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EXPERIMENTAL STUDY OF SINGLE-PLATE STEEL BEAM-TO-GIRDER CONNECTIONS

by
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COLLEGE OF ENGINEERING
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EXPERIMENTAL STUDY OF SINGLE-PLATE STEEL BEAM-TO-GIRDER CONNECTIONS

by

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ABSTRACT

The objective of this research was to study the behavior of single-plate beam-to-girder connections. Six full size specimens were subjected to loads and deformations in order to simulate gravity loads. In each test the shear-plate was welded to the girder web and bolted to the beam web. The top flange of the beam was coped. Girders with thick and thin webs were used in the study to determine the effect of girder web on connection stiffness and strength.

Specimens were loaded until failure occurred. Failure modes included weld fracture, bolt fracture, and girder web buckling. All specimens were instrumented and load-deformation relationships as well as local strain values were obtained during testing. Graphs of these relationships were then plotted and analyzed. From the analysis, design procedures were formulated and proposed.

All of the specimens were designed using current AISC specifications for beam-to-girder connections. The results of the research were compared to existing design procedures. Future research needs were identified.

ACKNOWLEDGEMENTS

This research project was conducted in the Structural Engineering Laboratory of the Department of Civil Engineering at the University of California at Berkeley as a CE-299 Individual Study project for the completion of the Master of Engineering degree. The authors wish to thank the American Iron and Steel Institute for funding this research, the Department of Civil Engineering for their support, and the efforts of Katleen Almand.

The fabrication of the specimens was completed by technicians in the UC Berkeley laboratory. Thanks is given particularly to Mark Troxler for his invaluable help in designing and constructing the test setup and instrument holding devices. The author also wishes to thank Roy Stephen, without whom the experimental work could not have been completed, William MacCracken for assisting with material testing, and Tony Costa for help with the instrumentation and computer equipment. Appreciation is also given to former graduate student Innes Ho for teaching the first author to use testing equipment and how to affix strain gages, and to Dave Reiser for dealing with the problems of receiving materials.

Finally, the first author wishes to thank Kevin Smathers for his support, tolerance, and computer use during the writing of this report.

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INTRODUCTION

1.1 BACKGROUND

Until very recently the design and detailing of connections in steel structures were left to the fabricators. Because of occasional questions about current design techniques, and because of changes in steel specifications, designers have been paying more attention to connection designs. As a result the behavior of the most common types of connections is being examined by researchers to ensure that current standards provide an adequate factor of safety. Since 1987 researchers at the University of California, Berkeley, have conducted a series of tests to study the behavior of shear connections[1]. This is the final study in the series, and concerns itself with single-plate beam-to-girder connections.

This type of connection is common in most steel construction, particularly in building structures. The most common connector is the single shear plate, welded to the web of the girder and bolted to the beam web. It is simplest to weld a shear plate in the shop and bolt it in the field. This guarantees a high quality connection. It is easier to place the beam in the field since the single plate allows maneuverability of the beam so that it can be brought around from the side of the

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INTRODUCTION

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Currently, it is assumed for analysis purposes that this type of connection acts as a simple pin. This is called a type II connection for purposes of design[2]. To classify as such, the connection must be able to provide sufficient rotation, as well as strength in shear. In general, if the connection develops less than 20% of the fixed end moment of the beam it is considered pinned[3]. For connections between beams and stiff elements (such as column flanges), the moment release associated with the rotation must be provided entirely by the connection. However in a beam-to-girder connection, the flexibility of the girder web may provide additional rotation, thus releasing some of the moment in the connection. The amount of rotation provided by the girder is primarily a function of the thickness of the web and the depth of the girder web relative to the connection length. The purpose of this report is to determine how the flexibility of the girder web affects the strength and deformation of the connection.

1.2 LITERATURE SURVEY

Although single-plate connections are quite popular, information regarding their behavior when used to connect a beam to a girder is limited. On related subjects, coped beams have received some attention in the last few years, due to the problems associated with stress concentration in the beam web[4]. Some of the research done in this series of connection tests at University of California, Berkeley was on single-plate shear connections[5]. However, the specimens were limited to beam-to-column connections. The special concerns associated with connecting shear plates to girder webs have not been examined.

Beam copes, which are often present in beam-to-girder connections, have been shown to lead to lateral-torsional buckling under certain circumstances[6]. While it is crucial for a designer to recognize this problem, it is also necessary to understand how the girder web behaves. In this study, beam webs were chosen to avoid lateral-torsional problems.

The work that has been done on single plate connections provided the basis for the design of the specimens tested in this study. These designs are consistent with AISC ASD specifications[7]. The results of this project support the design procedure presented in the previous study. However, the specific complexity of web deformation shows the limit of applicability, and the refinements needed in the current design procedure.

1.3 SCOPE OF THIS PROJECT

The objective of this project was to reexamine single-plate shear connections in the context of beam-to-girder connections. From this investigation, a more rational design technique was hoped to be developed.

The specimens studied were full size connections on beam and girder sections. Due to difficulty in testing whole girder lengths, only the section surrounding the connection was tested. This amounted to approximately thirty inches of girder. End plates were attached to simulate continuity of the girder and to restrain girder flanges. In this way flange movement of the girder was restricted to between the two ends. While this last condition may not resemble some actual designs, it is conservative as rotation of the entire girder would dissipate some stress in the connection and the web.

The parameters studied in this project were: girder web thickness to depth ratio, length of connection, and length of

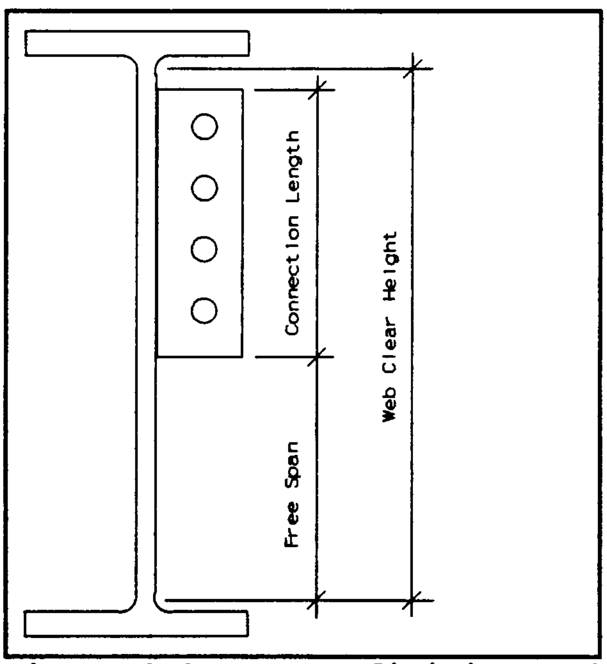


Figure 1.1 Definition of Parameters

connection to girder depth ratio. Details are given in Table 1.1. Free span is the length of girder web below the connection. It is usually described as a percentage of the entire height of the girder web. Connection length is the length of the shear plate in the direction of load. Height of the girder web excludes the fillets. See Figure 1.1.

Table 1.1 - Summary of Tests

	Girder Section	Beam Section	Girder Web Thickness (in.)	Web Clear Height	No. of Bolts	Plate Length (in.)	Hight/ Thickness	% Free Span	Free Span/ Thickness
Test 12	W24x146	W16x50	0.650	21.0	4	12	32	43	14
Test 13	W24x146	W21x50	0.650	21.0	6	18	32	14	4.6
Test 14	W18x35	W16x50	0.300	15.5	4	12	52	23	12
Test 15	W24x55	W21x50	0.395	21.0	6	18	53	14	7.6
Test 16	W24x55	W16x50	0.395	21.0	4	12	53	43	23
Test 17	W18x97	W16x50	0.535	15.5	4	12	29	23	6.5

Note: All Steel is A36

CHAPTER TWO

EXPERIMENTAL PROGRAM

2.1 INTRODUCTION

The experimental program consisted of six full scale tests. Each specimen was subjected to realistic shear and rotation in order to simulate connection shear and rotations in a simply supported beam. All tests were monotonic. The following sections describe the parameters of the study, the test specimens, the loading history, and the test procedure. Results are described and explained in subsequent chapters.

2.2 PARAMETERS OF STUDY

There were three main objectives for this study. The primary purpose for these tests was to verify that the beam-to-girder connections designed using the same procedures as beam-to-column connections can achieve sufficiently high shear loads. In addition, these tests were compared to similar beam-to-column connections in terms of rotation, moment capacity, and eccentricity of moment to see what quantitative effect the flexible web had on each parameter. A third purpose was to describe the behavior of the girder web when subjected to an eccentric shear load.

2.3 TEST SPECIMENS

A typical specimen is shown in Figure 2.1. Each test consisted of a 28 inch girder section, varying in depth from 18 in. to 24 in., connected to a beam section by a single shear plate and 4 or 6 A490N bolts. The girder sections were chosen to give a range of depth over thickness ratios for the webs. Different connection lengths (4 or 6 bolts) allowed different free lengths of the girder web below the shear tab.

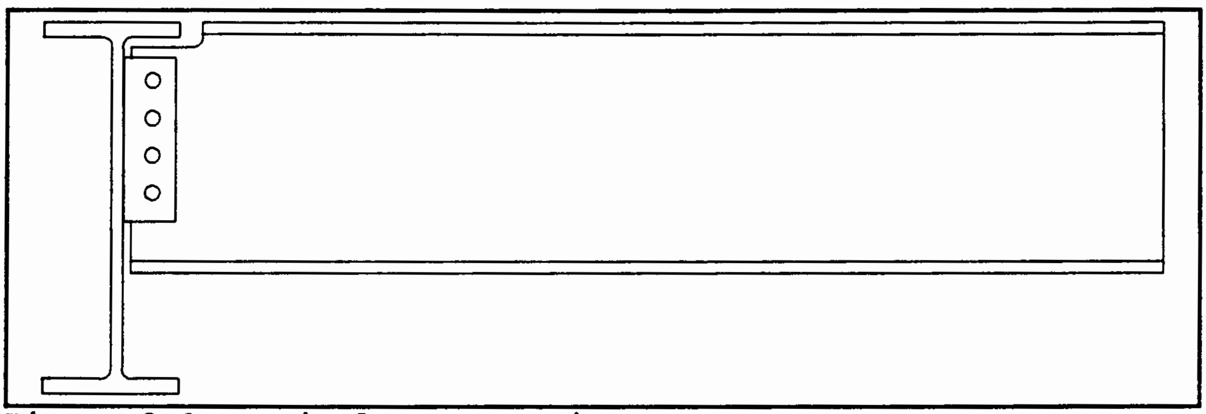


Figure 2.1 Typical Test Specimen

Beams were chosen so that they would not control the failure mode. The web thickness was in all cases thicker than the shear tab, and the beam depth was only enough to allow for the specified connection length (12 or 18 in.). The top flange was coped. All steel used was ASTM - A36.

Specifications for the shear tab, the weld size, and the number of bolts were based on the AISC design procedure for beam-to-column shear tab connections[8].

All bolts were 3/4 inch diameter A490 bolts with threads included in the shear plane. Bolts were tightened using the turn-of-the-nut method as required by the AISC Manual[9]. Bolt holes were punched in the shear tab at the fabricator's shop. Holes were 1/16 inch larger than the bolt to allow easier positioning.

Plates were attached by a fillet weld using an E70 electrode. In tests twelve and thirteen, wire weld was used. In the remaining four tests, stick weld was used. The quality of the welds in the last four tests was superior to the wire weld of the first two. All welds held up adequately under service conditions.

Shear tabs were purchased pre-cut from the fabricator. Holes were at 3 inch center-to-center. Edge distances were 2 3/4 inches from the side, 1 1/2 inches from the top and bottom.

2.4 TEST SET-UP

The test setup was designed to produce realistic loading on the beam. In a laboratory test it is not feasible to apply a large shear load without eccentricity. Even a small eccentricity can produce a much larger rotation than would actually occur in a simply supported beam. In order to control rotations, and the corresponding moment, two actuators were used in the test setup. The first, located a few inches from the connection, applied the main shear load, and is referred to as actuator S. The second, actuator R, was at the far end of the beam and controled the rotation. See Figure 2.2. This system was designed by the second author for use in the UC Berkeley series of connection tests and has proved to be an adequate means of creating realistic loads and rotations[10]. The advantage of this system is that it can used to test any flexible or semi-rigid connection.

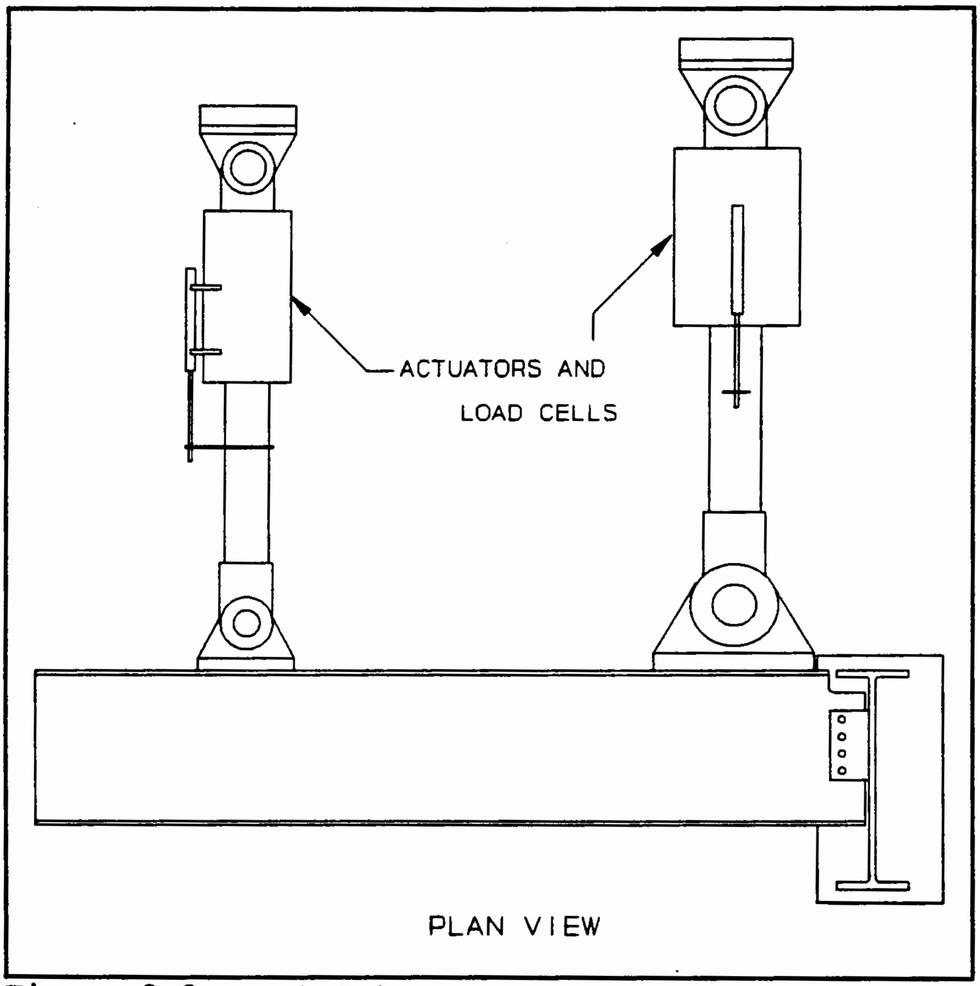


Figure 3.2 Test Set Up

The girder sections tested were about 30 inch long. Both space and budget constraints make it difficult to test full length girders in a laboratory. However it is possible to set up a realistic test using short sections. It is common to have a girder which is restrained against rotation at regular intervals, yet have a single beam attached between these restraints. Restraints may be column connections, shear studs imbedded in a concrete deck, or places where beams frame into either side of the girder. When the girder is restrained against rotation, higher stresses develop in the girder web. The web then acts similar to a plate being pulled out of plane. Because rotation of the girder would release much of the stress developed in the web, the constrained condition is conservative. If however restraints are not provided it is important to determine the impact of torsion and rotation on the girder.

2.5 LOADING HISTORY

The loading histories for the six tests were based on models created by A. Astaneh, and used in previous beam-column tests[11]. In order to simulate actual beam behavior, the specimen was first loaded in shear, with a small rotation, to imitate elastic behavior of the beam. After beam yielding was reached (assumed to be around 0.02 radians), the beam was rotated with less increase in shear up to 0.03 radians. Next, the rate of rotation was increased slightly to simulate plastic deformation up to 0.08 radians. After that the shear rate was doubled to represent strain hardening in the beam. In some cases the beam failed

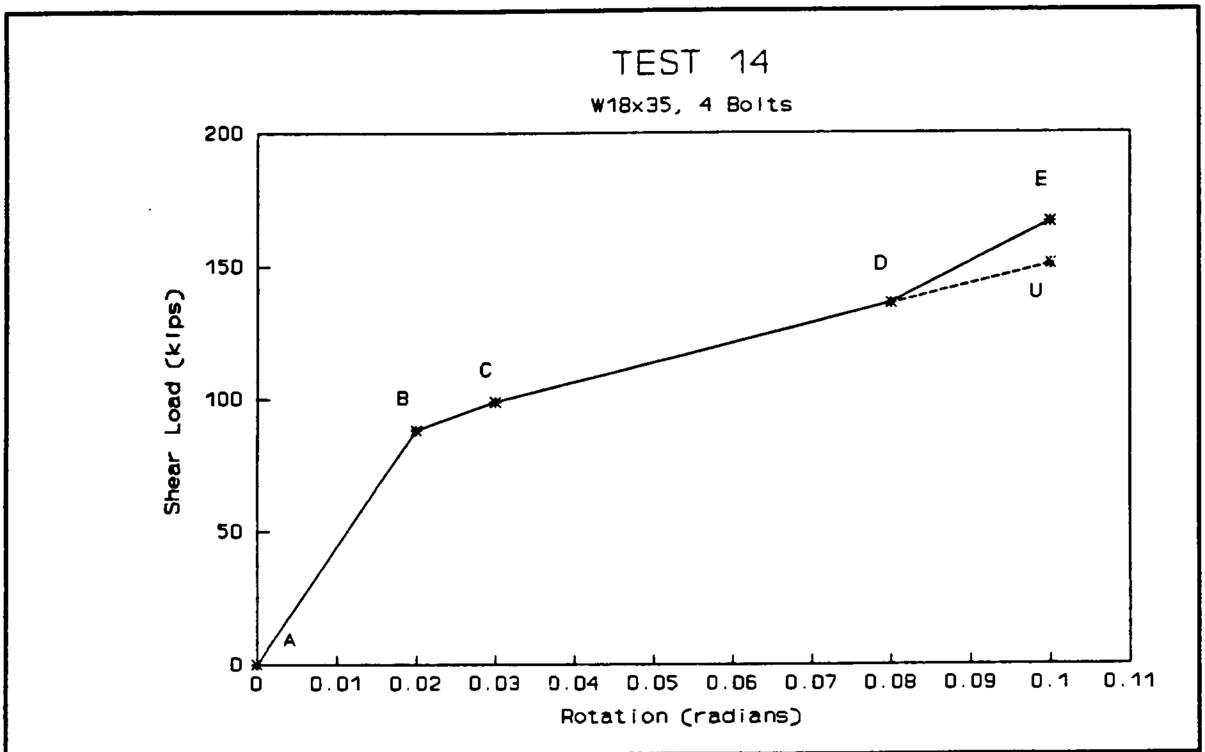


Figure 2.3 **Shear-Rotation Relationship Applied to Test Specimens**

before the end of these steps.

At the point were the rotation reached 0.02 radians, the

Test 14 -- Load Rotation Path Equations

	Rotation
Point B: V(B) = Vy = 12" x 3/8 x 0.577 x Fy = 88.3k	0.02
Point C: 1.12 x Vy = 98.9 k	0.03
Point U: Vult = Vy x Fu / Fy = 151.6 k	0.10
Point D: V(D) = {[Vu - V(C)] / 0.07} x 0.05 + V(C) = 136.5 k	0.08
Point E: V(E) = {[Vu - V(C)] / 0.05} x 2 x 0.02 = 166.6 k	0.10

Fy = actual yield strength of plate material = 34 ksi

Fu = ultimate strength of plate material = 58 ksi

Figure 2.4 **Example of Calculating Shear-Rotation Path**

shear was high enough to begin yielding of the shear plate on the gross section. This is called point B on the load curve. Point C was defined by 0.03 radians of rotation and 1.12 times the shear load at point B. After point D, the slope of the line was determined so that if shear was increased to ultimate, the rotation would reach 0.10 radians. However, a point D was defined along this line at 0.08 radians. After this, the slope was doubled, to hasten failure. An example of a load path curve, and the equations used to define it are found in Figures 2.3 and 2.4.

