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DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

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DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

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AND ENVIRONMENTAL SCIENCE

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Shear vs. Bolt Line Deflection
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Shear vs. Eccentricity

ABSTRACT

The single plate framing connection is one of the simplest and most economical beam to column or beam to girder connections. The connection is comprised of a single plate, with either prepunched or predrilled bolt holes that is shop welded to the supporting element. During erection, the beam with prepunched holes is brought into position and field bolted to the framing plate. The behavior of such a connection is rather complex, and involves the specification of numerous parameters for its design. Different procedures regarding the behavior and design of this connection have been suggested by different researchers, which give different values of the design parameters. In order to arrive at a common and rational procedure to characterize the behavior and design of this connection, full scale beam tests on 2-, 4- and 6-bolt connections have been conducted in this study. The beam tests were further supplemented by a series of single bolt lap tests followed by tensile coupon tests to investigate the effect of certain key parameters on the connection ductility. The test results are used to characterize the actual behavior of single plate framing connections. This is followed by the development of a design procedure for such a connection.

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

CHAPTER 1

INTRODUCTION

1.1 Description of Single Plate Connection

The single plate framing connection or shear tab is one of the simplest and most economical beam to column or beam to girder connections. Shear tabs are primarily used to transfer beam end reactions to the supporting elements. Fig. 1.1 shows typical single plate framing connections.

The connection is comprised of a single plate, with either prepunched or predrilled bolt holes, that is shop welded to the supporting beam or column. During erection, the beam with prepunched holes is brought into position and field bolted to the framing plate. The support to which the shear tab is welded may be either rigid such as a column flange, or flexible such as a column web, tube face, spandrel beam, or plate girder. The weld is usually the fillet type made with a single pass using E70XX electrodes. For the shear plate usually A36 grade steel is used. Bolt holes in the shear plate can be

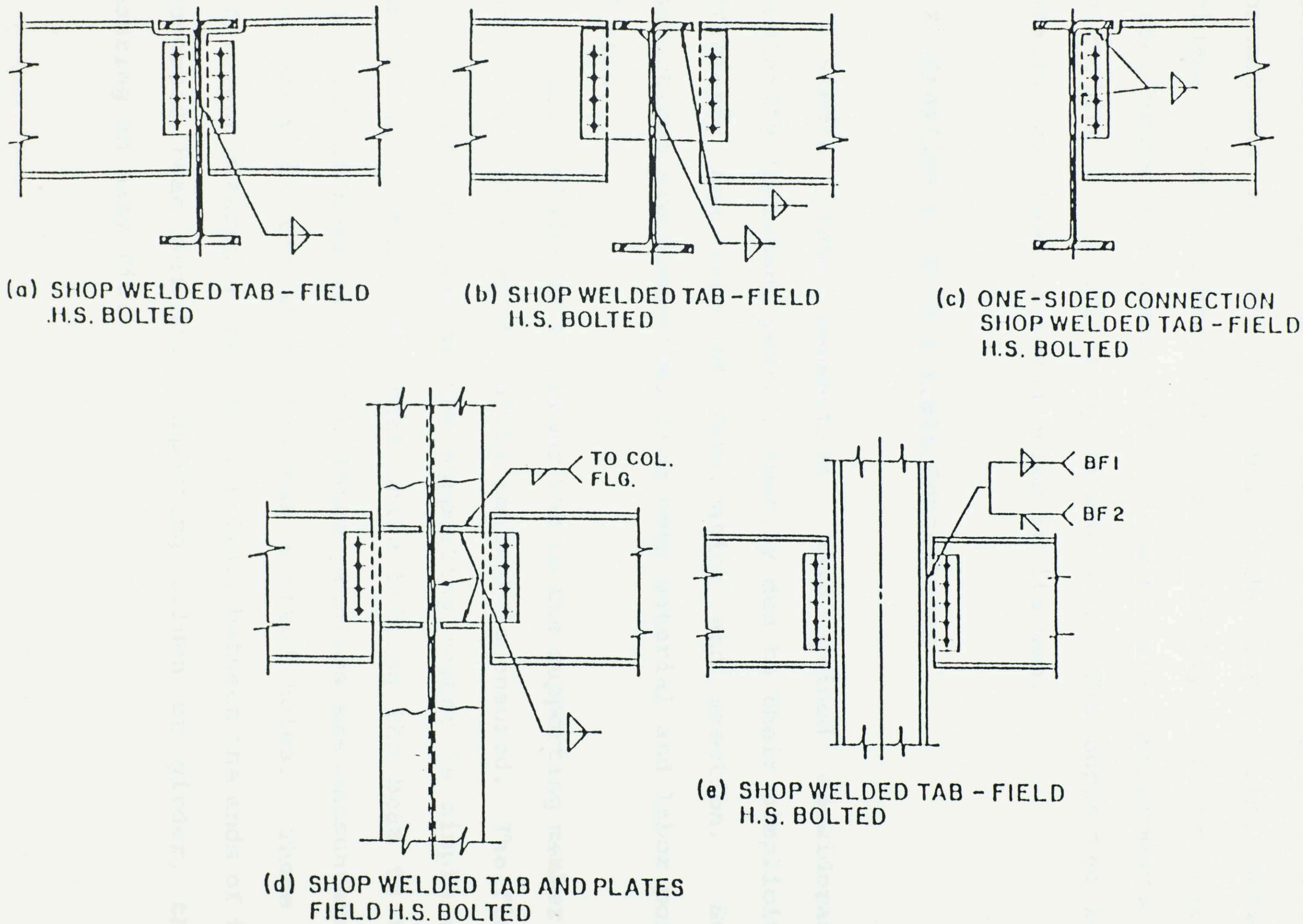

Fig. 1.1 Single Plate Framing Connections From Richard, 1

either standard round or slotted holes depending on the type of connection. Standard round holes are often used at column connections in order to control the bay spacing between columns. Short slotted holes are typically used for beam to girder connections because they allow for minor adjustments due to rolling and fabricating tolerances. The supported beam usually has standard round holes in its web.

1.2 Benefits of Single Plate Connections

Single plate connections have gained considerable popularity in recent years primarily due to their simplicity, efficiency and ease of fabrication and erection. Such connections are economical from both material and labor point of view.

Since welding of the shear tab to the supporting member is done in the shop, good quality control is ensured. The field erection of the beam to the supporting member is simple and convenient in the sense that while bolting the beam to the shear tab no beam length tolerance problems are encountered, especially if the shear tab has slotted holes. There is sufficient clearance (typically 1.5in.) between the ends of the supported beam and the supporting column or girder, thus ensuring an easy fit.

1.3 Objectives and Scope

1.3.1 Background Preview

As per the American Institute of steel Construction (AISC) Allowable Stress Design (ASD) manual (2) and the Load Resistance Factor Design (LRFD) manual (3) provisions, single plate connections should be flexible enough to accommodate end rotations of unrestrained (simple) beams, and so inelastic action in the connection is permitted.

The single plate connection is considered to be a simple and flexible connection primarily meant to transfer beam end shear reaction to the supporting members. In addition, the connection should also have enough rotational capacity, i.e., ductility to accomodate the end rotation demand of a simply supported beam.

The beam end rotational capacity of single plate connections is essentially derived from :

a) bolt deformation in shear,

b) plate and/or beam web hole distortion,

c) out-of-plane buckling of the plate and/or beam web,

d) bolt slippage if the bolts are not in bearing at the time of initial loading, and

e) in-plane yielding of the connection plate.

Another important aspect of the shear tab connection is the reaction eccentricity. This is defined as the distance from the inflection point of the moment diagram to the bolt line, and depends on a number of factors such as the number of bolts, the dimensions and material of the shear tab, the amount of beam end rotation and the relative rigidity of the supporting member or column. This eccentricity is used for proportioning connection parts in some design methods.

1.3.2 Previous Investigations

The behavior of single plate connections has been studied in the past by several investigators. For details, see Chapter 2. Among the many researchers, the design procedures developed by Richard (4) and Astaneh (5), in the year 1980 and 1989 respectively are of prominence and have become the center of attention of this current research. Although their designs are considered to be safe and conservative, they contrasted drastically in some important aspects. These two design procedures often resulted in quite different designs, particularly for shallower connections in which fewer bolts are required. The major differences in the two design procedures were in identifying the portion of the connection primarily responsible for the beam end rotation and the factors governing the determination of the reaction eccentricity.

1.3.3 Objectives

With two entirely different design procedures apparently resulting from the complex behavior of shear tab connection (involving numerous variables) as well as the possible drawbacks in test procedures of the previous researchers, the University of Oklahoma with the support of AISC set forward to conduct full scale simple span beam tests in its research facilities at Fears Structural Engineering Laboratory. The principal objective of the tests that followed was to develop rational design principles for single plate framing connections. A total of six simple span beam tests were performed, which involved 2, 4, and 6-bolt connections. These were accomplished using three different beams, with each beam being tested once, then turned over and tested again with a new set of connection plates. Connection plates with short slotted holes were used in the second set of tests involving 4-bolt and 6-bolt connections. In each test, the behavior of the shear tab connection was studied by subjecting the beam to two-point static loading essentially to simulate the effect of distributed load. The tests were designed such that the most important parameters necessary to understand the behavior of the shear tab connection could be measured and the most common failure modes identified. The beam tests were further supplemented by single bolt lap tests and tensile coupon tests to verify certain key parameters governing the design of such connections.

1.3.4 Scope

In order to restrict the number of beam tests, only two design parameters were kept constant. This was accomplished by using 3/8 in. thick connection plates of the same yield strength and 3/4 in. diameter bolts in all the beam tests.

CHAPTER 2

LITERATURE REVIEW

From a historical standpoint, the first standard design procedure for single plate framing connection was a simplified one, and had an apparent failure-free performance record. It was assumed that each bolt shares an equal portion of the total shear load, and relatively free rotation occurs between the end of the beam and the supporting member. Both the plate and weld were generally designed for shear and moment equal to the shear times the distance from the bolt line to the weld. Whether the connection possessed adequate ductility to accommodate rotations equal to those at the end of the simply supported beam, the possible sources of ductility, the relative degree of flexibility in the supporting member, and the failure modes of the connection, all remained a mystery until an extensive research program was pursued by Richard (4) at the University of Arizona in the late seventies.

Richard (4) conducted five full scale beam tests on 2, 3, 5 and 7-bolt connections. As shown in Fig. 2.1.1, his test setup consisted of a beam attached to a column through a shear tab connection at one end and propped at the other. The

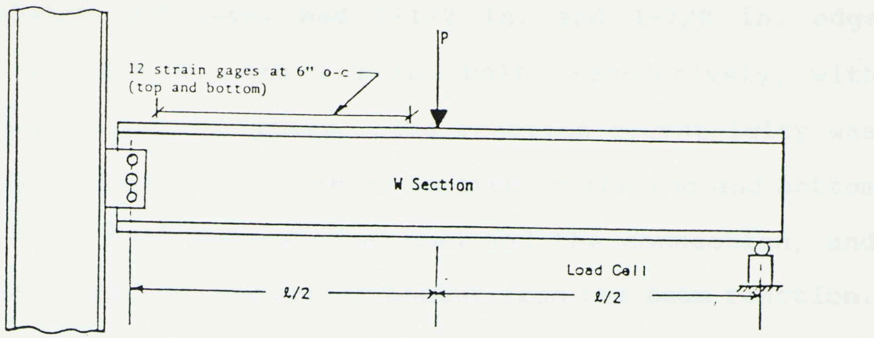

Fig. 2.1.1 Richard's Test Setup Richard, 4

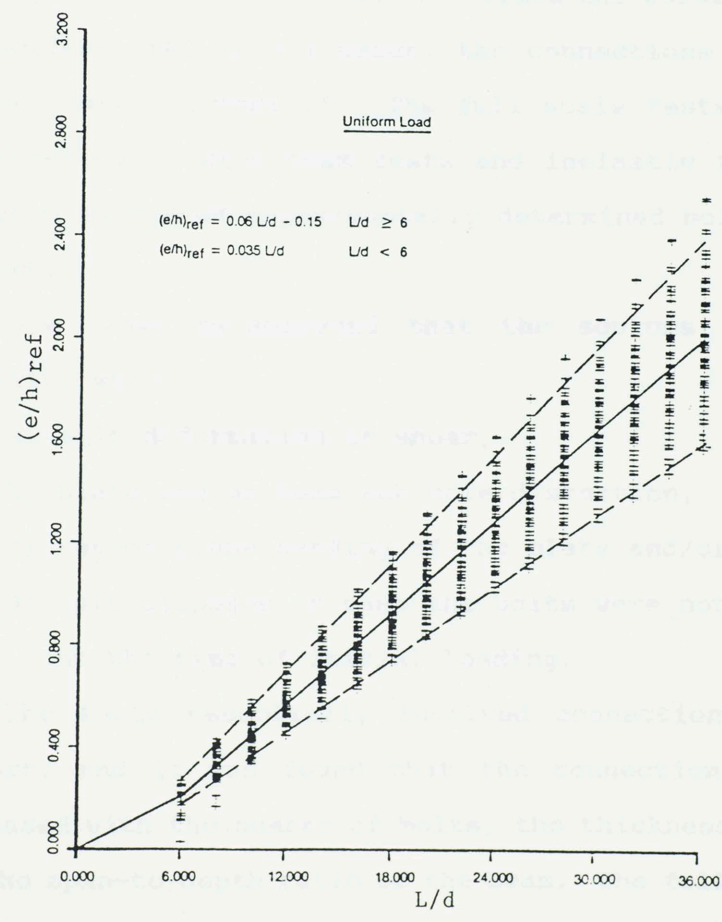

Fig. 2.1.2 Richard's Design Curve With ± 20% Bounds Richard, 4

framing plates and beams had 1-1/2 in. and 1-7/8 in. edge distances for 3/4 in. and 7/8 in. bolts respectively, with punched holes 3 in. on center. The reaction eccentricity was measured by means of strain gages located on the top and bottom flanges of the beam between the load and the connection, and also by computing the connection moment from the beam reaction. Moment-rotation curves generated in the tests and the beam line method of analysis formed the basis of Richard's design. All beams were loaded to at least 1.5 times the working load on the connections, and in all cases, the connections were found to perform "satisfactorily". The full scale tests were further supplemented by stub beam tests and inelastic finite element analyses that used experimentally determined bolt deformation results.

The tests established that the sources of connection ductility were:

a) bolt deformation in shear,

b) plate and/or beam web hole distortion,

c) out-of-plane bending of the plate and/or beam web,

d) bolt slippage in case the bolts were not in bearing at the time of initial loading.

The tests essentially involved connections to a rigid support, and it was found that the connection eccentricity increased with the number of bolts, the thickness of the plate and the span-to-depth ratio of the beam. The following failure modes were identified:

a) shear failure of the bolt,

b) bearing failure of the plate, and

c) transverse tension tearing of the plate.

Richard concluded that single plate connections can develop a significant end moment in the beam and supporting member. The maximum connection moment was found to occur around 1.5 times the working load. The magnitude of the moment was found to depend upon:

a) the number, size, and configuration of bolt pattern;

b) the thickness of the plate and/or beam web;

c) the beam span to depth ratio;

d) the loading (whether uniform or concentrated); and

e) the relative flexibility of the supporting member.

Based on the test results and observations, the design procedure that followed differed greatly from the previous simplified design procedure, but could be extended to a wide variety of single plate framing connections.

Richard's method assumed that the bolt group would withstand sufficient rotation to release part of the beam end moment. By limiting the connection plate thickness to half the bolt diameter and maintaining a 2 in. edge distance, the beam end rotation was allowed to be accommodated by bolt hole deformation. This ensured that bearing deformation would occur before the bolt shear capacity was exceeded, protecting the bolts from shear failure due to moment. So they could be designed only for the beam end shear reaction and not moment.

Richard developed a formula to determine the effective eccentricity of the connection, so that he could design the plate and weld to behave elastically under combined shear and bending. In essence, Richard assumed that the bolt group would be the ductile link to release the beam end moment, and designed the shear plate to remain elastic.

Richard's (4) design procedure is as follows:

1. Select A36 plate.

Plate thickness = Beam web thickness ± 1/16 in.

2. Compute number of bolts required based upon allowable beam shear and allowable bolt loads.

3. Use design curve to find $(e/h)_{ref}$ from beam L/d ratio. Compute h from:

$$h = (n-1) \times p \tag{2.1}$$

where n = number of bolts and p = bolt pitch

Find the eccentricity from the formula:

$$e/h = (e/h)_{ref} \times (n/N) \times (S_{ref}/S)^{0.4} \tag{2.2}$$

where N = 5 for 3/4 in. and 7/8 in. bolts, and
= 7 for 1 in. bolts;
S_{ref} = 100 for 3/4 in. bolts,
= 175 for 7/8 in. bolts, and
= 450 for 1 in. bolts;
S = Section modulus of the beam, and
$(e/h)_{ref}$ is obtained from the design curve in Fig. 2.1.2.

4. Compute the moment at the weldment:

$$M = V \times (e + a) \tag{2.3}$$

where V = beam shear force,

e = eccentricity from step 3, and

a = distance from the bolt line to the weldment (typically 3 in.)

5. Check the plate normal and shear stresses, respectively, from:

$$f_b = \frac{M}{{}^1/_4 tb^2} \leq 0.66\ f_y \tag{2.4}$$

$$f_v = \frac{V}{bt} \leq 0.40\ f_y \tag{2.5}$$

where t and b are the plate thickness and depth, respectively.

6. Design the weldment based upon the resultant (f_r) of the normal and shear stresses from step 5, which is given by:

$$f_r = (f_b^2 + f_v^2)^{0.5} \tag{2.6}$$

Richard's full scale tests were non-destructive since they involved loading the beams to only 1.5 times the service load, i.e., loading only in the elastic range. Consequently, it was felt that his test results were incomplete and did not provide sufficient information regarding ultimate strength and failure modes of the connection. Moreover, the inelastic finite element program used by Richard could only provide useful information on the state of strain and/or stress. The program was not capable of predicting failure modes and strengths such as weld fracture, bolt fracture, fracture of net section or fracture of the edge distance. The finite element program was used to simulate moment-rotation response. Again, similar to full scale tests, in the finite element analyses the maximum load was about 1.5 times the service load of the beams.

In order to identify the limit states of strength and to verify the validity of the design procedures that were developed and proposed by Richard, Astaneh (5) conducted five full scale tests on shear tab connections at the University of California, Berkeley. As shown in Fig. 2.2, Astaneh's test setup included two actuators. The actuator which was close to the connection was force controlled and provided the bulk of the shear force in the connection. The second actuator which was displacement controlled, provided and controlled the beam end rotation.

The bolt holes in all test specimens were standard round punched holes spaced 3 in. on center. All bolts were tightened to 70% of proof load using the turn-of-the-nut method. The yield stress and ultimate strength for materials of A36 shear tabs were 35.5 ksi and 61 ksi respectively (established from coupon tests). The edge distance in the horizontal as well as vertical direction was 1-1/2 in. with A325-N bolt connections and 1-1/8 in. with A490-N bolt connections. In all the tests, Astaneh (5) used the shear-rotation relationship shown as curve "abcd" in fig. 2.3 and tested to failure. Segment "ab", "bc" and "cd" corresponded to the elastic behavior, inelastic behavior and strain hardening of beam respectively. Points "b" and "c" were established considering the beam midspan moment as

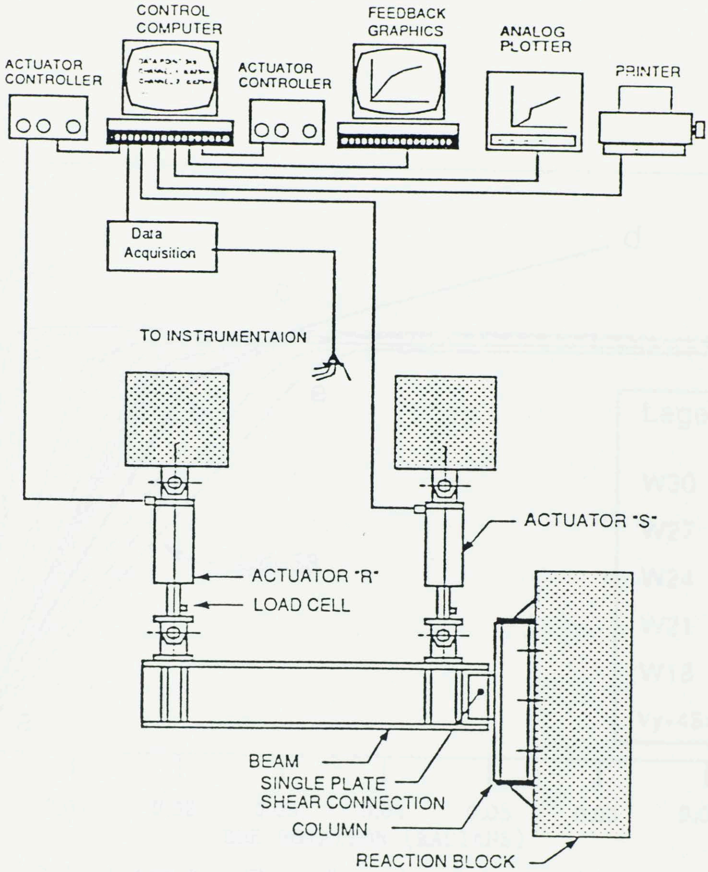

Fig. 2.2 Astaneh's Test Setup Astaneh, 5

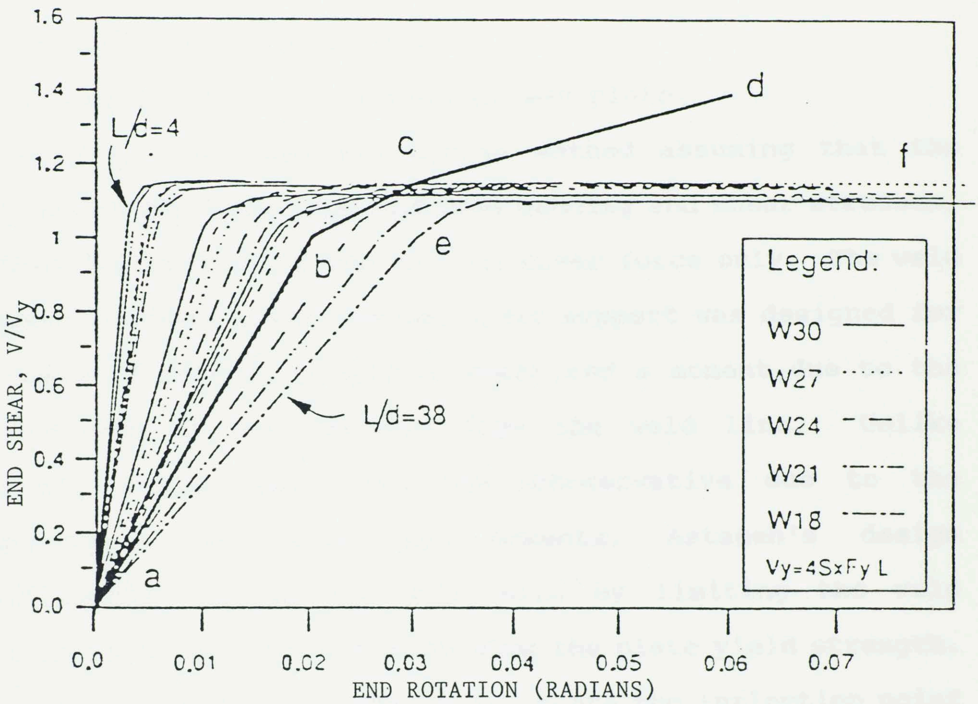

Fig. 2.3 Astaneh's Shear-Rotation Relationship   Astaneh, 5

yield and plastic moments respectively, and the corresponding end rotations as 0.02 and 0.03 rad. respectively. The following failure modes were identified:

a) shear failure of bolts,

b) yielding of the plate gross area,

c) fracture of the plate net area,

d) fracture of welds, and

e) bearing failure of beam or web plate.

Astaneh developed his design method assuming that the plate will yield under the combined bending and shear stresses, and then designed the plate for the shear force only. The weld between the shear plate and the rigid support was designed for the combined effects of direct shear and a moment due to the eccentricity of the reaction from the weld line. Unlike Richard's design which was over-conservative due to the establishment of large end moments, Astaneh's design significantly reduced the weld size by limiting the weld requirement to that needed to develop the plate yield strength. He developed a different formula to locate the inflection point of the moment diagram based on the movement of the point of inflection toward the support as the shear force was increased. The bolts were then designed for the combined effects of direct shear and moment due to the eccentricity of the reaction from the bolt line. In essence, Astaneh assumed that the plate will be the ductile link to release the beam end moment, and designed the bolt group to remain elastic.

The AISC adopted Astaneh's method and included it in the 9th edition ASD Manual of Steel Construction (2). Astaneh's (5) design procedure is as follows:

1. Calculate number of bolts n required to resist combined effects of shear, R, and moment, Re_b using Table X of AISC-ASD Manual of Steel Construction (2).

Find eccentricity from bolt line e_b (in.) from:

$$e_b = (n-1) - a \quad \text{(rigid support)} \tag{2.7}$$

$$e_b = \text{Max of } [(n-1) - a],\ a \quad \text{(flexible support)} \tag{2.8}$$

2. Calculate required gross area, A_{vg} of plate from:

$$A_{vg} \geq R/0.4F_y, \quad F_y = \text{plate yield strength} \tag{2.9}$$

Use A36 plate satisfying the following requirements:

a) edge distance $\geq$ (1.5)(bolt diameter d_b),

b) plate length $L_p \geq 2a$,

c) plate thickness $t_p \leq d_b/2 + 1/16$ in.,

d) $t_p \geq A_{vg}/L_p$, and

e) bolt spacing = 3 in.

3. Calculate the allowable shear strength, R_{ns}, of the effective net area from:

$$R_{ns} = [L_p - n(d_b + 1/16)(t_p)(0.3F_u) \tag{2.10}$$

$R_{ns} \geq R$, where F_u = plate ultimate strength

4. Calculate the actual allowable plate shear yield strength, R_o of the gross area from:

$$R_o = L_p t_p (0.4F_y) \tag{2.11}$$

Design fillet welds for the combined effects of shear, R_o, and moment, $R_o e_w$ using Table XIX of the AISC-ASD Manual of

Steel Construction (2).

$$e_w = \text{Max of } n, a \qquad (2.12)$$

For A36 steel and E70XX electrodes, limit weld size to 0.75 times the plate thickness, t_p, (which is sufficient to develop the plates).

5. Check bearing capacity of bolt group from:

$$(n)(t)(d_b)(1.2F_u) \geq R \qquad (2.13)$$

6. Check block shear failure in case of coped beam.

The most recent investigation into the shear tab connection to a rigid support was conducted by Owens and Moore (6) in the United Kingdom. The test setup consisted of an inverted "H" frame, which included a test beam connected to columns at each end by shear tabs. A two-point concentrated load was applied to the test beam by placing two hydraulic jacks at positions along the beam that gave the same elastic end rotation as uniformly distributed load. The testing scheme was split up into two phases: elastic test and test to failure. The following failure modes were identified:

a) combined shear and moment failure of the net section at the bolt area (typically for short plates),

b) combined shear and moment failure of the plate at the weld line (typically for long plates),

c) torsional-flexural buckling of the plate.

Another research was conducted by Sherman and Ales (7) at the University of Wisconsin, Milwaukee on shear tab connections to tubular columns, essentially considered as flexible supports. The researchers found that the reaction eccentricity was most affected by the width to thickness ratio of the tube wall and the span to depth ratio of the beam. The following failure modes were identified:

a) yielding of the gross area of the tab,

b) bearing failure of the tab,

c) fracture and yielding of the welds,

d) punching shear failure of the tube wall,

e) surface tearing of tube wall material,

f) lateral buckling of the tab, and

g) shear yielding and fracture of the bolts.

The work of the past researchers, Richard (1) (4) (8) (9) and Astaneh (5) (10) (11) became the focus of attention for this current research since comparison of the two methods revealed some striking similarities and also some fundamental differences. Both researchers agreed upon the use of A36 material for the shear tab. Both allowed the use of A325 and A490 bolts, both snug tight and fully torqued. Both used 3 in. bolt spacing and allowed either standard holes or short slots. And finally, both acknowledged the presence of shear and rotational yielding in the area between the welds and bolts as well as at the bolt line. The fundamental differences were mainly focused around the location of the point of inflection of the moment diagram and hence the load eccentricity, limiting plate thickness to bolt diameter ratio and the effect of edge distance on plowing of the bolts before bolt shear failure occurs. The most striking difference was focused around whether the bolt group needs to be designed for direct shear or a combination of shear and moment due to load eccentricity. The limiting weld thickness to develop the plate yield capacity and the relative efficiency of short slotted holes and standard round holes in the shear tab for improving the connection ductility were also considered in the current research.

CHAPTER 3

EXPERIMENTAL STUDY

3.1 Test Setup and Instrumentation

The general test setup used is shown in Fig. 3.1. For the sake of convenience, the actual test setup was inverted, i.e., the loads were applied upward. However, the description of the test setup that follows is written as if the test loads were applied downward, as would be the typical case. Fig. 3.2 shows details of the 2-bolt connection specimen. The 4- and 6-bolt connection specimens were the same except for the number of bolts, beam size, beam length, and hydraulic cylinder locations.

The beam was bolted to a single plate on either side. The plates were shop welded to the column flanges by the flux cored arc welding process using E70XX electrodes.

The main parameters to be measured included beam end shears and rotations, bolt line deflections, beam midspan deflection and the location of inflection points of the moment diagram from the bolt line.

