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by

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Report to sponsor:

American Institute of Steel Construction

JULY 1988

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BERKELEY, CALIFORNIA

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ABSTRACT

The main objective of the study was to develop a procedure for design of steel single plate framing connections. The new design procedure that is developed and presented in the paper is based on actual inelastic behavior of connection under realistic load and deformation conditions. The major difference between the proposed methods and currently available methods is in consideration of shear effects which is done realistically in the proposed methods.

The study consisted of developing design procedures and verifying them by a limited number of experiments. Three full-scale tests were performed to determine the behavior of single-plate framing connections when subjected to loading requirements accurately representing the actual behavior of a steel structure. A combination of moment, shear and rotation was applied to each connection. This combination reproduces the behavior of a pin connected steel beam as it forms a plastic hinge at midspan.

A review of past research is presented along with an analysis of testing procedures used in the past. A computer program estimating the behavior of steel beams is also reviewed. This computer analysis allows for the creation of a testing procedure which determines the true strength of a single-plate connection.

Graphs showing the relationships between various parameters were plotted and analyzed. Failure modes for these connections were identified and methods to predict the strength capacity of a connection were developed. A design procedure that will allow for the determination of plate, weld and bolt geometry was developed. Several examples of this design procedure are presented to show its application.

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The experiments reported here were conducted at the Civil Engineering Laboratories of the University of California, Berkeley. The authors would like to thank the laboratory staff, particularly Roy Stephens, for their dedicated and professional assistance throughout the project.

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NOTATION

A_b	Nominal cross section area of one bolt, in^2.
A_g	Gross area, in^2
A_{ns}	Net area, in_2
A_{nse}	Effective net area of plate in shear, in^2.
A_{vg}	Gross area of plate in shear, in^2.
B	Width of plate
C	Coefficient in the AISC Manual Tables X and XIX
C_1	Coefficient in the AISC Manual, Table XIX
D	Size of fillet welds, in.
D_{16}	Number of sixteenth of an inch in fillet weld size.
DL	Dead load
E	Modulus of elasticity of steel=29,000 ksi.
F_u	Specified minimum tensile strength of steel, ksi
F_{vb}	Allowable shear stress for bolts, ksi.
F_{vy}	Allowable shear stress for plate in yielding= $0.40F_y$, ksi.
F_{vu}	Allowable ultimate shear strength= $0.30F_u$, ksi.
F_y	Specified minimum yield stress of steel, ksi
L	Length of span, in.
L_p	Length of plate, in.
LL	Live load
M_u	Ultimate moment defined as $M_p(F_u/F_y)$, k-in.
M_y	Yield moment of beam cross section, k-in.
N	Number of bolts.

R	Reaction of the beam due to service load, kips.
R_{alw}	Allowable shear capacity, kips.
R_{blt}	Allowable shear capacity due to bole failure, kips.
R_{brg}	Allowable shear capacity due to bearing, kips.
R_{nse}	Allowable shear strength of effective net area, kips.
R_o	Allowable shear yield strength of plate, kips.
R_{wld}	Allowable shear capacity due to weld failure, kips.
R_y	Reaction corresponding to plastic collapse of beam, kips.
R_{yy}	Reaction corresponding to midspan yield moment, kips
S_x	Section Modulus, in^3
V_y	Shear yield capacity of the plate, kips
W	Width of plate, in.
Z_x	Plastic section modulus, in^3
a	Coefficient in the AISC Manual, Table XIX
a	Distance between bolt line and weld line, in.
d	Depth of beam, in.
d_b	Diameter of bolts, in.
e	Eccentricity of point of inflection from the support
e_b	Eccentricity of beam reaction from bolt line, in.
e_{coll}	Eccentricity of point of inflection at collapse load
e_{eff}	Von Mises' effective strain
e_{fixed}	Eccentricity of point of inflection for fixed beam.
e_w	Eccentricity of beam reaction from weld line, in.
e_1	First principal strain

e_2 Second principle strain

f Shape factor, Z_x/S_x

f_b Computed shear stress in bolt, ksi.

f_{vy} Computed shear stress in plate gross area, ksi.

f_{vu} Computed shear stress in plate effective net area, ksi.

k Coefficient in the AISC Manual, Table XIX

l_h Horizontal edge distance of bolts, in.

l_v Vertical edge distance of bolts, in.

r_v Allowable shear strength of one bolt, kips.

t Thickness of beam web or plate, in.

t_p Thickness of plate, in.

t_w Thickness of beam web, in.

CHAPTER ONE

INTRODUCTION

1.1 Background

Single plate framing connections have gained considerable popularity in recent years due to their ease of fabrication and erection. Figure 1.1 shows six common shear connections used in steel structures. Type (b) in the figure is a single plate shear connection which was the subject of this investigation. Single plate framing connections consist of a plate welded to the support and bolted to the beam as shown in Figure 1.2. This study is concerned with the strength and rotational ductility of these connections. Based on experimental and analytical studies, a new design procedure is developed and presented. The main concept used in developing the design procedure is to utilize inelastic behavior of the connection and its components to develop sufficient flexibility and ductility. The information presented in this report applies only to connections with seven or less bolts. Figure 1.3 shows typical applications of single plate connections where the supporting member is a column, a beam or a reinforced concrete wall.

Single plate framing connections are normally categorized as shear connections and are designed to transfer the end shear reaction of a beam to a supporting member. Like any shear connection, these connections should be designed to satisfy dual criteria of strength and ductility. The connection must have sufficient strength to transfer the end shear reaction and should have enough rotational ductility to accommodate the end rotation demand of the beam. The experimental studies of single plate framing connections reported herein as well as tests of tee framing connections (3), where the tee stem acts as a single plate, resulted in identifying limit states and stiffness characteristics of these connections. The tests indicated that due to shear yielding of the gross area of plate, stiffness of the plate is decreased. In fact, when the connection approaches ultimate capacity, extensive shear yielding takes place causing significant loss of elastic stiffness of the connection. Consequently, the end moments of the beam are released to midspan and the beam approaches the conditions that are assumed for a simply supported beam.

The current method of designing single plate framing connections is based on information obtained from studying moment rotation behavior of connections (18,19). However, since shear forces are the most significant forces in shear connections, any attempt to estimate realistic strength of shear connections should include shear forces and shear effects. In addition, since relatively large rotations develop in shear connections, the effect of rotations should be included in establishing the shear strength of a connection. In this study single plate connections were subjected to shear forces and rotations which realistically would occur in an actual structures when a beam is loaded uniformly until a plastic mechanism forms.

The investigation reported here was conducted to obtain more information on actual behavior of single plate framing connections and to verify, experimentally, a new design procedure proposed in this report. The experimental phase of the study consisted of conducting three full scale tests of single plate beam column assemblages.

The design procedures developed and presented are based on the actual behavior and limit states of the single plate framing connections that were tested. The main emphasis of this research was on studying the shear strength and rotational ductility of the connections.

In order to test connections under more realistic conditions, a special test set-up (1,2,3,14) was used that permitted application of any combination of shear and rotation.

To establish a realistic shear-rotation relationship to be applied to the connections, the results of a previous analytical study (1,2) were used. In that study, a computer program analyzed a large number of beams. The beams were loaded up to formation of plastic collapse mechanism, and their end shear and corresponding rotation were monitored and recorded. Then the established shear-rotation relationship was simulated in the laboratory and single plate framing connections were subjected to established shear-rotation combinations until failure occurred. Test results were recorded, processed and analyzed and design recommendations were formulated.

1.2 Literature Review

Past research for single-plate framing connections provided a guideline for developing the testing procedures used in this experimental program. Richard White (21) performed tests on 2, 3 and 4 bolt single plate framing connections. This testing used a cantilever beam test set-up to subject connections to rotations of about 0.06 radians with very small shear present. In addition, a direct shear test set-up was used later to subject each specimen to large shear and very small rotation. More information on test procedures and set-ups is provided in Section 2.4 of this report. White noted that nearly all of the rotational flexibility of the connection came from bearing deformation around the bolt holes in either the plate or the web. His experiments showed plate tearing as the dominant failure mode.

David Nethercot (15) summarized other research in his discussion of steel connections. He lists work done by McCormick (13) in suggesting that a bi-linear model be used for moment-rotation curves. Mansell and Pham (12,17) observed the initial slip that can occur between the beam web and the plate.

Other shear connections have been tested in the past and these projects show similar behavior to single-plate connections. Samuel Lipson (11) tested several single-angle welded-bolted specimens where he showed the advantages of a combined shear-rotation load application. This loading sequence allowed for a more accurate representation of the actual conditions that a connection would be subjected to during the life of the structure. This testing procedure was expanded and applied to tee framing connections by Astaneh and Nader (3) and later to double-angle connections by McMullin and Astaneh (14). Their research showed that as load increases, the point of inflection in the beam, connected with a tee or double-angle shear connections, moves rapidly toward the support thus decreasing eccentricity of the reaction from the support.

Ralph Richard et al. (19) have conducted studies on rotational stiffness of single plate connections and have developed an analytical model to predict the moment-rotation curve. The model is based on a vertical rigid bar supported by horizontal springs. The effect of shear is not included in the model. This model develops a non-linear relationship similar to the ideas proposed by McCormick (13). The Richard's model was shown to accurately predict the results of both the cantilever and the single span beam tests when significant shear is not present and connection is loaded up to the point of initial yielding of the beam's midspan section, (reaching M_y).

Richard (19) identifies four sources for the rotational flexibility in the single-plate connection. First is shear deformation of the bolt. Second is distortion of the bolt hole in either the plate or beam web. Third is out of plane bending of the supporting member and fourth is bolt slippage.

Patrick et al. (16) conducted experimental studies of single plate connections under a combination of shear and rotation. A total of four specimens were tested. In these tests, the connections were allowed to rotate to a maximum rotation of 0.01 radians during the test. This rotation simulates the response of a simply supported beam, with low L/d ratio, loaded until the midspan moment initiates yielding (M_y). The connections that were tested were part of the standardized connections used by the Australian steel industry (8,9). Two types of single plate connections were studied in the program. The first had a single vertical row of bolts (similar to the connections tested in this project), and these connections always failed by shearing of the bolts. The second series of tests were for connections with two vertical rows of bolts. This series always failed by shearing the plate at the edge of the bolt holes. This fracture was the final mode of failure but never occurred until after the gross area of the plate had shown significant yielding.

The literature review indicated that different methods have been developed for the design of single-plate framing connections. Ralph Richard's design method (18,24) is currently being used to design most single plate framing connections in the United States. Australian steel designers have tables that provide allowable shear capacity of their standardized connections (8,9).

1.3 Scope of the Research

The main objective of this study was to develop design procedures for single plate connections. The specific objectives were:

1. Develop design methods to determine the plate geometry required to resist a given shear force.
2. Provide methods for design of bolts and welds.
3. Obtain experimental data on the realistic behavior and shear strength of the single plate connections.

4. Provide methods that can be used to calculate the capacity of a given single plate connection.

CHAPTER TWO

EXPERIMENTAL PROGRAM

2.1 General

In the experimental phase, three full scale beam-to-column assemblages were tested. The experiments consisted of subjecting the test specimens to a combination of shear forces and rotations that would occur at the supports of a realistic simply supported beam. The following sections explain parameters of study, test specimens, loading history and test procedures. Test results are given in Chapter Three.

2.2 Parameters of Study

The main parameters affecting the behavior of single plate framing connections are:

a. Plate geometry and material

b. Number, size, type, and installation requirements of bolts.

c. Bolt hole geometry and method of fabrication (drilled or punched).

d. Weld size, type of electrode used and method of welding.

e. Type of loading and its application.

f. Behavior of connected beam as well as supporting element.

Currently two approaches are followed in developing design procedures. One approach, which is purely empirical, is to conduct a number of experiments and use numerical techniques to fit appropriate empirical design equations to test results. This approach almost entirely relies on test results thus a large number of tests are required if reliable design rules are to be developed. The second approach, which is followed in this study, is to develop design rules based on physical theory models which relies on basic theories and fundamental concepts of mechanics of material. In this approach experiments are conducted to verify the basic assumptions and to establish failure modes and the experimental data is not used extensively to develop empirical equations.

In experimental phase of this investigation, to vary all parameters over their possible range and consider all possible combinations would require a very large number of tests to be conducted which was not practical and necessary. Instead, an experimental program was designed which provided information on the behavior of the most common types of connections with parameters chosen to represent the most severe conditions.

Plate material was selected to be A36. Since the design concept developed in this report makes use of inelasticity in the connection, a low yield steel was used for the plate to facilitate early yielding and release of rotational stiffness.

According to the steel fabricators who were consulted, it appears that single plate connections with 2 to 7 bolts constitute more than 90% of all single plate connections used; therefore, connections with 3, 5 and 7 bolts were selected for testing. The bolts were 3/4" diameter A325 with threads included in the shear plane. A large volume of past research on bolts has been done using 3/4" diameter A325 bolts. Therefore, using this type of bolt enabled a comparison of the bolt limit states in these experiments with the past results. The bolt length was 1.75 inches. The bolts were tightened using the turn-of-the-nut method according to the requirements as specified in the AISC Manual (22). The bolt holes were punched to represent the most severe case of fabrication.

Welds were fillet welds with 1/4" leg size. Welds were done by Gas Metal Arc Welding (GMAW) with wires equivalent to E70XX electrodes.

The loads were applied such that the actual shear, rotation and moment combination in the connection was properly simulated. More data on loading is given in Sections 2.4 and 2.5.

2.3 Test Specimens

Test specimens are shown in Figures 2.1, 2.2 and 2.3. A typical test specimen consisted of a wide flange beam connected to a column with a single plate framing connection. Properties of the specimens are given in Table 2.1. The single plates used as test specimens were fabricated by a major steel fabricator and sent to the University of California. The plates then were fillet welded to the columns in the laboratory.

In all specimens 3/4 inch diameter A325-N bolts were used. The bolt holes in all specimens were standard round hole with a nominal diameter of 13/16 inch. The holes were punched in the fabricator's shop. The bolt spacing for all specimens was 3 inches center-to-center of the bolts. The edge distance of the bolts for all specimens was 1.50 inches from the top and bottom.

The spacing and edge distance satisfy the requirements of current AISC Specifications (22,23).

Welds connecting single plates to the column in all specimens were done using the GMAW procedure resulting in a nominal strength of 70 ksi for welds. The weld size was 1/4 inch for all specimens.

The column used was a W10x77. This column section was selected to ensure that the column remained elastic during experiments and did not influence the behavior of the connection as a major parameter of the study.

The beams used were W24x84 for the 7 bolt and W18x55 for the 5 bolt and 3 bolt connections. To prevent web buckling of the beam under the load, stiffeners were added along the line of application of concentrated loads. Lateral braces were provided at the end of the beam to prevent instability in out-of-plane direction.

2.4 Test Set-up

In a shear connection, such as a single plate, the main load is shear accompanied by a relatively small moment. In addition, a rotation is developed in the connection area that usually causes significant inelasticity in some connection elements. Therefore, even an experimental study of the behavior of flexible connections through testing is a complex task.

To perform a realistic test of a connection, the actual shear, moment and rotation values of a loaded beam must be simulated as closely as possible during the experiments. Researchers have used different test set-up to perform experiments and to study behavior of shear connections. A brief summary of these test set-ups follows. In most cases discussed here, test specimens consisted of a beam connected to a stub column using single plate shear connection.

One common test set-up used in studying shear connections in the past (10,19,21) is shown in Figure 2.4 and is usually referred to as cantilever beam test set-up. In this method, a load is applied at the tip of the beam. This test set-up can provide information on moment-rotation characteristics of the connection, but it fails to accurately measure the realistic shear strength. In this case, due to high flexibility of connection, upon application of very small shear, unrealistically large rotations take place and the connection fails in bending. The results of the cantilever test specimen can only be used as a measure of bending stiffness, bending strength and rotational ductility of the connection. Since large shear forces cannot be applied to the connection being tested, its shear strength cannot be measured.

Other researchers (20,21) have used the direct shear test set-up shown in Figure 2.5. The load is usually applied at a point close to the connection to generate large shear forces in the connection. As a result, small bending moments are generated in the beam. Therefore, the rotations experienced by the connection during the tests will be unrealistically smaller than the rotation in an actual beam in a buildings. Consequently, since the realistic rotations are not imposed on the connection under investigation, the measured shear strength at the best is an upper bound of strength and not the actual strength.

Another test set-up used by researchers (11,16) to study strength of shear connections is shown in Figure 2.6. In this set-up shear force is applied to the beam along with some rotation. However, the shear-rotation relationships that were used are much smaller than the actual shear-rotation relationship that a shear connection would experience. In both research projects, (11,16), the maximum connection rotation was less than or equal to rotations that a typical beam will experience during elastic range of behavior. As a result, the connection is tested under conditions that do not include the beam's post yielding rotations. It is established (2) that end rotations of the beams and the resulting rotation demand on the connection accelerates after beam passes yield point.

To perform more realistic tests and to simulate combined effects of shear, moment and rotation in a shear connection, one can fabricate an actual beam specimen with end connections, load until failure occurs and study the behavior of connections during the testing. In this case the cost of fabricating and testing the specimens would be prohibitively high.

To conduct realistic tests of flexible connections, the test set-up shown in Figure 2.7 was developed by Astaneh (1,2) and was used in this project to test single plate framing connections. The main components of the test set-up are a beam, a short column, two actuators, and support blocks. Actuator S, which is close to the support, is force controlled and provides the bulk of the shear force in the connection. Actuator R, which is displacement controlled, provides and controls the beam rotation.

Figure 2.8 shows the shear-rotation history that was developed in the connections using the above mentioned test set-ups and methods. Also shown in the figure is the actual shear-rotation relationship that will develop at the end connections of a simply supported uniformly loaded beam.

2.5 Loading History

To establish shear-rotation history to be applied to test specimens, Astaneh (1) developed and used a computer program which simulates loading of a beam supported by flexible connections until the beam collapses. During simulated loading, end shear and rotations are measured. Samples of the results are given in Reference 3.

The computer program was used to analyze all cross sections from W16 to W33 that are listed in the AISC Manuals (8,9). In the analysis spans of 10, 30, and 50 feet were considered for all beams. These analytical studies indicated that end shear vs. end rotation for these beams and spans are very stable and vary slightly with change of shape factor f, where f is equal to z_x/s_x. Figure 2.9 shows two curves representing the shear-rotation values for two extreme cases: a shallow beam with large span and a deep beam with small span. For other cases, the curves fell between the two curves. The material considered was A36 steel. The shear rotation curve that was selected and used in testing corresponds to a beam span to depth ratio (L/d) of about 25 for beams with 36 ksi yield strength. This would also compare with an L/d ratio of 18 for beams with 50 ksi yield strength. In actual structures the ratio rarely exceeds 25.

As the curves in Figure 2.9 indicate, the shear-rotation relationship follows an elastic path until yielding starts at midspan of the beam. At this point, the rate of increase of rotation increases rapidly, causing large rotations for relatively small load increases. In developing the curves in Figure 2.9, the material was assumed to be elastic-perfectly-plastic. However, more realistic curves are expected to exhibit strain hardening effects as shown in Figure 2.9 with dotted lines.

The loading history that was selected to be applied to test specimens, is shown in Figure 2.10. Also shown in the figure, are two dotted curves representing two extreme cases of shear-rotation demands as discussed above.

Segment AB on the loading curve corresponds to the elastic behavior of the beam. At point B, the midspan cross section of the beam reaches M_y and first fiber at midspan of the beam yields. Segment BC on the plot represents the region of inelastic behavior of the beam where the midspan cross sections are experiencing yielding in some fibers but other fibers are still elastic. Point C represents reaching M_p at midspan of the beam when a plastic hinge theoretically has formed. Segment CD represents strain hardening in the beam. Point D corresponds to the ultimate moment capacity at midspan of the beam while end rotations have reached 0.10 radians.

To establish the curve for each test, coupon tests of the plate material were conducted prior to testing to obtain the yield point and ultimate strength of the plate. The results of these coupon tests are in Appendix B. Yield capacity of each plate was calculated using actual plate dimensions and material properties. The yield capacity was denoted R_y to represent reaction corresponding to plastic collapse of the beam when midspan moment reaches M_p. The end reaction of the beam when midspan moment reaches M_y was denoted by R_{yy} and was calculated by dividing R_y by an assumed shape factor of 1.12. The ultimate moment capacity was calculated by multiplying plastic section modulus by ultimate strength of material. The rotations used to establish load path were 0.02 radians for point B and 0.03 radians for point C. The coordinates of point D were selected such that if loading continues, when rotation reaches 0.10 radian, the moment at midspan would reach a value of M_u defined as $M_p F_u / F_y$. The values were selected based on analytical studies of shear-rotation behavior of the beams as reported (2).

2.6 Instrumentation

The instrumentation used in this series of tests is shown in Figure 2.11. The instrumentation consisted of six Linear Variable Displacement Transducers (LVDT), three Tempasonic transducers and four Linear Potentiometers. Four LVDT's were used to measure displacements of top and bottom flanges to be used in calculation of rotation. Two LVDT's were attached to the plate to measure rotation of plate.

Three linear potentiometers were used to measure displacements of the beam. One linear potentiometer was used to measure shear deformation of the single plate while another linear potentiometer was used to measure vertical displacement of the beam end at the connection. The difference between these two measurements was considered to be shear deformation due to slip, hole and bolt deformations.

Actuator R, in Figure 2.11, was controlled by displacement and was used to control rotation at the connection and actuator S was force controlled and was used to control shear. The data acquisition system for the experiments consisted of an IBM-PC based system with capability of real-time recording and processing. Another IBM-PC was connected to the first PC and was used to plot shear vs. rotation to enable test conductors to monitor and follow shear-rotation history during the tests. The equipment is shown in Figure 2.7. In addition to quantitative data, slides and photographs were taken to record qualitative aspects of the research.

2.7 Test Procedures

The following steps were taken in conducting each test:

1. The specimen was prepared for testing by welding the single plate to the column flange.
2. The specimen was assembled by bolting the beam to the single plate. Bolts were tightened to the specified 70% of proof load using the turn-of-the-nut method.
3. Instrumentation was added and specimen was whitewashed. The whitewashing was done to enable the investigators to detect yielded areas.
4. The calibration of the instrumentation was rechecked.
5. The proper operation of the instrumentation and data acquisition systems was checked by applying a very small load and rotation.

6. The shear-rotation history shown in Figure 2.6 was applied to specimen until failure of the connection occurred. Figure 2.8 shows a photograph of the computer display during one of the tests with target and actual load paths. During the test, data was collected at discrete points and significant events were noted, recorded and photographed.

CHAPTER THREE

EXPERIMENTAL RESULTS

3.1 General

The quantitative and qualitative test results are presented in this Chapter. A summary of the behavior of each specimen is presented. The relevant plots of the experimental data for each specimen are given in Appendix A. A summary sheet for each test is provided in Appendix D.

3.2 Behavior of Test Specimens

As discussed in the previous chapter, each test consisted of subjecting the test specimen to shear-rotation history of Figure 2.10. The shears and rotations were monitored to be able to follow the realistic loading path suggested in Section 2.5.

3.2.1. Test Number One

Test specimen one had seven 3/4" diameter, A325 bolts. The plate was 4-1/4x3/8x21 inch and the beam was a a W24x84 with a web thickness of 0.47 inch. The plate and beam were both of A36 steel.

When the shear force reached 67 kips slip occurred in the connection. Significant rotational slip and shear slip were both noticeable by the time the shear force had been increased to 150 kips. Local Yielding of the plate was observed when the shear force reached 158 kips. The failure of the connection occurred when the shear force reached 160 kips and the rotation was 0.031 radians. The failure was due to sudden and brittle fracture of the bolts in shear.

Examination of the connection after the test was completed, showed no evidence of yielding of the weld. Figures 3.1 through 3.4 show this specimen during various stages of testing. Figure 3.5 shows actual load path followed during the tesing.

3.2.2. Test Number Two

Test specimen two had five 3/4" diameter A325 bolts. The plate was 4-1/4x3/8x15 inch. The beam was a W18x55, with web thickness of 0.39 inch. Both the plate and beam material were A36 steel.

When the shear force reached 54 kips slip occurred in the connection. Additional slip occurred when the shear force had been increased to 69 kips. Local yielding of the beam web due to bearing on the bolt holes was evident when the shear force reached 83 kips. When the shear force reached 130 kips it was observed that the single plate had separated about 1/8 inch from the beam web and that the beam web appeared to have buckled slightly. Failure of the connection occurred at 137 kips when the rotation was 0.054 radians.

After the test was completed, examination of the connection showed the single plate had a permanent set of about 0.1 inches in shear deformation. Significant local yielding due to bolt bearing was evident in both the beam web and the single plate. Also it was confirmed that the beam web had buckled. Figures 3.6

through 3.10 show Specimen 2 at various stages of testing and Figure 3.11 shows actual load path.

3.2.3. Test Number Three

Test specimen three had three 3/4" A325 bolts. The plate was 4-1/4x3/8x9 inch. The beam was a W18x55 with web thickness of 0.39 inch. The plate and beam material were both of A36 steel.

The behavior of this specimen was also similar to that of Specimen 2. Yield lines on the whitewash coat of the beam web were observed when the shear force reached 68 kips. Local yielding of the single-plate adjacent to the bolt holes was evident at a shear force of 76 kips. When the shear force had reached 85 kips the bolts had deformed noticeably and about 3/8 inch of slip between the single plate and beam web was present. When the rotation reached 0.063 radians (at a shear force of 86 kips) the displacement actuator was fully extended. For the remainder of the test as the shear force was increased the rotation of the beam could not be increased due to limitations of the actuator. When the shear force reached 94 kips and the rotation was 0.056 radians, the connection failed due to sudden fracture of the bolts.

Examination of the connection after failure showed about 0.20 inche of permanent set due to shear yielding of the plate. For the first time in any of the tests yielding of the weld was visible. Significant bolt hole deformation was observed in both the beam web and the single plate. Also, the bottom portion of the single plate had buckled. Figure 3.12 shows specimen 3 during testing and Figure 3.13 shows one of the bolts after failure. The actual load path for this specimen is shown in Figure 3.14

CHAPTER FOUR

ANALYSIS OF EXPERIMENTAL RESULTS

4.1. General

The experimental results obtained during the investigation are analyzed in this chapter and design procedures are developed. The design procedures are summarized in Chapter 5 along with examples of their applications.

4.2. Yielding of Gross Area of Single Plate

This yielding was primarily due to shear and was quite ductile. It was evident that considerable shear yielding occurred in the plate between the bolt line and weld line. The shear yielding was uniformly distributed throughout the depth of plate with more yielding being observed at mid-height. The amount of shear deformation was larger for specimens with small number of bolts, i.e. specimens 2 and 3. Figure 4.1 shows shear deformations of the plates at the conclusion of tests. Also shown in the Figure 4.1 (c) is the bottom view of the plate in specimen 3. Notice that in this specimen, the lower portion of the plate, which was in compression, has buckled. The buckling is attributed to relatively short depth of the connection and large rotations the were applied to the specimen (0.06 radian).

To prevent the buckling of compression zone of the plate, it is suggested that the height to width ratio, L_p/a, not to be less than 2.0.

Figure 4.2 shows plots of shear vs. vertical displacement of the beam along the bolt center line for each test specimen. The initial slope of shear vs. displacement curves for all three specimens was almost the same and was estimated to be 3,100 kips/inch.

After the onset of yielding, the shear displacement curves showed inelastic behavior until ultimate strength was reached and the bolts fractured. The slip in the specimens is evident in Figure 4.2 by a sudden increase of displacement. It is interesting to note that slip in Specimen Three with 3 bolts occurred suddenly, whereas, in Specimen One with seven bolts slip took place gradually.

To study inelastic activities specimens 2 and 3 had five rosette strain gages mounted on the plate. The locations and directions of these gages are shown in Figure 4.3. The strain gage readings were used to calculate normal strains along the lines making 45, 90 and 135 degrees with vertical line. These normal strains were used to calculate shear strains in the vertical plane and principal strains. In the elastic range of behavior, stress can be calculated by using Hooke's law. However, since inelasticity was dominant during the test, discussion is focused on strain rather than stress. Also, the Von Mises failure criterion was used to calculate effective strain as a predictor of yielding. The criterion can be written as:

$$e_{eff} = [e_1^2 + e_2^2 - e_1 e_2]^{1/2} \leq F_y/E \qquad (4.1)$$

where e_1 and e_2 are principal strains.

Figures 4.4(a) and 4.4(b) show the distribution of normal strains in Specimens 2 and 3 along a vertical plane one inch from the surface of the supporting column flange. The normal strains, plotted in Figure 4.4, represents strains due to bending moment in the connection.

The plate had material properties of yield strain equal to 0.00122 and yield stress of 35.5 ksi. As Figure 4.4 indicates, significant yielding occurred at the outer (top & bottom) fibers of the plate while normal strains along the interior fibers were relatively small.

Five curves are plotted in Figure 4.4 for each specimen. The curves correspond to five significant points during each experiment. The first curve corresponds to the point when maximum shear strength of connection was reached. The second curve correspond to a shear force V_y acting on connection, where V_y is equal to $0.577F_yL_pt$, the shear yield capacity of the plate. The load path used in the experiments, was designed such that when connection reaches its yield capacity, the typical beam supported by the connection, will reach its plastic collapse condition. Therefore, the second curve in the Figure 4.4 also corresponds to the condition of plastic hinge formation in the beam. The third curve in Figure 4.4 corresponds to the condition of beam midspan reaching yield moment. With a shape factor of 1.12, the third curve also corresponds to a shear value of $V_y/1.12$. The fourth curve in Figure 4.4 corresponds to a shear force equal to

$0.60V_y/1.12$ which approximates shear due to service load. The fifth curve corresponds to a shear value of $0.3V_y/1.12$ which approximates condition due to dead load, (DL/LL=1.0).

The shear strains computed from the strain gage measurements are plotted in Figures 4.5(a) and 4.5(b) for Specimens 2 and 3 respectively. The five curves correspond to same levels of shear as discussed above. The shear yield capacity of the plate in Specimens 2 and 3 was 202.35 kips and 121.40 kips respectively. The yield shear strain was 0.00183 and yield shear stress was 20.48 for the material of the plates. The curves in Figure 4.5 indicate that the distribution of the shear strain (and stress) was uniform over the depth of the plate for relatively small shear forces. However as shear increased to the ultimate shear force, (corresponding to M_u), the distribution of shear strain was non-uniform and was close to a parabolic curve.

A comparison of normal and shear strains as given in Figures 4.4 and 4.5 indicates that when approaching maximum shear capacity, shear strains were about 5 times greater than the normal strains. This is an indicator of importance of shear strains and shear deformations in these connections.

Figure 4.6 shows values of effective strain for Specimen 2, calculated using Von Mises' criterion (Equation 4.1). Figure 4.7 is a similar plot for Specimen 3. These figures show the shear load that causes average shear stress, acting on the gross area of the plate, to reach shear yield stress defined by Von Mises criterion. Applying Von Mises' criterion to an infinitesimal element with pure shear stresses has been shown that when shear stress reaches $0.577F_y$, yielding of the element is predicted. As Figures 4.6 and 4.7 indicate, Von Mises criterion used in the form of shear stress reaching $0.577F_y$ predicted the yielding of the plate quite accurately for these specimens, particularly for Specimen 3. For this reason it is recommended that the geometry of the plate be designed by considering the shear load alone, i.e. the moment resisted by the connection does not need to be included in design of plate. In such design the shear yield capacity of plate is limited to $0.577F_yL_pt$ and the corresponding allowable stress is $0.40F_y$.

4.3 Fracture of Net Area of Plate

In the single plate specimens that were tested, the net area of the plate did not fail. Nevertheless, this failure mode has been observed in several experiments on tee framing connections (3). The stem in a tee framing connection behaves similar to a single plate connections. Thus the information obtained from studying the behavior of the stem of the tee can be applied to single framing connections.

Figure 4.8 shows failure of the net section of the stem of a tee from a previous project (3). Figure 4.9 shows distribution of stresses around the bolt hole when a net section is subjected to tension or shear. In tension members, the net section along the centerline of the bolt has a smaller area but the same load as any other section. Thus, net section along the bolt centerline is the most critical section. However, by studying net sections subjected to shear, it appears that the net section along the bolt centerline may not be the most critical section. Referring to Figure 4.9(b), the net area along the bolt centerline is the smallest area but the shear force acting on this area is theoretically equal to only half of the total shear as the free body diagram in the figure indicates. Figure 4.9(c) shows a failure plane that does not have the smallest shear area but the shear force acting on it is maximum and is almost equal to the total applied shear. The experiments reported in Reference 3 on tee framing and in Reference 16 on single plate connection indicated that always shear failure of net area of the plate occurred as shown in Figure 4.9(c). Therefore,it is suggested that, in design of net section in shear, an effective net section for shear fracture be defined that is larger than the actual net section along the bolt centerline but smaller than the gross area of the plate. In Reference (3) an effective net section equal to the average of a gross section and net section along the bolt line was suggested. This effective net section is given by :

$$A_{nse} = (1/2)(A_g + A_{ns}) \tag{4.2}$$

or for single plates;

$$A_{nse} = A_g - 0.5N(d_b + 1/16)t_p \tag{4.3}$$

The current AISC Specifications (22) defines the net area for shear as the net area along the bolt line which can be written as:

$$A_{ns} = A_g - N(d_b + 1/16)t_p \tag{4.4}$$

4.4. Shear-Rotation Behavior

Test specimens were subjected to a combination of shear and rotation as defined by the curve shown in Figure 2.10. Figure 4.10 shows the actual shear-rotation relationship that was recorded during the tests. Two important observations can be made by studying Figure 4.10. First, contrary to direct shear slip, shear-rotation curves indicate that rotational slippage occurred suddenly in the 7 bolt connection whereas, the connections with 3 or 5 bolts slipped gradually. Second, the rotational ductility of the connections increased as the number of bolts decreased. The rotational ductility of the connection with 3 bolts was more than 0.06 radian which was about twice the rotational ductility of the connection with seven bolts.

4.5. Movement of Point of Inflection

Knowledge of the location of the point of inflection in a beam has many uses in design. In continuous beams, splices can be located at these points. In design of connections, knowledge of eccentricity of point of inflection can be used to analyze connection assemblage and obtain moment acting on the connection without performing analysis of the whole structure. In this section we will examine the movement of point of inflection in single span beams.

Consider a single span beam loaded uniformly and supported by completely rigid connections as shown in Figure 4.11(a). In this beam points of inflection, which are points of zero moment, are located at a distance equal to e_{fixed}, where:

$$e_{fixed}=(L/2)[1 - 1/(3)^{-1/2}] = 0.21L \quad (4.5)$$

The above equation is applicable only to the region of elastic behavior. As load increases, beam cross sections at the supports reach their yield point and enter inelastic region of behavior. Increased loading will cause midspan cross sections to enter the inelastic region of behavior until a plastic collapse mechanism forms. During elastic behavior, point of inflection is at a distance of 0.21L from support. During inelastic behavior, point of inflection moves toward the support and when beam collapses by forming plastic hinges at the support and at midspan, point of inflection is at $e_{coll.}$ given by:

$$e_{coll.}=(L/2)\ [1-1/(2)^{-1/2}\ ]=0.146L \quad (4.6)$$

The above discussion is valid only if beam supports are fixed. If beam is supported by semi-rigid or flexible connections, with nonlinear moment-rotation characteristics, the point of inflection in the beam moves almost immediately after loading starts, even though the beam itself might still be elastic.

During this experimental investigation of single plates, necessary displacements were measured and used to calculate eccentricity of the point of inflection. Figure 4.11(b) shows movement of point of inflection toward the support as shear increases. The curves indicate that as shear increased the point of inflection rapidly moved toward the connection (toward left in the figure) during the initial loading and then remained almost stationary while the loading was increased to failure. The stationary position of the point of inflection for test specimens was approximately located at 6, 4 and 2 inches from the support for 7, 5, and 3-bolt specimens respectively. Using the empirical results, the following simple equation was developed to define location of the point of inflection for test specimens.

$$e=(N-1)(1.0") \tag{4.7}$$

where N is the number of bolts used in the connection, and e is the eccentricity of point of inflection from the support (i.e. weld line).

The above eccentricity can be used to calculate bending moment acting on the connection elements such as bolt group and weld lines as discussed in the following two sections.

4.6. Behavior and Design of Bolts

Bolts in all specimens were 3/4 inch diameter ASTM-A325 bolts with threads included in the shear plane. All bolts were tightened using turn of nut method to achieve a tension equal to 70% of their proof load as specified in the AISC Manual (22). Information on mechanical properties and chemical composition of bolts is given in Appendix B.

In all specimens, bolts failed in relatively brittle manner. The failure mode was shear failure of bolts through threaded area. In all cases, an examination of bolts and bolt holes after failure indicated that bolts had experienced considerable deformations before failure. Figure 3.13 in Chapter 3 shows a bolt after failure.

The total shear displacement of the beam was due to two sources: (1) shear deformations of plate and (2) bolt and hole deformations. To separate the contribution of each source, Specimens 2 and 3 were instrumented such that the relative shear-displacement between the beam web and the plate, along the bolt line, could be measured and recorded. This displacement mainly is due to bolt and hole deformations. By subtracting this component of displacement from the total shear displacement, displacement due to plate shear deformations can be obtained. Figures 4.12 and 4.13 show total shear displacements and displacements due to bolt and hole deformations for Specimens 2 and 3. As the plots indicate under small shear forces only elastic deformations of plate were present in both specimens. After shear force reached 21 kips in Specimen 3 and 65 kips in Specimen 2, bolt and hole deformations as well as plate shear deformations contribute to the total shear displacement. As a result, in design, it is important to ensure that bolt and hole deformations take place in a ductile manner and contribute to the overall ductility of the connection.

Extensive studies by R. Richard et al. (19) on the behavior of single bolt in shear have indicated that for A325 bolts and A36 plate, if thickness of plate is less than 1/2 diameter of the bolt, considerable, but tolerable, bolt hole deformations will take place. Therefore, following their findings, it is recommended that in design of single plate connections, thickness of the plate be less than or equal to 1/2 of the bolt diameter.

In this project, bolts in single plate shear connections were subjected to direct shear and a small moment. This is based on measured shear and moments and a study of bolt hole deformation after the tests. Figures 4.14(a), (b) and (c) show bolt hole deformations in the beam web and the plate after conclusion of the tests. The arrows indicate direction of movement of the bolts which is expected to be close to the direction of the applied shear force to the bolt. The bolt movements indicate presence of large shear forces and small moments in the connections that were tested. In design, bolt groups should be designed for the combination of shear and moment. The shear force acting on the bolt group, for practical design purposes, is equal to the reaction of the beam. The moment is equal to the reaction multiplied by the eccentricity of the reaction from the bolt line given by:

$$e_b = (N-1)(1.0")-a \qquad (4.8)$$

where, a is the distance between bolt line and weld line.

After shear and moment acting on the bolt group is established design aids can be used to determine the bolts required. The AISC Manual (22) provides tables to aid in the design of welded or bolted connections under eccentric loads. These tables take into account the inefficiency of a connection that has both a shear and a moment applied. The efficiency can be defined as the ratio of the connection strength for eccentric loads divided by the strength for a concentric load. Figure 4.15 shows how the efficiency changes for bolted connections depending upon the number of bolts and the eccentricity of the shear.

Obviously as a load increases in eccentricity or a connection becomes more centralized, the efficiency decreases.

These tables are based on research performed in the past and are mathematical equations involving the moment of inertia for the connection. The table for design of eccentric bolted connection in the AISC Manual (22) is based on the research done by Crawford and Kulak (6). The design table for the bolts relates a coefficient C to a combination of geometric parameters. For single-plate framing connections the required bolt pattern can be determined from Table X in the AISC Manual (22), using eccentricity of the shear force from the bolt line. Since the eccentricity e_b is seldomly more than three inches for connections with seven bolts or less, the values given in the first line of Table X is all that must be considered.

With the advent of computers, it is useful to convert information in Table X of the AISC Manual (22) to a mathematical equation so it can be programmed and used in computerized applications. To obtain an equation, a cubic polynomial can be used to approximate the values in Table X in the AISC Manual (22). A regression technique was performed to calculate the coefficients for the polynomial. This technique is a common numerical procedure (7). Using this procedure the following equation was developed. This equation can calculate the efficiency of a bolt group.

$$C = -0.48357 + 0.47798N + 0.11226N^2 - 0.00667N^3 \qquad (4.9)$$

where N = number of bolts; not less than 2 nor more than 7, and

C is the number that is also given in Table X of the AISC Manual (22). The value of C can be considered the equivalent number of bolts required if the loading were in direct shear. Figure 4.16 shows a comparison of the above proposed equation with the values given in Table X of the AISC Manual (22).

4.7. Weld Behavior and Design

The welds in all specimens were fillet welds fabricated in a downward position using GMAW welding procedures. The size of fillet weld leg in all specimens was 1/4 inch. The welds showed very few visible signs of yielding on the whitewash coating during the tests. Only Specimen 3 showed signs of minor yielding, 1/2 inch from the top and 1/2 inch from the bottom of the fillet weld lines.

The fillet welds mainly experienced a direct shear accompanied by a relatively small moment in the connections that were tested. This is evident from strain measurements as was discussed in Section 4.2. Therefore, fillet welds should be designed for the combined effects of the shear and bending moment. Shear acting on the connection is equal to the reaction of the beam. Moment acting on the weld is equal to shear multiplied by the eccentricity of the point of inflection from weld line. As indicated in Section 4.5, the specimens studied had an eccentricity of point of inflection from weld line which could be related to number of bolts by Equation 4.7. However, it is suggested that for the design of welds, until more test results become available, conservatively the eccentricity e_w equal to N inches be used.

After e_w is established, welds can be designed for the combined effects of shear and bending moment using Table XIX in the AISC Manual (22). The table is based on the research conducted by Butler, Pal and Kulak (4). For the fillet welds in single plate framing connections, the special case of out-of-plane bending with k equal to 0.0, Table XIX of the AISC Manual (22) can be used. Figure 4.17 is a plot of values given for coeeficient "C" in the table. In this table, C is given as a function of the eccentricity. Using the same regression technique as applied in Section 4.6 the following equation was developed that provides values very close to those tabulated in the second column of Table XIX. The column corresponds to k equal to 0.0 and is applicable to the case of single plate welded to the support.

$$C = 1.8063 - 2.4665a + 1.2517a^2 - 0.20722a^3 \qquad (4.10)$$

where, a is the ratio of the eccentricity e_w and the weld length. (The AISC Manual (22), Table XIX uses the symbol "a" as one variable in determining the strength of an eccentrically loaded weld. Current steel design methods also use the symbol "a" to represent the distance between the bolt line and weld line of a single-plate connection. In this report both notations are used.) Figure 4.18 shows a comparison of the above proposed equation with the values given in Table XIX of the AISC Manual (22).

4.8. Moment-Shear Behavior

Figure 4.19, 4.20 and 4.21 show variation of shear versus moment at the bolt line and weld line during the tests. The curves show an initial segment where, due to the initial stiffness of the connection, moment and shear increased simultaneously. However, beyond the initial stage, bending moment decreased and then very slowly increased while shear force continued to increase.

4.9. Moment-Rotation Curves

Moment-rotation curves for test specimens are shown in Figures 4.22, 4.23 and 4.24. Moments are measured along the bolt line as well as weld line and plotted separately. Rotation for both plots in each figure corresponds to rotation of cross section along the bolt line.

As plots indicate, connections with fewer bolts developed smaller moment and exhibited larger rotational ductility. In all specimens, after the initial stage, rotational stiffness decreased suddenly and increased gradually. Compared to predictions of the Richard's Model (19) , relatively smaller bending moments were developed in the connections and moment rotation curves indicated that connections are more flexible than predicted by Richard's model. The release of connection moment is attributed to the loss of stiffness primarily due to inelastic shear deformations of the plate and secondarily due to deformations of bolts and bolt holes.

4.10. Movement of Neutral Axis in Connection

In a symmetric beam, neutral axis is located at mid-height of the beam. However, as we approach the connection, depending on response of connection elements, location of neutral axis can be on, below or above the neutral axis. Location of neutral axis in the connection depends on the stiffness of connection elements located above or below the centroid. If connection stiffness is symmetric with respect to the centroidal axis, then it is expected that neutral axis remains at the centroid. However, in some connections in which stiffness is not symmetric, neutral axis is not at mid-height. Examples of connections that are not symmetric with respect to rotational stiffness are double angle and tee framing connections shown in Figure 4.25. Both connections are geometrically symmetric. However, when moment is applied to these connections the portion of connection that is pulled away from the support, (top sketch in Figure 4.25), has much less stiffness than the portion which is pressed against the support, (bottom sketch in Figure 4.25(b) and (c)). Therefore, double angle and tee framing connections are not symmetric with respect to stiffness.

Experimental studies of double angle and tee framing connections (3,14) indicated that indeed neutral axis in these connections moves downward as the tension side of connection deforms and moves away from the support. Figure 4.26 shows movement of neutral axis in a tee connection studied (3).

For the single plate framing connections, the stiffness of the connection is almost symmetric. Referring to Figure 4.27, the top portion of connection being pulled away from the support, will exhibit almost the same stiffness as the bottom portion being pressed against the support. In experiments reported here, the location of neutral axis was computed by using displacement recordings. Figure 4.28 shows location of neutral axis for Specimens 1, 2 and 3. As the plots indicate, during the tests, neutral axis remained very close to the mid-height of the single plate. Referring to Figures 2.1, 2.2 and 2.3, in specimens 1 and 2, mid-height of the beam and the single plate was the same, however, for Specimen 3 mid-height of the plate was 3 inches above the mid-height of the beam.

One advantage of knowing location of neutral axis in the connection is that one can expect less rotational slippage from the bolts in the vicinity of neutral axis. The knowledge can be used in snug tight connections, where bolts in the vicinity of neutral axis can be tightened to achieve sufficient stiffness to assist erection process but do not contribute considerable rotational stiffness.

CHAPTER FIVE

CONCLUSIONS

5.1 General

The main objective of this research was to investigate the behavior of single plate connections and to develop design procedures. The specific findings and results have been presented in previous chapters. In this chapter, general conclusions are provided. The conclusions are based on the studies of single plate connections with seven or less bolts and should not be applied to cases that differ significantly from those studied here.

5.2 Conclusions

Based on the studies reported here, the following conclusions can be made:

1. The realistic testing of single plate connections indicated that considerable shear yielding occurred in the plate prior to the failure. The yielding caused reduction of the rotational stiffness which in turn caused release of the end moments to midspan of the beam. To ensure the desirable yielding, it is recommended that the material of

the plate be a steel with low yield stress such as A36 steel.

2. The connections that were studied showed some stiffness at the early stages of the loading but as yielding of plate and bolt hole assembly started, the rotational stiffness of the connections decreased significantly. While approaching ultimate shear strength, the connections were flexible enough to be considered type II (simple) connections in common applications.

3. The limit states associated with single plate connections are:

   a. Plate yielding.

   b. Fracture of net section of plate.

   c. Bolt fracture.

   d. Weld fracture.

   e. Bearing failure of bolt holes.

4. A design procedure for single plate framing connections is developed and recommended in Chapter 6. The procedure is based on a new concept that emphasizes facilitating shear and bearing yielding of the plate to reduce rotational stiffness of the connection.

5. To avoid bearing fracture, the horizontal and vertical edge distance of the bolt holes are recommended to be at least 1.5 diameter of the bolt as specified by the AISC Specification (22). The study reported here indicated that vertical edge distance, particularly, below the bottom bolt is the most critical edge distance.

6. Single plate connections that were tested were very ductile

and tolerated rotations from 0.03 to 0.054 radians at the point of maximum shear. Rotational ductility decreased with the increase in the number of bolts.

CHAPTER SIX

PROPOSED DESIGN PROCEDURES

6.1. General

The single plate framing connections covered by these procedures consist of a plate bolted to a beam web and welded to the support. The support can be a column, a beam or other steel elements such as embedded plates.

The design procedure that follows is based on the findings of the research program that was discussed in previous chapters. In Reference (2) an analysis was conducted to establish realistic shear-rotation relationship that a shear connection will experience in actual structures. Then by using a special test set-up, single plate connections were subjected to these realistic shear and rotations and their behavior was studied. The research program indicated that the single plates that are designed according to the following procedures, are rotationally flexible and possess sufficient ductility to be considered simple (shear) connections.

6.2. General Requirements

In design of single plate framing connections, the following requirements should be satisfied:

1. Material of the plate should be A36 steel.
2. The ratio of L_p/a of the plate should be more than 2.

3. ASTM A325 and A490 bolts may be used. Either slip critical as well as snug tight bolts are permitted. The bolts should be used in only one vertical row. Oversized and long slotted bolt holes should not be permitted. Punched or drilled holes are permitted. The number of bolts should not be more than seven or less than two.
4. Welds are fillet welds with E70xx or E60xx electrodes.
5. Vertical spacing between the bolts is equal to 3 inches.

6.3. Limit States

The following limit states are associated with the single plate framing connections.

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

In the design procedure given here, the basic consideration is to facilitate reaching limit state of yielding of gross area before limit states three, four or five are reached. If limit state of yielding is reached first, the behavior when approaching limit state, is expected to be quite ductile and desirable. Appendix C contains a computer output of the estimated shear capacity for a wide range of possible single-plate connection designs.

