .0625) = 5.31$ in.

L_t = tension rupture length

$= 1.5 - 0.5625 = 0.94$ $0.6*L_s = 0.6*5.31 = 3.19 \geq 0.94$ O.K.

$L_{eh} = 1.5$ in.

$R_n = (k*t_c)[0.6*F_{uc}*L_s + F_{yc}*L_{eh}]$

$= (1.0*0.371)[0.6*66.5*5.31 + 42.6*1.5]$

$$\mathbf{R_n = 102\ kips}$$

6.) Weld Shear by ultimate analysis: $R_n = C*D*L$ {HSS Equation 4-17}

$a = 6.86$ in. (Table 4.1)

$e_x = e_w = a - e_b = 6.86 - 4.86 = 2.0$ in.

$C = 2.57$ (Interpolation in Table 8-38 of AISC Manual

with $e_x = 2.0$ and $n = 3$ bolts)

$D = 5$ (# of sixteenths of the fillet weld)

$$R_n = C*D*L$$

$$R_n = 2.57*5*9$$

$$= 116 \text{ kips}$$

$$\mathbf{R_n = 116\ kips}$$

7. Web Mechanism: $R_n = [((2*h/L) + (4*L/h) + 4*(3)^{1/2})(F_{yw}*t_w^2/4)(L)]/e_w$

$F_{yw} = 55.2$ ksi (Table 5.1) {Equation 5.5}

$t_w = 0.288$ in. (Table 5.1)

$h = h/t_w*t_w = 22.2*0.288 = 6.39$

$e_w = 2.0$

$$R_n = [((2*h/L) + (4*L/h) + 4*(3)^{1/2})(F_{yw}*t_w^2/4)(L)]/e_w$$

$$= [((2*6.39/9) + (4*9/6.39) + 4*(3)^{1/2})(55.2*0.288^2/4)(9)]/2.0$$

$$\mathbf{R_n = 73.0\ kips}$$

Therefore, bolt shear controls and $\mathbf{V_{th} = R_{n,\,min} = 31.5\ kips}$.

A similar procedure was done for all of the other tests and the results are summarized in Table A.1.

Table A.1: AISC critical capacities

AISC CRITICAL CAPACITIES (KIPS) – USING AISC ECCENTRICITIES							
Test	Bolt Shear	Bearing	Shear Yield	Shear Rupture	Block Shear	Weld	Web Mechanism
1-U	**36.1**	60.4	85.3	94.4	102.0	149.0	---
2-U	**61.7**	103.0	142.0	157.0	165.0	278.0	---
3-U	**31.5**	52.8	85.3	94.4	102.0	160.0	73.0
4-U	**80.9**	177.0	184.0	204.0	215.0	236.0	120.0
1-A,B	**34.1**	39.9	61.0	65.9	71.9	96.3	---
2-A,B,C (Flexible)	**68.6**	82.8	98.3	113.0	118.0	167.0	---
2-A (Rigid)	117.0	141.0	**98.3**	113.0	118.0	153.0	---
2-B (Rigid)	107.0	129.0	**98.3**	113.0	118.0	161.0	---
3-A,B,D	**38.9**	47.0	59.0	68.0	73.1	96.3	---
3-E	**32.4**	39.1	59.0	68.0	73.1	100.0	---
4-A,C	**86.1**	101.0	102.0	110.0	116.0	153.0	---
4-B	**78.7**	92.1	102.0	110.0	116.0	161.0	---
AISC CRITICAL CAPACITIES (KIPS) – USING MEASURED ECCENTRICITIES							
Test	Bolt Shear	Bearing	Shear Yield	Shear Rupture	Block Shear	Weld	Web Mechanism
1-U	**45.6**	76.4	85.3	94.4	102.0	136.0	---
2-U	**70.2**	118.0	142.0	157.0	165.0	278.0	---
3-U	44.6	74.8	85.3	94.4	102.0	137.0	**41.0**
4-U	**64.3**	141.0	184.0	205.0	215.0	263.0	181.0
1-A	**49.8**	58.3	61.0	65.9	71.9	81.3	---
1-B	**56.7**	66.4	61.0	65.9	71.9	78.2	---
2-A	**85.5**	103.0	98.3	113.0	118.0	167.0	---
2-B	**77.9**	94.0	98.3	113.0	118.0	167.0	---
2-C	**78.9**	95.3	98.3	113.0	118.0	167.0	---
3-A	83.2	101.0	**59.0**	68.0	73.1	75.2	---
3-B	84.8	102.0	**59.0**	68.0	73.1	75.2	---
3-D	80.1	96.7	**59.0**	68.0	73.1	76.0	---
3-E	81.6	98.6	**59.0**	68.0	73.1	76.6	---
4-A	121.0	142.0	**102.0**	110.0	116.0	133.0	---
4-B	145.0	170.0	**102.0**	110.0	116.0	128.0	---
4-C	124.0	145.0	**102.0**	110.0	116.0	132.0	---