Beam end shears were determined by measuring the applied

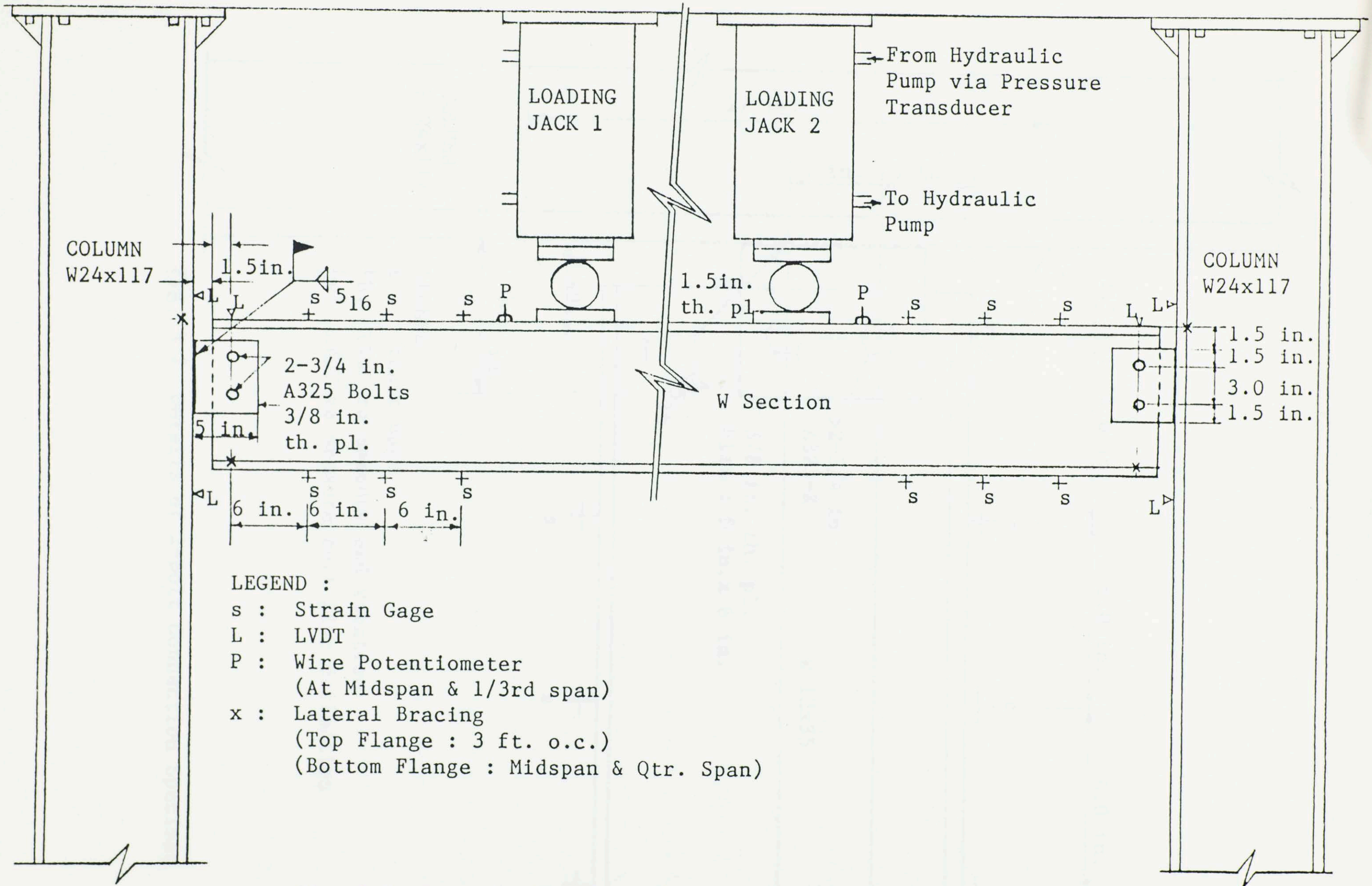

Fig. 3.1 General Test Setup for Single Plate Connections

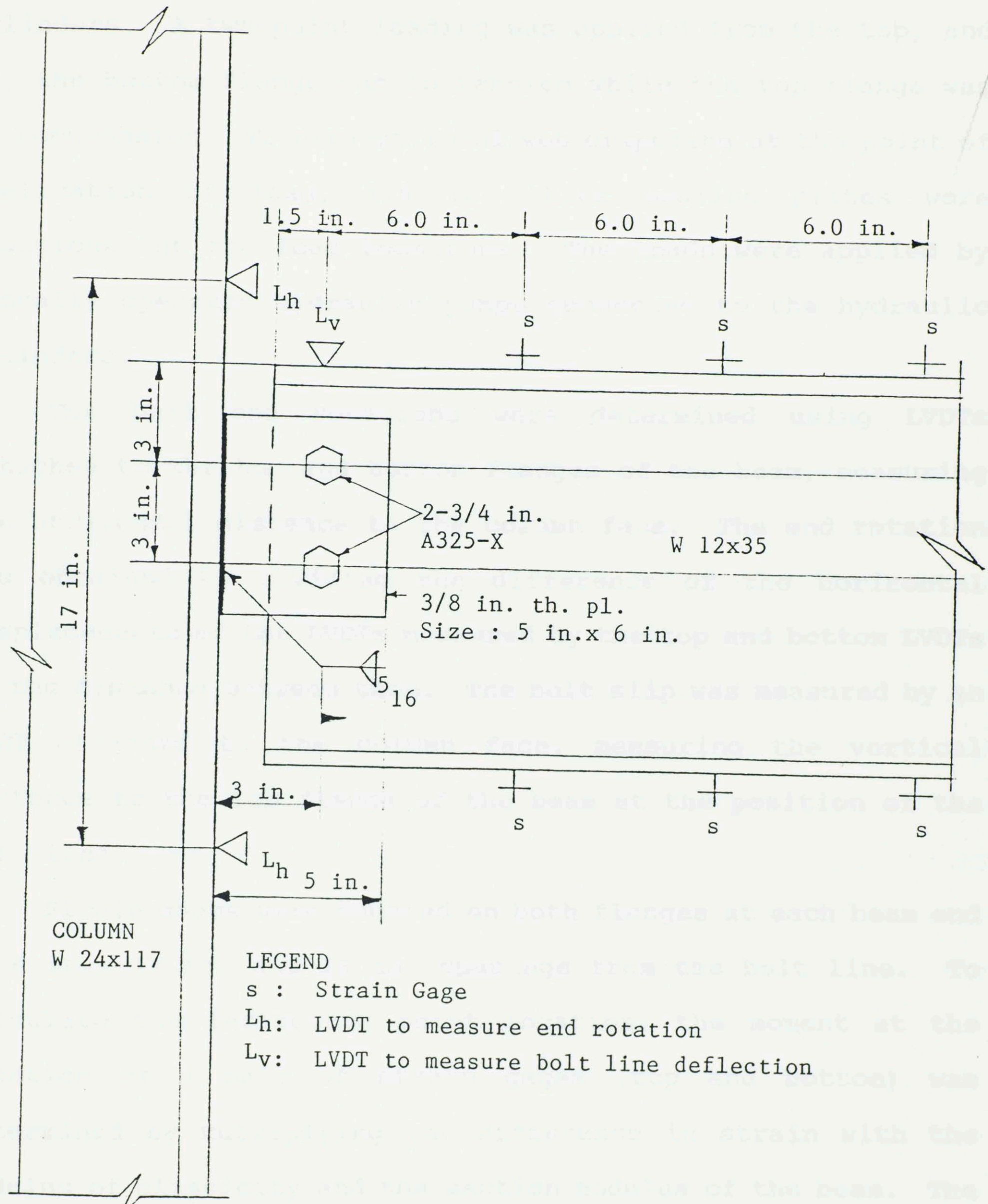

Fig. 3.2 Details of 2-Bolt Connection Specimen

load using pressure transducers calibrated with the hydraulic cylinders. A two-point loading was applied from the top, and so, the bottom flange was in tension while the top flange was in compression. To prevent local web crippling at the point of application of load, 1.5 in. thick bearing plates were positioned at the load locations. The loads were applied by manually operated hydraulic pumps connected to the hydraulic cylinders.

The beam end rotations were determined using LVDTs attached to the top and bottom flanges of the beam, measuring the horizontal distance to the column face. The end rotation was obtained by dividing the difference of the horizontal displacements of the LVDTs measured by the top and bottom LVDTs by the distance between them. The bolt slip was measured by an LVDT attached to the column face, measuring the vertical distance to the top flange of the beam at the position of the bolt line.

Strain gages were mounted on both flanges at each beam end at 6 in., 12 in. and 18 in. spacings from the bolt line. To calculate the inflection point location, the moment at the location of a pair of strain gages (top and bottom) was determined by multiplying the difference in strain with the modulus of elasticity and the section modulus of the beam. The eccentricity at that location was then obtained from the relationship :

e = distance of strain gage pair from the bolt line - (moment / end shear) (3.1)

Three wire potentiometers were positioned at one-third span locations from each end and at midspan to measure the corresponding deflections.

The entire test setup and instrumentation was monitored through a computerized data acquisition system. The setup file for the test had provisions for producing the appropriate plots during the test, so that the connection behavior could be assessed during the course of the loading.

To enable testing of the beam to large displacements and corresponding large end rotations without premature lateral buckling of the beam, lateral bracing of the compression flange (top flange) was provided by connecting L 2 1/2 x 2 1/2 x 1/4 braces to an adjacent girder at 3 ft. intervals. The tension flange (bottom flange) was also braced at midspan and at the quarter points. This bracing allowed significant beam yielding in bending without lateral torsional buckling. The beam as well as the connecting shear tabs were white-washed to identify the yield patterns during the test.

3.2 Loading Procedure

The initial distance of the loading points from the bolt line was established from the relationship:

$$a = (F_y \times S_x)/(2R) \quad (3.2)$$

where F_y = yield strength of beam

S_x = section modulus of beam

R = design reaction

This location was selected to obtain beam yielding and the corresponding beam end rotation at twice the design allowable load capacity of the connection. The two loads were applied simultaneously using manually operated hydraulic pumps. Readings were taken at specific intervals and the shear-rotation curve was carefully monitored during the course of the loading.

In order to decide on the extent of beam end rotation necessary to indicate adequate ductility in the connection, Astaneh's (5) shear-rotation relationship shown in Fig. 2.3 of Chapter 2 was used as a guideline. A beam span to depth ratio of 25 was used as a reasonable limit for attaining an end rotation of 0.02 rad. when the beam midspan moment reached its moment yield capacity (see point "b" of Fig. 2.3), and the beam softened. A 50% increase in beam end rotation which corresponded to an end rotation of 0.03 rad. would enable the beam to just attain the strain hardening stage (see point "c" of Fig. 2.3) with the beam midspan moment reaching its plastic moment capacity. Hence, loading the beam to an end rotation of 0.03 rad. seemed to be a reasonable indication of connection ductility.

The first stage of loading was carried out until failure was observed or an end rotation of 0.03 rad. was attained and then the load was released. If no failure was observed in the first stage of loading, the loading points were moved closer to the beam ends, and the test was restarted. This shift in the load points towards the beam end allowed the connections to be loaded to larger reactions in subsequent stages. This gradual loading and unloading was carried out in two or three different stages in which the shear-rotation curves were carefully observed. The behavior of the test specimens was observed from the appearance and gradual propagation of the yield lines on the test specimens, during the course of the loading.

3.3 Test Specimens

Three different beams were supplied for this research project by the W & W Steel Company, Oklahoma City. Each beam size and length was chosen so that the beam end shear-rotation relationship would be typical of commonly used beams loaded by a uniformly distributed load. All bolts were 3/4 in. diameter A325. The bolts in the first test (#1a) were inadvertently installed with the threads excluded from the shear plane. Bolts in all other tests were inserted through the thickest plate first and washers were used under the bolt head when required to ensure that the threads were included in the shear plane.

Originally only 3/8 in. thick connection plates with round bolt holes were to be tested. It was recognized that the beams could be inverted and used to test a second pair of connections with little additional effort, so this was done with each beam. The second two-bolt specimen utilized round holes in the connection plate and was essentially a repeat of the first test except that the bolts were chosen to ensure that threads were included in the shear plane. The second tests of the four- and six-bolt specimen beams utilized short horizontal slots in the connection plates to measure the effects of the slotted holes on the connection eccentricity and load capacity. Major parameters of the six tests performed are given in Table 3.1. All connection plates were cut from the same bar, which had a yield stress of 47.4 ksi established from tensile coupon tests.

Table 3.1 Test Beams and Connections

Test #	Beam Size	Beam Length (ft.)	Plate Thickness (in.)	Plate Holes	Bolts Number & Type
1a	W12x35	21	3/8	Round	2-A325-X
1b	W12x35	21	3/8	Round	2-A325-N
2a	W18x76	33	3/8	Round	4-A325-N
2b	W18x76	33	3/8	Short Slots	4-A325-N
3a	W21x93	25	3/8	Round	6-A325-N
3b	W21x93	25	3/8	Short Slots	6-A325-N

3.4 Experimental Results

3.4.1 Behavior of Test Specimen 1a:

This test was on a W12x35 beam with 2-bolt connections and other parameters as indicated in Table 3.2. For Test 1a, the initial load location was 57-7/8 in. from the beam ends. The beam was loaded with the hydraulic cylinders at this position until the end rotations reached 0.03 rad.s. The behavior of the test specimen at different stages of loading is described by Figs. 3.3 through 3.10. In each of these plots, the three successive stages of loading are indicated by the respective numbers.

Minor yielding of the beam web at the vicinity of the bottom bolt was observed at a load of 25 kips during the first stage of loading. At 30 kips, yielding at the bearing of the bottom bolt (near the tension flange) became just noticeable; simultaneously, yield lines appeared on the tension flange concentrated mostly in the central region of the beam. The compression flange was observed to yield at a load of 34 kips. At this stage, the rotation at both ends reached 0.03 rad. With the next increment of load, shear yielding of the North plate between the bolts and weld became apparent. Slight twisting of the shear tab was also noticeable. The load was then gradually released.

The test was restarted after moving the jacks further towards the support. This time the desired end shear was

Table 3.2: Details of Test Specimen 1a

BEAM SIZE AND DIMENSION :	
Beam Size :	W12x35
Beam Length :	21 ft.
Beam Material :	A36
Beam Yield Strength :	46 ksi (from mill certificate)
Design Reaction :	18.6 kips
Load Locations :	57-7/8 in. from beam ends
Bolt Holes :	13/16 in. round (drilled)
CONNECTION SIZE AND DIMENSION :	
No. of Bolts :	2
Type of Bolts :	A325-X
Bolt Spacing :	3 in.
Edge Distance :	1-1/2 in.
Plate Thickness :	3/8 in.
Plate Size :	5 in. x 6 in.
Plate Material :	A36
Plate Yield Strength :	47.4 ksi (from tensile tests)
Weld Size :	5/16 in. double fillet weld
Bolt Holes in Plate :	13/16 in. round (punched)
Column Size :	W24x117

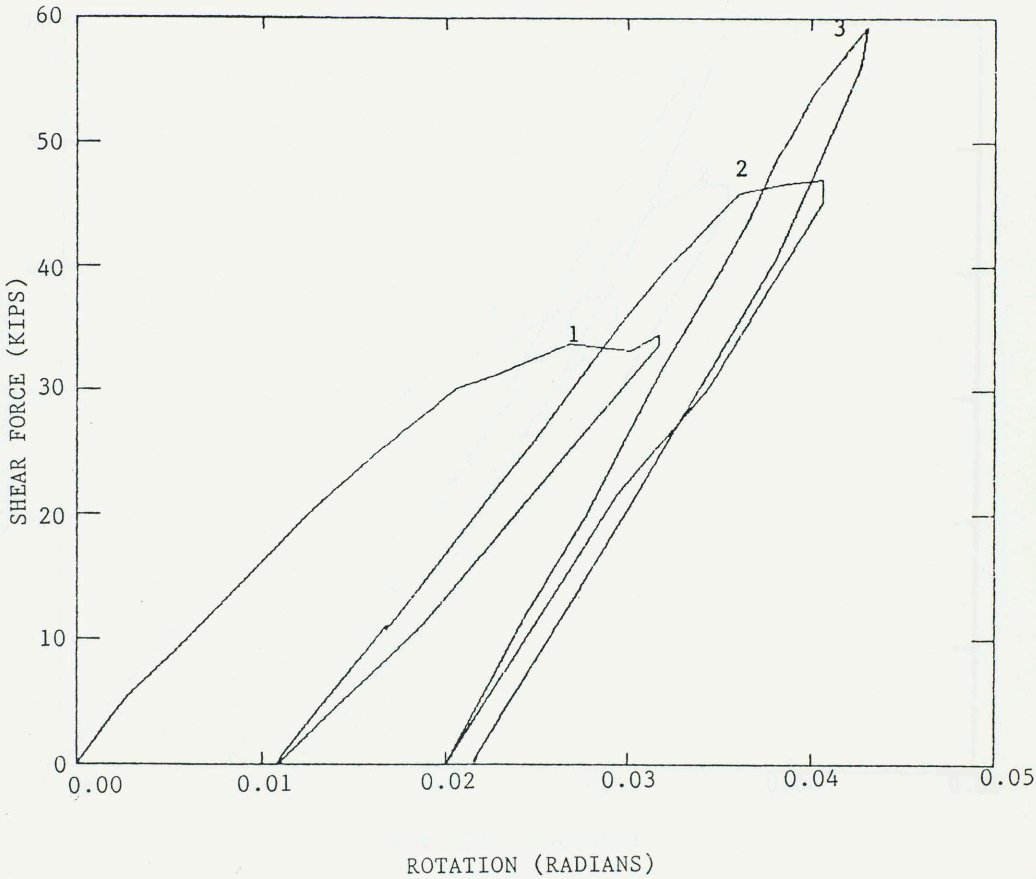

Fig. 3.3 Shear vs. Rotation (North End), Test 1a

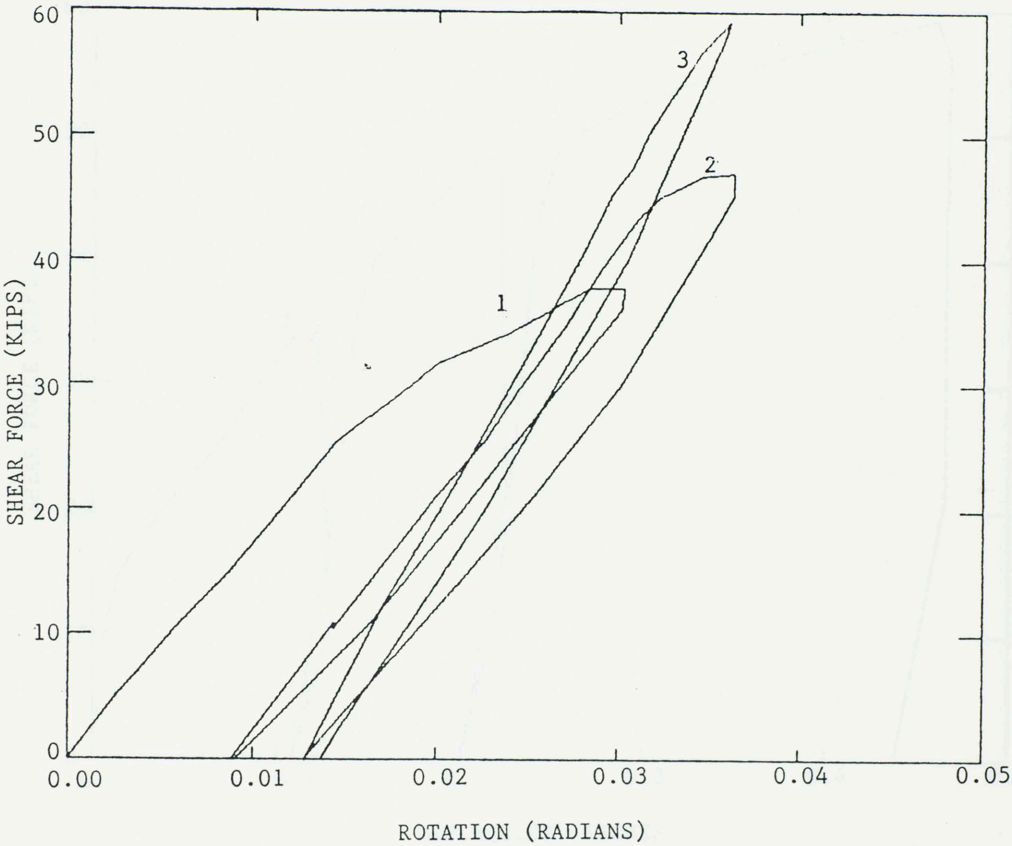

Fig. 3.4 Shear vs. Rotation (South End), Test 1a

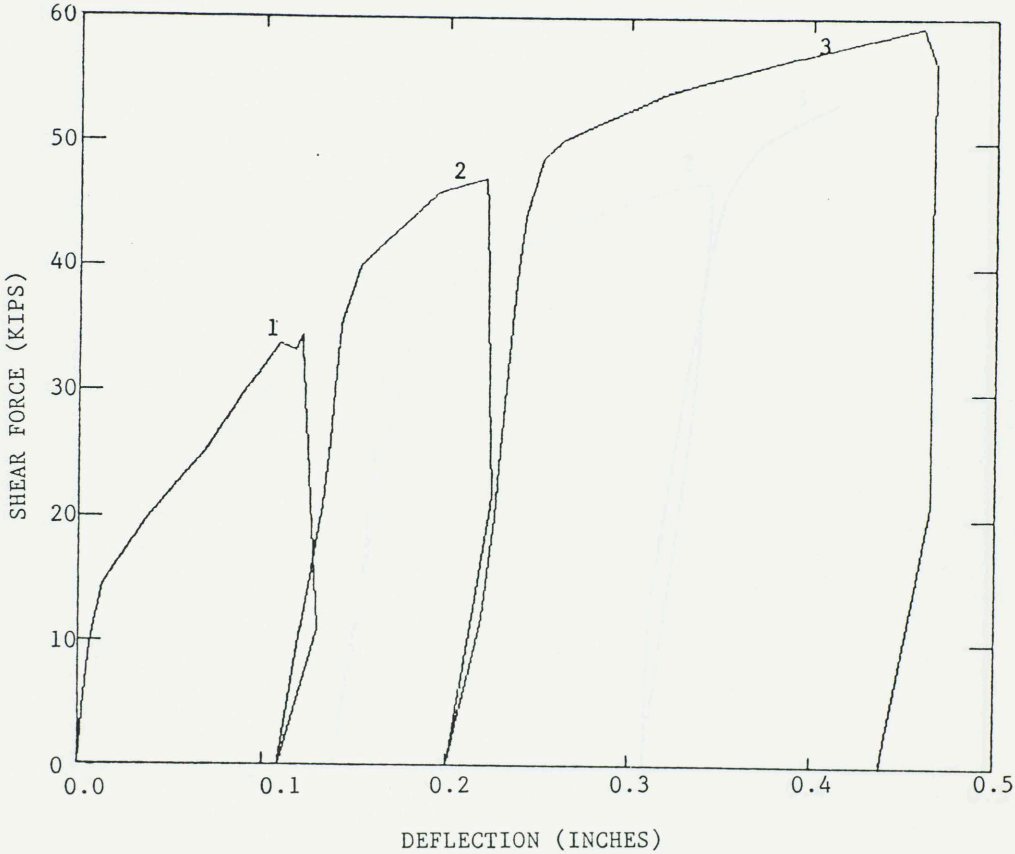

Fig. 3.5 Shear vs. Bolt Line Deflection (North End), Test 1a

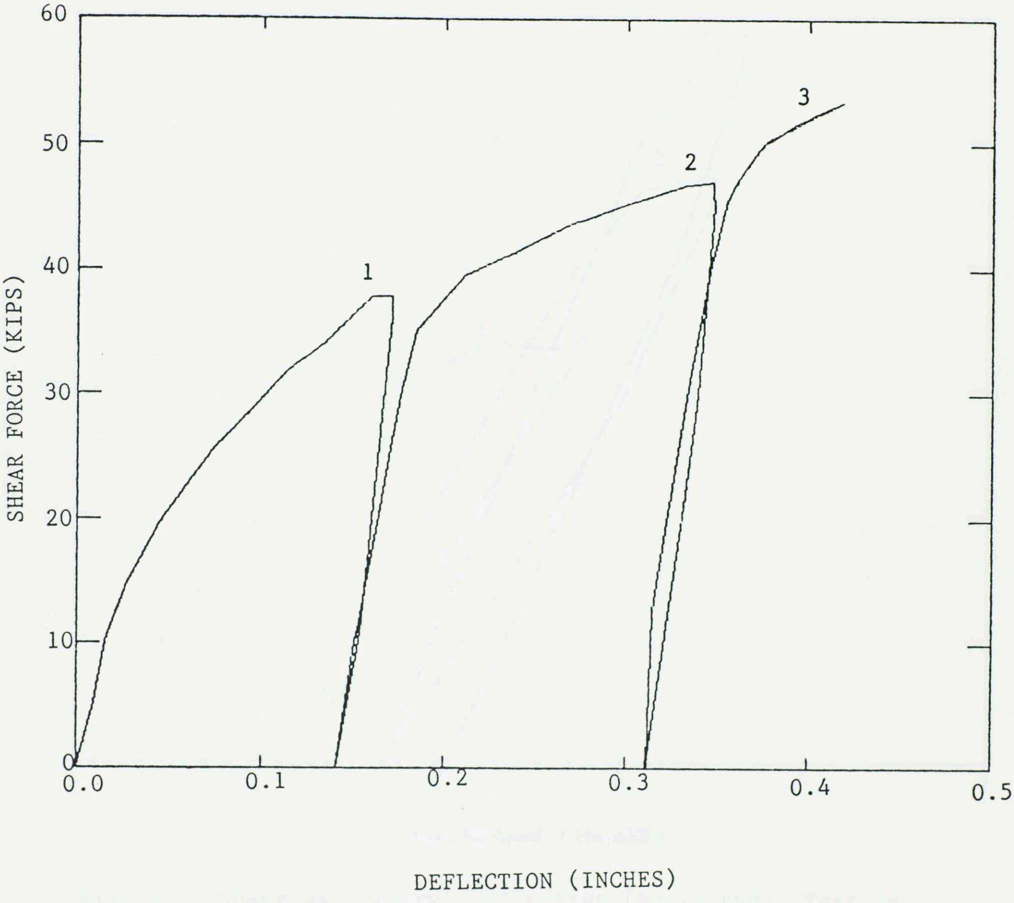

Fig. 3.6 Shear vs. Bolt Line Deflection (South End), Test la

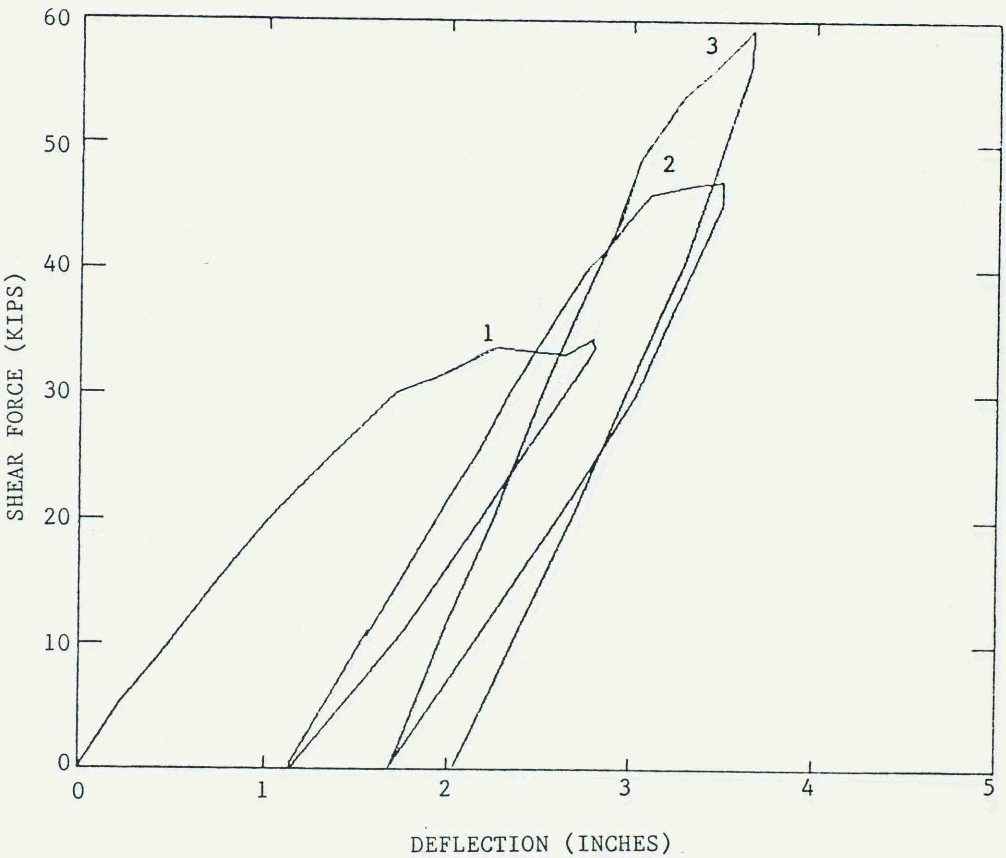

Fig. 3.7 Shear vs. Central Deflection (North End), Test 1a

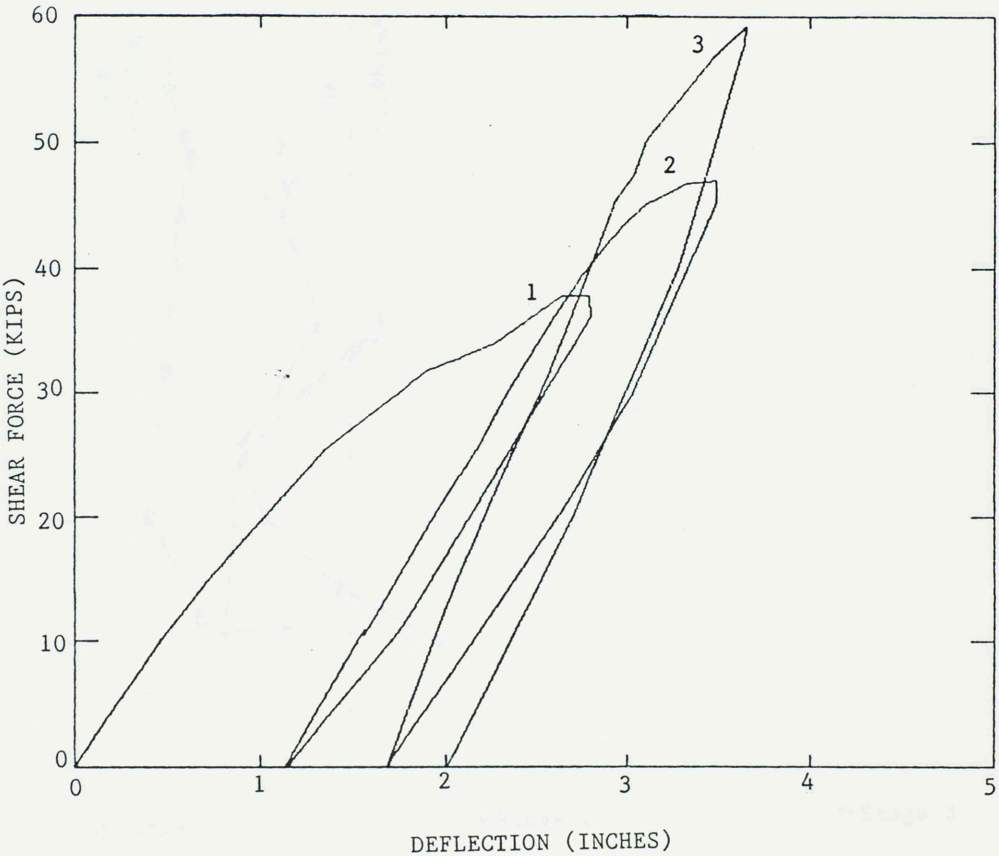

Fig. 3.8 Shear vs. Central Deflection (South End), Test 1a

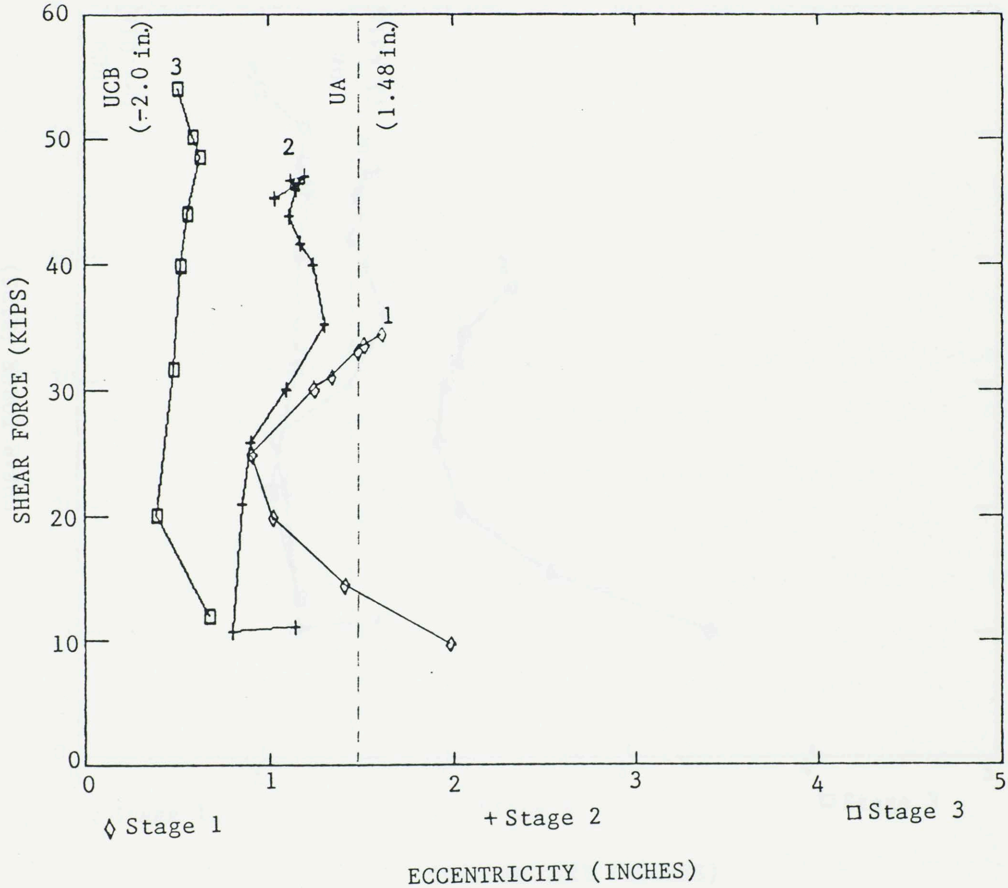

Fig. 3.9 Shear vs. Eccentricity (North End), Test la

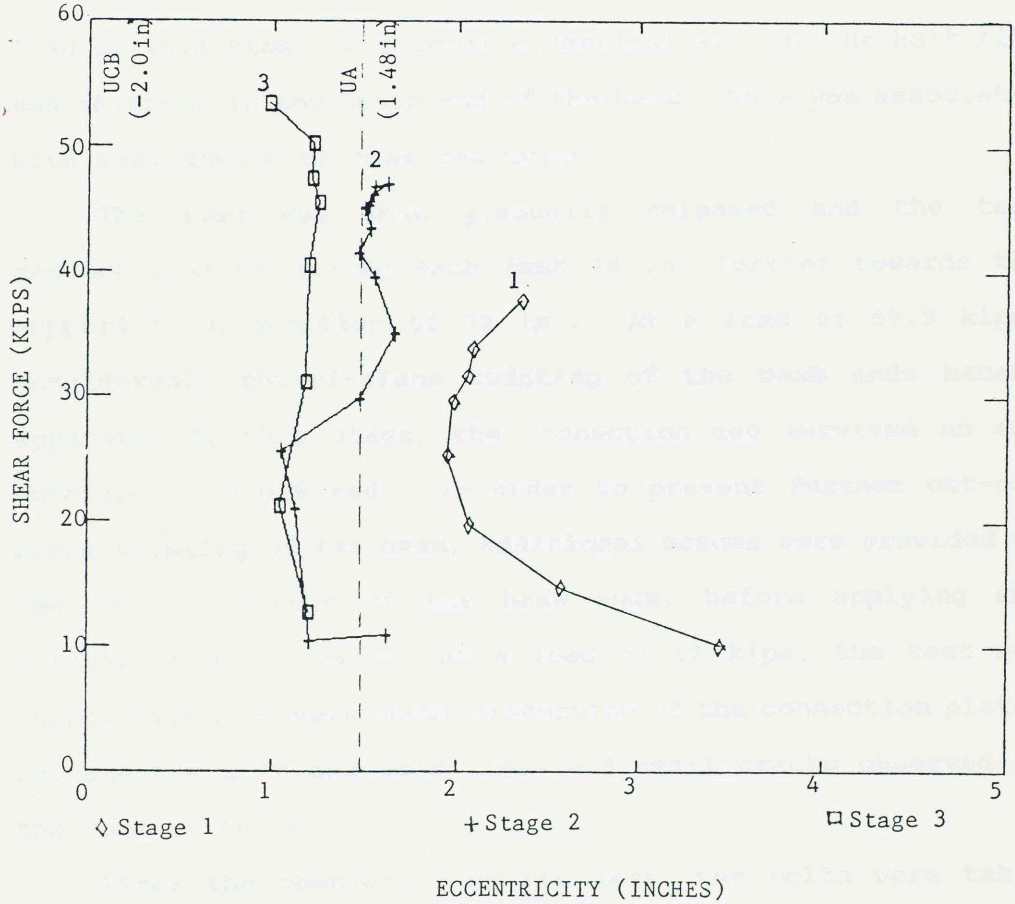

Fig. 3.10 Shear vs. Eccentricity (South End), Test 1a

increased by 25%, and hence, the loading points were established at 46 in. from the beam ends. At a load of 41.7 kips, significant shear yielding of the North end shear tab was observed in the vicinity of the weld line. Some shear yielding was also noticed in the South end shear tab. Up to this stage, the shear-rotation curve appeared to be fairly linear. At a load of 46.8 kips, considerable vertical slip at the bolt line was observed in the South end of the beam. This was associated with some amount of beam end twist.