6.3.1. Shear Failure of Bolts

Bolts are designed for the combined effects of direct shear and a moment due to the eccentricity of the reaction from the bolt line. The eccentricity e_b, for single plate connections covered in this section, can be assumed to be equal to a. The value is based on research results(1) and is conservative when single plate is welded to the flange of a relatively rigid column. The value is more realistic when supporting member is a column web or a beam. More realistic values for e_b can be obtained from the following equations:

If single plate is welded to a rotationally rigid element such as a column flange, e_b is larger value obtained from:

$$e_b = (N-1)1.0 - a \quad \text{and;} \tag{6.1a}$$

$$e_b = 0.0 \tag{6.1b}$$

If single plate is welded to a rotationally flexible element such as a column web or one side of a beam, e_b is larger value obtained from:

$$e_b = (N-1)1.0 - a \quad \text{and;} \tag{6.2a}$$

$$e_b = a \tag{6.2b}$$

where N is the number of bolts, a is the distance from bolt line to weld line and e_b is eccentricity in inches.

By using Tables X of the AISC-ASD Manual the bolts are designed for the combined effects of shear (R) and moment (Re_b).

6.3.2. Yielding of Gross Area of Plate

This is primarily a shear yielding limit state and is expected to be quite ductile. It is recommended that, whenever possible, this limit state be the governing limit state in design of single plate connections. The equation defining this limit state in allowable stress design (ASD) format is:

$$f_{vy} \leq F_{vy} \qquad (6.3a)$$

where,

$$f_{vy} = R / A_{vg} \qquad (6.3b)$$

$$F_{vy} = 0.40\ F_y \qquad (6.3c)$$

$$A_{vg} = L_p\ t_p \qquad (6.3d)$$

6.3.3. Fracture of Net Area of Plate

This limit state is reached when the effective net area of the plate fractures in shear. The effective net area in shear is conservatively equal to the average of net area and gross area in shear. The equation defining this limit state in allowable stress design (ASD) format is:

$$f_{vu} \leq F_{vu} \qquad (6.4a)$$

where,

$$f_{vu} = R / A_{ns} \qquad (6.4b)$$

$$F_{vu} = 0.30\ F_u \qquad (6.4c)$$

$$A_{nse} = [L_p - N(1/2)(d_b + 1/16)]t_p \qquad (6.4d)$$

If beam is coped, the block shear failure of beam web also should be considered as discussed in the AISC-ASD Specification.

Currently, the AISC Specifications (22,23) consider the net area in shear to be the same as the tension net area. This results in a more conservative design with fracture of the net area normally governing the connection's design.

6.3.4. Weld Failure

Welds connecting the plate to the support are designed for the combined effects of direct shear and a moment due to eccentricity of the reaction from the weld line, e_w. The eccentricity e_w is equal to larger value obtained from:

$$e_w = N(1.0) \quad \text{and;} \tag{6.5a}$$

$$e_w = a \tag{6.5b}$$

where N is the number of bolts and e_w is eccentricity in inches.

Using Tables XIX of the AISC-ASD Manual, fillet welds are designed for the combined effects of shear (R) and moment (Re_w).

6.3.5. Bearing Failure of Plate or Beam Web

This limit state is reached when due to insufficient horizontal or vertical edge distances, an edge distance fractures. It must be mentioned that limited yielding of bolt holes in bearing can be tolerated, however, large hole elongations are not desirable.

To avoid reaching this limit state, it is recommended that the established rule of horizontal and vertical edge distances being at least twice the bolt diameter be followed. The bolt spacings should satisfy requirements of the AISC-ASD Specifications. The bearing strength of connection can be calculated using the provisions of the AISC-ASD Specifications.

6.4. Summary of Design Procedure

Following steps are taken in design of single plate framing connections:

A. Calculate number of bolts required to resist combined effects of shear (R) and moment (Re_b) using Table X of the AISC-ASD Manual (22).

If single plate is welded to a rotationally rigid element such as a column flange, e_b is larger value obtained from:

$$e_b = (N-1)1.0-a \quad \text{and;} \tag{6.1a}$$

$$e_b = 0.0 \tag{6.1b}$$

If single plate is welded to a rotationally flexible element such as a column web or one side of a beam, e_b is larger value obtained from:

$$e_b = (N-1)1.0-a \quad \text{and;} \tag{6.2a}$$

$$e_b = a \tag{6.2b}$$

B. Calculate required gross area of plate:

$$A_{vg} = R \ / \ 0.40F_y \tag{6.6}$$

Use A36 steel and select a plate satisfying the following requirements:

a. l_h and $l_v \geq 2d_b$. (6.7a)

b. $L_p \geq 2a$ (6.7b)

c. $t_p \leq d_b/2$ (6.7c)

d. $t_p \geq A_{vg} / L_p$ (6.7d)

C. Check effective net section:

Calculate actual allowable shear yield strength of the selected plate:

$$R_o = L_p t_p \ (0.40F_y) \quad (6.8a)$$

Calculate allowable shear strength of the effective net area:

$$R_{nse} = [L_p - N(1/2)(d_b + 1/16)](t_p)(0.3F_u) \quad (6.8b)$$

and satisfy that $R_{nse} \geq R_o$.

D. Design fillet welds for the combined effects of shear (R_o) and moment ($R_o e_w$) using Table XIX of the AISC Manual (22).

e_w is larger of:

$e_b = N(1.0)$ and; (6.5a)

$e_b = a$ (6.5b)

The weld is designed for a capacity of R_o to insure that the plate will yield before the weld fails.

E. Check bearing capacity and satisfy the following equations for beam web and plate:

$$(N)(t)(d_b)(1.2F_u) \geq R_o \quad (6.9)$$

If the bolts are expected to resist a moment (as they normally would), this calculation should reflect the reduced strength as determined by Table X of the AISC Manual (22).

F. If beam is coped, the possibility of block shear failure should be investigated.

6.5. Application to Design Problems

The following examples show how the design procedure can be implemented into the design of steel structures.

6.5.1. Design Problem One

<u>Given:</u>

Beam: W27x114, t_w=0.570 in.

A36 steel

Support: A column flange

Reaction: 102 kips (Service Load)

Bolts: 7/8" dia. A490-N (snug tight)

Bolt Spacing: 3"

Welds: E70XX fillet welds

Design a single plate framing connection to transfer the beam reaction to supporting column.

<u>Solution:</u>

1. Calculate number of bolts:

Shear= 102 kips

Let us assume M=0, (will be verified later)

N= R/ r_v= 102/16.8=6.1

Try 7 bolts.

The distance between bolt line and weld line, a, is selected equal to 3 inches.

Check moment:

e_b=(N-1)1.0-a=7-1-3=3.0 in.

Moment= 3.0x102 =306 k-in.

Using Table X of the AISC-ASD Manual with eccentricity of 3 inches, a value of 6.06 is obtained for effective number of bolts (7 bolts are only as effective as 6.06 bolts).

Therefore,

$R=6.06x16.8=101.8 \approx 102$ kips say O.K.

Then;

<u>USE: Seven 7/8" dia. A490-N bolts.</u>

2. Calculate required gross area of plate:

$A_{vg}= R / 0.40F_y$

$A_{vg}=102/(0.40x36)=7.08$ in^2

Use A36 steel and select a plate satisfying the following requirements:

a. l_h and $l_v \geq 2d_b$.

$l_h = l_v = 2(7/8") = 1.75$ in.

$W = a+l_h = 3+1.75 = 4.75$

b. $L_p/a \geq 2.0$

$L_p = 2x1.75+6x3.0=21.5$ in.

Check: $L_p/a = 21.5/3 = 5.4 > 2$ O.K.

c. $t_p \leq d_b/2$

$t_p \leq (7/8)/2 = 7/16$ in.

d. $t_p = A_{vg} /L_p$

$t_p = 7.08/21.5= 0.329$ in.

<u>USE PL 12x3/8x4-3/4 A36</u>

3. Calculate actual allowable yield strength of the selected plate:

$R_o= L_p t_p (0.40F_y)$

$R_o= 21.5x0.375x0.40x36=116.1$ kips

Calculate allowable shear strength of the effective net area:

$R_{nse}=[L_p-N(1/2)(d_b+1/16)](t_p)(0.3F_u)$

R_{nse}=[21.5-7(1/2)(7/8+1/16)](3/8)(0.3x58)=118.9 kips

$R_{nse} \geq R_o$ is satisfied.

4. Design fillet welds for the combined effects of shear and moment:

Shear = R_o = 116.1 kips

e_b = MAX. N(1.0)=7(1.0)=7 in.

a =3 in.

Therefore, e_b=7.0 in.

Moment= $R_o e_w$ = 116.1x7= 812.7 kip-in.

a=7/21.5=0.326

C_1=1.0

C=1.10

D_{16}= R_o/CC_1L_p = 116.1/(1.0x1.1x21.5)=4.91

USE 5/16" E70 Fillet Welds.

5. Check bearing capacity:

For plate:

$r_v = d_b t_p(1.2F_u)$ = .875x.375x1.2x58 = 22.84

R_{brg} = 6.06(22.84) = 138.4 kips

Since the beam's web is thicker than the plate, the web will not fail.

6. Beam is not coped, therefore, there is no need for consideration of block shear failure.

6.5.2. Design Problem Two

Given:

Beam: W16x31, t_w=0.275

A572 Gr. 50 steel

Support: A column flange

Reaction: 35 kips (Service Load)

Bolts: 3/4" dia. A325-N (snug tight)

Bolt Spacing: 3"

Welds: E70XX fillet welds

Design a single plate framing connection to transfer the beam reaction to supporting column.

Solution:

1. Calculate number of bolts:

Shear= 35 kips

Let us assume M=0, (will be verified later)

$N= R/ r_v= 35/9.3 =3.76$

Try 4 bolts.

The distance between bolt line and weld line, W, is selected equal to 3 inches.

Check moment:

$e_b = (N-1)1.0-a = 4-1-3 = 0.0$ in.

Moment= 0.0x35 =0.0 k-in.

Therefore, the above assumption of M=0.0 is verified.

Then;

USE: Four 3/4" dia. A325N bolts.

2. Calculate required gross area of plate:

$A_{vg} = R / 0.40F_y$

$A_{vg} = 35/(0.40x36) = 2.43\ in^2$

Use A36 steel and select a plate satisfying the following requirements:

a. l_h and $l_v \geq 2d_b$.

$l_h = l_v = 2(3/4") = 1.50$ in.

$W = a + l_h = 3 + 1.50 = 4.50$ in.

b. $L_p/a \geq 2.0$

$L_p = 3" + 3x3" = 12"$

Check: $L_p/a = 12/3 = 4 > 2$ O.K.

c. $t_p \leq d_b/2$

$t_p \leq (3/4)/2 = 3/8"$

d. $t_p = A_{vg} / L_p$

$t_p = 2.43/12 = .203$ in.

<u>USE PL 12x1/4x4-1/2 A36</u>

3. Calculate actual allowable yield strength of the selected plate:

$R_o = L_p t_p (0.40F_y)$

$R_o = 12x0.25x0.40x36 = 43.2$ kips

Calculate allowable shear strength of the effective net area:

$R_{nse} = [L_p - N(1/2)(d_b + 1/16)](t_p)(0.3F_u)$

$R_{nse} = [12 - 4(1/2)(3/4 + 1/16)](1/4)(0.3x58) = 45.1$ kips

$R_{nse} \geq R_o$ is satisfied.

4. Design fillet welds for the combined effects of shear and moment:

Shear $= R_o = 43.2$ kips

e_b = MAX. $N(1.0) = 4(1.0) = 4$ in.

$a = 3.0$

Therefore, $e_b = 4.0$ in.

Moment= $R_o e_w$ = 34.2x4= 136.8 kip-in.

a=4/12=0.33

C_1=1.0

C=1.07

D_{16}= R_o/CC_1L_p = 43.2/(1.0x1.07x12)=3.36

<u>USE 1/4" E70 Fillet Welds.</u>

5. Check bearing capacity (remember that moment at boltline = 0)

For plate:

$Nd_bt_p(1.2F_u)$= 4x.75x.25x1.2x58= 52.2 kips > 43.2 kips.

and for beam:

$Nd_bt_w(1.2F_u)$= 4x.75x.25x1.2x65= 58.5 kips > 43.2 kips.

6. Beam is not coped, therefore, no need for consideration of block shear failure.

6.5.3. Design Problem Three

<u>Given:</u>

Beam: W14x22, t_w=0.23 in.

A36 steel

Support: A girder (only one beam from one side is attached to the girder)

Reaction: 11 kips (Service Load)

Bolts: 5/8" dia. A325-N (snug tight)

Bolt Spacing: 3"

Welds: E60XX fillet welds

Design a single plate framing connection to transfer the beam reaction to supporting girder.

Solution:

1. Calculate number of bolts:

Shear= 11 kips

Let us assume M=0, (will be verified later)

N= R/r_v= 11/6.4=1.7

Try 2 bolts.

The distance between bolt line and weld line, W, is selected equal to 3 inches.

Check moment:

e_b = (N-1)1.0-a = 2-1-3 = -2.0 in.

e_b =a= 3.0 in. (since flexible support is assumed)

Therefore, e_b=3.0 in.

Moment= 3x11 =33.0 k-in.

Using Table X of the AISC-ASD Manual with eccentricity of 3 inches, a value of 0.88 is obtained for effective number of bolts (i.e. 2 bolts are only as effective as 0.88 bolt).

Therefore,

R=0.88x6.4=5.63 < 11 kips N.G.

Try 3 bolts.

Check moment:

e_b = (N-1)1.0-a = 3-1-3 = -1.0 in.

e_b = a = 3.0 in.

Therefore, e_b = 3.0 in.

Moment= 3x11 =33.0 k-in.

Using Table X of the AISC-ASD Manual with an eccentricity of 3 inches, a value of 1.75 is obtained for effective number of bolts (i.e. 3 bolts are only as effective as 1.75 bolts).

Therefore,

$R=1.75x6.4=11.2 \ > 11$ kips. O.K.

Then;

<u>USE: Three 5/8" dia. A325-N bolts.</u>

2. Calculate required gross area of plate:

$A_{vg}= R / 0.40F_y$

$A_{vg}=11/(0.40x36)=0.76 \ in^2$

Use A36 steel and select a plate satisfying the following requirements:

a. l_h and $l_v \ \geq \ 2d_b$.

$l_h = l_v = 2(5/8") = 1.25$ in.

$W = a+l_h = 3+1.25 = 4.25$ in.

b. $L_p/a \geq 2.0$

$L_p = 2x1.25+2x3.0=8.5$ in.

Check: $L_p/a = 8.5/3 = 2.8 > 2$ O.K.

c. $t_p \leq d_b/2$

$t_p \leq (5/8)/2 = 5/16$ in.

d. $t_p = A_{vg} /L_p$

$t_p = 0.76/8.5= 0.089$ in.

<u>USE PL 8.5x3/16x4-1/4 A36</u>

3. Calculate actual allowable yield strength of the selected plate:

$R_o= L_pt_p \ (0.40F_y)$

$R_o= 8.5x(3/16)x0.40x36 = 23$ kips

Calculate allowable shear strength of the effective net area:

$R_{nse}=[L_p-N(1/2)(d_b+1/16)](t_p)(0.3F_u)$

$R_{nse}=[8.5-3(1/2)(5/8+1/16)](3/16)(0.3x58)=24.36$ kips

$R_{nse} \geq R_o$ is satisfied.

4. Design fillet welds for the combined effects of shear and

moment:

Shear = R_o = 23.0 kips

e_b = MAX. N(1.0)=3(1.0)=3 in.

a=3.0 in.

Therefore, e_b=3.0 in.

Moment= $R_o e_w$ = 23x3= 69 kip-in.

a=3/8.5=0.352

C_1=0.857 (for E60xx electrode)

C=1.035

D_{16}= R_o/CC_1L_p = 23/(0.857x1.035x8.5)=3.05

<u>USE 3/16" E60 Fillet Welds.</u>

5. Check bearing capacity:

For plate:

r_v = $d_b t_p(1.2F_u)$= .625x(3/16)x1.2x58 = 8.16

R_{brg} = 1.75(8.16) = 14.3 kips

and for beam, similarly bearing is satisfied because the web is thicker than the plate.

6. Beam is coped and block shear failure needs to be checked.

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Table 2.1.-Properties of Test Specimens

TEST NUMBER	SPECIMEN	NO. OF BOLTS	DIA. OF BOLTS	TYPE OF BOLTS	PLATE (inxinxin)	WELD SIZE
1	7-3/4-S-3/8	7	3/4	A325-N	21x3/8x4-1/4	1/4
2	5-3/4-S-3/8	5	3/4	A325-N	15x3/8x4-1/4	1/4
3	3-3/4-S-3/8	3	3/4	A325-N	9x3/8x4-1/4	1/4

Note: 1) All specimens were fabricated with standard size, round holes that were punched.

2) All bolts were tightened to 70% of proof load by using turn-of-the-nut method.