2.6 INSTRUMENTATION

The instrumentation consisted of three linear potentiometers (linear pots), five rosette strain gages, six linear variable

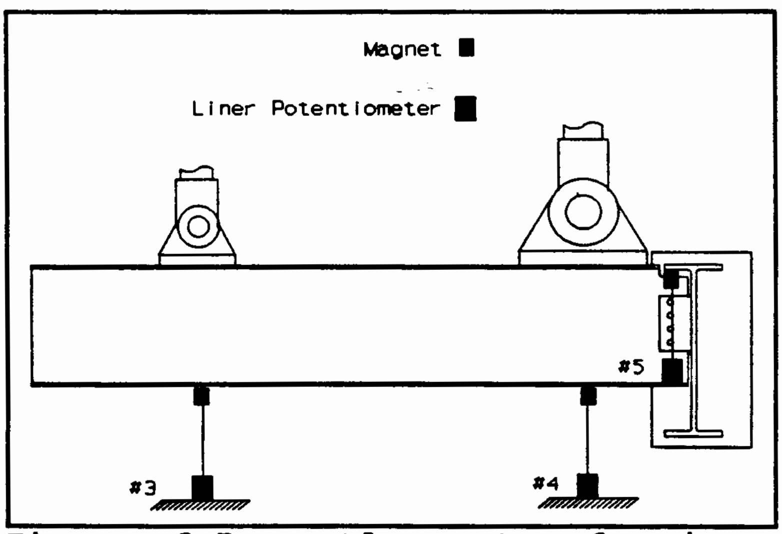


Figure 2.5 Placement of Linear Potentiometer

displacement transducers (LVDT), and two load cells for each test. See Figures 2.5, 2.6, and 2.7. The linear pots were used to measure the rotation of the beam and the slip of the connection. The strain gages were used to determine deformations in the girder web. The LVDTs were used to determine the relative movement of the connection. Data from these instruments were used to create graphs (see Results), and to monitor the progress on the load-rotation curve, described in section 2.5.

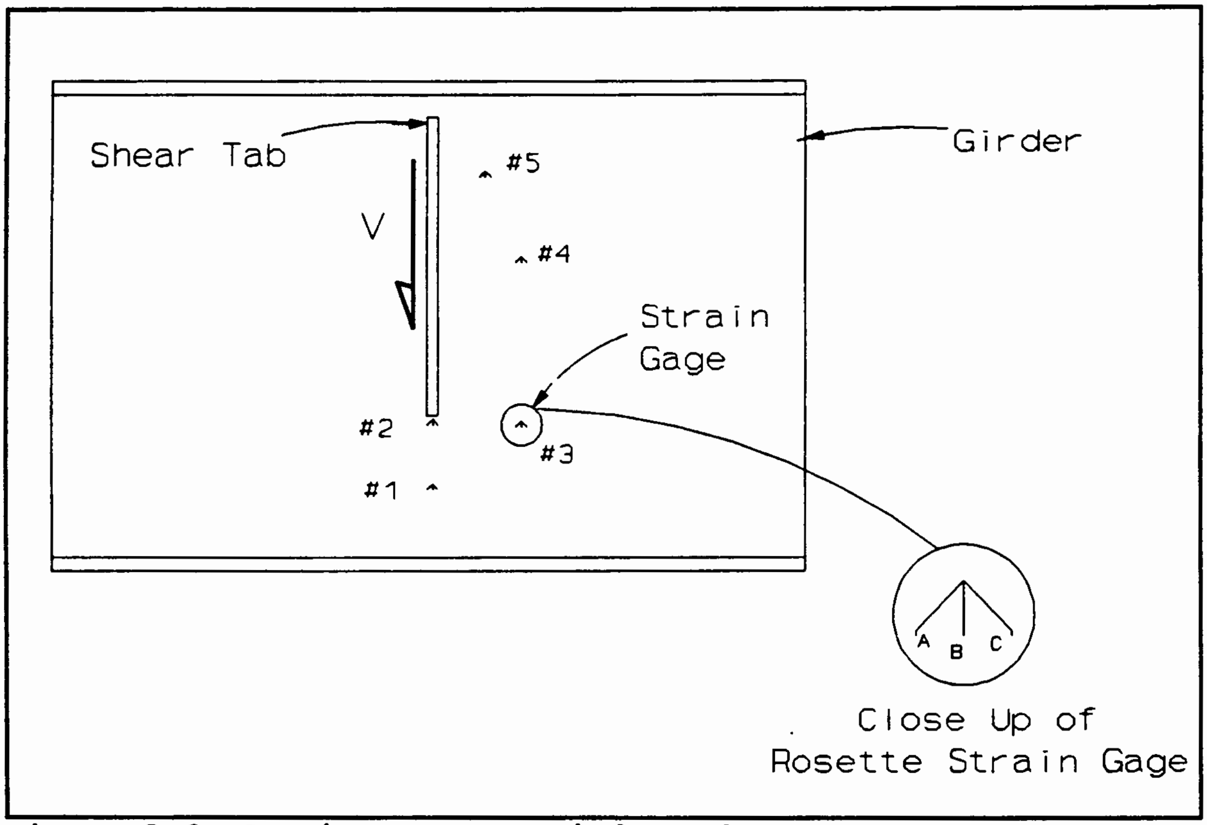


Figure 2.6 Strain Gages on Girder Web

Data acquisition was accomplished by an IBM-PC based system which could continuously monitor the instruments, and record the results when requested. These were printed and recorded on disk. A few important readings such as shear load and girder web strain were printed directly on the screen. A second PC was used to display the progress along the load-rotation curve.

Total shear on the connection was determined by summing the loads from each of the load cells. Rotation was determined by taking the difference of the displacements measured by linear pots #3 and #4, Figure 2.5, and dividing by their separation. This was done automatically by the second PC so that the result could by plotted on the load-rotation curve and compared with the desired progress.

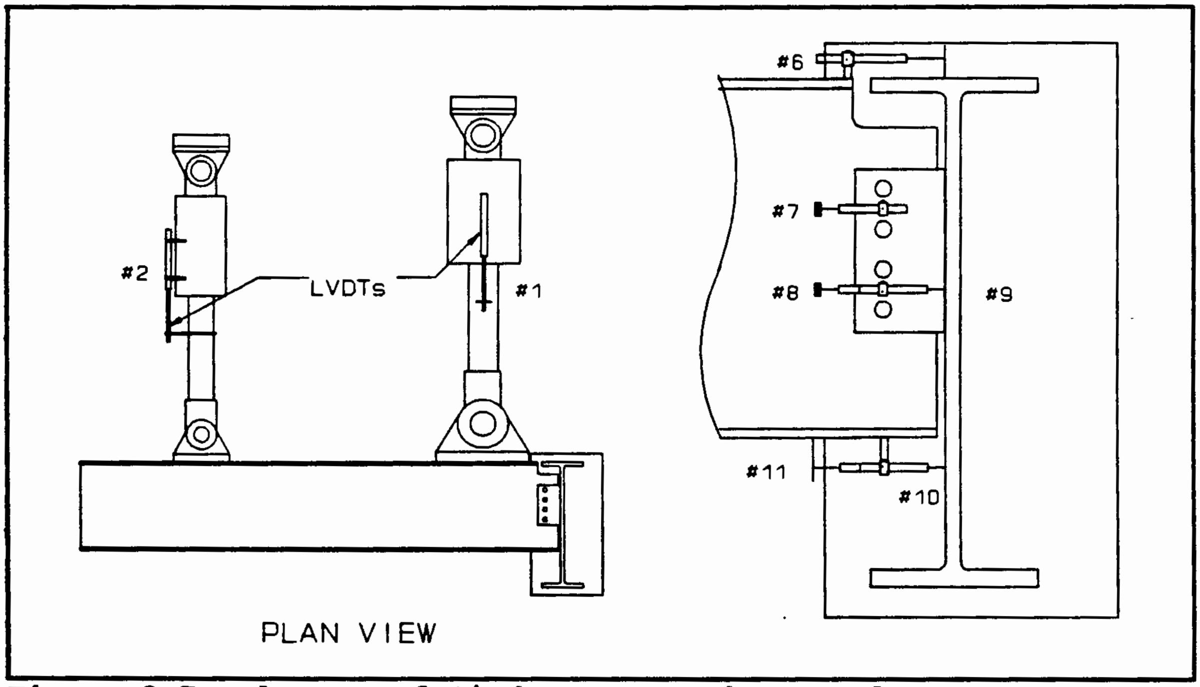


Figure 2.7 Blow Up of Girder Area and LVDT Placement

2.7 TEST PROCEDURES

Test procedures for the specimens proceeded as follows:

1. Specimens were selected to give a range of girder web thicknesses and depths.

2. Materials were ordered and received into the UC Berkeley laboratory. Technicians cut girders to appropriate lengths and welded shear tab onto web.

3. Shear-Rotation path was constructed for the specimen to be tested.

4. Strain gages were attached to girder web. Girder was then placed into test set-up.

5. Beam section was coped and fitted into set-up. Bolts were tightened.

6. LVDT's and Linear Potentiometer were added and calibrated.

7. Specimen was whitewashed in order to detect yielding on the surface of the specimen.

8. Specimen was loaded in small increments in a manner consistent with the desired Shear-Rotation path.

9. Video recording, photographs, and hand notes were taken during loading to record results.

10. Specimen was loaded until failure.

CHAPTER THREE

EXPERIMENTAL RESULTS

3.1 INTRODUCTION

This chapter presents the qualitative and quantitative results from the experimental program. Details of each test are given, followed by a description of the similarities in the tests which are of concern in design. A brief discussion is presented on the behavior of the shear tab and how its role is different for this type of connection than for the beam-to-column connection. Other failure modes are described and problems associated with them are addressed. A summary of the failure modes is given. Quantitative and qualitative results are given.

3.2 BEHAVIOR OF TEST NUMBER TWELVE

Test number twelve was a four bolt test with a large girder section. The girder web was relatively thick, but the free span of the girder web was long. As early as 60k shear load there was yielding in the girder web, although there was no discernable deformation. The web continued to yield and rotate until the weld began to develop a crack at 127k (0.068 radians). The connection continued to take load until the actuator ran out of displacement potential. The final weld crack was about three inches in length at a load of 193k and 0.103 radians rotation. There was a noticeable twist in the shear plate at the end of the test (See Figure 3.1).

Figure 3.1 Test #12 Twisting of Shear plate. Note: black flecks above shear plate indicate yielding of girder web.

The combination of a large free-span to web thickness ratio and a heavy girder section made this a very tough connection.

Similar to the other connections, yielding occurred early in the girder web, releasing some of the moment from the connection. However, the thick web appeared to keep the connection from twisting excessively, despite eccentricities. Thus the bolts were not over stressed in tension, and the connection continued to take load well over the predicted capacity.

3.3 BEHAVIOR OF TEST NUMBER THIRTEEN

This test used the same heavy girder section employed in test twelve, but a six bolt connection and a larger beam section were used. This resulted in a very short, thick free span, and

Figure 3.2 Test #13 Yielding in girder web below shear plate.

the stiffest connection tested. Despite this, there was significant deformation in the girder web before failure. Yielding was first developed below the shear tab around 39 kips load (See Figure 3.2). Weld crack was first noticed at 109 kips, 0.017 radians rotation.

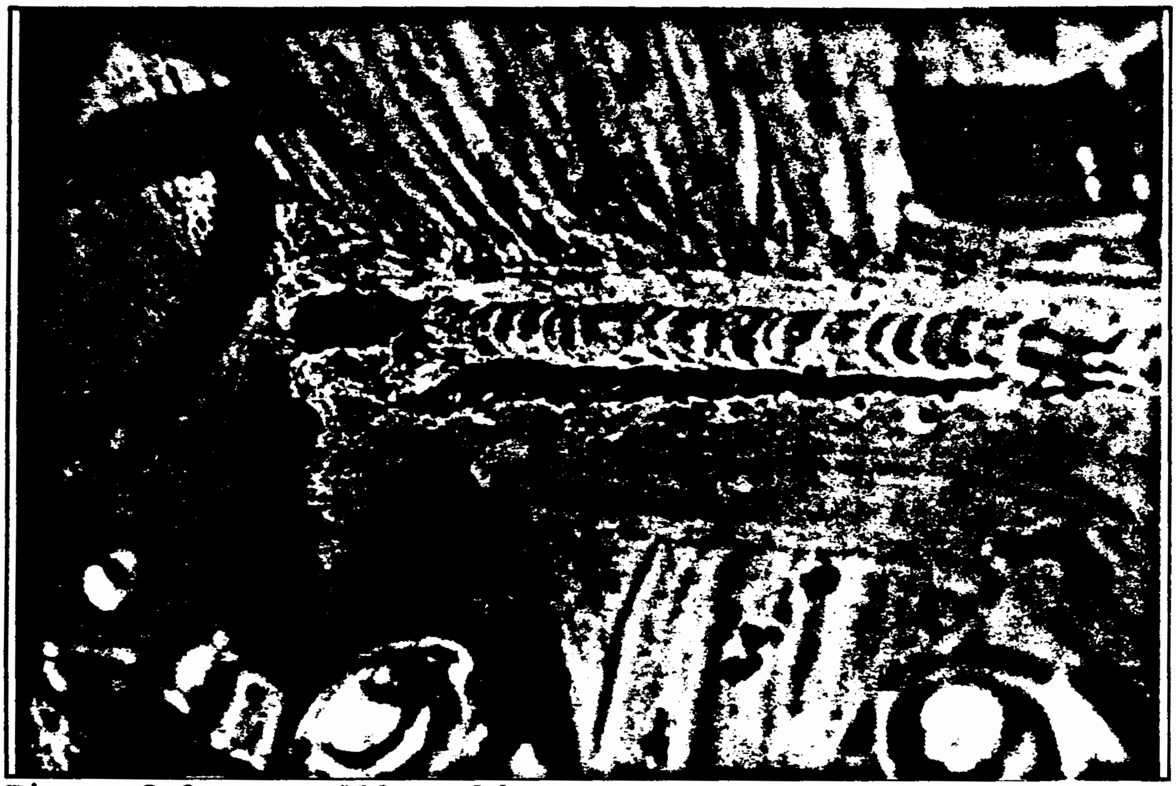


Figure 3.3 Test #13 Weld tear

Unfortunately, the weld joining the tab and the girder was not a good fillet weld. A wire weld was used instead of a stick weld by mistake. In general, wire welds are cooler and do not bind as well to the base material as do stick welds. Thus the weld broke earlier than it should have. Figure 3.3 emphasizes the severity of the weld crack. By only 125 kips a 4 inch tear had been developed. Even so, the connection did achieve more than full design strength and displayed ductility. This test should further emphasize the flexibility and stress-release capability of beam-to-girder connections. It is interesting to note that the same weld was used in test twelve and caused no problems due to the release of the moment by the flexible web. The obvious conclusion is that the weld must be inspected carefully if the web is stiff and the free span short. Otherwise, poor workmanship may result in weld tears under relatively small loads.

3.4 BEHAVIOR OF TEST NUMBER FOURTEEN

Test number fourteen was another four bolt test. The free span distance was relatively short, but the web was very thin (0.300 in.), making it the third most flexible specimen. Yielding was clearly visible on the specimen by only 88 kips shear load and 0.02 radians. The girder web soon made a perceptible s-shape with a relative displacement of 1/4 inch at 92 kips. By 110 kips and 0.05 radians lines of stress outlining a tear shape were well-defined, especially on the lower part of the specimen. Also at this time the weld crack appeared. This opened into a crack of about 1/4 in. by about 1 1/2 in. long by the final load of 122 kips. The test was stopped before failure due to the fact that the beam rotated enough (0.062 radians.) that the flange of the beam came in contact with the flange of the girder.

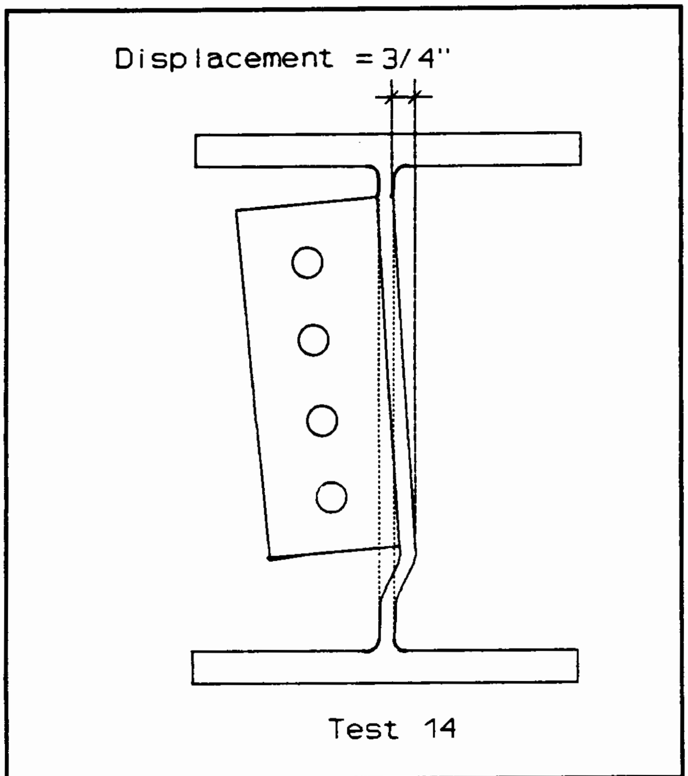


Figure 3.4 Girder Web Deformation

Examining the specimen after testing showed warping of the shear tab due to the high flexibility of the girder web and the slightly eccentric loads. The final relative displacement from top to bottom of shear tab was almost 3/4 in, as shown in Figures 3.4 and 3.5. The bolt hole elongation was not as eccentric as one would expect for a similar beam column connection indicating very little moment development, shown in Figure 3.6.

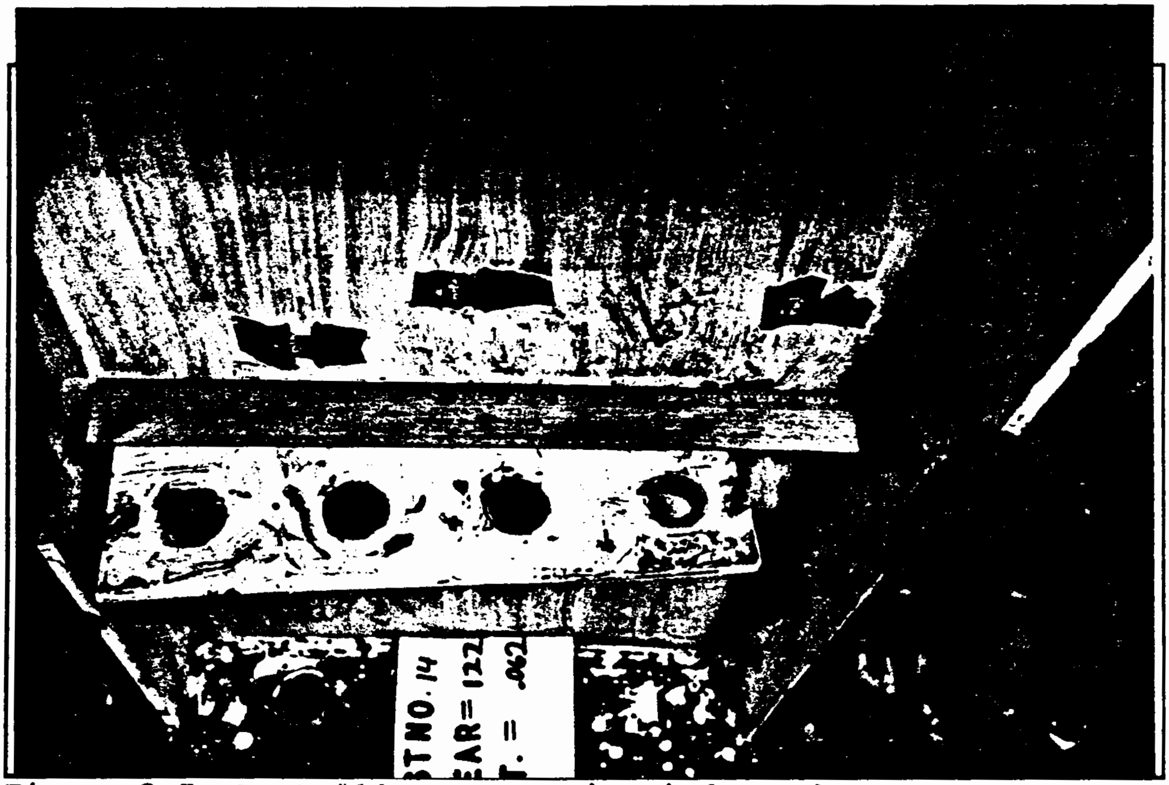


Figure 3.5 Test #14 S-Curve in girder web.