The load was then gradually released and the test restarted after moving each jack 14 in. further towards the support to a location of 32 in.. At a load of 59.3 kips, considerable out-of-plane twisting of the beam ends became apparent. At this stage, the connection had survived an end rotation of 0.036 rad. In order to prevent further out-of-plane twisting of the beam, additional braces were provided to the tension flange of the beam ends, before applying any further load. Finally, at a load of 62 kips, the test was stopped due to severe shear distortion of the connection plates between the bolt and weld lines and small cracks observed at the root of the welds.

After the completion of the test, the bolts were taken out; and the shear deformation in the bolts was found to be almost insignificant. This might be attributed to the fact that the gross area instead of the net area of the bolts was allowed to resist the shear. Permanent bearing deformation of approximately 3/32 in. was observed in the bolt holes of the beam web as well as the shear tabs.

A careful observation of the shear vs. eccentricity graphs (see Figs. 3.9 and 3.10) indicates that the location of the inflection point of the moment diagram was about 1.5 in. away from the bolt line towards the midspan. The vertical lines denoted as UA and UCB in the shear vs. eccentricity graphs respectively indicate Richard's and Astaneh's prediction of eccentricity for this test.

3.4.2 Behavior of Test Specimen 1b:

The same W12x35 beam used in test 1a was used again in test 1b. The beam was flipped over and additional bolt holes were drilled at the beam ends. The beam was then bolted to a single plate on either side. New plates cut from the same bar used in this test. The plates were shop welded to the column flanges by the flux cored arc welding process using E70XX electrodes, but with smaller amperage equipment and smaller diameter electrodes. These welds were made to the 5/16 in. outside dimension but small root openings and shallower penetration resulted in welds with an effective size of approximately 1/4 in. Bolts were installed so that the bolt threads were included in the shear plane. Details of this test specimen are listed in Table 3.3.

During the first stage of loading, the connection survived an end rotation of 0.033 rad., so further loading was stopped

Table 3.3: Details of Test Specimen 1b

BEAM SIZE AND DIMENSION :

Beam Size :	W12x35
Beam Length :	21 ft.
Beam Material :	A36
Beam Yield Strength :	46 ksi (from mill certificate)
Design Reaction :	18.6 kips
Load Locations :	57-7/8 in. from beam ends
Bolt Holes :	13/16 in. round (drilled)

CONNECTION SIZE AND DIMENSION :

No. of Bolts :	2
Type of Bolts :	A325-N
Bolt Spacing :	3 in.
Edge Distance :	1-1/2 in.
Plate Thickness :	3/8 in.
Plate Size :	5 in. x 6 in.
Plate Material :	A36
Plate Yield Strength :	47.4 ksi (from tensile tests)
Weld Size :	1/4 in. double fillet weld
Bolt Holes in Plate :	13/16 in. round (punched)
Column Size :	W24x117

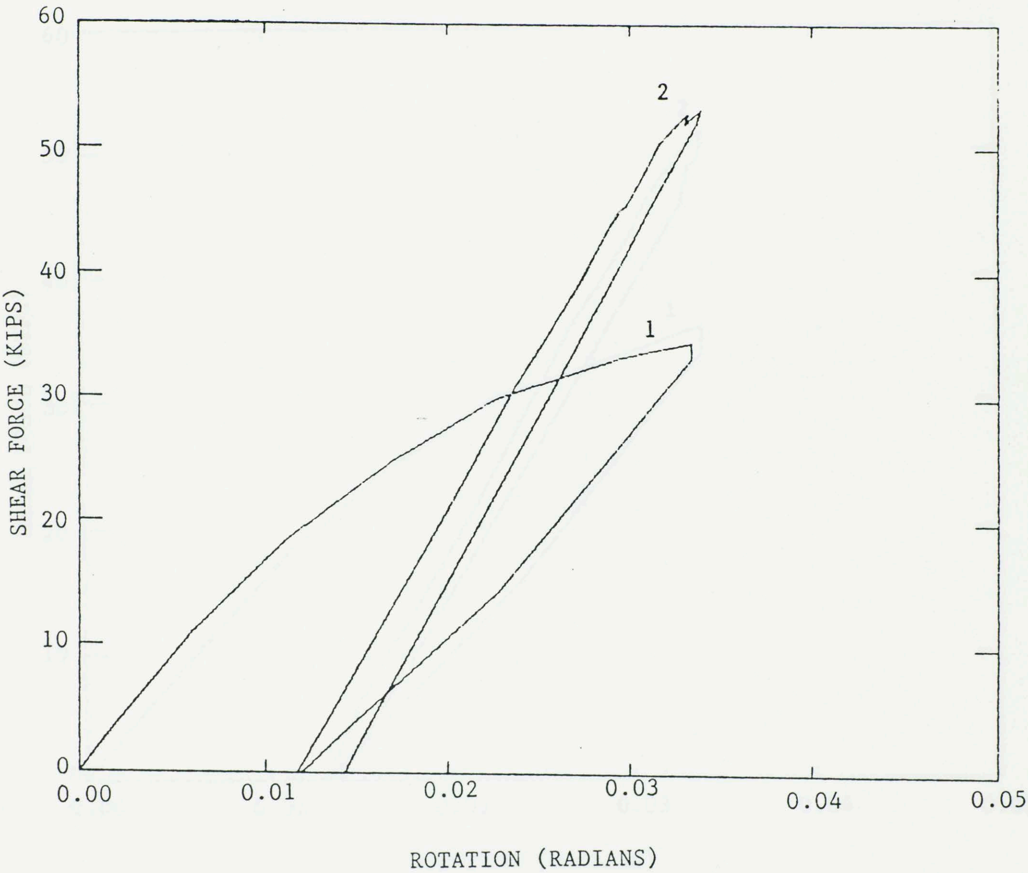

Fig. 3.11 Shear vs. Rotation (North End), Test 1b

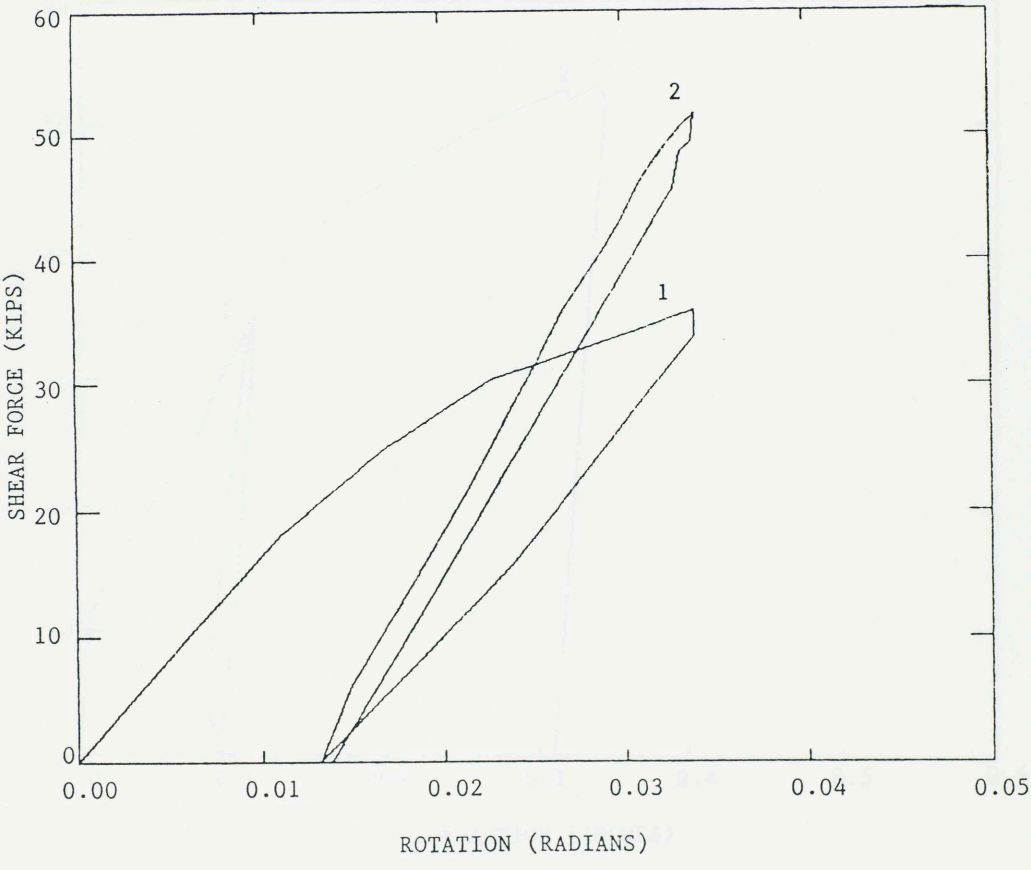

Fig. 3.12 Shear vs. Rotation (South End), Test 1b

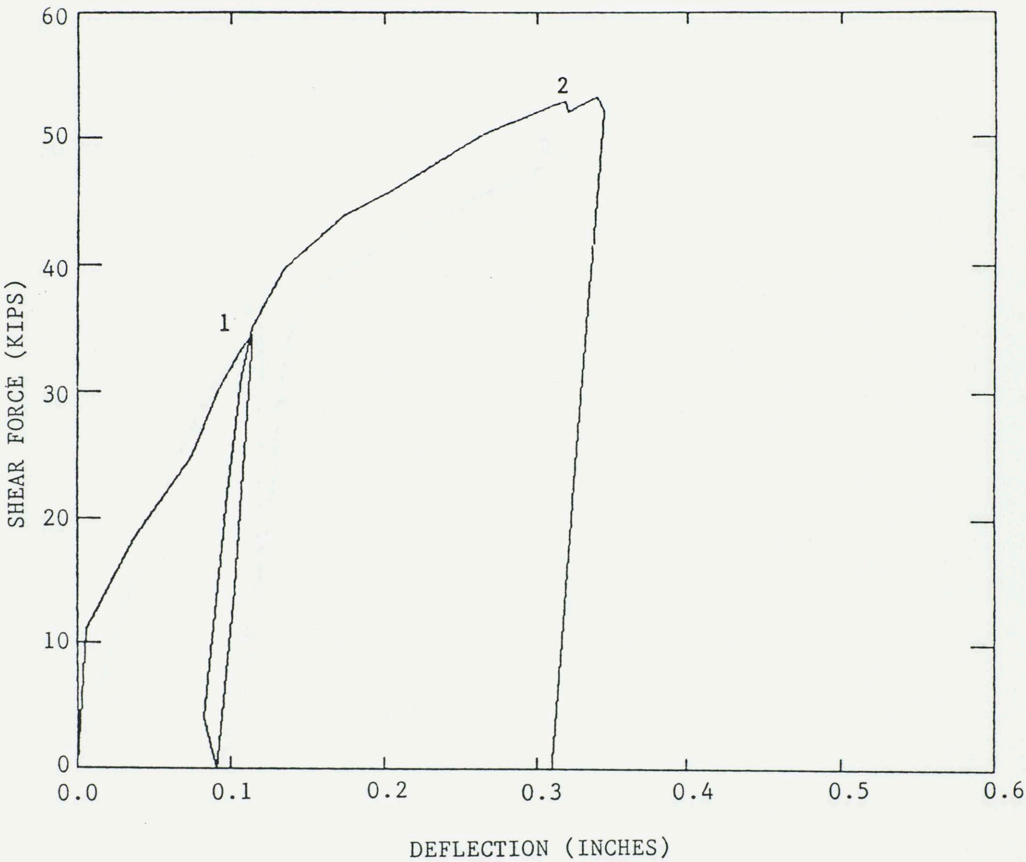

Fig. 3.13 Shear vs. Bolt Line Deflection (North End), Test 1b

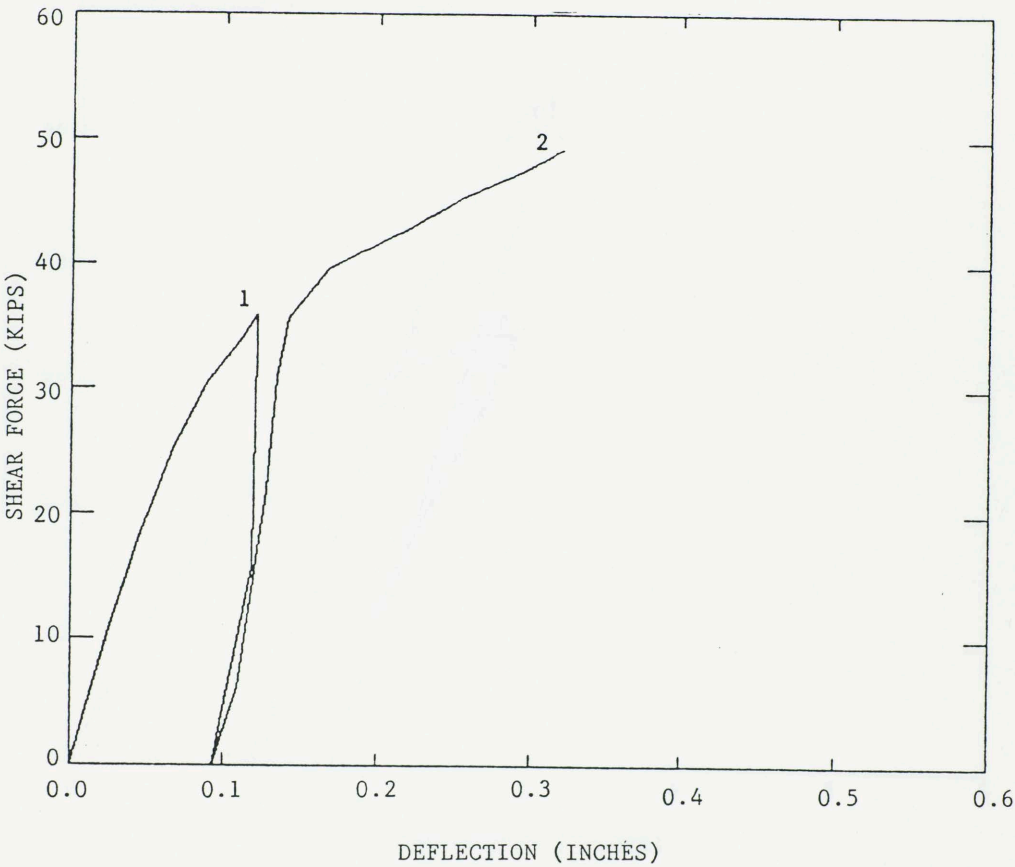

Fig. 3.14 Shear vs. Bolt Line Deflection (South End), Test 1b

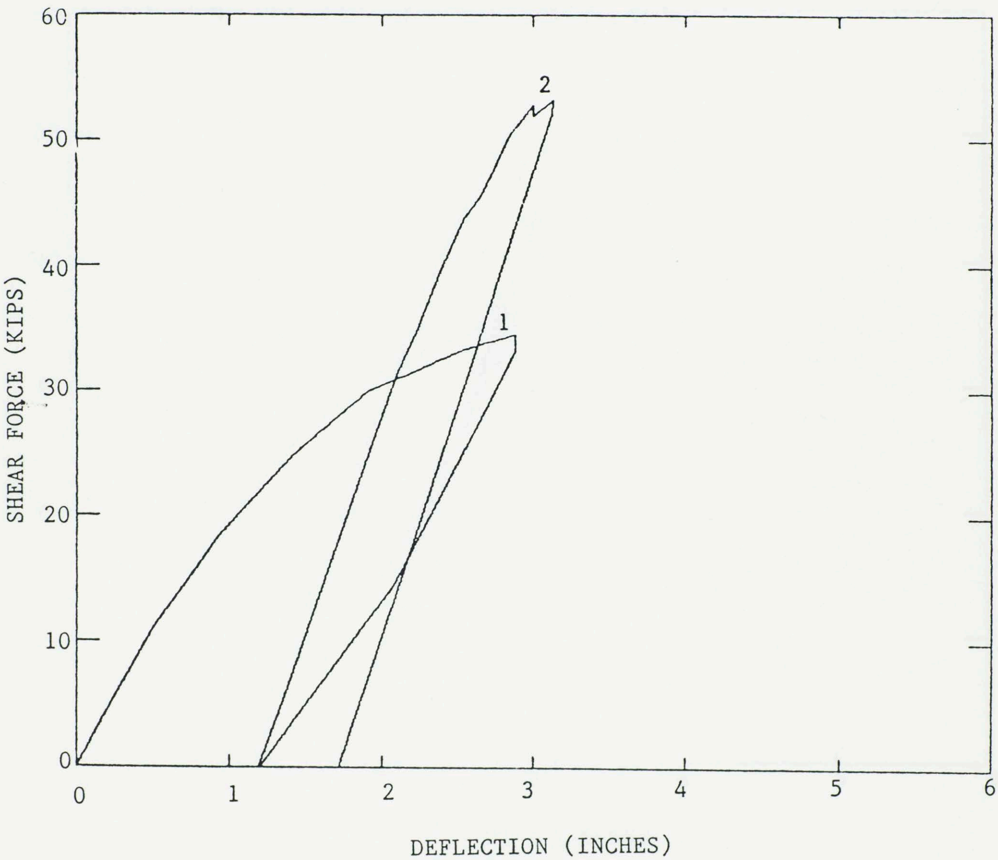

Fig. 3.15 Shear vs. Central Deflection (North End), Test 1b

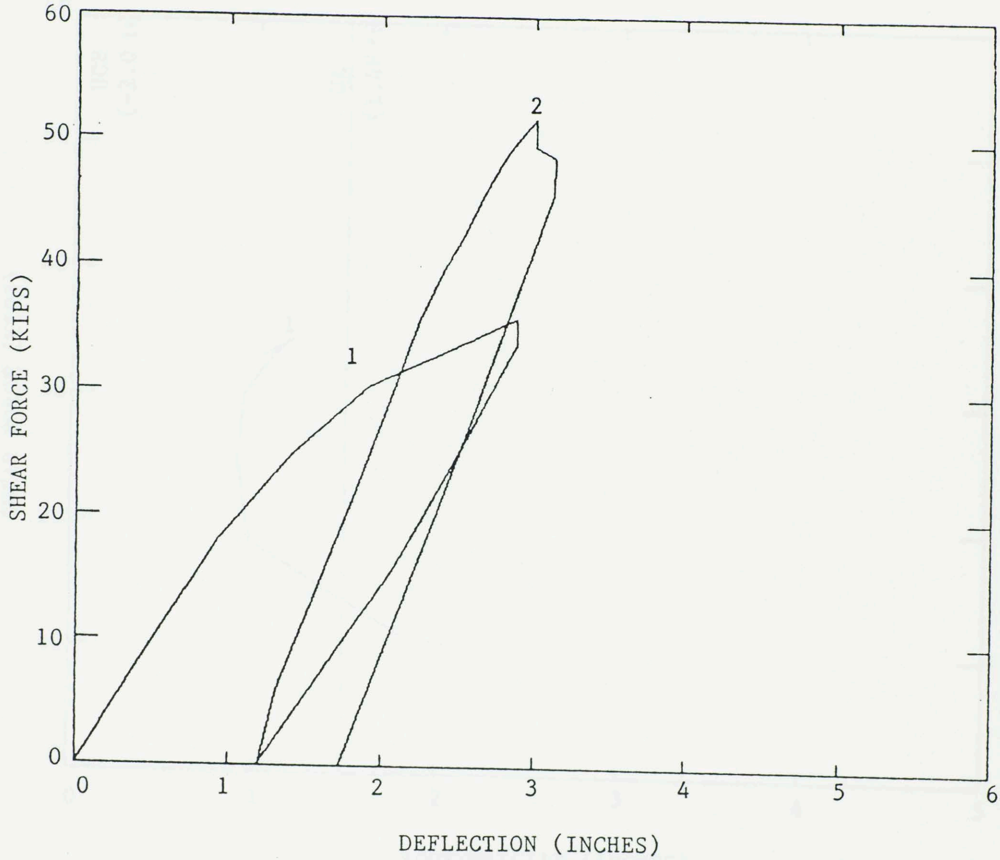

Fig. 3.16 Shear vs. Central Deflection (South End), Test 1b

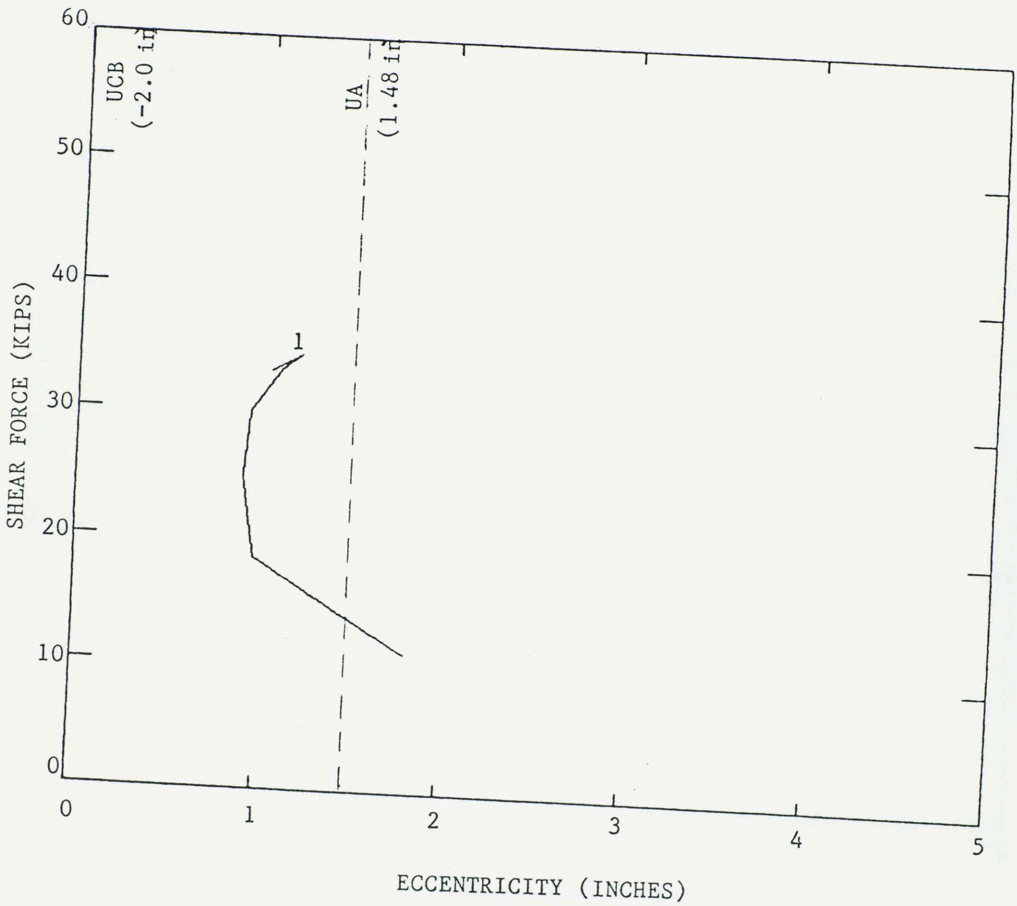

Fig. 3.17 Shear vs. Eccentricity (North End), Test 1b

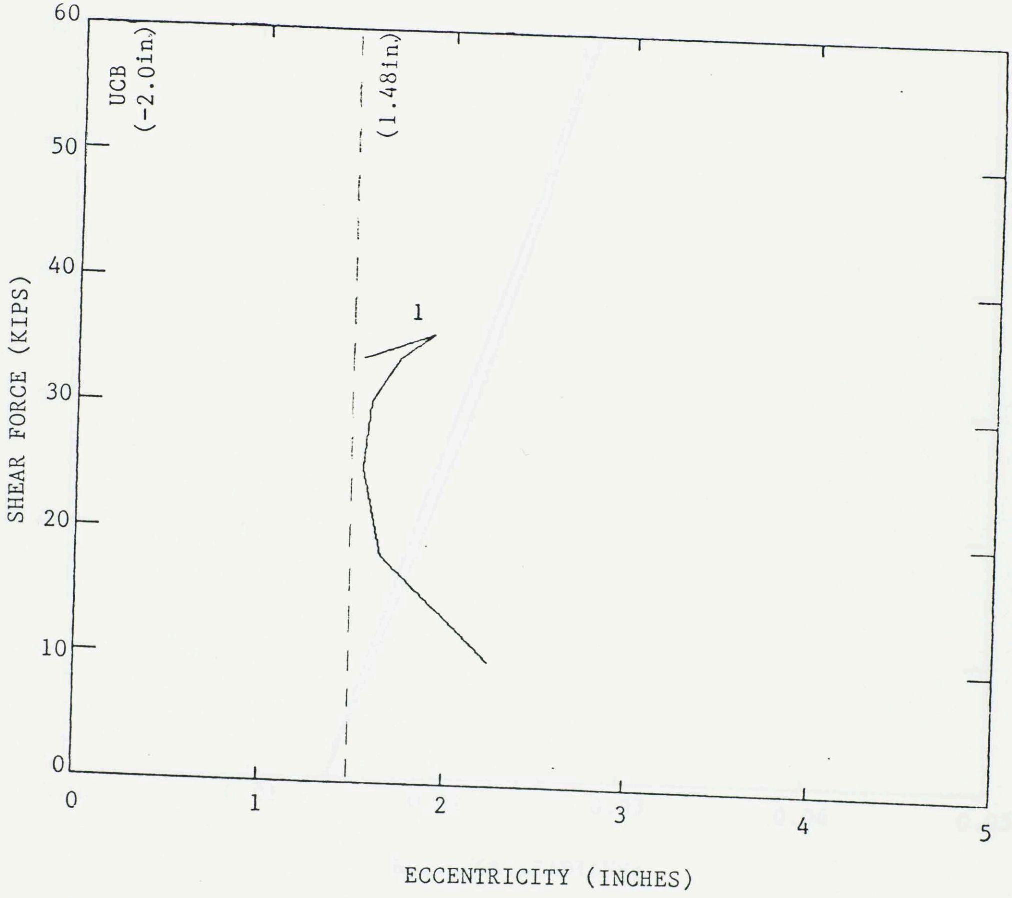

Fig. 3.18 Shear vs. Eccentricity (South End), Test 1b

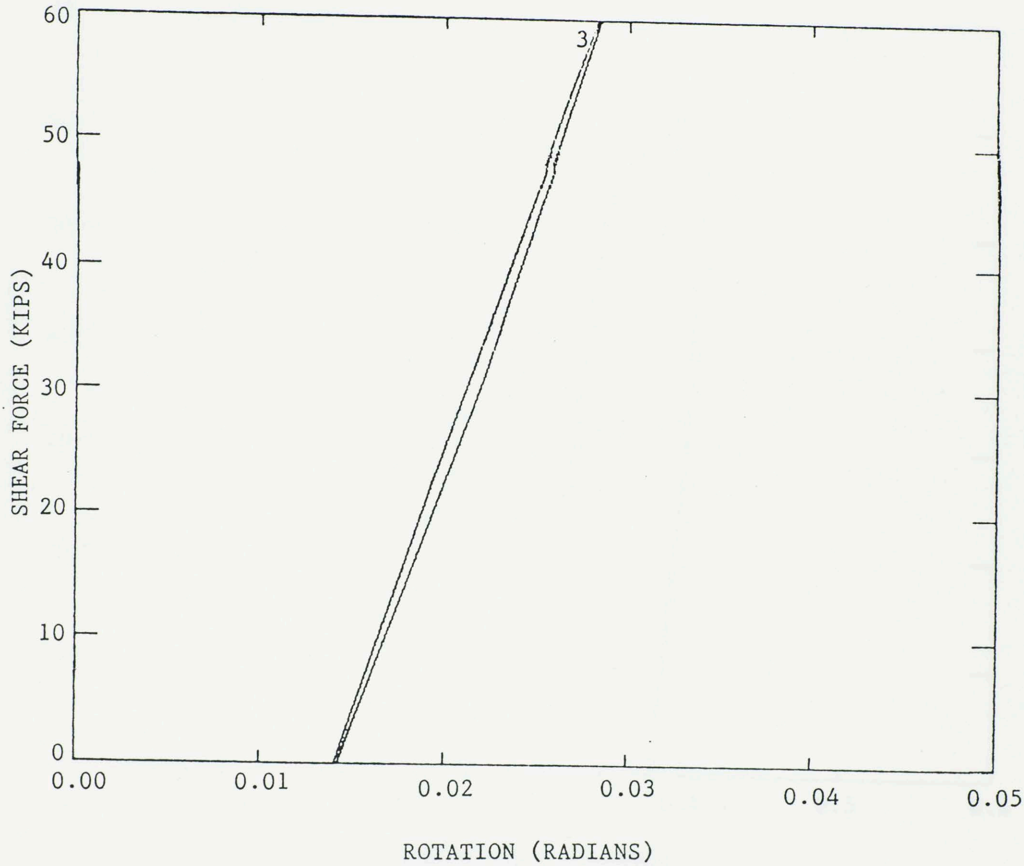

Fig. 3.19 Shear vs. Rotation
(Load applied at North End only), Test 1b

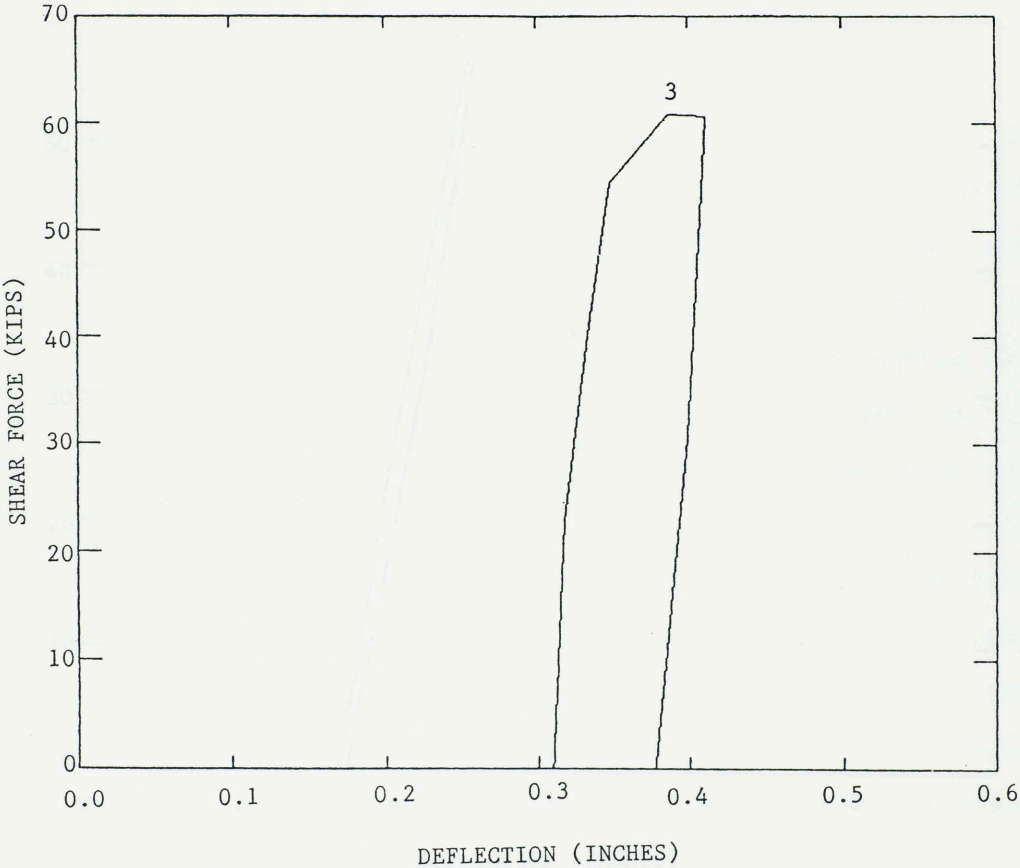

Fig. 3.20 Shear vs. Bolt Line Deflection
(Load applied at North End only), Test 1b

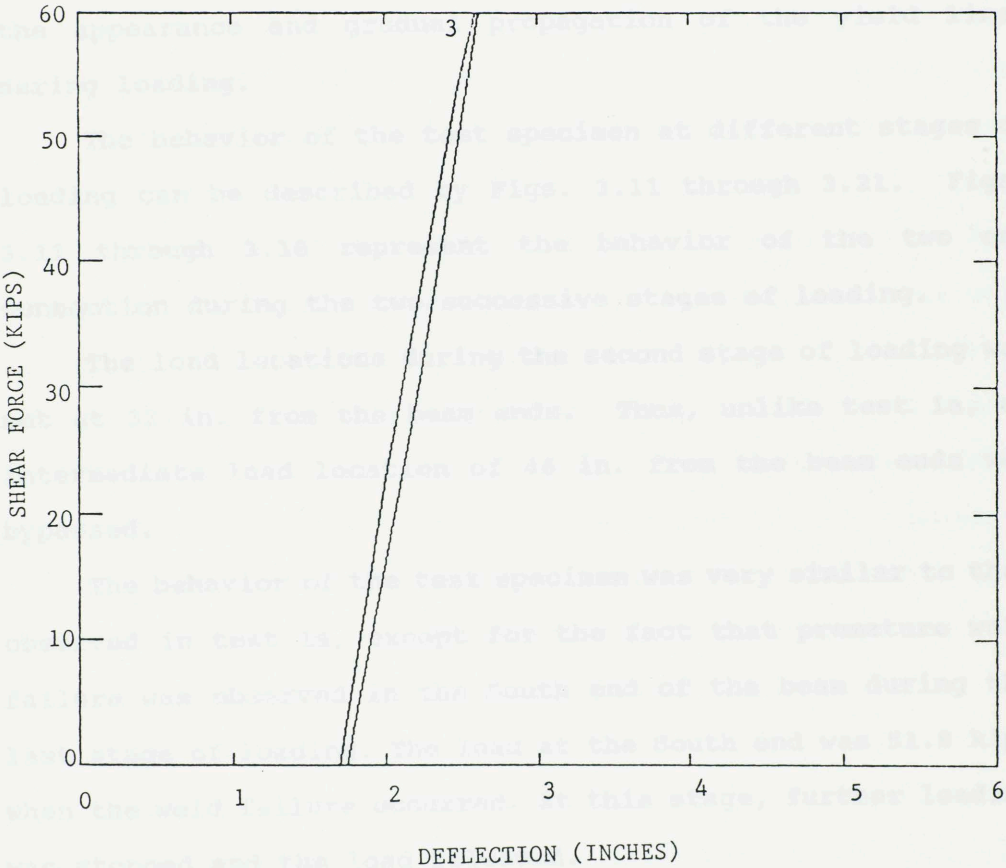

Fig. 3.21 Shear vs. Central Deflection
(Load applied at North End only), Test 1b

and the load released. The loading points were then moved closer to the beam ends (32 in. from the beam ends), and the test was restarted.