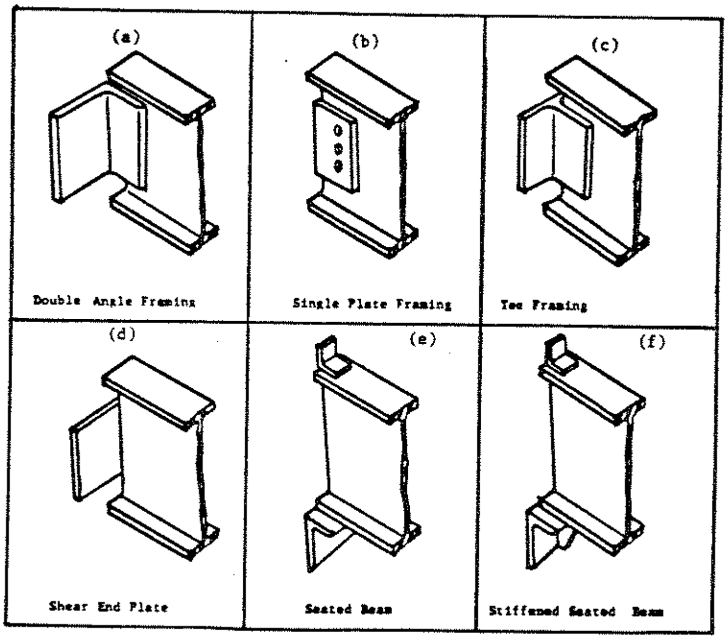


Figure 1.1 Common Types of Steel Shear Connections

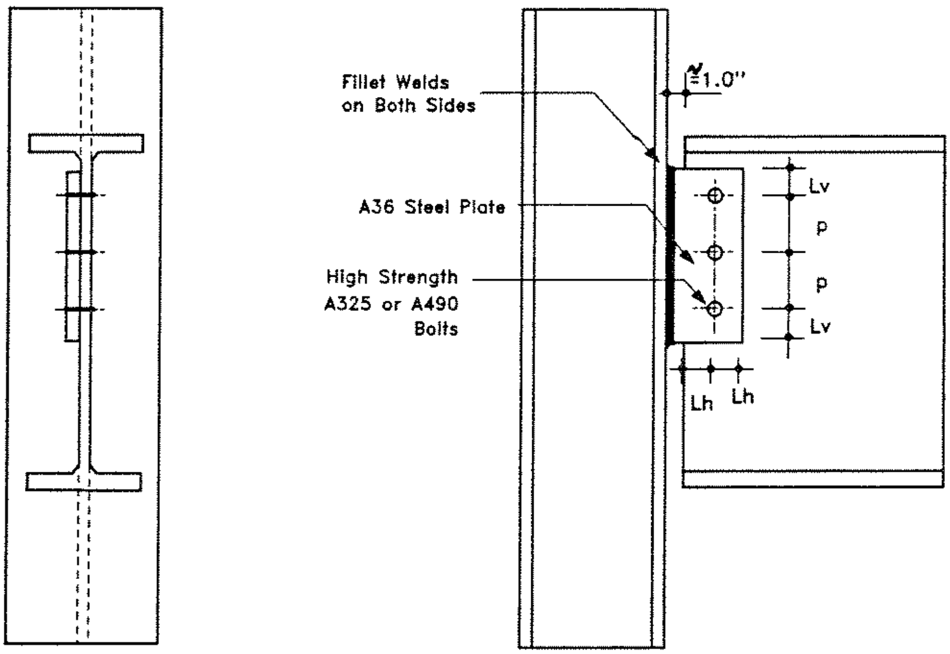


Figure 1.2. Single Plate Framing Connections

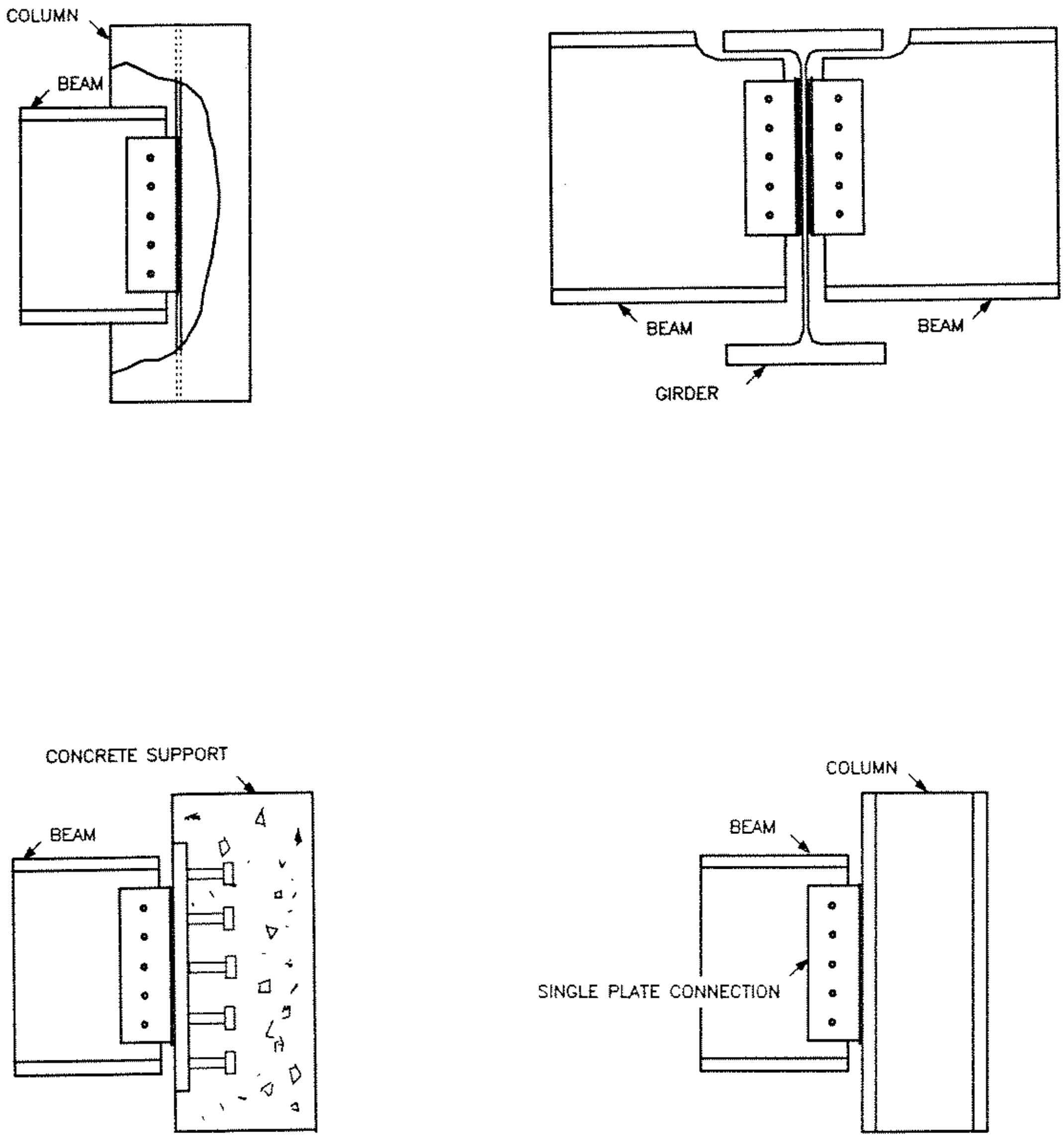


Figure 1.3. Typical Applications of Single Plate Framing Connections

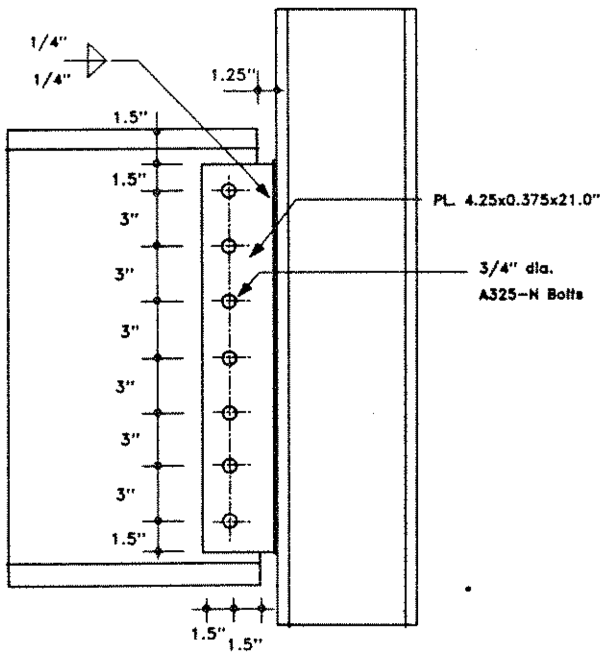


Figure 2.1. Test Specimen One

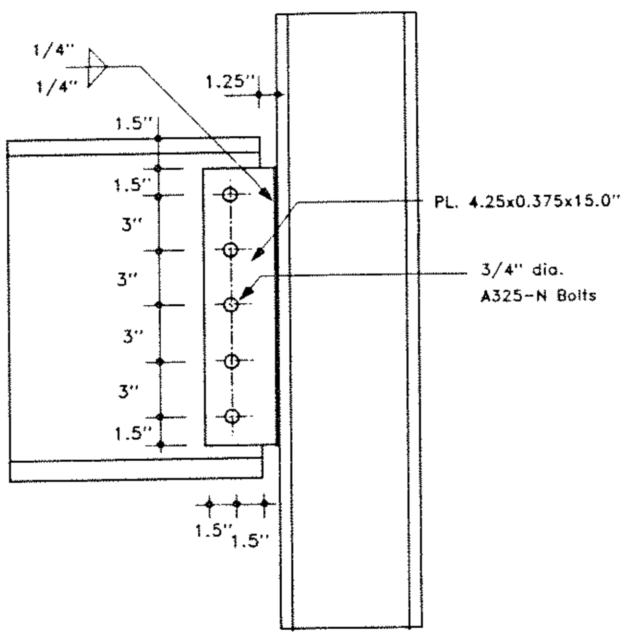


Figure 2.2. Test Specimen Two

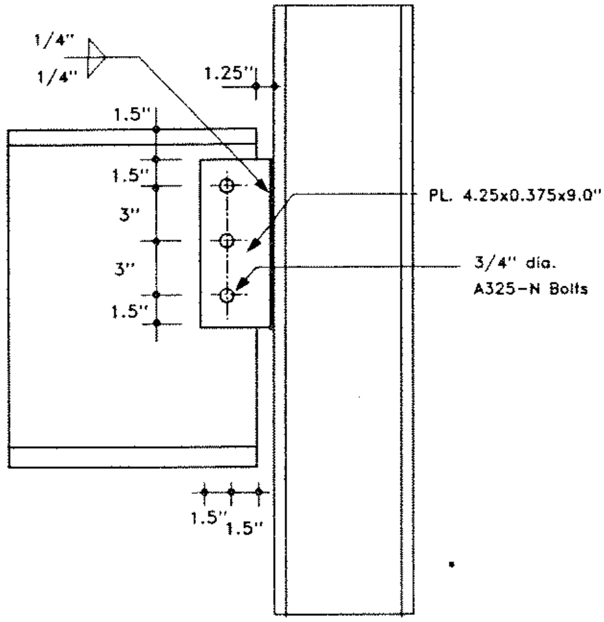


Figure 2.3. Test Specimen Three

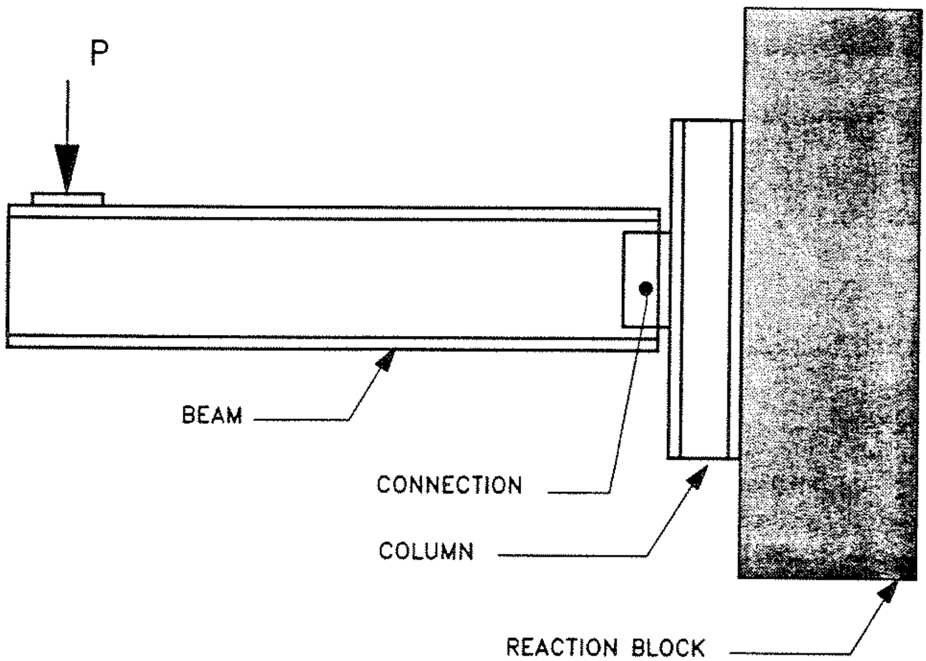


Figure 2.4. Cantilever Test Set-up Used in the Past

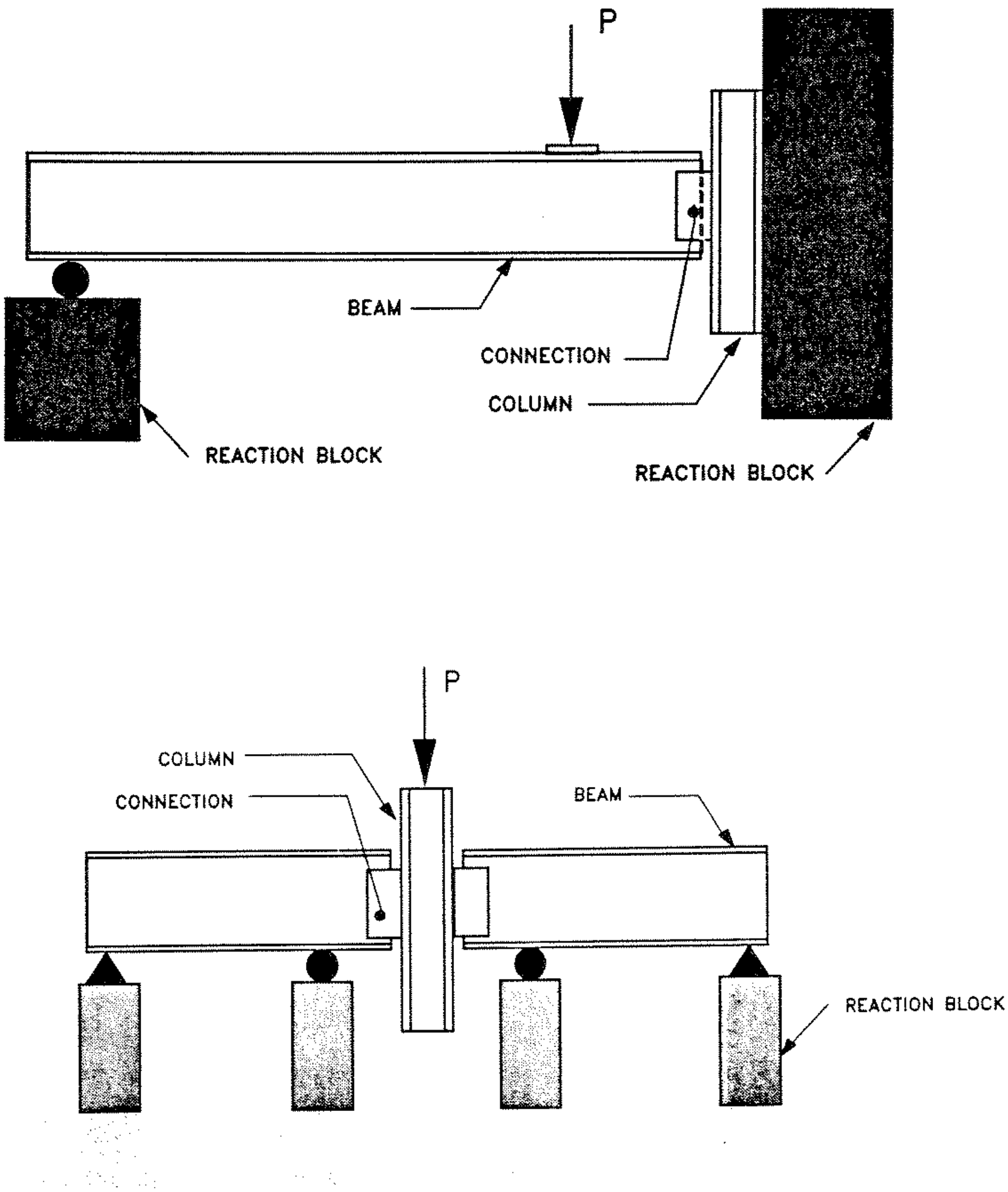


Figure 2.5. Direct Shear Test Set-up Used in Past

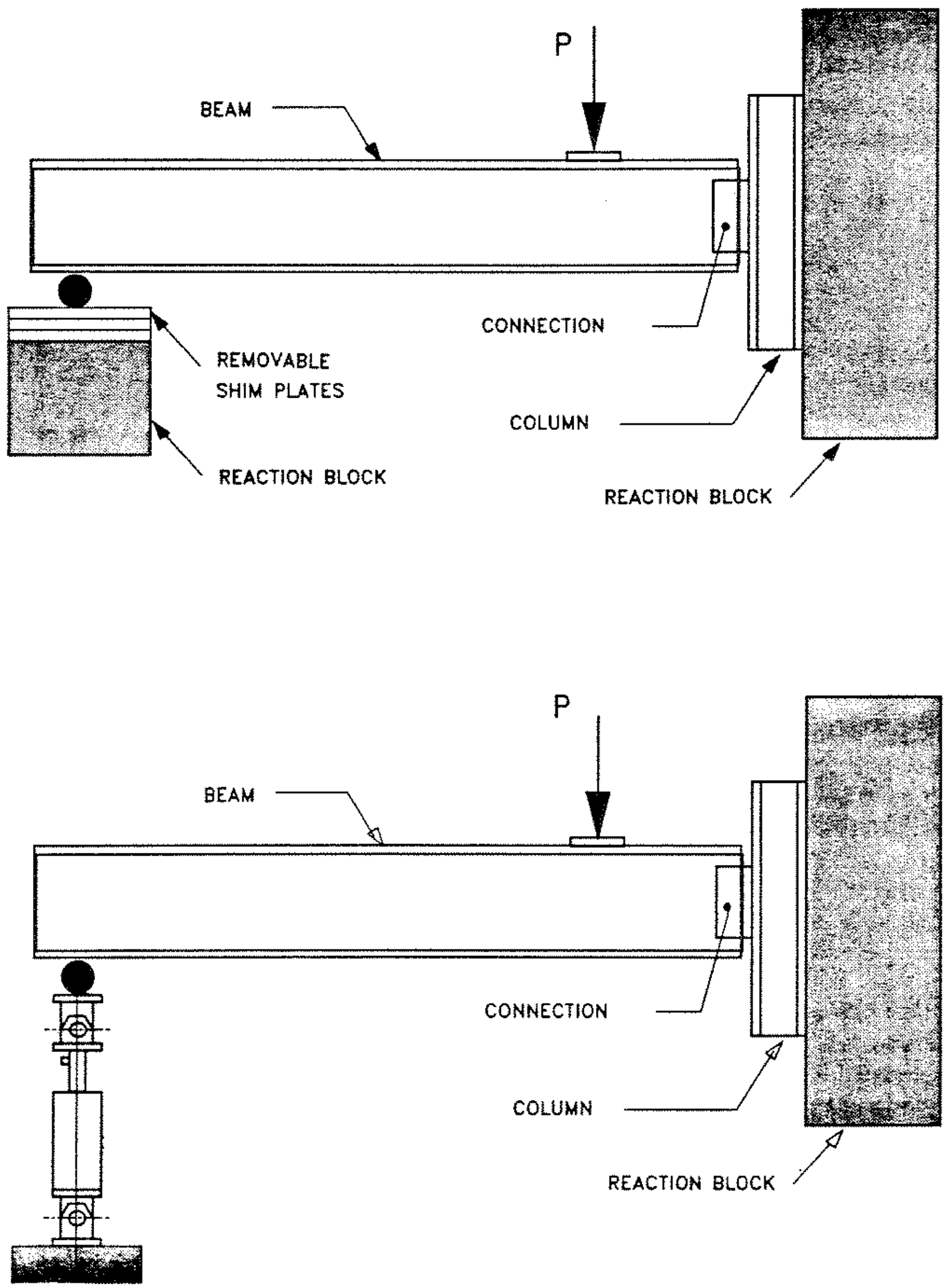


Figure 2.6. Shear-Rotation Test Set-up Used in Past

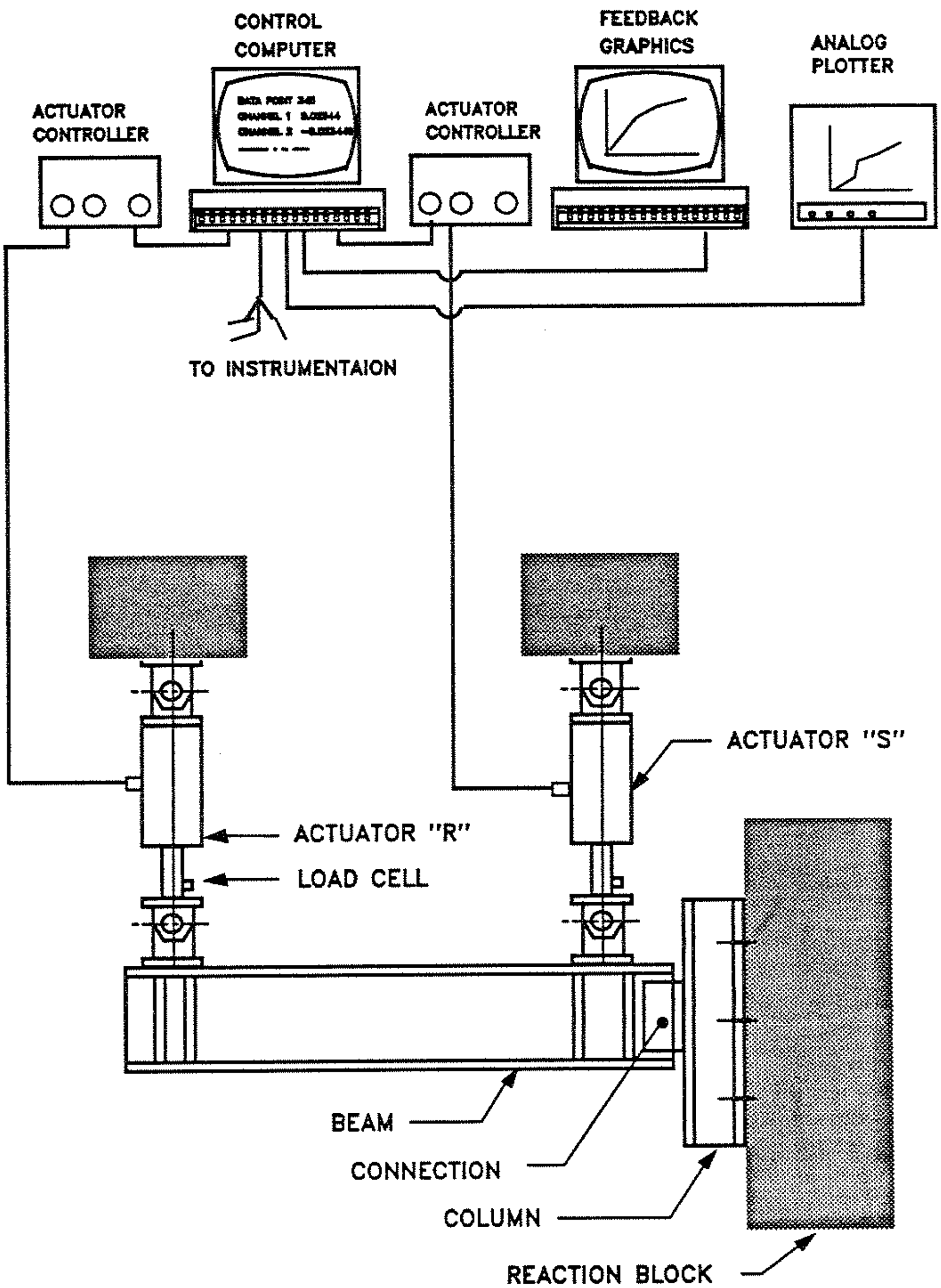


Figure 2.7. Test Set-up Used in the Investigation

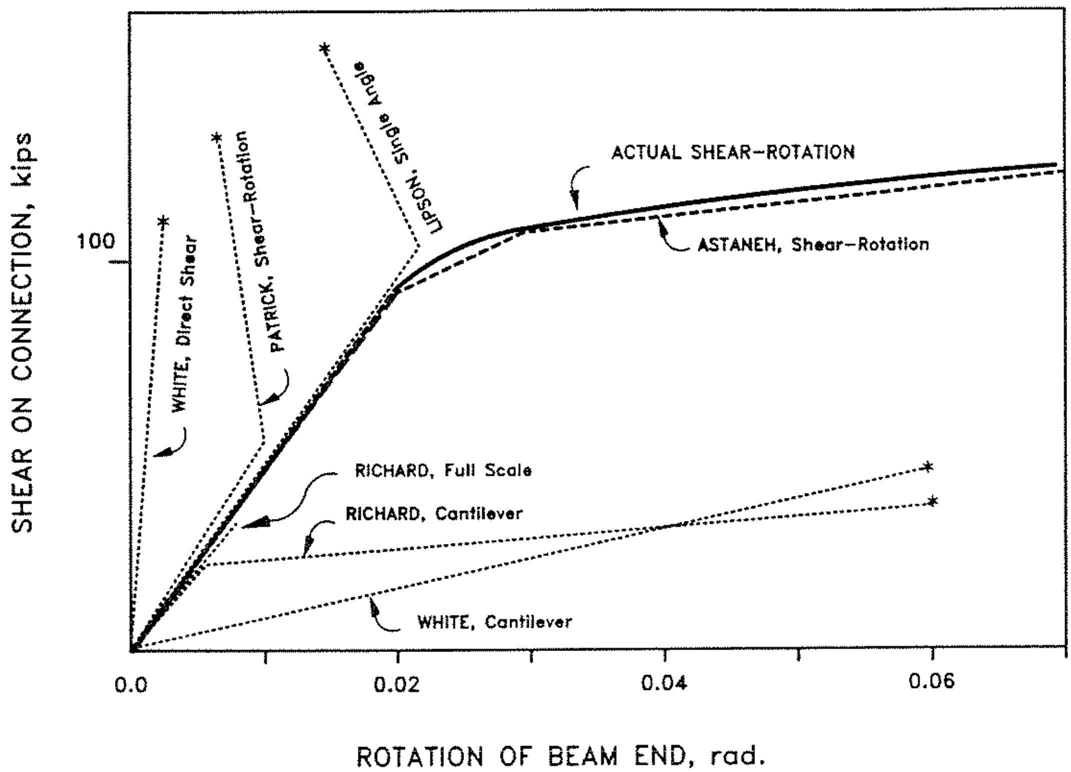


Figure 2.8. Shear-Rotation History of Test Set-ups

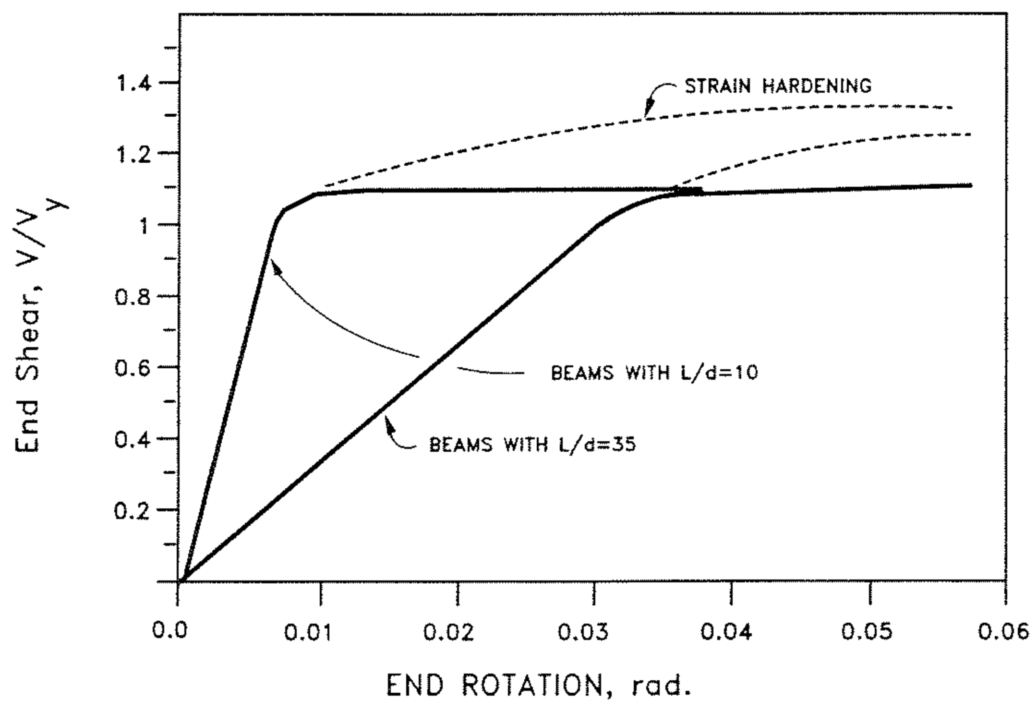


Figure 2.9. Shear-Rotation Curves of Typical Beam

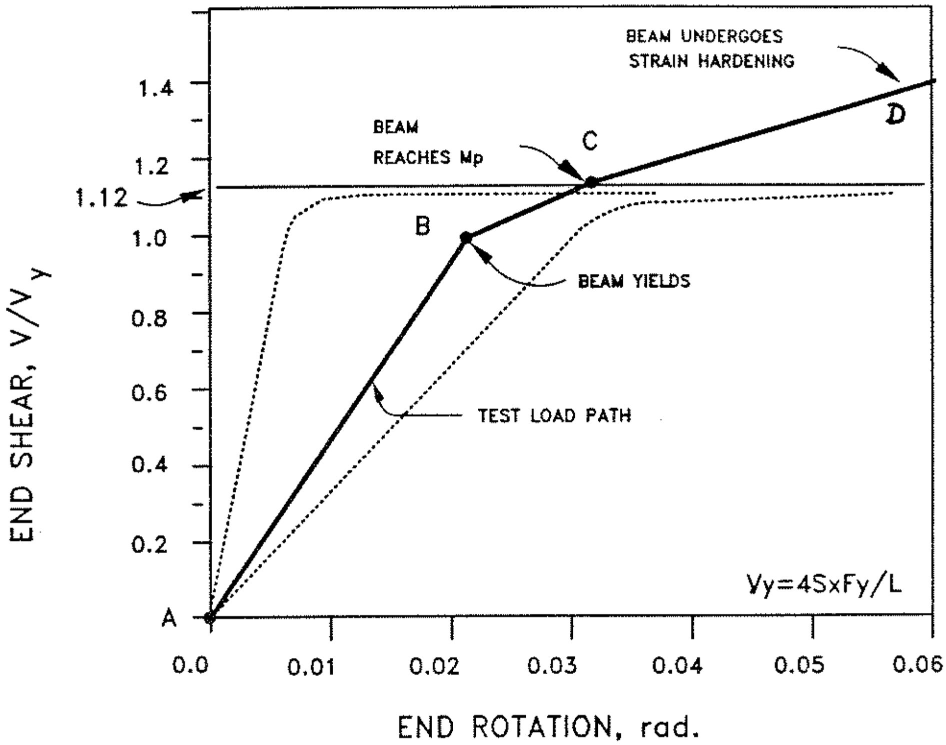


Figure 2.10. Shear-Rotation Curve Used for this Project

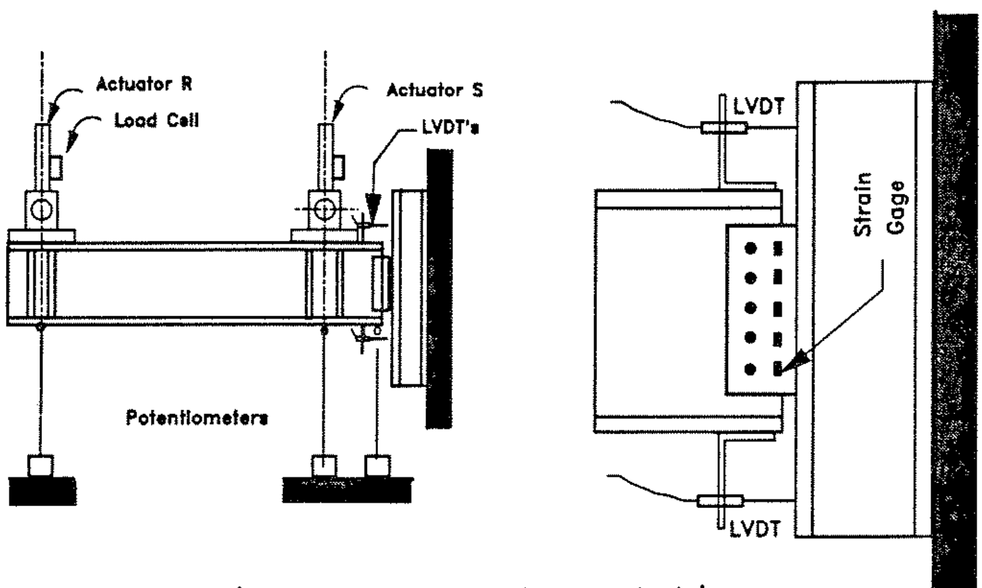


Figure 2.11. Instrumentation

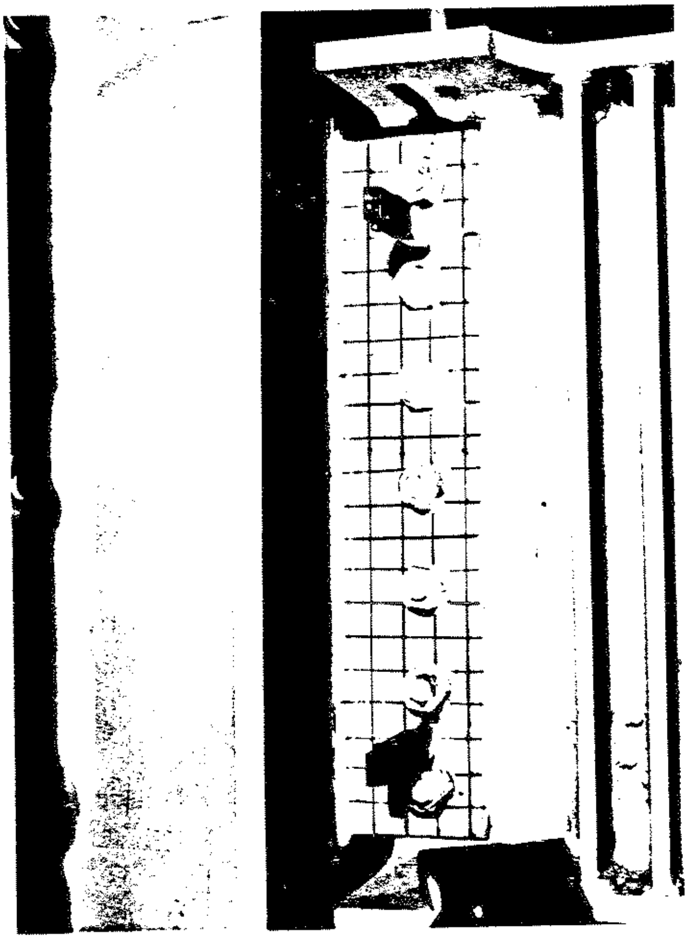


Figure 3.1. Initial Condition of Specimen One Before Testing

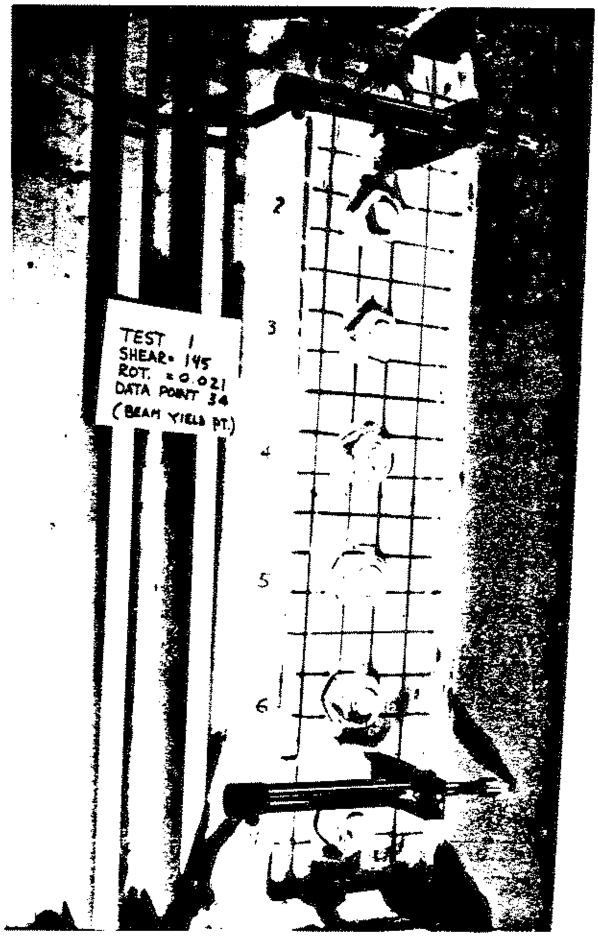


Figure 3.2. Specimen One During Testing

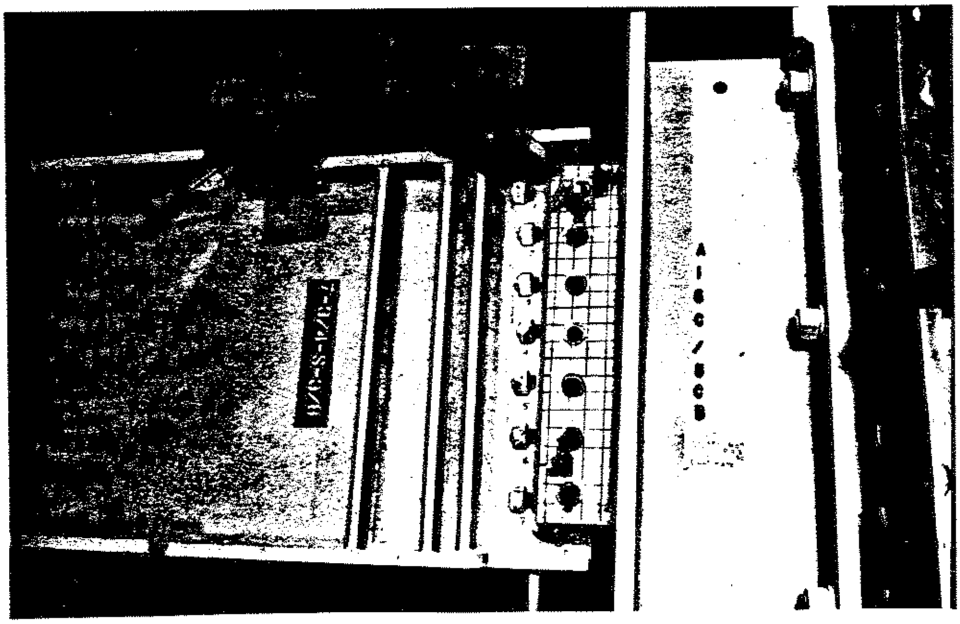


Figure 3.3. Specimen One After Failure of Bolts

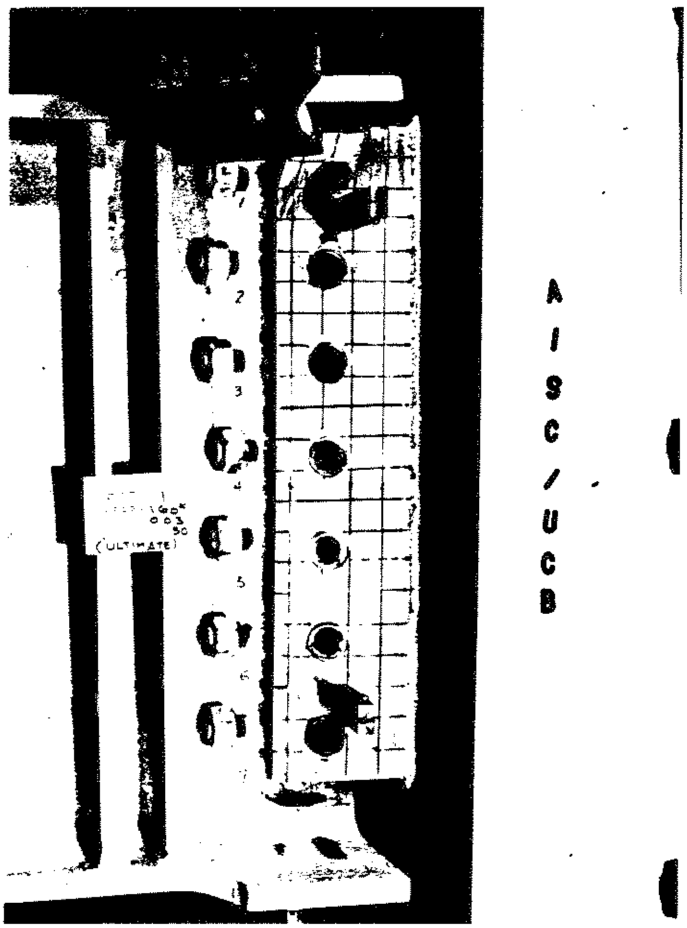


Figure 3.4. Close-up of Specimen One After Failure

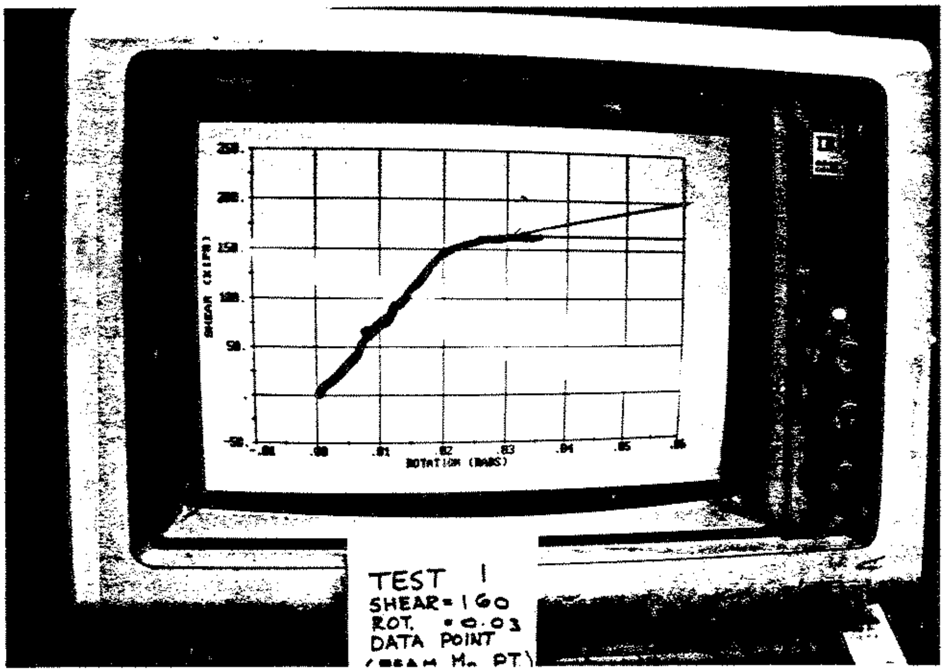


Figure 3.5. Load Path During Testing of Specimen One

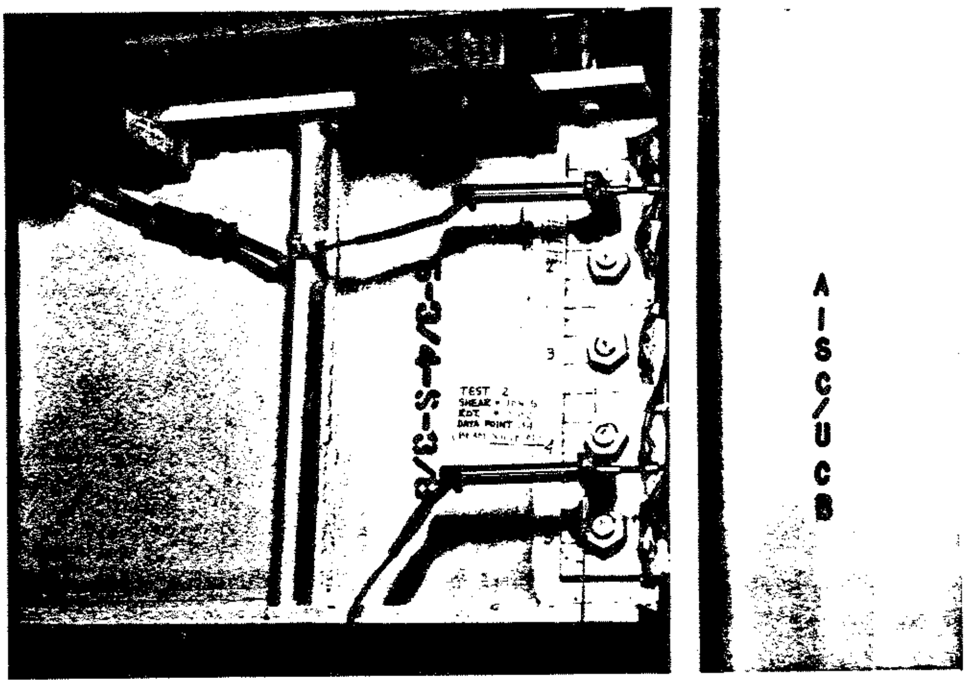


Figure 3.6. Specimen Two During Testing

Figure 3.7. Specimen Two During Testing

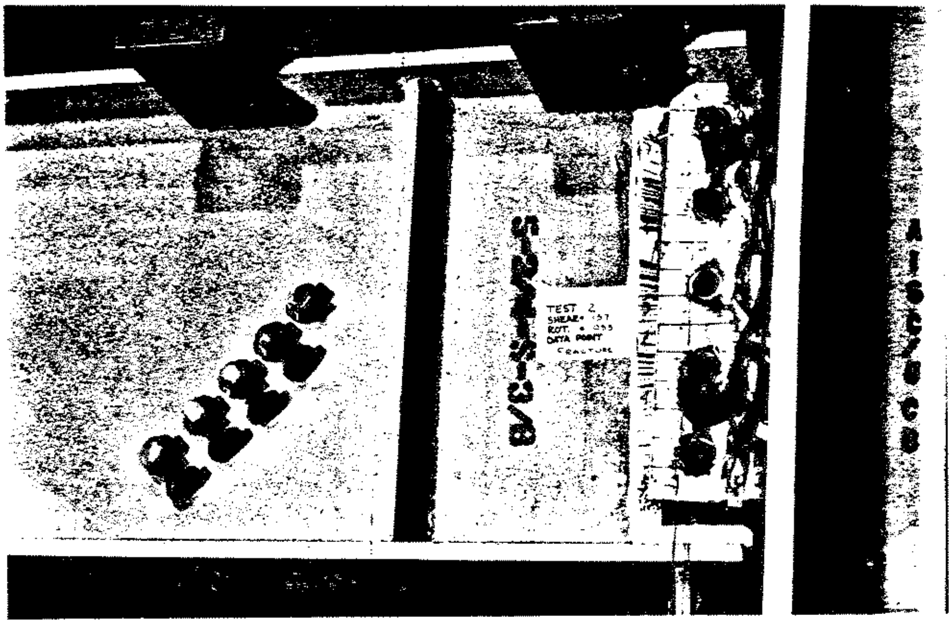


Figure 3.8. Specimen Two After Failure of Bolts

Figure 3.9. Single Plate in Specimen Two After Failure of Bolts

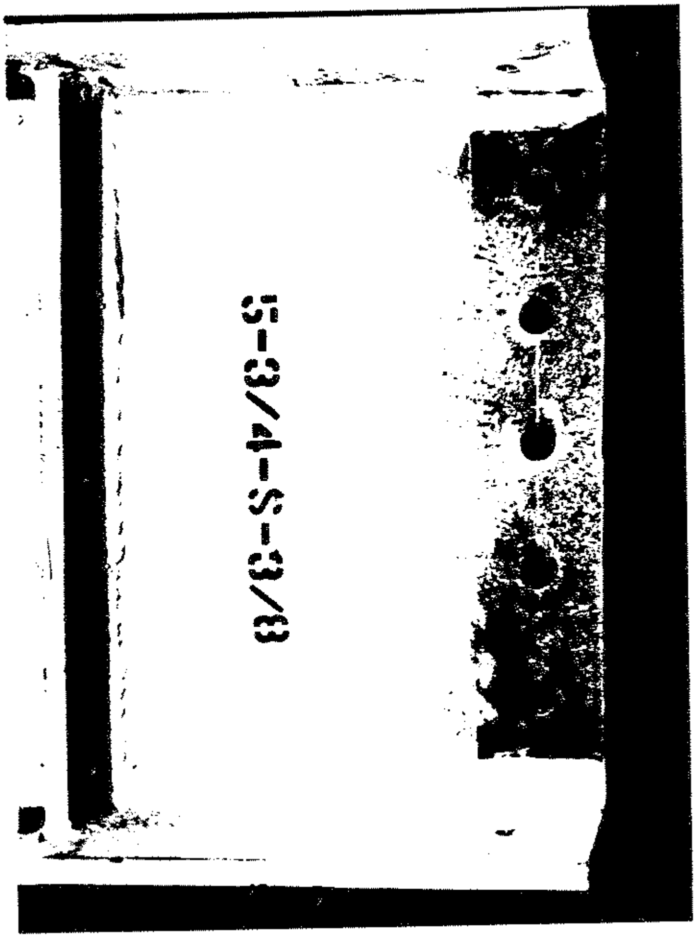


Figure 3.10. Beam End in Specimen Two After Failure of Bolts

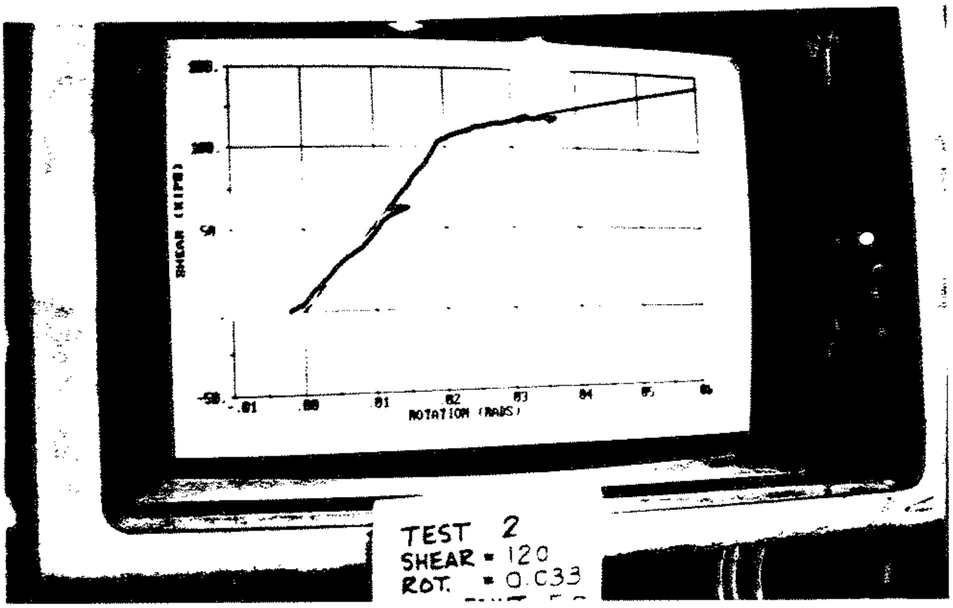


Figure 3.11. Load Path Used During Test Two

Figure 3.12. Specimen Three During Testing

Figure 3.13. Deformation and Fracture of Bolt

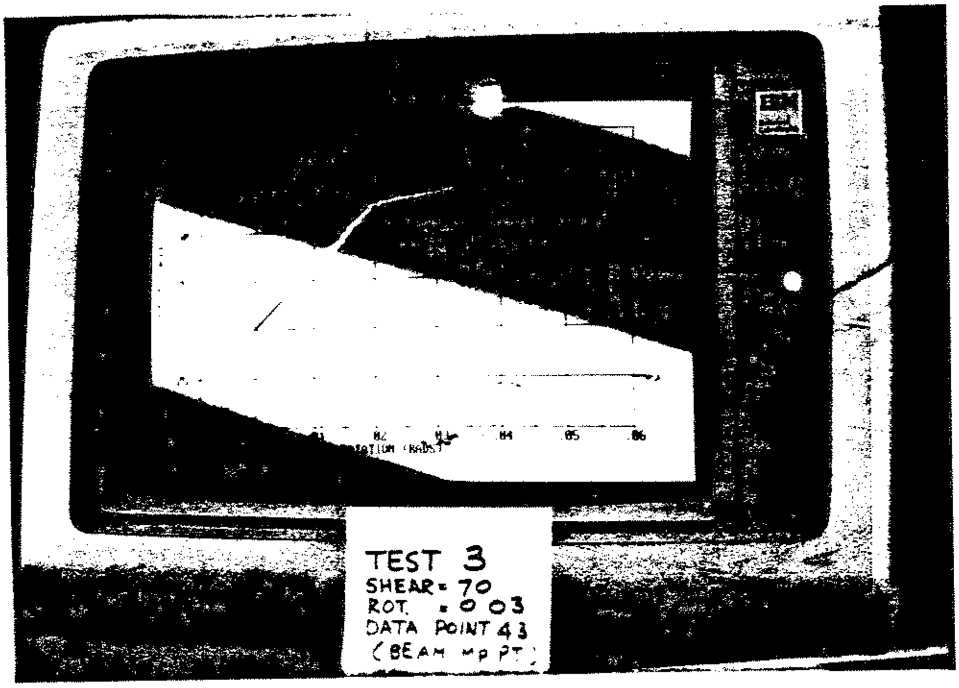


Figure 3.14. Load Path Used During Test Three

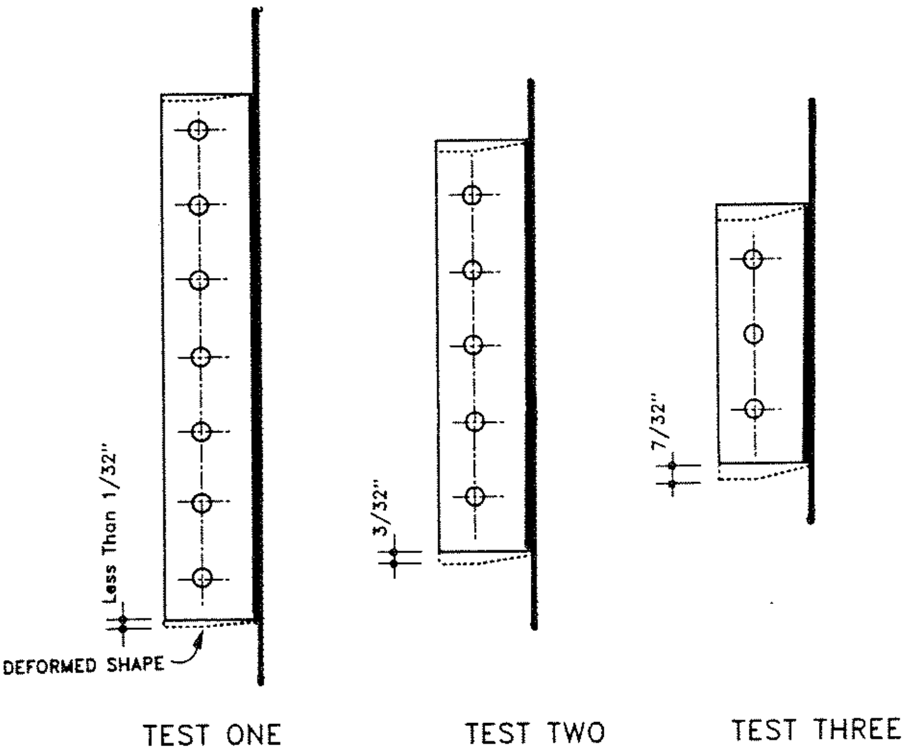


Figure 4.1. Shear Deformation of Plates at Conclusion of Tests

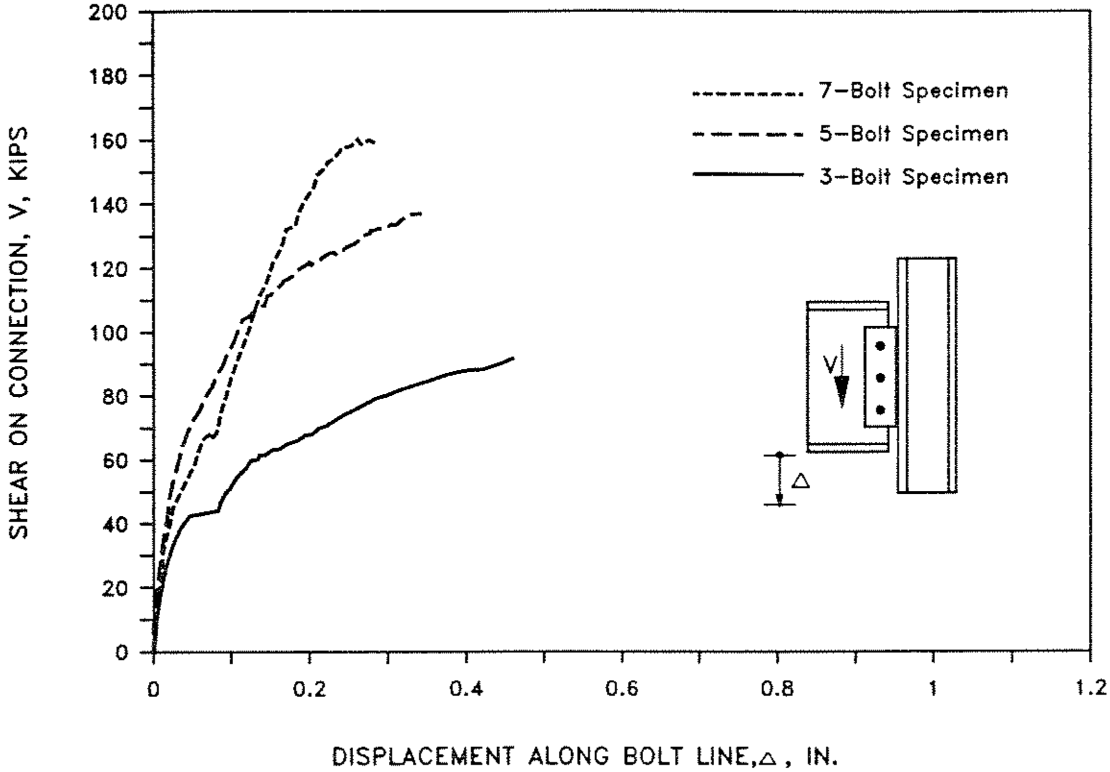


Figure 4.2. Shear vs. Vertical Displacement of Beam Along Boltline

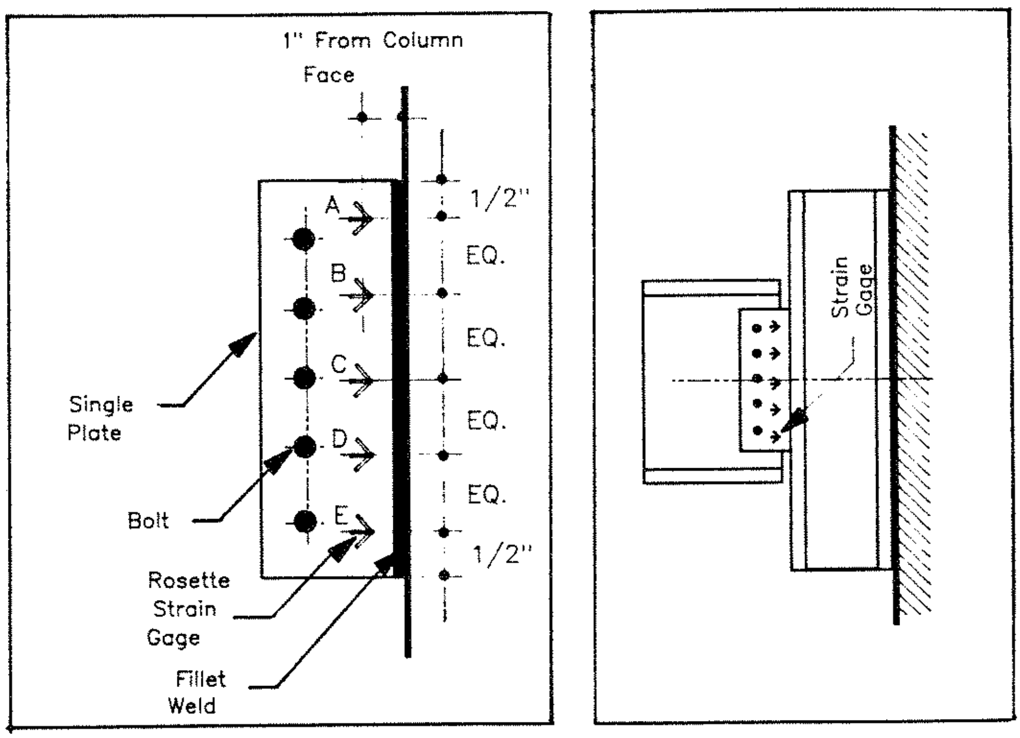
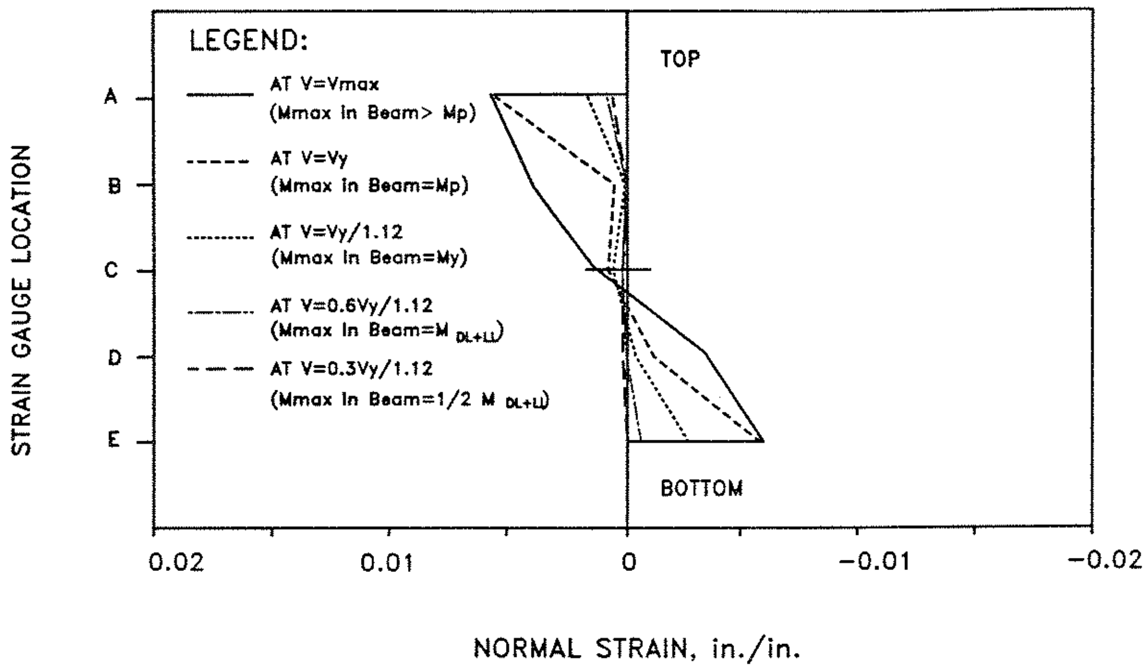
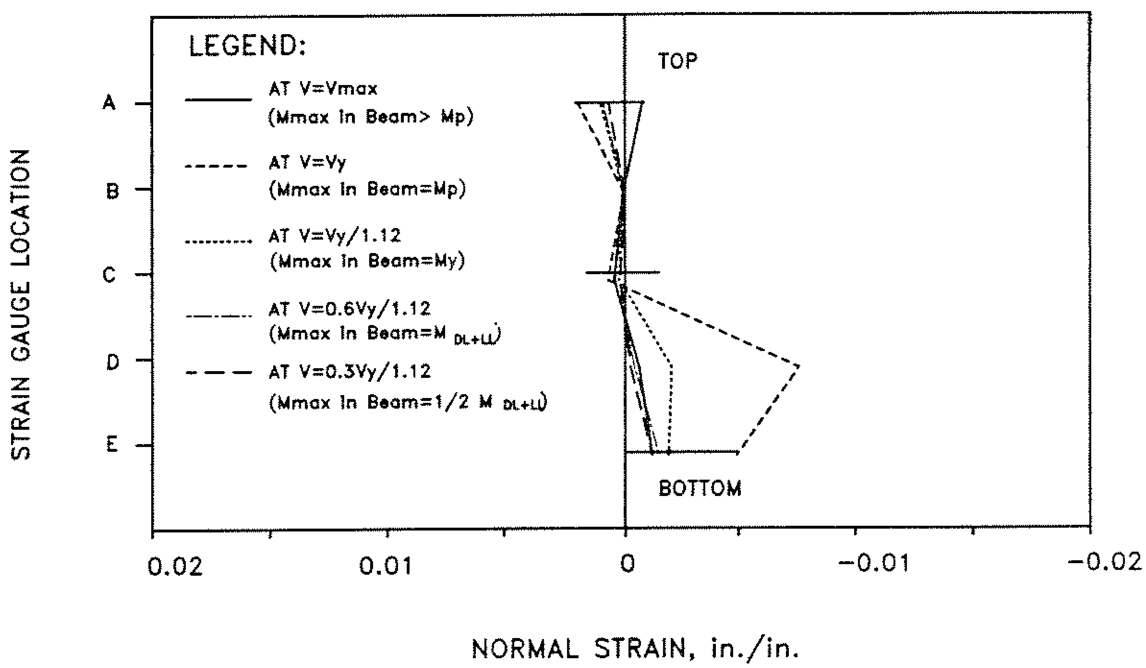


Figure 4.3. Location of Strain Gauges



(a)



(b)

Figure 4.4. Distribution of Normal Strains

a) Specimen 2

b) Specimen 3

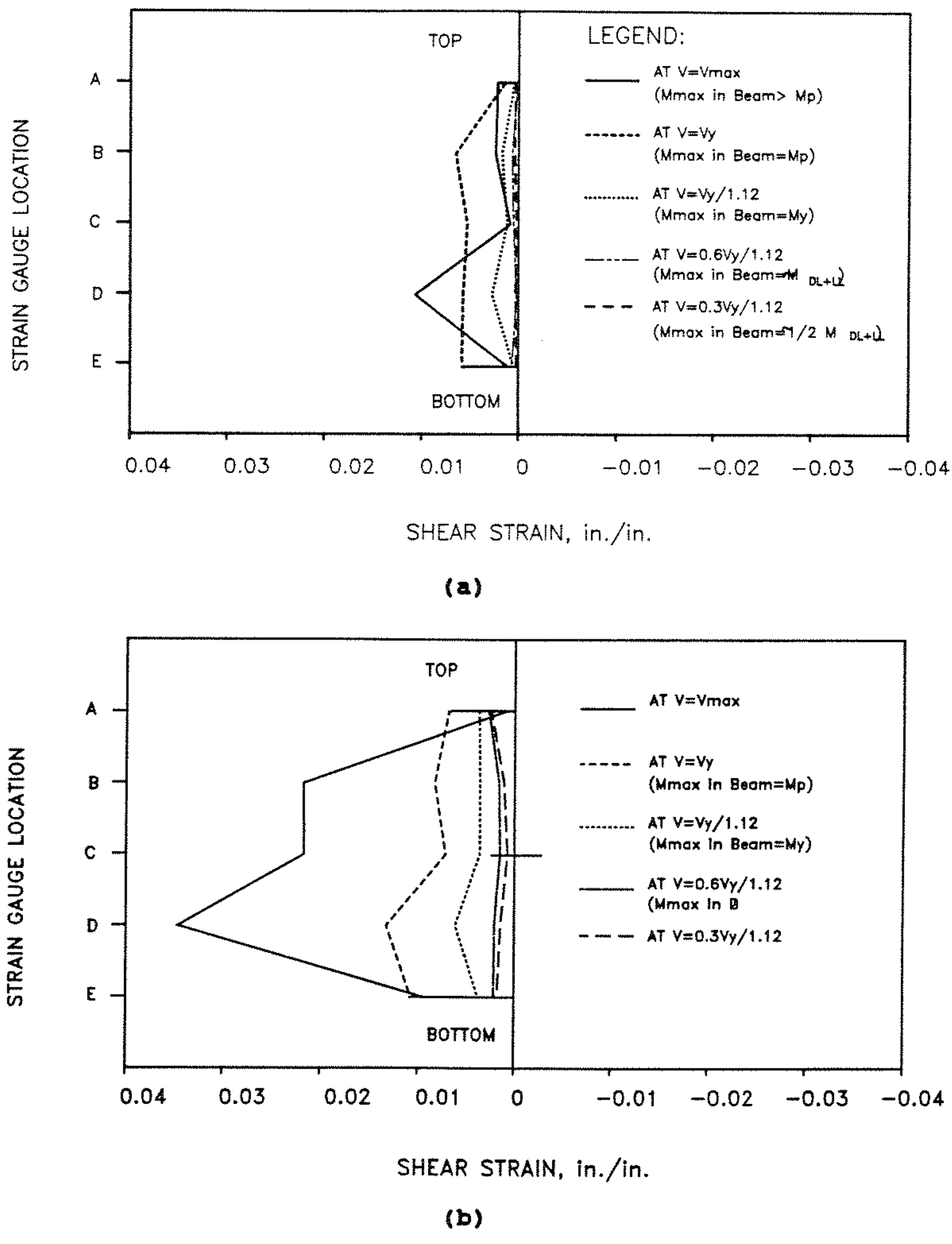


Figure 4.5. Distribution of Shear Strain

a) Specimen 2

b) Specimen 3

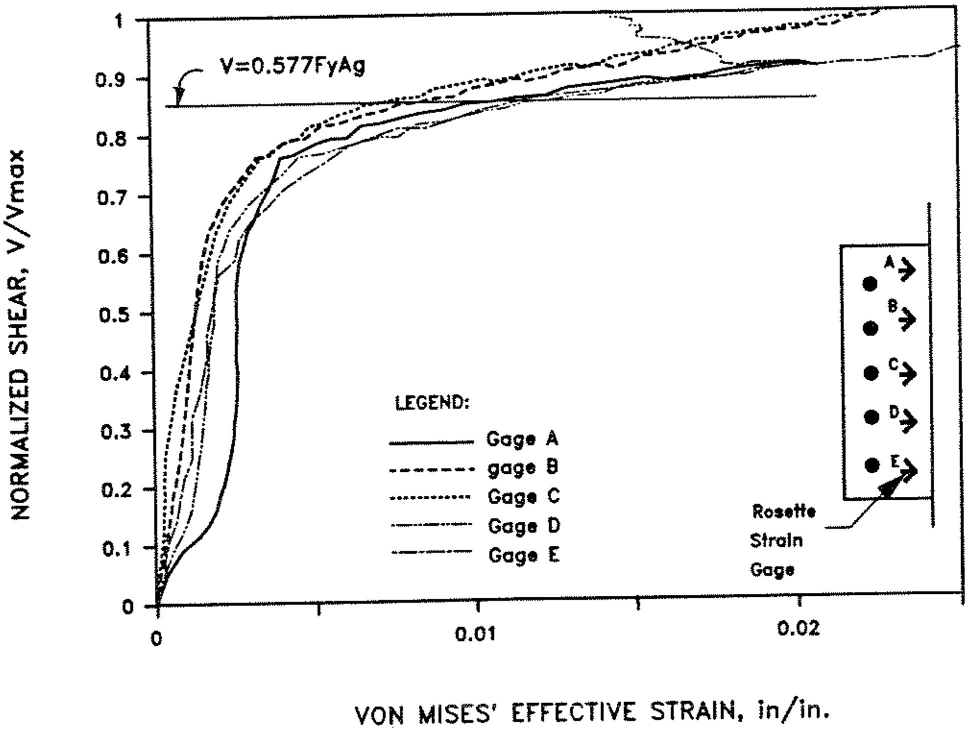


Figure 4.6. Distribution of Effective Strain for Test 2

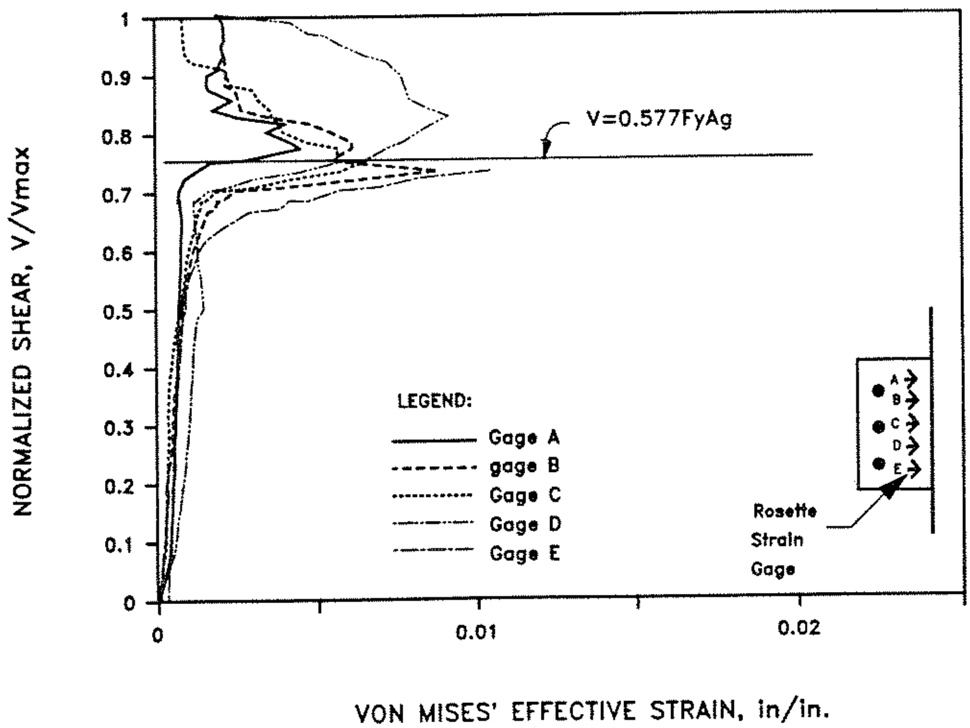


Figure 4.7. Distribution of Effective Strain for Test 3

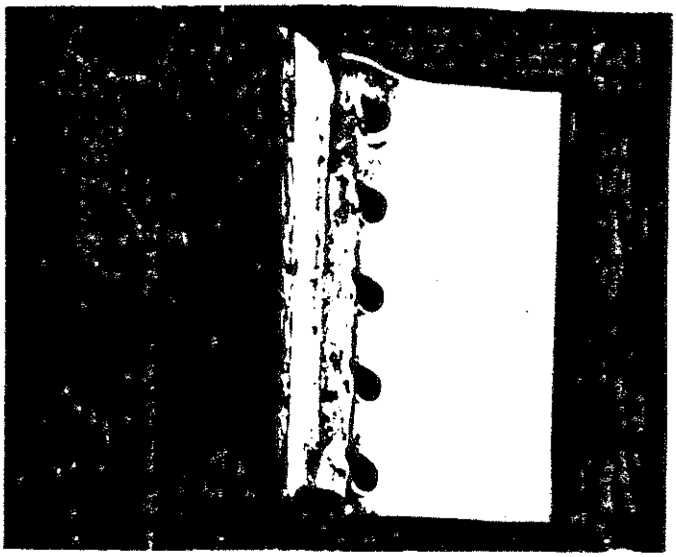


Figure 4.8. Failure of Net Section of Stem of A Tee Connection (Ref. 3)

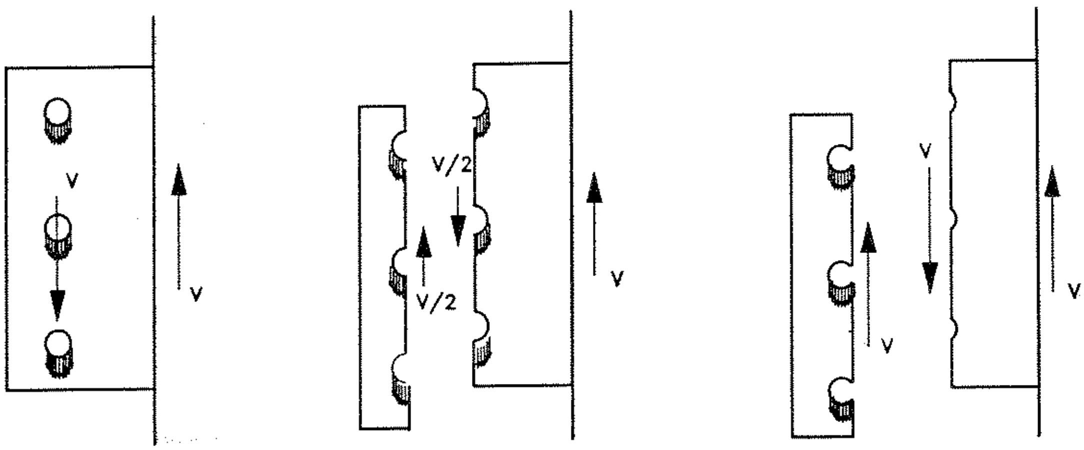


Figure 4.9. Distribution of Stress Around Bolt Hole

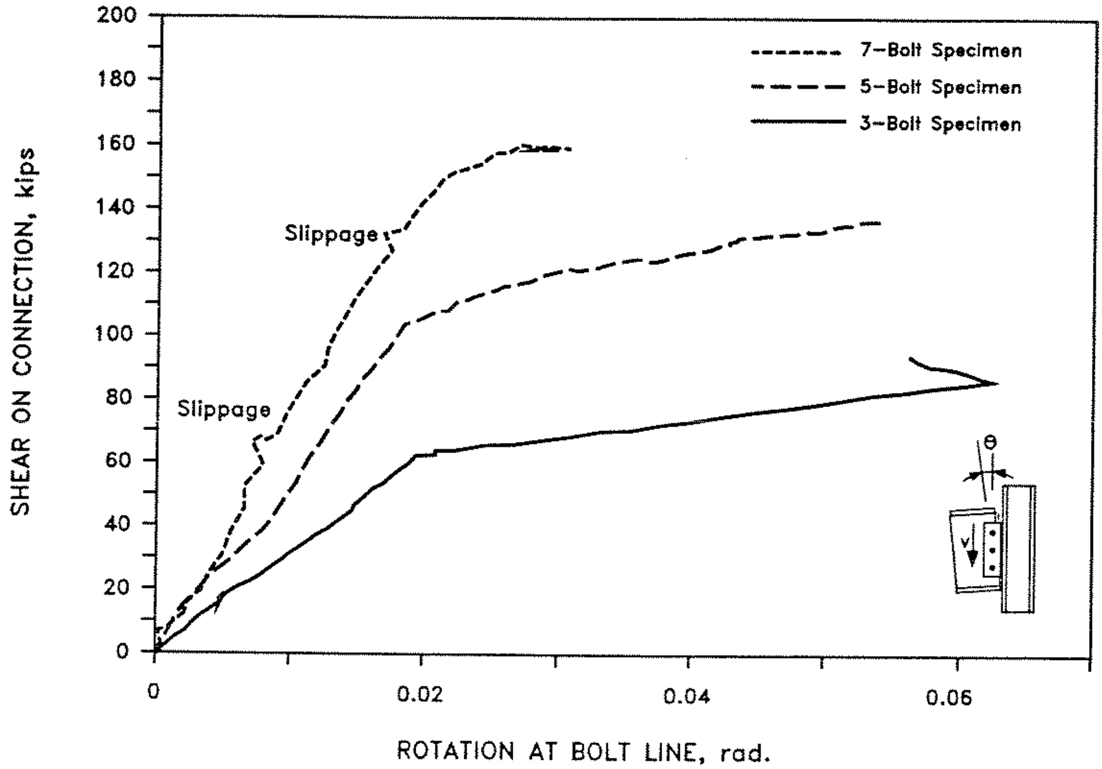
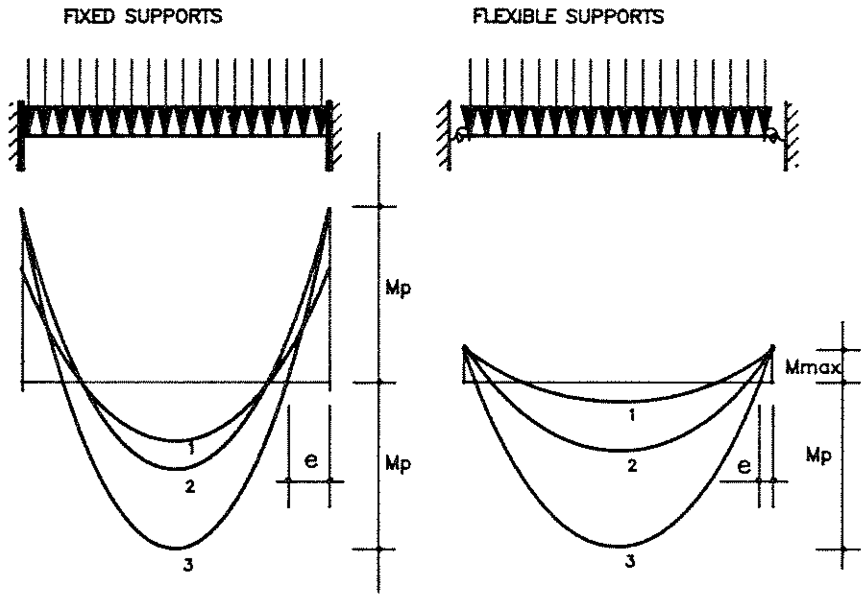
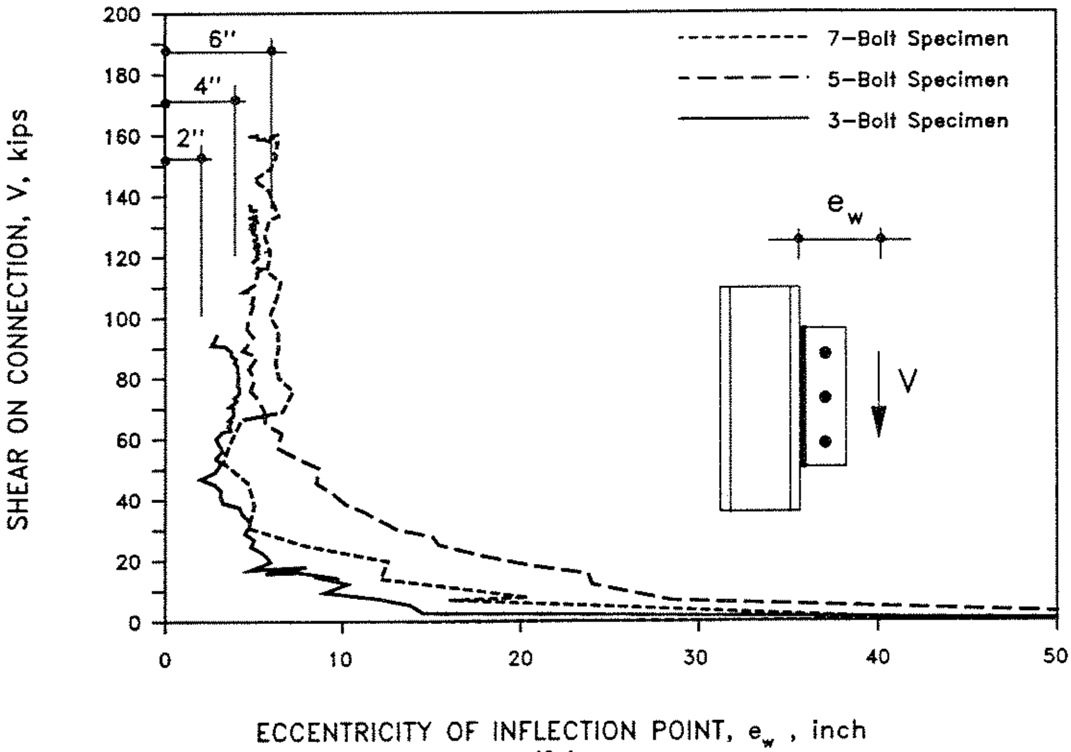