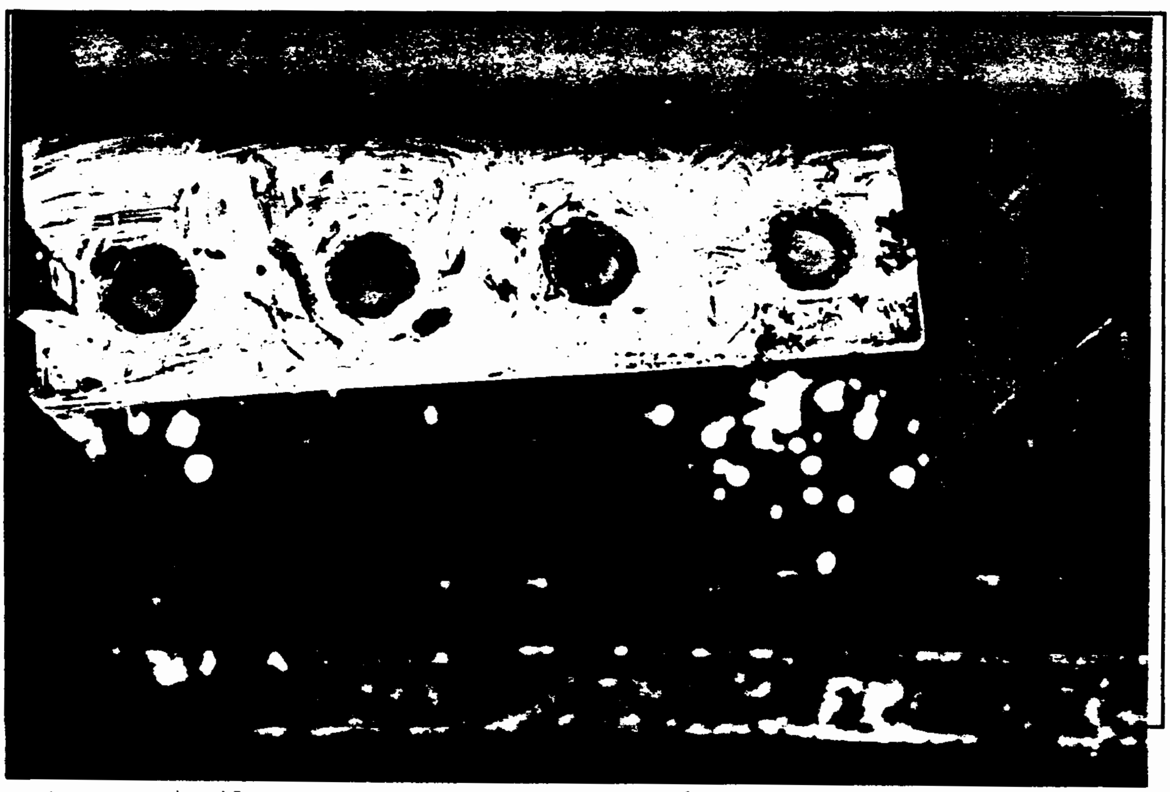


Figure 3.6 Test #14 Hole elongation.

3.5 BEHAVIOR OF TEST NUMBER FIFTEEN

This test was a six bolt connection, with a rather thin girder web (0.395 in.). Despite this, the connection proved quite stiff because of the very short free span of the girder web. The result was some unpredicted but very interesting behavior. At relatively low loads the web began to yield near the top and bottom of the connection, and by the time there was 94 kips total shear load, there were well defined stress lines in the white paint shown in Figure 3.7. The lines continued to progress until the weld began to fracture at the very top of the connection at about 153k. Even with the weld cracked, the connection was so stiff that beam web (not the girder) began to yield in the coped region by around 170k. By 175k the beam was

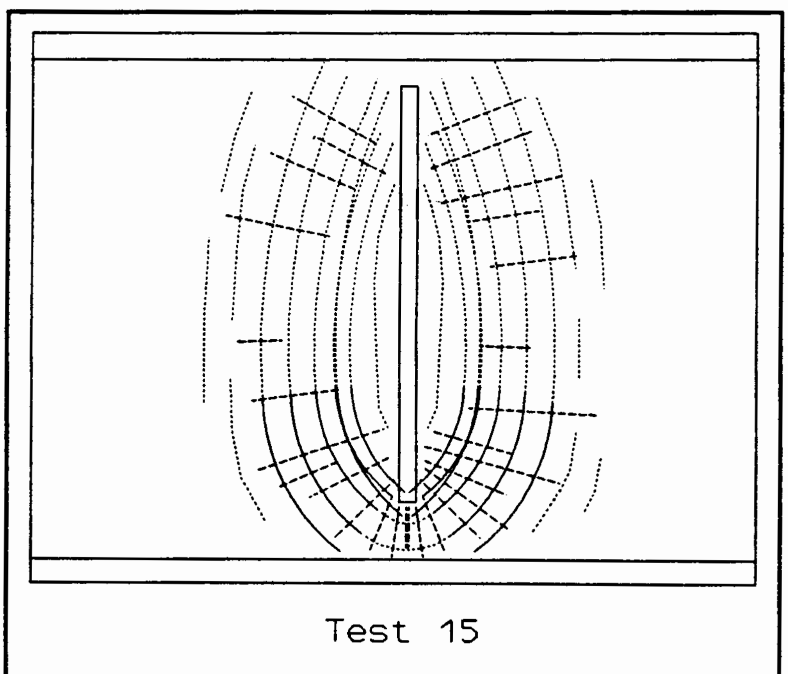


Figure 3.7 Yield Lines in Girder Web.

seriously damaged, and the weld crack had progressed to 1/4 inch. By this time, however, rotation was already at 0.05 radians.

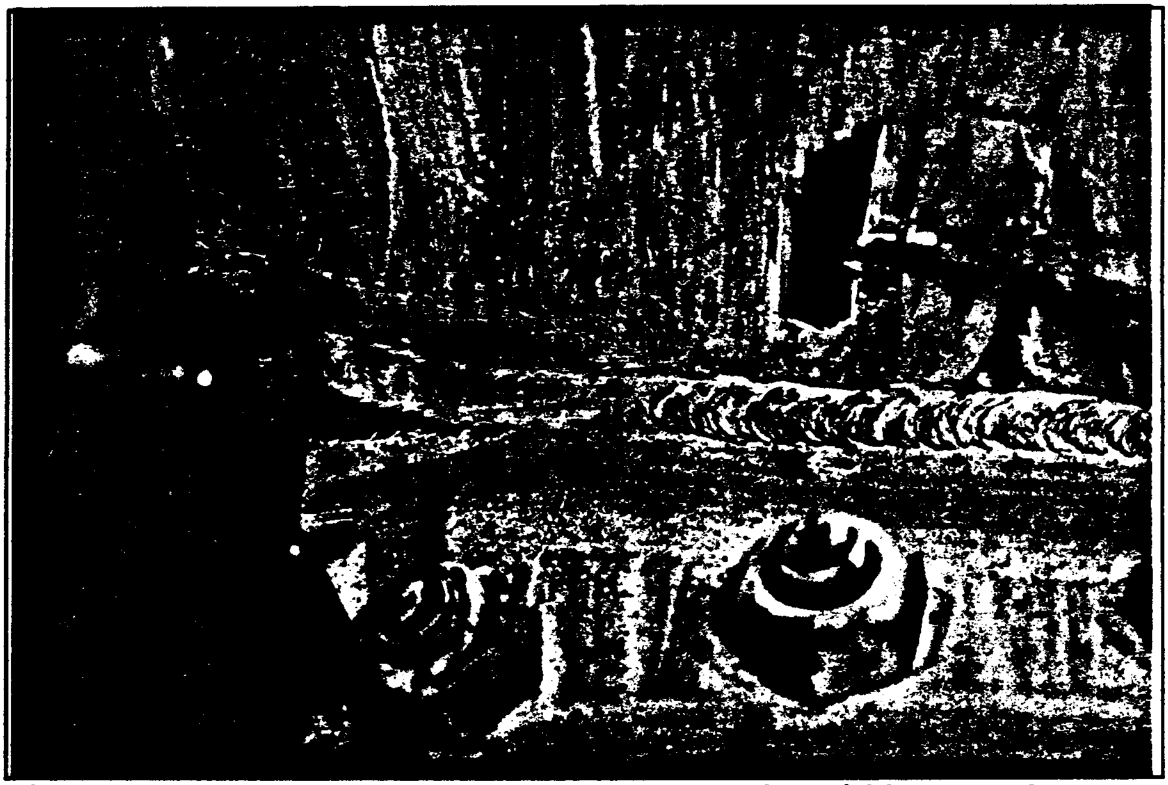


Figure 3.8 Test #15 Weld crack. Note the difference between this "good" crack and the on shown in Figure 3.3.

It seems that the length of the connection (18 in.) combined with a short free span determined the failure of this connection. At low loads, the connection behaved quite flexibly, as it was designed to. However, because of the connection length the moment applied to the girder web was high, and the web began to yield early. Yet it could not yield far because of the short free span of the web, and thus began to stiffen due to catenary action. This resulted in the top section of weld taking additional load, amplified because of the length of the

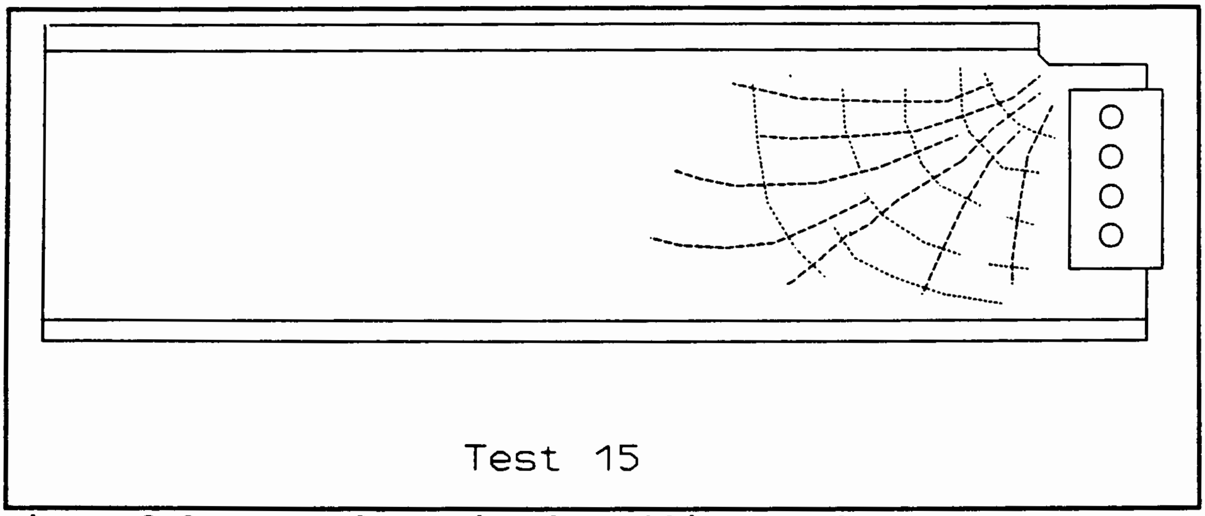


Figure 3.9 Lateral-Torsional Buckling of the Beam

connection, and eventually fracturing. Also, because of slight eccentricity of the connection, accentuated by the deflection of

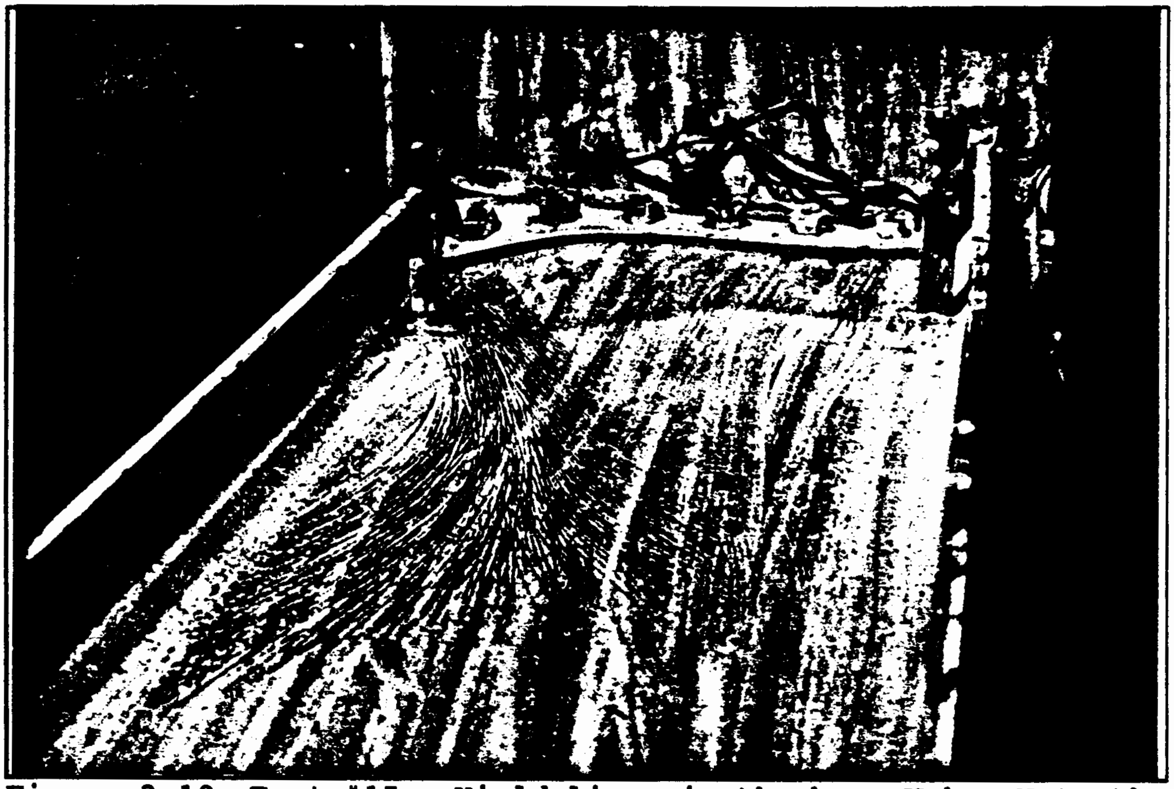


Figure 3.10 Test #15. Yield lines in the beam Web. Note the twisting of the shear plate.

the girder web, the beam was susceptible to a lateral buckling mode, depicted in Figures 3.9 and 3.10. The decreased cross-sectional area was enough to cause the beam web to become over stressed. The twisting action was exhibited in the shear plate at the end of the test, shown in Figure 3.11.

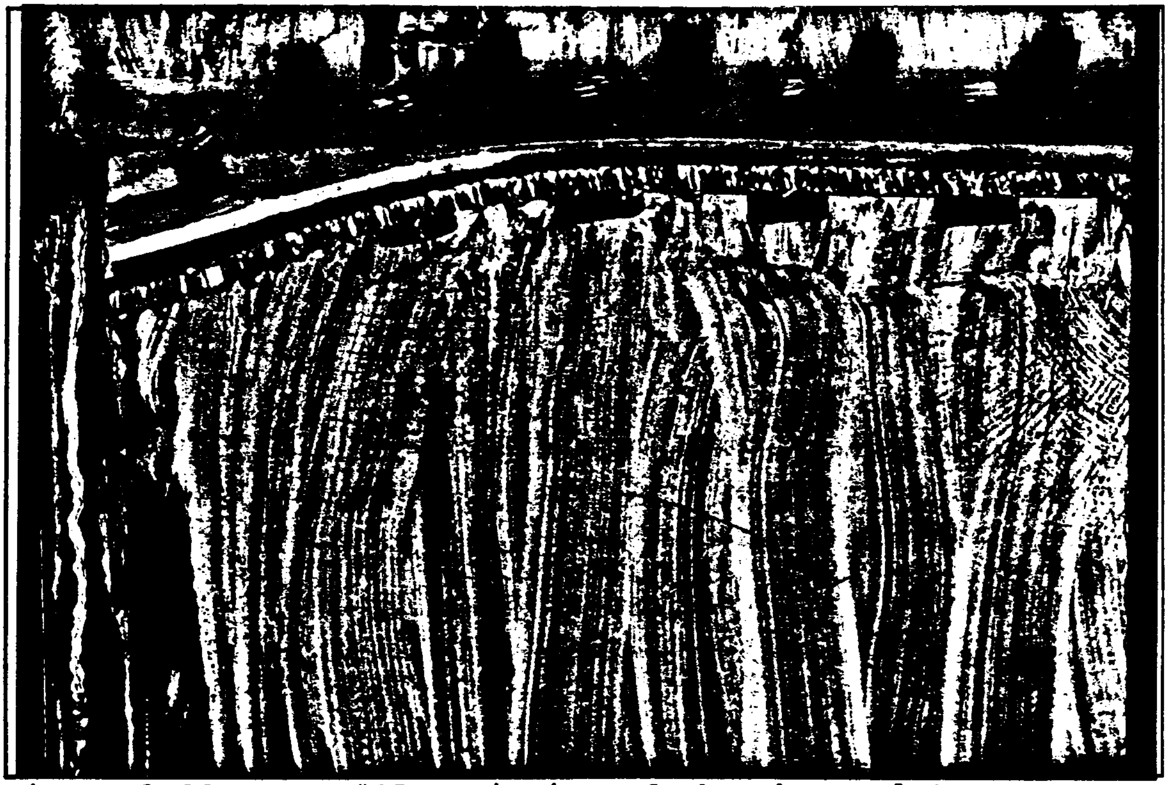


Figure 3.11 Test #15. Twisting of the shear plate

3.6 BEHAVIOR OF TEST NUMBER SIXTEEN

This test was particularly interesting because of the very long free span of the girder web. This, coupled with a thin web, resulted in the most flexible of all the connections tested. At a very low load (62k) the beam began to twist and the girder web to yield. The beam proceeded to rotate easily while the web continued to yield and the shear plate twisted and warped, as seen in Figure 3.12. The weld did crack about 118k, and reached 1/4 inches in length by the time the shear was 127k. At 133k the crack had lengthened to 1-1/2 inches and separated by 1/8 inch from the web. Eventually the bolts sheared off at 133k.

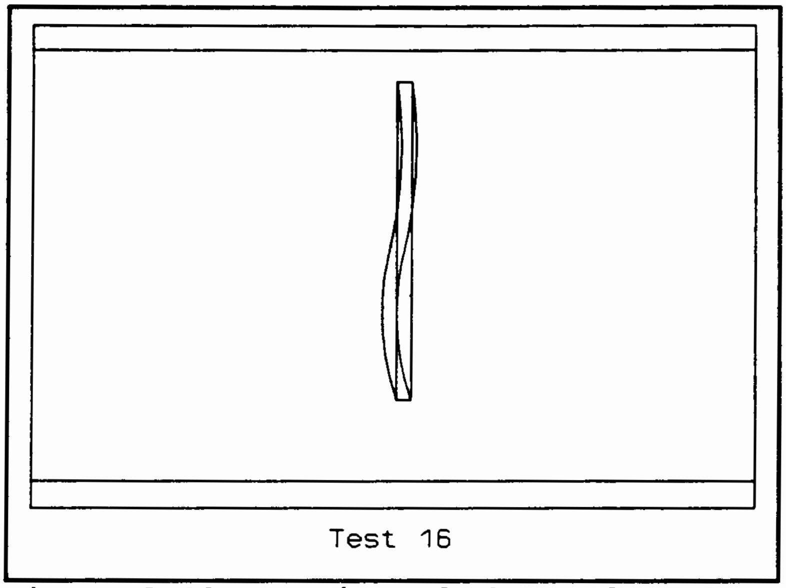


Figure 3.12 Warping of Shear Plate

The extreme flexibility of the girder web and slight eccentricity caused to beam to twist early. While the connection remained flexible and continued to rotate as desired, the twisting effect may have caused additional stresses on the weld. This twisting may also have put tensile stress on the bolts, increasing the total stress, which is why they failed before the weld.