The gradual loading and unloading was carried out in two stages in which the shear-rotation curves were carefully observed. The behavior of the test specimen was observed from the appearance and gradual propagation of the yield lines during loading.

The behavior of the test specimen at different stages of loading can be described by Figs. 3.11 through 3.21. Figs. 3.11 through 3.18 represent the behavior of the two end connection during the two successive stages of loading.

The load locations during the second stage of loading was set at 32 in. from the beam ends. Thus, unlike test 1a, an intermediate load location of 46 in. from the beam ends was bypassed.

The behavior of the test specimen was very similar to that observed in test 1a, except for the fact that premature weld failure was observed in the South end of the beam during the last stage of loading. The load at the South end was 51.8 kips when the weld failure occurred. At this stage, further loading was stopped and the load released.

Since the North end did not show any weld failure, it was decided that the load be applied only at the North end; and the load location was maintained at 32 in. from the beam end. Figs. 3.19 through 3.21 represent the behavior of the connection due to load applied at the North end only. At a load of 60.8 kips, the North end of the beam started showing weld failure and the test was stopped.

3.4.3 <u>Behavior of Test Specimen 2a:</u>

This test was the first on the 4-bolt connections as shown in Table 3.1. Details of this specimen are listed in Table 3.4. The loads were initially placed 92 in. from the beam ends. The recorded behavior is shown in Figs. 3.22 through 3.29.

In the first stage of loading (load location 92 in. from beam ends) no yielding was observed in the connection as well as in the beam up to a load of 49.9 kips. This can be recognized from the linearity in the shear-rotation and shear-central deflection curves (zone a-b). The shear-eccentricity curves (see Figs. 3.28 and 3.29) show a significant change in eccentricity, particularly at the North end of the beam. This may be attributed to the plowing of the bolts through the bolt holes, thereby releasing some end moment while reducing the eccentricity.

On further increment of load, tension yield was observed in the bottom flange of the beam. At a load of 65 kips, the North end of the beam showed some yielding near the topmost bolt. Finally, at 66.5 kips load, the topmost bolt at the North end failed due to shear. Data readings were taken immediately after the bolt failure, and it was found that the

Table 3.4: Details of Test Specimen 2a

BEAM SIZE AND DIMENSION :

Beam Size :	W18x76
Beam Length :	33 ft.
Beam Material :	A36
Beam Yield Strength :	46.1 ksi (from mill certificate)
Design Reaction :	37.2 kips
Load Locations :	92 in. from beam ends
Bolt Holes :	13/16 in. round (drilled)

CONNECTION SIZE AND DIMENSION :

No. of Bolts :	4
Type of Bolts :	A325-N
Bolt Spacing :	3 in.
Edge Distance :	1-1/2 in.
Plate Thickness :	3/8 in.
Plate Size :	5 in. x 12 in.
Plate Material :	A36
Plate Yield Strength :	47.4 ksi (from tensile tests)
Weld Size :	5/16 in. double fillet
Bolt Holes in Plate :	13/16 in. round (punched)
Column Size :	W24x117

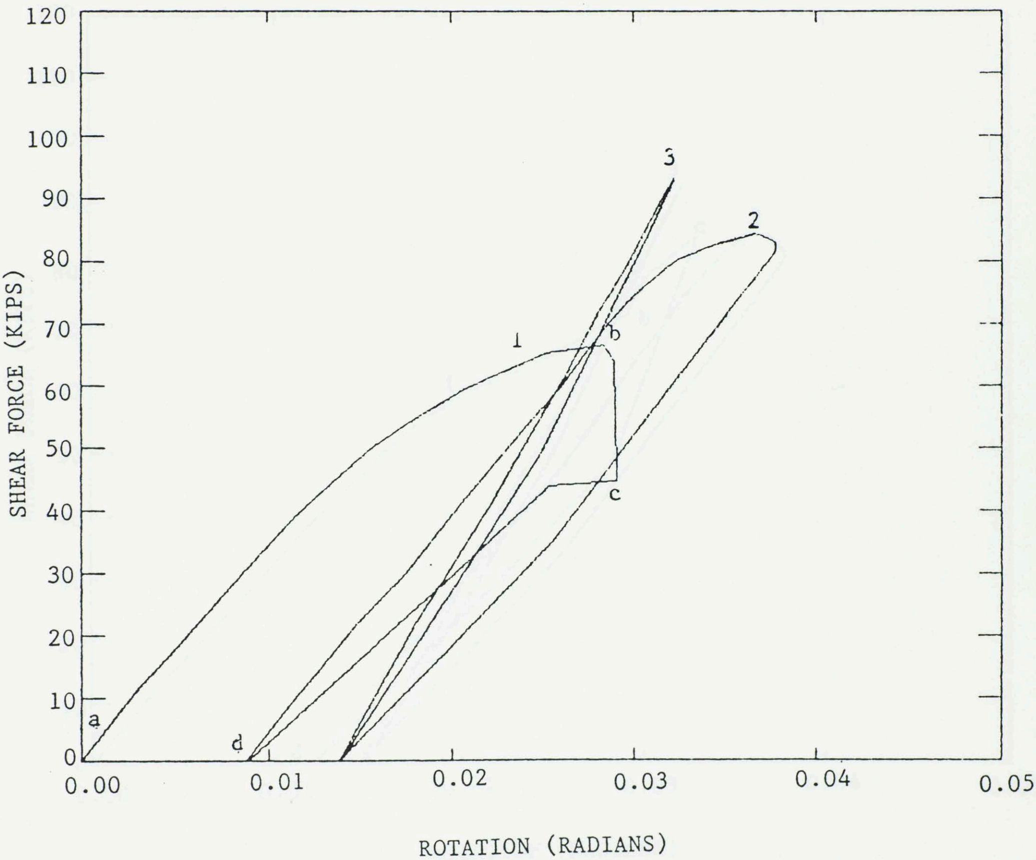

Fig. 3.22 Shear vs. Rotation (North End), Test 2a

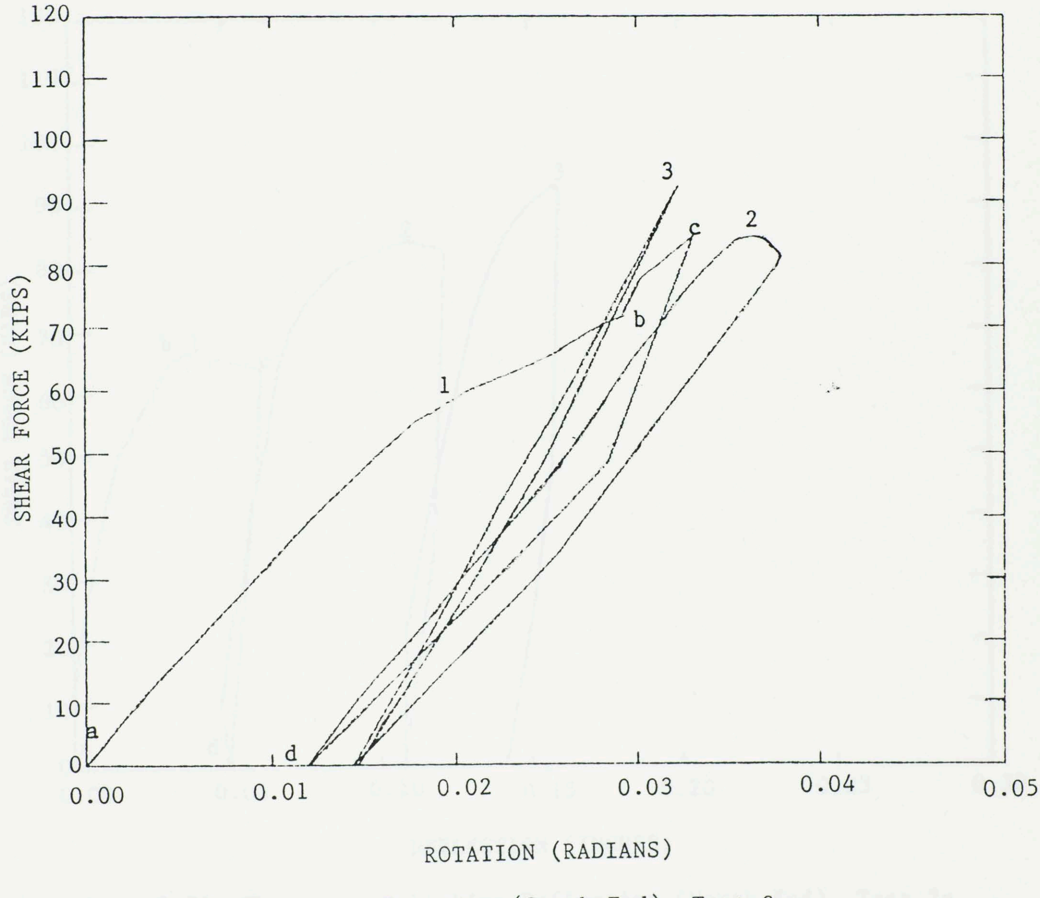

Fig. 3.23 Shear vs. Rotation (South End), Test 2a

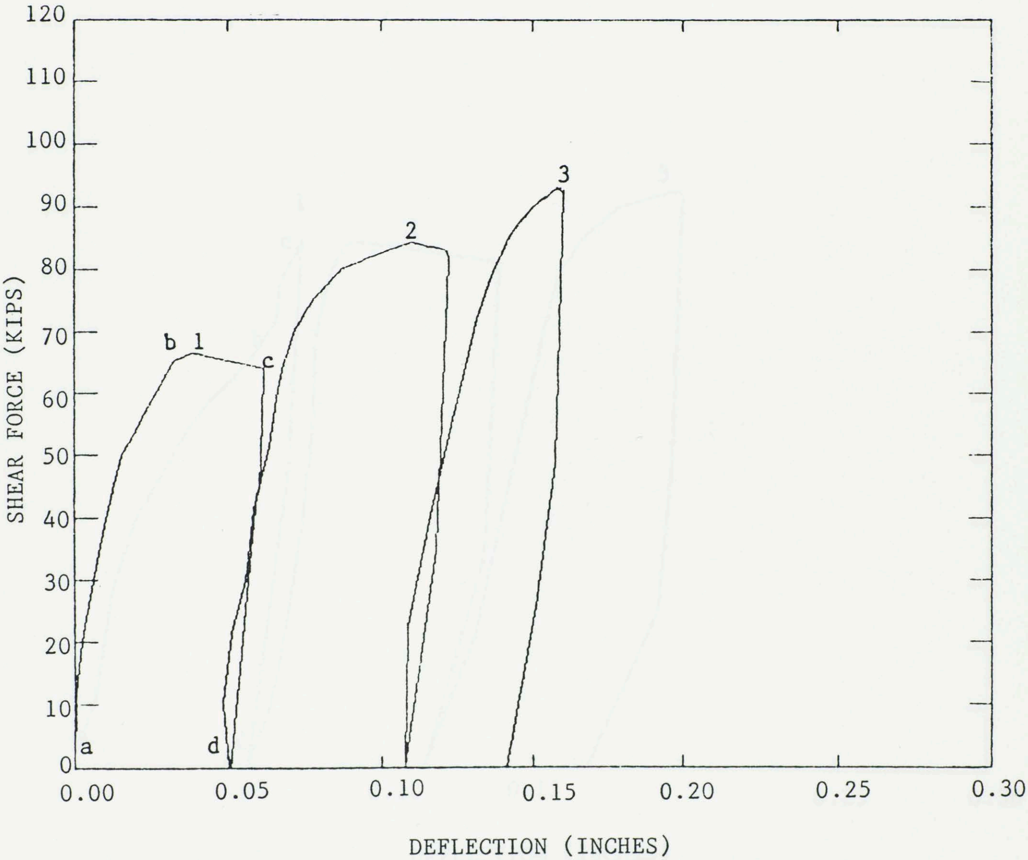

Fig. 3.24 Shear vs. Bolt Line Deflection (North End), Test 2a

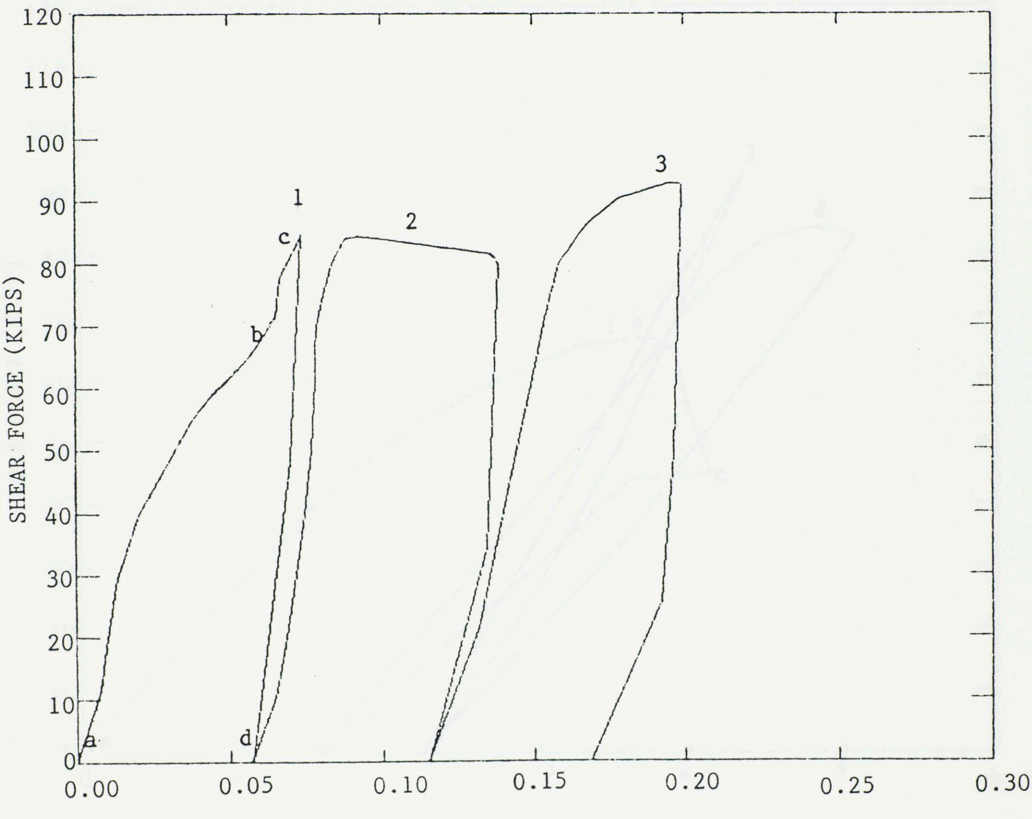

Fig. 3.25 Shear vs. Bolt Line Deflection (South End), Test 2a

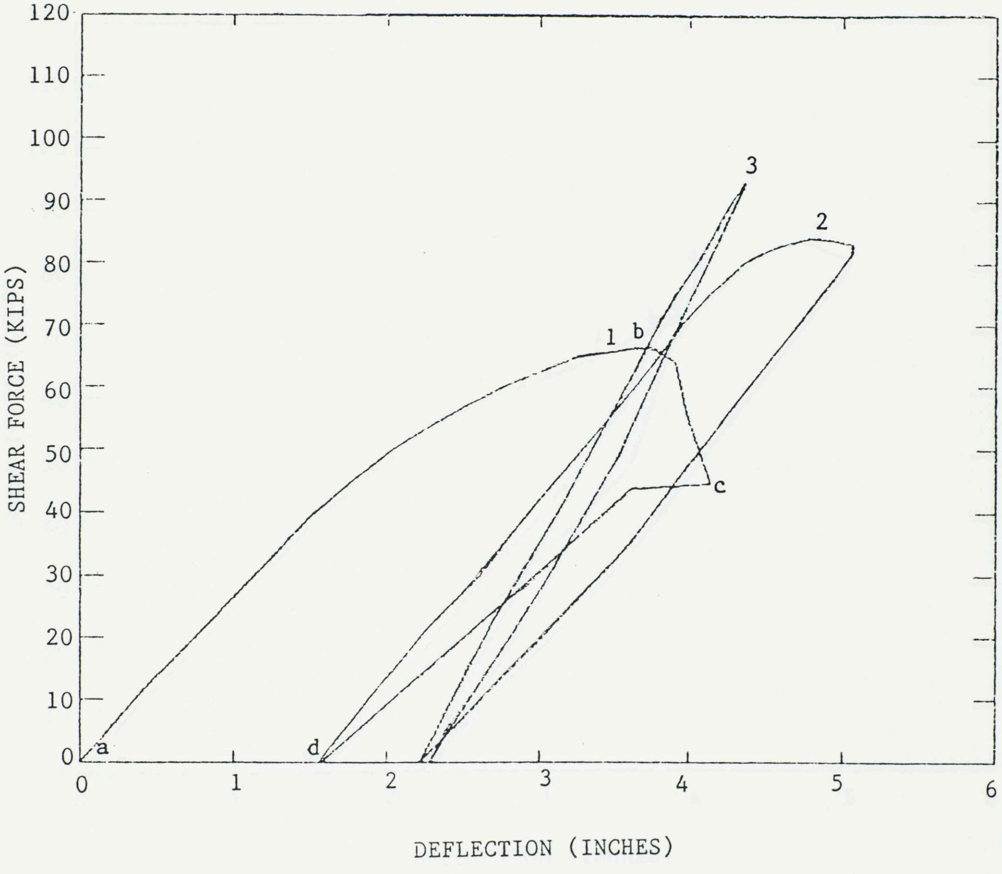

Fig. 3.26 Shear vs. Central Deflection (North End), Test 2a

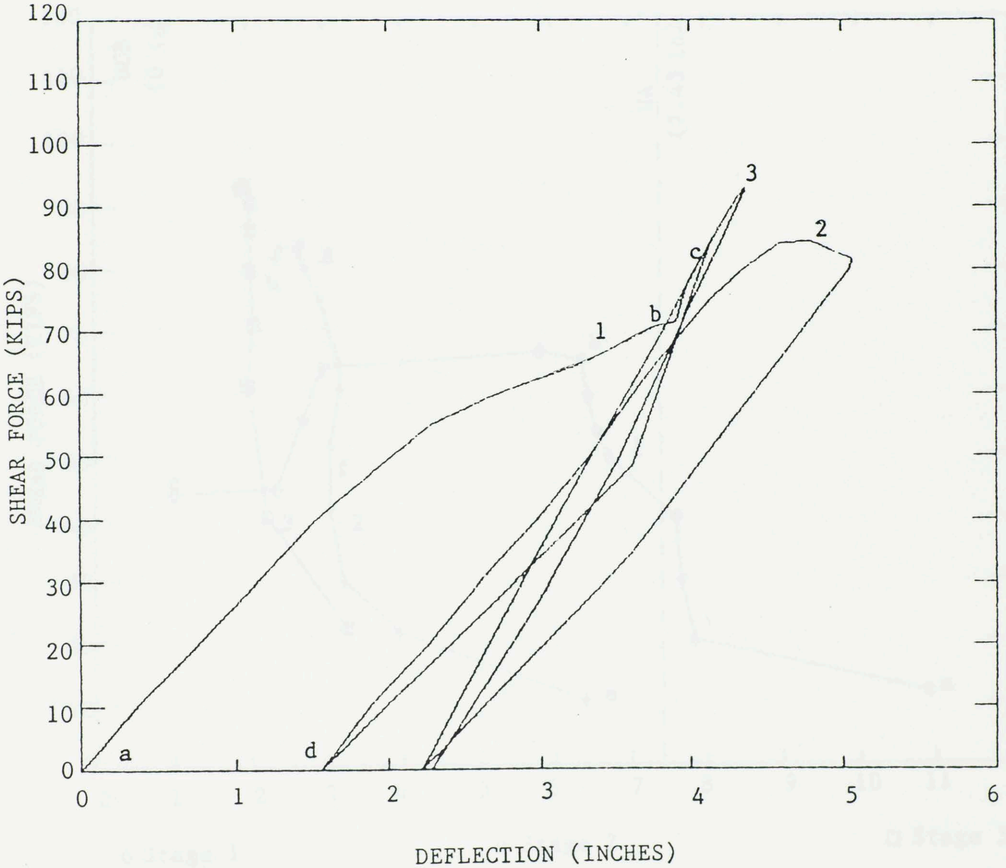

Fig. 3.27 Shear vs. Central Deflection (South End), Test 2a

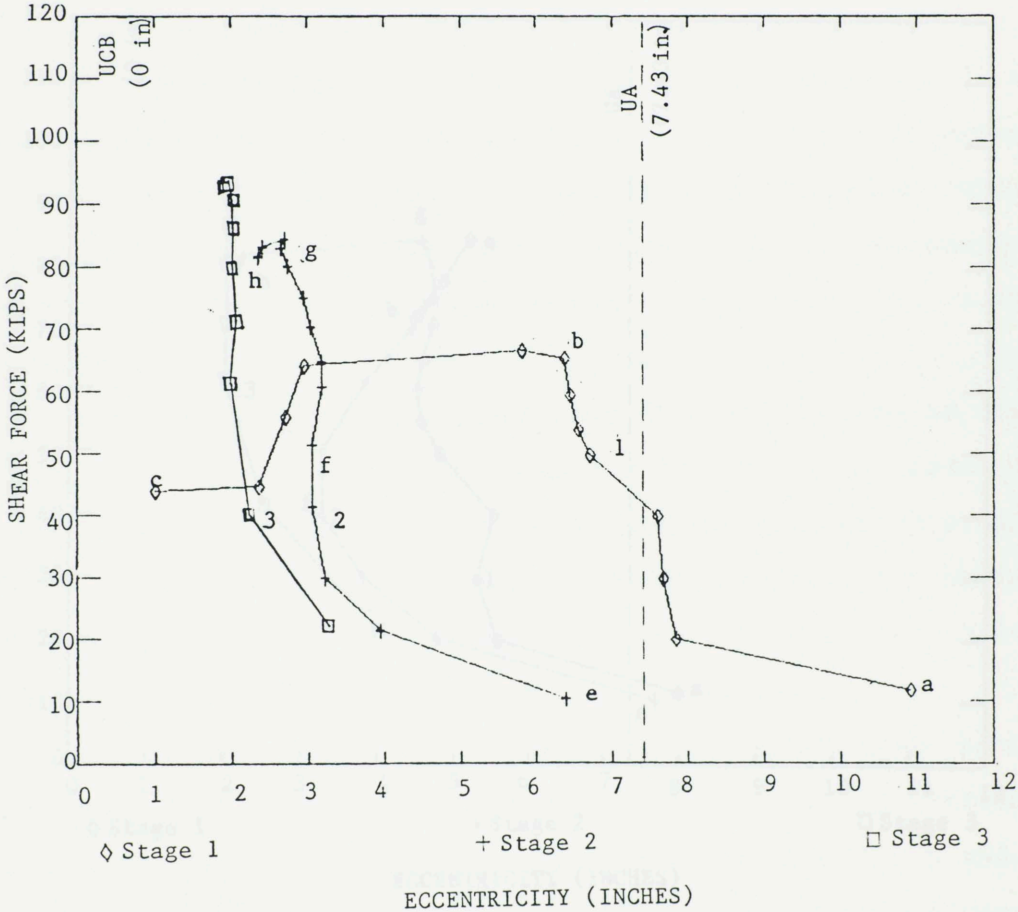

Fig. 3.28 Shear vs. Eccentricity (North End), Test 2a

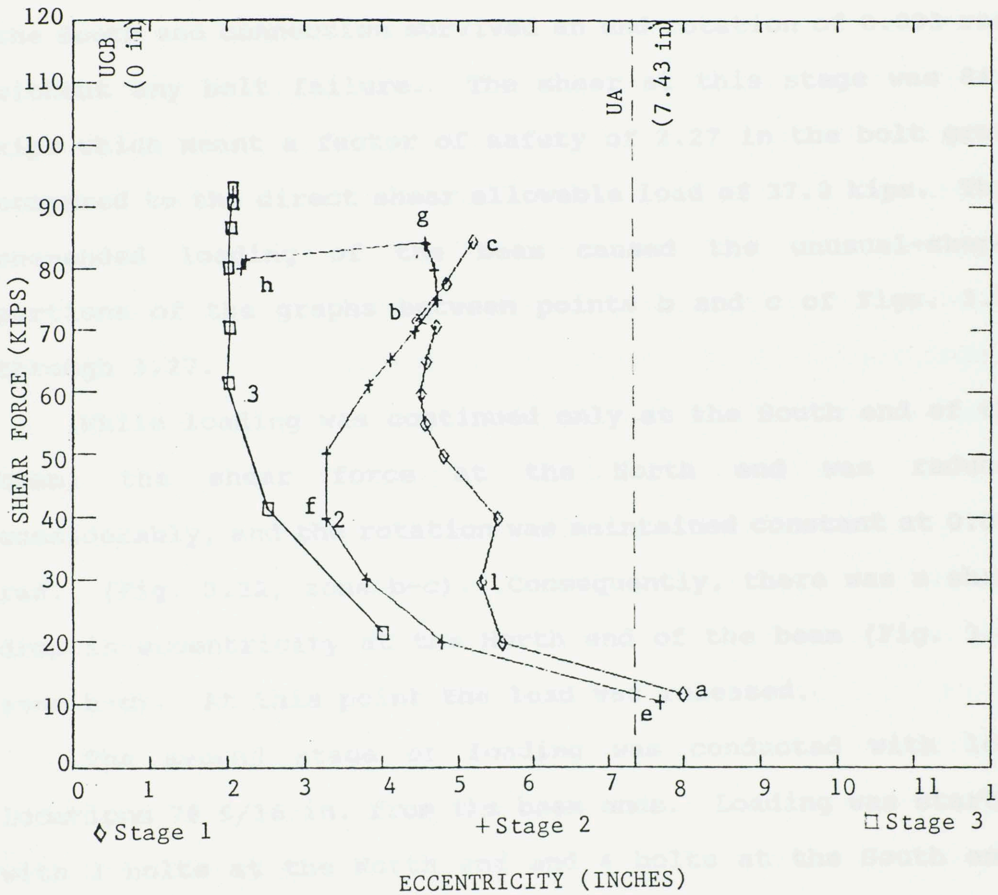

Fig. 3.29 Shear vs. Eccentricity (South End), Test 2a

shear load was still sustained in the connection. The rotation at both ends was held at 0.028 rad. It was then decided to stop further loading at the North end and continue loading at the South end of the beam. This was done to find out how much reserve rotational capacity was available in the South end connection before any bolt failure occurred. It was found that the South end connection survived an end rotation of 0.033 rad. without any bolt failure. The shear at this stage was 84.6 kips which meant a factor of safety of 2.27 in the bolt group compared to the direct shear allowable load of 37.2 kips. This one-ended loading of the beam caused the unusual-shaped portions of the graphs between points b and c of Figs. 3.22 through 3.27.

While loading was continued only at the South end of the beam, the shear force at the North end was reduced considerably, and the rotation was maintained constant at 0.029 rad. (Fig. 3.22, zone b-c). Consequently, there was a sharp drop in eccentricity at the North end of the beam (Fig. 3.28 zone b-c). At this point the load was released.

The second stage of loading was conducted with load locations 78 9/16 in. from the beam ends. Loading was started with 3 bolts at the North end and 4 bolts at the South end. The first occurrence of yielding was observed on the plate near the top bolt at both ends of the beam at a load of 80 kips. At this stage, the North connection had survived an end rotation of 0.034 rad. The eccentricity remained constant at 3 in. in the North end (see Fig. 3.28, zone e-f-g). At the South end the bolts were able to travel a short distance through the plate holes before going into bearing due to deformations from the previous loading. This caused a gradual reduction in eccentricity (see Fig. 3.29, zone e-f), to just above 3 in. at a load of 50 kips. After that, the eccentricity began to increase (see Fig. 3.29, zone f-g). Eventually, at a South end rotation of 0.038 rad., the topmost bolt at the South end also failed in shear. When this bolt failed the shear was still held constant at 81.6 kips, while the eccentricity dropped considerably at the South end of the beam (Fig. 3.29 zone g-h).

Now that there were 3 bolts at either end of the beam, it was decided to release the loads, change the load locations to 50 in. from the beam ends, and restart the loading to verify the capacity of the remaining 3-bolt connection. This is labeled on the figures as Loading Stage 3 (load location 50 in. from beam ends). With this load location no failure was observed in the connection even up to a load of 93 kips at each end. This actually meant a factor of safety of 3.33 in the bolt group compared to the 3-bolt concentric shear capacity of 27.9 kips. At this point the connection had survived an end rotation of 0.032 rad. and the eccentricity was 2 in. in both ends.

3.4.4 <u>Behavior of Test Specimen 2b:</u>

In this test short slotted holes instead of standard round holes were punched into the plate. The basic purpose for this variation was to study the comparative effectiveness of short slotted holes over standard round holes in the shear tab. In order to accomplish this objective, the same load locations as in test 2a were maintained at the different stages of loading.

During the first stage of loading, the connection behavior was very similar to that observed in test 2a up to a load of 59.4 kips at each end. There was slight seating of the bolts within the slotted holes. This is evident from the slight non-linearity in the shear-rotation curves (see Figs. 3.30 and 3.31, curve 1). Due to plowing of the bolts through the bolt holes and subsequent release of the beam end moment, there was a sharp drop in eccentricity in the initial stage (see Figs. 3.36 and 3.37, zone a-b). Later on, the eccentricity was maintained constant at nearly 4 in. away from the bolt line toward the beam center.

Finally, the connection survived an end rotation of 0.03 rad. at loads of 64 and 68 kips at the North and South ends, respectively. Slight yielding of the plate along the bolt line was also noticed at both ends. The central deflection of the beam was found to be 4 in. (see Figs. 3.34 and 3.35, curve 1).