Figure 4.10. Shear-Rotation Relationship During Tests



(a)



(b)

Figure 4.11. Location of Point of Inflection
a) Beam Supported by Rigid and Flexible Connections

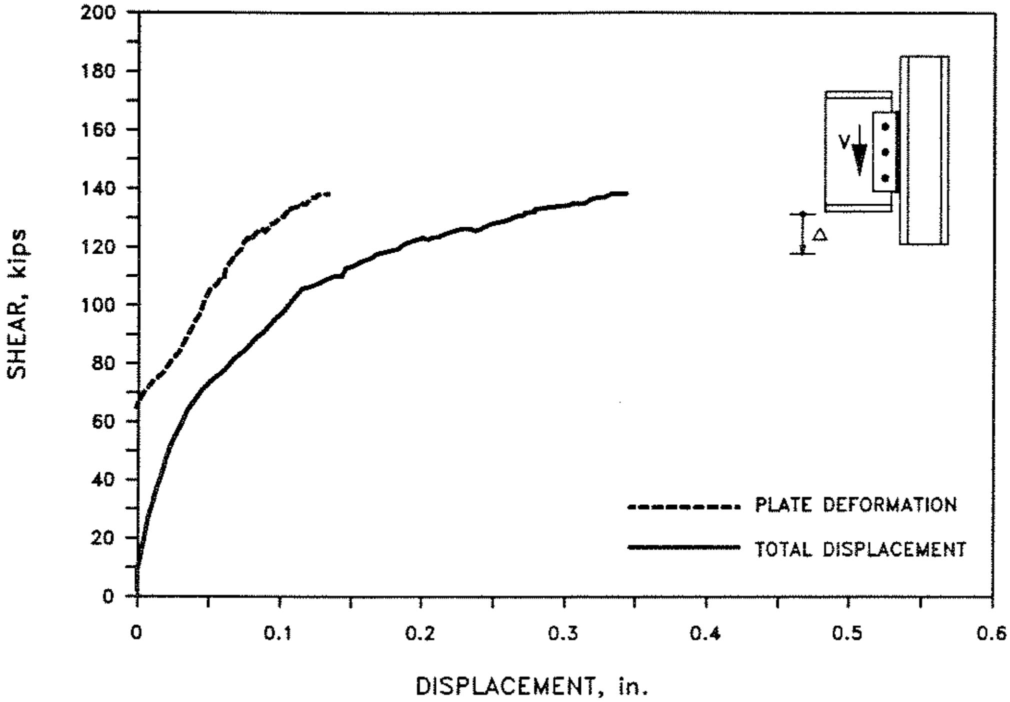


Figure 4.12. Bolt Shear vs. Boltline Displacement for Specimen 2

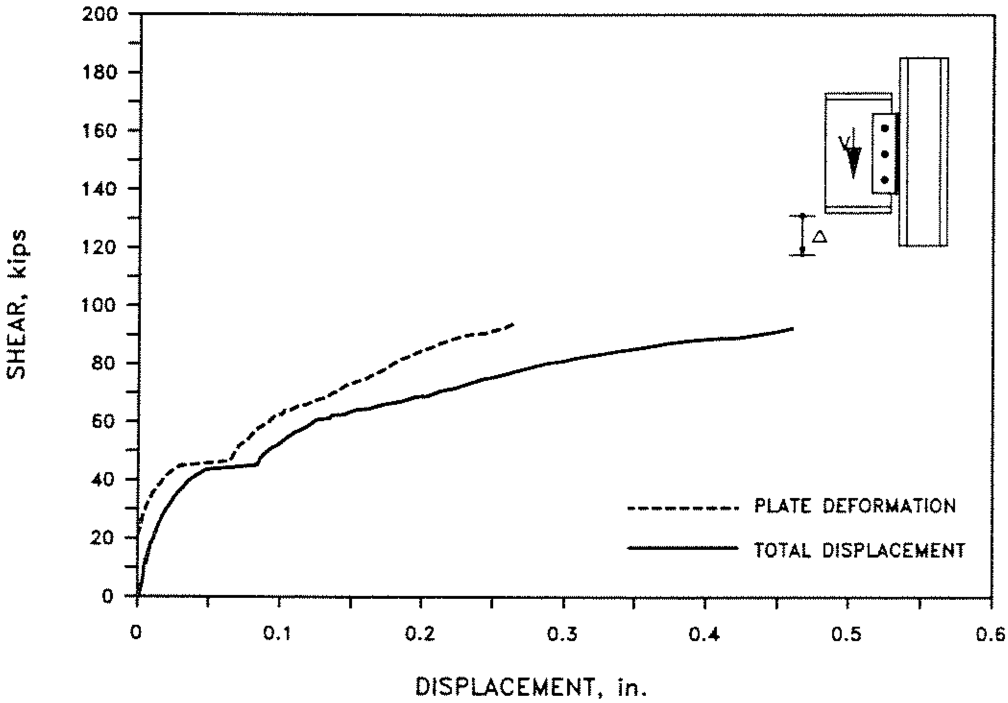


Figure 4.13. Bolt Shear vs. Boltline Displacement for Specimen 3

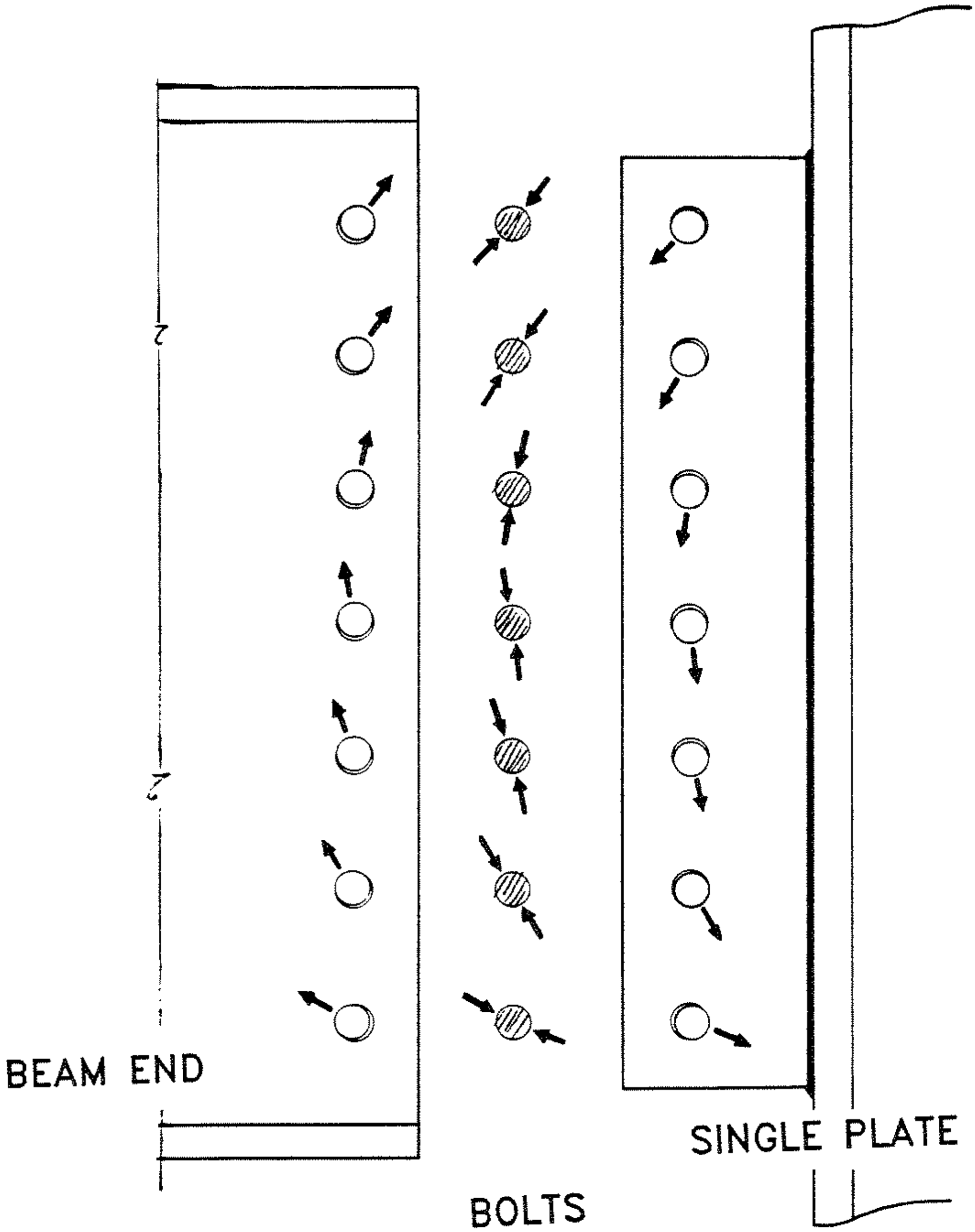


Figure 4.14(a) Bolt Hole Deformations in Specimen 1

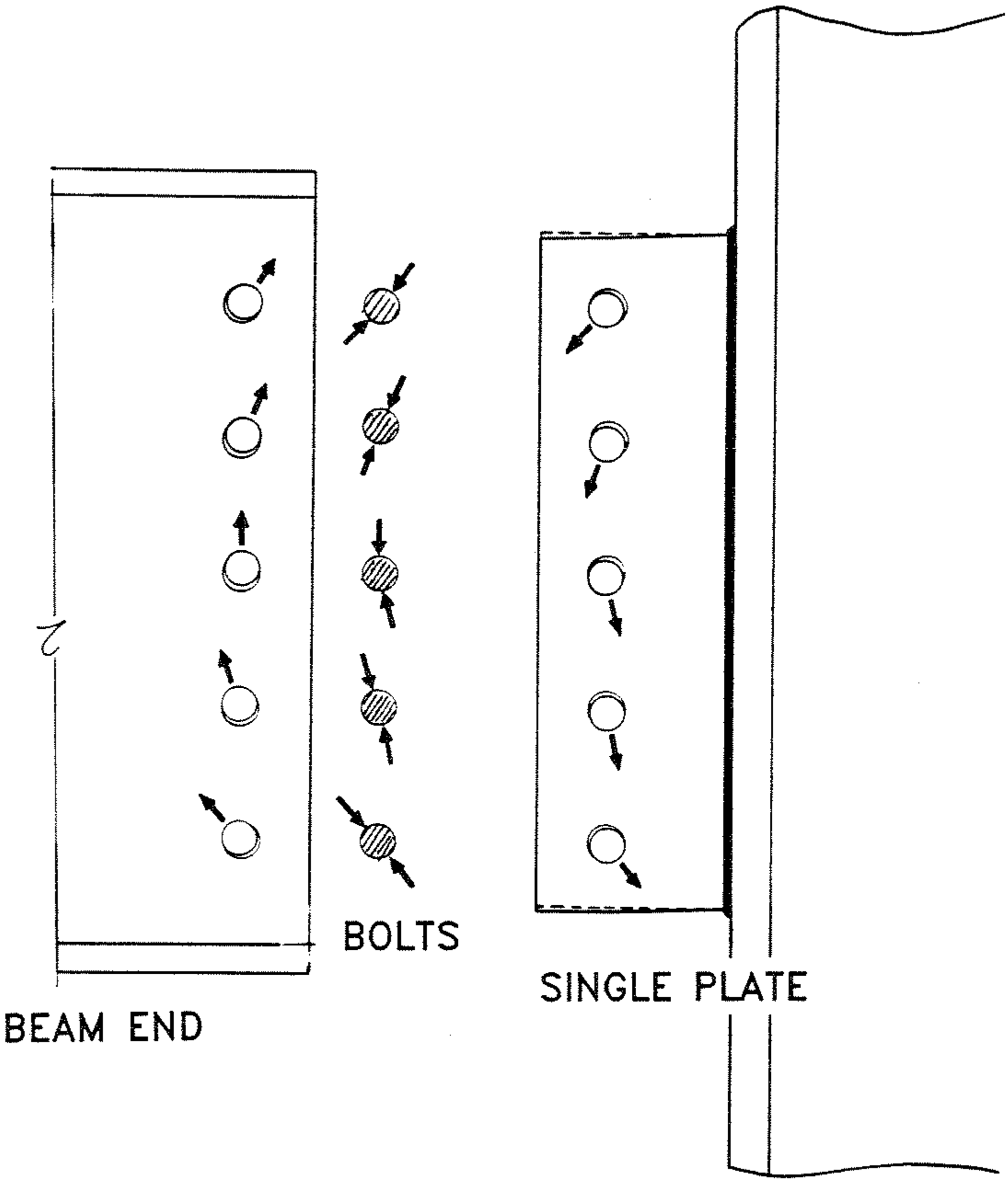


Figure 4.14(b) Bolt Hole Deformations in Specimen 2

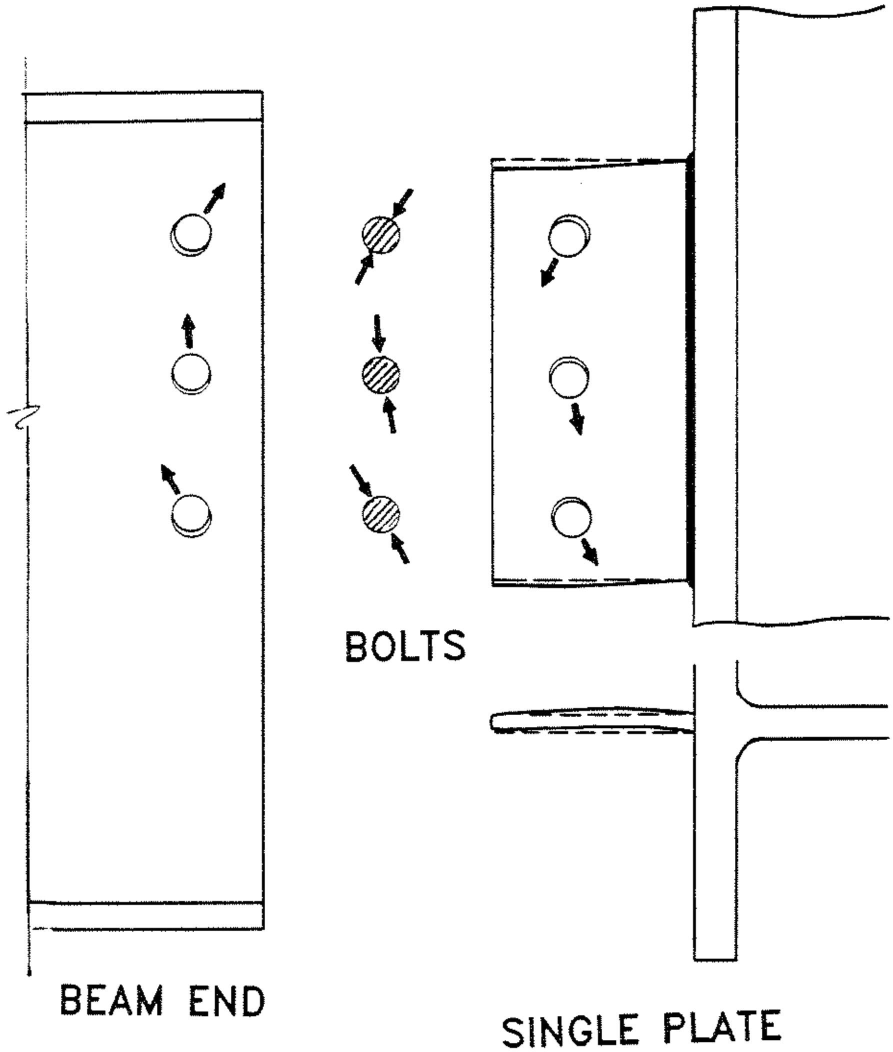


Figure 4.14(c) Bolt Hole Deformations in Specimen 3

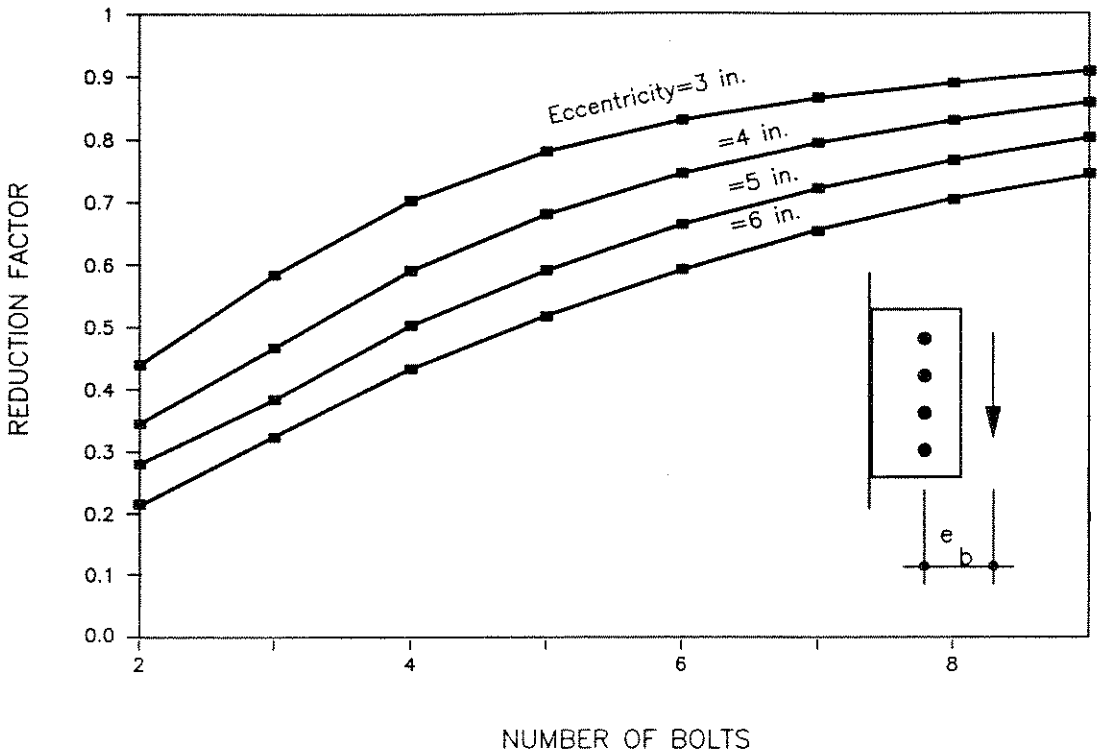


Figure 4.15. Efficiency of Bolt Groups in Eccentric Shear

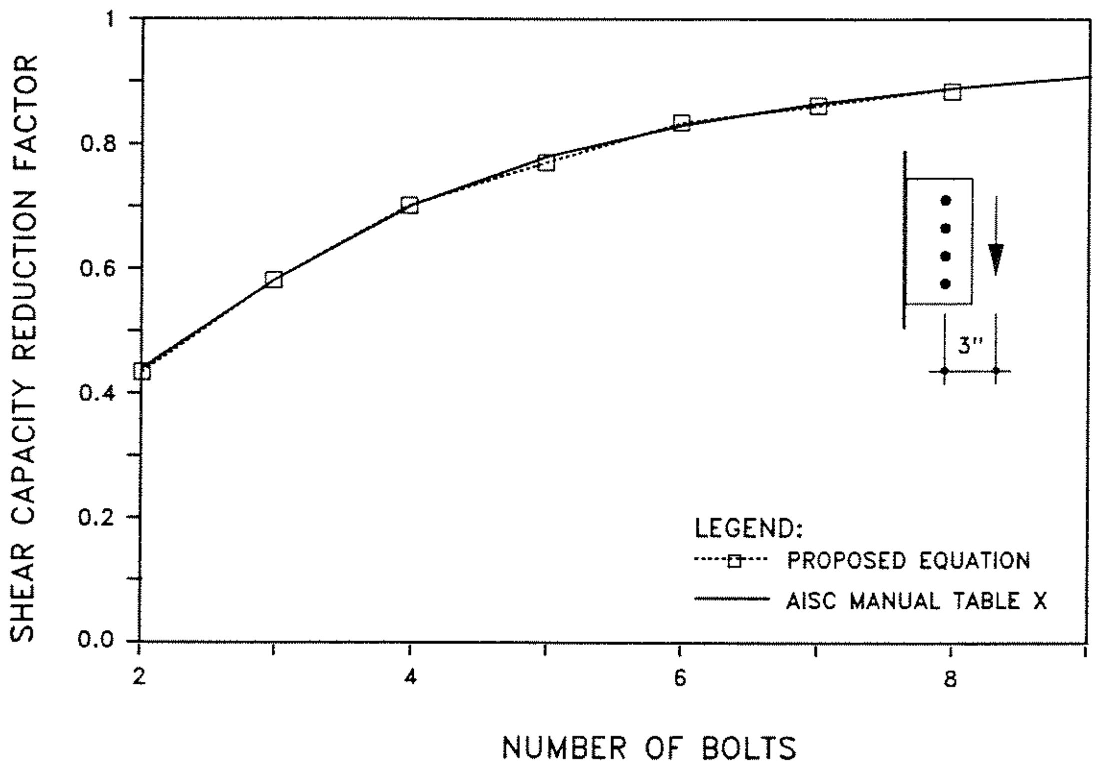


Figure 4.16. Approximation of Coefficient C in Table X of the AISC Manual

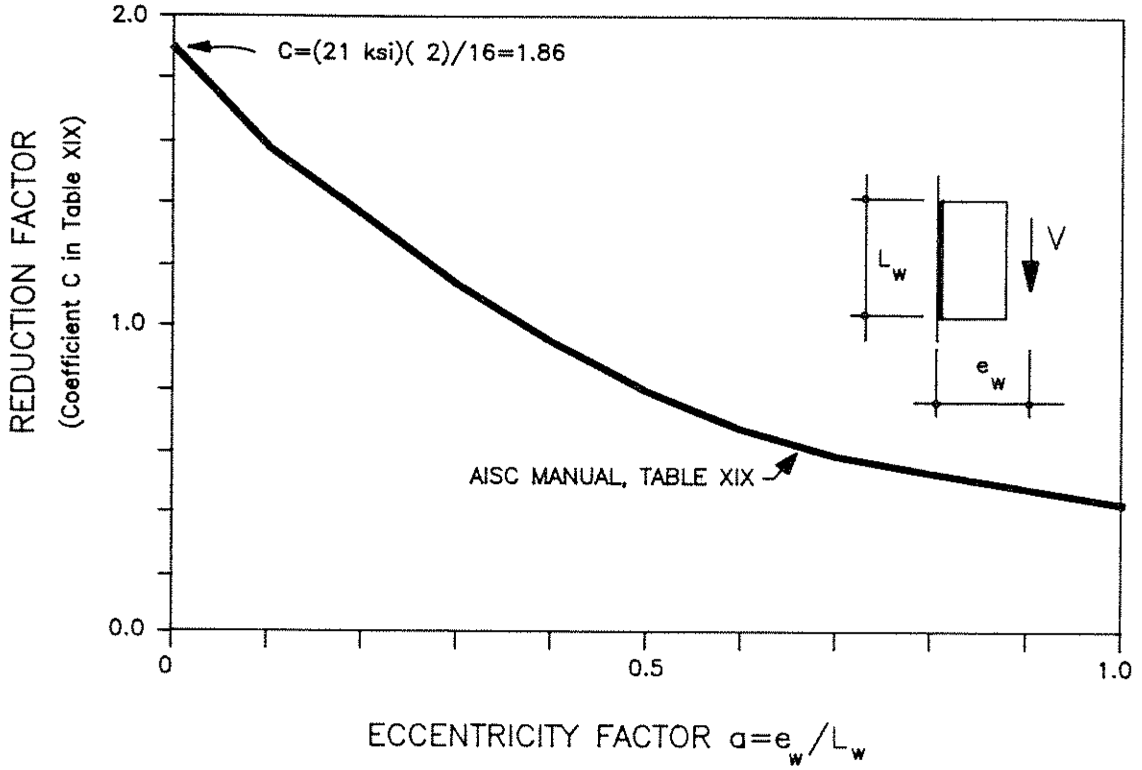


Figure 4.17. Efficiency of Weld Line in Eccentric Shear

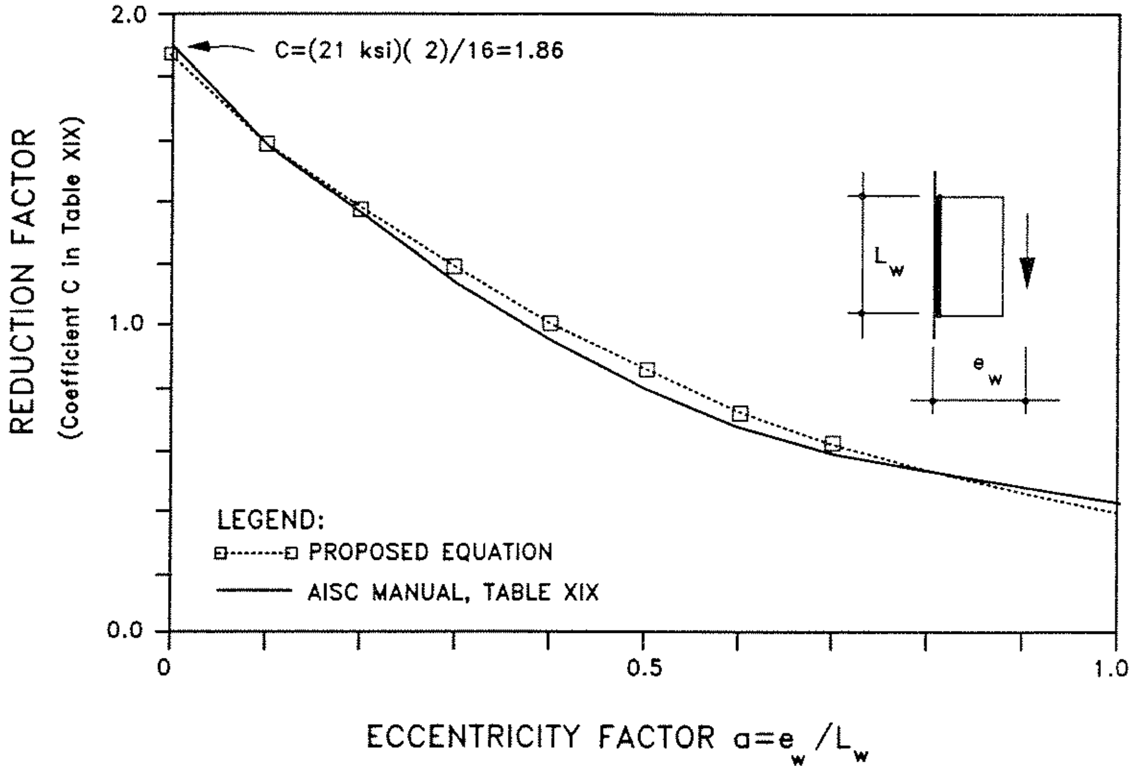


Figure 4.18. Approximation of Coefficient C in Table XIX of the AISC Manual

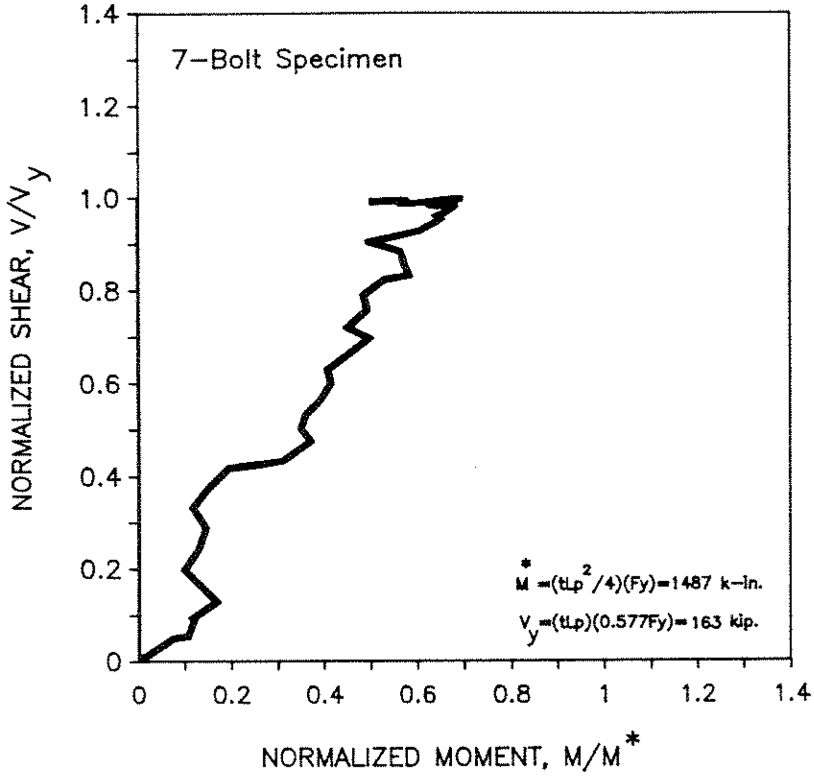


Figure 4.19. Shear vs. Moment for Seven Bolt Specimen

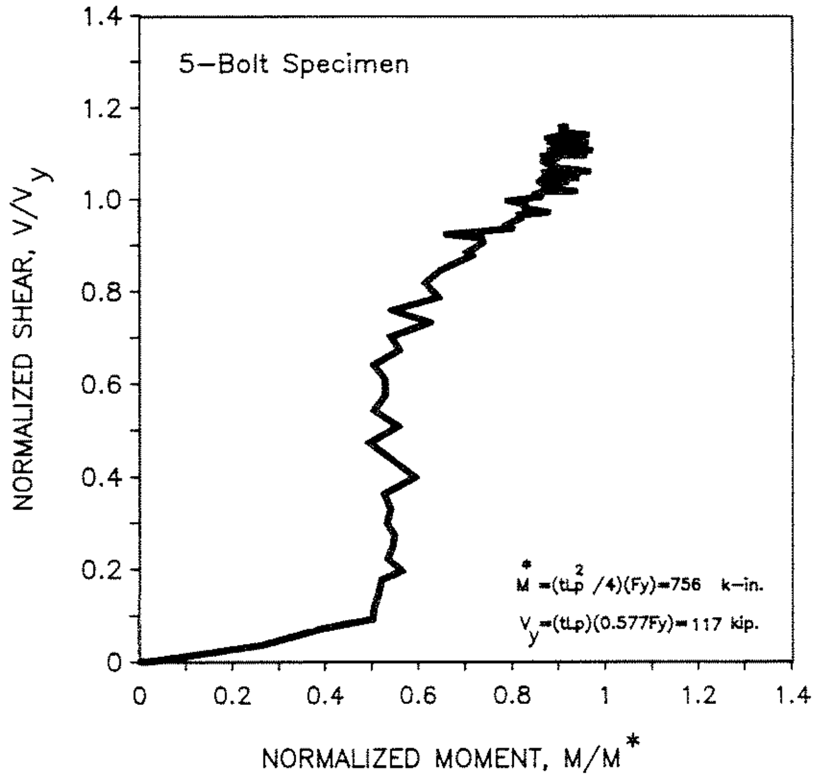


Figure 4.20. Shear vs. Moment for Five Bolt Specimen

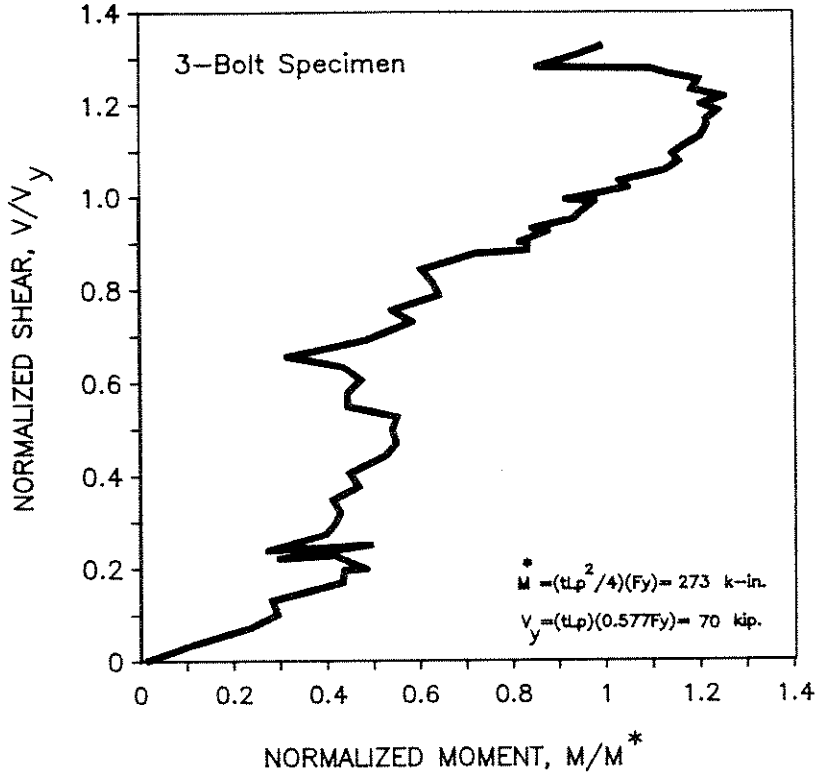


Figure 4.21. Shear vs. Moment for Three Bolt Specimen

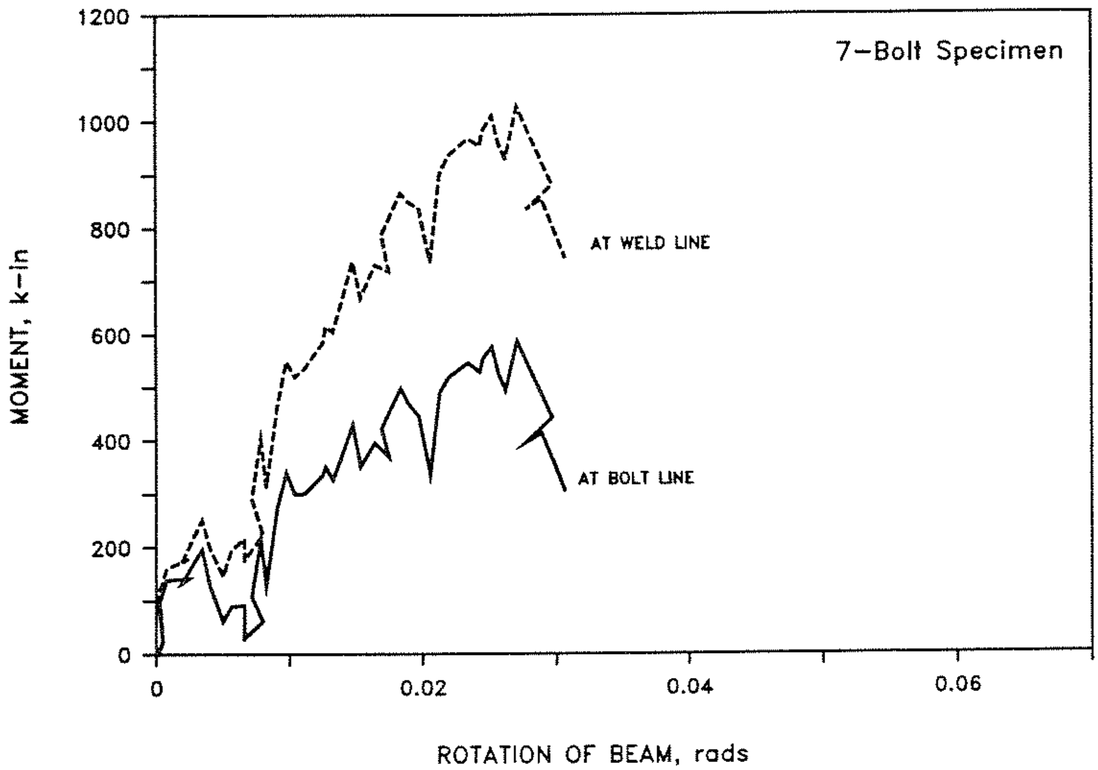


Figure 4.22. Moment vs. Rotation for Seven Bolt Specimen

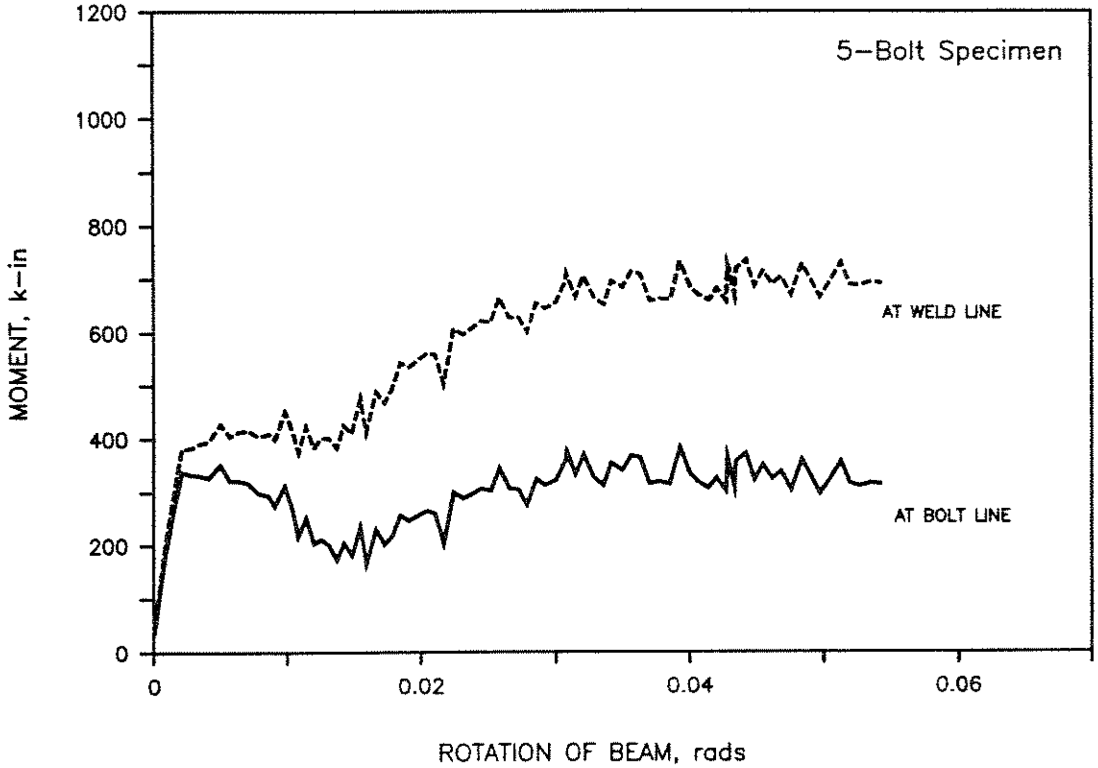


Figure 4.23. Moment vs. Rotation for Five Bolt Specimen

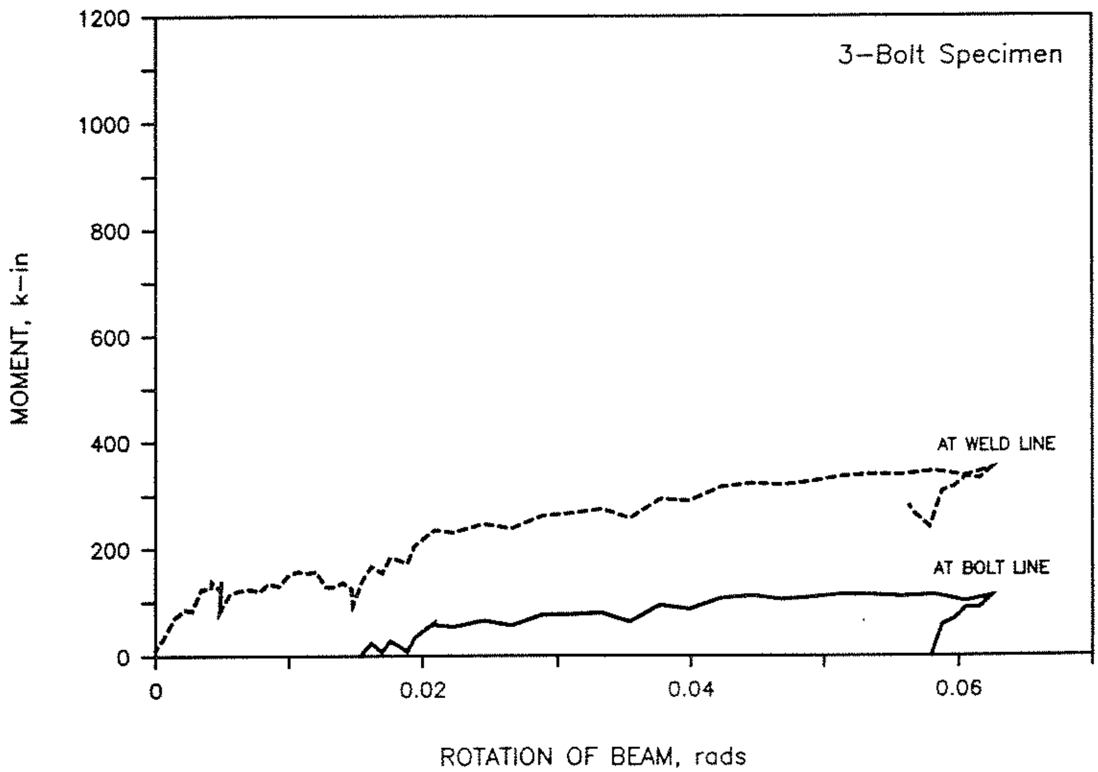


Figure 4.24. Moment vs. Rotation for Three Bolt Specimen

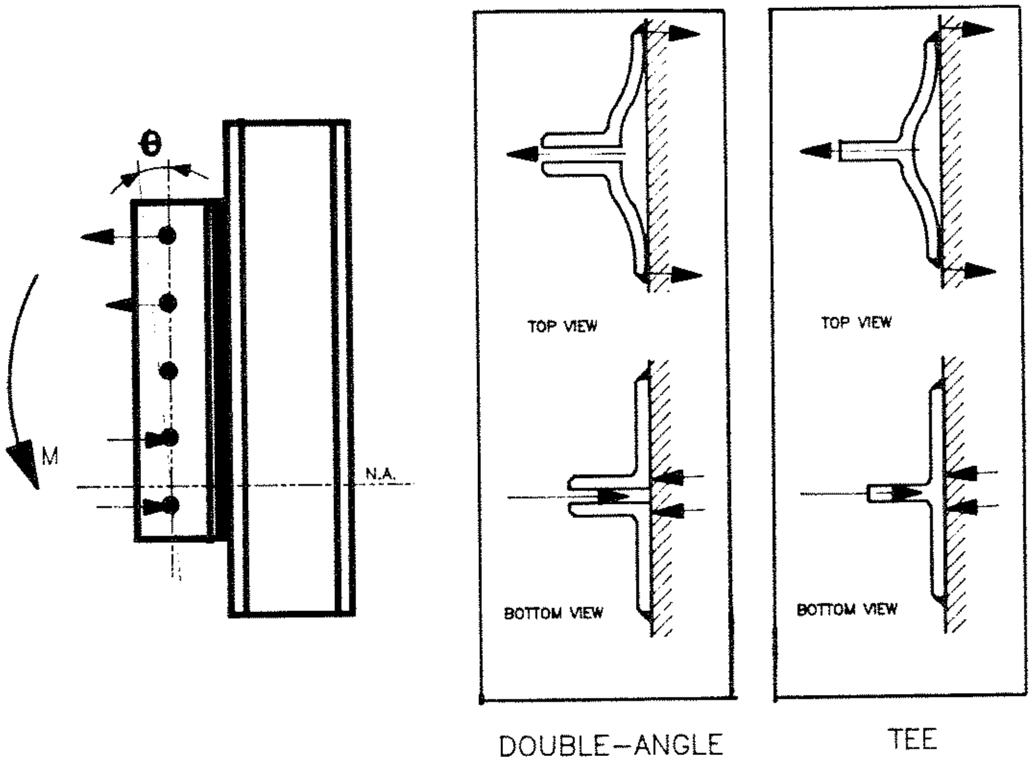


Figure 4.25. Double-Angle and Tee Framing Connections

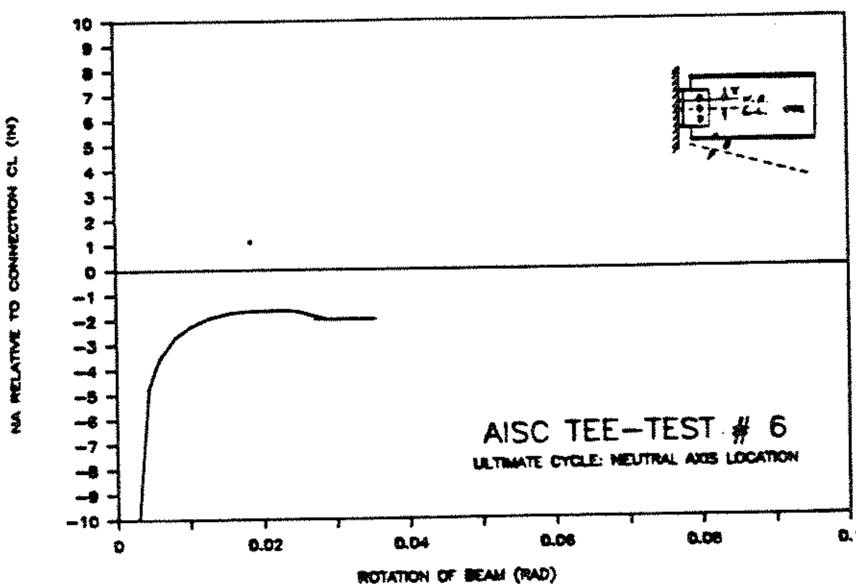


Figure 4.26. Movement of Neutral Axis During Testing of Tee Framing Connection, (Ref. 3)