The numbers that appear in bold are the numbers that are reported in Table 5.3.

APPENDIX I: PHOTOGRAPHS OF FAILURE MODES

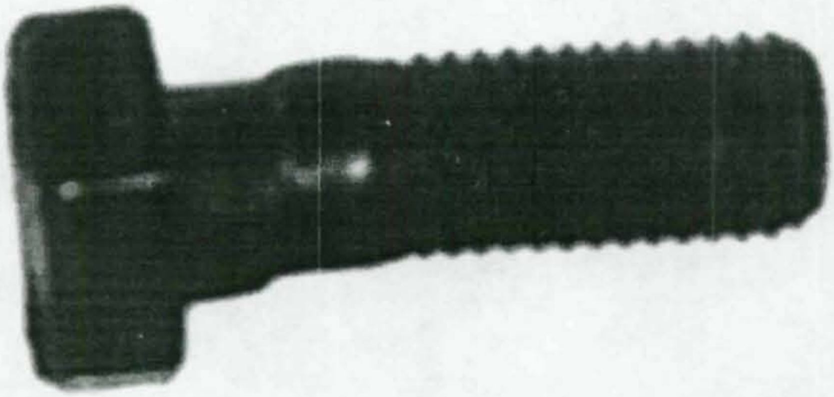


Figure H.1: Bolt Shear (Test 1-U)

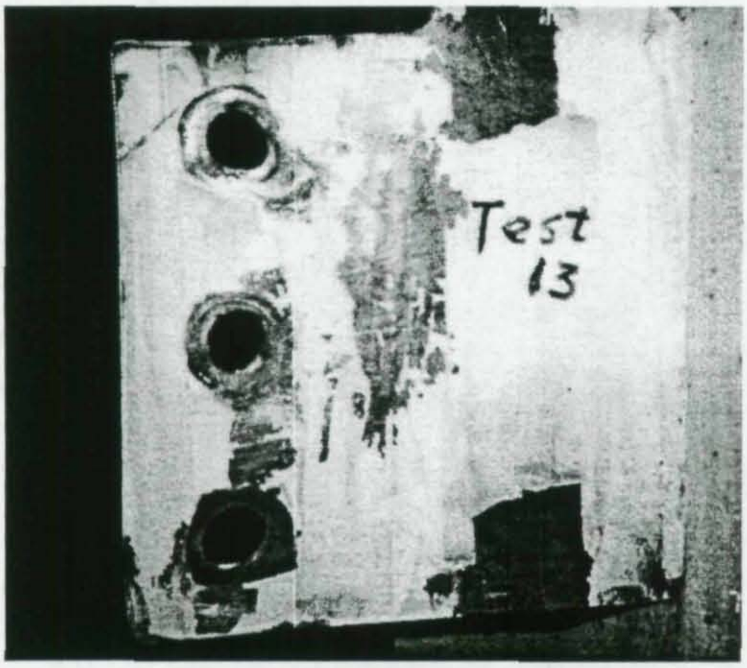


Figure H.2: Bolt Bearing (Test 1-A)