It is interesting to note that with slotted holes, an end rotation of 0.03 rad. could be attained without any bolt failure like those observed in test 2a. This demonstrates the

Table 3.5: Details of Test Specimen 2b

BEAM SIZE AND DIMENSION :

Beam Size :	W18x76
Beam Length :	33 ft.
Beam Material :	A36
Beam Yield Strength :	46.1 ksi (from mill certificate)
Design Reaction :	37.2 kips
Load Location :	92 in. from beam ends
Bolt Holes :	13/16 in. round (drilled)

CONNECTION SIZE AND DIMENSION :

No. of Bolts :	4
Type of Bolts :	A325-N
Bolt Spacing :	3 in.
Edge Distance :	1-1/2 in.
Plate Thickness :	3/8 in.
Plate Size :	5 in. x 12 in.
Plate Material :	A36
Plate Yield Strength :	47.4 ksi (from tensile tests)
Weld Size :	5/16 in. double fillet
Bolt Holes in Plate :	13/16 in. x 1 in. horizontal short slots (punched)
Column Size :	W24x117

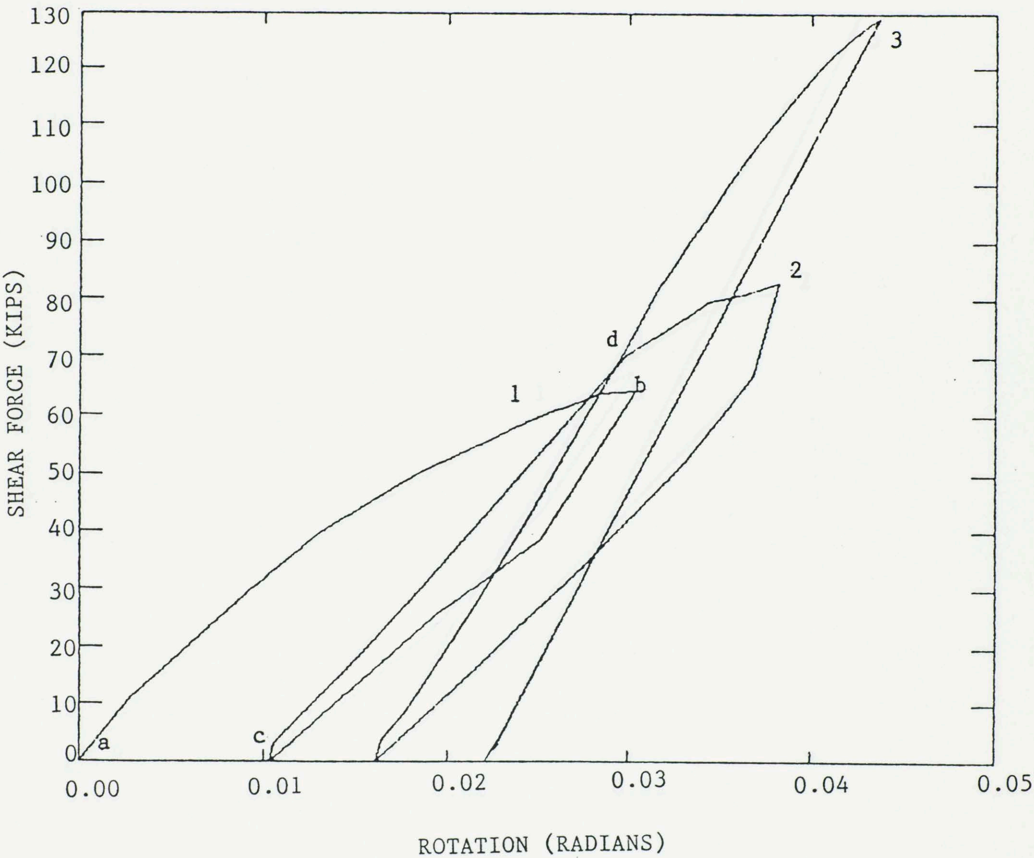

Fig. 3.30 Shear vs. Rotation (North End), Test 2b

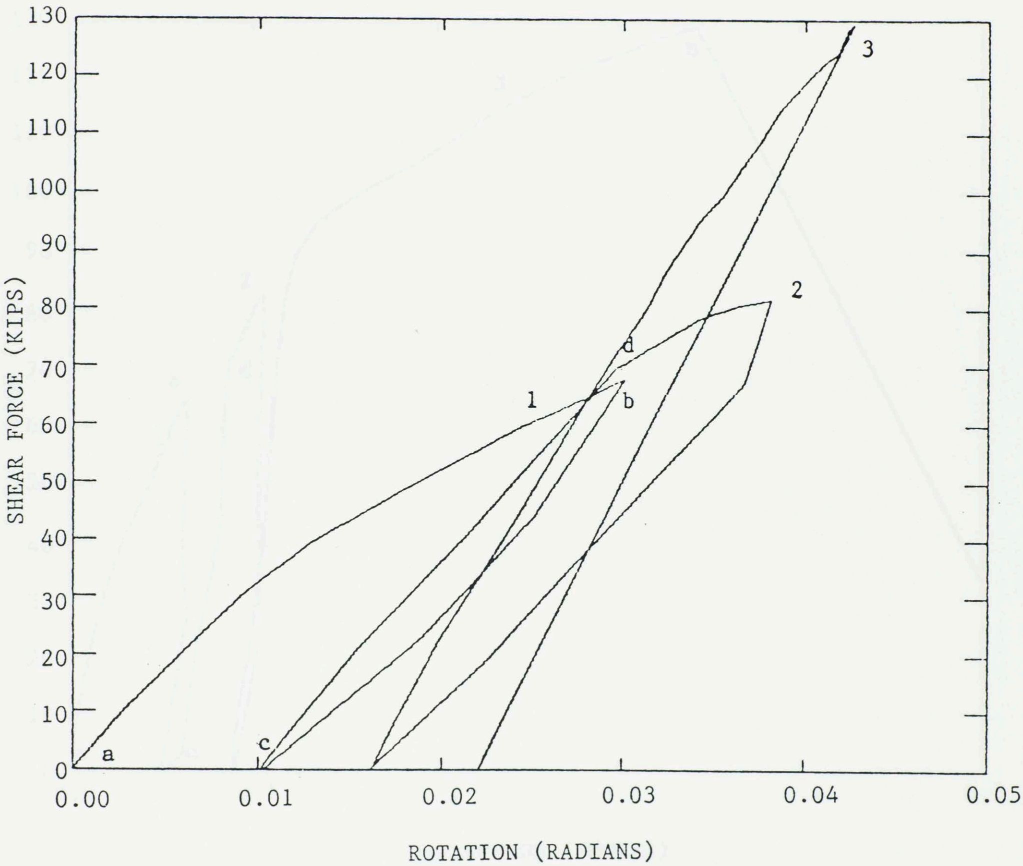

Fig. 3.31 Shear vs. Rotation (South End), Test 2b

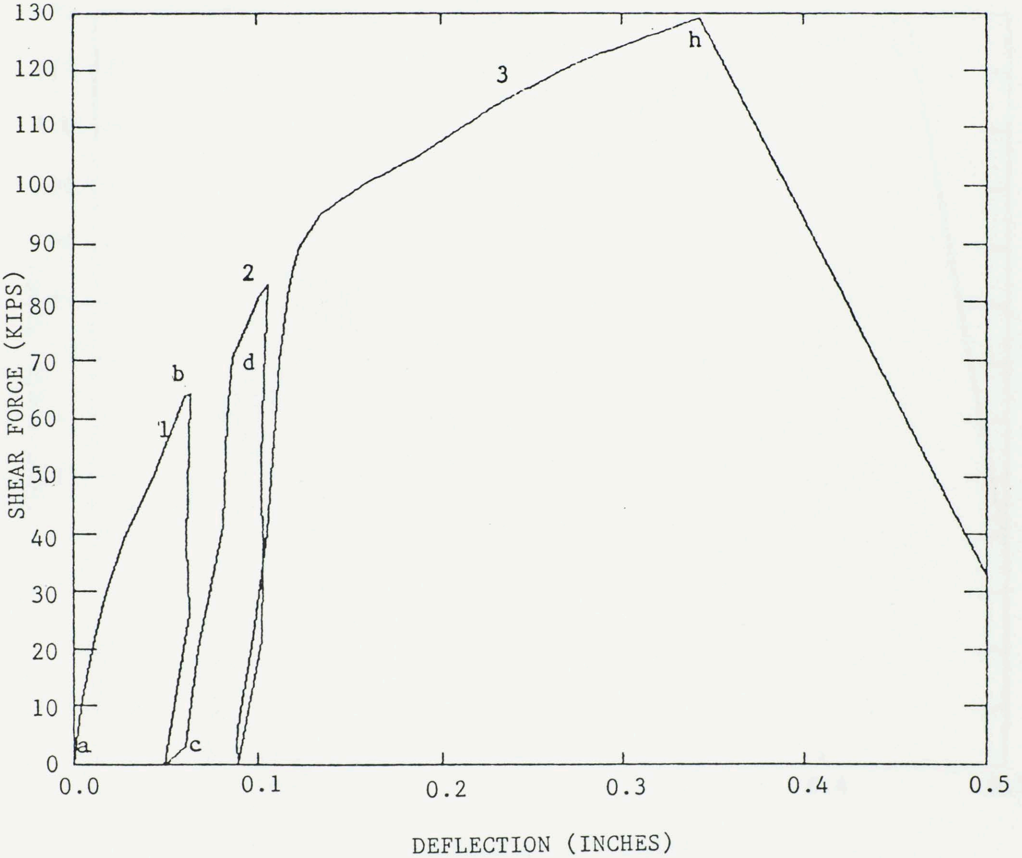

Fig. 3.32 Shear vs. Bolt Line Deflection (North End), Test 2b

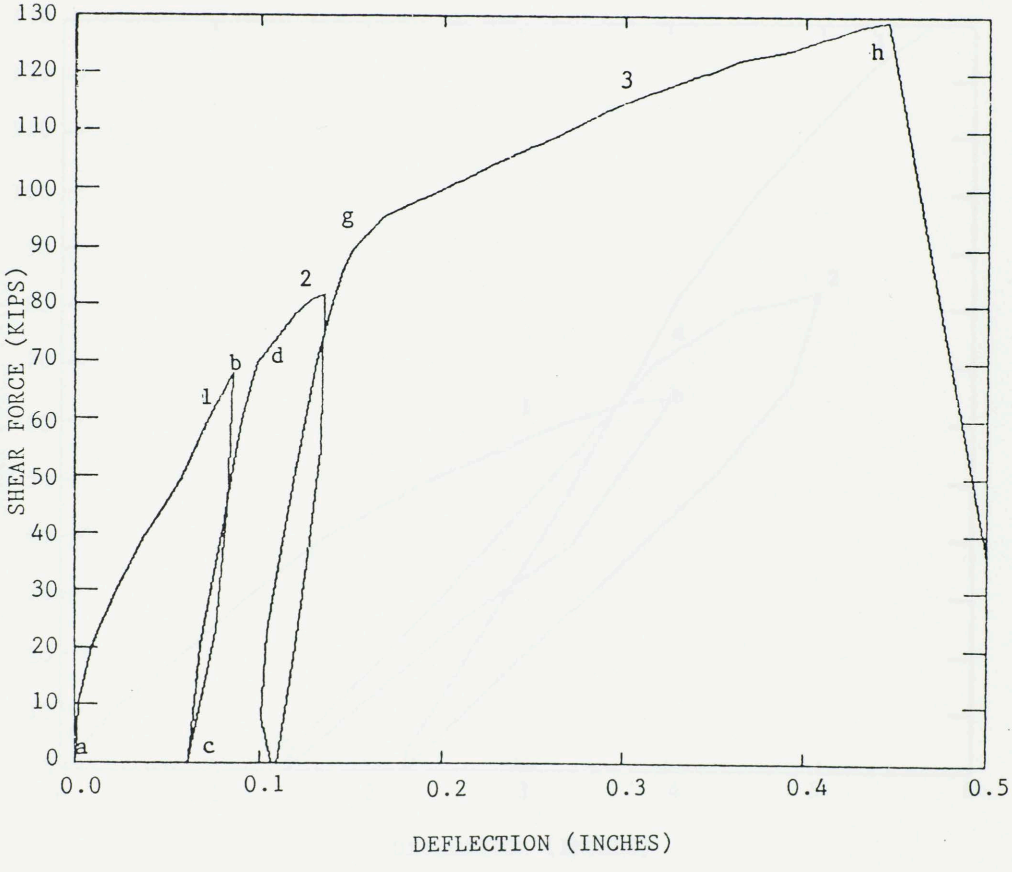

Fig. 3.33 Shear vs. Bolt Line Deflection (South End), Test 2b

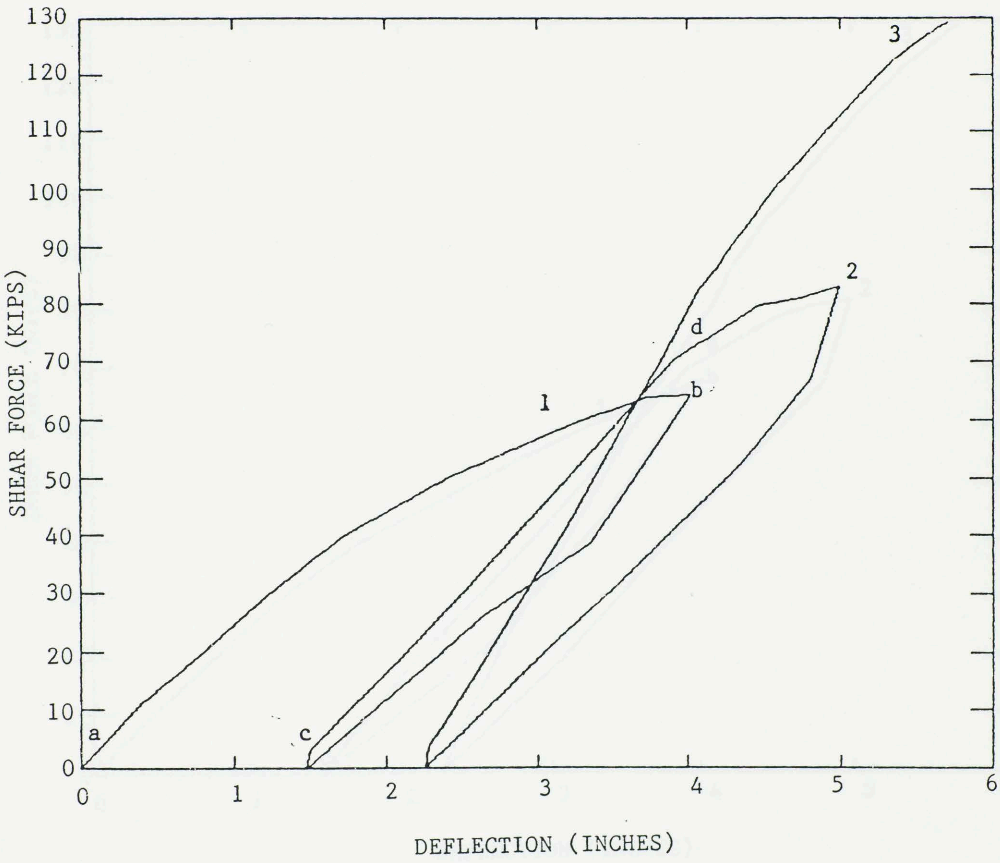

Fig. 3.34 Shear vs. Central Deflection (North End), Test 2b

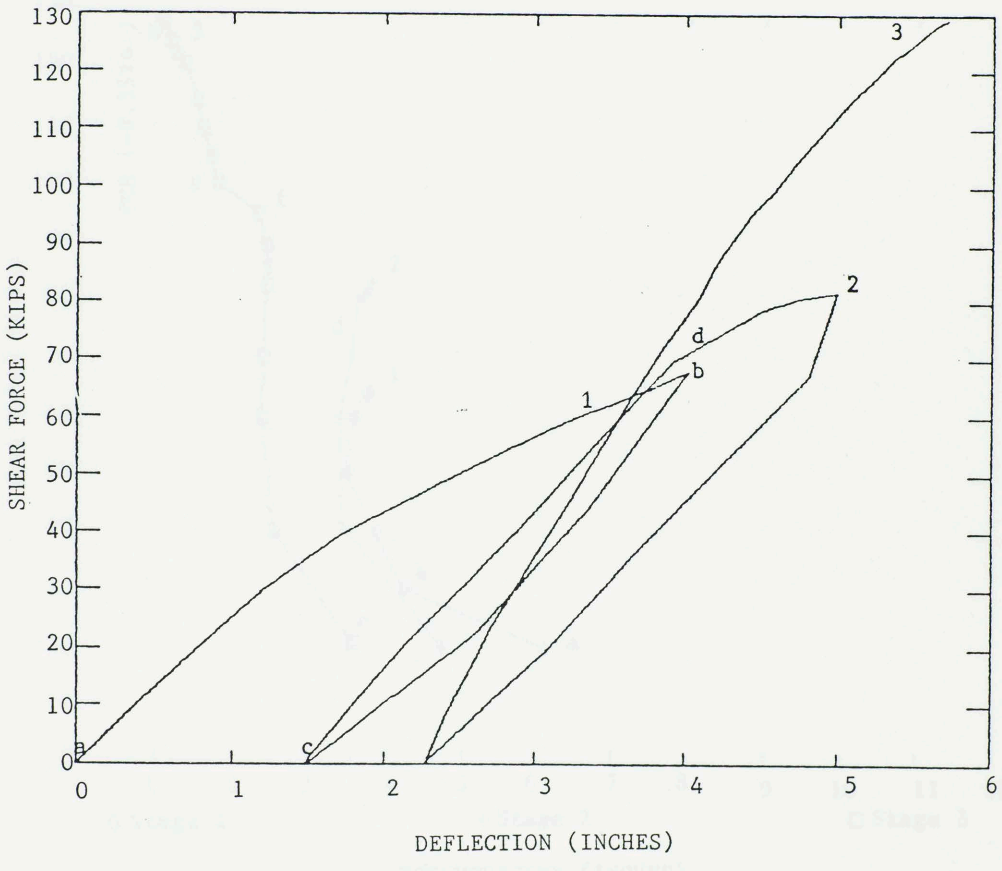

Fig. 3.35 Shear vs. Central Deflection (South End), Test 2b

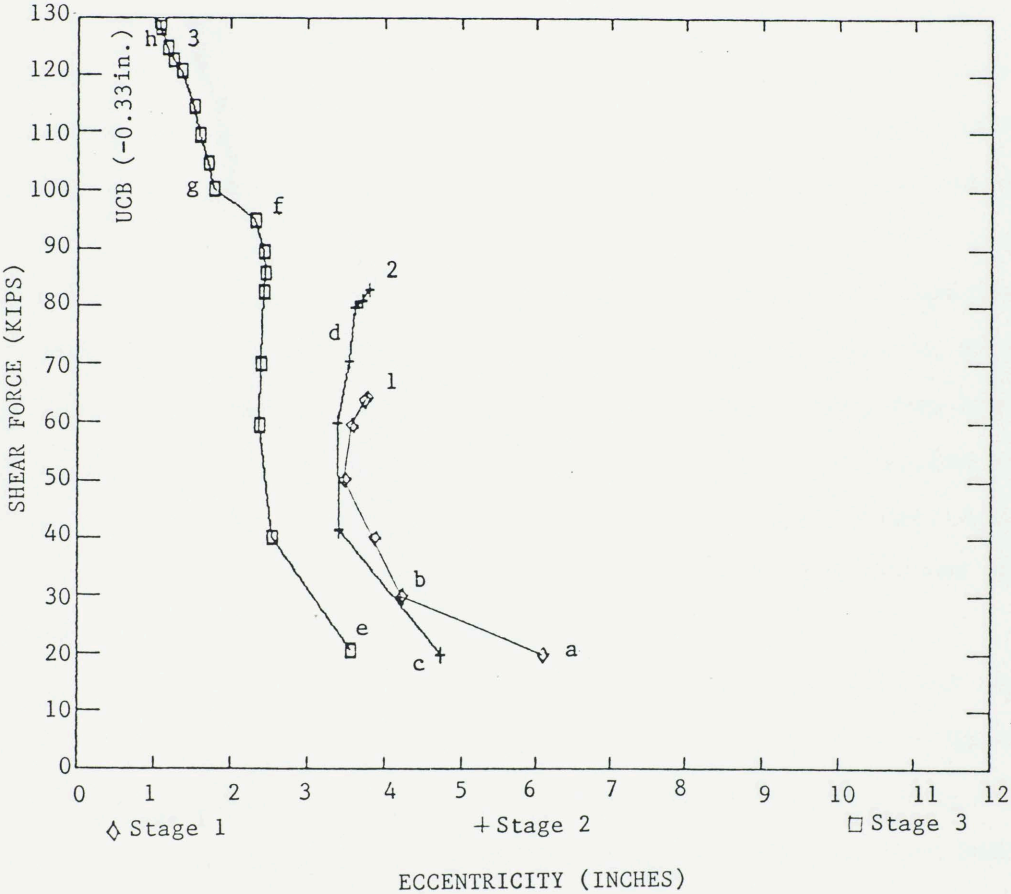

Fig. 3.36 Shear vs. Eccentricity (North End), Test 2b

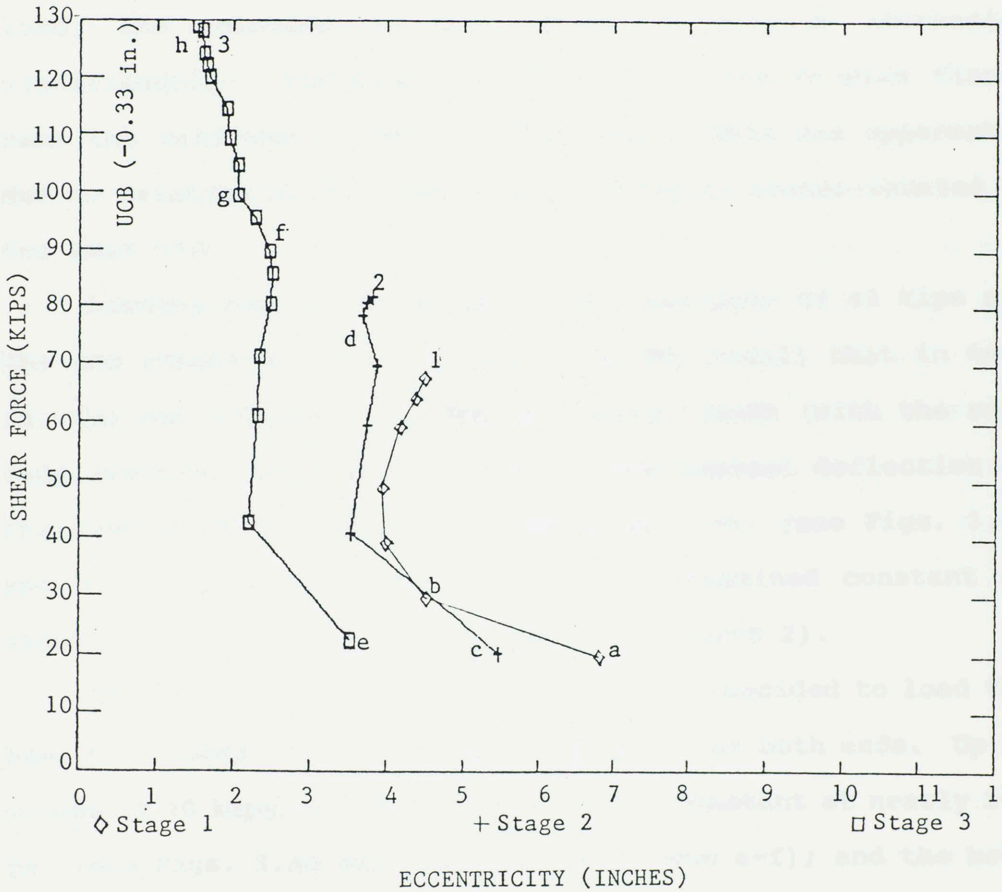

Fig. 3.37 Shear vs. Eccentricity (South End), Test 2b

obvious contribution of the slotted holes to the rotational capacity of the connection.

A linear shear-rotation relationship (see Fig. 3.30 and 3.31, curve 2, zone c-d) was maintained up to a load of 70 kips at each end during the second stage of loading. The connection survived an end rotation of 0.03 rad. On further increments of load, the rotation of the connection started increasing significantly. Yielding of the plate at the tension flange near the weld was noticed at both ends. This was apparently due to twisting of the beam ends, limited by braces located at the beam ends.

Loading was continued up to end reactions of 83 kips and the end rotations of 0.038 rad. One may recall that in test 2a, the end rotation attained at similar loads (with the same load location) was just 0.033 rad. The central deflection of the beam at this stage was found to be 5 in. (see Figs. 3.34 and 3.35, curve 2). The eccentricity remained constant at about 3.5 in. (see Figs. 2.36 and 2.37, curve 2).

For the third stage of loading, it was decided to load the beam to ultimate failure of the connection at both ends. Up to a load of 90 kips, eccentricity remained constant at nearly 2.5 in. (see Figs. 3.36 and 3.37, curve 3, zone e-f); and the bolt line deflection was constant at around 0.1 in. (see Figs. 3.32 and 3.33, curve 3, zone e-f).

Beyond 90 kips, the bolt line deflection started to increase significantly (see Figs. 3.32 and 3.33, curve 3, zone g-h); and so, anticipating sudden failure in the bolts or otherwise, it was decided to take the readings at every 5 kips interval, and observe the connection behavior at every step. Due to the onset of excessive seating of the bolts through the bolt holes, some end moments were released, and consequently, there was a gradual drop in eccentricity (see Figs. 3.36 and 3.37, curve 3, zone g-h). At a load of 105 kips significant plate shear yielding was observed near the weld, concentrated mostly between the weld line and the bolt line. This load corresponds to a factor of safety of 2.82 for the bolt group.

At a load of 120 kips, shear yielding of the beam web near the connection became apparent. Finally, at a load of 129 kips at each end, all the bolts at both ends failed in shear. This sudden bolt failure is indicated by the sharp drop in shear-bolt line deflection curve (see Fig. 3.32 and 3.33, curve 3, point h). The connections attained an end rotation of 0.042 rad. prior to failure.

3.4.5 <u>Behavior of Test Specimen 3a:</u>

This test specimen used a 6-bolt connection with round holes as listed previously in Table 3.1. Details of the test specimen are listed in Table 3.6. The first load locations were at 79 3/4 in. from the beam ends, as determined by the beam yield strength and section modulus and bolt group concentric shear allowable load. Results of this test are presented in Figs. 3.38 through 3.45.

Table 3.6: Details of Test Specimen 3a

BEAM SIZE AND DIMENSION :

Beam Size :	W21x93
Beam Length :	25 ft.
Beam Material :	A36
Beam Yield Strength :	45.5 ksi (from mill certificate)
Design Reaction :	55.8 kips
Load Location :	79-3/4 in. from beam ends
Bolt Holes:	13/16 in. round (drilled)

CONNECTION SIZE AND DIMENSION :

No. of Bolts :	6
Type of Bolts :	A325-N
Bolt Spacing :	3 in.
Edge Distance :	1-1/2 in.
Plate Thickness :	3/8 in.
Plate Size :	5 in. x 18 in.
Plate Material :	A36
Plate Yield Strength:	47.4 ksi (from tensile tests)
Weld Size :	5/16 in. double fillet
Bolt Holes in Plate:	13/16 in. round (punched)
Column Size :	W24x117