3.7 BEHAVIOR OF TEST NUMBER SEVENTEEN

This four bolt connection was one of the stiffest of the connections tested in that it had a short free-span length, and a web thickness of 0.535 inch. At 84 kips the web began to yield visibly. The white wash paint continued to chip off with little discernable deformation until shear was over 100 kips. By 121 kips there was noticeable bolt twisting. Failure was finally determined by the beam flange rotating into the girder flange at 138 kips and 0.079 radians (see Figure 3.13).

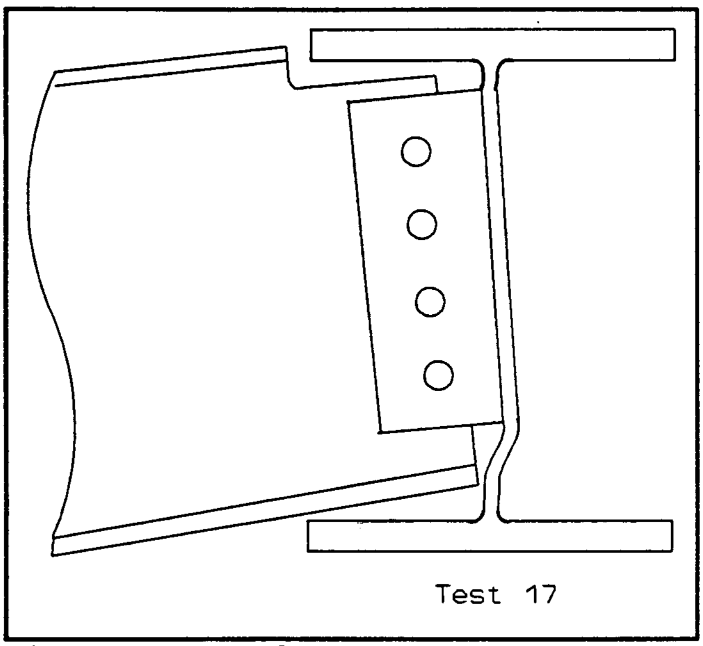


Figure 3.13 Flange Contact

Despite the fact that this was a relatively stiff connection, the girder still allowed rotations to take place by yielding early. Eventually the slight eccentricities and deformations caused the bolts to twist slightly. However, the flanges touching was a more important limit state.

3.8 TYPICAL BEHAVIOR OF CONNECTIONS

The most important effect of the beam-to-girder connection appears to be its ability to release stresses under low loads, yet pick them up as the stress increases. In all cases, the girder web did yield significantly before failure. At very low loads, the flexibility of the web allowed the connection to act as a pin. In cases where the free span of the girder was long, the initial flexibility was due to elastic bending of the plate, followed by yielding. Stiffer webs or ones with little free span showed little or no perceptible bending before yielding. In both cases however, yielding occurred in the web at a shear far below the ultimate load for the connection.

As loading continued, the girder web stiffened up, allowing the connection to take more of the load. The benefit from this is that such a connection will not continue to rotate so much under large shears. However, because of the increased stiffness, and because of rotation, the upper portion of the connection weld becomes stressed in tension. Eventually this results in weld fracture and tearing. Eccentricities also lead to twisting of the plate, especially in the more flexible connections.

3.9 SHEAR TAB BEHAVIOR

Because of the softness of the shear tab material (see section 4.1), yielding in the shear tab was expected at a low

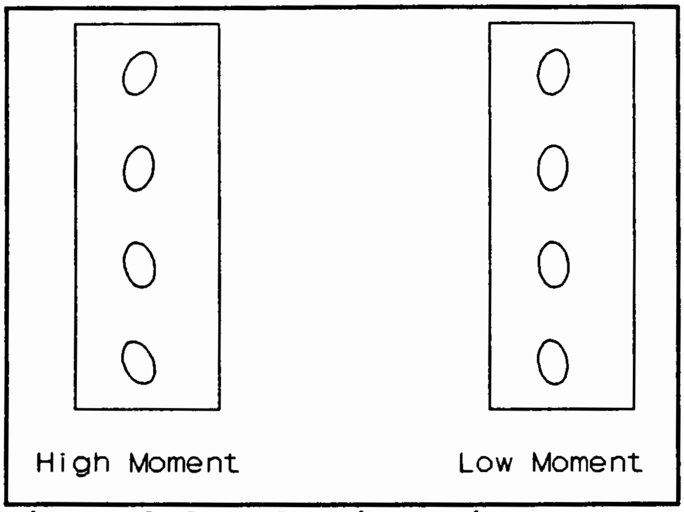


Figure 3.6 Hole Distortions

load. However, unlike previous beam-to-column tests, where the ductility of the connection was supplied primarily by the shear tab yielding, the girder web provided much of the rotation to release moment-induced stresses. Thus at the end of the tests, there was little apparent yielding of the plate.

There were hole distortions caused by the bolts bearing on the soft steel. However, unlike previous tests, the distortions did not seem to describe an arc, shown in Figure 3.6, which would indicate a high moment. Rather, the distortions appeared to indicate almost all of the load was shear. This is supported by the fact that at loads high enough to cause yielding in bearing, in many cases the eccentricity of the inflection point was very close to the connection, as shown in eccentricity graphs in Appendix D.

The other major deformation seen was bending and/or twisting of the shear plate. As described earlier, the slight eccentricity caused by having a tab on only one side of the beam, coupled with the extreme flexibility of the girder web, often caused some twisting in the beam. This led to out-of-plane distortions of the shear plate. This effect did not directly cause failure in any of these tests. If the beam is restrained against twisting, additional stresses will develop in the connection. Twisting can eventually lead to tension in the bolts, and a sudden brittle failure, as in Test Sixteen.

3.10 OTHER FAILURE MODES

When the difference between the depth of the beam and the depth of the girder is small there is the possibility that the flanges of the two sections will touch before the capacity of the connection is reached. In such a case the shear load is transferred from the connection to the girder flange. Although this mechanism may offer a certain factor of safety, it is probably not a desirable failure mode in most cases. If the flange is stiff enough to carry the load there will be a large moment applied to the top of the connection. This will probably lead to large bolt forces and possibly bolt failure. If the flange cannot carry the load, there will be rotation of the flange, which may not be desirable. However, it should be noted that the connections which displayed this behavior during this course of tests did not do so until after their ultimate design value was reached. Thus their performance can be considered adequate.

Lateral-torsional buckling of the beam can be a problem if the connection is very flexible. In this case, the small eccentricity in load causes twisting, which has undesirable effects on bolts and welds described earlier. However, designers must also be aware of the fact that twisting can damage the beam as well as the connection. Even though this failure mode is observed occasionally in the laboratories, it appears that in actual buildings with adequate floor bracing it is not likely to occur.

3.11 SUMMARY OF FAILURE MODES

Several failure modes were established to be considered in the design of beam-to-girder connections. These are:

1) Yielding of shear plate
2) Bolt bearing on plate holes
3) Edge distance failure of bolt holes
4) Fracture of net area of shear plate
5) Bolt Fracture
6) Weld Fracture
7) Block shear failure of the beam web.

Weld and bolt failures were the expected failure modes in the test specimens. Plate yielding and bolt hole deformation occurred but were not catastrophic. Edge distances were chosen so that fracture would not occur before bolt failure.

Additional failure modes experienced in beam-to-girder connections are:

8) Twisting due to eccentric loading, causing additional tension stresses in bolts and shear stresses in welds
9) Contact between the flanges of the beam and girder due to large rotations, leading to additional moment in the connection
10) Lateral-torsional buckling of the coped beam.

The last three are the result of the geometry of the members. Twisting is a special problem caused by very flexible girder webs, and can lead to lateral-torsional buckling of the beam. The excessive deformations are caused by an eccentric load. Flange contact is only a problem if sufficient clearance is not provided. If this is the case, a calculation should be made to determine what rotation will cause the flange of the beam to come in contact with the girder flange. If this value is low (less than 0.03 radians), additional strength must be designed into the weld and bolts to take the high moment that will be developed. Better yet, it would be advisable to provide clearance between the flanges of the beam and the girder.

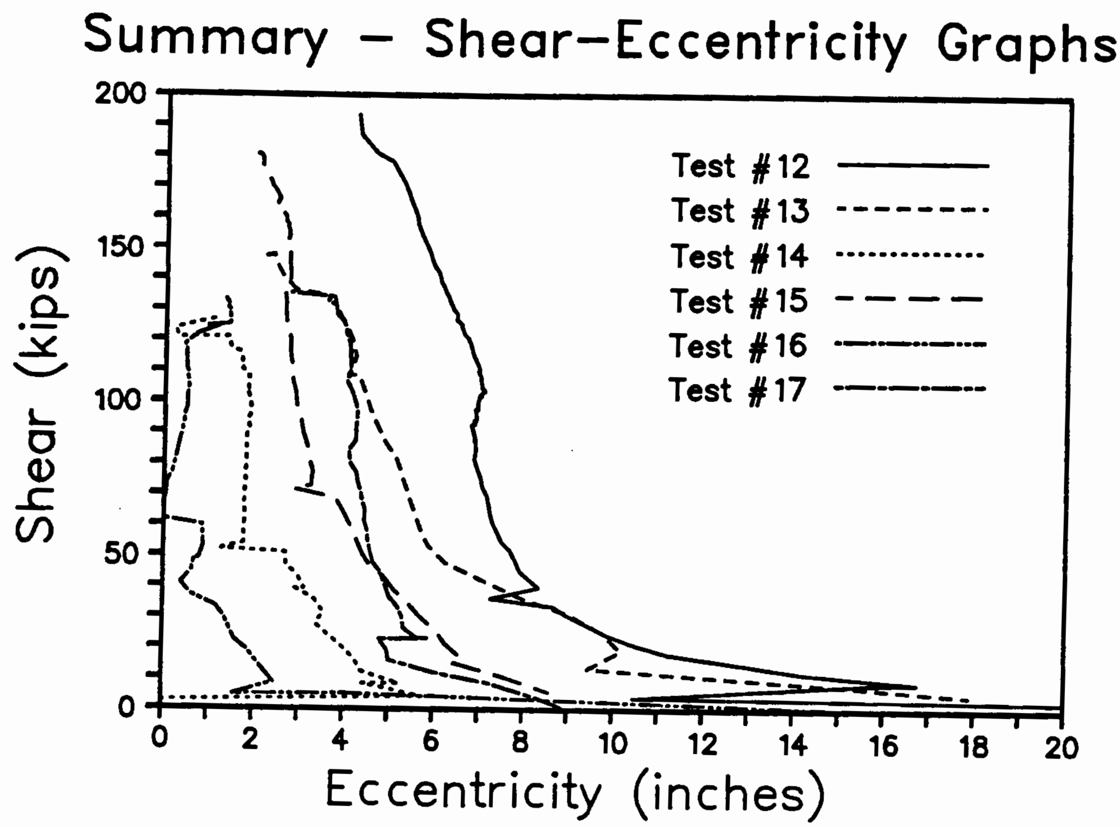
3.12 TEST RESULTS

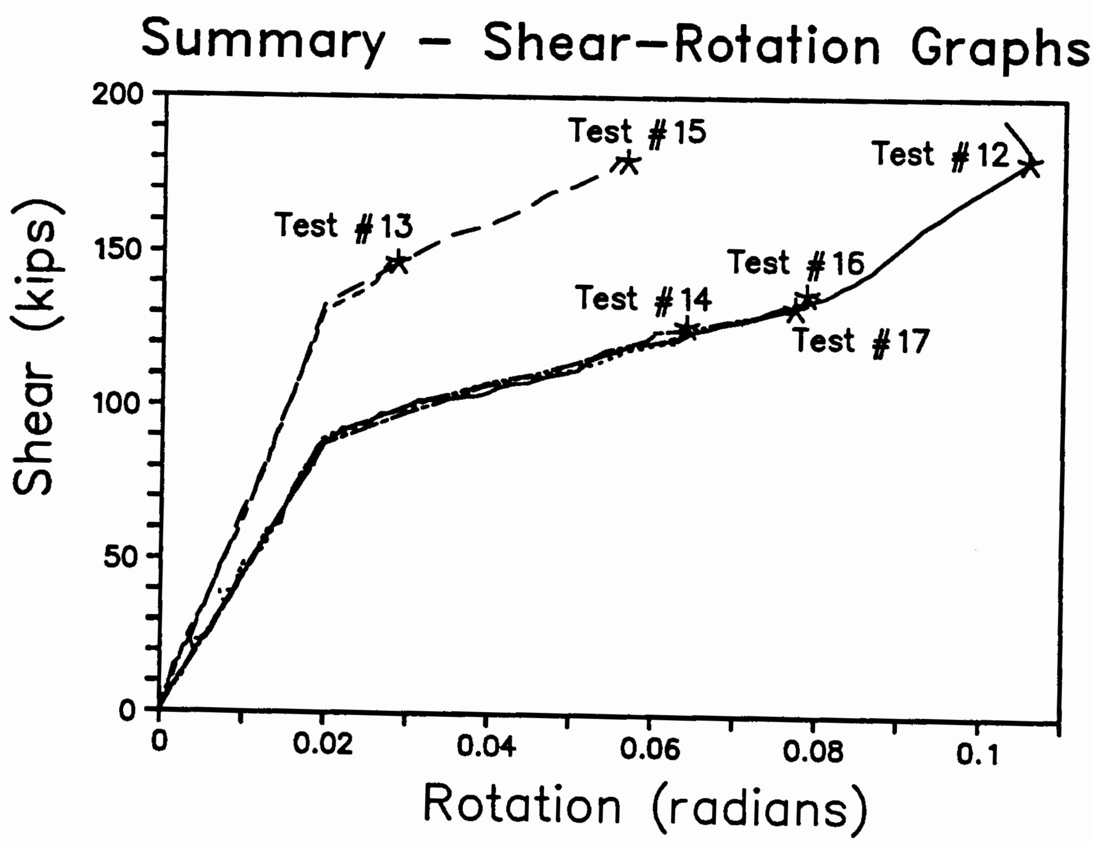
Following each test a summary sheet was made containing the specifications for the specimen, and some qualitative notes on the behavior taken during the test. These can be found in Appendix E. Additionally, graphs were made describing the relationships between various parameters in the test. Graphs include:

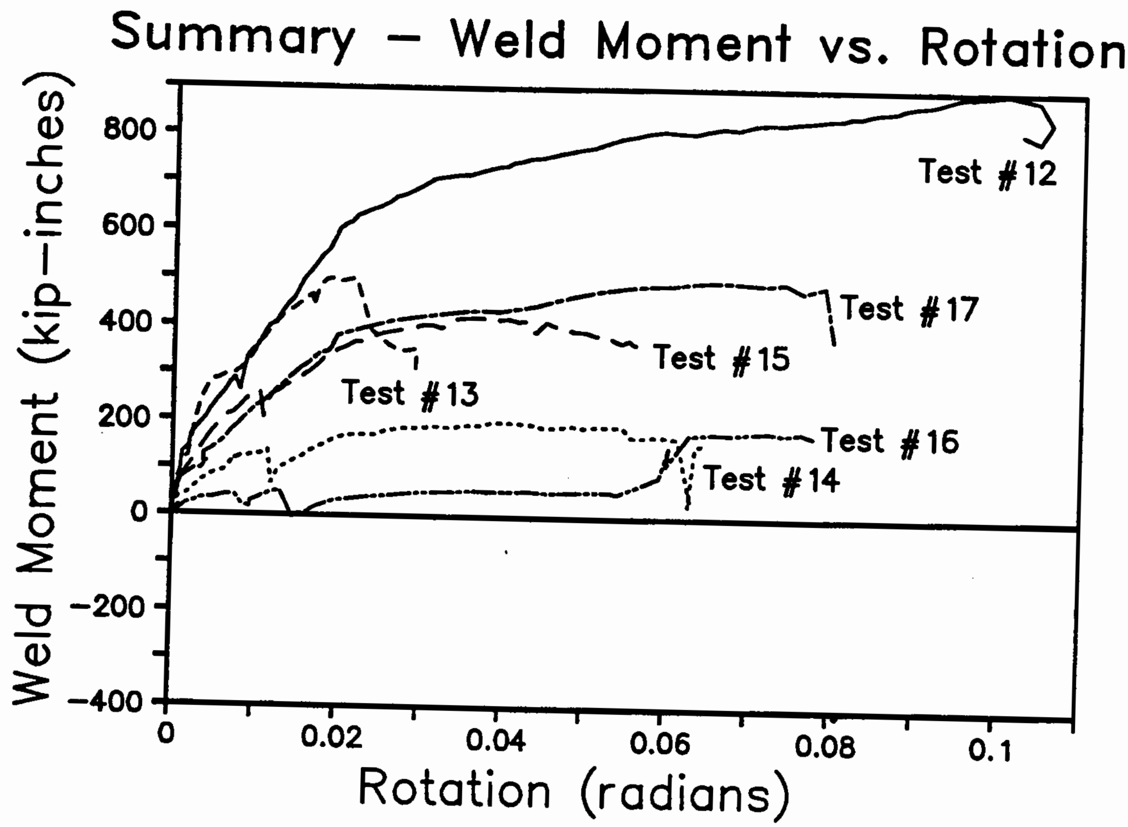
1)	Shear	vs.	Eccentricity of the Inflection Point of the Beam.
2)	Shear	vs.	Rotation
3)	Moment at Weld	vs.	Rotation
4)	Moment at Boltline	vs.	Rotation
5)	Moment at Weld	vs.	Shear
6)	Moment at Boltline	vs.	Shear
7)	Shear	vs.	Deflection at Boltline
8)	Shear	vs.	Shear Strain in Girder Web

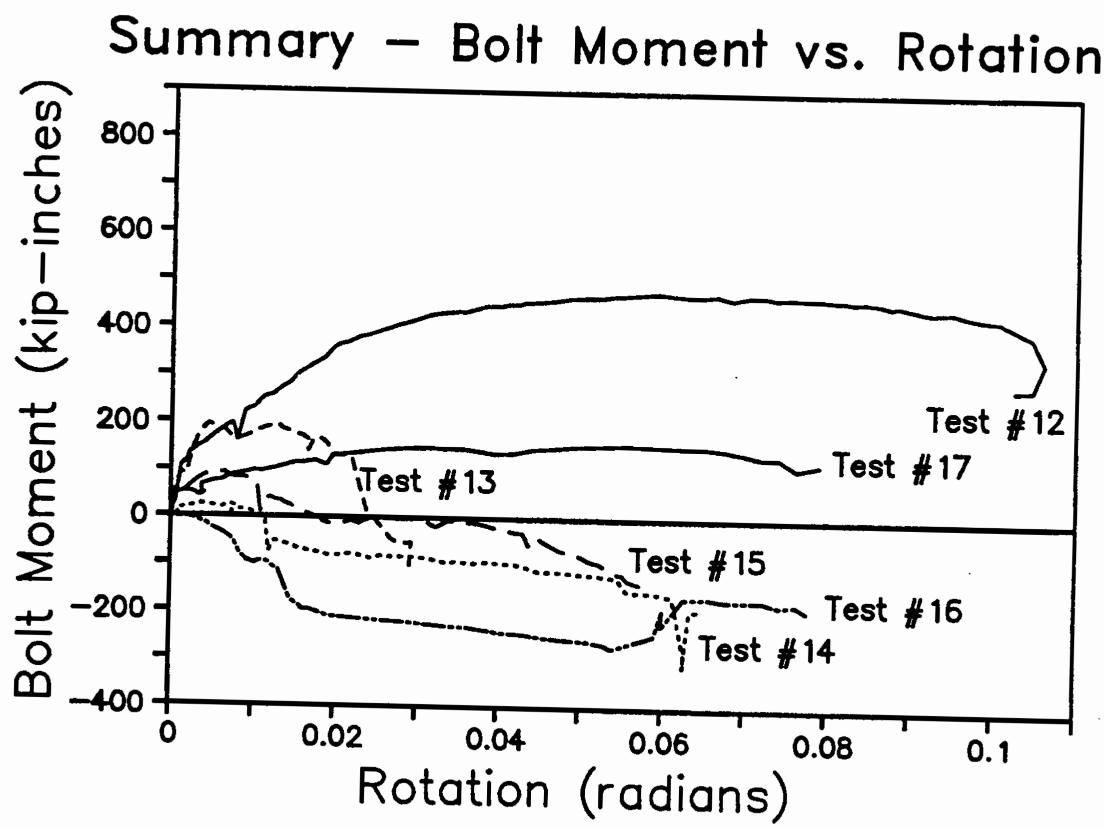
These graphs can be found in Appendix D.