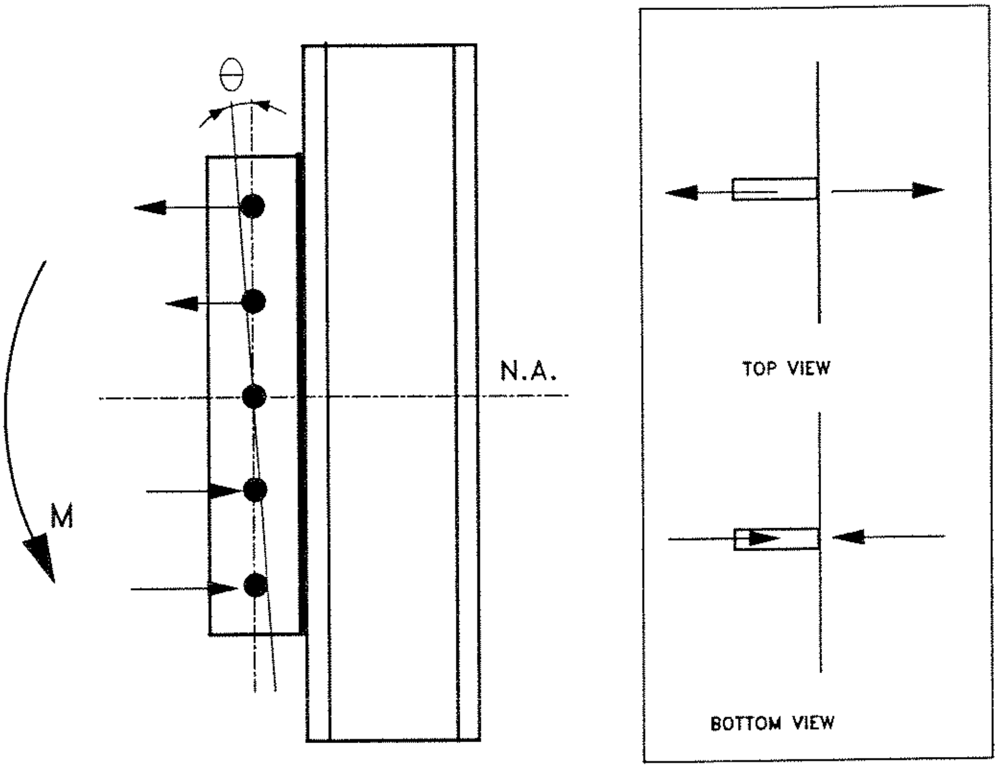


Figure 4.27. Elements of Single Plate Under Tension and Compression

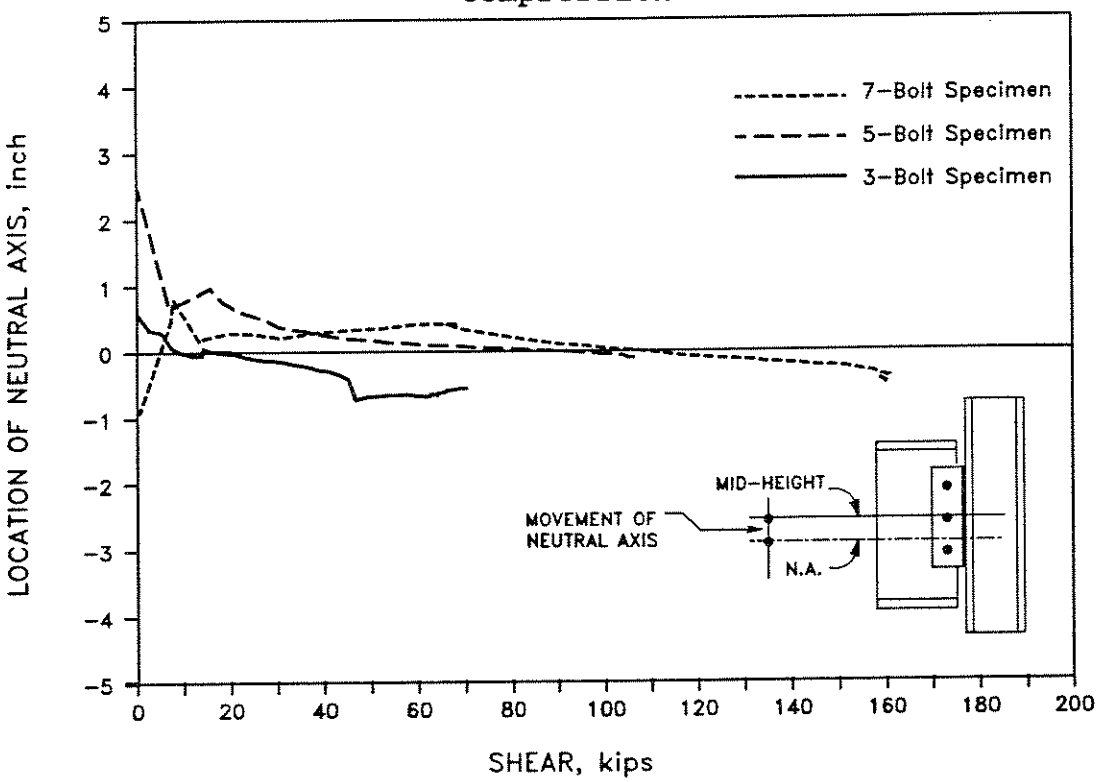
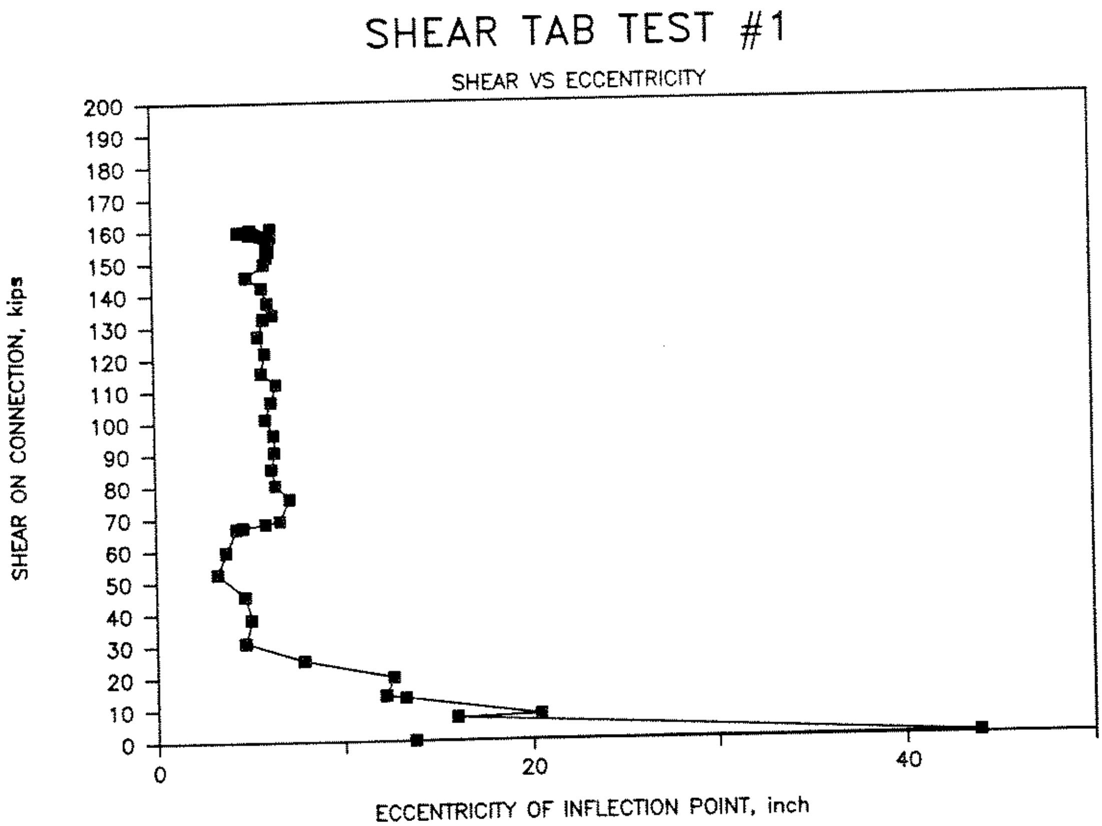


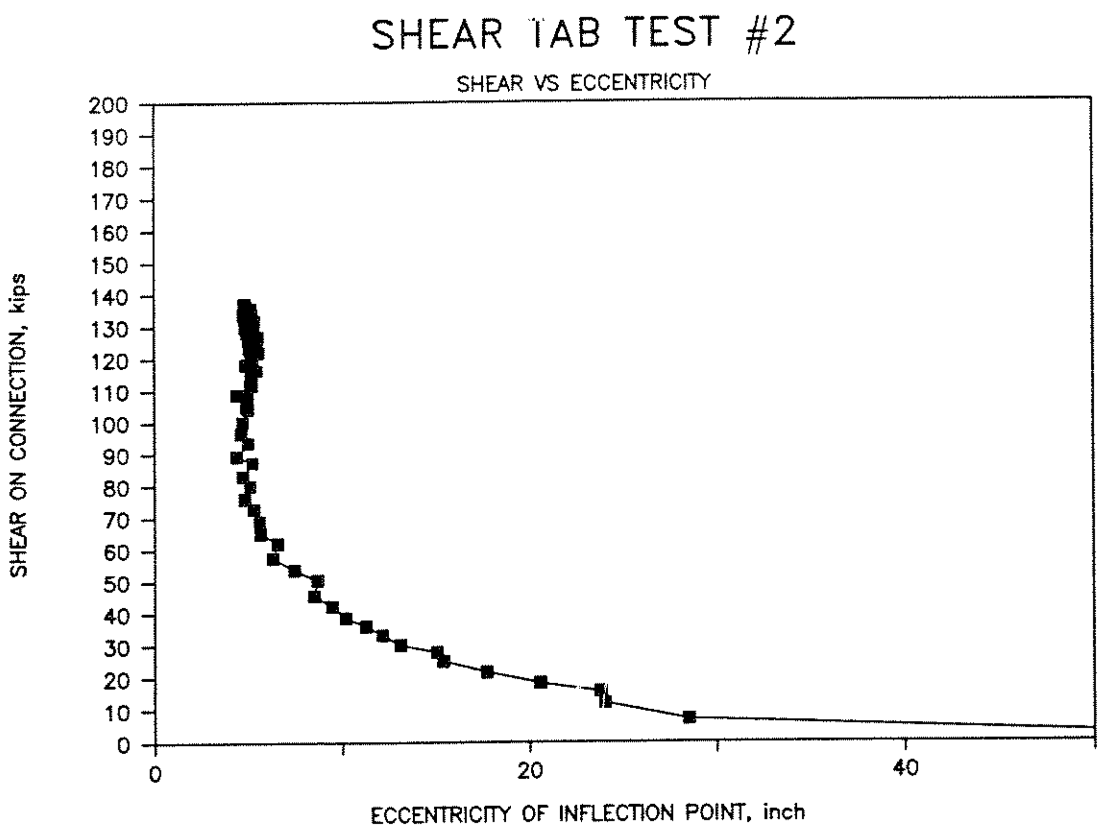
Figure 4.28. Movement of Neutral Axis During Testing of Single-Plate Connection

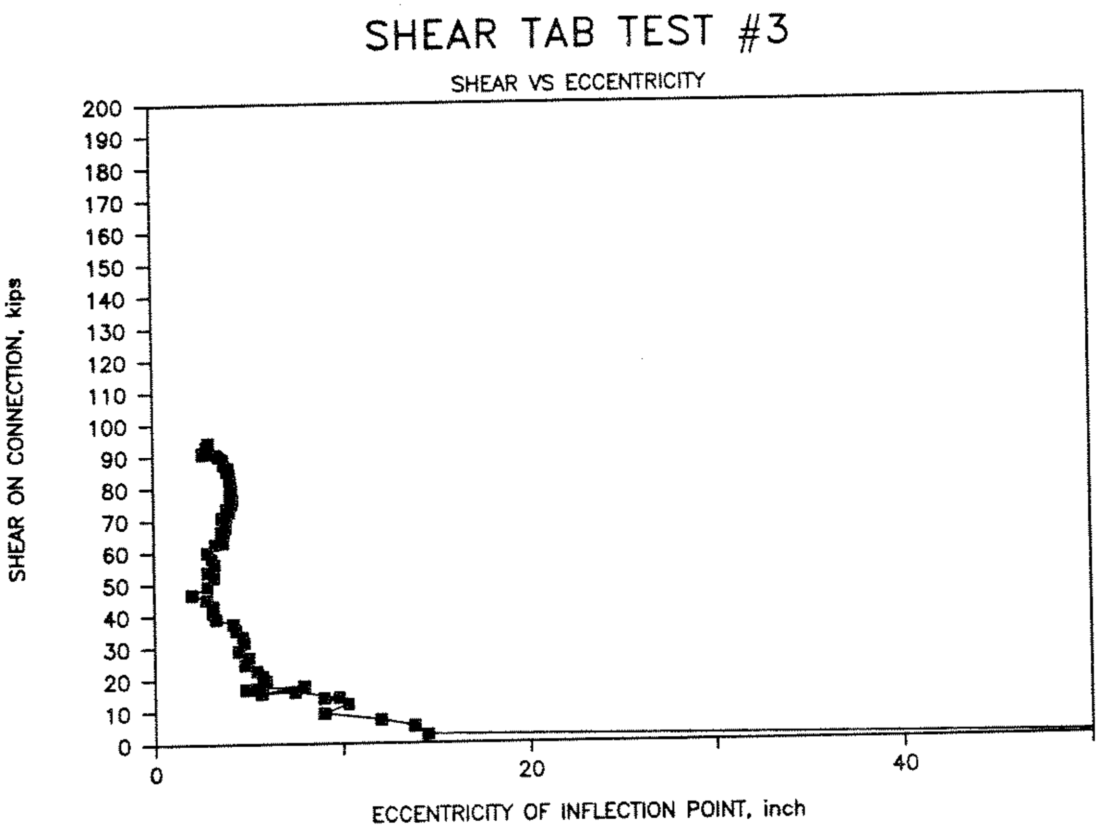
APPENDIX A

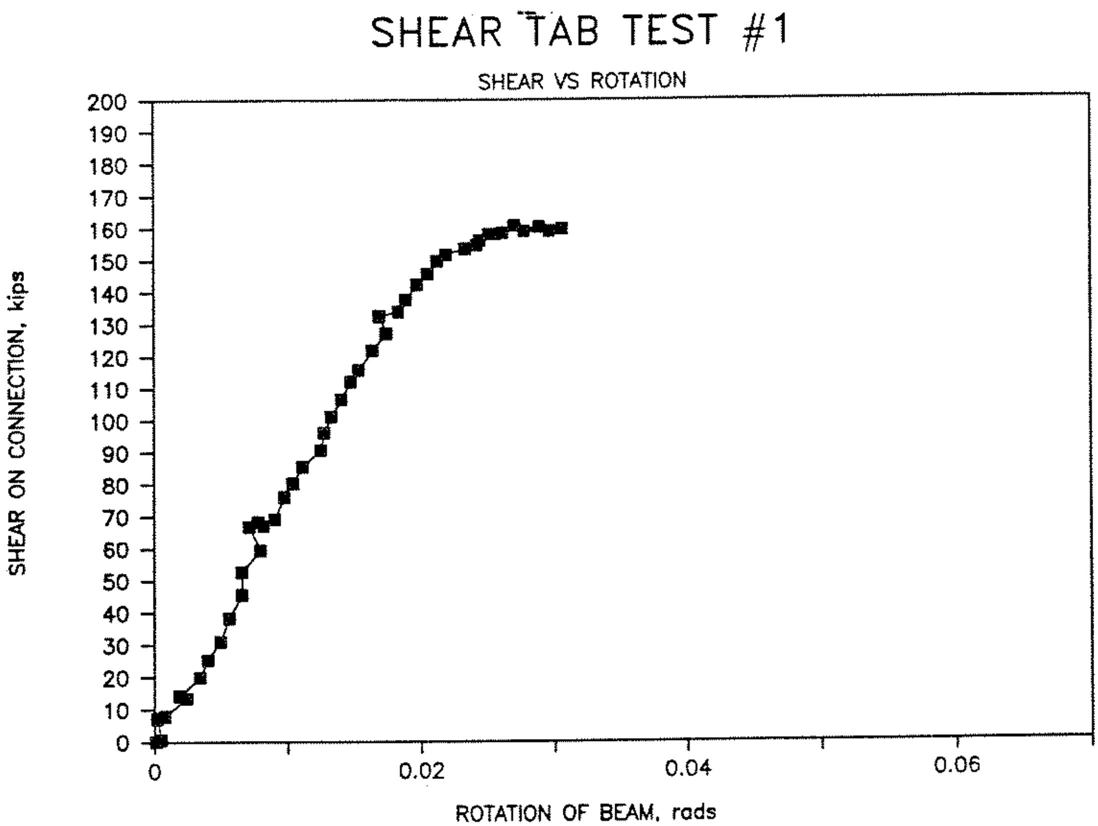
EXPERIMENTAL DATA

The experimental data collected during the tests are given in this appendix. The data is in the form of plots. For information on variables see Chapters 3 and 4.

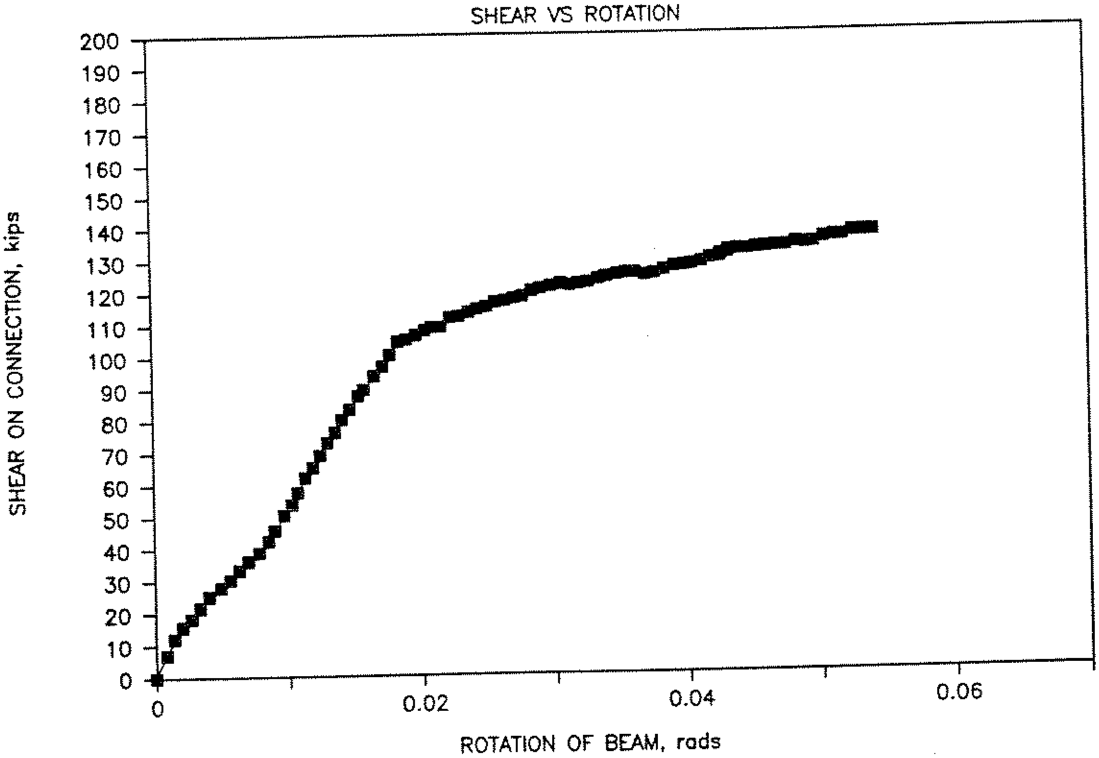

SHEAR TAB TEST #1
SHEAR VS ECCENTRICITY
SHEAR ON CONNECTION, kips
ECCENTRICITY OF INFLECTION POINT, inch
0
20
40
200
190
180
170
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0


SHEAR TAB TEST #2
SHEAR VS ECCENTRICITY
SHEAR ON CONNECTION, kips
ECCENTRICITY OF INFLECTION POINT, inch
0
20
40
200
190
180
170
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0

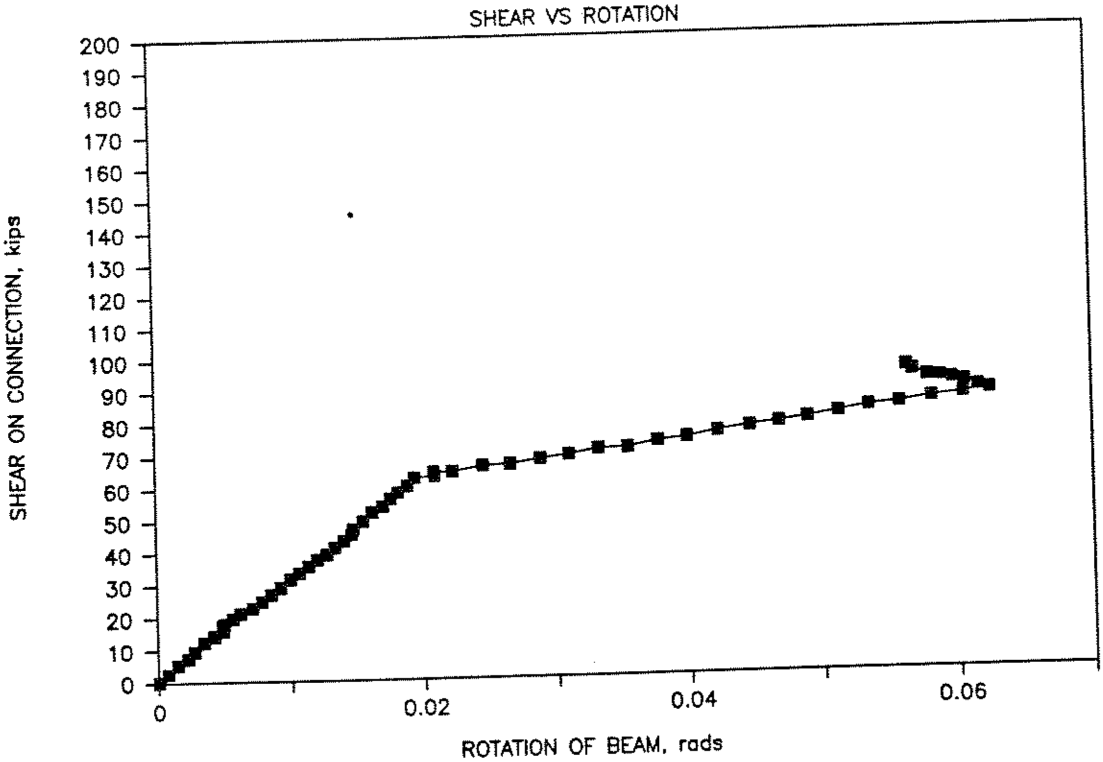

SHEAR TAB TEST #3
SHEAR VS ECCENTRICITY
SHEAR ON CONNECTION, kips
ECCENTRICITY OF INFLECTION POINT, inch
0
20
40
10
30
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200

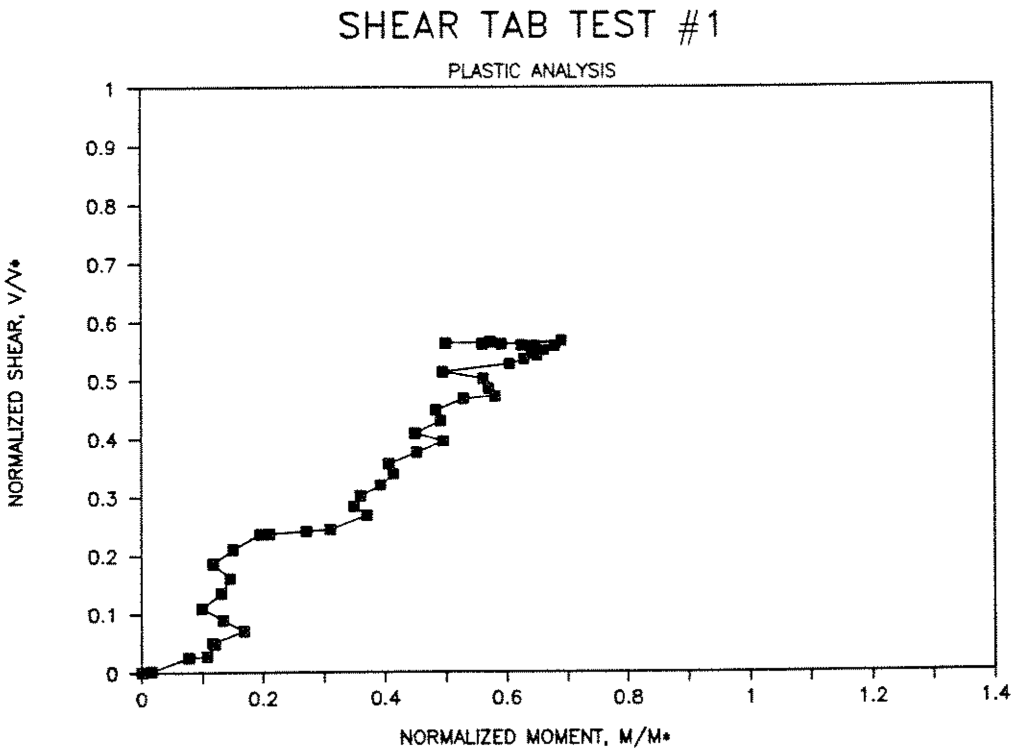

SHEAR TAB TEST #1
SHEAR VS ROTATION
SHEAR ON CONNECTION, kips
ROTATION OF BEAM, rads
0
0.02
0.04
0.06
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200

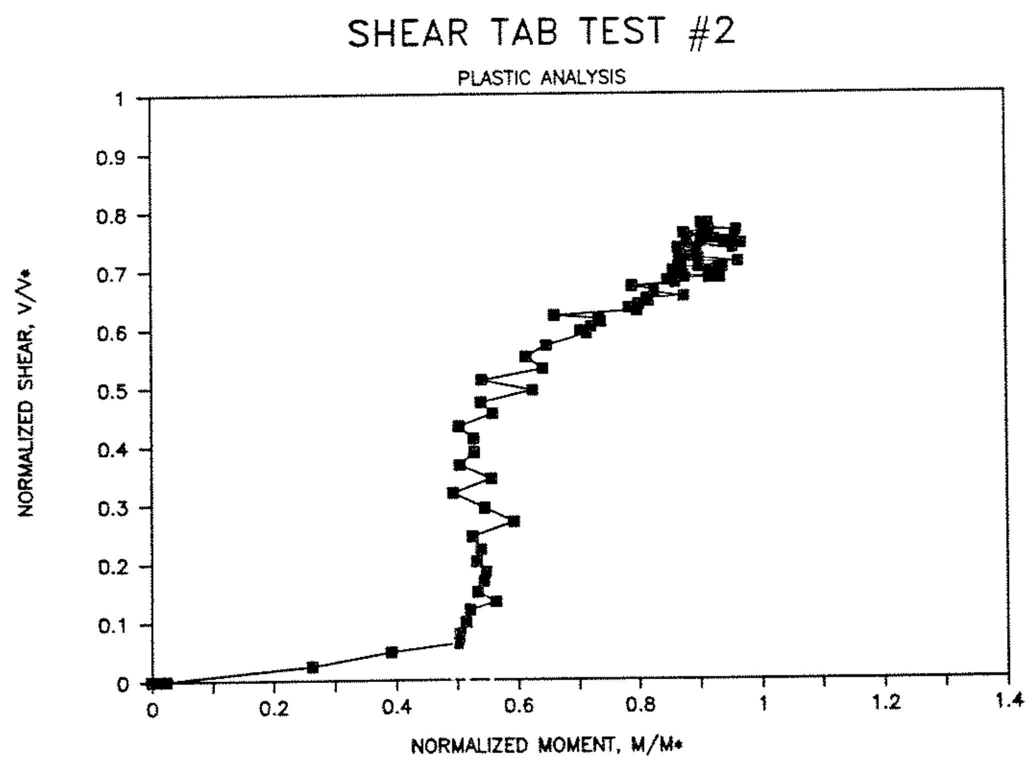
SHEAR TAB TEST #2

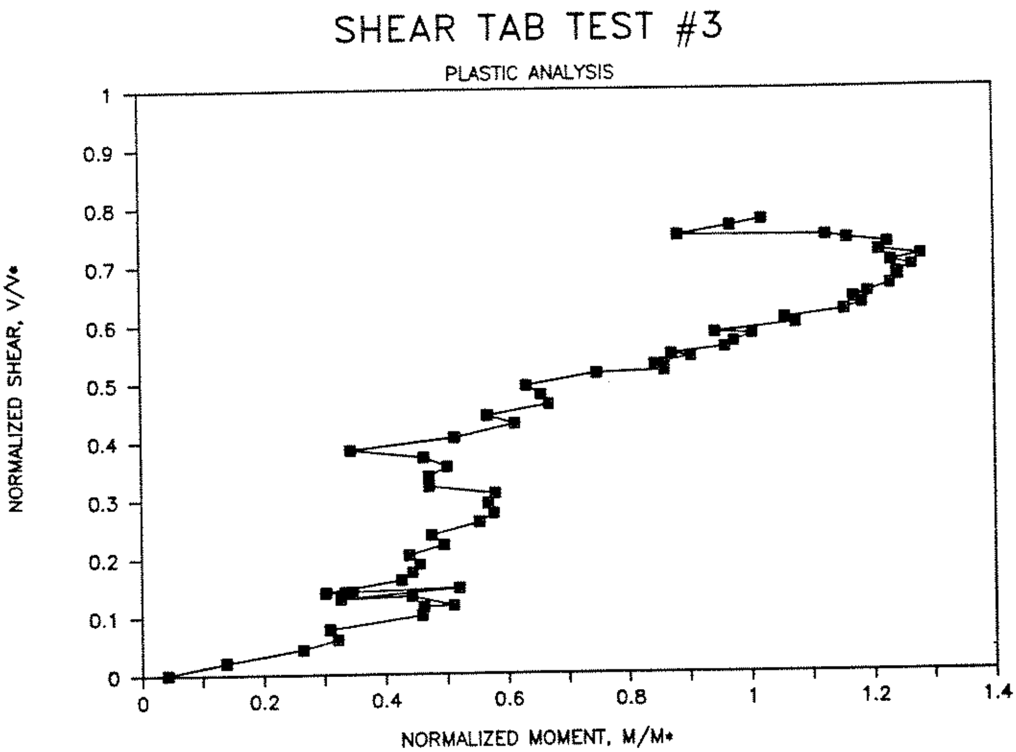


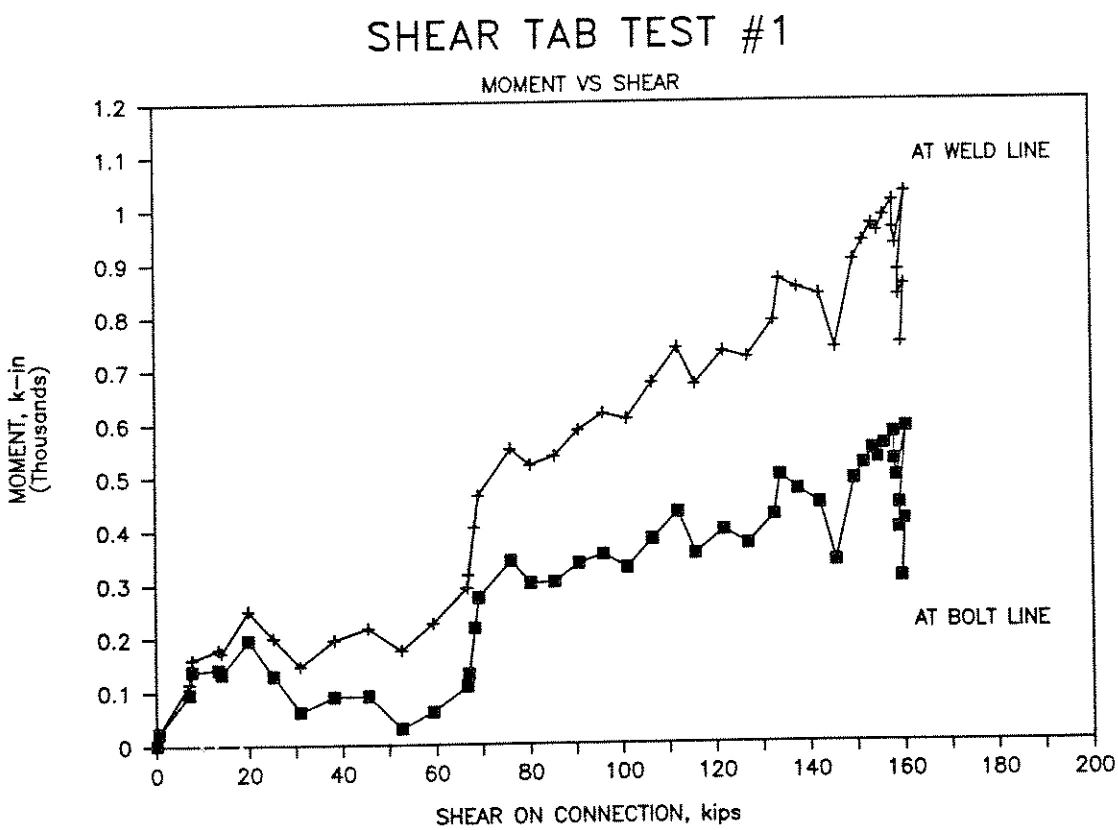
SHEAR TAB TEST #3



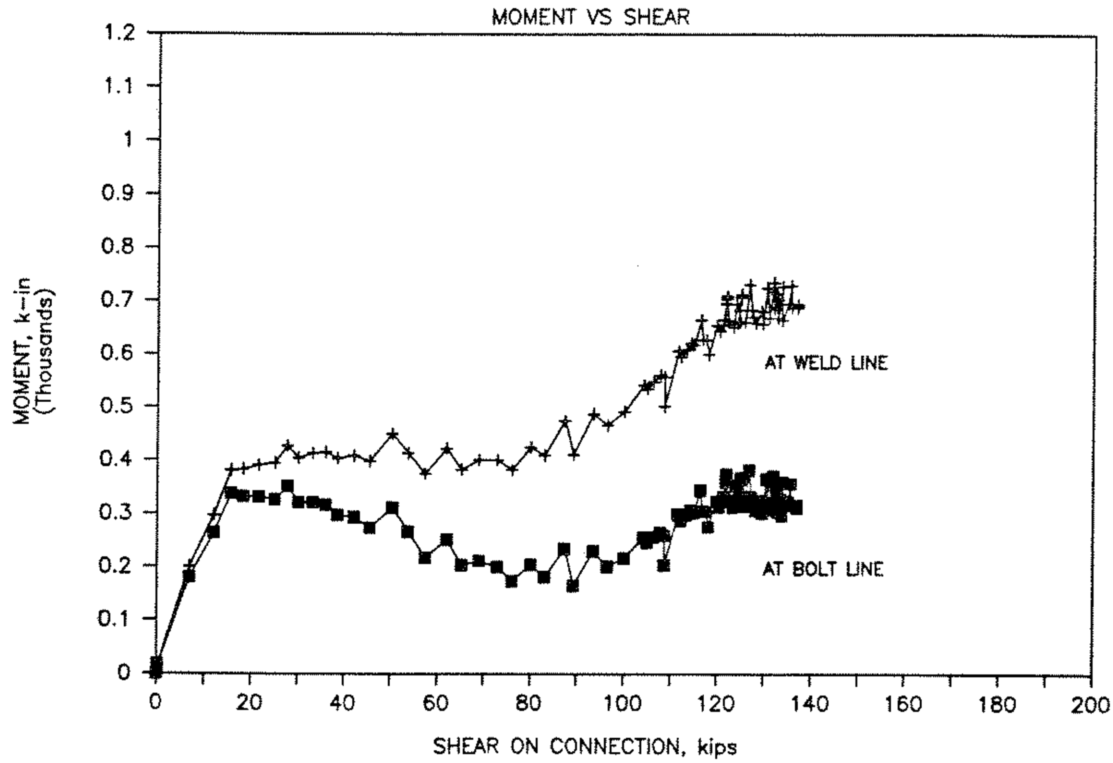

SHEAR TAB TEST #1
PLASTIC ANALYSIS
NORMALIZED SHEAR, V/V*
NORMALIZED MOMENT, M/M*
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
0.2
0.4
0.6
0.8
1.2
1.4


SHEAR TAB TEST #2
PLASTIC ANALYSIS
NORMALIZED SHEAR, V/V*
NORMALIZED MOMENT, M/M*
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
0.2
0.4
0.6
0.8
1.2
1.4

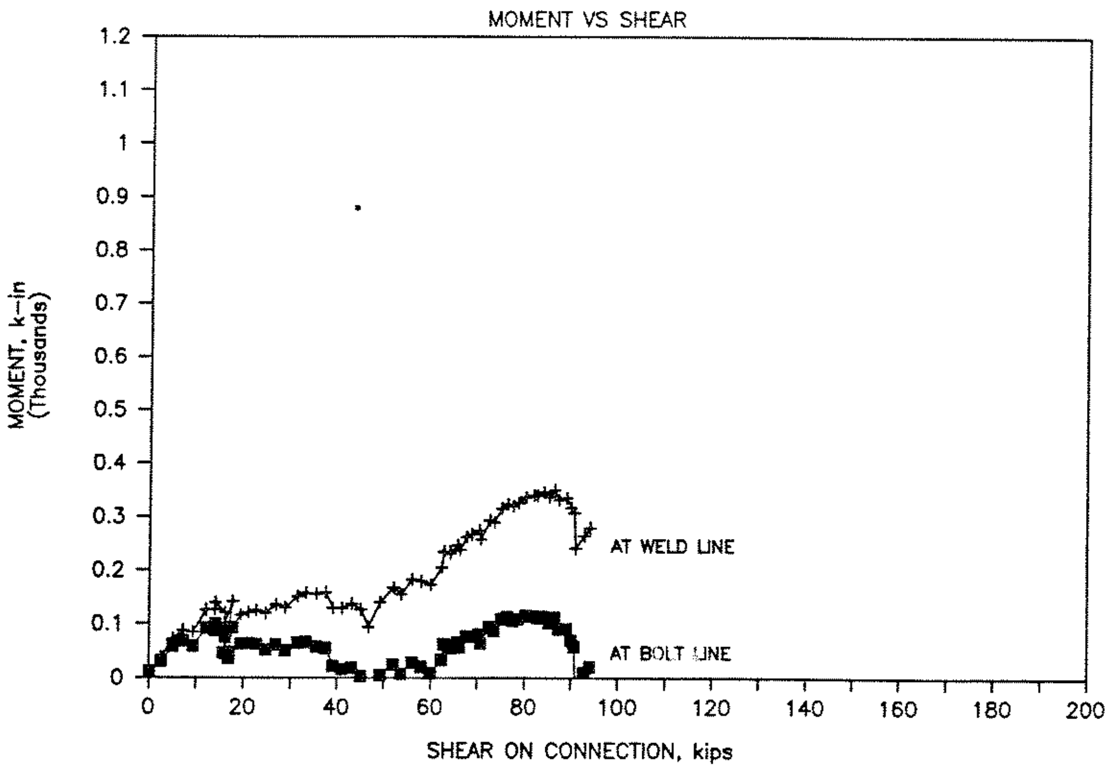

SHEAR TAB TEST #3
PLASTIC ANALYSIS
NORMALIZED SHEAR, V/V*
NORMALIZED MOMENT, M/M*
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0.2
0.4
0.6
0.8
1
1.2
1.4

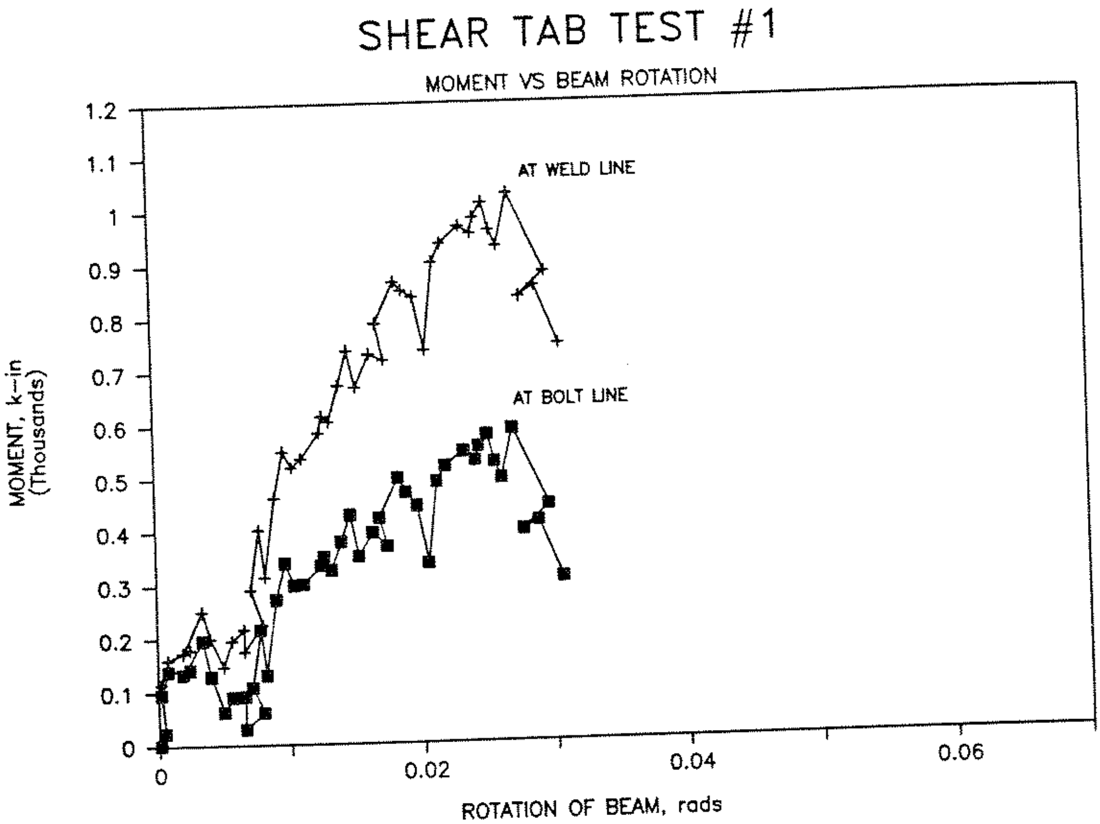

SHEAR TAB TEST #1
MOMENT VS SHEAR
AT WELD LINE
AT BOLT LINE
MOMENT, k-in
(Thousands)
SHEAR ON CONNECTION, kips
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
20
40
60
80
100
120
140
160
180
200