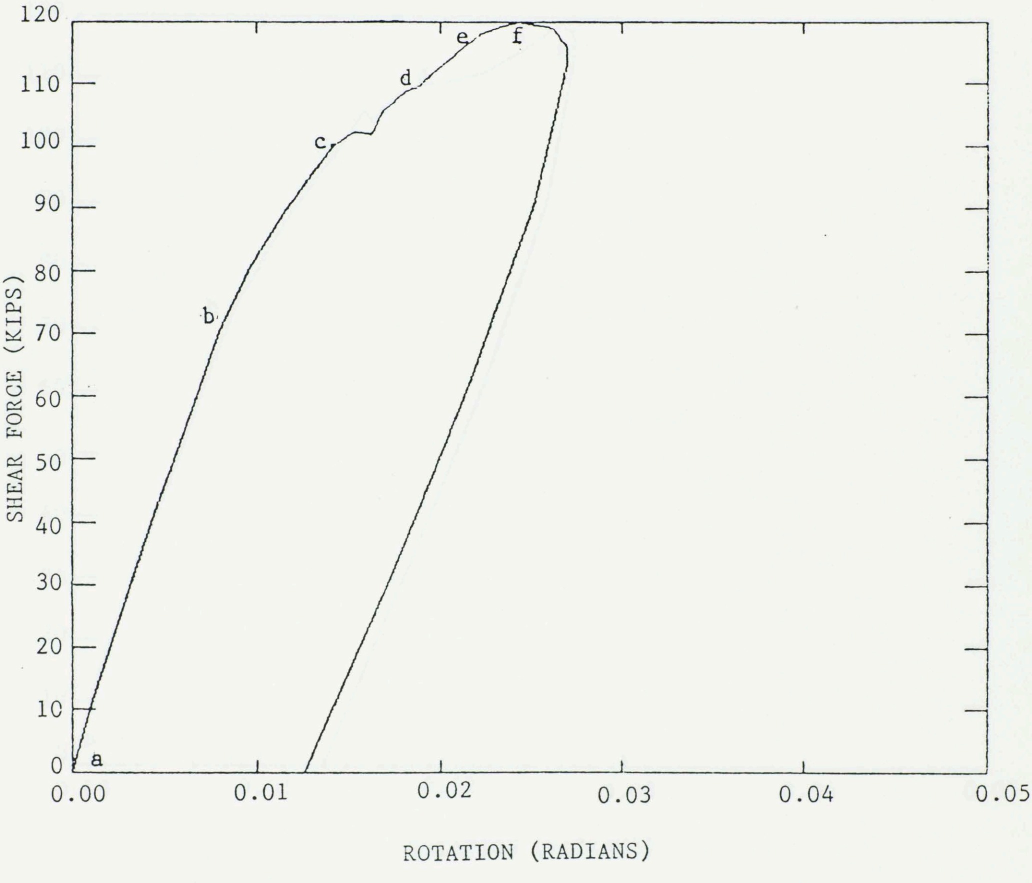

Fig. 3.38 Shear vs. Rotation (North End), Test 3a

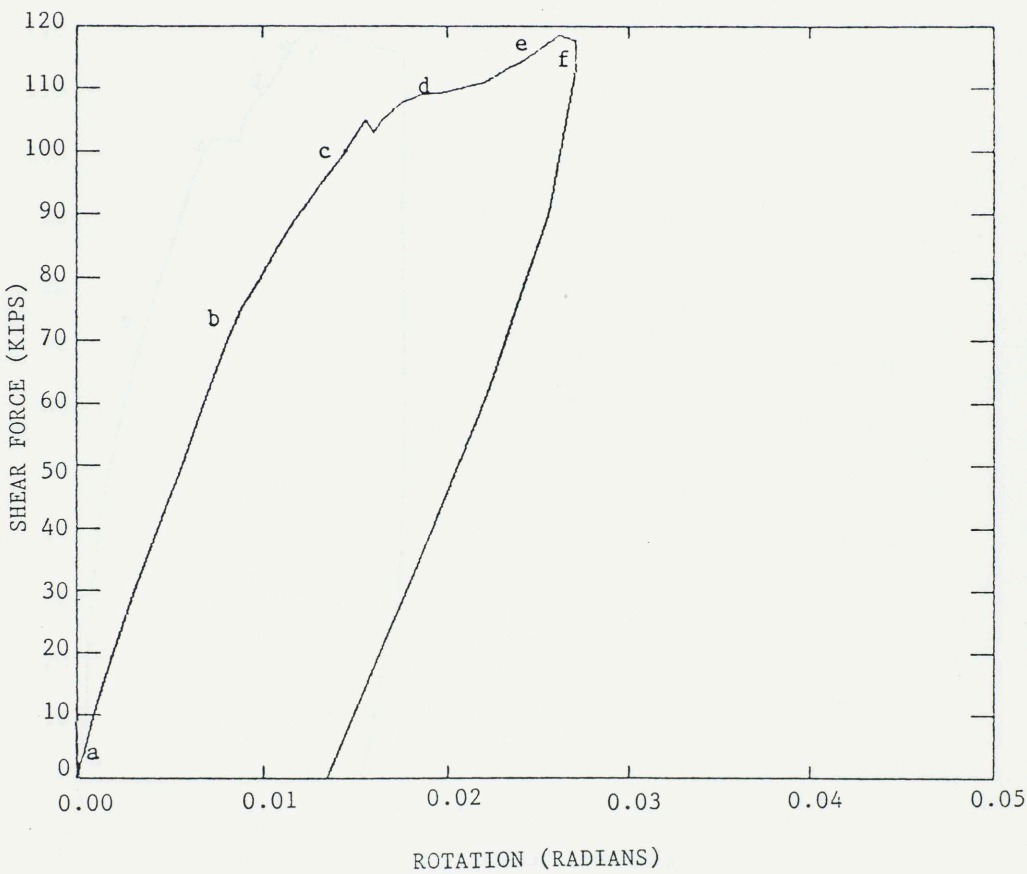

Fig. 3.39 Shear vs. Rotation (South End), Test 3a

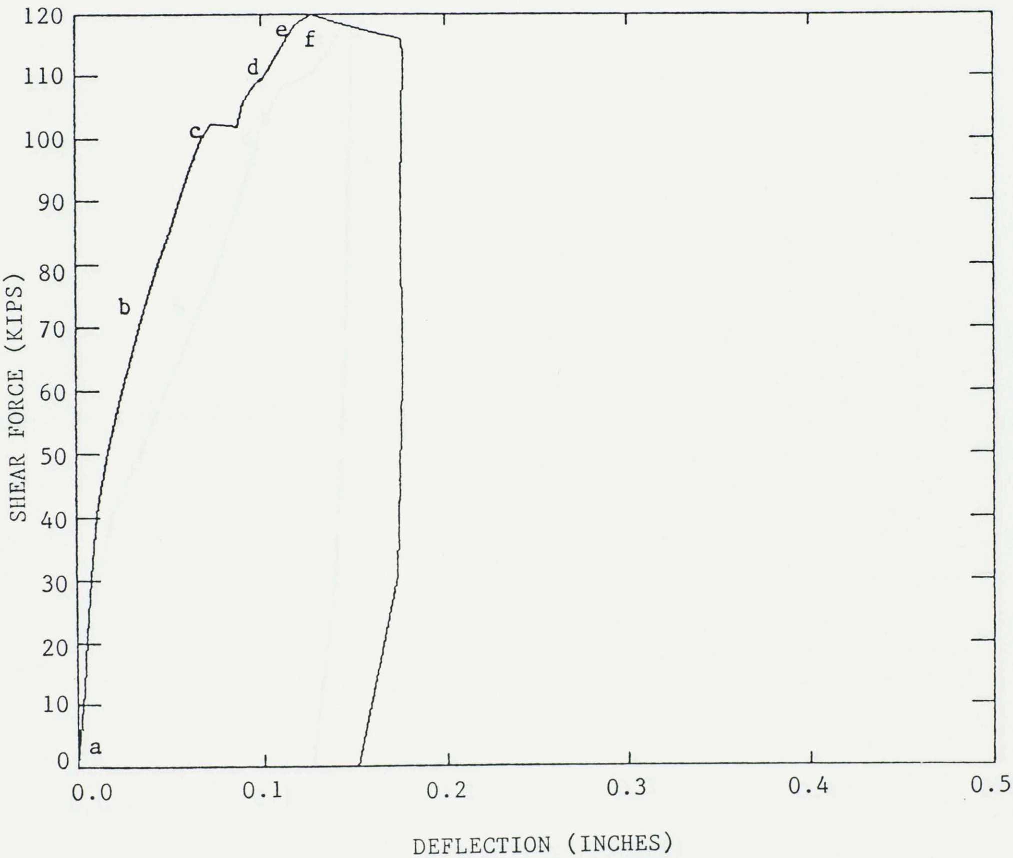

Fig. 3.40 Shear vs. Bolt Line Deflection (North End), Test 3a

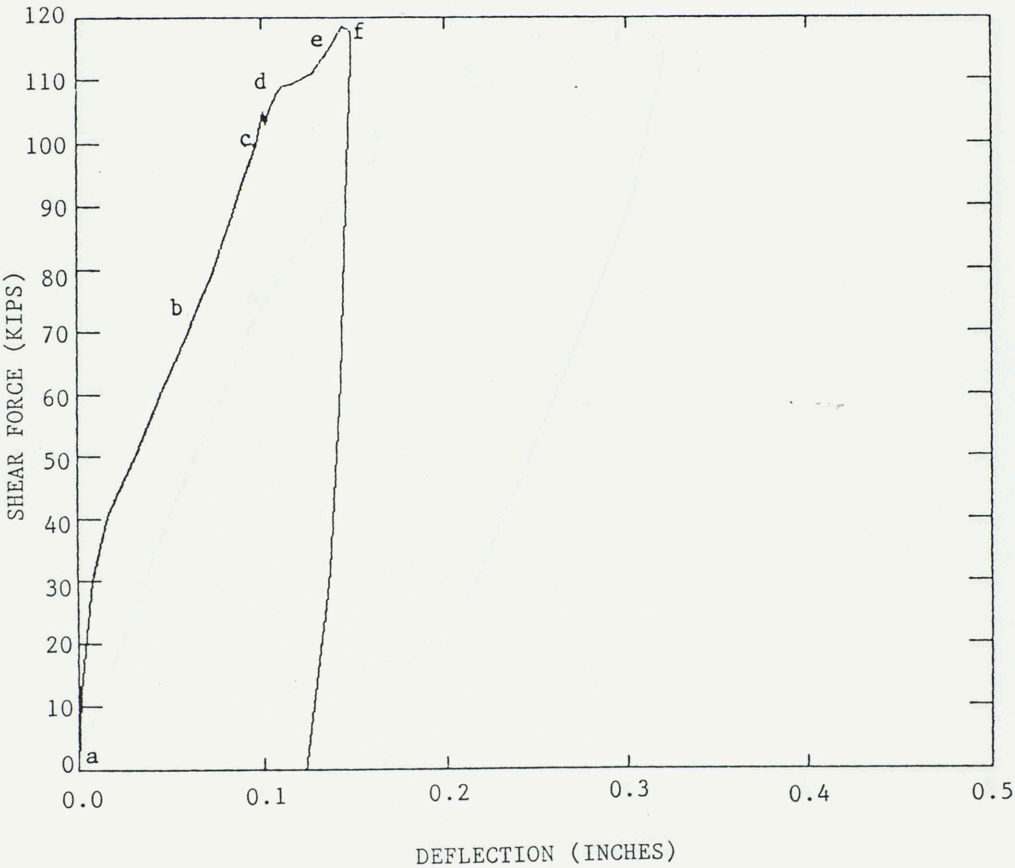

Fig. 3.41 Shear vs. Bolt Line Deflection (South End), Test 3a

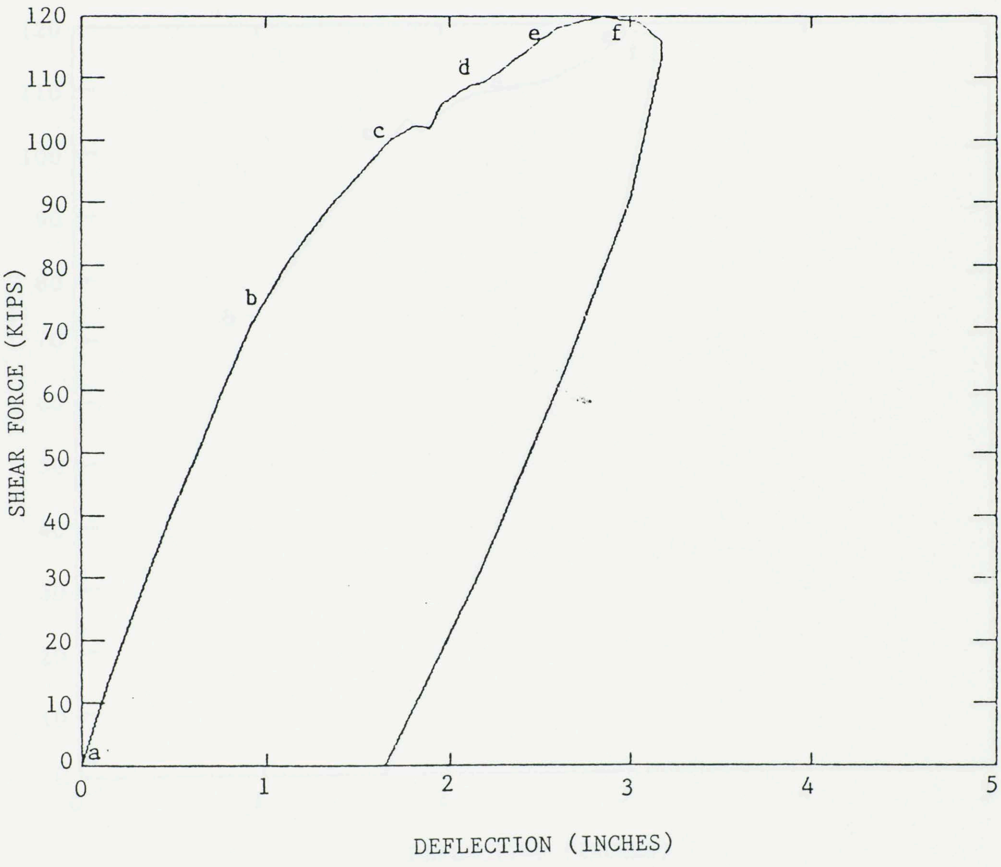

Fig. 3.42 Shear vs. Central Deflection (North End), Test 3a

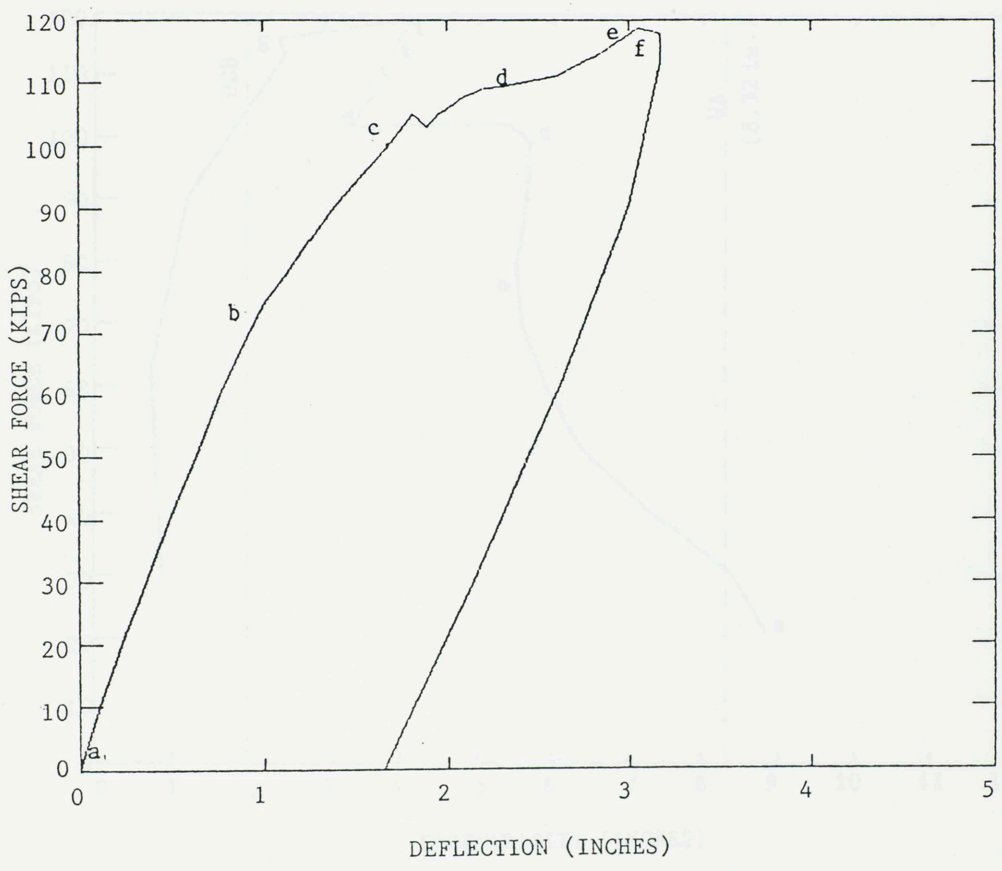

Fig. 3.43 Shear vs. Central Deflection (South End), Test 3a

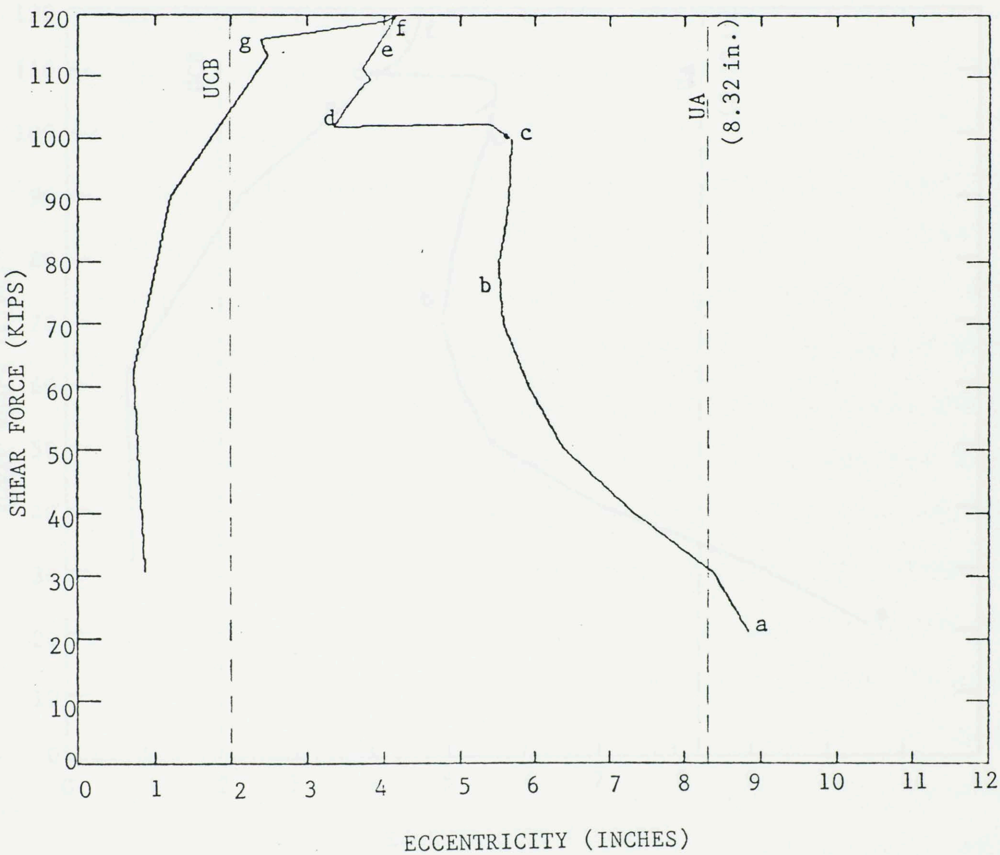

Fig. 3.44 Shear vs. Eccentricity (North End), Test 3a

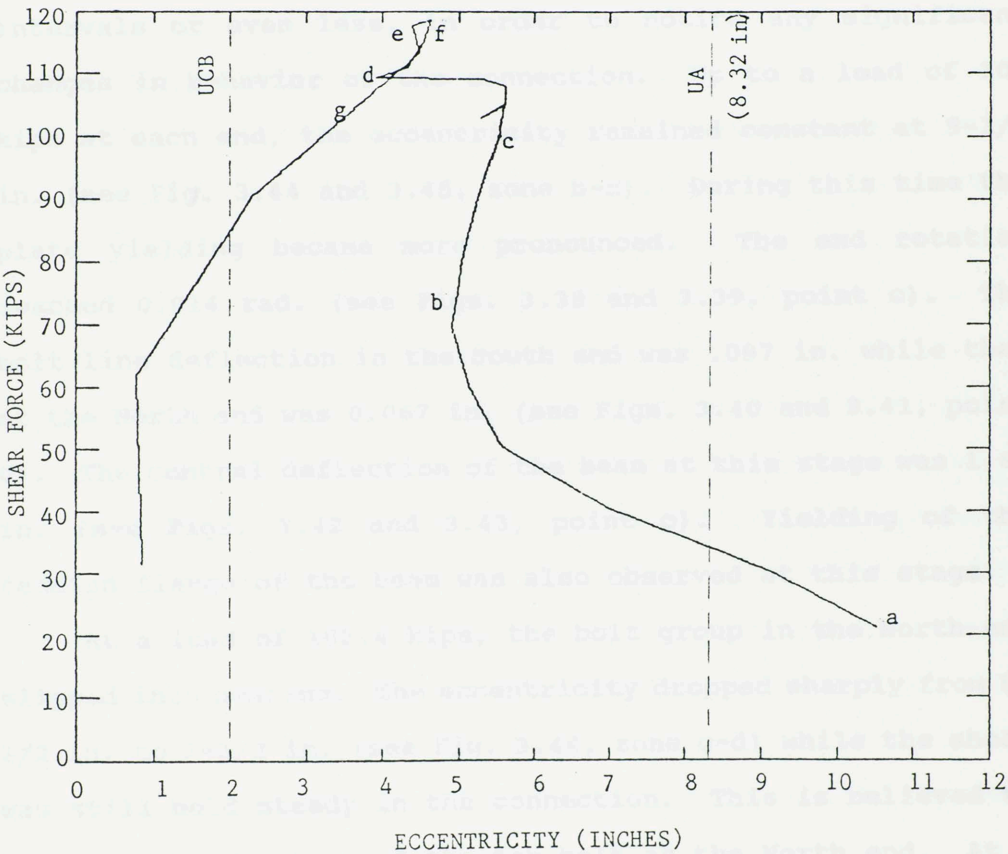

Fig. 3.45 Shear vs. Eccentricity (South End), Test 3a

Yielding of the plates near the topmost bolts was first observed at a load of 70 kips at each end. During this stage the gradual release of end moment due to plowing of the bolts through the bolt holes caused the eccentricity to drop to nearly 5-1/2 in. at both ends (see Figs. 3.44 and 3.45, zone a-b). From 70 kips onwards, readings were taken at every 5 kips intervals or even less, in order to notice any significant changes in behavior of the connection. Up to a load of 100 kips at each end, the eccentricity remained constant at 5-1/2 in. (see Fig. 3.44 and 3.45, zone b-c). During this time the plate yielding became more pronounced. The end rotation reached 0.014 rad. (see Figs. 3.38 and 3.39, point c). The bolt line deflection in the South end was .097 in. while that at the North end was 0.067 in. (see Figs. 3.40 and 3.41, point c). The central deflection of the beam at this stage was 1.68 in. (see Figs. 3.42 and 3.43, point c). Yielding of the tension flange of the beam was also observed at this stage.

At a load of 102.4 kips, the bolt group in the North end slipped into bearing. The eccentricity dropped sharply from 5-1/2 in. to 3-1/2 in. (see Fig. 3.44, zone c-d) while the shear was still held steady in the connection. This is believed to be caused by fracture of the top bolt at the North end. At a load of 109.3 kips, the bolt group in the South end also slipped into bearing caused by the top bolt shearing. The eccentricity dropped from 5-1/2 in. to 4 in. (see Fig. 3.45, zone c-d). As before, the shear remained steady in the connection. The rotation at both ends was 0.0195 rad. (see Figs. 3.38 and 3.39, point d).

It was then decided to take the readings based on rotation increments instead of load increments. When the rotation reached 0.0243 rad. at each end, plate yielding between the weld and bolt lines became apparent. The bolt line deflection was 0.13 in. indicating significant plowing of the bolts, particularly the topmost ones at both ends (see Figs. 3.40 and 3.41, zone d-e). Once again, the eccentricity started picking up gradually (see Figs. 3.44 and 3.45, zone d-e). The reason for this increase is that there was no significant release of end moment after the bolt groups went into bearing, while the shear in the connection increased steadily.

Finally, at an end rotation of 0.027 rad. at each end, the second North end bolt failed in shear and the two topmost bolts in the North and one in the South end fell from the specimen. The point at which the failure occurred is indicated by point "f" in Figs. 3.38 through 3.45. Once again, there was drop in eccentricity associated with the bolt failure (see Figs. 3.44 and 3.45, zone f-g). An end shear of 119 kips was still held steady in the connection at both ends. At this point, the test was stopped and the load released.

Since the behavior of this 6-bolt connection with standard round plate holes turned out to be similar to that of the corresponding 4-bolt connection, it was felt that there was no reason in testing the beam further by changing the load location. Hence, all the figures 3.38 through 3.45 indicate single stage loading.

3.4.6 Behavior of Test Specimen 3b:

This test was a repeat of test 3a, except that the connection plates had horizontal short slots instead of round holes as listed in Table 3.7. The load locations for the first stage were the same as in test 3a (79-3/4 in. from beam ends). Results of this test are presented in Figs. 3.46 through 3.53.

Except for the magnitude and location of load, the behavior of the test specimens in the 6-bolt connection with slotted plate holes was almost the same as that of the corresponding 4-bolt connection. Similar to the 4-bolt slotted specimen, the eccentricity dropped gradually in the initial stages of loading until it remained constant at 4 in. away from the bolt line toward the beam center up to a load of 80 kips at each end (see Figs. 3.52 and 3.53, curve 1). At 90 kips load, plate yielding along the bolt line became noticeable. At a load of 105 kips the rotation was 0.022 rad. (see Figs. 3.46 and 3.47, zone a-b). It was then decided to control the loading through rotation increments instead of load increments. At 0.026 rad. end rotation, plate yielding became apparent around the top four bolts. Yielding of the beam tension flange was observed at 0.028 rad. end rotation. At 0.03 rad., plate yielding was observed along the entire bolt line. The central deflection of the beam reached 3.22 in. (see Figs. 3.50 and

Table 3.7: Details of Test Specimen 3b

BEAM SIZE AND DIMENSION :

Beam Size :	W21x93
Beam Length :	25 ft.
Beam Material :	A36
Beam Yield Strength :	45.5 ksi (from mill certificate)
Design Reaction :	55.8 kips
Load Location :	79-3/4 in. from beam ends
Bolt Holes:	13/16 in. round (drilled)

CONNECTION SIZE AND DIMENSION :

No. of Bolts :	6
Type of Bolts :	A325-N
Bolt Spacing :	3 in.
Edge Distance :	1-1/2 in.
Plate Thickness :	3/8 in.
Plate Size :	5 in. x 18 in.
Plate Material :	A36
Plate Yield Strength:	47.4 ksi (from tensile tests)
Weld Size :	5/16 in. double fillet
Bolt Holes in Plate:	13/16 in. x 1 in. horizontal short slots (punched)
Column Size :	W24x117