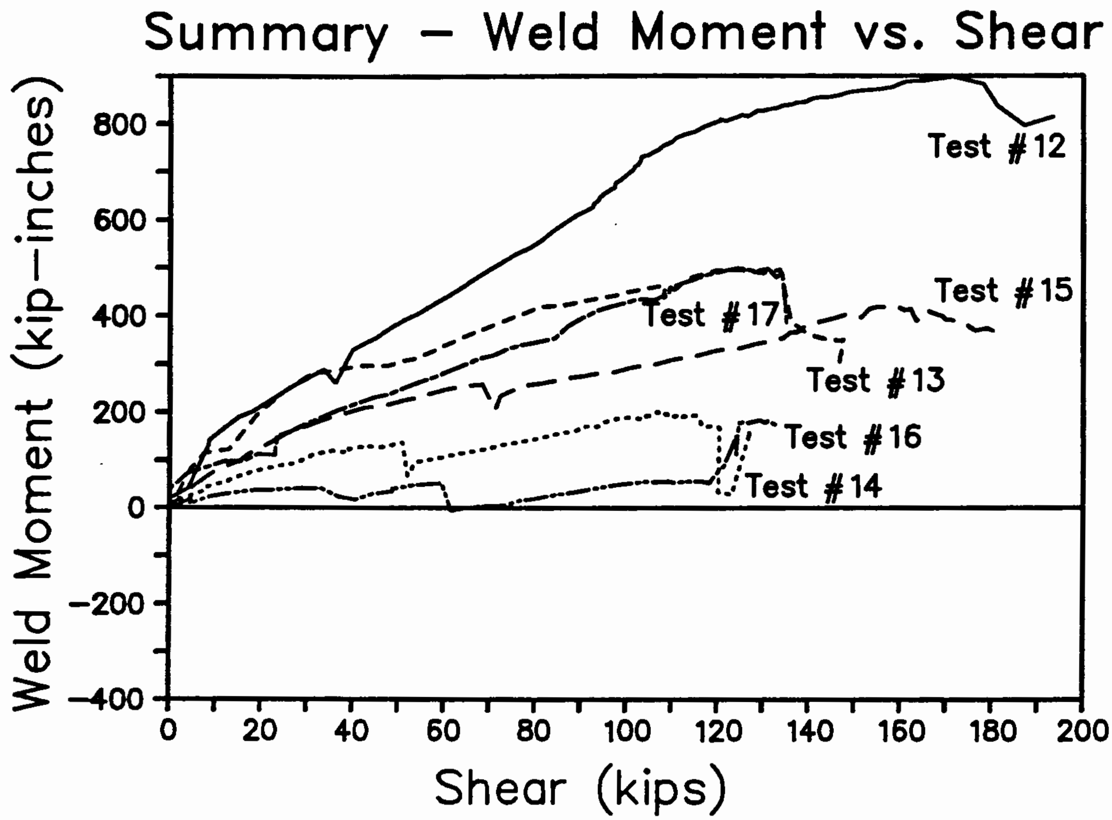
Following are a summary of test results.

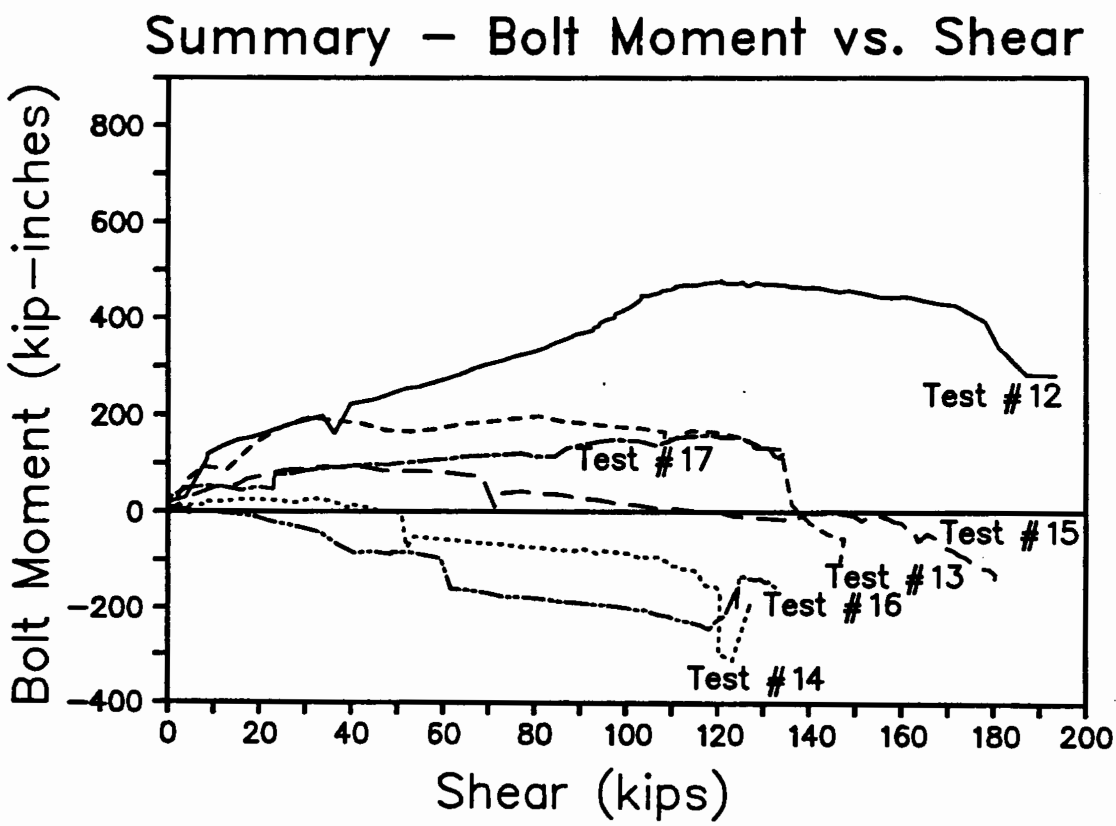

Summary – Shear–Eccentricity Graphs
Shear (kips)
Eccentricity (inches)
Test #12
Test #13
Test #14
Test #15
Test #16
Test #17
0
50
100
150
200
0
2
4
6
8
10
12
14
16
18
20

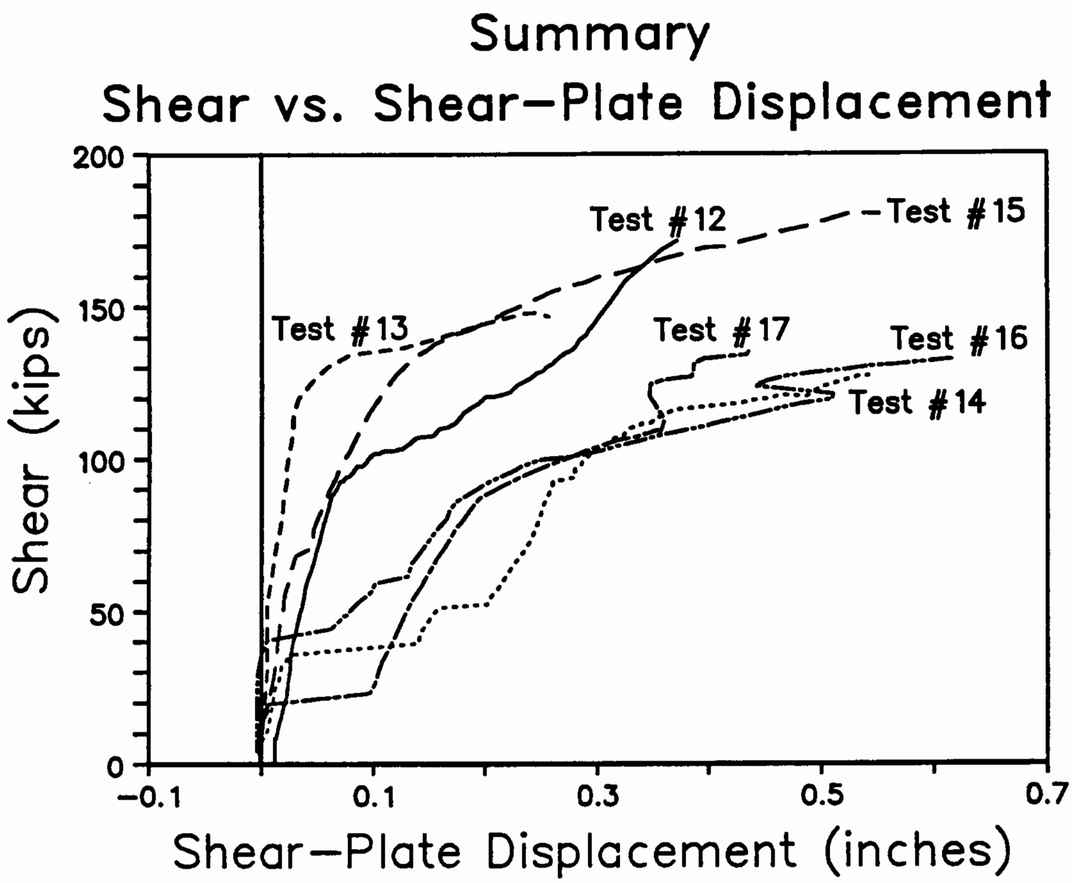

Summary – Shear–Rotation Graphs
Shear (kips)
Rotation (radians)
Test #15
Test #12
Test #13
Test #16
Test #14
Test #17
0
50
100
150
200
0
0.02
0.04
0.06
0.08
0.1


Summary – Weld Moment vs. Rotation
Weld Moment (kip–inches)
800
600
400
200
0
–200
–400
0
0.02
0.04
0.06
0.08
0.1
Rotation (radians)
Test #12
Test #17
Test #15
Test #13
Test #16
Test #14


Summary – Bolt Moment vs. Rotation
Bolt Moment (kip–inches)
800
600
400
200
0
–200
–400
0
0.02
0.04
0.06
0.08
0.1
Rotation (radians)
Test #12
Test #17
Test #13
Test #15
Test #16
Test #14


Summary – Weld Moment vs. Shear
Weld Moment (kip–inches)
800
600
400
200
0
−200
−400
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
Test #12
Test #15
Test #17
Test #13
Test #16
Test #14


Summary – Bolt Moment vs. Shear
Bolt Moment (kip–inches)
800
600
400
200
0
−200
−400
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
Test #12
Test #17
Test #15
Test #13
Test #16
Test #14


Summary
Shear vs. Shear-Plate Displacement
Shear (kips)
Shear-Plate Displacement (inches)
200
150
100
50
0
-0.1
0.1
0.3
0.5
0.7
Test #12
Test #13
Test #14
Test #15
Test #16
Test #17

CHAPTER FOUR

MATERIAL TESTS

4.1 RESULTS OF COUPON TESTS

Tension tests were performed on coupons made from girder sections and shear plate material. One coupon was taken from the girder web and one from the flange of each section. Tests were performed according to the ASTM-E6 standard governing "Standard Methods of Tension Testing of Metallic Materials". Properties of the coupons can be found in Table 4.1 Stress-strain curves, such as the one in Figure 4.1, can be found in Appendix D.

Results from the shear tab tests were used to determine the shear rotation curves, as was done in the earlier beam-to-column connections. This is explained in the section 2.5. As shown in the stress-strain curve, the shear tab material acted as a cold worked section. It did not have a well defined yield point, and the 2% offset yield was only around 32 ksi. Ultimate shear was about 58 ksi. This indicates that significant yielding should have been seen in the connection.

Coupons from the girder sections performed on a normal manner. In general steel was somewhat stronger than A36, but in no case was it weaker. Thus results dependant on the flexibility of the connection are conservative. However, if the material strength is lower, deformations will be larger.

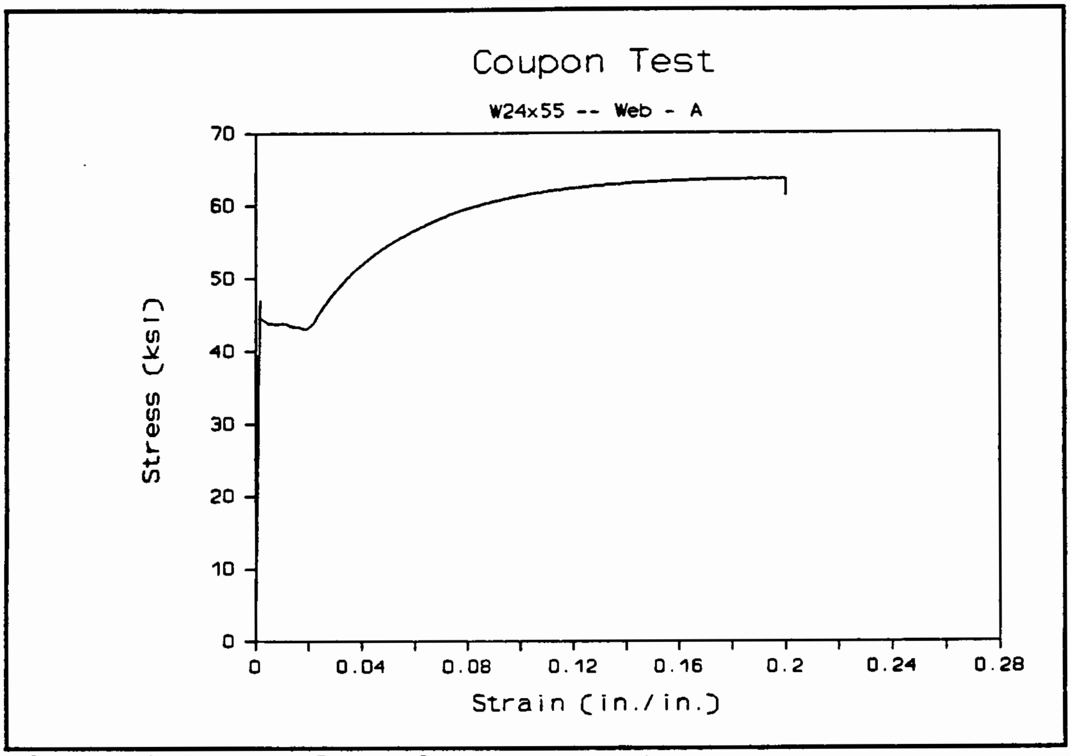


Figure 4.1 Results of Coupon Test

CHAPTER FIVE

ANALYSIS OF EXPERIMENTAL RESULTS

5.1 INTRODUCTION

One purpose of this project was to establish design procedures based on the behavior of beam-to-girder connections. For the most part, it appears that the current design methods provide an adequate factor of safety against failure due to shear loads. Thus, most of the conclusions of this report are qualitative, with the exception of the determination of the eccentricity. By observing how the specimen fails, it is possible to understand the failure modes, and the potential problems of beam-to-girder connections.

5.2 DETERMINATION OF INFLECTION POINT

As seen in previous connection tests, the inflection point on the beam tends to move closer to the connection as the shear load and rotation increase. However, in these tests the girder web contributes significantly to the flexibility of the connection. Much of the movement is not caused by yielding in the shear plate, but by bending, and eventually yielding, in the girder web. This is a very significant divergence from beam-to-column shear tab connections, where connection flexibility had to come from the stiff shear plate. This can result in a significant moment transferred by the connection. However, from comparing the graphs of strain in the girder web to the deflection of the shear tab, it is apparent that girder web yielding and plate yielding occurred simultaneously in most cases. Thus both mechanisms contribute to connection softening. In general, first yielding occurred between 60% and 85% of estimated ultimate capacity.

In order to be able to correlate eccentricity with other variables, a value for eccentricity had to be chosen from the plots, shown in Figure 5.1. This was developed by determining a lower and upper bound for each curve and then averaging them to determine the eccentricity as shown in the graphs in Appendix C. Each of the values have some uncertainty associated with it, which will be shown in the correlation graphs.

The stiffer the connection, the greater will be the eccentricity. In beam-to-column connections (where the shear plate was welded to the flange) it was found that the connection length, directly related to the number of bolts, was the most important variable in determining eccentricity. Other variables which were thought might contribute to eccentricity in beam-to-girder connections are girder web thickness, girder web clear height, and percent free span in the girder web. From these variables, correlations were made.

Connection stiffness is directly related to girder web clear

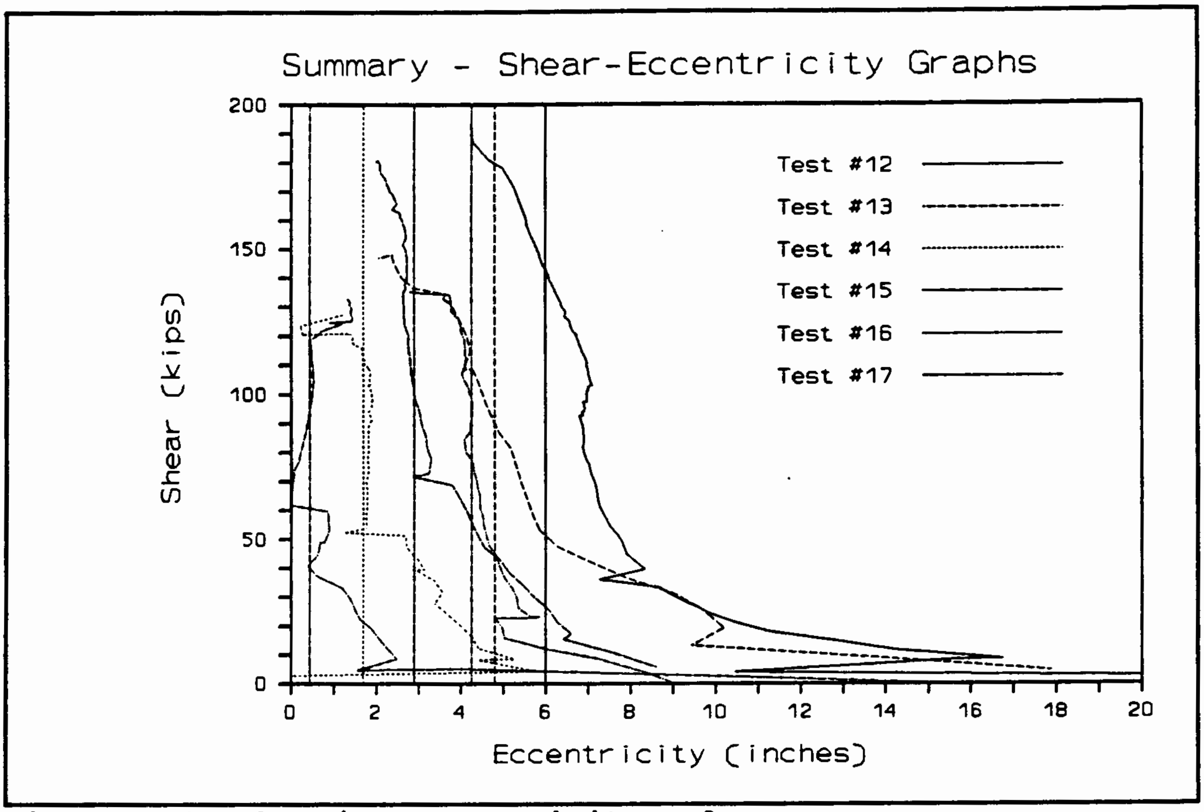


Figure 5.1 Choosing eccentricity values

height and free span, and indirectly to girder web thickness and connection length. Several correlations were looked at with these four variables. It appeared that the most important variables in determining the eccentricity were shear plate length, girder web thickness, and girder clear web height. The test points, their uncertainties, and the chosen correlation are shown in Figure 5.2.