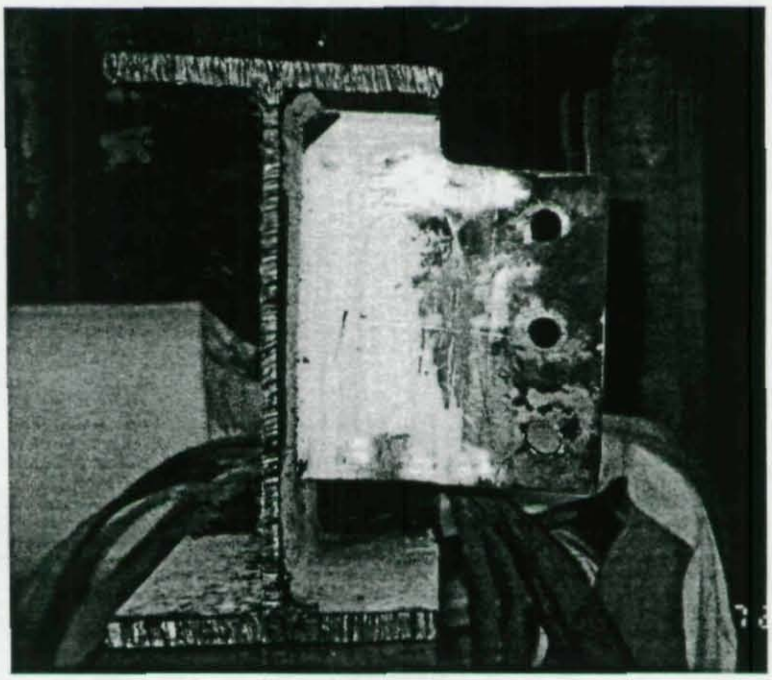


Figure H.3: Shear Yield (Test 1-A)

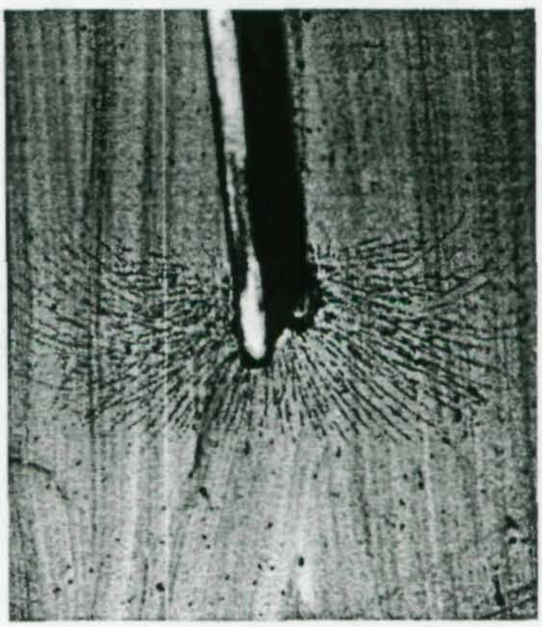


Figure H.4: Web Mechanism (Test 2-A)