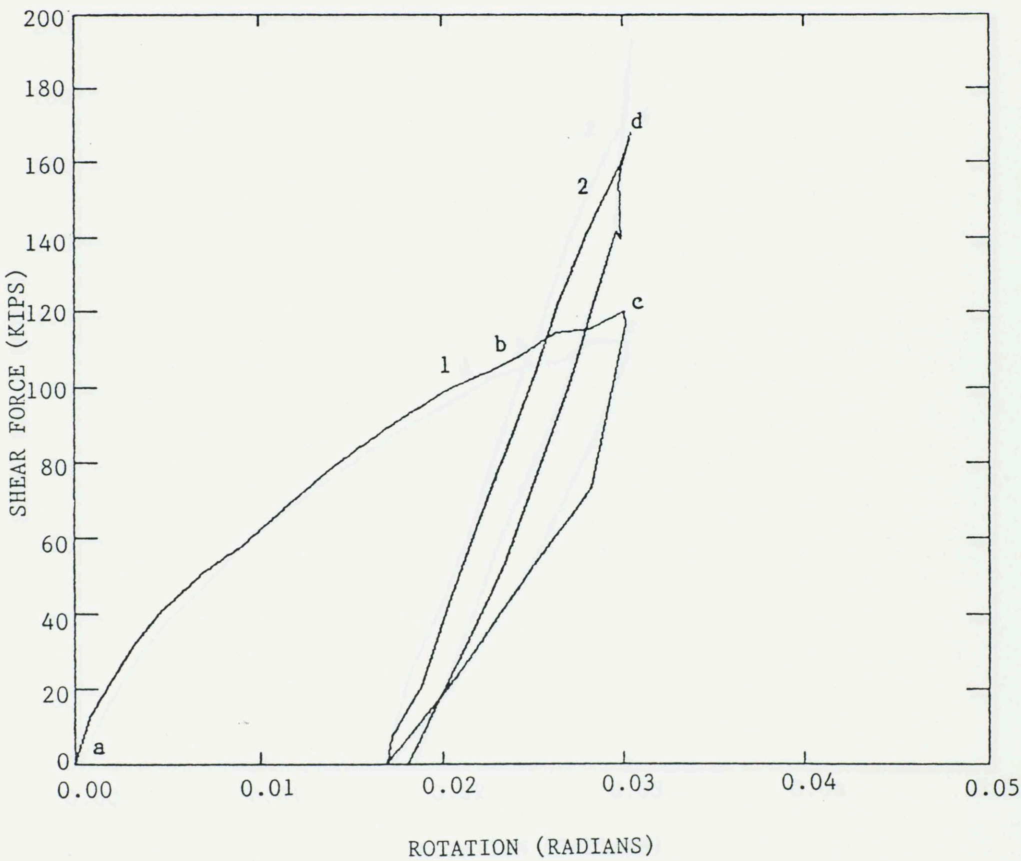

Fig. 3.46 Shear vs. Rotation (North End), Test 3b

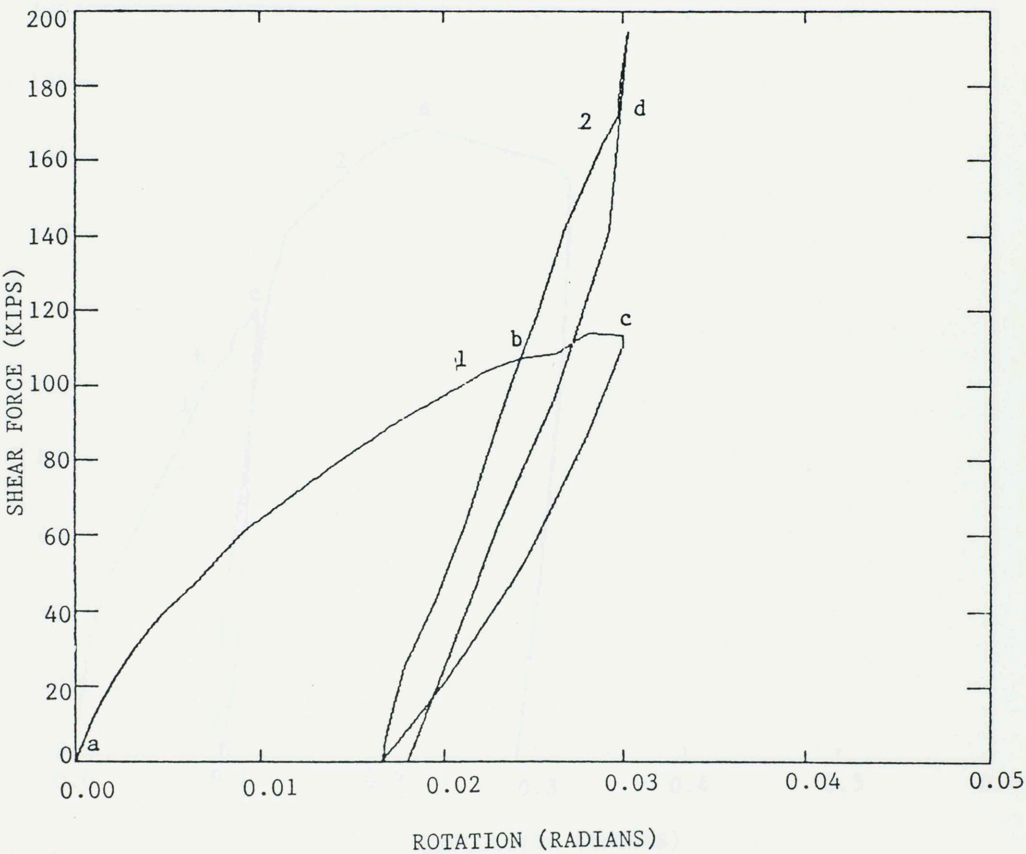

Fig. 3.47 Shear vs. Rotation (South End), Test 3b

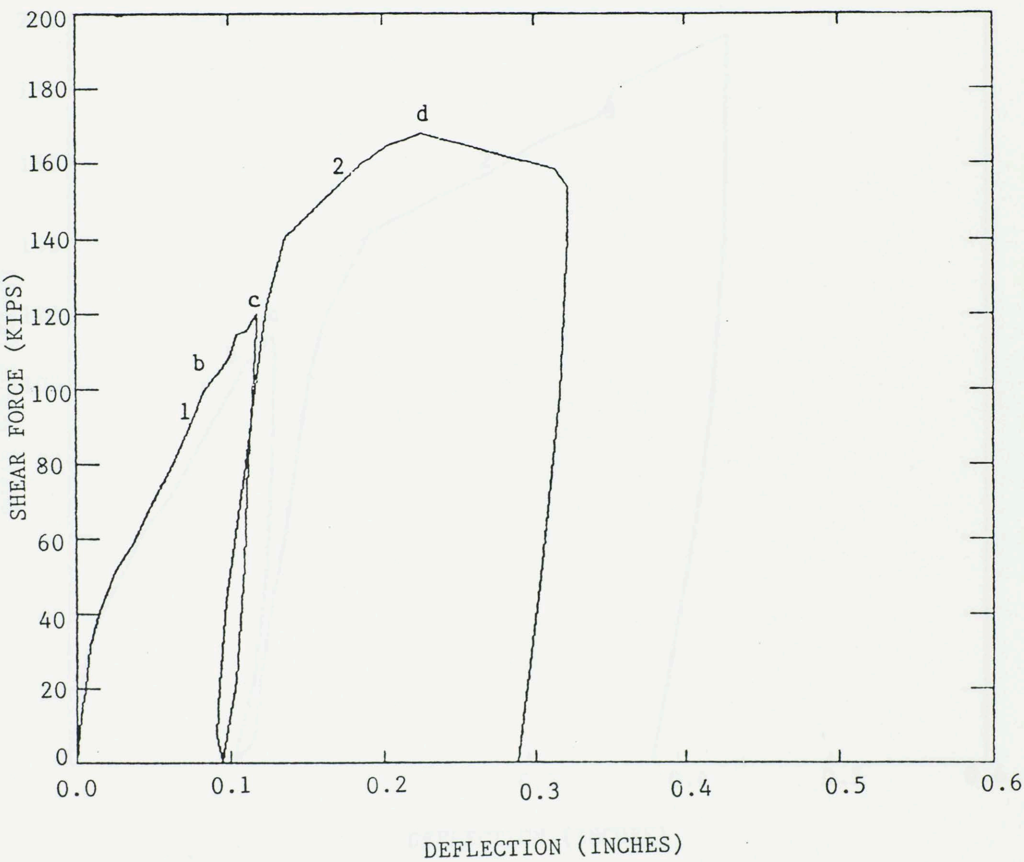

Fig. 3.48 Shear vs. Bolt Line Deflection (North End), Test 3b

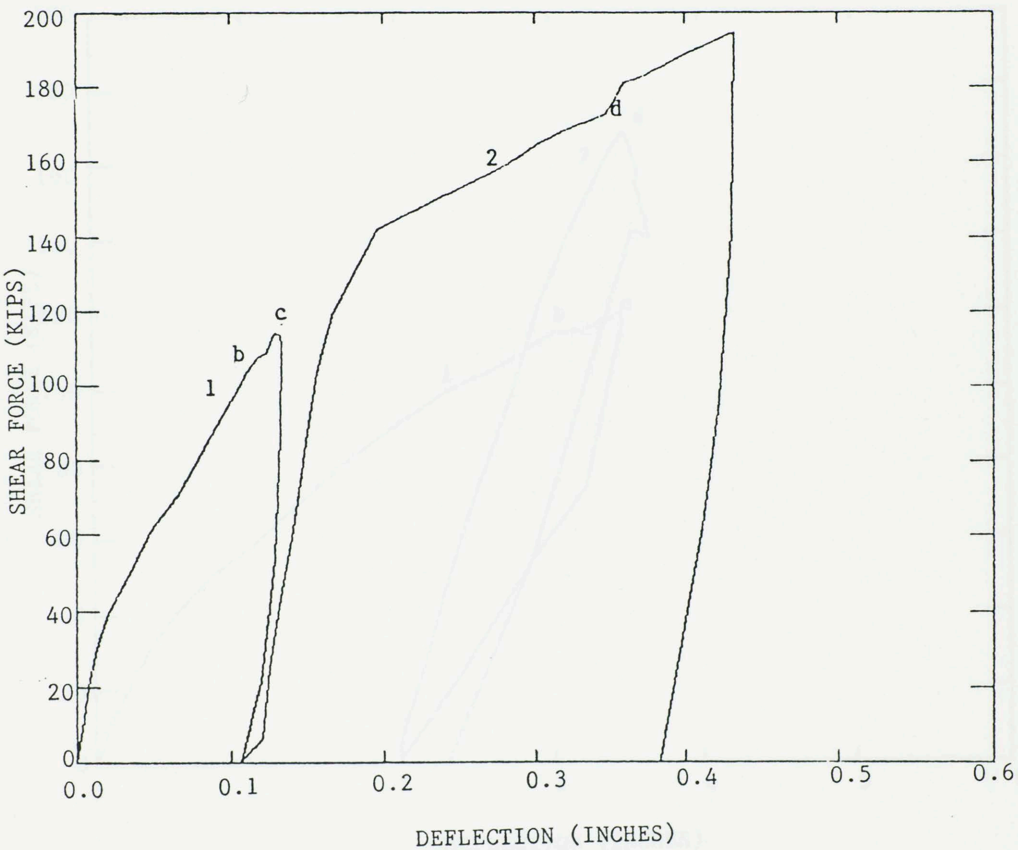

Fig. 3.49 Shear vs. Bolt Line Deflection (South End), Test 3b

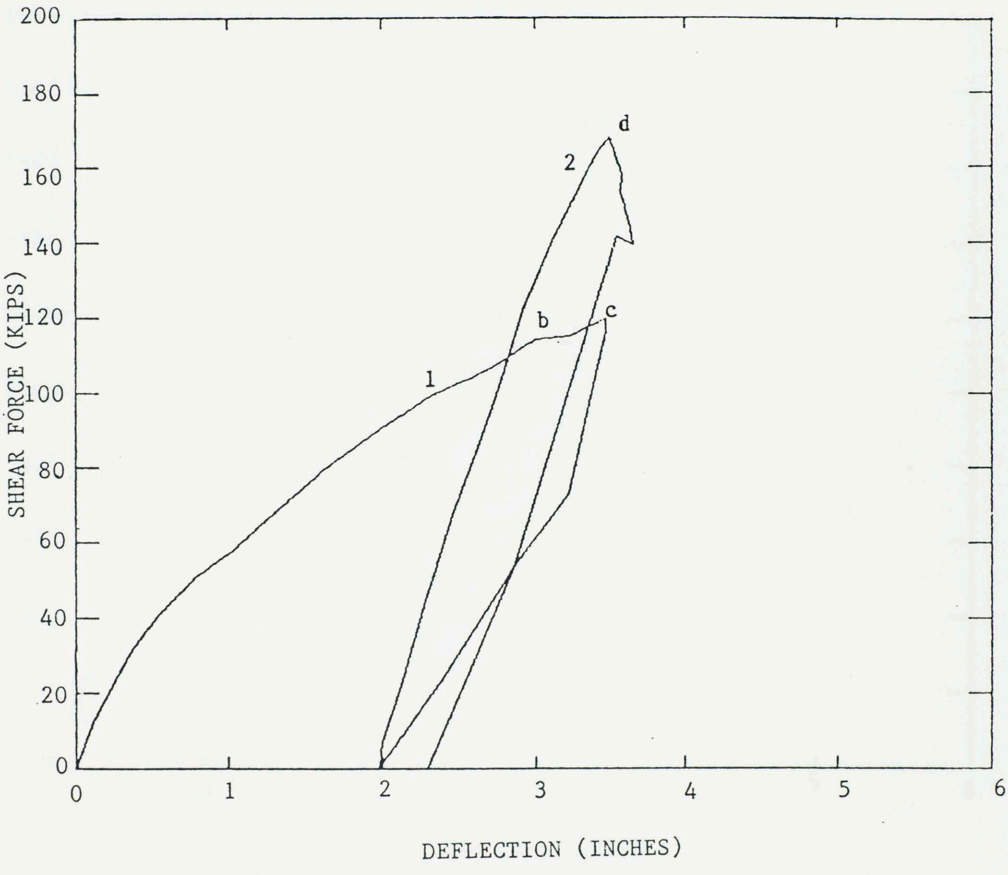

Fig. 3.50 Shear vs. Central Deflection (North End), Test 3b

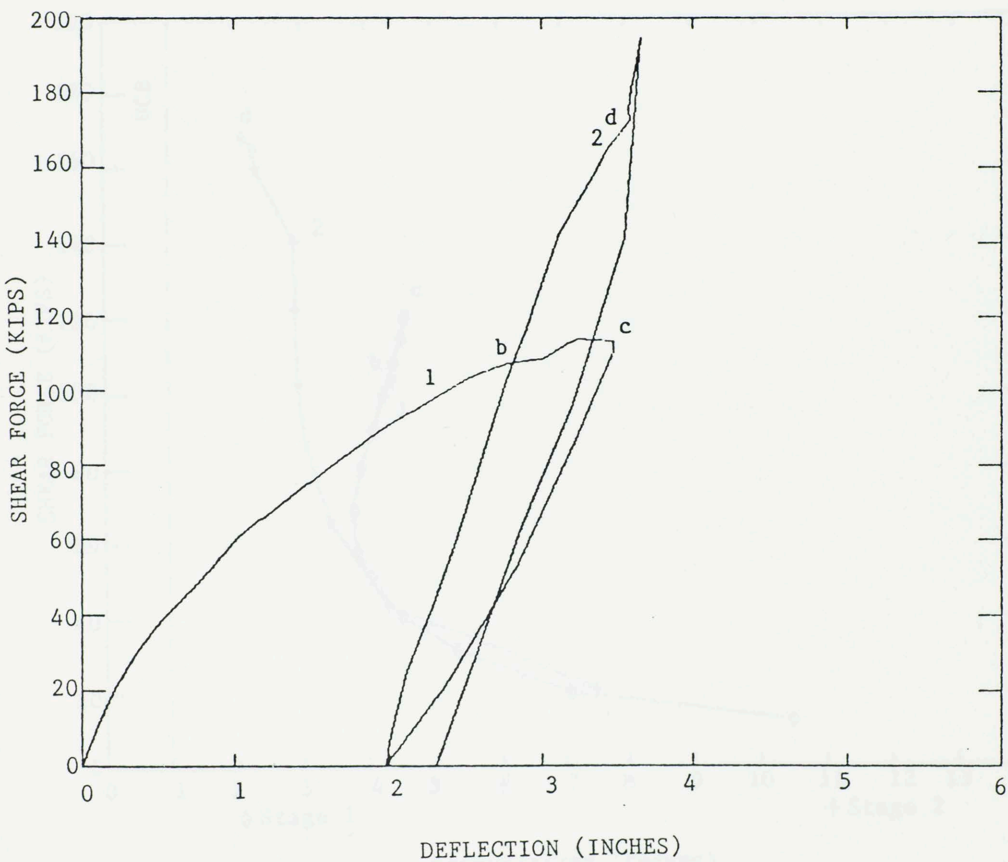

Fig. 3.51 Shear vs. Central Deflection (South End), Test 3b

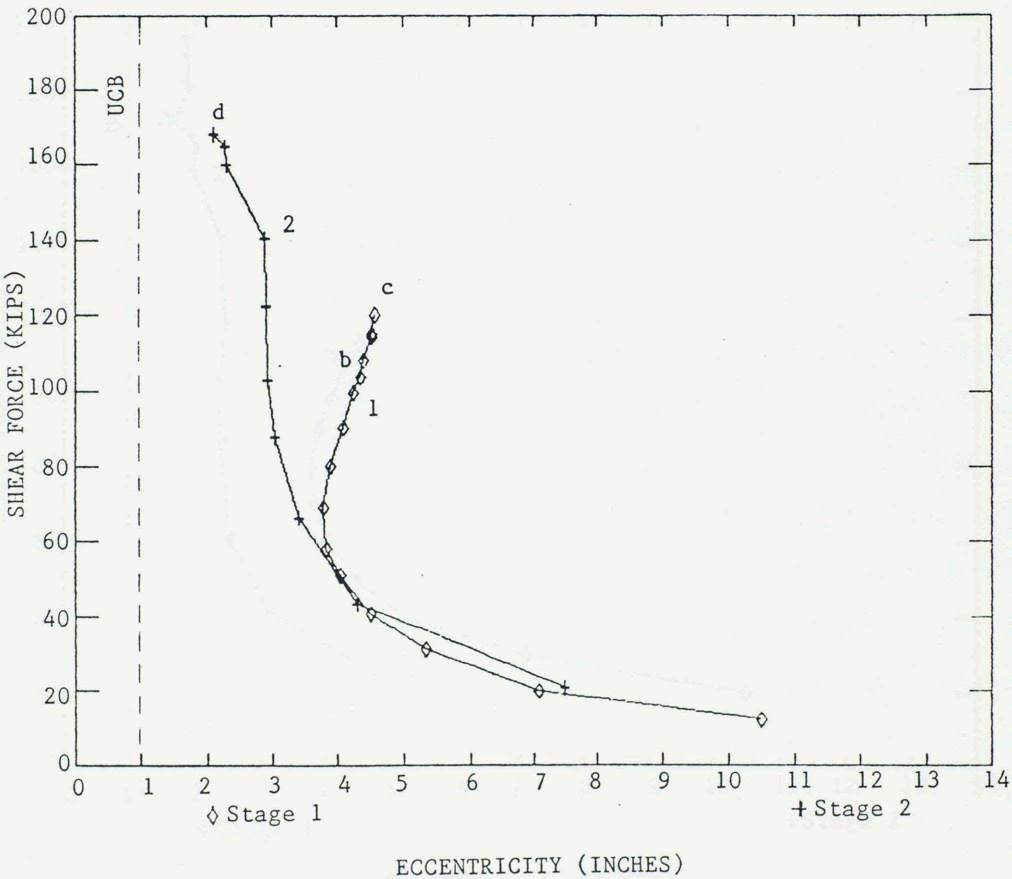

Fig. 3.52 Shear vs. Eccentricity (North End), Test 3b

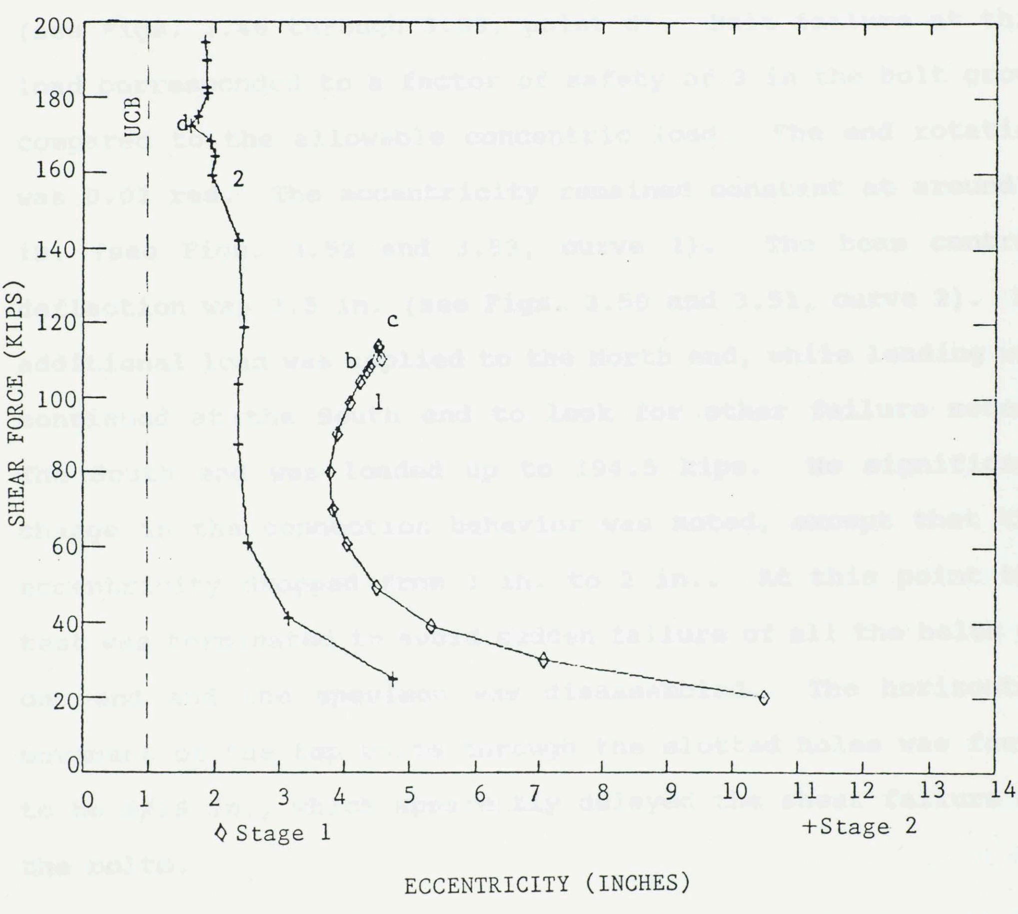

Fig. 3.53 Shear vs. Eccentricity (South End), Test 3b

3.51, zone b-c). Since the connection survived an end rotation of 0.03 rad., the load was released and the test restarted with a load location of 45 in. from the beam ends.

A linear shear-rotation relationship (see Figs. 3.46 and 3.47, curve 2) was observed up to a load of 168 kips at each end, until the topmost bolt at the North end failed in shear (see Figs. 3.46 through 3.53, point d). Bolt failure at this load corresponded to a factor of safety of 3 in the bolt group compared to the allowable concentric load. The end rotation was 0.03 rad. The eccentricity remained constant at around 3 in. (see Figs. 3.52 and 3.53, curve 2). The beam central deflection was 3.5 in. (see Figs. 3.50 and 3.51, curve 2). No additional load was applied to the North end, while loading was continued at the South end to look for other failure modes. The South end was loaded up to 194.5 kips. No significant change in the connection behavior was noted, except that the eccentricity dropped from 3 in. to 2 in.. At this point the test was terminated to avoid sudden failure of all the bolts at one end and the specimen was disassembled. The horizontal movement of the top bolts through the slotted holes was found to be 5/16 in., which apparently delayed the shear failure of the bolts.

CHAPTER 4

ANALYSIS OF CONNECTION BEHAVIOR

4.1 Summary of Test Results

Highlights of all six tests are summarized in Table 4.1. The maximum shear resisted by each connection immediately before and after each bolt failure, the corresponding end rotation and average reaction eccentricity, and the factor of safety in the bolt group considering eccentricity are summarized in Tables 4.2 and 4.3. As shown in Table 4.4, all of the tests resulted in factors of safety above 2.1 for the maximum load attained as compared to the allowable bolt shear capacities with no eccentricity for the original number of bolts. It should be noted that lower factors of safety were observed for tests in which bolt fracture occurred in round hole specimens. It is also obvious that the slotted holes greatly increased (93.8% for 4-bolt connection and 64.4% for 6-bolt connection at first bolt failure) the shear capacity of the connection, while reducing the reaction eccentricity.

One unexpected observation from these test results is the relatively small amount of bolt hole deformation observed

Table 4.1: Summary of Test Results

Test	Hole type	Eccentricity (in.)	Bolt No.	Load (kips)	Rotation (rad)	Observations
1a	Round	1.6	2(x)	34	0.032	No failure, loads moved.
		1.6		41.7	0.033	Shear yielding of North plate.
		1.6		46.8	0.035	Beam end twisted slighty, loads moved.
		0.8		59.3	0.043	Beam ends twisted, braced added.
		0.8		64.3	0.025	Severe shear distortion of connection plates, test stopped.
1b	Round	1.5	2(n)	34	0.033	No failure, loads moved.
				51.8	0.033	Weld tearing at South end, loading stopped at South end.
				60.8	0.028	Weld tearing at North end, test stopped.

Table 4.1: Summary of Test Results (Contd.)

Test	Hole type	Eccentricity (in.)	Bolt No.	Load (kips)	Rotation (rad)	Observations
2a	Round	6	4	66.5	0.028	Top North bolt sheared, stopped loading of North end only.
		3(N) 5(S)	3(N) 4(S)	84.6	0.029(N) 0.033(S)	No failure noted, loads moved.
		3(N) 6(S)	3(N) 4(S)	81.6	0.038(S)	Top South bolt sheared, loads moved.
		2	3	93	0.032	Bolt line deflections increasing, test stopped.
2b	Slot	4	4	64(N) 68(S)	0.030	Slight plate yielding @ bolts, loads moved.
		3.5	4	83	0.038	Some plate yielding @ bolts, loads moved.
		2.5	4	90	0.033	Bolt line deflections starting to increase significantly.
		1.5	4	105	0.036	Shear yielding of plate noticeable.
		1.25	4	129	0.042	All bolts sheared simultaneously.

Table 4.1: Summary of Test Results (Contd.)

Test	Hole type	Eccentricity (in.)	Bolt No.	Load (kips)	Rotation (rad)	Observations
3a	Round	5.5	6	70	0.007	
		3.5(N) 5.5(S)	5(N) 6(S)	102	0.014	Top North bolt sheared.
		3.5(N) 4(S)	5	109	0.019	Top South bolt sheared.
		4.2	5	119	0.024	General plate yielding significant.
		2.3(N) 4.5(S)	4(N) 5(S)	119	0.027	Second North bolt sheared, test stopped.
3b	Slot	4	6	90	0.018	Plate yielding noticed @ top of bolt line.
		4.7	6	120	0.030	Plate yielding at all bolts, loads moved.
		3	5(N) 6(S)	168	0.030	Top North bolt sheared, loading of North end stopped.
		2(S)	5(N) 6(S)	194		Test stopped.

Table 4.2: Connections with Standard Round Holes

Test No.	No. of Bolts	Load (kips)	Rot. (rad)	Ecc. (in.)	Bolt Failure	Coefficient (Table XI)	Allowable Load (Kips)	Factor of Safety
1a	2	34.56	0.0317	1.60		1.34	17.82	2.77
		46.82	0.0346	1.57		1.36	18.09	3.70
		59.31	0.0430	0.81		1.67	22.21	3.82
1b	2	34.52	0.0334	1.37		1.44	13.39	2.58
		51.80	0.0340	0.33		1.86	17.30	2.99
		60.81	0.0285	1.69		1.31	12.18	4.99
2a	4	66.54	0.0284	5.86		1.77	16.46	4.04
		64.18	0.0290	2.99	1[1]-N end	1.77	16.46	3.90
		84.49	0.0368	4.54		2.18	20.27	4.17
		81.56	0.0378	2.23	1[1]-S end	2.12	19.72	4.14
		93.23	0.0323	1.94		2.25	20.93	4.45
3a	6	102.35	0.0154	5.38		3.84	35.71	2.87
		101.70	0.0163	3.35	1[1]-N end	3.74	34.78	2.92
		109.08	0.0186	5.30		3.88	36.08	3.00
		109.27	0.0196	3.91	1[1]-S end	3.46	32.18	3.40
		118.99	0.0262	4.03		3.41	31.71	3.75
		115.94	0.0270	2.38	2[2]-S end	3.13	29.11	3.98

Table 4.3: Connections with Short Slotted Holes

Test No.	No. of Bolts	Load (kips)	Rot. (rad)	Ecc. (in.)	Bolt Failure	Coefficient (Table XI)	Allowable Load (Kips)	Factor of Safety
2b	4	128.99	0.0422	1.22		3.17	29.48	3.47
		129.02	0.0426	1.52		2.97	27.62	3.47
3b	6	168.23	0.0305	2.11		5.40	50.22	3.01
		158.72	0.0299	1.26	1[1]-N end	4.24	39.43	2.84

[1]Topmost bolt
[2]Second bolt from top

Table 4.4: Observed Factors of Safety

Test	Allowable load (kips)	Maximum test load (kips)	Factor of Safety
1a	18.6	62.0	3.33
1b	18.6	51.8 (South) 60.8 (North)	2.78 3.27
2a	37.2	93	2.50[1]
2b	37.2	129	3.47
3a	55.8	119	2.13[2]
3b	55.8	168	3.08

[1]Top bolts sheared at 66.5 kips (Factor of Safety = 2.19), remaining 3 bolts carried 93 kips ultimate load.

[2]Top bolts sheared at 102 kips (Factor of Safety = 1.83) and 109 kips (F.S.=1.95), remaining 5 bolts carried the 119 kips ultimate load.

before the bolts failed in shear. This is inconsistent with the earlier research work done by Richard (4). One probable reason for this inconsistency is that A36 plate material has a higher yield strength than the tests conducted by Richard. This increase in plate yield strength can be seen by examining the average web yield stress determined in NBS Special Publication 577, which documents material properties up to about 1980, establishing yield strength at 1.10 x 36 = 39.6 ksi. Later testing for end-plate research and the AISC LRFD Specification established A36 plate yield strength at 37.2 ksi (12, 13). As shown in Table 4.5, results of tensile coupon tests performed from the same stock as the test plates used in this study indicated that the actual yield strength of the test specimen A36 plate material was 47.4 ksi. The bolt tensile strength was found to be 120 ksi. These numbers indicate that the typical plate yield strength has increased nearly 30% while the bolt tensile strength has increased only 2%. Undoubtedly, this will reduce the maximum plate thickness to bolt diameter ratio to ensure significant bolt hole distortion before bolt shear failure. Moreover, these findings are the likely reason why Richard found bolt hole deformation to be a more viable ductile link to accommodate the beam end rotation than Astaneh.

4.2 Analysis of Failure Modes

The following failure modes have been identified from the tests of single plate framing connections:

1. Yielding of gross area of plate and/or beam web.
2. Yielding of the bolt holes in bearing.
3. Lateral buckling of the shear tab.
4. Shear failure of bolts.
5. Fracture of weld.

The predicted capacities of the connections with round holes are shown in Table 4.6. Besides the above-mentioned failure modes, the predicted capacity of the connections due to shear fracture of the net effective plate area and failure due to bolt bearing have also been included in the Table. The yielding of the plate gross area and shear fracture of the plate net area are based on von Mises failure criteria for measured plate strengths and eccentricity measured at failure. The bolt shear capacity, the weld capacity (considering eccentricity) and the bolt bearing shear capacity (without considering eccentricity) were computed using the allowable loads multiplied by a factor of 2.

4.2.1 Connection Behavior with Standard Round Holes

In the first 2-bolt connection, the observed failure mode was yielding of the gross area of the plate and beam web (see Table 4.1, Test 1a) which occurred after the end rotation reached 0.03 rad. and the loads were moved to generate larger

Table 4.5

Beam Connection Test Plate Tensile Coupon Results

Test No.	t (in.)	w (in.)	P_y (kips)	P_u (kips)	F_y (ksi)	F_u (ksi)
1	0.375	0.496	8.9	12.77	47.85	68.66
2	0.374	0.496	8.6	12.68	46.36	68.35
3	0.376	0.500	9.0	12.92	47.87	68.72
Avg.	3/8				47.36	68.58

Table 4.6 Predicted Capacities of the Connections

Test No.	Bolts	Plate Yield (kips)	Shear Fracture (kips)	Bolt Shear (kips)	Bolt Bearing (kips)	Weld Capacity (kips)
1a	2(X)	30.3	52.4	53.2	87.6	33.0
1b	2(N)	34.7	52.4	37.2	87.6	31.2
2a	4(N)	80.6	104.6	74.4	175.2	94.4
3a	6(N)	125.8	157.0	111.6	262.8	156.6

end shear reactions. This is consistent with the failure mode predicted by comparing the various capacities in the first row of Table 4.6. The second 2-bolt test suffered lateral buckling of the shear tab and subsequent fracture along the weld lines (see Table 4.1, Test 1b). These particular welds were undersized due to root openings and low welding heat input. Once again the failure mode is consistent with the smallest predicted capacity of Table 4.6. None of the welds on the other specimens failed suggesting that the weld designs were adequate, but probably not overly conservative.

The 4-bolt and 6-bolt connections showed similar traits as far as the sequence of failure mode is concerned. In both cases, bolt shear failure was the governing failure mode, which was almost simultaneously followed by yielding of the plate gross area. These observations compare well with the predicted capacities (see Table 4.6, Tests 2a and 3a). In both of these connections, the end rotation was primarily due to slip of bolts into bearing against the hole sides and to a certain extent, yielding of bolt holes in bearing. Before the bolts could slip into bearing, there was a significant release of end moment which was further associated with the movement of the point of inflection toward the support. The bolt group in both the connections plowed through a distance of 1/32 in. before any bolt fractured. Plate yielding was confined to near the top bolts prior to bolt failure.

The reaction eccentricity was more or less constant prior to bolt failure. As shown in each of the shear versus eccentricity graphs of Chapter 3, Richard's equation (2.2) over-predicts eccentricity as indicated by the vertical lines marked UA, whereas Astaneh's equation (2.7) under-predicts the eccentricity, as shown by the vertical lines marked UCB. Moreover, analysis of test data and graphical plots indicates that load location to beam span ratio affects the eccentricity. A sudden drop in eccentricity was associated with bolt failure, which was a direct consequence of release of end moment.

Since no weld failure was associated with these connections, the concept of limiting the required weld size to 0.75 times the plate thickness, as suggested by Astaneh (5) appears valid.

4.2.2 Connection Behavior with Short Slotted Holes

The same list of possible failure modes established for connections with round holes was examined for the connections with short slotted holes. The experimental results indicated that connections with short slotted holes have an obvious advantage over connections with standard round holes. The former can carry greater shear, and yet sustain more end rotation than the latter. The end rotation demand for such connection, was primarily accommodated by the yielding of bolt holes in bearing, and to a certain extent, shear yielding of the plate. Moreover, the eccentricity of the point of inflection for such connections was much smaller than in the corresponding connections with standard round holes. The primary reason for this low eccentricity is that short slots allow the bolts to slide horizontally through the slots thereby causing considerable release of end moment until the bolts finally reached the end of the slot. After attaining this stage, the bolts plowed through the plate through a distance of 3/32 in. to 1/8 in. before ultimately failing in shear.

Due to its relatively higher shear carrying capacity and rotational ductility, single plate connections with short slotted plate holes are more efficient than similar connections with standard round holes.

4.3 Implication of Test Results

The most obvious implications from these six test results is that the previous design rules for A325-N bolt diameter to plate thickness ratio is not sufficient to ensure significant bolt hole deformation before bolt shear failure. This is most probably due to the increase in the typical A36 steel plate yield strength over the last decade. A more general ratio rule which depends on the plate yield strength seems to be warranted.

Another implication of these test results is that if bolt shear due to eccentricity and non-yielding plates is a problem, then deeper connections (with more bolts) suffer bolt failure at smaller beam end rotations than shallower connections. This can be seen by comparing the rotations at first bolt failure of

Tests 2a and 3a in Table 4.1. This is a direct consequence of the relationship among beam end rotation, bolt pattern depth, and the resulting horizontal bolt movement. This reduced rotational capacity of deeper connections means that if bolt shear due to beam end rotation and a lack of other rotational deformation mechanisms is a problem, then adding bolts (which results in a deeper connection) will not help.

The shearing of one bolt in Test 3b occurred when the bolt was forced beyond the end of the slotted hole. This occurrence should serve as a reminder that if one utilizes slotted holes to accomodate beam end rotation, a rational analysis of the expected movement is needed to ensure that adequate slot length is provided.

Finally, the effect of the chosen load path on the observed test results is always of concern. The difficulty in choosing a load path (beam end shear versus rotation relationship) for each test is caused by the somewhat wide range of plausible load paths and the fact that the paths are non-linear. It is quite conceivable that two different but plausible load paths may result in significantly different connection behavior. For instance, one load path would be to apply a large shear force to the beam with little simultaneous rotation, and then hold that shear force constant while applying the remainder of the prescribed maximum rotation. This would correspond to the expected end behavior of beams with small span:depth ratio. This load path would enhance the deformation behavior as described by Astaneh (5), where moment capacity of the connection plate is reduced by the early application of the large shear force. A slower application of the shear force (or a quicker application of the beam end rotation) may actually be a more severe test of the bolts, which would be subjected to a larger moment before the shear load "softened" the plate. The opposite type of loading may be a more severe loading for other failure modes. For instance, the later application of shear may be a less severe test of connections with slotted holes, as more rotation would be completed before the vertical load caused the bolts to dent deep seats in the slots.

CHAPTER 5

SINGLE BOLT LAP TESTS

5.1 Introduction

In order to further investigate the contribution of bolt plowing toward the development of ductility in single plate connections, a series of single bolt lap tests were performed on plates of various thickness and edge distance as listed in Table 5.1. The purpose of varying the plate thickness while using the same bolt configuration was to find out the plate thickness to bolt diameter ratio that could cause considerable bolt plowing before bolt shear failure occurred. For one case the edge distance has been varied to study the possible effects of edge distance toward connection ductility. And finally, to study the effect of bolt strength, one additional configuration was tested with A490-N bolts. All bolt holes were standard 13/16 in. round drilled holes.

Three specimens of each configuration were tested, for a total of 16 tests in the tension region of a universal testing machine. As shown in Fig. 5.1, the instrumentation included a pair of LVDTs for measurement of slip between the two plates.

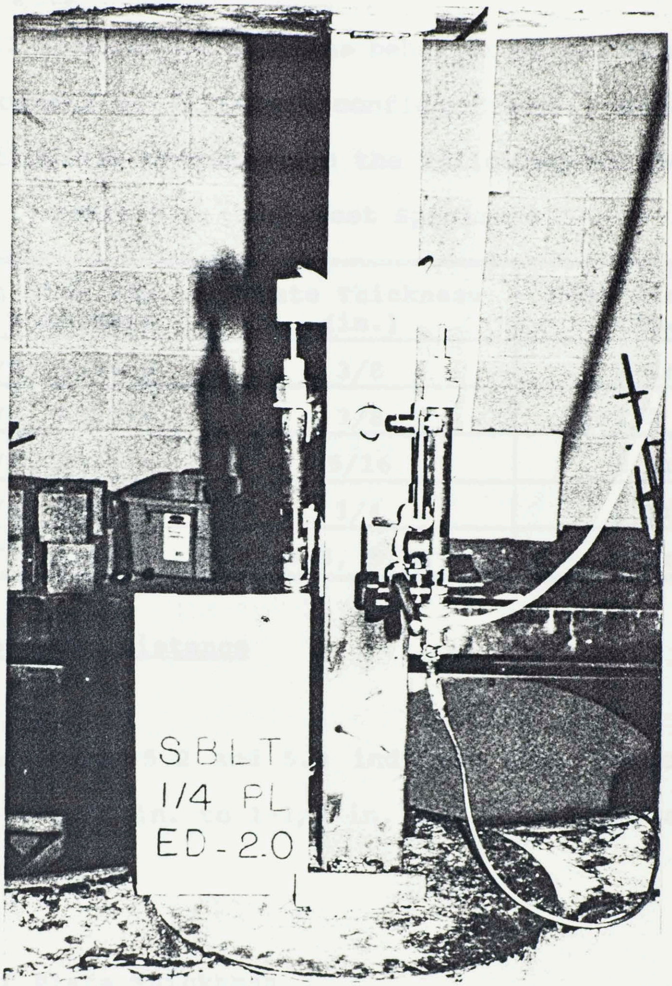

Fig. 5.1 Experimental Setup for Single Bolt Lap Tests

The two LVDTs were mounted on opposite sides of the specimen and positioned equidistant from the bolt centerline and connection faying surface.

Figs. 5.2 through 5.6 show the behavior of the single bolt lap test specimens of different configurations. Conclusions from the testing are presented in the following sections.

Table 5.1: Lap Test Specimens

Group No.	Bolt Size (in.) & Bolt Type	Plate Thickness (in.)	Edge Distance (in.)
1	3/4, A325-N	3/8	2 in.
2	3/4, A325-N	3/8	1-1/2 in.
3	3/4, A325-N	5/16	2 in.
4	3/4, A325-N	1/4	2 in.
5	3/4, A490-N	3/8, 5/16	2 in.

5.2 Effect of Edge Distance

Comparing Figs. 5.2 and 5.3 indicate that reducing the edge distance from 2 in. to 1-1/2 in. did not facilitate bolt plowing.

5.3 Effect of Plate Thickness

Comparing Fig. 5.2 with Figs. 5.4 and 5.5 indicate that reducing the plate thickness to 5/16 in. and subsequently to 1/4 in. improved the ductility. With 3/8 in. plates, the maximum plate slip before bolt failure was only 0.3 in., whereas for 5/16 in. and 1/4 in. thick plates, it was 0.6 in. and 0.65 in. respectively. The bolt failure load was essentially the same for specimens of all thicknesses. However, in test 4c (with 1/4 in. thick plate), an additional failure mechanism, tension tearing of net area, was observed. The plate yielding around the bolt holes was particularly significant with 5/16 in. and 1/4 in. thick plates rather than 3/8 in. thick plates. The obvious conclusion is that, with reduced plate thickness to bolt diameter ratio, the single plate connection ductility can be improved significantly. These conclusions are consistent with the previously mentioned theory that the common higher yield strength of recently produced A36 plates requires a lower plate thickness to bolt diameter ratio than in the past. Tensile coupon test results of these specimens are shown in Table 5.2. Since the 5/16 in. thick plate proved adequate ductility through plate yielding and bolt plowing only, a limiting value of 0.4 (0.3125/0.75) for the plate thickness to bolt diameter ratio appears appropriate for this plate and bolt material.

5.4 Effect of Bolt Strength

The tests categorized under Group 5 aimed at investigating the effect of increased bolt strength toward the ductility mechanism. Comparing Figs. 5.2 and 5.6 indicate that for 3/8 in. thick plates, the A490-N bolts proved to be stronger and more effective in plowing than A325-N bolts of the same size. With A325-N bolts, the maximum plate slip before bolt failure was only 0.3 in. and the failure load was 28 kips. With A490-N bolts, the same plate slip was attained at 35 kips load, and finally, the failure occurred at a load of 38 kips with bolt plowing reaching a plate slip of 0.4 in. By reducing the plate thickness to 5/16 in. for A490-N bolts (Test 5d), the plate slip before bolt yielding was found to be much more pronounced. But the drawback was that tension tearing of the plate was initiated when the plate slip reached 0.7 in. and the load was approximately 30 kips.