In all tests bolt spacing was chosen to be 3 inches. Plate length was then 3 times the number of bolts. Thus the two important variables in the correlation line are number of bolts and the AISC tabulated value $(h_c/t)_w$. Eccentricity can then be defined as:

$$e_w \cong \frac{40}{(h_c/t)_w} \times N \quad \text{(inches)}$$

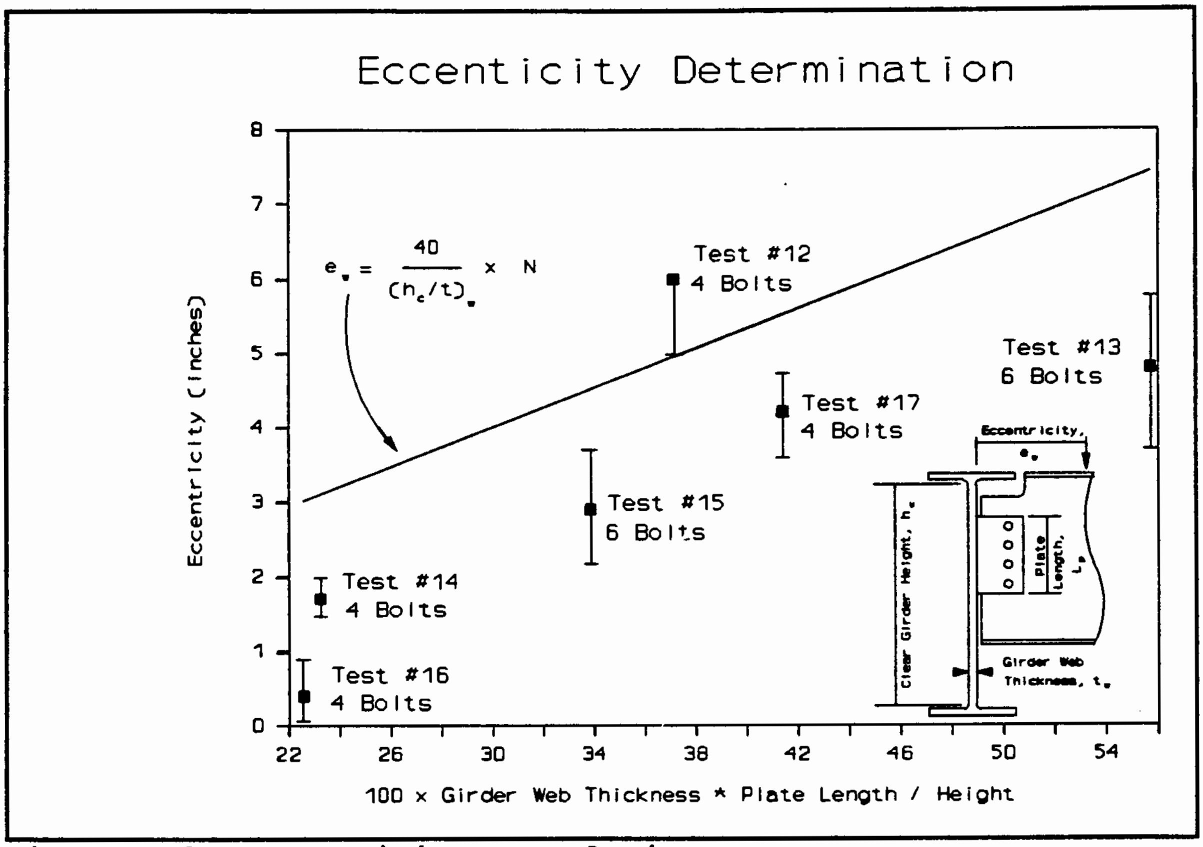


Figure 5.2 Eccentricity correlation

5.3 FAILURE PREDICTION USED DURING TESTING

In the tested connections, it was believed that ultimate failure mode would be either weld fracture or bolt failure. Using information gained in previous tests, welds were made so that bolt failure would occur first. The ultimate shear capacity of an A490N bolt was estimated to be 30 kips in shear. Thus capacity for 4-bolt connections was predicted to be 120 kips, while the 6-bolt connections were estimated to withstand 180 kips shear force.

The eccentrically applied shear force led to additional tension stresses on the top of the weld. Thus in all cases there was some weld fracture before the ultimate expected shear capacity was reached. It should be noted though, that the weld fractures were quite ductile in manner. For example, in the first test, the connection achieved 50% more load even after the weld began to crack. Only in the second test, where the weld was of poor quality and the connection very stiff did the weld tear progress rapidly.

5.4 SPECIAL CONSIDERATIONS FOR DESIGN

As stated earlier, the design of beam-to-girder connections based on specifications for beam-to-column connections appears to be valid. There are however a few considerations of which designers must be aware.

From the results of test #13, it was seen that weld quality had an impact on connection reliability. Weld strength and ductility will be more important in stiffer connections in which higher moments are developed. Further more, the twisting of the beam seen often in these tests can lead to additional stresses on the weld. Finally, the out of plane bending of the girder web causes additional tensile stresses to develop in the weld, especially at the ends. If weld failure is a concern, either because of a very stiff connection, or a very flexible girder web, the shear plate could be welded to the girder flanges as well as the web.

In a one-sided connection there may be problems with rotation of the girder. Even in cases where the flanges are braced at regular intervals, some rotation will occur between the braced points. Additionally, the girder web may experience some deformation, and possibly yielding at loads just above service conditions. For most applications this will not be a problem. However, in situations where the girder web can not be allowed to rotate, plates should be welded to the top flange of the girder.

One final consideration is short free depth of girder web. If this distance is too short, the failure mode may be rotation of the beam until its flange touches the girder flange (See Figure 3.13). If this occurs, the beam will begin to rotate about the point of contact, causing stresses to increase on the top part of the connection. Additionally, the flange of the girder could be damaged. A scale drawing showing the beam rotated so that the flanges touch will give the rotation angle at which this will occur. This angle must be greater that the design angle of rotation based on simple span behavior.

CHAPTER SIX

CONCLUSIONS

6.1 CONCLUSION

The results of these tests provide both qualitative and quantitative considerations for design. An equation was developed in section 5.2 to determine the design eccentricity in beam-to-girder connections. Special failure modes which occur in beam-to-girder connections were described in the experimental results, section 3.10. An examination of how these failure modes should be considered in design was discussed in section 5.4. Results show that in general current design practice is adequate for beam-to-girder connections. Designers should be aware of peculiarities exhibited in particularly large or stiff connections.

6.2 FUTURE RESEARCH NEEDS

Beam-to-girder connections experience complex behavior which is difficult to completely ascertain in six connection tests. Further research should be done to further evaluate the behavior of these connections.

Experimentally, there are many variables in the specimens: girder web height, web thickness, connection length, and free span. Tests should be run varying each one of these parameters while holding all others constant.

Analytically, the behavior of the girder web could be examined. It could be modeled as a plate with out-of-plane forces acting on it. The free span could be looked at as a column and compared to elastic or inelastic buckling. Catenary action in the deformed girder could be looked at. Along with experimental data, analytical models could explain much more about the behavior of beam-to-girder connections.

Appendix A

References

1. A series of connection reports have been written at UC Berkeley under the supervision of A. Astaneh. These include double-angle, T-section, shear plate, and moment connections. Some consider cyclic and/or axial loads. All are published by the University of California, Berkeley, Department of Structural Engineering, Mechanics, and Materials, and are available upon request.

2. Salmon, C. G., and Johnson, J. E., *Steel Structures, Design and Behavior,* 2nd edition, Harper and Row, New York, 1980.

3. Salmon, C. G., and Johnson, J.E.

4. Cheng, J.R., Yura, J. A., "Local Web Buckling of Coped Beams," *Journal of Structural Engineering,* Oct. 1986.

5. Astaneh, A., McMullin, K. A., "Design of Single Plate Framing Connections," *Report No. UCB/SEMM-88/12,* Department of Civil Engineering, University of California, Berkeley, July, 1988.

6. Cheng, J. R., Yura, J. A., "Lateral Buckling Tests on Coped Steel Beams," *Journal of Structural Engineering,* Jan. 1988.

7. American Institute of Steel Construction, *Manual of Steel Construction, Allowable Stress Design,* 9th edition, 1989.

8. American Institute of Steel Construction.

9. American Institute of Steel Construction.

10. Astaneh, A. "Demand and Supply of Ductility in Steel Shear Connections", *Journal of Construction and Steel Research,* **14**(1989).

11. Astaneh, A.

Appendix B

Notation

d	depth of beam
e_w	eccentricity of beam inflection point from weld line
F_y	yield strength of material
F_u	ultimate strength of material
h_c	clear height of girder web
L_p	length of shear plate
M_b	moment at bolt line
M_w	moment at weld line
N	number of bolts
t_w	thickness of web
V_y	yield capacity of plate in shear
V_u	ultimate capacity of plate in shear

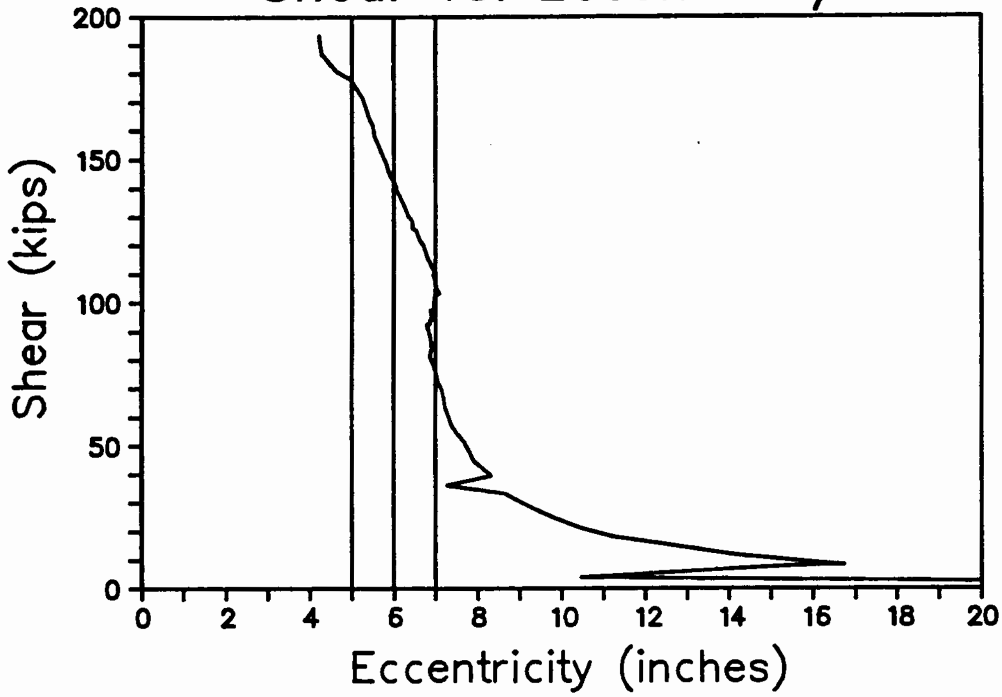
Appendix C

Eccenticity Determination

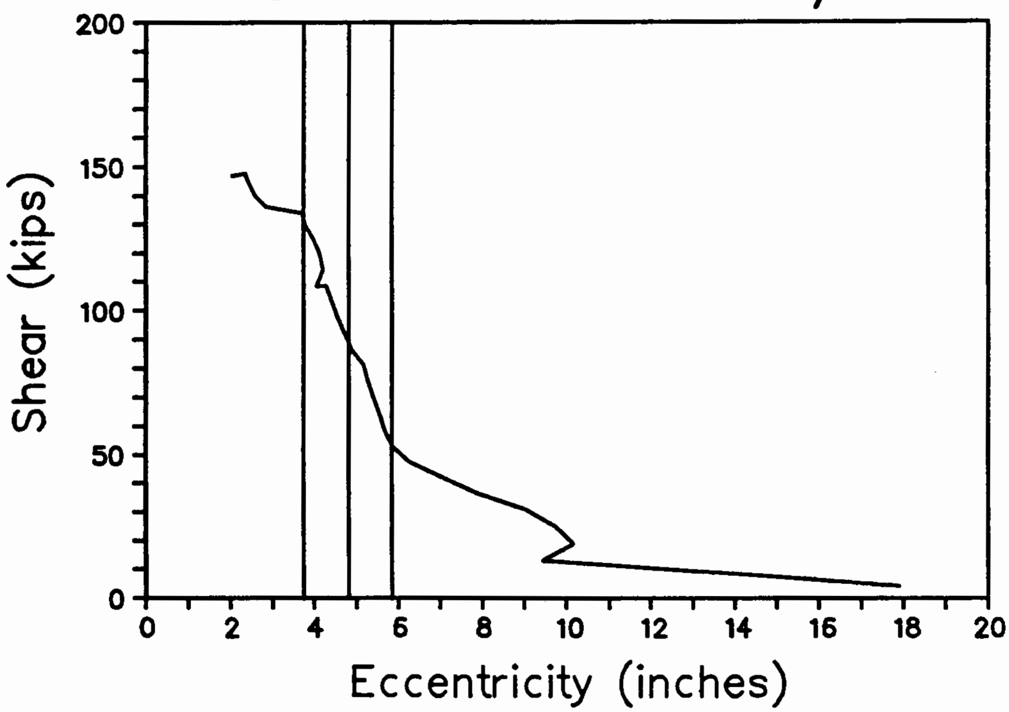
Eccentricity Determination

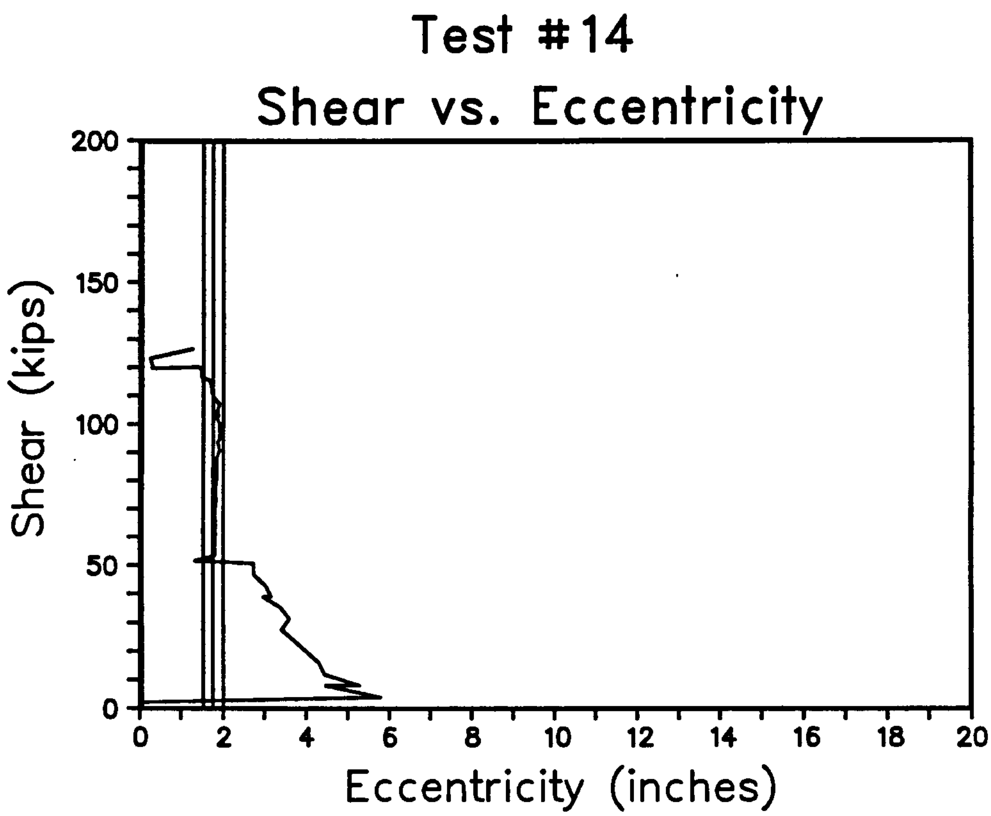
	Lower Bound	Upper Bound	Average	Upper − Lower
Test #12	5.0	7.0	6.0	2.0
Test #13	3.7	5.8	4.8	2.1
Test #14	1.5	2.0	1.7	0.5
Test #15	2.2	3.7	2.9	1.5
Test #16	0.1	0.9	0.4	0.8
Test #17	3.6	4.7	4.2	1.1

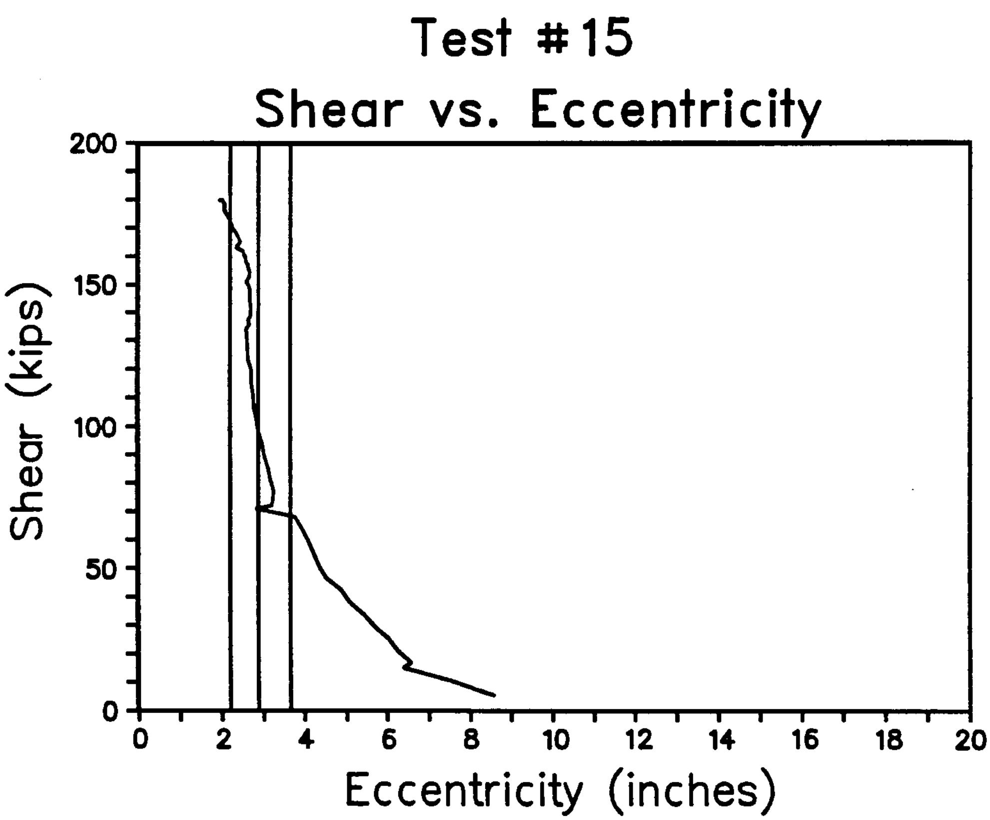
Test #12
Shear vs. Eccentricity



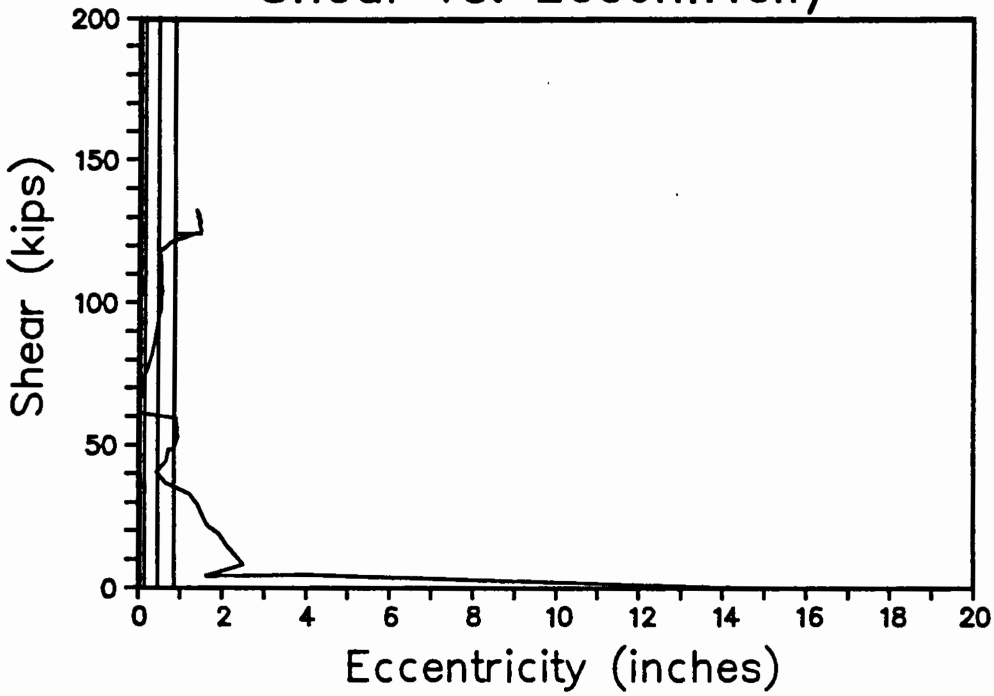
Test #13
Shear vs. Eccentricity



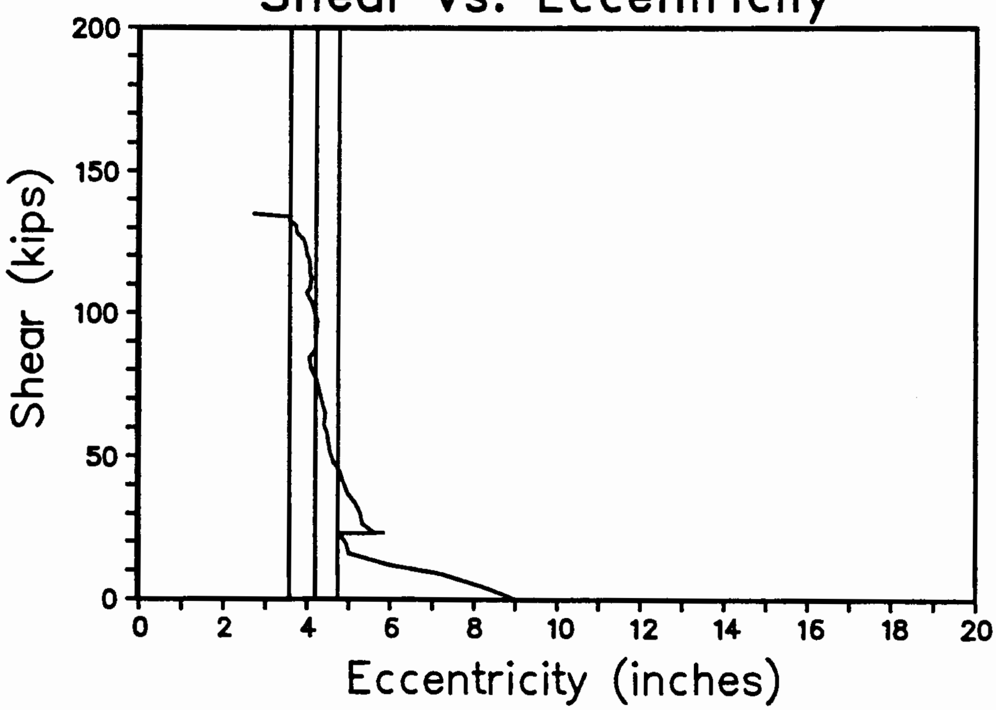

Test #14
Shear vs. Eccentricity
Shear (kips)
Eccentricity (inches)
0
50
100
150
200
0
2
4
6
8
10
12
14
16
18
20