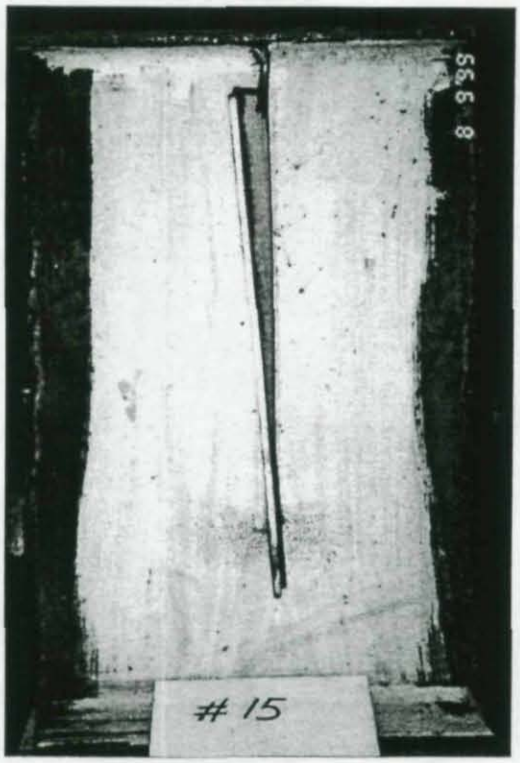


Figure H.5: Twist (Test 2-A)

APPENDIX J: TWIST LIMIT STATE EQUATION
(FINITE ELEMENT RESULTS)

As mentioned in Section 6.2, a finite element analysis correlation was performed on Test 1-U. This correlation was used to determine the proper eccentricity of the load with respect to the center of the tab cross section. This location is used to calculate the torsion on the rectangular cross section. This torsion is used to derive the twist limit state equation (Equation 5.5). The results of the F.E. analysis are shown in the following two tables. Table I.1 lists the results for when the load was applied to the shear tab face. Table I.2 lists the results for when the load was applied at 1/3 the thickness of the shear tab. As seen, F.E.A. strains correlate much closer with experimental strains for the load applied at 1/3 the thickness of the shear tab.

Figures I.1 and I.2 show the model of the shear tab used for the F.E. analysis and the mesh sized used. The point of loading was determined from eccentricity results obtained from strain gages mounted on the supported beam. The eccentricity of the shear force changes with increasing shear so four loads were chosen and the load was applied at the appropriate eccentricity for each load case.