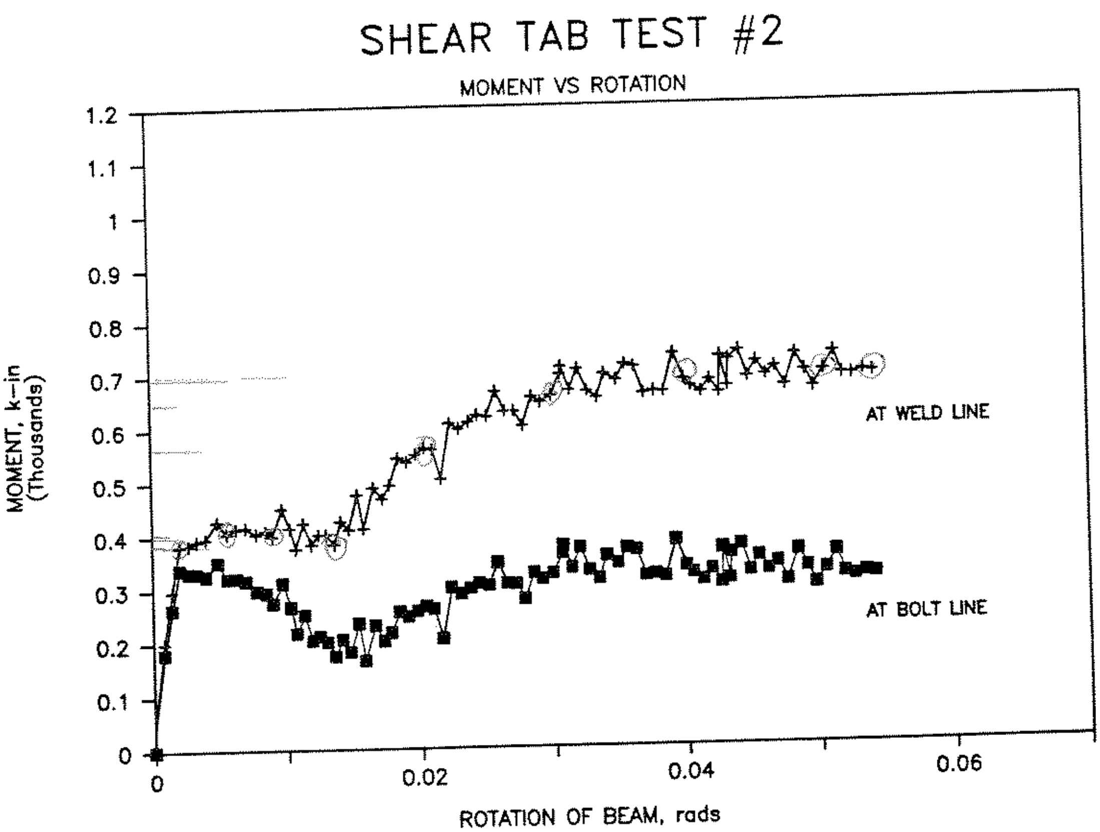
SHEAR TAB TEST #2

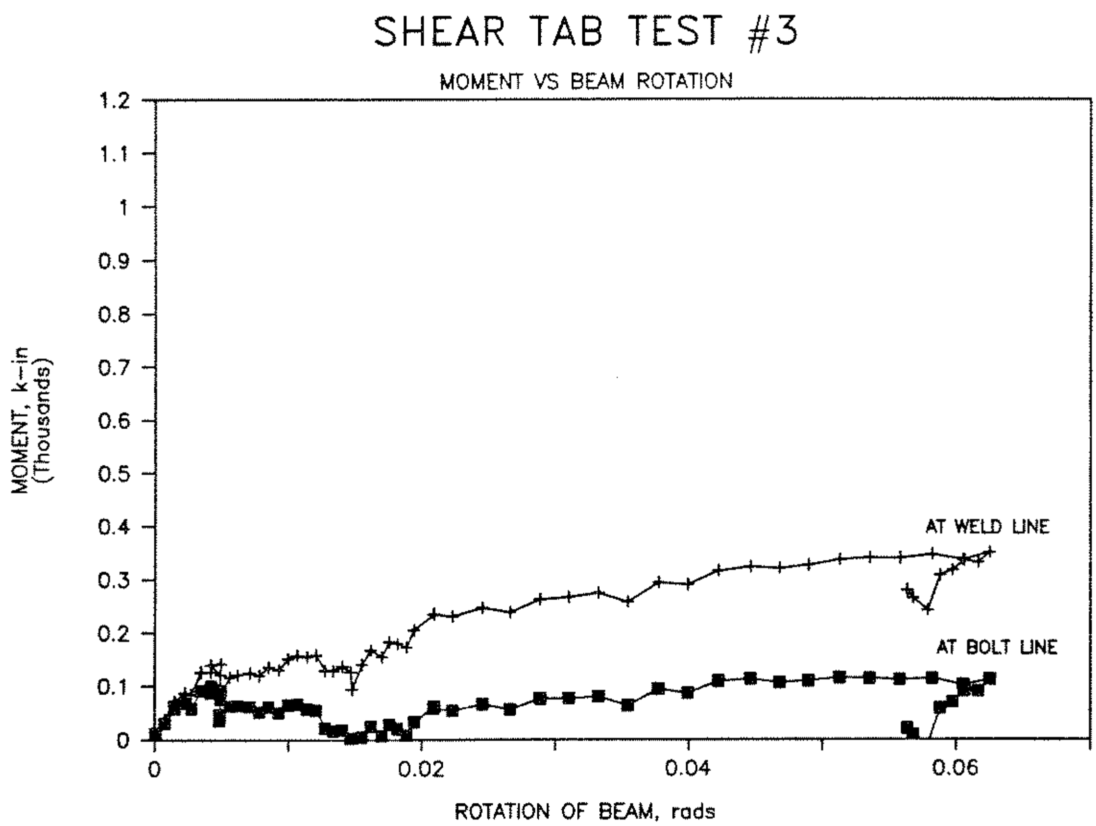


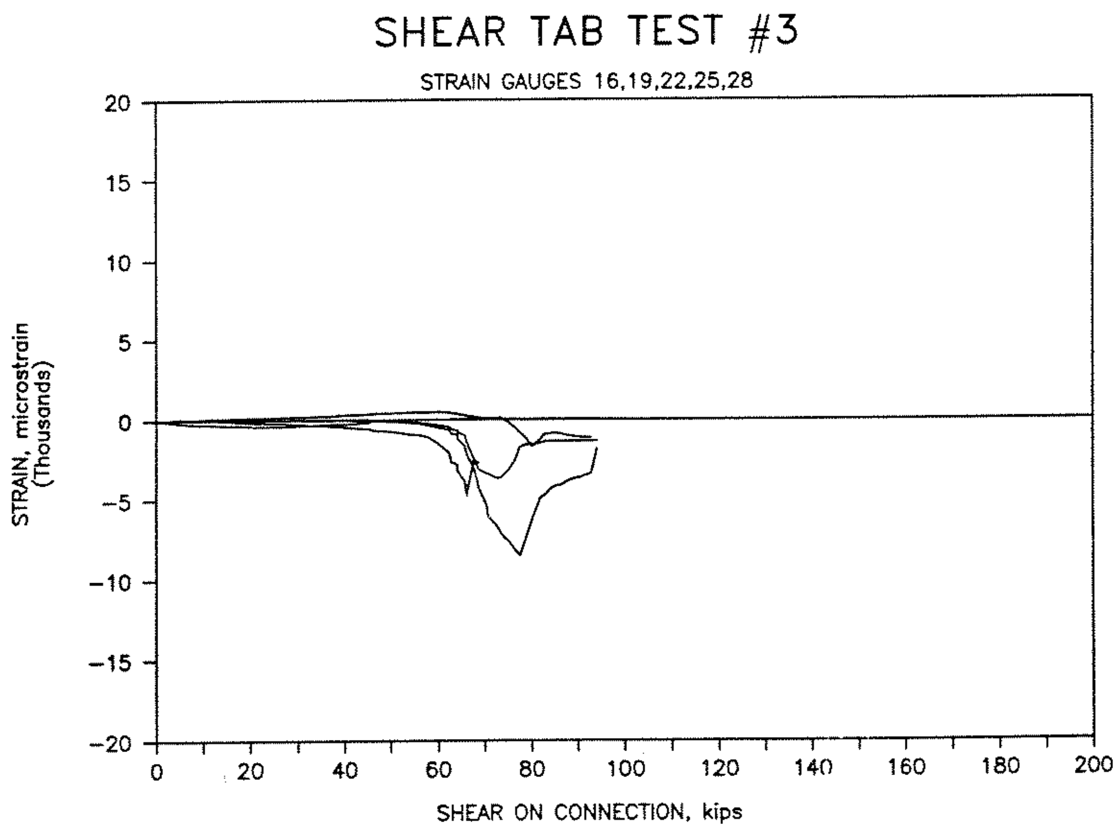
SHEAR TAB TEST #3

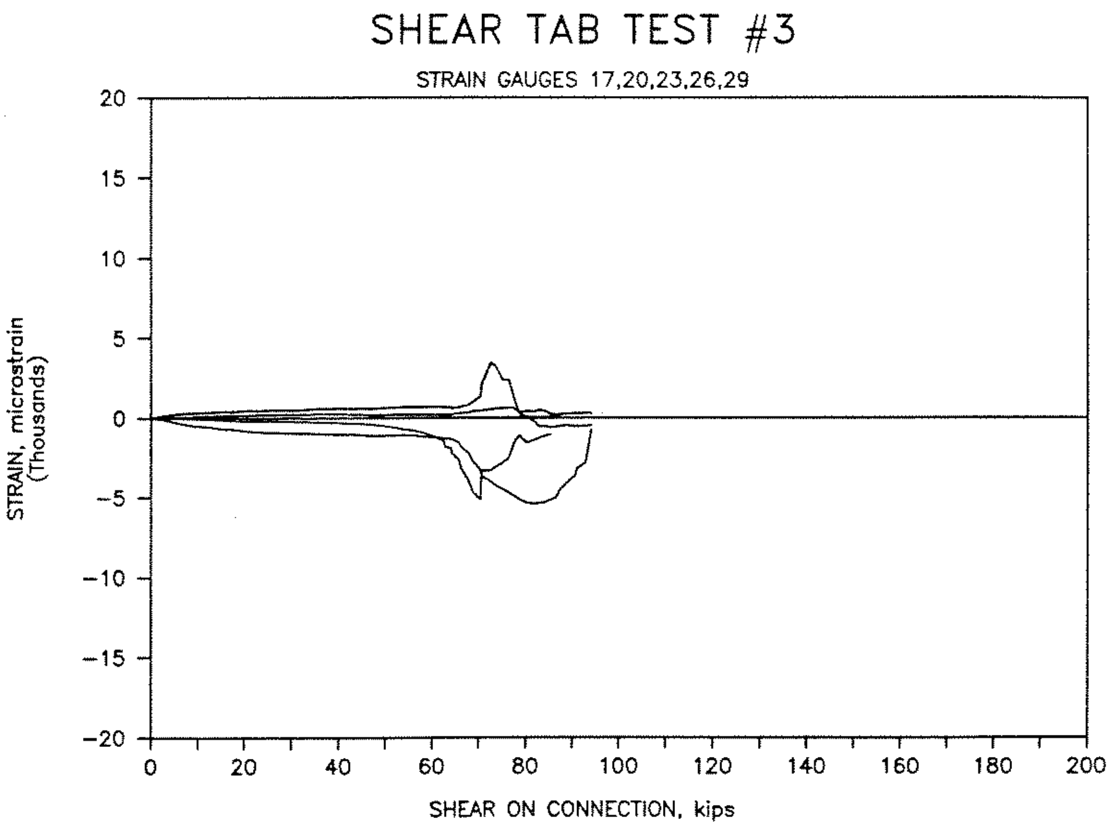


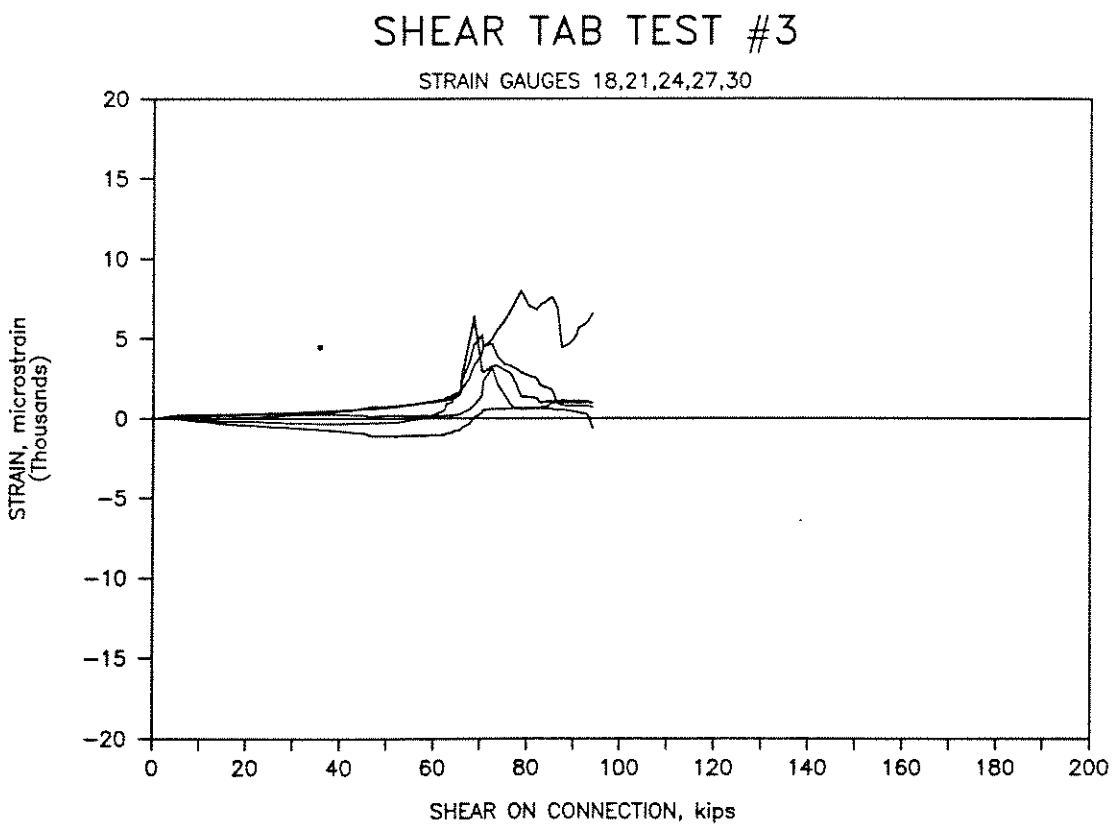

SHEAR TAB TEST #1
MOMENT VS BEAM ROTATION
AT WELD LINE
AT BOLT LINE
MOMENT, k-in
(Thousands)
ROTATION OF BEAM, rads
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
0.02
0.04
0.06


SHEAR TAB TEST #2
MOMENT VS ROTATION
AT WELD LINE
AT BOLT LINE
MOMENT, k-in
(Thousands)
ROTATION OF BEAM, rads
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
0.02
0.04
0.06

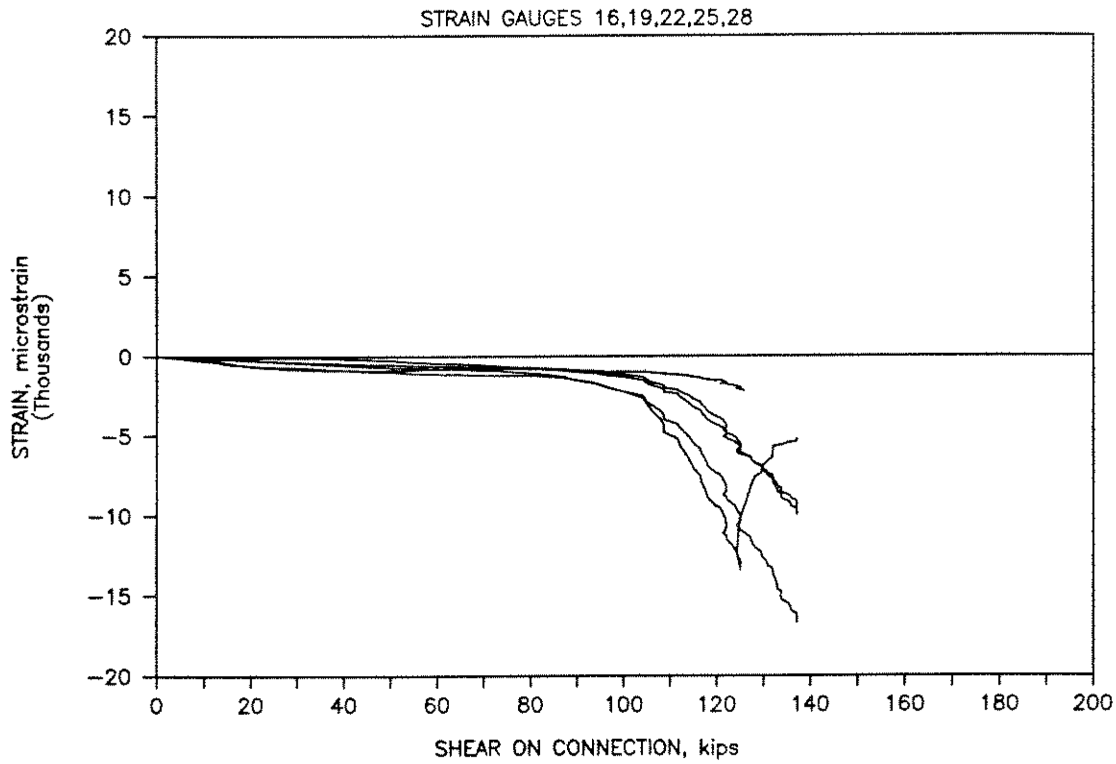

SHEAR TAB TEST #3
MOMENT VS BEAM ROTATION
MOMENT, k-in
(Thousands)
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
AT WELD LINE
AT BOLT LINE
0
0.02
0.04
0.06
ROTATION OF BEAM, rads


SHEAR TAB TEST #3
STRAIN GAUGES 16,19,22,25,28
STRAIN, microstrain
(Thousands)
20
15
10
5
0
−5
−10
−15
−20
0
20
40
60
80
100
120
140
160
180
200
SHEAR ON CONNECTION, kips

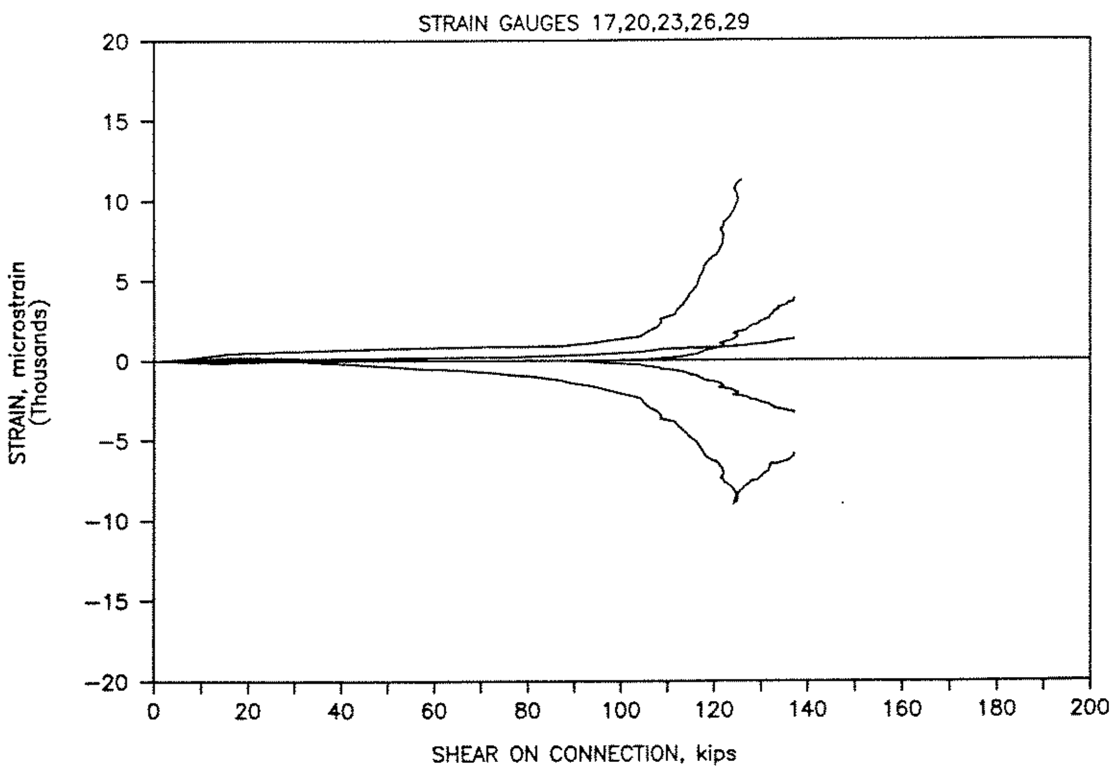

SHEAR TAB TEST #3
STRAIN GAUGES 17,20,23,26,29
STRAIN, microstrain (Thousands)
20
15
10
5
0
−5
−10
−15
−20
0
20
40
60
80
100
120
140
160
180
200
SHEAR ON CONNECTION, kips


SHEAR TAB TEST #3
STRAIN GAUGES 18,21,24,27,30
STRAIN, microstrain (Thousands)
20
15
10
5
0
−5
−10
−15
−20
0
20
40
60
80
100
120
140
160
180
200
SHEAR ON CONNECTION, kips

SHEAR TAB TEST #2

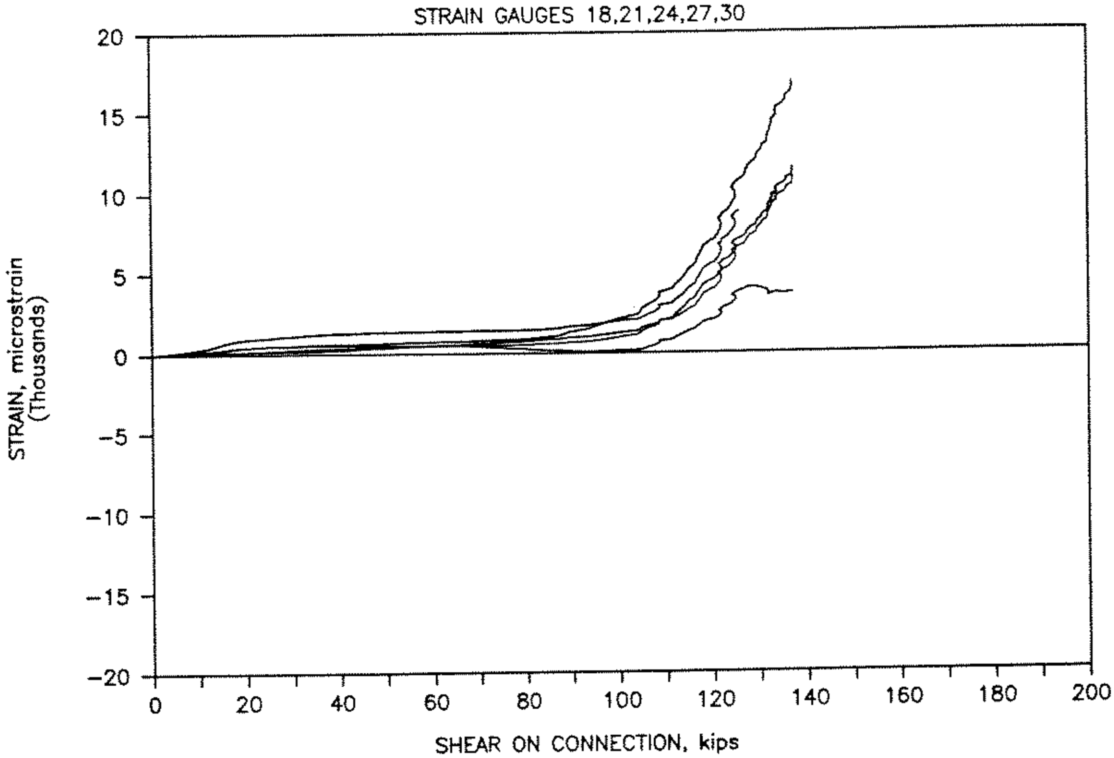


SHEAR TAB TEST #2



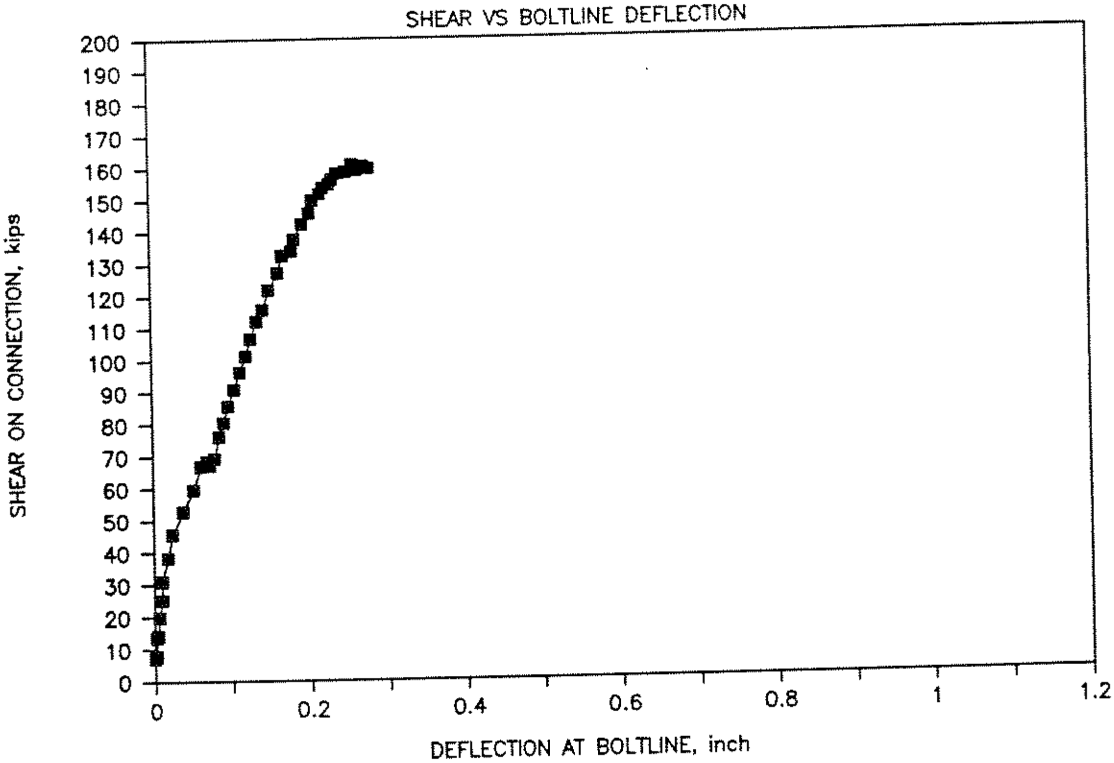
SHEAR TAB TEST #2

STRAIN GAUGES 18,21,24,27,30

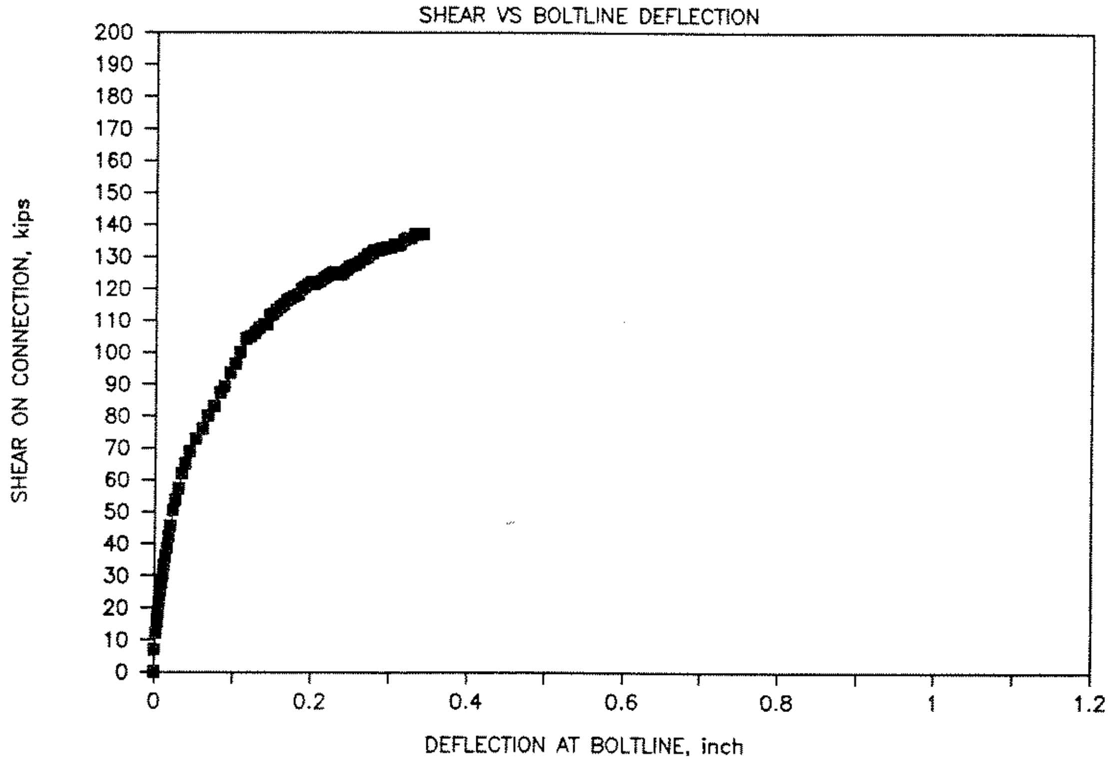


SHEAR TAB TEST #1

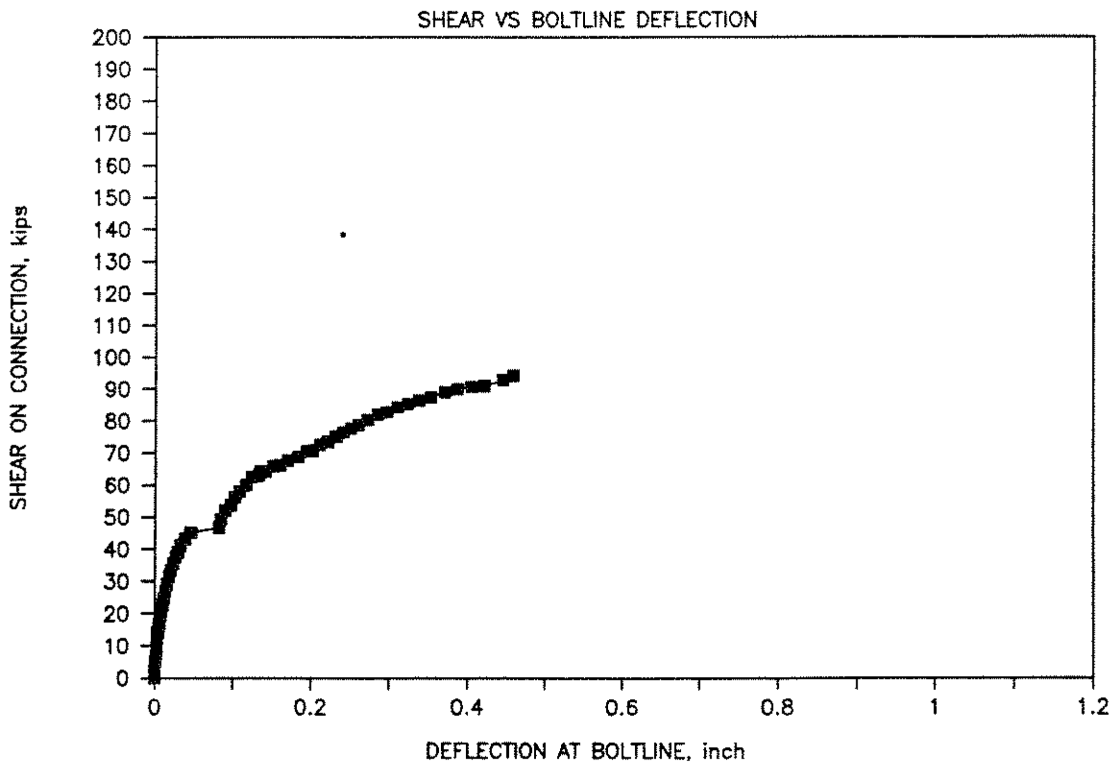
SHEAR VS BOLTLINE DEFLECTION



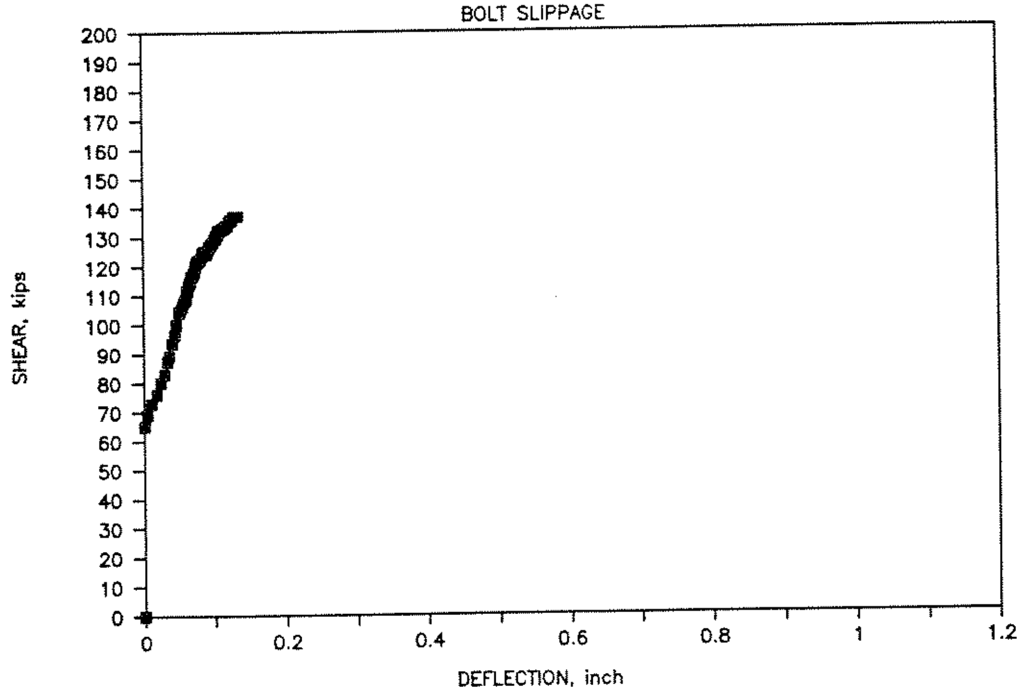
SHEAR TAB TEST #2



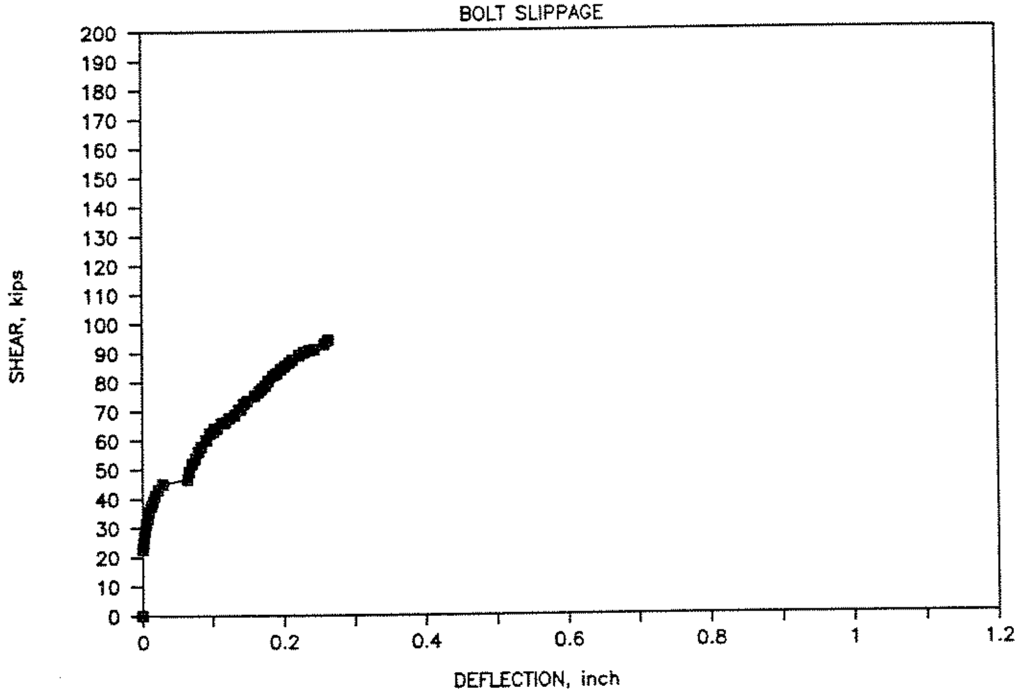
SHEAR TAB TEST #3



SHEAR TAB TEST #2



SHEAR TAB TEST #3



APPENDIX B

MATERIAL PROPERTIES

The material ordered for the single plates was A36 steel. In order to establish actual properties of material, standard coupon tests were conducted. The coupons were fabricated according to ASTM Standard E8 with an eight inches long coupon that had four inches of gage length. The results of these tests are reported in this appendix.

The coupon tests indicated that material is A36 steel with yield point of 35.5 ksi and ultimate strength of 61 ksi.

The mill reports on steel and bolts which were provided by the fabricator are also given in the following tables. The bolts were ASTM-A325.

TABLE B.1. Mill Report on Mechanical Properties and Chemical Composition of Steel Plates

[redacted]

SOLD TO: [redacted]

DATE SHIPPED: 4/28/88
CUSTOMER PO#: [redacted]

CERTIFIED MATERIAL TEST REPORT

PART NUMBER	QUANTITY	LOT NUMBER	DESCRIPTION
169550	23,400	060688	3/4-10 X 1 3/4 A325 HVY HX STRUC SCREW PLAIN PER ASTM A325

CHEMISTRY

(WT %)

LOT NUMBER	HEAT NUMBER	C	MN	P	S	SI
RP001823	US L60936	.4070	.7300	.0120	.0130	.2100

MECHANICAL PROPERTIES AVERAGE VALUES FROM 13 TESTS

LOT NUMBER	SURFACE HARDNESS (R30N)	CORE HARDNESS (RC)	PROOF LOAD/ELONGATION (LBS)	PROOF LOAD/ELONGATION (IN)	TENSILE STRENGTH AXIAL LOAD (LBS)	TENSILE STRENGTH 06 DEG- WEDGE STRESS (PSI)
060838	49.6	28.0	28,400	.0001	47,262	141,502

SURFACE COATINGS

LOT NUMBER	PLATING	OTHER	HEAT TREAT
060980	N/A	N/A	OIL QUENCH & TEMPER

ALL TESTS ARE IN ACCORDANCE WITH THE METHODS PRESCRIBED IN THE APPLICABLE SAE AND ASTM SPECIFICATIONS. WE CERTIFY THAT THIS DATA IS A TRUE REPRESENTATION OF INFORMATION PROVIDED BY THE MATERIAL SUPPLIER AND OUR TESTING LABORATORY.

[redacted]

PRODUCTS MANUFACTURED IN [redacted] U.S.A.
STEEL MELTED AND MANUFACTURED IN U.S.A.

TABLE A.2. Mill Report on Chemical Composition of Bolt Rods

[redacted]

DATE 3/8/88

[redacted]

CUST. ORDER NO. [redacted]

INVOICE NO.

[redacted]

SHIP TO

[redacted]

PROJECT

Truck

SPECIFICATIONS

SAE Grade C-1045 Fine Grain.

CHEMICAL ANALYSIS

HEAT NO.	C	P	MN	S	SI	V	
26887	.46	.008	.78	.030	.28		
17152	.47	.009	.80	.030	.26		
	CU	NI	CR	SN	MO	AL	C
26887	.23	.081	.122	.012	.018	.002	C
17152	.21	.089	.125	.013	.016	.002	C

HEAT NO.	DESCRIPTION	PCS.	WT.	YIELD POINT PSI	TENSILE STRENGTH PSI	% ELONG IN ___	RED AREA
26887	15/16" RDS. 27'0"/30'0"						
17152	15/16" RDS. 27'0"/30'0"						
	This material was 100% melted and manufactured in the U. S. A.						

Subscribed and sworn to before me a Notary Public, in and for the [redacted]
this ________ Day of ________
My commission expires ________

I hereby certify that the above tests are correct as contained in the records of the Company.

[redacted]

TABLE A.3. Mill Report on Mechanical Properties of Bolts

CERTIFICATE OF TESTS

FROM: [redacted]

INVOICE NO:
INVOICE DATE:
TERMS:

SOLD TO: [redacted]

SHIP TO: [redacted]

SOLD TO PURCHASE ORDER: [redacted]

DATE SHIPPED:
SHIP VIA: TRUCK

ROUTE CARRIER: [redacted]
VEHICLE I.D.

DESCRIPTION OF PRODUCT:
GRADE: ASTM A 36
HEAT NUMBER: SEE BELOW

SHIPPERS NO: [redacted]
SPECIFICATION: SPFC
FOB

IDENTITY	[redacted] ORDER NO.	DIMENSIONS	THEO WEIGHT
	[redacted]	.375X90X240	

I HEREBY CERTIFY THAT THE MATERIAL LISTED HERIN HAS BEEN INSPECTED AND TESTED IN ACCORDANCE WITH THE METHODS PRESCRIBED IN THE GOVERNING SPECIFICATIONS AND BASED UPON THE RESULTS OF SUCH INSPECTION AND TESTING HAS BEEN APPROVED FOR CONFORMANCE TO THE SPECIFICATIONS.

HEAT NUMBER:	C	MN	P	S	SI	CB	V	AL
Y66007	.20	.46	.009	.009	.007			
Y66004	.21	.38	.008	.010	.008			

HEAT NO.	GAUGE	YIELD(KSI)	TENSILE(KSI)	ELONGATION 2"	ELONGATION 8"	BENDS
Y66004		38.5	60.1	30.6		
		42.8	63.7	39.8		
Y66004		39.6	59.0	27.0		
		43.7	65.1	31.7		

Y66007-8C0300
Y66004-8C0301

[redacted]

APPENDIX C

DESIGN TABLES

The new design procedures that were developed, were discussed and presented in Chapters 5 and 6. Based on these procedures, 840 most common cases of single plate connections are designed and tabulated in this appendix. The tables apply for the cases where following limitations are observed.

Limitations used in tables:

1. Steel is A36
2. Bolts are A325 or A490. Bolts can be snug tight or tightened to achieve 70% of their proof load. Bolt holes are standard round or short slotted holes. Drilled or punched holes are permitted. Bolt threads can be included (N) in or excluded (X) from shear plane.
3. Welds are fillet welds. Electrodes are E70xx. E60xx electrodes are also permitted. However, the weld capacity should be reduced accordingly.
4. Bolt pitch and distance from bolt line to weld line are 3 inches.
5. Eccentricity of reaction from bolt line is assumed to be 3 inches whereas eccentricity of reaction from weld line is equal to N inches where n is the number of bolts.

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A325-N	3/16,	3.75x3/16x 4.50	3.6	12.2	11.0	12.8	12.7	5.7	3.6
3	1/2	A325-N	3/16,	3.75x3/16x 7.50	7.3	20.3	19.0	21.7	22.7	11.6	7.3
4	1/2	A325-N	3/16,	3.75x3/16x10.50	11.5	28.4	26.9	30.6	32.7	18.3	11.5
5	1/2	A325-N	3/16,	3.75x3/16x13.50	16.0	36.5	34.9	39.5	42.7	25.3	16.0
6	1/2	A325-N	3/16,	3.75x3/16x16.50	20.6	44.6	42.8	48.3	52.7	32.5	20.6
7	1/2	A325-N	3/16,	3.75x3/16x19.50	25.0	52.7	50.8	57.2	62.7	39.6	25.0

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A325-N	3/16,	4.00x3/16x 5.00	5.6	13.5	11.8	14.1	15.1	7.1	5.6
3	5/8	A325-N	3/16,	4.00x3/16x 8.00	11.5	21.6	19.4	22.7	25.1	14.5	11.5
4	5/8	A325-N	3/16,	4.00x3/16x11.00	18.0	29.7	26.9	31.4	35.1	22.8	18.0
5	5/8	A325-N	3/16,	4.00x3/16x14.00	25.0	37.8	34.5	40.1	45.2	31.6	25.0
6	5/8	A325-N	3/16,	4.00x3/16x17.00	32.1	45.9	42.0	48.7	55.2	40.7	32.1
7	5/8	A325-N	3/16,	4.00x3/16x20.00	39.1	54.0	49.5	57.4	65.2	49.6	39.1

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A325-N	3/16,	4.25x3/16x 5.25	8.1	14.2	11.8	14.5	16.3	8.5	8.1
3	3/4	A325-N	3/16,	4.25x3/16x 8.25	16.5	22.3	19.0	22.9	26.4	17.4	16.5
4	3/4	A325-N	3/16,	4.25x3/16x11.25	26.0	30.4	26.1	31.4	36.4	27.4	26.0
5	3/4	A325-N	3/16,	4.25x3/16x14.25	36.0	38.5	33.2	39.9	46.4	38.0	33.2
6	3/4	A325-N	3/16,	4.25x3/16x17.25	46.2	46.6	40.4	48.3	56.5	48.8	40.4
7	3/4	A325-N	3/16,	4.25x3/16x20.25	56.4	54.7	47.5	56.8	66.5	59.5	47.5

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A325-N	3/16,	4.50x3/16x 5.75	11.0	15.5	12.6	15.7	18.8	9.9	9.9
3	7/8	A325-N	3/16,	4.50x3/16x 8.75	22.5	23.6	19.4	24.0	28.9	20.3	19.4
4	7/8	A325-N	3/16,	4.50x3/16x11.75	35.3	31.7	26.1	32.2	38.9	31.9	26.1
5	7/8	A325-N	3/16,	4.50x3/16x14.75	49.0	39.8	32.8	40.5	48.9	44.3	32.8
6	7/8	A325-N	3/16,	4.50x3/16x17.75	62.9	47.9	39.6	48.7	59.0	56.9	39.6
7	7/8	A325-N	3/16,	4.50x3/16x20.75	76.7	56.0	46.3	57.0	69.0	69.4	46.3

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A325-N	3/16,	4.50x3/16x 6.00	14.3	16.2	12.6	16.1	20.1	11.3	11.3
3	1.0	A325-N	3/16,	4.50x3/16x 9.00	29.4	24.3	19.0	24.2	30.1	23.2	19.0
4	1.0	A325-N	3/16,	4.50x3/16x12.00	46.1	32.4	25.3	32.2	40.2	36.5	25.3
5	1.0	A325-N	3/16,	4.50x3/16x15.00	64.0	40.5	31.6	40.3	50.2	50.6	31.6
6	1.0	A325-N	3/16,	4.50x3/16x18.00	82.2	48.6	37.9	48.3	60.2	65.1	37.9
7	1.0	A325-N	3/16,	4.50x3/16x21.00	100.2	56.7	44.2	56.4	70.3	79.3	44.2

NOTES:

1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips.
   Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A325-N	1/4	3.75x 1/4 x 4.50	3.6	16.2	14.7	17.1	16.9	7.6	3.6
3	1/2	A325-N	1/4	3.75x 1/4 x 7.50	7.3	27.0	25.3	29.0	30.2	15.5	7.3
4	1/2	A325-N	1/4	3.75x 1/4 x10.50	11.5	37.8	35.9	40.8	43.5	24.3	11.5
5	1/2	A325-N	1/4	3.75x 1/4 x13.50	16.0	48.6	46.5	52.6	56.9	33.7	16.0
6	1/2	A325-N	1/4	3.75x 1/4 x16.50	20.6	59.4	57.1	64.4	70.3	43.4	20.6
7	1/2	A325-N	1/4	3.75x 1/4 x19.50	25.0	70.2	67.7	76.3	83.7	52.9	25.0

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A325-N	1/4	4.00x 1/4 x 5.00	5.6	18.0	15.8	18.8	20.1	9.4	5.6
3	5/8	A325-N	1/4	4.00x 1/4 x 8.00	11.5	28.8	25.8	30.3	33.5	19.4	11.5
4	5/8	A325-N	1/4	4.00x 1/4 x11.00	18.0	39.6	35.9	41.9	46.9	30.4	18.0
5	5/8	A325-N	1/4	4.00x 1/4 x14.00	25.0	50.4	45.9	53.4	60.2	42.2	25.0
6	5/8	A325-N	1/4	4.00x 1/4 x17.00	32.1	61.2	56.0	65.0	73.6	54.2	32.1
7	5/8	A325-N	1/4	4.00x 1/4 x20.00	39.1	72.0	66.1	76.5	87.0	66.1	39.1

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A325-N	1/4	4.25x 1/4 x 5.25	8.1	18.9	15.8	19.3	21.8	11.3	8.1
3	3/4	A325-N	1/4	4.25x 1/4 x 8.25	16.5	29.7	25.3	30.6	35.1	23.2	16.5
4	3/4	A325-N	1/4	4.25x 1/4 x11.25	26.0	40.5	34.8	41.9	48.5	36.5	26.0
5	3/4	A325-N	1/4	4.25x 1/4 x14.25	36.0	51.3	44.3	53.2	61.9	50.6	36.0
6	3/4	A325-N	1/4	4.25x 1/4 x17.25	46.2	62.1	53.8	64.4	75.3	65.1	46.2
7	3/4	A325-N	1/4	4.25x 1/4 x20.25	56.4	72.9	63.3	75.7	88.7	79.3	56.4

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A325-N	1/4	4.50x 1/4 x 5.75	11.0	20.7	16.9	20.9	25.1	13.2	11.0
3	7/8	A325-N	1/4	4.50x 1/4 x 8.75	22.5	31.5	25.8	31.9	38.5	27.1	22.5
4	7/8	A325-N	1/4	4.50x 1/4 x11.75	35.3	42.3	34.8	43.0	51.9	42.6	34.8
5	7/8	A325-N	1/4	4.50x 1/4 x14.75	49.0	53.1	43.8	54.0	65.3	59.1	43.8
6	7/8	A325-N	1/4	4.50x 1/4 x17.75	62.9	63.9	52.7	65.0	78.6	75.9	52.7
7	7/8	A325-N	1/4	4.50x 1/4 x20.75	76.7	74.7	61.7	76.0	92.0	92.5	61.7

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A325-N	1/4	4.50x 1/4 x 6.00	14.3	21.6	16.9	21.5	26.8	15.1	14.3
3	1.0	A325-N	1/4	4.50x 1/4 x 9.00	29.4	32.4	25.3	32.2	40.2	31.0	25.3
4	1.0	A325-N	1/4	4.50x 1/4 x12.00	46.1	43.2	33.7	43.0	53.5	48.7	33.7
5	1.0	A325-N	1/4	4.50x 1/4 x15.00	64.0	54.0	42.1	53.7	66.9	67.5	42.1
6	1.0	A325-N	1/4	4.50x 1/4 x18.00	82.2	64.8	50.6	64.4	80.3	86.7	50.6
7	1.0	A325-N	1/4	4.50x 1/4 x21.00	100.2	75.6	59.0	75.2	93.7	105.7	59.0

NOTES:
1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips.
   Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A325-N	5/16,	3.75x 5/16x 4.50			**** tp > db/2 ****				
3	1/2	A325-N	5/16,	3.75x 5/16x 7.50			**** tp > db/2 ****				
4	1/2	A325-N	5/16,	3.75x 5/16x 10.50			**** tp > db/2 ****				
5	1/2	A325-N	5/16,	3.75x 5/16x 13.50			**** tp > db/2 ****				
6	1/2	A325-N	5/16,	3.75x 5/16x 16.50			**** tp > db/2 ****				
7	1/2	A325-N	5/16,	3.75x 5/16x 19.50			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A325-N	5/16,	4.00x 5/16x 5.00	5.6	22.5	19.7	23.4	25.2	11.8	5.6
3	5/8	A325-N	5/16,	4.00x 5/16x 8.00	11.5	36.0	32.3	37.9	41.9	24.2	11.5
4	5/8	A325-N	5/16,	4.00x 5/16x11.00	18.0	49.5	44.9	52.3	58.6	38.0	18.0
5	5/8	A325-N	5/16,	4.00x 5/16x14.00	25.0	63.0	57.4	66.8	75.3	52.7	25.0
6	5/8	A325-N	5/16,	4.00x 5/16x17.00	32.1	76.5	70.0	81.2	92.0	67.8	32.1
7	5/8	A325-N	5/16,	4.00x 5/16x20.00	39.1	90.0	82.6	95.7	108.7	82.6	39.1

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A325-N	5/16,	4.25x 5/16x 5.25	8.1	23.6	19.7	24.1	27.2	14.2	8.1
3	3/4	A325-N	5/16,	4.25x 5/16x 8.25	16.5	37.1	31.6	38.2	43.9	29.0	16.5
4	3/4	A325-N	5/16,	4.25x 5/16x11.25	26.0	50.6	43.5	52.3	60.7	45.6	26.0
5	3/4	A325-N	5/16,	4.25x 5/16x14.25	36.0	64.1	55.4	66.4	77.4	63.3	36.0
6	3/4	A325-N	5/16,	4.25x 5/16x17.25	46.2	77.6	67.3	80.5	94.1	81.3	46.2
7	3/4	A325-N	5/16,	4.25x 5/16x20.25	56.4	91.1	79.2	94.6	110.8	99.1	56.4

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A325-N	5/16,	4.50x 5/16x 5.75	11.0	25.9	21.1	26.2	31.4	16.5	11.0
3	7/8	A325-N	5/16,	4.50x 5/16x 8.75	22.5	39.4	32.3	39.9	48.1	33.9	22.5
4	7/8	A325-N	5/16,	4.50x 5/16x11.75	35.3	52.9	43.5	53.7	64.8	53.2	35.3
5	7/8	A325-N	5/16,	4.50x 5/16x14.75	49.0	66.4	54.7	67.5	81.6	73.8	49.0
6	7/8	A325-N	5/16,	4.50x 5/16x17.75	62.9	79.9	65.9	81.2	98.3	94.9	62.9
7	7/8	A325-N	5/16,	4.50x 5/16x20.75	76.7	93.4	77.1	95.0	115.0	115.6	76.7