Test #15
Shear vs. Eccentricity
Shear (kips)
Eccentricity (inches)
0
50
100
150
200
0
2
4
6
8
10
12
14
16
18
20

Test #16
Shear vs. Eccentricity



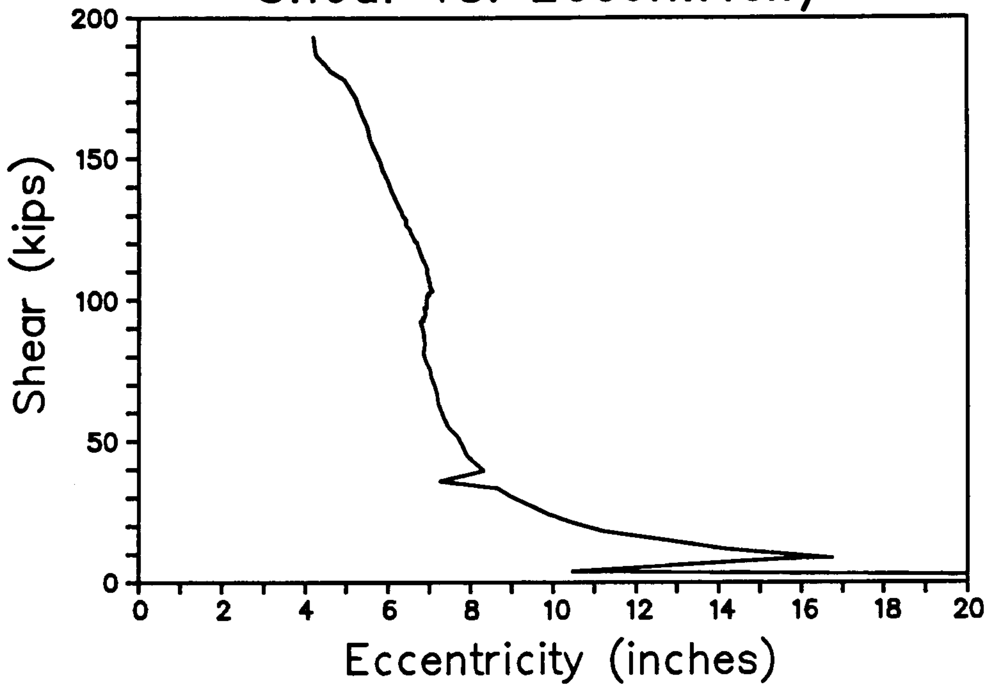
Test #17
Shear vs. Eccentricity



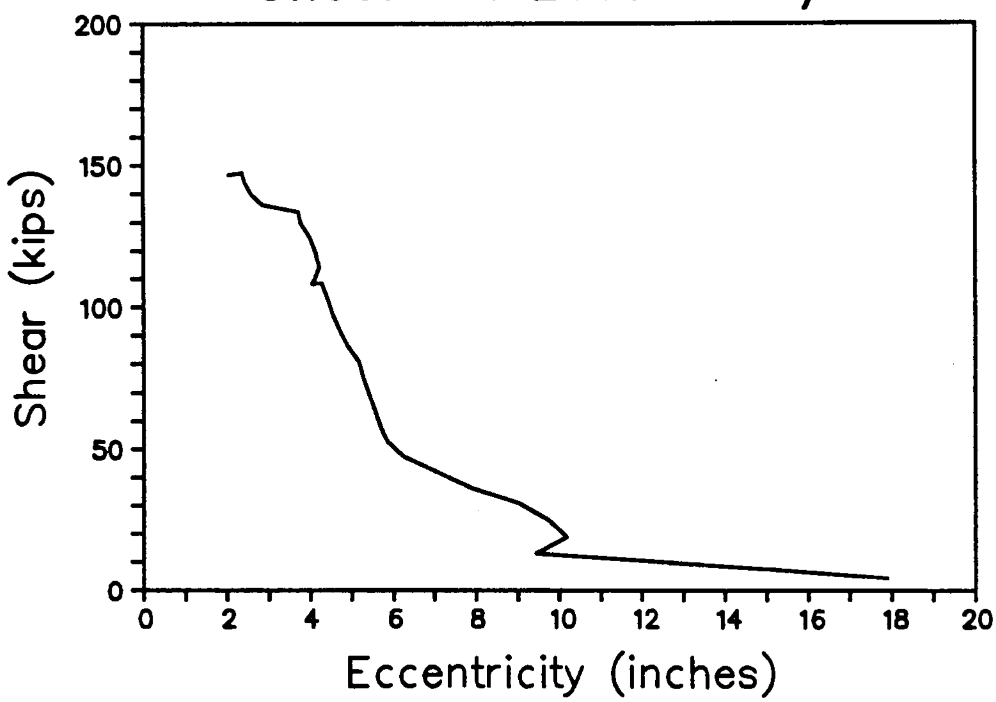
Appendix D

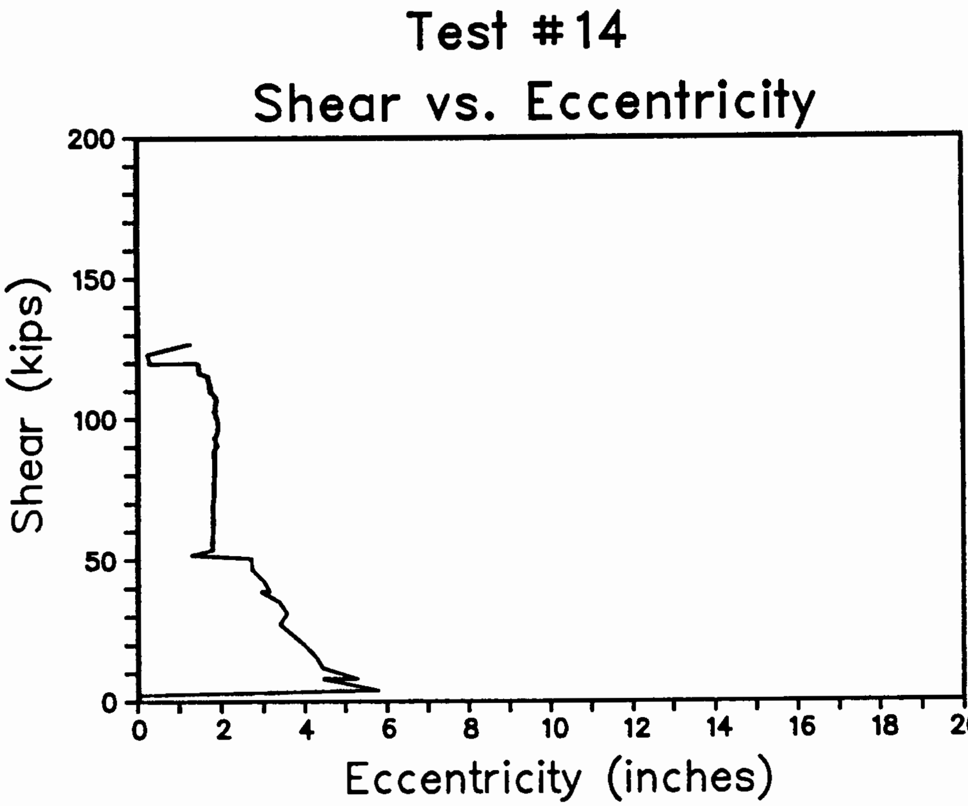
Graphs

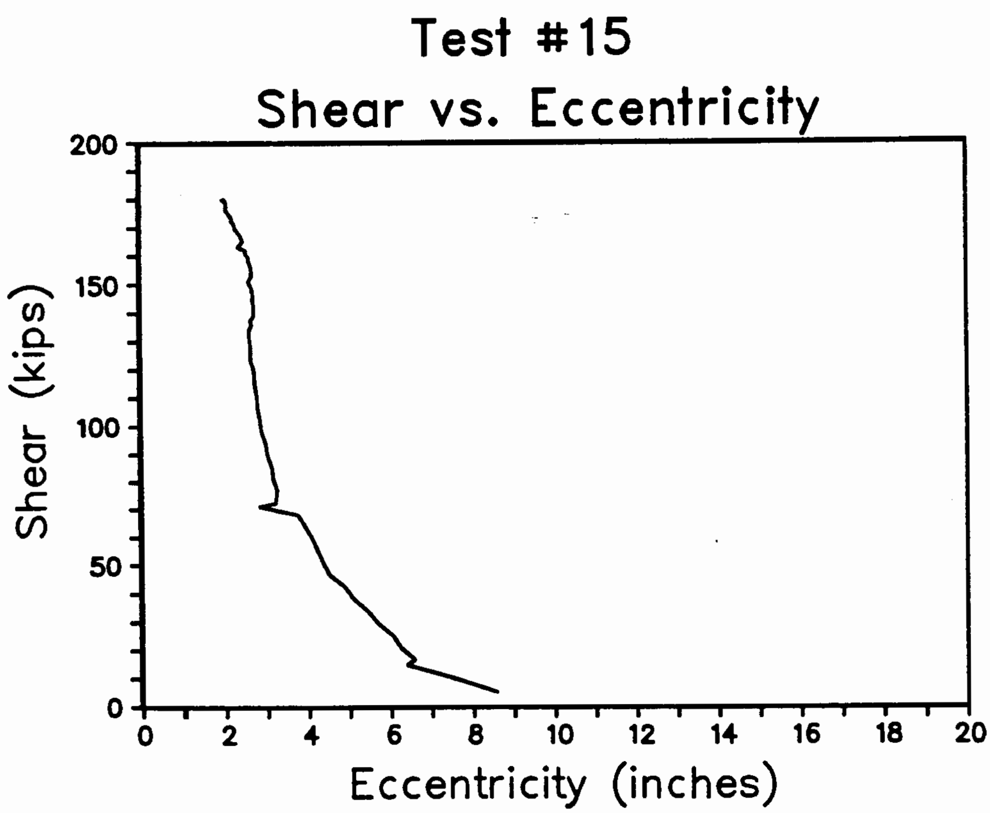
Test #12
Shear vs. Eccentricity



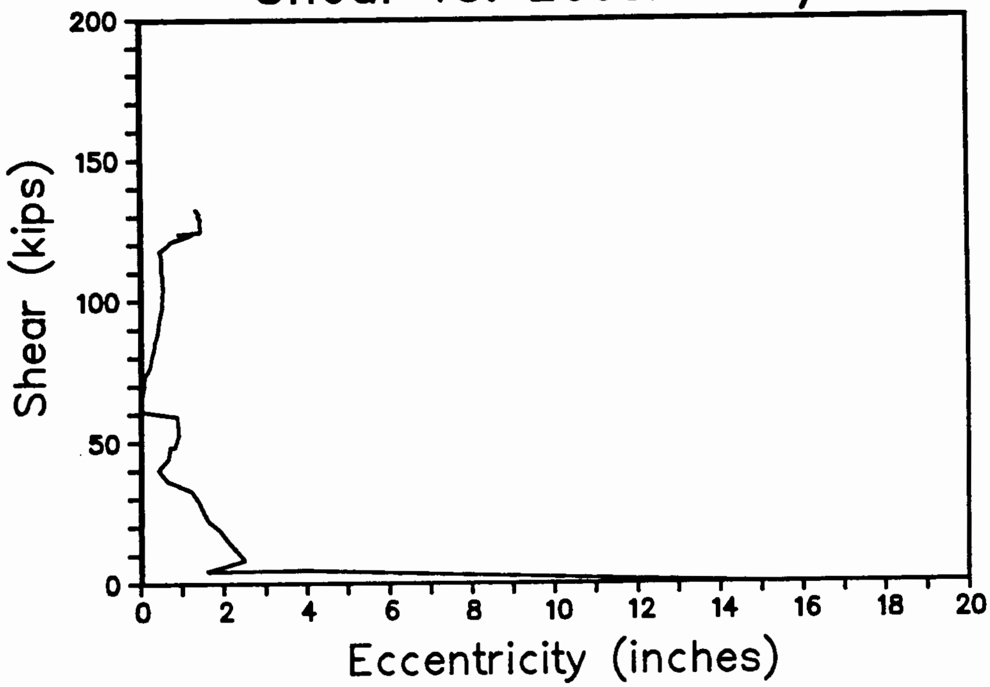
Test #13
Shear vs. Eccentricity



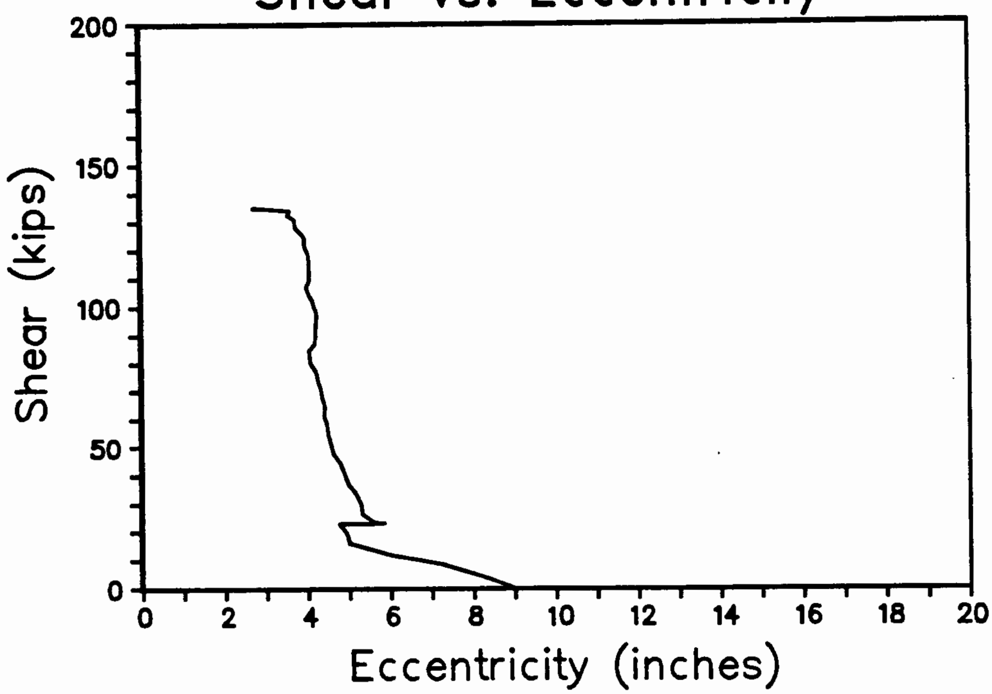

Test #14
Shear vs. Eccentricity
200
150
100
50
0
Shear (kips)
0
2
4
6
8
10
12
14
16
18
20
Eccentricity (inches)


Test #15
Shear vs. Eccentricity
200
150
100
50
0
Shear (kips)
0
2
4
6
8
10
12
14
16
18
20
Eccentricity (inches)