Table 5.2: Tensile Coupon Test Results

Test No.	w (in.)	t (in.)	P_y (kips)	P_u (kips)	F_y (ksi)	F_u (ksi)
1	0.504	0.303	7.25	10.10	47.47	66.14
2	0.500	0.303	7.17	9.90	47.33	65.35
3	0.502	0.303	7.17	9.90	47.14	65.09
Avg. (1,2,3)		5/16			47.31	65.52
4	0.504	0.250	6.00	8.25	47.62	65.48
5	0.502	0.250	6.00	8.25	47.81	65.74
6	0.496	0.249	5.83	8.20	47.21	66.40
Avg. (4,5,6)		1/4			47.54	65.87
7	0.505	0.376	9.00	12.75	47.40	67.15
8	0.504	0.377	9.17	12.75	48.26	67.10
9	0.503	0.377	8.92	12.70	47.04	66.97
Avg. (7,8,9)		3/8			47.57	67.07

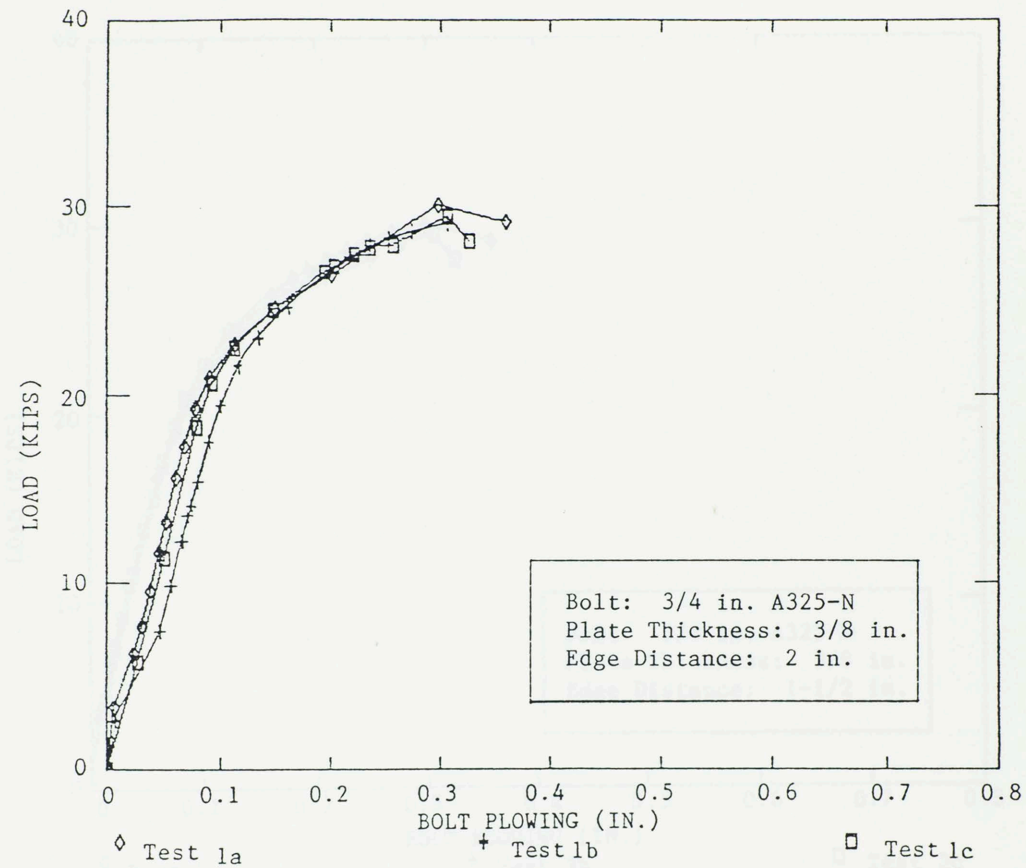

Fig. 5.2 Load vs. Bolt Plowing (Group 1)

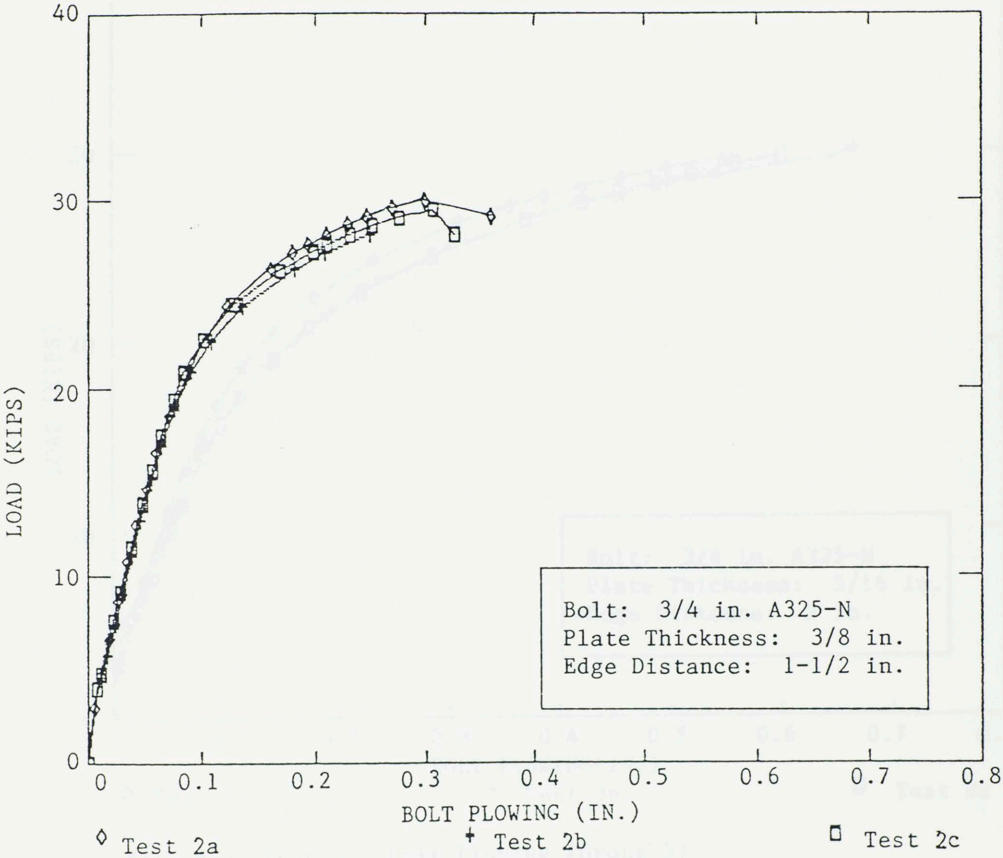

Fig. 5.3 Load vs. Bolt Plowing (Group 2)

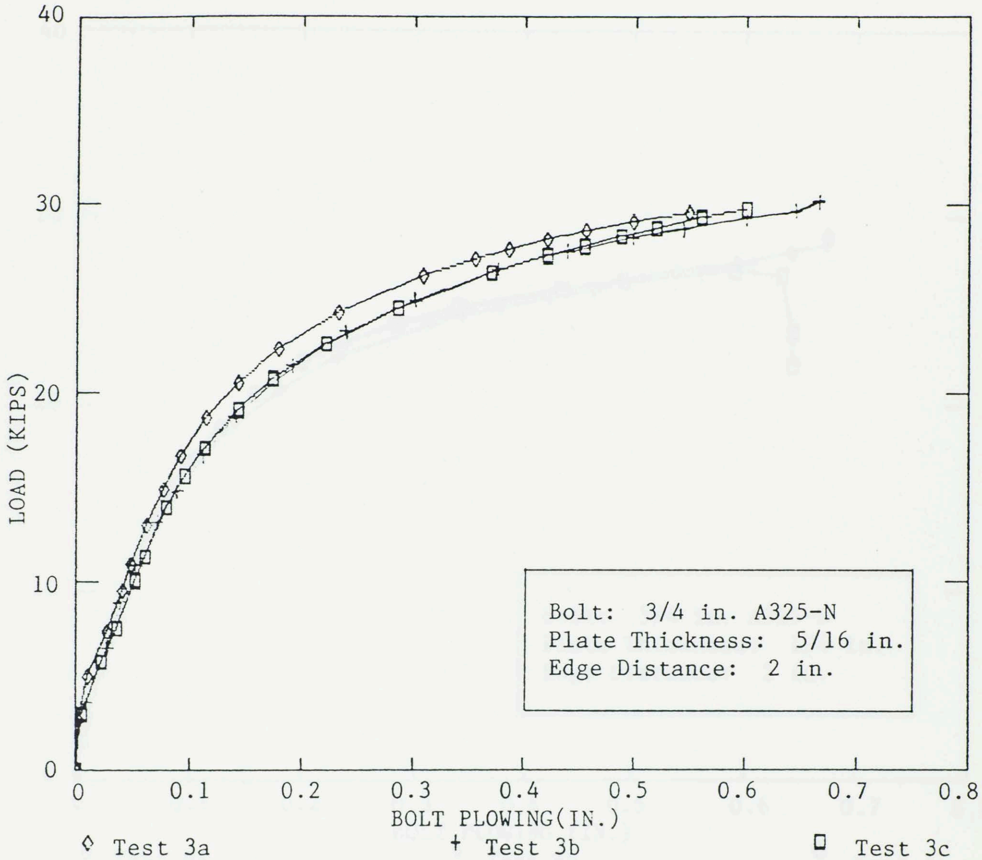

Fig. 5.4 Load vs. Bolt Plowing (Group 3)

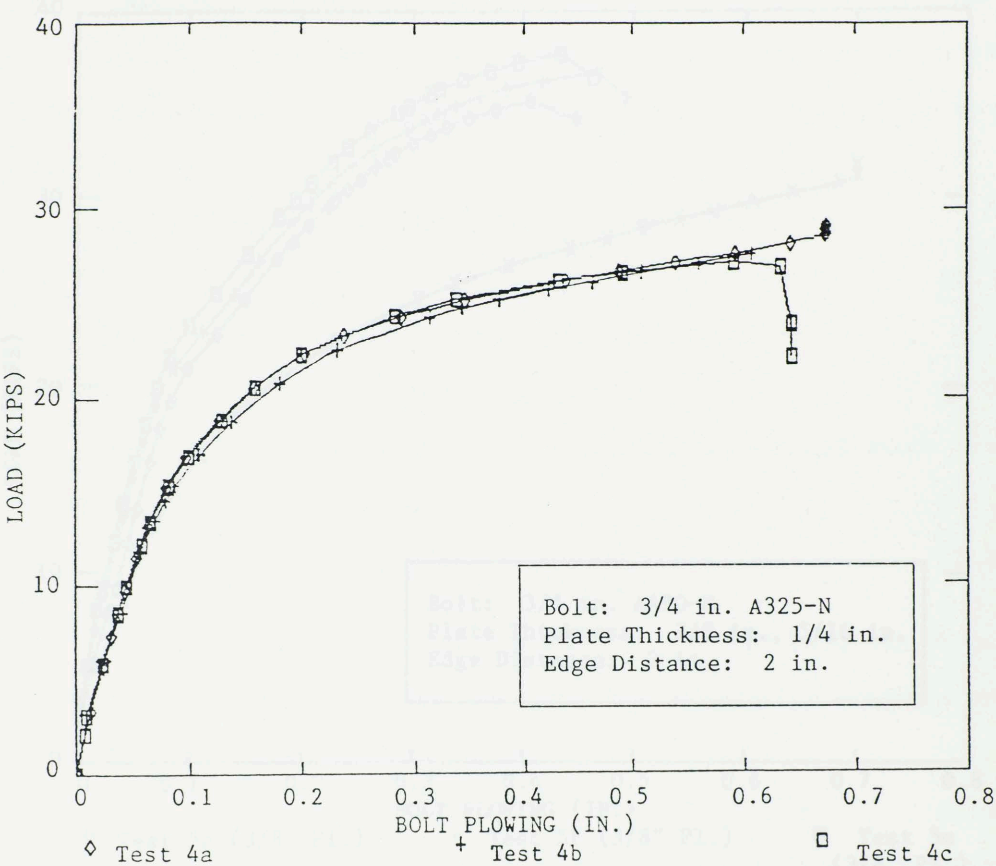

Fig. 5.5 Load vs. Bolt Plowing (Group 4)

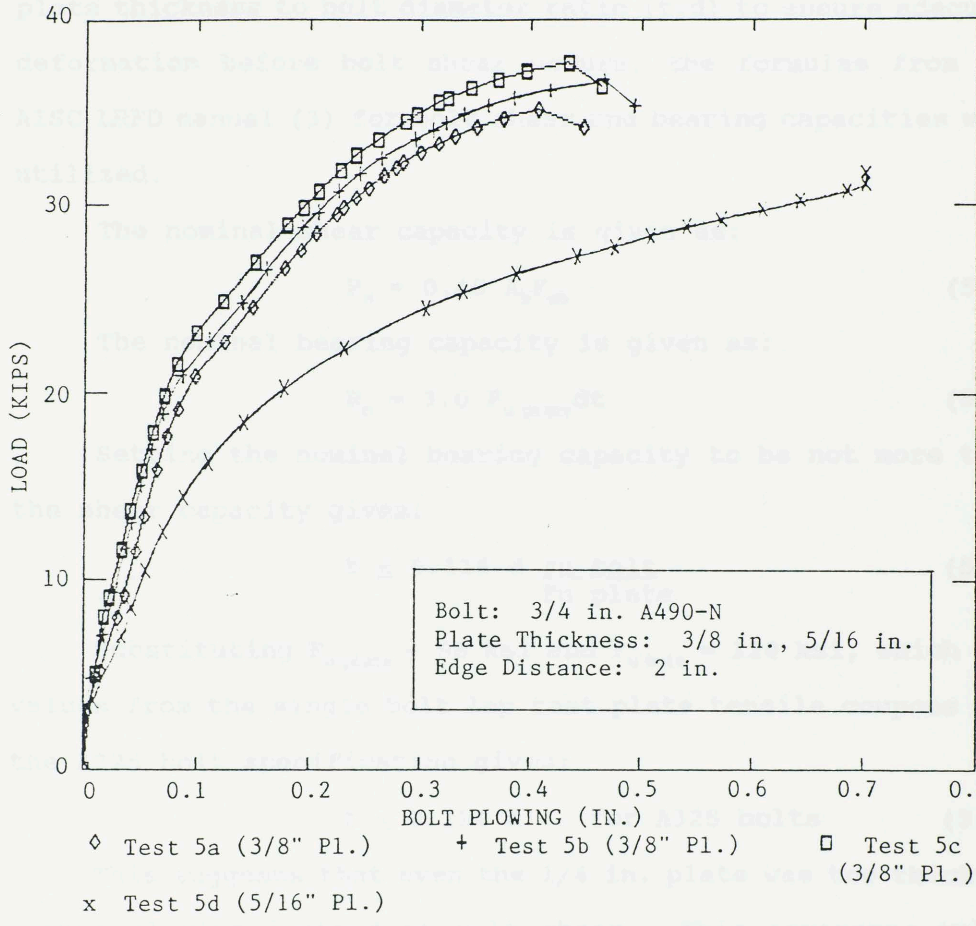

Fig. 5.6 Load vs. Bolt Plowing (Group 5)

5.5 Modified Plate Thickness to Bolt Diameter Ratio Rule

In order to develop a rational expression for the maximum plate thickness to bolt diameter ratio (t:d) to ensure adequate deformation before bolt shear occurs, the formulas from the AISC LRFD manual (3) for bolt shear and bearing capacities were utilized.

The nominal shear capacity is given as:

$$R_n = 0.45\ A_b F_{ub} \tag{5.1}$$

The nominal bearing capacity is given as:

$$R_n = 3.0\ F_{u\ plate} dt \tag{5.2}$$

Setting the nominal bearing capacity to be not more than the shear capacity gives:

$$t \leq 0.118\ d\ \frac{\text{Fu bolt}}{\text{Fu plate}} \tag{5.3}$$

Substituting $F_{u\ plate}$ = 66 ksi and $F_{u\ bolt}$ = 120 ksi, which are values from the single bolt lap test plate tensile coupons and the A325 bolt specification gives:

$$t \leq 0.214\ d \qquad \text{for A325 bolts} \tag{5.4}$$

This suggests that even the 1/4 in. plate was too thick to ensure plowing instead of bolt shear. This erroneous value obtained is because the bolt shear nominal capacity equation is more conservative than the bearing nominal capacity equation. A more appropriate equation may be obtained by utilizing the form of this equation, but changing the constant such that the 5/16 in. plate tested is barely acceptable. This results in the equation:

$$t \le 0.229\, d\, \frac{Fu\ bolt}{Fu\ plate} \quad \text{for A325 bolts} \tag{5.5}$$

This gives:

$$t \le 0.42\, d \quad \text{for A325 bolts} \tag{5.6}$$

$$t \le 0.52\, d \quad \text{for A490 bolts} \tag{5.7}$$

CHAPTER 6

DESIGN PRINCIPLES AND PROCEDURE

6.1 Introduction

The following suggested design principles have been developed from the previous design procedures by Richard (4) and Astaneh (5), and also from the analysis of the experimentally obtained results. The design principles recognize Richard's design requirement for the bolt group and Astaneh's design requirement for the plate as well as the weld. The plate thickness to bolt diameter ratio has been modified to consider the increased yield strength of A36 plate. The advantage of using slotted plate holes as an alternative for thicker plates is also discussed in the following sections.

6.2 Design Criteria for the Plate

1. To ensure sufficient ductility and facilitate plate yielding, the plate material should be A36 steel.
2. Bearing yielding of bolt holes due to bolt plowing or adequate sliding of bolts in slots must precede bolt

fracture. Hence, to facilitate bolt plowing, the plate thickness to bolt diameter ratio should be limited to 0.42 and 0.52 for A325 and A490 bolts, respectively. These ratios have been established to consider the current tensile strength of the connection plate. These ratios are in contrast with the design procedure by Astaneh (5) which considers this ratio as 0.5 or slightly higher.

3. In case it becomes necessary to increase the plate thickness based on its shear and bending capacity, slotted holes may be used to provide the bolt ductility and this ratio may be neglected. If this is done, the minimum slot length must be calculated for the desired rotation capacity and bolt pattern depth.

4. To avoid edge distance failure, the horizontal and vertical edge distances should follow the standard AISC guidelines (2), (3).

5. The plate should be designed to carry direct shear and relatively small moment. The moment is equal to the shear times the "equilibrium eccentricity", which is equal to the distance between the bolt line and the weld line considering the inflection point of the moment diagram right at the bolt line. For all practical purposes and from static equilibrium considerations, this equilibrium eccentricity should govern the design. The same criteria is applicable even with slotted holes, no matter how effective they are in reducing the eccentricity. The

compatibility eccentricity values obtained in the experiments (see Tables 4.2 and 4.3), if used for design, will only increase the moment capacity of the connection, and will not necessarily increase its rotational ductility. It may be recalled that the previous design procedure by Richard (4) used compatibility eccentricity instead of equilibrium eccentricity which often resulted in overdesigning of the plate. The resulting plate shear and bending stress combination may be checked using the Von Mises criterion.

6. The plate should be designed for both shear fracture of net area and shear yielding of the gross area.

6.3 Design Criteria for the Bolts

Designing the bolt group for eccentric loading increases the number of bolts, and hence, the shear capacity of the connection. But experimental results indicate that increasing the connection shear capacity does not automatically increase the rotation capability and may even decrease it. This is evident from the fact that the first bolt fracture may not define the maximum connection capacity for shear and rotation, and that the 6-bolt connection fractured the first bolt at a much smaller rotation than the 4-bolt connection. The current design procedure by Astaneh (5), however, utilizes the bolt group for both shear and moment due to eccentricity.

Bolt ductility may be ensured by sufficiently long slots. Considering both the top and the bottom bolts to move equally in the slots, the required horizontal slip is obtained as a product of the bolt group depth and the expected end rotation (typically 0.03 rad.). Assuming that the bolts are centered in slots at zero beam end rotation, the required slot length may be obtained from the sum of the bolt diameter and the expected horizontal bolt slip. Example: For a 6-bolt connection (bolt diameter 3/4 in.) with 3 in. bolt pitch and 0.03 rad. end rotation, the required horizontal bolt slip is (6-1)(3)(0.03) = 0.45 in. This would establish the required slot length as 0.75 + 0.45 = 1.20 in.

The minimum eccentricity required for equilibrium must somehow be resisted at the bolt line. In case of rigid supports this may be zero. For the case of flexible supports this will be the end shear reaction times the horizontal distance between the bolts and the support. It may be possible to mobilize some effects from floor slabs to assist in resisting the resulting moment.

6.4 Design Criteria for the Weldment

Fillet welds are acceptable when applied on both sides of the plate along its entire length. The weld should be designed for the combined effect of shear and moment in order to develop the plate yield capacity. In other words, the plate should be guaranteed to yield before the weld. The weld size may be limited to 0.75 times the plate thickness as previously established by Astaneh (5). It should be noted that the weld size calculated from the minimum eccentricity required for equilibrium may be too small for the compatibility eccentricity. Therefore these welds should always be sized to "develop" the plate.

6.5 Design Procedure

The recommended design procedure following ASD rules is shown below. This procedure can easily be adopted to the LRFD format by substitution of appropriate LRFD equation in steps 1, 5, 7 and 8.

1. Number of bolts: $n = \dfrac{\text{Design Reaction}}{\text{Allowable Bolt Shear}}$ (7.1)
   (single row)

   Select A325 or A490 Bolts.

2. Select A36 plate thickness from:

   $t \leq 0.42\, d^*$ for A325 bolts (7.2)

   $t \leq 0.52\, d^*$ for A490 bolts (7.3)

   where d = bolt diameter

   The above criterion is strictly applicable for round holes, and not required for slotted holes.

   * Note: These formulas assume plate and bolt ultimate strengths as 66 and 120 ksi respectively. See equations 5.4 and 5.5 for other strength materials. If thicker plates are desired, use slotted holes to protect the bolts

from shear due to beam end rotation. See section 6.3 for details regarding the slot length.

3. Choose: Bolt pitch: 3 in. is preferable.

   Edge distance $\geq$ (Bolt Pitch)/2

   Minimum vertical edge distance = 1.5 in.

4. Determine preliminary plate length (L) from (1) and (3) and plate width (b) from (3).

5. Check allowable shear strength R_{ns} of the plate effective net area.

$$R_{ns} = [L - n(d + 1/16)](t)(0.3F_u) \geq R \tag{7.4}$$

6. Calculate moment, M, at weld line using:

$$M = (V) \times (a) \tag{7.5}$$

   where V = design reaction

   a = distance between weld line and bolt line

7. Check combined shear and bending of the plate:

   Plate bending stress: $f_b = M/Z \leq 0.60f_y$ (7.6)

   Plate shear stress: $f_v = V/A \leq 0.40f_y$ (7.7)

   Z = Plate plastic section modulus = $(b)(d^2)/4$ (7.8)

   A = Area cross-sectional area = (b)(t) (7.9)

   Check the equivalent Von Mises stress:

$$f_{VM} = (f_b^2 + 3f_v^2)^{0.5} \leq 0.60f_y \tag{7.10}$$

8. Weld the plate to the support to fully develop the plate capacity. Fillet welds using E70XX electrodes should be applied on both sides of the plate along its entire length. The weld size should be computed from:

$$\text{Weld Size} = (0.75)(t) \tag{7.11}$$

6.6 Design Examples

6.6.1 Design Example 1: 2-Bolt, Round Holes

Given:

Beam: W12x35, A36 steel, $t_w = 0.3$ in.

Support: Column flange (assumed rigid)

Welds: E70XX fillet welds

Bolts: 3/4 in. A325-N

Design Reaction: 18.6 kips

Solution:

1. Number of bolts required: n = (18.6)/(9.3) = 2
2. Select A36 plate.

   Thickness $t \leq (0.42)(3/4) = 0.315$ in.

   Choose t = 5/16 in.
3. Choose bolt pitch = 3 in. and

   edge distance = 3/2 = 1.5 in.
4. Plate length L = 6 in.
5. Allowable shear strength of effective plate net area

   $$R_{ns} = [6 - 2(3/4 + 1/16)](5/16)(0.3 \times 66)$$

   $$= 27.1 > 18.6 \text{ kips}$$
6. Take a = 3 in. and compute moment.

   Moment M = (V) x (a) = (18.6)(3) = 55.8 kip-in.
7. Compute plate sectional properties and stresses and check if they are less than the allowable limit:

   $$Z = \frac{(5/16)(6)^2}{4} = 2.81 \text{ in}^3$$

   $$A = (6)(5/16) = 1.875 \text{ in}^2$$

Plate Bending Stress: $f_b = \frac{M}{Z} = \frac{55.8}{2.81} = 19.86 \text{ ksi} \leq 0.60f_y$

Plate Shear Stress: $f_v = \frac{V}{A} = \frac{18.6}{1.875} = 9.92 \text{ ksi} \leq 0.40f_y$

Compute equivalent Von Mises stress:

$$f_r = [19.86^2 + 3\ (9.92^2)]^{0.5} = 26.3 \text{ ksi} > 0.60f_y$$

Plate should be lengthened or made thicker with slotted holes. Try 7in.long x 5/16 in. plate.

$Z = 3.83 \text{ in.}^3$,

$A = 2.19 \text{ in.}^2$,

$f_b = 14.57$ ksi,

$f_v = 8.49$ ksi,

$f_{VM} = [(14.57)^2 + 3(8.49)^2]^{0.5} = 20.7 \leq 22$ ksi O.K.

8. Fillet welds using E70XX electrodes should be applied on both sides of the plate along its entire length.

Weld Size = (0.75)(5/16) = 0.234 in.

Use 1/4 in. fillet welds.

<u>Note:</u> In the experiments, for this problem:

plate thickness used = 3/8 in.,

$f_b = 16.53$ ksi,

$f_v = 8.27$ ksi,

$f_{VM} = 21.87$ ksi, and

Weld thickness required = 5/16 in. (same as in tests)

6.6.2 Design Example 2: 4-Bolt, Round Holes

<u>Given:</u>

Beam: W18x76, A36 steel, $t_w = 0.425$ in.

Support: Column flange (assumed rigid)

Welds: E70XX fillet welds

Bolts: 3/4 in. A325-N

Design Reaction: 37.2 kips

Solution:

1. Number of bolts required: n = (37.2)/(9.3) = 4
2. Select A36 plate.

   Thickness t $\leq$ (0.42)(3/4) = 0.315 in.

   Choose t = 5/16 in.
3. Choose bolt pitch = 3 in. and

   edge distance = 3/2 = 1.5 in.
4. Plate length L = 12 in.
5. Allowable shear strength of effective plate net area

   R_{ns} = [12 - 4(3/4 + 1/16)](5/16)(0.3 x 66)

   = 54.1 > 37.2 kips
6. Take a = 3 in. and compute moment.

   Moment M = (V) x (a) = (37.2)(3) = 111.6 kip-in.
7. Compute plate sectional properties and stresses and check if they are less than the allowable limit:

   $Z = \frac{(5/16)(12)^2}{4} = 11.25\ in^3$

   $A = (12)(5/16) = 3.75\ in^2$

   Plate Bending Stress: $f_b = \frac{M}{Z} = \frac{111.6}{11.25} = 9.92\ ksi \leq 0.60f_y$

   Plate Shear Stress: $f_v = \frac{V}{A} = \frac{37.2}{3.75} = 9.92\ ksi \leq 0.40f_y$

   Compute equivalent Von Mises stress:

   $$f_{VM} = [(9.92)^2 + 3(9.92)^2]^{0.5} = 17.47\ ksi \leq 0.60f_y$$

8. Fillet welds using E70XX electrodes should be applied on both sides of the plate along its entire length.

   Weld Size = (0.75)(5/16) = 0.234 in.

   Use 1/4 in. fillet welds.

Note: In the experiments, for this problem:

plate thickness used = 3/8 in.,

f_b = 8.27 ksi,

f_v = 8.27 ksi,

f_{VM} = 16.54 ksi, and

Weld thickness required = 5/16 in. (same as in tests)

6.6.3 Design Example 3: 4-Bolt, Slotted Holes

Given:

Beam: W18x76, A36 steel, t_w = 0.425 in.

Support: Column flange (assumed rigid)

Welds: E70XX fillet welds

Bolts: 3/4 in. A325-N

Design Reaction: 37.2 kips

Solution:

1. Number of bolts required: n = (37.2)/(9.3) = 4
2. Select A36 plate.

   Thickness t $\leq$ (0.42)(3/4) = 0.315 in. for round holes.

   Choose t = 3/8 in. plate with slotted holes.
3. Choose bolt pitch = 3 in. and

   edge distance = 3/2 = 1.5 in.

   Considering 0.03 rad. end rotation,

slot length required = (4-1)(3)(0.03) + 3/4 = 1.02 in.

Provide 1 in. slot or longer.

4. Plate length L = 12 in.

5. Allowable shear strength of effective plate net area

R_{ns} = [12 - 4(3/4 + 1/16)](5/16)(0.3 x 66)

= 54.1 > 37.2 kips

6. Take a = 3 in. and compute moment.

Moment M = (V) x (a) = (37.2)(3) = 111.6 kip-in.

7. Compute plate sectional properties and stresses and check if they are less than the allowable limit:

$$Z = \frac{(3/8)(12)^2}{4} = 13.5 \text{ in}^3$$

$$A = (12)(3/8) = 4.5 \text{ in}^2$$

Plate Bending Stress: $f_b = \frac{M}{Z} = \frac{111.6}{13.5} = 8.27 \text{ ksi} \leq 0.60f_y$

Plate Shear Stress: $f_v = \frac{V}{A} = \frac{37.2}{4.5} = 8.27 \text{ ksi} \leq 0.40f_y$

Compute equivalent Von Mises stress:

$$f_{VM} = [(8.27)^2 + 3(8.27)^2]^{0.5} = 16.54 \text{ ksi} \leq 0.6f_y$$

8. Fillet welds using E70XX electrodes should be applied on both sides of the plate along its entire length.

Weld Size = (0.75)(5/16) = 0.234 in.

Use 1/4 in. fillet welds.

Note: In the experiments, for this problem:

plate thickness used = 3/8 in.,

f_b = 8.27 ksi,

$_{fv}$ = 8.27 ksi,

f_{VM} = 16.54 ksi, and

Weld thickness required = 5/16 in. (same as in tests)

6.6.4 Design Example 4: 6-Bolt, Round Holes

Given:

Beam: W21x93, A36 steel, $t_w = 0.58$ in.

Support: Column flange (assumed rigid)

Welds: E70XX fillet welds

Bolts: 3/4 in. A325-N

Design Reaction: 55.8 kips

Solution:

1. Number of bolts required: n = (55.8)/(9.3) = 6
2. Select A36 plate.

   Thickness $t \leq (0.42)(3/4) = 0.315$ in.

   Choose t = 5/16 in.
3. Choose bolt pitch = 3 in. and

   edge distance = 3/2 = 1.5 in.
4. Plate length L = 18 in.
5. Allowable shear strength of effective plate net area

   $R_{ns} = [18 - 6(3/4 + 1/16)](5/16)(0.3 \times 66)$

   $= 81.2 > 55.8$ kips
6. Take a = 3 in. and compute moment.

   Moment M = (V) x (a) = (55.8)(3) = 167.4 kip-in.
7. Compute plate sectional properties and stresses and check if they are less than the allowable limit:

   $Z = \frac{(5/16)(18)^2}{4} = 25.31 \text{ in}^3$

   $A = (18)(5/16) = 5.62 \text{ in}^2$

Plate Bending Stress: $f_b = \frac{M}{Z} = \frac{167.4}{25.31} = 6.61 \text{ ksi} \leq 0.60 f_y$

Plate Shear Stress: $f_v = \frac{V}{A} = \frac{55.8}{5.62} = 9.93 \text{ ksi} \leq 0.40 f_y$

Compute equivalent Von Mises stress:

$$f_{VM} = [(6.61)^2 + 3(9.93)^2]^{0.5} = 18.43 \text{ ksi} \leq 0.6 f_y$$

Note: In the experiments, for this problem:

plate thickness used = 3/8 in.,

$f_b = 5.51$ ksi,

$f_v = 8.27$ ksi,

$f_{VM} = 15.35$ ksi, and

Weld thickness required = 5/16 in. (same as in tests)

CHAPTER 7

SUMMARY AND CONCLUSIONS

7.1 <u>Summary</u>

The main objective of this investigation was to study experimentally the behavior of single plate framing connections to rigid supports and develop a design procedure for such connections. Previous design procedures regarding the behavior and design of this connection have been suggested by different researchers, notably Richard (4) and Astaneh (5), and their design philosophies for this connection were remarkably different. In order to arrive at a common and rational procedure to characterize the behavior and design of this connection, full scale beam tests on 2-, 4- and 6-bolt connections have been conducted in this study. The beam tests were further supplemented by a series of single bolt lap tests followed by tensile coupon tests to investigate the effect of edge distance, plate thickness and bolt strength on the connection ductility. The test results are used to characterize the actual behavior of single plate framing connections. This is followed by the development of a design procedure for such a connection. Design examples are given illustrating the suggested design procedure.

7.2 Conclusions

1. The following failure modes were identified from full scale tests of single plate framing connections:
   a) Yielding of gross area of plate and/or beam web,
   b) Yielding of bolt holes in bearing,
   c) Shear failure of bolts,
   d) Lateral buckling of the shear tab, and
   e) Fracture of weld.
2. Based on experimental observations and analysis of test data and failure modes, a new design procedure is developed and recommended.
3. The design procedure recognizes the increased value of plate yield strength to establish a modified plate thickness to bolt diameter ratio to facilitate bolt plowing prior to bolt fracture. This is further supported by the results obtained from single bolt lap tests.
4. Both shear fracture of the net area and shear yielding of the gross area of the plate should be checked.
5. Rotational ductility of the connection is not necessarily increased by increasing the shear and/or moment capacity of the connection.
6. For all practical purposes and from static equilibrium

considerations, equilibrium eccentricity, which locates the inflection point at the bolt line for connections to rigid supports, should govern the design.

7. The experiments established that the bolt group itself can withstand sufficient end rotation to release the beam end moment if proper t:d ratios or slotted holes are provided. Therefore, the bolt group can be designed for direct shear only.

8. Limiting the weld size to 0.75 times the plate thickness to develop the plate yield capacity is valid.

9. Connections with short slotted plate holes proved to be more efficient than those with round plate holes. Slotted holes increase connection shear capacity and rotational ductility while reducing the eccentricity. With slotted holes, bolts were found to move considerably until they reached the end of the slot and started bearing. Moreover, plates with slotted holes have been found to exhibit relatively more bolt plowing perpendicular to the slot before bolt fracture, thus avoiding early bolt shear due to misalignment of holes. Hence, slotted plate holes are highly recommended for the design of single plate framing connections.

10. The new design procedure for single plate framing connections is simple and economical in terms of plate dimensions, number of bolts and weld size.

7.3 Recommendations for Future Research

The following recommendations are made for future research:

1. The validity of the experimentally determined plate thickness to bolt diameter ratio needs further investigation.
2. The suggested design procedure for single plate framing connections will result in the rotational ductility required for connections to rigid supports. This design may, however, not result in sufficient moment capacity to satisfy equilibrium for flexible supports. Other factors such as the presence of floor slabs may assist in the development of this moment capacity. Hence, a more appropriate design procedure for single plate connections to flexible supports is needed.
3. The connection behavior should be studied for cyclic loading.
4. The contribution of X-bolts toward connection ductility needs further investigation.

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