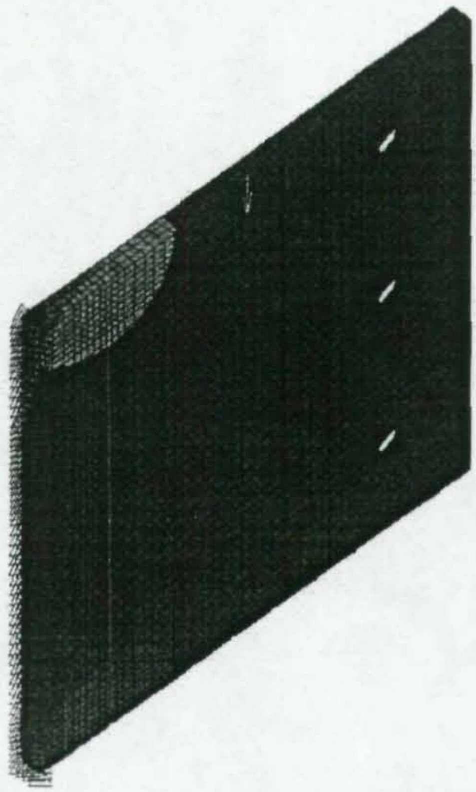


Figure I.1: F.E.A. model

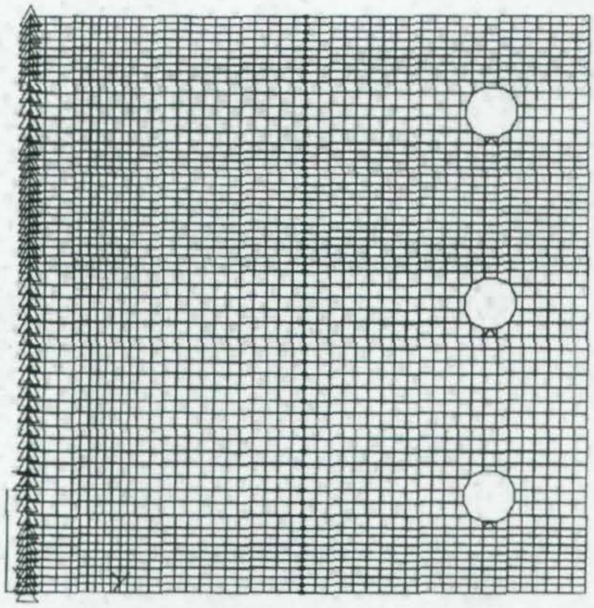


Figure I.2: F.E.A. mesh size and load location

Table I.1: Finite element results for torsion load location

FINITE ELEMENT SUMMARY				
Case 1: Load Applied to Face of Shear tab				
Gage	**Shear (kips)**	**Eccentricity (in.)**	**Experimental Strain (μE)**	**F.E.A. Strain (μE)**
C	5.06	-1.7	69	49
C	10.64	-2.1	114	87
C	22.11	-2.7	361	155
C	40.82	-3.1	516	196
D	5.06	-1.7	44	213
D	10.64	-2.1	175	419
D	22.11	-2.7	443	919
D	40.82	-3.1	867	1030
J	5.06	-1.7	-91	-48
J	10.64	-2.1	-193	-85
J	22.11	-2.7	-522	-155
J	40.82	-3.1	-691	-188
K	5.06	-1.7	-3	-217
K	10.64	-2.1	-126	-427
K	22.11	-2.7	-486	-918
K	40.82	-3.1	-888	-1340

Table I.2: Finite element results for torsion load location

FINITE ELEMENT SUMMARY				
Case 2: Load Applied at 1/3 the Thickness of Shear Tab				
Gage	**Shear (kips)**	**Eccentricity (in.)**	**Experimental Strain (μE)**	**F.E.A. Strain (μE)**
C	5.06	-1.7	69	104
C	10.64	-2.1	114	198
C	22.11	-2.7	361	409
C	40.82	-3.1	516	564
D	5.06	-1.7	44	213
D	10.64	-2.1	175	419
D	22.11	-2.7	443	919
D	40.82	-3.1	867	1030
J	5.06	-1.7	-91	-104
J	10.64	-2.1	-193	-199
J	22.11	-2.7	-522	-409
J	40.82	-3.1	-691	-570
K	5.06	-1.7	-3	-160
K	10.64	-2.1	-126	-313
K	22.11	-2.7	-486	-664
K	40.82	-3.1	-888	-953

APPENDIX K: DERIVATION OF TWIST LIMIT STATE EQUATION

Maximum torsional shear stress for a rectangular cross section:

$$\tau_{max} = (k_1 * T)/(b * t^2) \qquad (1)$$

where : b = plate depth

t = plate thickness

T = torsion on the plate (shear force multiplied by the eccentricity from the plate center of gravity

k_1 = a function of the b/t ratio of the section (k_1 = f (b/t):

$k_1 = 3.00$ for $b/t \geq 5 \rightarrow k_1 = 3.00$

b/t ratios:	1/4-in. tab	(b/t) = 15/.25 = 60
		(b/t) = 9/.25 = 36
	3/8-in. tab	(b/t) = 15/.375 = 40
		(b/t) = 9/.375 = 24
	1/2-in. tab	(b/t) = 15/.5 = 30

All b/t ratios exceed 5, therefore $k_1 = 3.00$ for all tests

Maximum direct shear stress for a rectangular cross section:

$$\tau_{max} = 3/2\ (V/A) \qquad (2)$$

Shear yield occurs at:

$$\tau_y = 0.6\ F_y \qquad (3)$$

Setting Equation (3) equal to the summation of Equations (1) and (2) and substituting $k_1 = 3.00$:

$$0.6\ F_y = (3*T)/(b*t^2) + 3/2\ (V/A) \qquad (4)$$

Torsion from the finite element analysis:

$$T = 1/6*t*V \qquad (5)$$

Substitute (5) into (4) and substituting A=b*t and solving for V:

$$V_{max} = V_t = 0.3*L*T*F_y \quad \rightarrow \quad \text{Equation 6.1}$$

APPENDIX L: DESIGN EXAMPLE

The following is an example of how the design shear strengths, V_n, in Table 7.1 were calculated. The proposed design procedure for extended shear tabs is followed, including the proposed new eccentricities for the shear reaction. The following example is for the Group 1 unstiffened test.