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A325-N	5/16,	4.50x 5/16x 6.00	14.3	27.0	21.1	26.8	33.5	18.9	14.3
3	1.0	A325-N	5/16,	4.50x 5/16x 9.00	29.4	40.5	31.6	40.3	50.2	38.7	29.4
4	1.0	A325-N	5/16,	4.50x 5/16x12.00	46.1	54.0	42.1	53.7	66.9	60.8	42.1
5	1.0	A325-N	5/16,	4.50x 5/16x15.00	64.0	67.5	52.7	67.1	83.7	84.4	52.7
6	1.0	A325-N	5/16,	4.50x 5/16x18.00	82.2	81.0	63.2	80.5	100.4	108.4	63.2
7	1.0	A325-N	5/16,	4.50x 5/16x21.00	100.2	94.5	73.7	94.0	117.1	132.1	73.7

NOTES:

1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips.
   Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A325-N	3/8 ,	3.75x 3/8 x 4.50			**** tp > db/2 ****				
3	1/2	A325-N	3/8 ,	3.75x 3/8 x 7.50			**** tp > db/2 ****				
4	1/2	A325-N	3/8 ,	3.75x 3/8 x 10.50			**** tp > db/2 ****				
5	1/2	A325-N	3/8 ,	3.75x 3/8 x 13.50			**** tp > db/2 ****				
6	1/2	A325-N	3/8 ,	3.75x 3/8 x 16.50			**** tp > db/2 ****				
7	1/2	A325-N	3/8 ,	3.75x 3/8 x 19.50			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A325-N	3/8 ,	4.00x 3/8 x 5.00			**** tp > db/2 ****				
3	5/8	A325-N	3/8 ,	4.00x 3/8 x 8.00			**** tp > db/2 ****				
4	5/8	A325-N	3/8 ,	4.00x 3/8 x 11.00			**** tp > db/2 ****				
5	5/8	A325-N	3/8 ,	4.00x 3/8 x 14.00			**** tp > db/2 ****				
6	5/8	A325-N	5/16,	4.00x 3/8 x 17.00			**** tp > db/2 ****				
7	5/8	A325-N	5/16,	4.00x 3/8 x 20.00			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A325-N	3/8 ,	4.25x 3/8 x 5.25	8.1	28.4	23.7	29.0	32.7	17.0	8.1
3	3/4	A325-N	3/8 ,	4.25x 3/8 x 8.25	16.5	44.6	37.9	45.9	52.7	34.9	16.5
4	3/4	A325-N	3/8 ,	4.25x 3/8 x11.25	26.0	60.8	52.2	62.8	72.8	54.8	26.0
5	3/4	A325-N	5/16,	4.25x 3/8 x14.25	36.0	77.0	66.5	79.7	77.4	75.9	36.0
6	3/4	A325-N	5/16,	4.25x 3/8 x17.25	46.2	93.2	80.7	96.7	94.1	97.6	46.2
7	3/4	A325-N	5/16,	4.25x 3/8 x20.25	56.4	109.4	95.0	113.6	110.8	118.9	56.4

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A325-N	5/16,	4.50x 3/8 x 5.75	11.0	31.1	25.3	31.4	31.4	19.8	11.0
3	7/8	A325-N	5/16,	4.50x 3/8 x 8.75	22.5	47.3	38.7	47.9	48.1	40.7	22.5
4	7/8	A325-N	5/16,	4.50x 3/8 x11.75	35.3	63.5	52.2	64.4	64.8	63.9	35.3
5	7/8	A325-N	5/16,	4.50x 3/8 x14.75	49.0	79.7	65.7	81.0	81.6	88.6	49.0
6	7/8	A325-N	5/16,	4.50x 3/8 x17.75	62.9	95.9	79.1	97.5	98.3	113.8	62.9
7	7/8	A325-N	5/16,	4.50x 3/8 x20.75	76.7	112.1	92.6	114.0	115.0	138.7	76.7

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A325-N	5/16,	4.50x 3/8 x 6.00	14.3	32.4	25.3	32.2	33.5	22.7	14.3
3	1.0	A325-N	5/16,	4.50x 3/8 x 9.00	29.4	48.6	37.9	48.3	50.2	46.5	29.4
4	1.0	A325-N	5/16,	4.50x 3/8 x12.00	46.1	64.8	50.6	64.4	66.9	73.0	46.1
5	1.0	A325-N	5/16,	4.50x 3/8 x15.00	64.0	81.0	63.2	80.5	83.7	101.2	63.2
6	1.0	A325-N	5/16,	4.50x 3/8 x18.00	82.2	97.2	75.9	96.7	100.4	130.1	75.9
7	1.0	A325-N	5/16,	4.50x 3/8 x21.00	100.2	113.4	88.5	112.8	117.1	158.6	88.5

NOTES:

1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips.
   Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A325-N	7/16,	3.75x 7/16x 4.50			**** tp > db/2 ****				
3	1/2	A325-N	7/16,	3.75x 7/16x 7.50			**** tp > db/2 ****				
4	1/2	A325-N	7/16,	3.75x 7/16x 10.50			**** tp > db/2 ****				
5	1/2	A325-N	3/8 ,	3.75x 7/16x 13.50			**** tp > db/2 ****				
6	1/2	A325-N	3/8 ,	3.75x 7/16x 16.50			**** tp > db/2 ****				
7	1/2	A325-N	3/8 ,	3.75x 7/16x 19.50			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A325-N	7/16,	4.00x 7/16x 5.00			**** tp > db/2 ****				
3	5/8	A325-N	7/16,	4.00x 7/16x 8.00			**** tp > db/2 ****				
4	5/8	A325-N	3/8 ,	4.00x 7/16x 11.00			**** tp > db/2 ****				
5	5/8	A325-N	3/8 ,	4.00x 7/16x 14.00			**** tp > db/2 ****				
6	5/8	A325-N	3/8 ,	4.00x 7/16x 17.00			**** tp > db/2 ****				
7	5/8	A325-N	3/8 ,	4.00x 7/16x 20.00			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A325-N	7/16,	4.25x 7/16x 5.25			**** tp > db/2 ****				
3	3/4	A325-N	3/8 ,	4.25x 7/16x 8.25			**** tp > db/2 ****				
4	3/4	A325-N	3/8 ,	4.25x 7/16x 11.25			**** tp > db/2 ****				
5	3/4	A325-N	3/8 ,	4.25x 7/16x 14.25			**** tp > db/2 ****				
6	3/4	A325-N	3/8 ,	4.25x 7/16x 17.25			**** tp > db/2 ****				
7	3/4	A325-N	3/8 ,	4.25x 7/16x 20.25			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A325-N	3/8 ,	4.50x 7/16x 5.75	11.0	36.2	29.5	36.6	37.6	23.1	11.0
3	7/8	A325-N	3/8 ,	4.50x 7/16x 8.75	22.5	55.1	45.2	55.9	57.7	47.4	22.5
4	7/8	A325-N	3/8 ,	4.50x 7/16x11.75	35.3	74.0	60.9	75.2	77.8	74.5	35.3
5	7/8	A325-N	3/8 ,	4.50x 7/16x14.75	49.0	92.9	76.6	94.4	97.9	103.4	49.0
6	7/8	A325-N	3/8 ,	4.50x 7/16x17.75	62.9	111.8	92.3	113.7	118.0	132.8	62.9
7	7/8	A325-N	3/8 ,	4.50x 7/16x20.75	76.7	130.7	108.0	133.0	138.0	161.9	76.7

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A325-N	3/8 ,	4.50x 7/16x 6.00	14.3	37.8	29.5	37.6	40.2	26.4	14.3
3	1.0	A325-N	3/8 ,	4.50x 7/16x 9.00	29.4	56.7	44.2	56.4	60.2	54.2	29.4
4	1.0	A325-N	3/8 ,	4.50x 7/16x12.00	46.1	75.6	59.0	75.2	80.3	85.2	46.1
5	1.0	A325-N	3/8 ,	4.50x 7/16x15.00	64.0	94.5	73.7	94.0	100.4	118.1	64.0
6	1.0	A325-N	3/8 ,	4.50x 7/16x18.00	82.2	113.4	88.5	112.8	120.5	151.8	82.2
7	1.0	A325-N	3/8 ,	4.50x 7/16x21.00	100.2	132.3	103.2	131.6	140.6	185.0	100.2

NOTES:

1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips. Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A325-N	1/2 ,	3.75x 1/2 x 4.50			**** tp > db/2 ****				
3	1/2	A325-N	1/2 ,	3.75x 1/2 x 7.50			**** tp > db/2 ****				
4	1/2	A325-N	7/16,	3.75x 1/2 x 10.50			**** tp > db/2 ****				
5	1/2	A325-N	7/16,	3.75x 1/2 x 13.50			**** tp > db/2 ****				
6	1/2	A325-N	7/16,	3.75x 1/2 x 16.50			**** tp > db/2 ****				
7	1/2	A325-N	7/16,	3.75x 1/2 x 19.50			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A325-N	1/2 ,	4.00x 1/2 x 5.00			**** tp > db/2 ****				
3	5/8	A325-N	7/16,	4.00x 1/2 x 8.00			**** tp > db/2 ****				
4	5/8	A325-N	7/16,	4.00x 1/2 x 11.00			**** tp > db/2 ****				
5	5/8	A325-N	7/16,	4.00x 1/2 x 14.00			**** tp > db/2 ****				
6	5/8	A325-N	7/16,	4.00x 1/2 x 17.00			**** tp > db/2 ****				
7	5/8	A325-N	7/16,	4.00x 1/2 x 20.00			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A325-N	7/16,	4.25x 1/2 x 5.25			**** tp > db/2 ****				
3	3/4	A325-N	7/16,	4.25x 1/2 x 8.25			**** tp > db/2 ****				
4	3/4	A325-N	7/16,	4.25x 1/2 x 11.25			**** tp > db/2 ****				
5	3/4	A325-N	7/16,	4.25x 1/2 x 14.25			**** tp > db/2 ****				
6	3/4	A325-N	7/16,	4.25x 1/2 x 17.25			**** tp > db/2 ****				
7	3/4	A325-N	7/16,	4.25x 1/2 x 20.25			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A325-N	7/16,	4.50x 1/2 x 5.75			**** tp > db/2 ****				
3	7/8	A325-N	7/16,	4.50x 1/2 x 8.75			**** tp > db/2 ****				
4	7/8	A325-N	7/16,	4.50x 1/2 x 11.75			**** tp > db/2 ****				
5	7/8	A325-N	7/16,	4.50x 1/2 x 14.75			**** tp > db/2 ****				
6	7/8	A325-N	7/16,	4.50x 1/2 x 17.75			**** tp > db/2 ****				
7	7/8	A325-N	7/16,	4.50x 1/2 x 20.75			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A325-N	7/16,	4.50x 1/2 x 6.00	14.3	43.2	33.7	43.0	46.9	30.2	14.3
3	1.0	A325-N	7/16,	4.50x 1/2 x 9.00	29.4	64.8	50.6	64.4	70.3	62.0	29.4
4	1.0	A325-N	7/16,	4.50x 1/2 x12.00	46.1	86.4	67.4	85.9	93.7	97.4	46.1
5	1.0	A325-N	7/16,	4.50x 1/2 x15.00	64.0	108.0	84.3	107.4	117.1	135.0	64.0
6	1.0	A325-N	7/16,	4.50x 1/2 x18.00	82.2	129.6	101.1	128.9	140.6	173.5	82.2
7	1.0	A325-N	7/16,	4.50x 1/2 x21.00	100.2	151.2	118.0	150.3	164.0	211.4	100.2

NOTES:

1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips.
   Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A325-N	9/16,	3.75x 9/16x	4.50		**** tp > db/2 ****				
3	1/2	A325-N	9/16,	3.75x 9/16x	7.50		**** tp > db/2 ****				
4	1/2	A325-N	1/2 ,	3.75x 9/16x	10.50		**** tp > db/2 ****				
5	1/2	A325-N	1/2 ,	3.75x 9/16x	13.50		**** tp > db/2 ****				
6	1/2	A325-N	1/2 ,	3.75x 9/16x	16.50		**** tp > db/2 ****				
7	1/2	A325-N	1/2 ,	3.75x 9/16x	19.50		**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A325-N	9/16,	4.00x 9/16x	5.00		**** tp > db/2 ****				
3	5/8	A325-N	1/2 ,	4.00x 9/16x	8.00		**** tp > db/2 ****				
4	5/8	A325-N	1/2 ,	4.00x 9/16x	11.00		**** tp > db/2 ****				
5	5/8	A325-N	1/2 ,	4.00x 9/16x	14.00		**** tp > db/2 ****				
6	5/8	A325-N	1/2 ,	4.00x 9/16x	17.00		**** tp > db/2 ****				
7	5/8	A325-N	1/2 ,	4.00x 9/16x	20.00		**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A325-N	1/2 ,	4.25x 9/16x	5.25		**** tp > db/2 ****				
3	3/4	A325-N	1/2 ,	4.25x 9/16x	8.25		**** tp > db/2 ****				
4	3/4	A325-N	1/2 ,	4.25x 9/16x	11.25		**** tp > db/2 ****				
5	3/4	A325-N	1/2 ,	4.25x 9/16x	14.25		**** tp > db/2 ****				
6	3/4	A325-N	1/2 ,	4.25x 9/16x	17.25		**** tp > db/2 ****				
7	3/4	A325-N	1/2 ,	4.25x 9/16x	20.25		**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A325-N	1/2 ,	4.50x 9/16x	5.75		**** tp > db/2 ****				
3	7/8	A325-N	1/2 ,	4.50x 9/16x	8.75		**** tp > db/2 ****				
4	7/8	A325-N	1/2 ,	4.50x 9/16x	11.75		**** tp > db/2 ****				
5	7/8	A325-N	1/2 ,	4.50x 9/16x	14.75		**** tp > db/2 ****				
6	7/8	A325-N	1/2 ,	4.50x 9/16x	17.75		**** tp > db/2 ****				
7	7/8	A325-N	1/2 ,	4.50x 9/16x	20.75		**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A325-N	1/2 ,	4.50x 9/16x	6.00		**** tp > db/2 ****				
3	1.0	A325-N	1/2 ,	4.50x 9/16x	9.00		**** tp > db/2 ****				
4	1.0	A325-N	1/2 ,	4.50x 9/16x	12.00		**** tp > db/2 ****				
5	1.0	A325-N	1/2 ,	4.50x 9/16x	15.00		**** tp > db/2 ****				
6	1.0	A325-N	1/2 ,	4.50x 9/16x	18.00		**** tp > db/2 ****				
7	1.0	A325-N	1/2 ,	4.50x 9/16x	21.00		**** tp > db/2 ****				

NOTES:

1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips.
   Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A325-X	3/16,	3.75x3/16x 4.50	5.1	12.2	11.0	12.8	12.7	5.7	5.1
3	1/2	A325-X	3/16,	3.75x3/16x 7.50	10.5	20.3	19.0	21.7	22.7	11.6	10.5
4	1/2	A325-X	3/16,	3.75x3/16x10.50	16.5	28.4	26.9	30.6	32.7	18.3	16.5
5	1/2	A325-X	3/16,	3.75x3/16x13.50	22.8	36.5	34.9	39.5	42.7	25.3	22.8
6	1/2	A325-X	3/16,	3.75x3/16x16.50	29.4	44.6	42.8	48.3	52.7	32.5	29.4
7	1/2	A325-X	3/16,	3.75x3/16x19.50	35.8	52.7	50.8	57.2	62.7	39.6	35.8

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A325-X	3/16,	4.00x3/16x 5.00	8.0	13.5	11.8	14.1	15.1	7.1	7.1
3	5/8	A325-X	3/16,	4.00x3/16x 8.00	16.4	21.6	19.4	22.7	25.1	14.5	14.5
4	5/8	A325-X	3/16,	4.00x3/16x11.00	25.7	29.7	26.9	31.4	35.1	22.8	22.8
5	5/8	A325-X	3/16,	4.00x3/16x14.00	35.7	37.8	34.5	40.1	45.2	31.6	31.6
6	5/8	A325-X	3/16,	4.00x3/16x17.00	45.9	45.9	42.0	48.7	55.2	40.7	40.7
7	5/8	A325-X	3/16,	4.00x3/16x20.00	55.9	54.0	49.5	57.4	65.2	49.6	49.5

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A325-X	3/16,	4.25x3/16x 5.25	11.5	14.2	11.8	14.5	16.3	8.5	8.5
3	3/4	A325-X	3/16,	4.25x3/16x 8.25	23.6	22.3	19.0	22.9	26.4	17.4	17.4
4	3/4	A325-X	3/16,	4.25x3/16x11.25	37.1	30.4	26.1	31.4	36.4	27.4	26.1
5	3/4	A325-X	3/16,	4.25x3/16x14.25	51.4	38.5	33.2	39.9	46.4	38.0	33.2
6	3/4	A325-X	3/16,	4.25x3/16x17.25	66.1	46.6	40.4	48.3	56.5	48.8	40.4
7	3/4	A325-X	3/16,	4.25x3/16x20.25	80.5	54.7	47.5	56.8	66.5	59.5	47.5

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A325-X	3/16,	4.50x3/16x 5.75	15.7	15.5	12.6	15.7	18.8	9.9	9.9
3	7/8	A325-X	3/16,	4.50x3/16x 8.75	32.1	23.6	19.4	24.0	28.9	20.3	19.4
4	7/8	A325-X	3/16,	4.50x3/16x11.75	50.5	31.7	26.1	32.2	38.9	31.9	26.1
5	7/8	A325-X	3/16,	4.50x3/16x14.75	70.0	39.8	32.8	40.5	48.9	44.3	32.8
6	7/8	A325-X	3/16,	4.50x3/16x17.75	89.9	47.9	39.6	48.7	59.0	56.9	39.6
7	7/8	A325-X	3/16,	4.50x3/16x20.75	109.6	56.0	46.3	57.0	69.0	69.4	46.3

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A325-X	3/16,	4.50x3/16x 6.00	20.5	16.2	12.6	16.1	20.1	11.3	11.3
3	1.0	A325-X	3/16,	4.50x3/16x 9.00	42.0	24.3	19.0	24.2	30.1	23.2	19.0
4	1.0	A325-X	3/16,	4.50x3/16x12.00	65.9	32.4	25.3	32.2	40.2	36.5	25.3
5	1.0	A325-X	3/16,	4.50x3/16x15.00	91.4	40.5	31.6	40.3	50.2	50.6	31.6
6	1.0	A325-X	3/16,	4.50x3/16x18.00	117.5	48.6	37.9	48.3	60.2	65.1	37.9
7	1.0	A325-X	3/16,	4.50x3/16x21.00	143.1	56.7	44.2	56.4	70.3	79.3	44.2

NOTES:

1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips.
   Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A325-X	1/4 ,	3.75x 1/4 x 4.50	5.1	16.2	14.7	17.1	16.9	7.6	5.1
3	1/2	A325-X	1/4 ,	3.75x 1/4 x 7.50	10.5	27.0	25.3	29.0	30.2	15.5	10.5
4	1/2	A325-X	1/4 ,	3.75x 1/4 x10.50	16.5	37.8	35.9	40.8	43.5	24.3	16.5
5	1/2	A325-X	1/4 ,	3.75x 1/4 x13.50	22.8	48.6	46.5	52.6	56.9	33.7	22.8
6	1/2	A325-X	1/4 ,	3.75x 1/4 x16.50	29.4	59.4	57.1	64.4	70.3	43.4	29.4
7	1/2	A325-X	1/4 ,	3.75x 1/4 x19.50	35.8	70.2	67.7	76.3	83.7	52.9	35.8

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A325-X	1/4 ,	4.00x 1/4 x 5.00	8.0	18.0	15.8	18.8	20.1	9.4	8.0
3	5/8	A325-X	1/4 ,	4.00x 1/4 x 8.00	16.4	28.8	25.8	30.3	33.5	19.4	16.4
4	5/8	A325-X	1/4 ,	4.00x 1/4 x11.00	25.7	39.6	35.9	41.9	46.9	30.4	25.7
5	5/8	A325-X	1/4 ,	4.00x 1/4 x14.00	35.7	50.4	45.9	53.4	60.2	42.2	35.7
6	5/8	A325-X	1/4 ,	4.00x 1/4 x17.00	45.9	61.2	56.0	65.0	73.6	54.2	45.9
7	5/8	A325-X	1/4 ,	4.00x 1/4 x20.00	55.9	72.0	66.1	76.5	87.0	66.1	55.9

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A325-X	1/4 ,	4.25x 1/4 x 5.25	11.5	18.9	15.8	19.3	21.8	11.3	11.3
3	3/4	A325-X	1/4 ,	4.25x 1/4 x 8.25	23.6	29.7	25.3	30.6	35.1	23.2	23.2
4	3/4	A325-X	1/4 ,	4.25x 1/4 x11.25	37.1	40.5	34.8	41.9	48.5	36.5	34.8
5	3/4	A325-X	1/4 ,	4.25x 1/4 x14.25	51.4	51.3	44.3	53.2	61.9	50.6	44.3
6	3/4	A325-X	1/4 ,	4.25x 1/4 x17.25	66.1	62.1	53.8	64.4	75.3	65.1	53.8
7	3/4	A325-X	1/4 ,	4.25x 1/4 x20.25	80.5	72.9	63.3	75.7	88.7	79.3	63.3

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A325-X	1/4 ,	4.50x 1/4 x 5.75	15.7	20.7	16.9	20.9	25.1	13.2	13.2
3	7/8	A325-X	1/4 ,	4.50x 1/4 x 8.75	32.1	31.5	25.8	31.9	38.5	27.1	25.8
4	7/8	A325-X	1/4 ,	4.50x 1/4 x11.75	50.5	42.3	34.8	43.0	51.9	42.6	34.8
5	7/8	A325-X	1/4 ,	4.50x 1/4 x14.75	70.0	53.1	43.8	54.0	65.3	59.1	43.8
6	7/8	A325-X	1/4 ,	4.50x 1/4 x17.75	89.9	63.9	52.7	65.0	78.6	75.9	52.7
7	7/8	A325-X	1/4 ,	4.50x 1/4 x20.75	109.6	74.7	61.7	76.0	92.0	92.5	61.7

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A325-X	1/4 ,	4.50x 1/4 x 6.00	20.5	21.6	16.9	21.5	26.8	15.1	15.1
3	1.0	A325-X	1/4 ,	4.50x 1/4 x 9.00	42.0	32.4	25.3	32.2	40.2	31.0	25.3
4	1.0	A325-X	1/4 ,	4.50x 1/4 x12.00	65.9	43.2	33.7	43.0	53.5	48.7	33.7
5	1.0	A325-X	1/4 ,	4.50x 1/4 x15.00	91.4	54.0	42.1	53.7	66.9	67.5	42.1
6	1.0	A325-X	1/4 ,	4.50x 1/4 x18.00	117.5	64.8	50.6	64.4	80.3	86.7	50.6
7	1.0	A325-X	1/4 ,	4.50x 1/4 x21.00	143.1	75.6	59.0	75.2	93.7	105.7	59.0

NOTES:

1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips.
   Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A325-X	5/16,	3.75x 5/16x 4.50			**** tp > db/2 ****				
3	1/2	A325-X	5/16,	3.75x 5/16x 7.50			**** tp > db/2 ****				
4	1/2	A325-X	5/16,	3.75x 5/16x 10.50			**** tp > db/2 ****				
5	1/2	A325-X	5/16,	3.75x 5/16x 13.50			**** tp > db/2 ****				
6	1/2	A325-X	5/16,	3.75x 5/16x 16.50			**** tp > db/2 ****				
7	1/2	A325-X	5/16,	3.75x 5/16x 19.50			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A325-X	5/16,	4.00x 5/16x 5.00	8.0	22.5	19.7	23.4	25.2	11.8	8.0
3	5/8	A325-X	5/16,	4.00x 5/16x 8.00	16.4	36.0	32.3	37.9	41.9	24.2	16.4
4	5/8	A325-X	5/16,	4.00x 5/16x11.00	25.7	49.5	44.9	52.3	58.6	38.0	25.7
5	5/8	A325-X	5/16,	4.00x 5/16x14.00	35.7	63.0	57.4	66.8	75.3	52.7	35.7
6	5/8	A325-X	5/16,	4.00x 5/16x17.00	45.9	76.5	70.0	81.2	92.0	67.8	45.9
7	5/8	A325-X	5/16,	4.00x 5/16x20.00	55.9	90.0	82.6	95.7	108.7	82.6	55.9

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A325-X	5/16,	4.25x 5/16x 5.25	11.5	23.6	19.7	24.1	27.2	14.2	11.5
3	3/4	A325-X	5/16,	4.25x 5/16x 8.25	23.6	37.1	31.6	38.2	43.9	29.0	23.6
4	3/4	A325-X	5/16,	4.25x 5/16x11.25	37.1	50.6	43.5	52.3	60.7	45.6	37.1
5	3/4	A325-X	5/16,	4.25x 5/16x14.25	51.4	64.1	55.4	66.4	77.4	63.3	51.4
6	3/4	A325-X	5/16,	4.25x 5/16x17.25	66.1	77.6	67.3	80.5	94.1	81.3	66.1
7	3/4	A325-X	5/16,	4.25x 5/16x20.25	80.5	91.1	79.2	94.6	110.8	99.1	79.2

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A325-X	5/16,	4.50x 5/16x 5.75	15.7	25.9	21.1	26.2	31.4	16.5	15.7
3	7/8	A325-X	5/16,	4.50x 5/16x 8.75	32.1	39.4	32.3	39.9	48.1	33.9	32.1
4	7/8	A325-X	5/16,	4.50x 5/16x11.75	50.5	52.9	43.5	53.7	64.8	53.2	43.5
5	7/8	A325-X	5/16,	4.50x 5/16x14.75	70.0	66.4	54.7	67.5	81.6	73.8	54.7
6	7/8	A325-X	5/16,	4.50x 5/16x17.75	89.9	79.9	65.9	81.2	98.3	94.9	65.9
7	7/8	A325-X	5/16,	4.50x 5/16x20.75	109.6	93.4	77.1	95.0	115.0	115.6	77.1

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A325-X	5/16,	4.50x 5/16x 6.00	20.5	27.0	21.1	26.8	33.5	18.9	18.9
3	1.0	A325-X	5/16,	4.50x 5/16x 9.00	42.0	40.5	31.6	40.3	50.2	38.7	31.6
4	1.0	A325-X	5/16,	4.50x 5/16x12.00	65.9	54.0	42.1	53.7	66.9	60.8	42.1
5	1.0	A325-X	5/16,	4.50x 5/16x15.00	91.4	67.5	52.7	67.1	83.7	84.4	52.7
6	1.0	A325-X	5/16,	4.50x 5/16x18.00	117.5	81.0	63.2	80.5	100.4	108.4	63.2
7	1.0	A325-X	5/16,	4.50x 5/16x21.00	143.1	94.5	73.7	94.0	117.1	132.1	73.7

NOTES:
1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips.
   Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A325-X	3/8 ,	3.75x 3/8 x 4.50			**** tp > db/2 ****				
3	1/2	A325-X	3/8 ,	3.75x 3/8 x 7.50			**** tp > db/2 ****				
4	1/2	A325-X	3/8 ,	3.75x 3/8 x 10.50			**** tp > db/2 ****				
5	1/2	A325-X	3/8 ,	3.75x 3/8 x 13.50			**** tp > db/2 ****				
6	1/2	A325-X	3/8 ,	3.75x 3/8 x 16.50			**** tp > db/2 ****				
7	1/2	A325-X	3/8 ,	3.75x 3/8 x 19.50			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A325-X	3/8 ,	4.00x 3/8 x 5.00			**** tp > db/2 ****				
3	5/8	A325-X	3/8 ,	4.00x 3/8 x 8.00			**** tp > db/2 ****				
4	5/8	A325-X	3/8 ,	4.00x 3/8 x 11.00			**** tp > db/2 ****				
5	5/8	A325-X	3/8 ,	4.00x 3/8 x 14.00			**** tp > db/2 ****				
6	5/8	A325-X	5/16,	4.00x 3/8 x 17.00			**** tp > db/2 ****				
7	5/8	A325-X	5/16,	4.00x 3/8 x 20.00			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A325-X	3/8 ,	4.25x 3/8 x 5.25	11.5	28.4	23.7	29.0	32.7	17.0	11.5
3	3/4	A325-X	3/8 ,	4.25x 3/8 x 8.25	23.6	44.6	37.9	45.9	52.7	34.9	23.6
4	3/4	A325-X	3/8 ,	4.25x 3/8 x11.25	37.1	60.8	52.2	62.8	72.8	54.8	37.1
5	3/4	A325-X	5/16,	4.25x 3/8 x14.25	51.4	77.0	66.5	79.7	77.4	75.9	51.4
6	3/4	A325-X	5/16,	4.25x 3/8 x17.25	66.1	93.2	80.7	96.7	94.1	97.6	66.1
7	3/4	A325-X	5/16,	4.25x 3/8 x20.25	80.5	109.4	95.0	113.6	110.8	118.9	80.5

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A325-X	5/16,	4.50x 3/8 x 5.75	15.7	31.1	25.3	31.4	31.4	19.8	15.7
3	7/8	A325-X	5/16,	4.50x 3/8 x 8.75	32.1	47.3	38.7	47.9	48.1	40.7	32.1
4	7/8	A325-X	5/16,	4.50x 3/8 x11.75	50.5	63.5	52.2	64.4	64.8	63.9	50.5
5	7/8	A325-X	5/16,	4.50x 3/8 x14.75	70.0	79.7	65.7	81.0	81.6	88.6	65.7
6	7/8	A325-X	5/16,	4.50x 3/8 x17.75	89.9	95.9	79.1	97.5	98.3	113.8	79.1
7	7/8	A325-X	5/16,	4.50x 3/8 x20.75	109.6	112.1	92.6	114.0	115.0	138.7	92.6

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A325-X	5/16,	4.50x 3/8 x 6.00	20.5	32.4	25.3	32.2	33.5	22.7	20.5
3	1.0	A325-X	5/16,	4.50x 3/8 x 9.00	42.0	48.6	37.9	48.3	50.2	46.5	37.9
4	1.0	A325-X	5/16,	4.50x 3/8 x12.00	65.9	64.8	50.6	64.4	66.9	73.0	50.6
5	1.0	A325-X	5/16,	4.50x 3/8 x15.00	91.4	81.0	63.2	80.5	83.7	101.2	63.2
6	1.0	A325-X	5/16,	4.50x 3/8 x18.00	117.5	97.2	75.9	96.7	100.4	130.1	75.9
7	1.0	A325-X	5/16,	4.50x 3/8 x21.00	143.1	113.4	88.5	112.8	117.1	158.6	88.5

NOTES:
1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips.
   Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A325-X	7/16,	3.75x 7/16x 4.50			**** tp > db/2 ****				
3	1/2	A325-X	7/16,	3.75x 7/16x 7.50			**** tp > db/2 ****				
4	1/2	A325-X	7/16,	3.75x 7/16x 10.50			**** tp > db/2 ****				
5	1/2	A325-X	3/8 ,	3.75x 7/16x 13.50			**** tp > db/2 ****				
6	1/2	A325-X	3/8 ,	3.75x 7/16x 16.50			**** tp > db/2 ****				
7	1/2	A325-X	3/8 ,	3.75x 7/16x 19.50			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A325-X	7/16,	4.00x 7/16x 5.00			**** tp > db/2 ****				
3	5/8	A325-X	7/16,	4.00x 7/16x 8.00			**** tp > db/2 ****				
4	5/8	A325-X	3/8 ,	4.00x 7/16x 11.00			**** tp > db/2 ****				
5	5/8	A325-X	3/8 ,	4.00x 7/16x 14.00			**** tp > db/2 ****				
6	5/8	A325-X	3/8 ,	4.00x 7/16x 17.00			**** tp > db/2 ****				
7	5/8	A325-X	3/8 ,	4.00x 7/16x 20.00			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A325-X	7/16,	4.25x 7/16x 5.25			**** tp > db/2 ****				
3	3/4	A325-X	3/8 ,	4.25x 7/16x 8.25			**** tp > db/2 ****				
4	3/4	A325-X	3/8 ,	4.25x 7/16x 11.25			**** tp > db/2 ****				
5	3/4	A325-X	3/8 ,	4.25x 7/16x 14.25			**** tp > db/2 ****				
6	3/4	A325-X	3/8 ,	4.25x 7/16x 17.25			**** tp > db/2 ****				
7	3/4	A325-X	3/8 ,	4.25x 7/16x 20.25			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A325-X	3/8 ,	4.50x 7/16x 5.75	15.7	36.2	29.5	36.6	37.6	23.1	15.7
3	7/8	A325-X	3/8 ,	4.50x 7/16x 8.75	32.1	55.1	45.2	55.9	57.7	47.4	32.1
4	7/8	A325-X	3/8 ,	4.50x 7/16x11.75	50.5	74.0	60.9	75.2	77.8	74.5	50.5
5	7/8	A325-X	3/8 ,	4.50x 7/16x14.75	70.0	92.9	76.6	94.4	97.9	103.4	70.0
6	7/8	A325-X	3/8 ,	4.50x 7/16x17.75	89.9	111.8	92.3	113.7	118.0	132.8	89.9
7	7/8	A325-X	3/8 ,	4.50x 7/16x20.75	109.6	130.7	108.0	133.0	138.0	161.9	108.0

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A325-X	3/8 ,	4.50x 7/16x 6.00	20.5	37.8	29.5	37.6	40.2	26.4	20.5
3	1.0	A325-X	3/8 ,	4.50x 7/16x 9.00	42.0	56.7	44.2	56.4	60.2	54.2	42.0
4	1.0	A325-X	3/8 ,	4.50x 7/16x12.00	65.9	75.6	59.0	75.2	80.3	85.2	59.0
5	1.0	A325-X	3/8 ,	4.50x 7/16x15.00	91.4	94.5	73.7	94.0	100.4	118.1	73.7
6	1.0	A325-X	3/8 ,	4.50x 7/16x18.00	117.5	113.4	88.5	112.8	120.5	151.8	88.5
7	1.0	A325-X	3/8 ,	4.50x 7/16x21.00	143.1	132.3	103.2	131.6	140.6	185.0	103.2

NOTES:

1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips. Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A325-X	1/2 ,	3.75x 1/2 x 4.50		**** tp > db/2 ****					
3	1/2	A325-X	1/2 ,	3.75x 1/2 x 7.50		**** tp > db/2 ****					
4	1/2	A325-X	7/16,	3.75x 1/2 x 10.50		**** tp > db/2 ****					
5	1/2	A325-X	7/16,	3.75x 1/2 x 13.50		**** tp > db/2 ****					
6	1/2	A325-X	7/16,	3.75x 1/2 x 16.50		**** tp > db/2 ****					
7	1/2	A325-X	7/16,	3.75x 1/2 x 19.50		**** tp > db/2 ****					

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A325-X	1/2 ,	4.00x 1/2 x 5.00		**** tp > db/2 ****					
3	5/8	A325-X	7/16,	4.00x 1/2 x 8.00		**** tp > db/2 ****					
4	5/8	A325-X	7/16,	4.00x 1/2 x 11.00		**** tp > db/2 ****					
5	5/8	A325-X	7/16,	4.00x 1/2 x 14.00		**** tp > db/2 ****					
6	5/8	A325-X	7/16,	4.00x 1/2 x 17.00		**** tp > db/2 ****					
7	5/8	A325-X	7/16,	4.00x 1/2 x 20.00		**** tp > db/2 ****					

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A325-X	7/16,	4.25x 1/2 x 5.25		**** tp > db/2 ****					
3	3/4	A325-X	7/16,	4.25x 1/2 x 8.25		**** tp > db/2 ****					
4	3/4	A325-X	7/16,	4.25x 1/2 x 11.25		**** tp > db/2 ****					
5	3/4	A325-X	7/16,	4.25x 1/2 x 14.25		**** tp > db/2 ****					
6	3/4	A325-X	7/16,	4.25x 1/2 x 17.25		**** tp > db/2 ****					
7	3/4	A325-X	7/16,	4.25x 1/2 x 20.25		**** tp > db/2 ****					

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A325-X	7/16,	4.50x 1/2 x 5.75		**** tp > db/2 ****					
3	7/8	A325-X	7/16,	4.50x 1/2 x 8.75		**** tp > db/2 ****					
4	7/8	A325-X	7/16,	4.50x 1/2 x 11.75		**** tp > db/2 ****					
5	7/8	A325-X	7/16,	4.50x 1/2 x 14.75		**** tp > db/2 ****					
6	7/8	A325-X	7/16,	4.50x 1/2 x 17.75		**** tp > db/2 ****					
7	7/8	A325-X	7/16,	4.50x 1/2 x 20.75		**** tp > db/2 ****					

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A325-X	7/16,	4.50x 1/2 x 6.00	20.5	43.2	33.7	43.0	46.9	30.2	20.5
3	1.0	A325-X	7/16,	4.50x 1/2 x 9.00	42.0	64.8	50.6	64.4	70.3	62.0	42.0
4	1.0	A325-X	7/16,	4.50x 1/2 x12.00	65.9	86.4	67.4	85.9	93.7	97.4	65.9
5	1.0	A325-X	7/16,	4.50x 1/2 x15.00	91.4	108.0	84.3	107.4	117.1	135.0	84.3
6	1.0	A325-X	7/16,	4.50x 1/2 x18.00	117.5	129.6	101.1	128.9	140.6	173.5	101.1
7	1.0	A325-X	7/16,	4.50x 1/2 x21.00	143.1	151.2	118.0	150.3	164.0	211.4	118.0

NOTES:
1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips. Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A325-X	9/16,	3.75x 9/16x	4.50		**** tp > db/2 ****				
3	1/2	A325-X	9/16,	3.75x 9/16x	7.50		**** tp > db/2 ****				
4	1/2	A325-X	1/2 ,	3.75x 9/16x	10.50		**** tp > db/2 ****				
5	1/2	A325-X	1/2 ,	3.75x 9/16x	13.50		**** tp > db/2 ****				
6	1/2	A325-X	1/2 ,	3.75x 9/16x	16.50		**** tp > db/2 ****				
7	1/2	A325-X	1/2 ,	3.75x 9/16x	19.50		**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A325-X	9/16,	4.00x 9/16x	5.00		**** tp > db/2 ****				
3	5/8	A325-X	1/2 ,	4.00x 9/16x	8.00		**** tp > db/2 ****				
4	5/8	A325-X	1/2 ,	4.00x 9/16x	11.00		**** tp > db/2 ****				
5	5/8	A325-X	1/2 ,	4.00x 9/16x	14.00		**** tp > db/2 ****				
6	5/8	A325-X	1/2 ,	4.00x 9/16x	17.00		**** tp > db/2 ****				
7	5/8	A325-X	1/2 ,	4.00x 9/16x	20.00		**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A325-X	1/2 ,	4.25x 9/16x	5.25		**** tp > db/2 ****				
3	3/4	A325-X	1/2 ,	4.25x 9/16x	8.25		**** tp > db/2 ****				
4	3/4	A325-X	1/2 ,	4.25x 9/16x	11.25		**** tp > db/2 ****				
5	3/4	A325-X	1/2 ,	4.25x 9/16x	14.25		**** tp > db/2 ****				
6	3/4	A325-X	1/2 ,	4.25x 9/16x	17.25		**** tp > db/2 ****				
7	3/4	A325-X	1/2 ,	4.25x 9/16x	20.25		**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A325-X	1/2 ,	4.50x 9/16x	5.75		**** tp > db/2 ****				
3	7/8	A325-X	1/2 ,	4.50x 9/16x	8.75		**** tp > db/2 ****				
4	7/8	A325-X	1/2 ,	4.50x 9/16x	11.75		**** tp > db/2 ****				
5	7/8	A325-X	1/2 ,	4.50x 9/16x	14.75		**** tp > db/2 ****				
6	7/8	A325-X	1/2 ,	4.50x 9/16x	17.75		**** tp > db/2 ****				
7	7/8	A325-X	1/2 ,	4.50x 9/16x	20.75		**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A325-X	1/2 ,	4.50x 9/16x	6.00		**** tp > db/2 ****				
3	1.0	A325-X	1/2 ,	4.50x 9/16x	9.00		**** tp > db/2 ****				
4	1.0	A325-X	1/2 ,	4.50x 9/16x	12.00		**** tp > db/2 ****				
5	1.0	A325-X	1/2 ,	4.50x 9/16x	15.00		**** tp > db/2 ****				
6	1.0	A325-X	1/2 ,	4.50x 9/16x	18.00		**** tp > db/2 ****				
7	1.0	A325-X	1/2 ,	4.50x 9/16x	21.00		**** tp > db/2 ****				

NOTES:

1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips.
   Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A490-N	3/16,	3.75x3/16x 4.50	4.8	12.2	11.0	12.8	12.7	5.7	4.8
3	1/2	A490-N	3/16,	3.75x3/16x 7.50	9.8	20.3	19.0	21.7	22.7	11.6	9.8
4	1/2	A490-N	3/16,	3.75x3/16x10.50	15.4	28.4	26.9	30.6	32.7	18.3	15.4
5	1/2	A490-N	3/16,	3.75x3/16x13.50	21.3	36.5	34.9	39.5	42.7	25.3	21.3
6	1/2	A490-N	3/16,	3.75x3/16x16.50	27.4	44.6	42.8	48.3	52.7	32.5	27.4
7	1/2	A490-N	3/16,	3.75x3/16x19.50	33.4	52.7	50.8	57.2	62.7	39.6	33.4

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A490-N	3/16,	4.00x3/16x 5.00	7.5	13.5	11.8	14.1	15.1	7.1	7.1
3	5/8	A490-N	3/16,	4.00x3/16x 8.00	15.3	21.6	19.4	22.7	25.1	14.5	14.5
4	5/8	A490-N	3/16,	4.00x3/16x11.00	24.0	29.7	26.9	31.4	35.1	22.8	22.8
5	5/8	A490-N	3/16,	4.00x3/16x14.00	33.3	37.8	34.5	40.1	45.2	31.6	31.6
6	5/8	A490-N	3/16,	4.00x3/16x17.00	42.8	45.9	42.0	48.7	55.2	40.7	40.7
7	5/8	A490-N	3/16,	4.00x3/16x20.00	52.2	54.0	49.5	57.4	65.2	49.6	49.5

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A490-N	3/16,	4.25x3/16x 5.25	10.7	14.2	11.8	14.5	16.3	8.5	8.5
3	3/4	A490-N	3/16,	4.25x3/16x 8.25	22.0	22.3	19.0	22.9	26.4	17.4	17.4
4	3/4	A490-N	3/16,	4.25x3/16x11.25	34.6	30.4	26.1	31.4	36.4	27.4	26.1
5	3/4	A490-N	3/16,	4.25x3/16x14.25	48.0	38.5	33.2	39.9	46.4	38.0	33.2
6	3/4	A490-N	3/16,	4.25x3/16x17.25	61.7	46.6	40.4	48.3	56.5	48.8	40.4
7	3/4	A490-N	3/16,	4.25x3/16x20.25	75.1	54.7	47.5	56.8	66.5	59.5	47.5

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A490-N	3/16,	4.50x3/16x 5.75	14.6	15.5	12.6	15.7	18.8	9.9	9.9
3	7/8	A490-N	3/16,	4.50x3/16x 8.75	30.0	23.6	19.4	24.0	28.9	20.3	19.4
4	7/8	A490-N	3/16,	4.50x3/16x11.75	47.1	31.7	26.1	32.2	38.9	31.9	26.1
5	7/8	A490-N	3/16,	4.50x3/16x14.75	65.3	39.8	32.8	40.5	48.9	44.3	32.8
6	7/8	A490-N	3/16,	4.50x3/16x17.75	83.9	47.9	39.6	48.7	59.0	56.9	39.6
7	7/8	A490-N	3/16,	4.50x3/16x20.75	102.3	56.0	46.3	57.0	69.0	69.4	46.3

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A490-N	3/16,	4.50x3/16x 6.00	19.1	16.2	12.6	16.1	20.1	11.3	11.3
3	1.0	A490-N	3/16,	4.50x3/16x 9.00	39.2	24.3	19.0	24.2	30.1	23.2	19.0
4	1.0	A490-N	3/16,	4.50x3/16x12.00	61.5	32.4	25.3	32.2	40.2	36.5	25.3
5	1.0	A490-N	3/16,	4.50x3/16x15.00	85.3	40.5	31.6	40.3	50.2	50.6	31.6
6	1.0	A490-N	3/16,	4.50x3/16x18.00	109.6	48.6	37.9	48.3	60.2	65.1	37.9
7	1.0	A490-N	3/16,	4.50x3/16x21.00	133.6	56.7	44.2	56.4	70.3	79.3	44.2

NOTES:
1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips. Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A490-N	1/4 ,	3.75x 1/4 x 4.50	4.8	16.2	14.7	17.1	16.9	7.6	4.8
3	1/2	A490-N	1/4 ,	3.75x 1/4 x 7.50	9.8	27.0	25.3	29.0	30.2	15.5	9.8
4	1/2	A490-N	1/4 ,	3.75x 1/4 x10.50	15.4	37.8	35.9	40.8	43.5	24.3	15.4
5	1/2	A490-N	1/4 ,	3.75x 1/4 x13.50	21.3	48.6	46.5	52.6	56.9	33.7	21.3
6	1/2	A490-N	1/4 ,	3.75x 1/4 x16.50	27.4	59.4	57.1	64.4	70.3	43.4	27.4
7	1/2	A490-N	1/4 ,	3.75x 1/4 x19.50	33.4	70.2	67.7	76.3	83.7	52.9	33.4

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A490-N	1/4 ,	4.00x 1/4 x 5.00	7.5	18.0	15.8	18.8	20.1	9.4	7.5
3	5/8	A490-N	1/4 ,	4.00x 1/4 x 8.00	15.3	28.8	25.8	30.3	33.5	19.4	15.3
4	5/8	A490-N	1/4 ,	4.00x 1/4 x11.00	24.0	39.6	35.9	41.9	46.9	30.4	24.0
5	5/8	A490-N	1/4 ,	4.00x 1/4 x14.00	33.3	50.4	45.9	53.4	60.2	42.2	33.3
6	5/8	A490-N	1/4 ,	4.00x 1/4 x17.00	42.8	61.2	56.0	65.0	73.6	54.2	42.8
7	5/8	A490-N	1/4 ,	4.00x 1/4 x20.00	52.2	72.0	66.1	76.5	87.0	66.1	52.2

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A490-N	1/4 ,	4.25x 1/4 x 5.25	10.7	18.9	15.8	19.3	21.8	11.3	10.7
3	3/4	A490-N	1/4 ,	4.25x 1/4 x 8.25	22.0	29.7	25.3	30.6	35.1	23.2	22.0
4	3/4	A490-N	1/4 ,	4.25x 1/4 x11.25	34.6	40.5	34.8	41.9	48.5	36.5	34.6
5	3/4	A490-N	1/4 ,	4.25x 1/4 x14.25	48.0	51.3	44.3	53.2	61.9	50.6	44.3
6	3/4	A490-N	1/4 ,	4.25x 1/4 x17.25	61.7	62.1	53.8	64.4	75.3	65.1	53.8
7	3/4	A490-N	1/4 ,	4.25x 1/4 x20.25	75.1	72.9	63.3	75.7	88.7	79.3	63.3

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A490-N	1/4 ,	4.50x 1/4 x 5.75	14.6	20.7	16.9	20.9	25.1	13.2	13.2
3	7/8	A490-N	1/4 ,	4.50x 1/4 x 8.75	30.0	31.5	25.8	31.9	38.5	27.1	25.8
4	7/8	A490-N	1/4 ,	4.50x 1/4 x11.75	47.1	42.3	34.8	43.0	51.9	42.6	34.8
5	7/8	A490-N	1/4 ,	4.50x 1/4 x14.75	65.3	53.1	43.8	54.0	65.3	59.1	43.8
6	7/8	A490-N	1/4 ,	4.50x 1/4 x17.75	83.9	63.9	52.7	65.0	78.6	75.9	52.7
7	7/8	A490-N	1/4 ,	4.50x 1/4 x20.75	102.3	74.7	61.7	76.0	92.0	92.5	61.7

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A490-N	1/4 ,	4.50x 1/4 x 6.00	19.1	21.6	16.9	21.5	26.8	15.1	15.1
3	1.0	A490-N	1/4 ,	4.50x 1/4 x 9.00	39.2	32.4	25.3	32.2	40.2	31.0	25.3
4	1.0	A490-N	1/4 ,	4.50x 1/4 x12.00	61.5	43.2	33.7	43.0	53.5	48.7	33.7
5	1.0	A490-N	1/4 ,	4.50x 1/4 x15.00	85.3	54.0	42.1	53.7	66.9	67.5	42.1
6	1.0	A490-N	1/4 ,	4.50x 1/4 x18.00	109.6	64.8	50.6	64.4	80.3	86.7	50.6
7	1.0	A490-N	1/4 ,	4.50x 1/4 x21.00	133.6	75.6	59.0	75.2	93.7	105.7	59.0

NOTES:

1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips.
   Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A490-N	5/16,	3.75x 5/16x 4.50	**** tp > db/2 ****						
3	1/2	A490-N	5/16,	3.75x 5/16x 7.50	**** tp > db/2 ****						
4	1/2	A490-N	5/16,	3.75x 5/16x 10.50	**** tp > db/2 ****						
5	1/2	A490-N	5/16,	3.75x 5/16x 13.50	**** tp > db/2 ****						
6	1/2	A490-N	5/16,	3.75x 5/16x 16.50	**** tp > db/2 ****						
7	1/2	A490-N	5/16,	3.75x 5/16x 19.50	**** tp > db/2 ****						