Step 1: Determine the distance *a* from the centroid of the weld group to the bolt line.

Solution: For an unstiffened W8X31 support column, $a = 6.86$ in.

This distance includes 3.86 in. from support web to flange tips plus 3-in. from the flange tips to the bolt line.

Step 2: Determine the tab thickness, *t*, for stability.

Solution: For an unstiffened shear tab, the thickness required is the largest from Equations 4.8 and 4.9.

Equation 4.8: $t \geq (Va^2/12000L) = (60*6.86^2)/(12000*9) = 0.297$ in.

Equation 4.9: $t \geq L/64 = 9/64 = 0.141$ in.

$t \geq 1/4$-in. $= 0.25$ in.

t provided $= 0.371$ in. O.K.

Step 3: Determine the eccentricity, *e*, of the shear force relative to the bolt line.

Solution: $e = 0.5a$ for unstiffened connections when the supporting member has $h/t_w \leq 35$ ($h/t_w = 22.2$ for a W8X31)

$e = 0.5a = 0.5*6.86 = 3.43$ in.

Step 4: Using AISC criteria, determine the critical limit state based on:

Solution: 1.) Bolt Shear $R_n = C*A_b*F_v = 1.61*.4418*60 = 43$ kips

R_n = 43 kips

2.) Bolt Bearing $R_n = C(2.4*d_b*t_c*F_{uc})$

(in the tab) $= 1.61(2.4*0.75*0.371*66.5) = 72$ kips

R_n = 72 kips

3.) 90% of Shear Yield $R_n = 0.9*L(k*t_c)(0.6*F_{yc})$

(on the gross area) $= 0.9*9(1.0*0.371)(0.6*42.6) = 77$ kips

R_n = 77 kips

4.) Shear Rupture $R_n = [L - n(d_h + 1/16)](k*t_c)(0.6*F_{uc})$

(of the net section) $= [9 - 3(0.875)](1.0*0.371)(0.6*66.5)$

$= 94$ kips

R_n = 94 kips

5.) Block Shear Rupture $R_n = (k*t_c)[0.6*F_{uc}*L_s + F_{yc}*L_{eh}]$

$= (1.0*0.371)[0.6*66.5*5.31 + 42.6*1.5]$

$= 102$ kips

R_n = 102 kips

Step 6: For unstiffened connections to column webs, also determine the yield line mechanism by Equation 5.5.

Solution: The example is for the unstiffened W8X31 support column. Therefore, the yield line mechanism limit state must be figured.

$R_n = [((2*h/L) + (4*L/h) + 4*(3)^{1/2})(F_{yw}*t_w^2/4)(L)]/e_w$

$= [((2*6.39/9) + (4*9/6.39) + 4*(3)^{1/2})(55.2*0.288^2/4)(9)]/3.43$

$= 43$ kips

$R_n = 43$ kips

Therefore, $V_n = R_{n,min} = 43$ kips

$V_n = 43$ kips

Step 7: Use a weld size equal to 3/4 of the plate thickness.

Solution: Weld size = 3/4*0.371 = 0.75*0.371 = 0.278 in.

Weld size provided = 5/16 in. = 0.313 in. O.K.

Step 8: Examine the beam web for bolt bearing.

Solution: $R_n = C(2.4*d_b*t_c*F_{uc})$

$= 1.61(2.4*0.75*0.515*50) = 75$ kips

$R_n = 75$ kips (Does not control)

Step 9: Accept the design.

Solution: The W8X31 support column and 3/8-in. extended shear tab has a nominal capacity, V_n, of 43 kips.

Note:

It is not the focus of this study to address the design of supporting members in shear tab connections. However, it is apparent that the resulting shear and moment forces from the connections do affect the design of the supporting members and will need to be considered.

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