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A490-N	5/16,	4.00x 5/16x 5.00	7.5	22.5	19.7	23.4	25.2	11.8	7.5
3	5/8	A490-N	5/16,	4.00x 5/16x 8.00	15.3	36.0	32.3	37.9	41.9	24.2	15.3
4	5/8	A490-N	5/16,	4.00x 5/16x11.00	24.0	49.5	44.9	52.3	58.6	38.0	24.0
5	5/8	A490-N	5/16,	4.00x 5/16x14.00	33.3	63.0	57.4	66.8	75.3	52.7	33.3
6	5/8	A490-N	5/16,	4.00x 5/16x17.00	42.8	76.5	70.0	81.2	92.0	67.8	42.8
7	5/8	A490-N	5/16,	4.00x 5/16x20.00	52.2	90.0	82.6	95.7	108.7	82.6	52.2

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A490-N	5/16,	4.25x 5/16x 5.25	10.7	23.6	19.7	24.1	27.2	14.2	10.7
3	3/4	A490-N	5/16,	4.25x 5/16x 8.25	22.0	37.1	31.6	38.2	43.9	29.0	22.0
4	3/4	A490-N	5/16,	4.25x 5/16x11.25	34.6	50.6	43.5	52.3	60.7	45.6	34.6
5	3/4	A490-N	5/16,	4.25x 5/16x14.25	48.0	64.1	55.4	66.4	77.4	63.3	48.0
6	3/4	A490-N	5/16,	4.25x 5/16x17.25	61.7	77.6	67.3	80.5	94.1	81.3	61.7
7	3/4	A490-N	5/16,	4.25x 5/16x20.25	75.1	91.1	79.2	94.6	110.8	99.1	75.1

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A490-N	5/16,	4.50x 5/16x 5.75	14.6	25.9	21.1	26.2	31.4	16.5	14.6
3	7/8	A490-N	5/16,	4.50x 5/16x 8.75	30.0	39.4	32.3	39.9	48.1	33.9	30.0
4	7/8	A490-N	5/16,	4.50x 5/16x11.75	47.1	52.9	43.5	53.7	64.8	53.2	43.5
5	7/8	A490-N	5/16,	4.50x 5/16x14.75	65.3	66.4	54.7	67.5	81.6	73.8	54.7
6	7/8	A490-N	5/16,	4.50x 5/16x17.75	83.9	79.9	65.9	81.2	98.3	94.9	65.9
7	7/8	A490-N	5/16,	4.50x 5/16x20.75	102.3	93.4	77.1	95.0	115.0	115.6	77.1

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A490-N	5/16,	4.50x 5/16x 6.00	19.1	27.0	21.1	26.8	33.5	18.9	18.9
3	1.0	A490-N	5/16,	4.50x 5/16x 9.00	39.2	40.5	31.6	40.3	50.2	38.7	31.6
4	1.0	A490-N	5/16,	4.50x 5/16x12.00	61.5	54.0	42.1	53.7	66.9	60.8	42.1
5	1.0	A490-N	5/16,	4.50x 5/16x15.00	85.3	67.5	52.7	67.1	83.7	84.4	52.7
6	1.0	A490-N	5/16,	4.50x 5/16x18.00	109.6	81.0	63.2	80.5	100.4	108.4	63.2
7	1.0	A490-N	5/16,	4.50x 5/16x21.00	133.6	94.5	73.7	94.0	117.1	132.1	73.7

NOTES:

1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips.
   Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A490-N	3/8 ,	3.75x 3/8 x 4.50			**** tp > db/2 ****				
3	1/2	A490-N	3/8 ,	3.75x 3/8 x 7.50			**** tp > db/2 ****				
4	1/2	A490-N	3/8 ,	3.75x 3/8 x 10.50			**** tp > db/2 ****				
5	1/2	A490-N	3/8 ,	3.75x 3/8 x 13.50			**** tp > db/2 ****				
6	1/2	A490-N	3/8 ,	3.75x 3/8 x 16.50			**** tp > db/2 ****				
7	1/2	A490-N	3/8 ,	3.75x 3/8 x 19.50			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A490-N	3/8 ,	4.00x 3/8 x 5.00			**** tp > db/2 ****				
3	5/8	A490-N	3/8 ,	4.00x 3/8 x 8.00			**** tp > db/2 ****				
4	5/8	A490-N	3/8 ,	4.00x 3/8 x 11.00			**** tp > db/2 ****				
5	5/8	A490-N	3/8 ,	4.00x 3/8 x 14.00			**** tp > db/2 ****				
6	5/8	A490-N	5/16,	4.00x 3/8 x 17.00			**** tp > db/2 ****				
7	5/8	A490-N	5/16,	4.00x 3/8 x 20.00			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A490-N	3/8 ,	4.25x 3/8 x 5.25	10.7	28.4	23.7	29.0	32.7	17.0	10.7
3	3/4	A490-N	3/8 ,	4.25x 3/8 x 8.25	22.0	44.6	37.9	45.9	52.7	34.9	22.0
4	3/4	A490-N	3/8 ,	4.25x 3/8 x11.25	34.6	60.8	52.2	62.8	72.8	54.8	34.6
5	3/4	A490-N	5/16,	4.25x 3/8 x14.25	48.0	77.0	66.5	79.7	77.4	75.9	48.0
6	3/4	A490-N	5/16,	4.25x 3/8 x17.25	61.7	93.2	80.7	96.7	94.1	97.6	61.7
7	3/4	A490-N	5/16,	4.25x 3/8 x20.25	75.1	109.4	95.0	113.6	110.8	118.9	75.1

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A490-N	5/16,	4.50x 3/8 x 5.75	14.6	31.1	25.3	31.4	31.4	19.8	14.6
3	7/8	A490-N	5/16,	4.50x 3/8 x 8.75	30.0	47.3	38.7	47.9	48.1	40.7	30.0
4	7/8	A490-N	5/16,	4.50x 3/8 x11.75	47.1	63.5	52.2	64.4	64.8	63.9	47.1
5	7/8	A490-N	5/16,	4.50x 3/8 x14.75	65.3	79.7	65.7	81.0	81.6	88.6	65.3
6	7/8	A490-N	5/16,	4.50x 3/8 x17.75	83.9	95.9	79.1	97.5	98.3	113.8	79.1
7	7/8	A490-N	5/16,	4.50x 3/8 x20.75	102.3	112.1	92.6	114.0	115.0	138.7	92.6

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A490-N	5/16,	4.50x 3/8 x 6.00	19.1	32.4	25.3	32.2	33.5	22.7	19.1
3	1.0	A490-N	5/16,	4.50x 3/8 x 9.00	39.2	48.6	37.9	48.3	50.2	46.5	37.9
4	1.0	A490-N	5/16,	4.50x 3/8 x12.00	61.5	64.8	50.6	64.4	66.9	73.0	50.6
5	1.0	A490-N	5/16,	4.50x 3/8 x15.00	85.3	81.0	63.2	80.5	83.7	101.2	63.2
6	1.0	A490-N	5/16,	4.50x 3/8 x18.00	109.6	97.2	75.9	96.7	100.4	130.1	75.9
7	1.0	A490-N	5/16,	4.50x 3/8 x21.00	133.6	113.4	88.5	112.8	117.1	158.6	88.5

NOTES:

1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips.
   Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A490-N	7/16,	3.75x 7/16x 4.50			**** tp > db/2 ****				
3	1/2	A490-N	7/16,	3.75x 7/16x 7.50			**** tp > db/2 ****				
4	1/2	A490-N	7/16,	3.75x 7/16x 10.50			**** tp > db/2 ****				
5	1/2	A490-N	3/8 ,	3.75x 7/16x 13.50			**** tp > db/2 ****				
6	1/2	A490-N	3/8 ,	3.75x 7/16x 16.50			**** tp > db/2 ****				
7	1/2	A490-N	3/8 ,	3.75x 7/16x 19.50			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A490-N	7/16,	4.00x 7/16x 5.00			**** tp > db/2 ****				
3	5/8	A490-N	7/16,	4.00x 7/16x 8.00			**** tp > db/2 ****				
4	5/8	A490-N	3/8 ,	4.00x 7/16x 11.00			**** tp > db/2 ****				
5	5/8	A490-N	3/8 ,	4.00x 7/16x 14.00			**** tp > db/2 ****				
6	5/8	A490-N	3/8 ,	4.00x 7/16x 17.00			**** tp > db/2 ****				
7	5/8	A490-N	3/8 ,	4.00x 7/16x 20.00			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A490-N	7/16,	4.25x 7/16x 5.25			**** tp > db/2 ****				
3	3/4	A490-N	3/8 ,	4.25x 7/16x 8.25			**** tp > db/2 ****				
4	3/4	A490-N	3/8 ,	4.25x 7/16x 11.25			**** tp > db/2 ****				
5	3/4	A490-N	3/8 ,	4.25x 7/16x 14.25			**** tp > db/2 ****				
6	3/4	A490-N	3/8 ,	4.25x 7/16x 17.25			**** tp > db/2 ****				
7	3/4	A490-N	3/8 ,	4.25x 7/16x 20.25			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A490-N	3/8 ,	4.50x 7/16x 5.75	14.6	36.2	29.5	36.6	37.6	23.1	14.6
3	7/8	A490-N	3/8 ,	4.50x 7/16x 8.75	30.0	55.1	45.2	55.9	57.7	47.4	30.0
4	7/8	A490-N	3/8 ,	4.50x 7/16x11.75	47.1	74.0	60.9	75.2	77.8	74.5	47.1
5	7/8	A490-N	3/8 ,	4.50x 7/16x14.75	65.3	92.9	76.6	94.4	97.9	103.4	65.3
6	7/8	A490-N	3/8 ,	4.50x 7/16x17.75	83.9	111.8	92.3	113.7	118.0	132.8	83.9
7	7/8	A490-N	3/8 ,	4.50x 7/16x20.75	102.3	130.7	108.0	133.0	138.0	161.9	102.3

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A490-N	3/8 ,	4.50x 7/16x 6.00	19.1	37.8	29.5	37.6	40.2	26.4	19.1
3	1.0	A490-N	3/8 ,	4.50x 7/16x 9.00	39.2	56.7	44.2	56.4	60.2	54.2	39.2
4	1.0	A490-N	3/8 ,	4.50x 7/16x12.00	61.5	75.6	59.0	75.2	80.3	85.2	59.0
5	1.0	A490-N	3/8 ,	4.50x 7/16x15.00	85.3	94.5	73.7	94.0	100.4	118.1	73.7
6	1.0	A490-N	3/8 ,	4.50x 7/16x18.00	109.6	113.4	88.5	112.8	120.5	151.8	88.5
7	1.0	A490-N	3/8 ,	4.50x 7/16x21.00	133.6	132.3	103.2	131.6	140.6	185.0	103.2

NOTES:

1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips. Ralw is the governing allowable shear capacity of connection in kips. Plate material is A36 steel
2. E70xx electrodes are used.
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A490-N	1/2 ,	3.75x 1/2 x 4.50			**** tp > db/2 ****				
3	1/2	A490-N	1/2 ,	3.75x 1/2 x 7.50			**** tp > db/2 ****				
4	1/2	A490-N	7/16,	3.75x 1/2 x 10.50			**** tp > db/2 ****				
5	1/2	A490-N	7/16,	3.75x 1/2 x 13.50			**** tp > db/2 ****				
6	1/2	A490-N	7/16,	3.75x 1/2 x 16.50			**** tp > db/2 ****				
7	1/2	A490-N	7/16,	3.75x 1/2 x 19.50			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A490-N	1/2 ,	4.00x 1/2 x 5.00			**** tp > db/2 ****				
3	5/8	A490-N	7/16,	4.00x 1/2 x 8.00			**** tp > db/2 ****				
4	5/8	A490-N	7/16,	4.00x 1/2 x 11.00			**** tp > db/2 ****				
5	5/8	A490-N	7/16,	4.00x 1/2 x 14.00			**** tp > db/2 ****				
6	5/8	A490-N	7/16,	4.00x 1/2 x 17.00			**** tp > db/2 ****				
7	5/8	A490-N	7/16,	4.00x 1/2 x 20.00			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A490-N	7/16,	4.25x 1/2 x 5.25			**** tp > db/2 ****				
3	3/4	A490-N	7/16,	4.25x 1/2 x 8.25			**** tp > db/2 ****				
4	3/4	A490-N	7/16,	4.25x 1/2 x 11.25			**** tp > db/2 ****				
5	3/4	A490-N	7/16,	4.25x 1/2 x 14.25			**** tp > db/2 ****				
6	3/4	A490-N	7/16,	4.25x 1/2 x 17.25			**** tp > db/2 ****				
7	3/4	A490-N	7/16,	4.25x 1/2 x 20.25			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A490-N	7/16,	4.50x 1/2 x 5.75			**** tp > db/2 ****				
3	7/8	A490-N	7/16,	4.50x 1/2 x 8.75			**** tp > db/2 ****				
4	7/8	A490-N	7/16,	4.50x 1/2 x 11.75			**** tp > db/2 ****				
5	7/8	A490-N	7/16,	4.50x 1/2 x 14.75			**** tp > db/2 ****				
6	7/8	A490-N	7/16,	4.50x 1/2 x 17.75			**** tp > db/2 ****				
7	7/8	A490-N	7/16,	4.50x 1/2 x 20.75			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A490-N	7/16,	4.50x 1/2 x 6.00	19.1	43.2	33.7	43.0	46.9	30.2	19.1
3	1.0	A490-N	7/16,	4.50x 1/2 x 9.00	39.2	64.8	50.6	64.4	70.3	62.0	39.2
4	1.0	A490-N	7/16,	4.50x 1/2 x12.00	61.5	86.4	67.4	85.9	93.7	97.4	61.5
5	1.0	A490-N	7/16,	4.50x 1/2 x15.00	85.3	108.0	84.3	107.4	117.1	135.0	84.3
6	1.0	A490-N	7/16,	4.50x 1/2 x18.00	109.6	129.6	101.1	128.9	140.6	173.5	101.1
7	1.0	A490-N	7/16,	4.50x 1/2 x21.00	133.6	151.2	118.0	150.3	164.0	211.4	118.0

NOTES:

1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips.
   Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A490-N	9/16,	3.75x 9/16x 4.50			**** tp > db/2 ****				
3	1/2	A490-N	9/16,	3.75x 9/16x 7.50			**** tp > db/2 ****				
4	1/2	A490-N	1/2 ,	3.75x 9/16x 10.50			**** tp > db/2 ****				
5	1/2	A490-N	1/2 ,	3.75x 9/16x 13.50			**** tp > db/2 ****				
6	1/2	A490-N	1/2 ,	3.75x 9/16x 16.50			**** tp > db/2 ****				
7	1/2	A490-N	1/2 ,	3.75x 9/16x 19.50			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A490-N	9/16,	4.00x 9/16x 5.00			**** tp > db/2 ****				
3	5/8	A490-N	1/2 ,	4.00x 9/16x 8.00			**** tp > db/2 ****				
4	5/8	A490-N	1/2 ,	4.00x 9/16x 11.00			**** tp > db/2 ****				
5	5/8	A490-N	1/2 ,	4.00x 9/16x 14.00			**** tp > db/2 ****				
6	5/8	A490-N	1/2 ,	4.00x 9/16x 17.00			**** tp > db/2 ****				
7	5/8	A490-N	1/2 ,	4.00x 9/16x 20.00			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A490-N	1/2 ,	4.25x 9/16x 5.25			**** tp > db/2 ****				
3	3/4	A490-N	1/2 ,	4.25x 9/16x 8.25			**** tp > db/2 ****				
4	3/4	A490-N	1/2 ,	4.25x 9/16x 11.25			**** tp > db/2 ****				
5	3/4	A490-N	1/2 ,	4.25x 9/16x 14.25			**** tp > db/2 ****				
6	3/4	A490-N	1/2 ,	4.25x 9/16x 17.25			**** tp > db/2 ****				
7	3/4	A490-N	1/2 ,	4.25x 9/16x 20.25			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A490-N	1/2 ,	4.50x 9/16x 5.75			**** tp > db/2 ****				
3	7/8	A490-N	1/2 ,	4.50x 9/16x 8.75			**** tp > db/2 ****				
4	7/8	A490-N	1/2 ,	4.50x 9/16x 11.75			**** tp > db/2 ****				
5	7/8	A490-N	1/2 ,	4.50x 9/16x 14.75			**** tp > db/2 ****				
6	7/8	A490-N	1/2 ,	4.50x 9/16x 17.75			**** tp > db/2 ****				
7	7/8	A490-N	1/2 ,	4.50x 9/16x 20.75			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A490-N	1/2 ,	4.50x 9/16x 6.00			**** tp > db/2 ****				
3	1.0	A490-N	1/2 ,	4.50x 9/16x 9.00			**** tp > db/2 ****				
4	1.0	A490-N	1/2 ,	4.50x 9/16x 12.00			**** tp > db/2 ****				
5	1.0	A490-N	1/2 ,	4.50x 9/16x 15.00			**** tp > db/2 ****				
6	1.0	A490-N	1/2 ,	4.50x 9/16x 18.00			**** tp > db/2 ****				
7	1.0	A490-N	1/2 ,	4.50x 9/16x 21.00			**** tp > db/2 ****				

NOTES:
1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips.
   Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A490-X	3/16,	3.75x3/16x 4.50	6.8	12.2	11.0	12.8	12.7	5.7	5.7
3	1/2	A490-X	3/16,	3.75x3/16x 7.50	14.0	20.3	19.0	21.7	22.7	11.6	11.6
4	1/2	A490-X	3/16,	3.75x3/16x10.50	22.0	28.4	26.9	30.6	32.7	18.3	18.3
5	1/2	A490-X	3/16,	3.75x3/16x13.50	30.5	36.5	34.9	39.5	42.7	25.3	25.3
6	1/2	A490-X	3/16,	3.75x3/16x16.50	39.2	44.6	42.8	48.3	52.7	32.5	32.5
7	1/2	A490-X	3/16,	3.75x3/16x19.50	47.7	52.7	50.8	57.2	62.7	39.6	39.6

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A490-X	3/16,	4.00x3/16x 5.00	10.7	13.5	11.8	14.1	15.1	7.1	7.1
3	5/8	A490-X	3/16,	4.00x3/16x 8.00	21.9	21.6	19.4	22.7	25.1	14.5	14.5
4	5/8	A490-X	3/16,	4.00x3/16x11.00	34.3	29.7	26.9	31.4	35.1	22.8	22.8
5	5/8	A490-X	3/16,	4.00x3/16x14.00	47.6	37.8	34.5	40.1	45.2	31.6	31.6
6	5/8	A490-X	3/16,	4.00x3/16x17.00	61.2	45.9	42.0	48.7	55.2	40.7	40.7
7	5/8	A490-X	3/16,	4.00x3/16x20.00	74.6	54.0	49.5	57.4	65.2	49.6	49.5

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A490-X	3/16,	4.25x3/16x 5.25	15.3	14.2	11.8	14.5	16.3	8.5	8.5
3	3/4	A490-X	3/16,	4.25x3/16x 8.25	31.5	22.3	19.0	22.9	26.4	17.4	17.4
4	3/4	A490-X	3/16,	4.25x3/16x11.25	49.4	30.4	26.1	31.4	36.4	27.4	26.1
5	3/4	A490-X	3/16,	4.25x3/16x14.25	68.5	38.5	33.2	39.9	46.4	38.0	33.2
6	3/4	A490-X	3/16,	4.25x3/16x17.25	88.1	46.6	40.4	48.3	56.5	48.8	40.4
7	3/4	A490-X	3/16,	4.25x3/16x20.25	107.4	54.7	47.5	56.8	66.5	59.5	47.5

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A490-X	3/16,	4.50x3/16x 5.75	20.9	15.5	12.6	15.7	18.8	9.9	9.9
3	7/8	A490-X	3/16,	4.50x3/16x 8.75	42.8	23.6	19.4	24.0	28.9	20.3	19.4
4	7/8	A490-X	3/16,	4.50x3/16x11.75	67.3	31.7	26.1	32.2	38.9	31.9	26.1
5	7/8	A490-X	3/16,	4.50x3/16x14.75	93.3	39.8	32.8	40.5	48.9	44.3	32.8
6	7/8	A490-X	3/16,	4.50x3/16x17.75	119.9	47.9	39.6	48.7	59.0	56.9	39.6
7	7/8	A490-X	3/16,	4.50x3/16x20.75	146.1	56.0	46.3	57.0	69.0	69.4	46.3

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A490-X	3/16,	4.50x3/16x 6.00	27.3	16.2	12.6	16.1	20.1	11.3	11.3
3	1.0	A490-X	3/16,	4.50x3/16x 9.00	55.9	24.3	19.0	24.2	30.1	23.2	19.0
4	1.0	A490-X	3/16,	4.50x3/16x12.00	87.9	32.4	25.3	32.2	40.2	36.5	25.3
5	1.0	A490-X	3/16,	4.50x3/16x15.00	121.9	40.5	31.6	40.3	50.2	50.6	31.6
6	1.0	A490-X	3/16,	4.50x3/16x18.00	156.6	48.6	37.9	48.3	60.2	65.1	37.9
7	1.0	A490-X	3/16,	4.50x3/16x21.00	190.9	56.7	44.2	56.4	70.3	79.3	44.2

NOTES:

1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips.
   Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A490-X	1/4 ,	3.75x 1/4 x 4.50	6.8	16.2	14.7	17.1	16.9	7.6	6.8
3	1/2	A490-X	1/4 ,	3.75x 1/4 x 7.50	14.0	27.0	25.3	29.0	30.2	15.5	14.0
4	1/2	A490-X	1/4 ,	3.75x 1/4 x10.50	22.0	37.8	35.9	40.8	43.5	24.3	22.0
5	1/2	A490-X	1/4 ,	3.75x 1/4 x13.50	30.5	48.6	46.5	52.6	56.9	33.7	30.5
6	1/2	A490-X	1/4 ,	3.75x 1/4 x16.50	39.2	59.4	57.1	64.4	70.3	43.4	39.2
7	1/2	A490-X	1/4 ,	3.75x 1/4 x19.50	47.7	70.2	67.7	76.3	83.7	52.9	47.7

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A490-X	1/4 ,	4.00x 1/4 x 5.00	10.7	18.0	15.8	18.8	20.1	9.4	9.4
3	5/8	A490-X	1/4 ,	4.00x 1/4 x 8.00	21.9	28.8	25.8	30.3	33.5	19.4	19.4
4	5/8	A490-X	1/4 ,	4.00x 1/4 x11.00	34.3	39.6	35.9	41.9	46.9	30.4	30.4
5	5/8	A490-X	1/4 ,	4.00x 1/4 x14.00	47.6	50.4	45.9	53.4	60.2	42.2	42.2
6	5/8	A490-X	1/4 ,	4.00x 1/4 x17.00	61.2	61.2	56.0	65.0	73.6	54.2	54.2
7	5/8	A490-X	1/4 ,	4.00x 1/4 x20.00	74.6	72.0	66.1	76.5	87.0	66.1	66.1

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A490-X	1/4 ,	4.25x 1/4 x 5.25	15.3	18.9	15.8	19.3	21.8	11.3	11.3
3	3/4	A490-X	1/4 ,	4.25x 1/4 x 8.25	31.5	29.7	25.3	30.6	35.1	23.2	23.2
4	3/4	A490-X	1/4 ,	4.25x 1/4 x11.25	49.4	40.5	34.8	41.9	48.5	36.5	34.8
5	3/4	A490-X	1/4 ,	4.25x 1/4 x14.25	68.5	51.3	44.3	53.2	61.9	50.6	44.3
6	3/4	A490-X	1/4 ,	4.25x 1/4 x17.25	88.1	62.1	53.8	64.4	75.3	65.1	53.8
7	3/4	A490-X	1/4 ,	4.25x 1/4 x20.25	107.4	72.9	63.3	75.7	88.7	79.3	63.3

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A490-X	1/4 ,	4.50x 1/4 x 5.75	20.9	20.7	16.9	20.9	25.1	13.2	13.2
3	7/8	A490-X	1/4 ,	4.50x 1/4 x 8.75	42.8	31.5	25.8	31.9	38.5	27.1	25.8
4	7/8	A490-X	1/4 ,	4.50x 1/4 x11.75	67.3	42.3	34.8	43.0	51.9	42.6	34.8
5	7/8	A490-X	1/4 ,	4.50x 1/4 x14.75	93.3	53.1	43.8	54.0	65.3	59.1	43.8
6	7/8	A490-X	1/4 ,	4.50x 1/4 x17.75	119.9	63.9	52.7	65.0	78.6	75.9	52.7
7	7/8	A490-X	1/4 ,	4.50x 1/4 x20.75	146.1	74.7	61.7	76.0	92.0	92.5	61.7

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A490-X	1/4 ,	4.50x 1/4 x 6.00	27.3	21.6	16.9	21.5	26.8	15.1	15.1
3	1.0	A490-X	1/4 ,	4.50x 1/4 x 9.00	55.9	32.4	25.3	32.2	40.2	31.0	25.3
4	1.0	A490-X	1/4 ,	4.50x 1/4 x12.00	87.9	43.2	33.7	43.0	53.5	48.7	33.7
5	1.0	A490-X	1/4 ,	4.50x 1/4 x15.00	121.9	54.0	42.1	53.7	66.9	67.5	42.1
6	1.0	A490-X	1/4 ,	4.50x 1/4 x18.00	156.6	64.8	50.6	64.4	80.3	86.7	50.6
7	1.0	A490-X	1/4 ,	4.50x 1/4 x21.00	190.9	75.6	59.0	75.2	93.7	105.7	59.0

NOTES:

1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips.
   Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A490-X	5/16,	3.75x 5/16x 4.50		**** tp > db/2 ****					
3	1/2	A490-X	5/16,	3.75x 5/16x 7.50		**** tp > db/2 ****					
4	1/2	A490-X	5/16,	3.75x 5/16x 10.50		**** tp > db/2 ****					
5	1/2	A490-X	5/16,	3.75x 5/16x 13.50		**** tp > db/2 ****					
6	1/2	A490-X	5/16,	3.75x 5/16x 16.50		**** tp > db/2 ****					
7	1/2	A490-X	5/16,	3.75x 5/16x 19.50		**** tp > db/2 ****					

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A490-X	5/16,	4.00x 5/16x 5.00	10.7	22.5	19.7	23.4	25.2	11.8	10.7
3	5/8	A490-X	5/16,	4.00x 5/16x 8.00	21.9	36.0	32.3	37.9	41.9	24.2	21.9
4	5/8	A490-X	5/16,	4.00x 5/16x11.00	34.3	49.5	44.9	52.3	58.6	38.0	34.3
5	5/8	A490-X	5/16,	4.00x 5/16x14.00	47.6	63.0	57.4	66.8	75.3	52.7	47.6
6	5/8	A490-X	5/16,	4.00x 5/16x17.00	61.2	76.5	70.0	81.2	92.0	67.8	61.2
7	5/8	A490-X	5/16,	4.00x 5/16x20.00	74.6	90.0	82.6	95.7	108.7	82.6	74.6

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A490-X	5/16,	4.25x 5/16x 5.25	15.3	23.6	19.7	24.1	27.2	14.2	14.2
3	3/4	A490-X	5/16,	4.25x 5/16x 8.25	31.5	37.1	31.6	38.2	43.9	29.0	29.0
4	3/4	A490-X	5/16,	4.25x 5/16x11.25	49.4	50.6	43.5	52.3	60.7	45.6	43.5
5	3/4	A490-X	5/16,	4.25x 5/16x14.25	68.5	64.1	55.4	66.4	77.4	63.3	55.4
6	3/4	A490-X	5/16,	4.25x 5/16x17.25	88.1	77.6	67.3	80.5	94.1	81.3	67.3
7	3/4	A490-X	5/16,	4.25x 5/16x20.25	107.4	91.1	79.2	94.6	110.8	99.1	79.2

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A490-X	5/16,	4.50x 5/16x 5.75	20.9	25.9	21.1	26.2	31.4	16.5	16.5
3	7/8	A490-X	5/16,	4.50x 5/16x 8.75	42.8	39.4	32.3	39.9	48.1	33.9	32.3
4	7/8	A490-X	5/16,	4.50x 5/16x11.75	67.3	52.9	43.5	53.7	64.8	53.2	43.5
5	7/8	A490-X	5/16,	4.50x 5/16x14.75	93.3	66.4	54.7	67.5	81.6	73.8	54.7
6	7/8	A490-X	5/16,	4.50x 5/16x17.75	119.9	79.9	65.9	81.2	98.3	94.9	65.9
7	7/8	A490-X	5/16,	4.50x 5/16x20.75	146.1	93.4	77.1	95.0	115.0	115.6	77.1

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A490-X	5/16,	4.50x 5/16x 6.00	27.3	27.0	21.1	26.8	33.5	18.9	18.9
3	1.0	A490-X	5/16,	4.50x 5/16x 9.00	55.9	40.5	31.6	40.3	50.2	38.7	31.6
4	1.0	A490-X	5/16,	4.50x 5/16x12.00	87.9	54.0	42.1	53.7	66.9	60.8	42.1
5	1.0	A490-X	5/16,	4.50x 5/16x15.00	121.9	67.5	52.7	67.1	83.7	84.4	52.7
6	1.0	A490-X	5/16,	4.50x 5/16x18.00	156.6	81.0	63.2	80.5	100.4	108.4	63.2
7	1.0	A490-X	5/16,	4.50x 5/16x21.00	190.9	94.5	73.7	94.0	117.1	132.1	73.7

NOTES:

1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips.
   Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A490-X	3/8 ,	3.75x 3/8 x 4.50		**** tp > db/2 ****					
3	1/2	A490-X	3/8 ,	3.75x 3/8 x 7.50		**** tp > db/2 ****					
4	1/2	A490-X	3/8 ,	3.75x 3/8 x 10.50		**** tp > db/2 ****					
5	1/2	A490-X	3/8 ,	3.75x 3/8 x 13.50		**** tp > db/2 ****					
6	1/2	A490-X	3/8 ,	3.75x 3/8 x 16.50		**** tp > db/2 ****					
7	1/2	A490-X	3/8 ,	3.75x 3/8 x 19.50		**** tp > db/2 ****					

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A490-X	3/8 ,	4.00x 3/8 x 5.00		**** tp > db/2 ****					
3	5/8	A490-X	3/8 ,	4.00x 3/8 x 8.00		**** tp > db/2 ****					
4	5/8	A490-X	3/8 ,	4.00x 3/8 x 11.00		**** tp > db/2 ****					
5	5/8	A490-X	3/8 ,	4.00x 3/8 x 14.00		**** tp > db/2 ****					
6	5/8	A490-X	5/16,	4.00x 3/8 x 17.00		**** tp > db/2 ****					
7	5/8	A490-X	5/16,	4.00x 3/8 x 20.00		**** tp > db/2 ****					

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A490-X	3/8 ,	4.25x 3/8 x 5.25	15.3	28.4	23.7	29.0	32.7	17.0	15.3
3	3/4	A490-X	3/8 ,	4.25x 3/8 x 8.25	31.5	44.6	37.9	45.9	52.7	34.9	31.5
4	3/4	A490-X	3/8 ,	4.25x 3/8 x11.25	49.4	60.8	52.2	62.8	72.8	54.8	49.4
5	3/4	A490-X	5/16,	4.25x 3/8 x14.25	68.5	77.0	66.5	79.7	77.4	75.9	66.5
6	3/4	A490-X	5/16,	4.25x 3/8 x17.25	88.1	93.2	80.7	96.7	94.1	97.6	80.7
7	3/4	A490-X	5/16,	4.25x 3/8 x20.25	107.4	109.4	95.0	113.6	110.8	118.9	95.0

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A490-X	5/16,	4.50x 3/8 x 5.75	20.9	31.1	25.3	31.4	31.4	19.8	19.8
3	7/8	A490-X	5/16,	4.50x 3/8 x 8.75	42.8	47.3	38.7	47.9	48.1	40.7	38.7
4	7/8	A490-X	5/16,	4.50x 3/8 x11.75	67.3	63.5	52.2	64.4	64.8	63.9	52.2
5	7/8	A490-X	5/16,	4.50x 3/8 x14.75	93.3	79.7	65.7	81.0	81.6	88.6	65.7
6	7/8	A490-X	5/16,	4.50x 3/8 x17.75	119.9	95.9	79.1	97.5	98.3	113.8	79.1
7	7/8	A490-X	5/16,	4.50x 3/8 x20.75	146.1	112.1	92.6	114.0	115.0	138.7	92.6

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A490-X	5/16,	4.50x 3/8 x 6.00	27.3	32.4	25.3	32.2	33.5	22.7	22.7
3	1.0	A490-X	5/16,	4.50x 3/8 x 9.00	55.9	48.6	37.9	48.3	50.2	46.5	37.9
4	1.0	A490-X	5/16,	4.50x 3/8 x12.00	87.9	64.8	50.6	64.4	66.9	73.0	50.6
5	1.0	A490-X	5/16,	4.50x 3/8 x15.00	121.9	81.0	63.2	80.5	83.7	101.2	63.2
6	1.0	A490-X	5/16,	4.50x 3/8 x18.00	156.6	97.2	75.9	96.7	100.4	130.1	75.9
7	1.0	A490-X	5/16,	4.50x 3/8 x21.00	190.9	113.4	88.5	112.8	117.1	158.6	88.5

NOTES:

1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips.
   Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A490-X	7/16,	3.75x 7/16x 4.50			**** tp > db/2 ****				
3	1/2	A490-X	7/16,	3.75x 7/16x 7.50			**** tp > db/2 ****				
4	1/2	A490-X	7/16,	3.75x 7/16x 10.50			**** tp > db/2 ****				
5	1/2	A490-X	3/8 ,	3.75x 7/16x 13.50			**** tp > db/2 ****				
6	1/2	A490-X	3/8 ,	3.75x 7/16x 16.50			**** tp > db/2 ****				
7	1/2	A490-X	3/8 ,	3.75x 7/16x 19.50			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A490-X	7/16,	4.00x 7/16x 5.00			**** tp > db/2 ****				
3	5/8	A490-X	7/16,	4.00x 7/16x 8.00			**** tp > db/2 ****				
4	5/8	A490-X	3/8 ,	4.00x 7/16x 11.00			**** tp > db/2 ****				
5	5/8	A490-X	3/8 ,	4.00x 7/16x 14.00			**** tp > db/2 ****				
6	5/8	A490-X	3/8 ,	4.00x 7/16x 17.00			**** tp > db/2 ****				
7	5/8	A490-X	3/8 ,	4.00x 7/16x 20.00			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A490-X	7/16,	4.25x 7/16x 5.25			**** tp > db/2 ****				
3	3/4	A490-X	3/8 ,	4.25x 7/16x 8.25			**** tp > db/2 ****				
4	3/4	A490-X	3/8 ,	4.25x 7/16x 11.25			**** tp > db/2 ****				
5	3/4	A490-X	3/8 ,	4.25x 7/16x 14.25			**** tp > db/2 ****				
6	3/4	A490-X	3/8 ,	4.25x 7/16x 17.25			**** tp > db/2 ****				
7	3/4	A490-X	3/8 ,	4.25x 7/16x 20.25			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A490-X	3/8 ,	4.50x 7/16x 5.75	20.9	36.2	29.5	36.6	37.6	23.1	20.9
3	7/8	A490-X	3/8 ,	4.50x 7/16x 8.75	42.8	55.1	45.2	55.9	57.7	47.4	42.8
4	7/8	A490-X	3/8 ,	4.50x 7/16x11.75	67.3	74.0	60.9	75.2	77.8	74.5	60.9
5	7/8	A490-X	3/8 ,	4.50x 7/16x14.75	93.3	92.9	76.6	94.4	97.9	103.4	76.6
6	7/8	A490-X	3/8 ,	4.50x 7/16x17.75	119.9	111.8	92.3	113.7	118.0	132.8	92.3
7	7/8	A490-X	3/8 ,	4.50x 7/16x20.75	146.1	130.7	108.0	133.0	138.0	161.9	108.0

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A490-X	3/8 ,	4.50x 7/16x 6.00	27.3	37.8	29.5	37.6	40.2	26.4	26.4
3	1.0	A490-X	3/8 ,	4.50x 7/16x 9.00	55.9	56.7	44.2	56.4	60.2	54.2	44.2
4	1.0	A490-X	3/8 ,	4.50x 7/16x12.00	87.9	75.6	59.0	75.2	80.3	85.2	59.0
5	1.0	A490-X	3/8 ,	4.50x 7/16x15.00	121.9	94.5	73.7	94.0	100.4	118.1	73.7
6	1.0	A490-X	3/8 ,	4.50x 7/16x18.00	156.6	113.4	88.5	112.8	120.5	151.8	88.5
7	1.0	A490-X	3/8 ,	4.50x 7/16x21.00	190.9	132.3	103.2	131.6	140.6	185.0	103.2

NOTES:

1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips.
   Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A490-X	1/2 ,	3.75x 1/2 x 4.50		**** tp > db/2 ****					
3	1/2	A490-X	1/2 ,	3.75x 1/2 x 7.50		**** tp > db/2 ****					
4	1/2	A490-X	7/16,	3.75x 1/2 x 10.50		**** tp > db/2 ****					
5	1/2	A490-X	7/16,	3.75x 1/2 x 13.50		**** tp > db/2 ****					
6	1/2	A490-X	7/16,	3.75x 1/2 x 16.50		**** tp > db/2 ****					
7	1/2	A490-X	7/16,	3.75x 1/2 x 19.50		**** tp > db/2 ****					

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A490-X	1/2 ,	4.00x 1/2 x 5.00		**** tp > db/2 ****					
3	5/8	A490-X	7/16,	4.00x 1/2 x 8.00		**** tp > db/2 ****					
4	5/8	A490-X	7/16,	4.00x 1/2 x 11.00		**** tp > db/2 ****					
5	5/8	A490-X	7/16,	4.00x 1/2 x 14.00		**** tp > db/2 ****					
6	5/8	A490-X	7/16,	4.00x 1/2 x 17.00		**** tp > db/2 ****					
7	5/8	A490-X	7/16,	4.00x 1/2 x 20.00		**** tp > db/2 ****					

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A490-X	7/16,	4.25x 1/2 x 5.25		**** tp > db/2 ****					
3	3/4	A490-X	7/16,	4.25x 1/2 x 8.25		**** tp > db/2 ****					
4	3/4	A490-X	7/16,	4.25x 1/2 x 11.25		**** tp > db/2 ****					
5	3/4	A490-X	7/16,	4.25x 1/2 x 14.25		**** tp > db/2 ****					
6	3/4	A490-X	7/16,	4.25x 1/2 x 17.25		**** tp > db/2 ****					
7	3/4	A490-X	7/16,	4.25x 1/2 x 20.25		**** tp > db/2 ****					

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A490-X	7/16,	4.50x 1/2 x 5.75		**** tp > db/2 ****					
3	7/8	A490-X	7/16,	4.50x 1/2 x 8.75		**** tp > db/2 ****					
4	7/8	A490-X	7/16,	4.50x 1/2 x 11.75		**** tp > db/2 ****					
5	7/8	A490-X	7/16,	4.50x 1/2 x 14.75		**** tp > db/2 ****					
6	7/8	A490-X	7/16,	4.50x 1/2 x 17.75		**** tp > db/2 ****					
7	7/8	A490-X	7/16,	4.50x 1/2 x 20.75		**** tp > db/2 ****					

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A490-X	7/16,	4.50x 1/2 x 6.00	27.3	43.2	33.7	43.0	46.9	30.2	27.3
3	1.0	A490-X	7/16,	4.50x 1/2 x 9.00	55.9	64.8	50.6	64.4	70.3	62.0	50.6
4	1.0	A490-X	7/16,	4.50x 1/2 x12.00	87.9	86.4	67.4	85.9	93.7	97.4	67.4
5	1.0	A490-X	7/16,	4.50x 1/2 x15.00	121.9	108.0	84.3	107.4	117.1	135.0	84.3
6	1.0	A490-X	7/16,	4.50x 1/2 x18.00	156.6	129.6	101.1	128.9	140.6	173.5	101.1
7	1.0	A490-X	7/16,	4.50x 1/2 x21.00	190.9	151.2	118.0	150.3	164.0	211.4	118.0

NOTES:

1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips.
   Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

DESIGN OF SINGLE PLATE FRAMING CONNECTIONS

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1/2	A490-X	9/16,	3.75x 9/16x 4.50			**** tp > db/2 ****				
3	1/2	A490-X	9/16,	3.75x 9/16x 7.50			**** tp > db/2 ****				
4	1/2	A490-X	1/2 ,	3.75x 9/16x 10.50			**** tp > db/2 ****				
5	1/2	A490-X	1/2 ,	3.75x 9/16x 13.50			**** tp > db/2 ****				
6	1/2	A490-X	1/2 ,	3.75x 9/16x 16.50			**** tp > db/2 ****				
7	1/2	A490-X	1/2 ,	3.75x 9/16x 19.50			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	5/8	A490-X	9/16,	4.00x 9/16x 5.00			**** tp > db/2 ****				
3	5/8	A490-X	1/2 ,	4.00x 9/16x 8.00			**** tp > db/2 ****				
4	5/8	A490-X	1/2 ,	4.00x 9/16x 11.00			**** tp > db/2 ****				
5	5/8	A490-X	1/2 ,	4.00x 9/16x 14.00			**** tp > db/2 ****				
6	5/8	A490-X	1/2 ,	4.00x 9/16x 17.00			**** tp > db/2 ****				
7	5/8	A490-X	1/2 ,	4.00x 9/16x 20.00			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	3/4	A490-X	1/2 ,	4.25x 9/16x 5.25			**** tp > db/2 ****				
3	3/4	A490-X	1/2 ,	4.25x 9/16x 8.25			**** tp > db/2 ****				
4	3/4	A490-X	1/2 ,	4.25x 9/16x 11.25			**** tp > db/2 ****				
5	3/4	A490-X	1/2 ,	4.25x 9/16x 14.25			**** tp > db/2 ****				
6	3/4	A490-X	1/2 ,	4.25x 9/16x 17.25			**** tp > db/2 ****				
7	3/4	A490-X	1/2 ,	4.25x 9/16x 20.25			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	7/8	A490-X	1/2 ,	4.50x 9/16x 5.75			**** tp > db/2 ****				
3	7/8	A490-X	1/2 ,	4.50x 9/16x 8.75			**** tp > db/2 ****				
4	7/8	A490-X	1/2 ,	4.50x 9/16x 11.75			**** tp > db/2 ****				
5	7/8	A490-X	1/2 ,	4.50x 9/16x 14.75			**** tp > db/2 ****				
6	7/8	A490-X	1/2 ,	4.50x 9/16x 17.75			**** tp > db/2 ****				
7	7/8	A490-X	1/2 ,	4.50x 9/16x 20.75			**** tp > db/2 ****				

No.	Dia.	Type	Dweld	Plate	Rblt	Ryg	Rsn	Rsne	Rwld	Rbrg	Ralw
2	1.0	A490-X	1/2 ,	4.50x 9/16x 6.00			**** tp > db/2 ****				
3	1.0	A490-X	1/2 ,	4.50x 9/16x 9.00			**** tp > db/2 ****				
4	1.0	A490-X	1/2 ,	4.50x 9/16x 12.00			**** tp > db/2 ****				
5	1.0	A490-X	1/2 ,	4.50x 9/16x 15.00			**** tp > db/2 ****				
6	1.0	A490-X	1/2 ,	4.50x 9/16x 18.00			**** tp > db/2 ****				
7	1.0	A490-X	1/2 ,	4.50x 9/16x 21.00			**** tp > db/2 ****				

NOTES:
1. Rblt, Ryg, Rsn, Rsne, Rwld, Rbrg are the allowable values of shear based on bolt failure, yielding of plate gross area, fracture of plate net net area per AISC, fracture of effective net area, weld fracture, and bolt bearing failure respectively all in kips.
   Ralw is the governing allowable shear capacity of connection in kips.
2. E70xx electrodes are used. Plate material is A36 steel
3. Bolt pitch=3 in., and distance from bolt line to weld line is 3.0 in.
4. Eccentricity of reaction from bolt line is assumed to be 3.0 in.
5. Eccentricity of reaction from weld line is equal to (N)(1.0 in.).

APPENDIX D

TEST SUMMARY SHEETS

This appendix provides three summary sheets for experiments. Each sheet summarizes properties and behavior of each specimen.

AISC SINGLE PLATE SHEAR CONNECTIONS

SUMMARY OF TEST NUMBER 1

SPECIMEN 7-3/4-s-3/8

OBJECTIVE: To study actual behavior of single plate shear connections under realistic loading conditions.

TEST DATE: 7/11/1988

CONDUCTED BY: K. M. McMullin, S. M. Call, R. Stephen and A. Astaneh-Asl

LABORATORY: 200 Davis Hall, University of California, Berkeley.

PROPERTIES OF TEST SPECIMEN:

PLATE DEPTH : 21 in	**PLATE WIDTH :** 4 1/4 in	**PL. THICKNESS:** 3/8 in
PLATE F_y: 35.5 ksi	**PLATE F_u:** 60 ksi	**PL. MATERIAL:** -
NUMBER OF BOLTS: 7	**BOLT DIAMETER:** 3/4 in	**TYPE OF BOLTS:** A325-N
HOLE DIAMETER : 13/16 in	**EDGE DISTANCE:** 1.5 in	**TYPE OF HOLES:** Standard
FILLET WELD SIZE: 1/4 in	**WELD LENGTH:** 21 in	**WELD ELECTRODE:** E70XX

TEST RESULTS:

MAXIMUM SHEAR: 160 kips, **AT ROTATION OF:** 0.031 rad.

MAJOR OBSERVATION: All bolts suddenly sheared off

GENERAL COMMENTS AND DISCUSSION:

- Very small slippage occurred at the start of the test.

- A second slip occurred at about 70 kips shear.

- Yielding that could be observed on the plate, was very minor. Shear deformations were very small.

- Failure occurred when load reached 160 kips of shear acting on the connection. Failure mode was brittle fracture of all bolts in shear.

AISC SINGLE PLATE SHEAR CONNECTIONS

SUMMARY OF TEST NUMBER 2

SPECIMEN 5-3/4-s-3/8

OBJECTIVE: To study actual behavior of single plate shear connections under realistic loading conditions.

TEST DATE: 7/13/1988

CONDUCTED BY: K. M. McMullin, S. M. Call, R. Stephen and A. Astaneh-Asl

LABORATORY: 200 Davis Hall, University of California, Berkeley.

PROPERTIES OF TEST SPECIMEN:

PLATE DEPTH : 15 in	**PLATE WIDTH :** 4 1/4 in	**PL. THICKNESS:** 3/8 in
PLATE F_y: 35.5 ksi	**PLATE F_u:** 60 ksi	**PL. MATERIAL:** -
NUMBER OF BOLTS: 5	**BOLT DIAMETER:** 3/4 in	**TYPE OF BOLTS:** A325-N
HOLE DIAMETER : 13/16 in	**EDGE DISTANCE:** 1.5 in	**TYPE OF HOLES:** Standard
FILLET WELD SIZE: 1/4 in	**WELD LENGTH:** 15 in	**WELD ELECTRODE:** E70XX

TEST RESULTS:

MAXIMUM SHEAR: 137 kips, **AT ROTATION OF:** 0.054 rad.

MAJOR OBSERVATION: All bolts suddenly sheared off

GENERAL COMMENTS AND DISCUSSION:

- No slippage occurred in this specimen until load reached about 100 kips of shear on connection

- Behavior was very similar to specimen 1.

- Failure occurred when load reached a value of 137 kips shear acting on connection.

AISC SINGLE PLATE SHEAR CONNECTIONS

SUMMARY OF TEST NUMBER 3

SPECIMEN 3-3/4-s-3/8

OBJECTIVE: To study actual behavior of single plate shear connections under realistic loading conditions.

TEST DATE: 7/14/1988

CONDUCTED BY: K. M. McMullin, S. M. Call, R. Stephen and A. Astaneh-Asl

LABORATORY: 200 Davis Hall, University of California, Berkeley.

PROPERTIES OF TEST SPECIMEN:

PLATE DEPTH : 9 in	**PLATE WIDTH :** 4 1/4 in	**PL. THICKNESS:** 3/8 in
PLATE F_y: 35.5 ksi	**PLATE F_u:** 60 ksi	**PL. MATERIAL:** -
NUMBER OF BOLTS: 3	**BOLT DIAMETER:** 3/4 in	**TYPE OF BOLTS:** A325-N
HOLE DIAMETER : 13/16 in	**EDGE DISTANCE:** 1.5 in	**TYPE OF HOLES:** Standard
FILLET WELD SIZE: 1/4 in	**WELD LENGTH:** 9 in	**WELD ELECTRODE:** E70XX

TEST RESULTS:

MAXIMUM SHEAR: 94 kips, **AT ROTATION OF:** 0.056 rad.

MAJOR OBSERVATION: All bolts suddenly sheared off

GENERAL COMMENTS AND DISCUSSION:

- This specimen also behaved as specimen 1.
- Very minor shear yielding could be observed on the plate.
- Failure occurred when all three bolts suddenly fractured in shear.