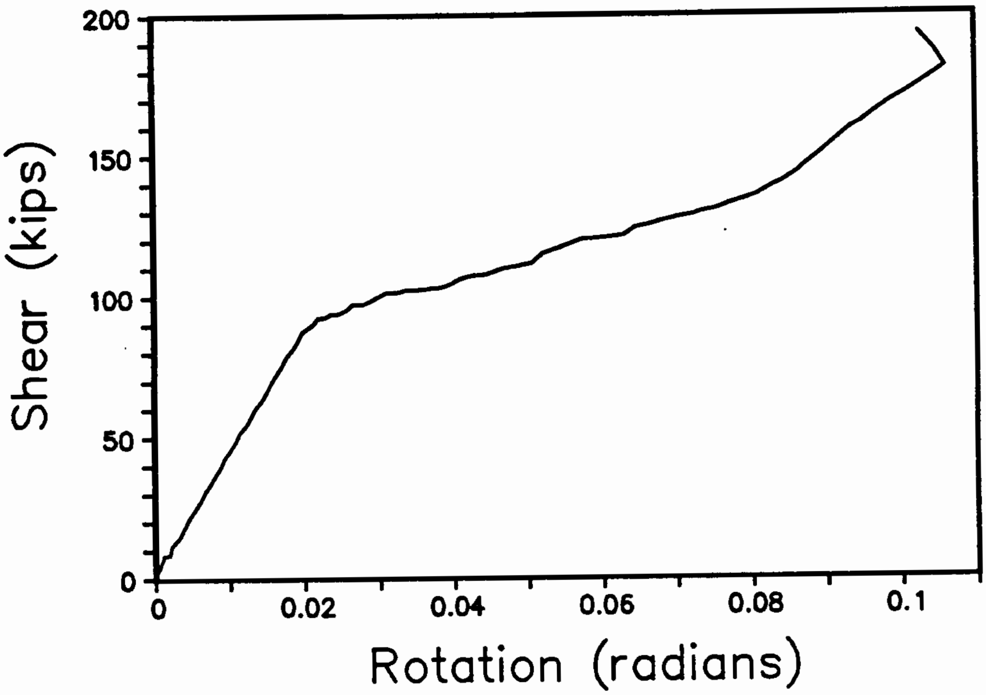
Test #16
Shear vs. Eccentricity



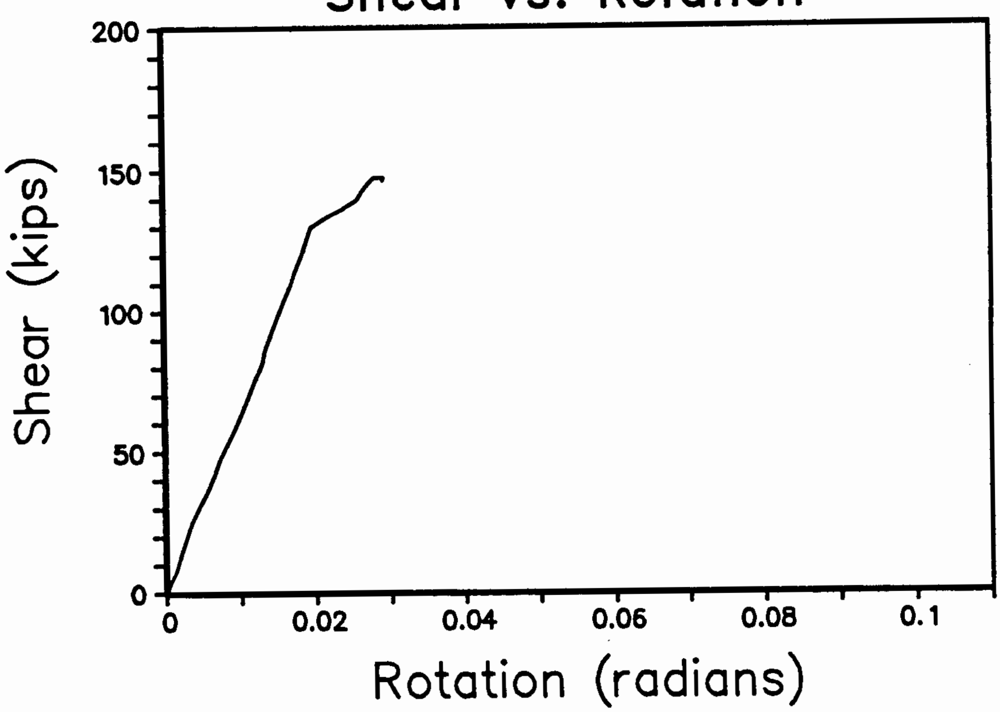
Test #17
Shear vs. Eccentricity



Test #12
Shear vs. Rotation

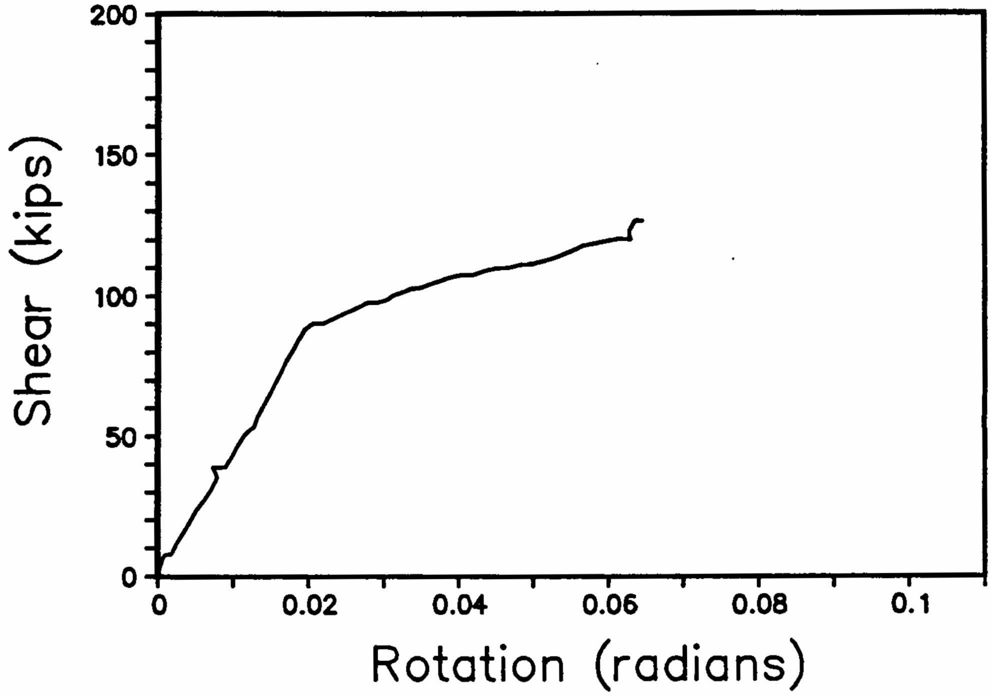


Test #13
Shear vs. Rotation



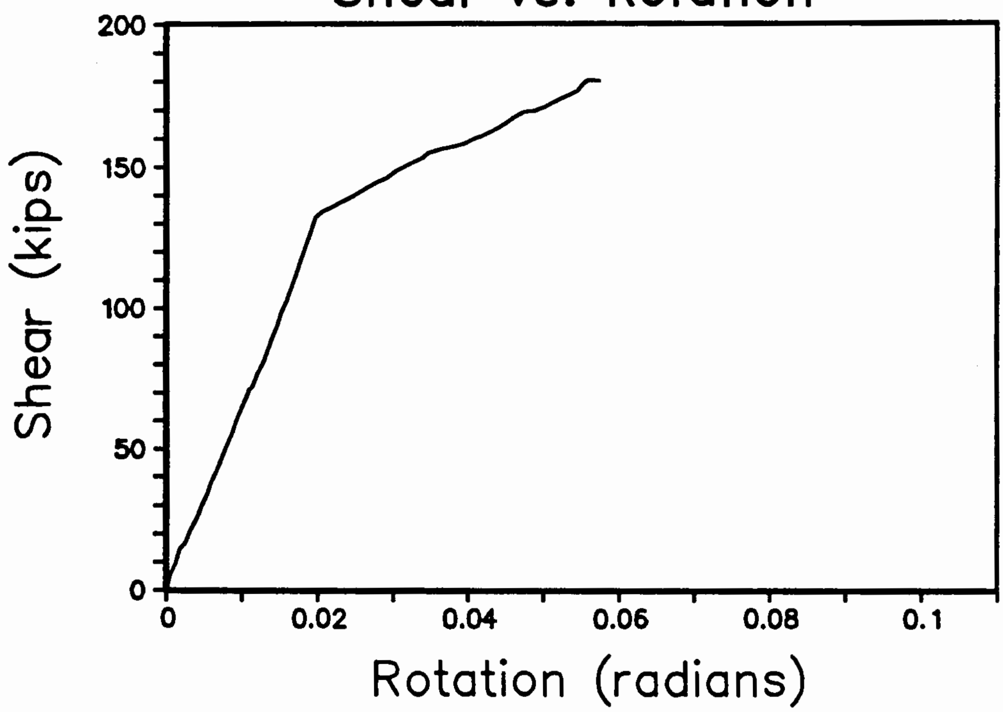
Test #14

Shear vs. Rotation



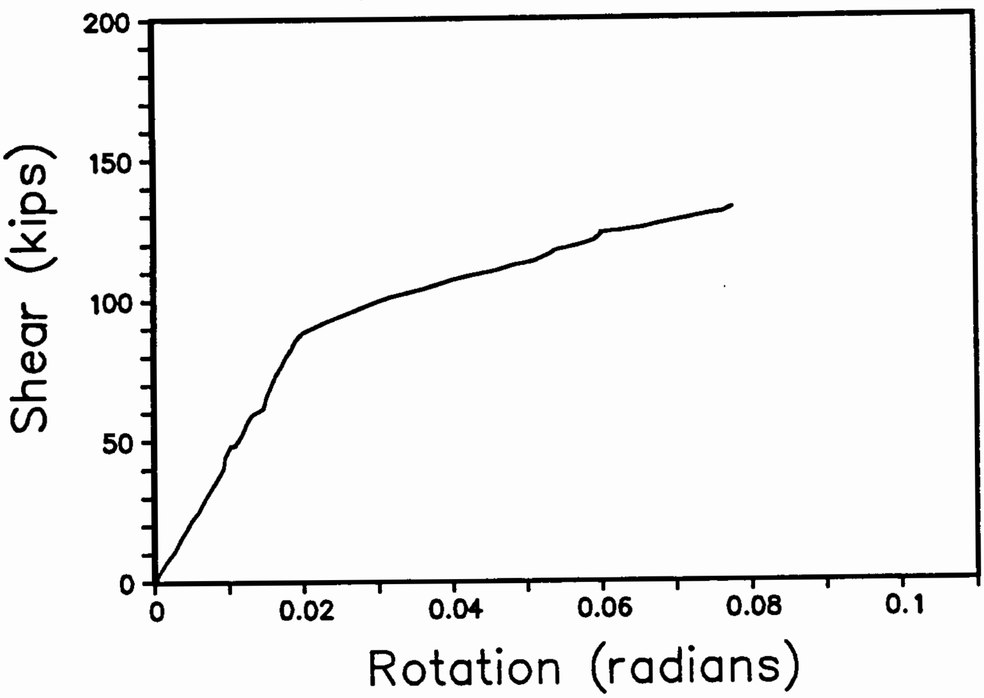
Test #15

Shear vs. Rotation



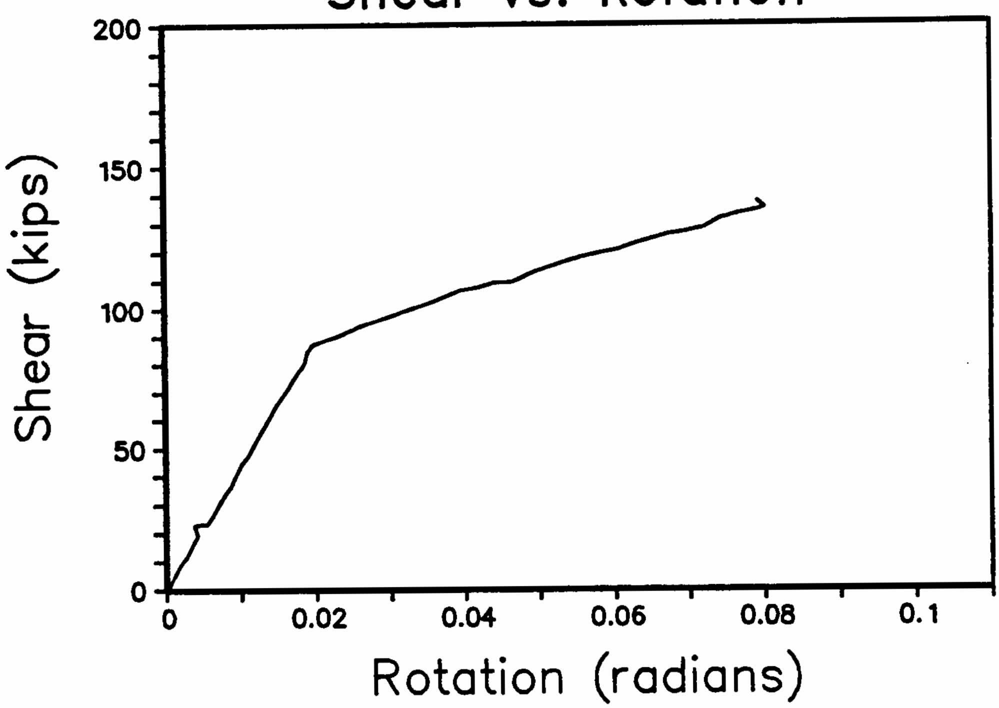
Test #16

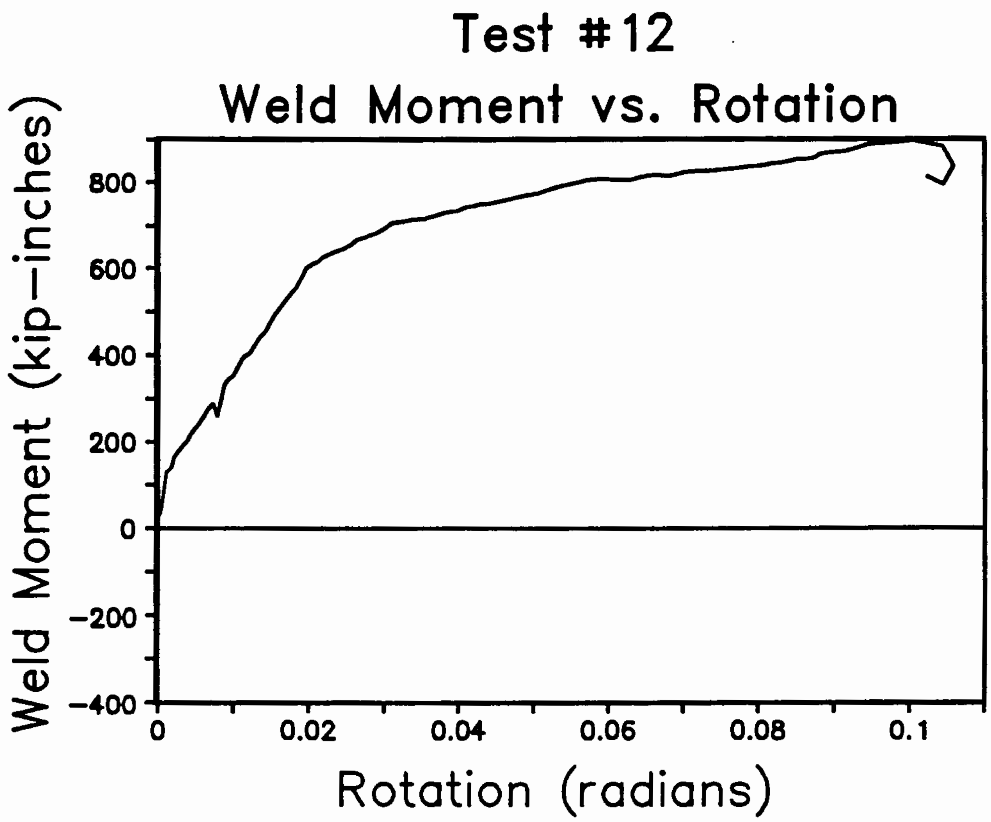
Shear vs. Rotation

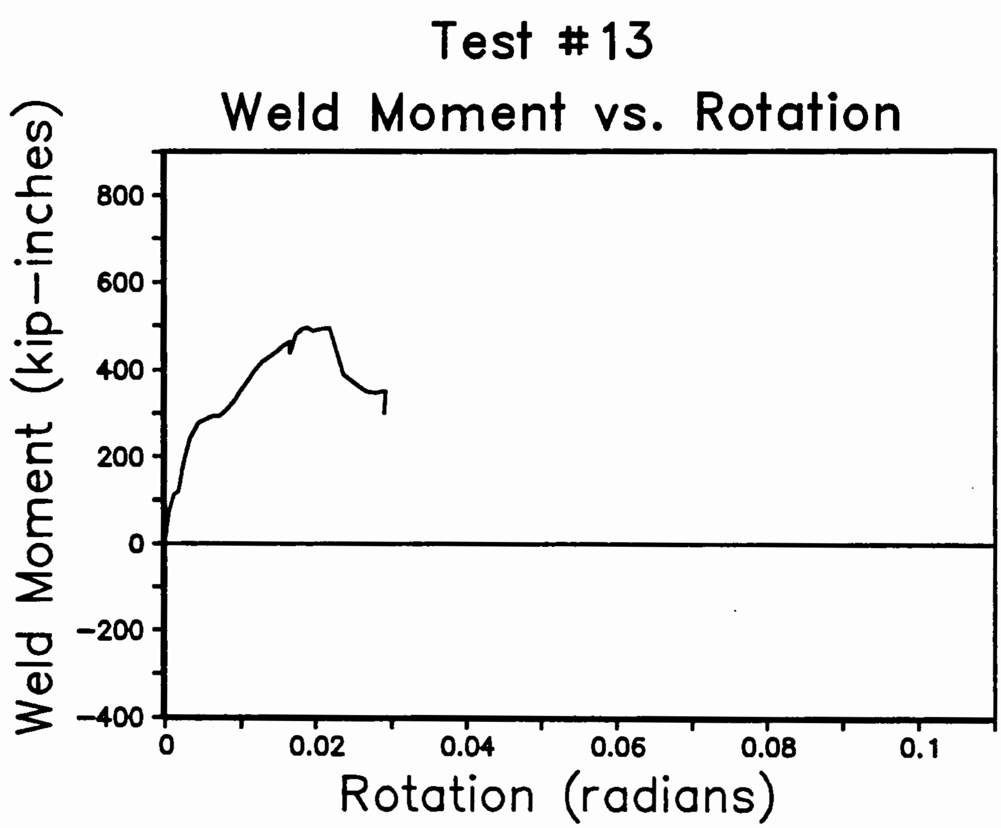


Test #17

Shear vs. Rotation

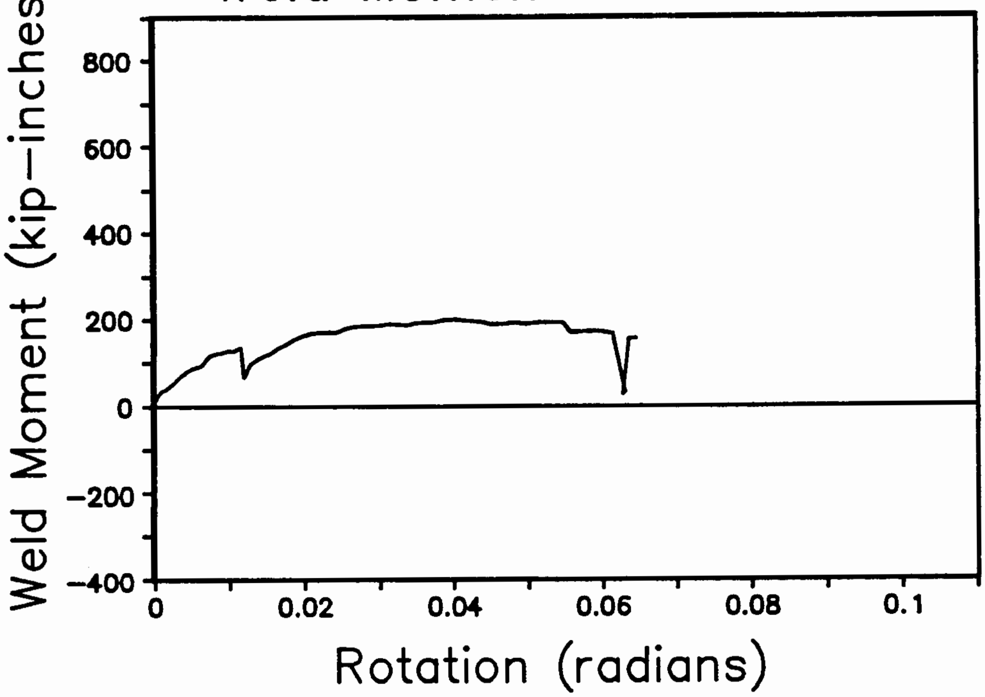



Test #12
Weld Moment vs. Rotation
Weld Moment (kip-inches)
800
600
400
200
0
-200
-400
0
0.02
0.04
0.06
0.08
0.1
Rotation (radians)


Test #13
Weld Moment vs. Rotation
Weld Moment (kip-inches)
800
600
400
200
0
-200
-400
0
0.02
0.04
0.06
0.08
0.1
Rotation (radians)

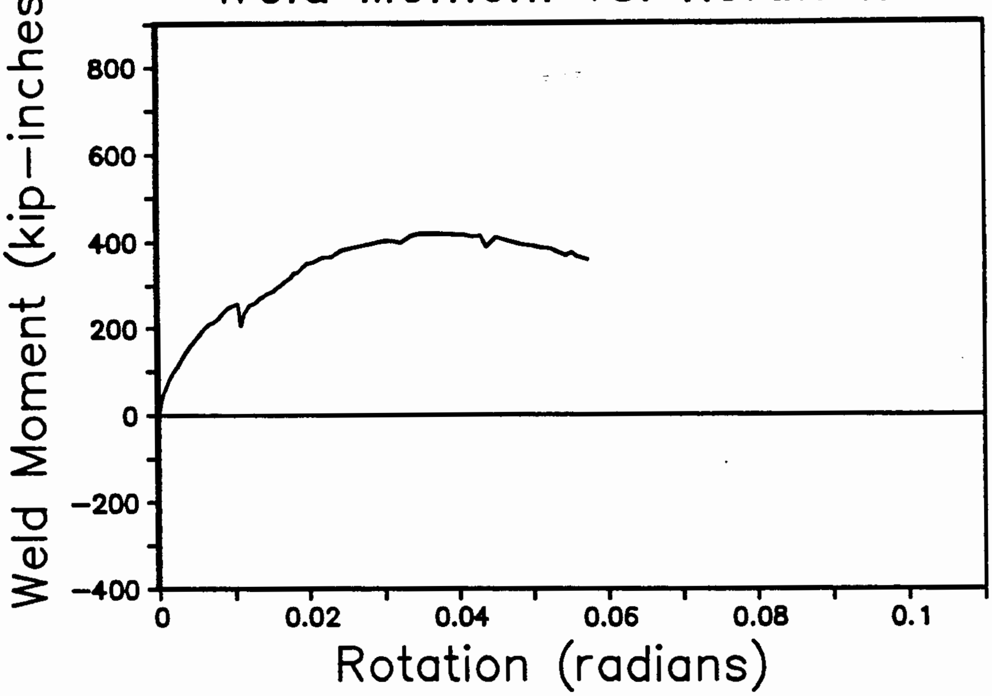
Test #14

Weld Moment vs. Rotation

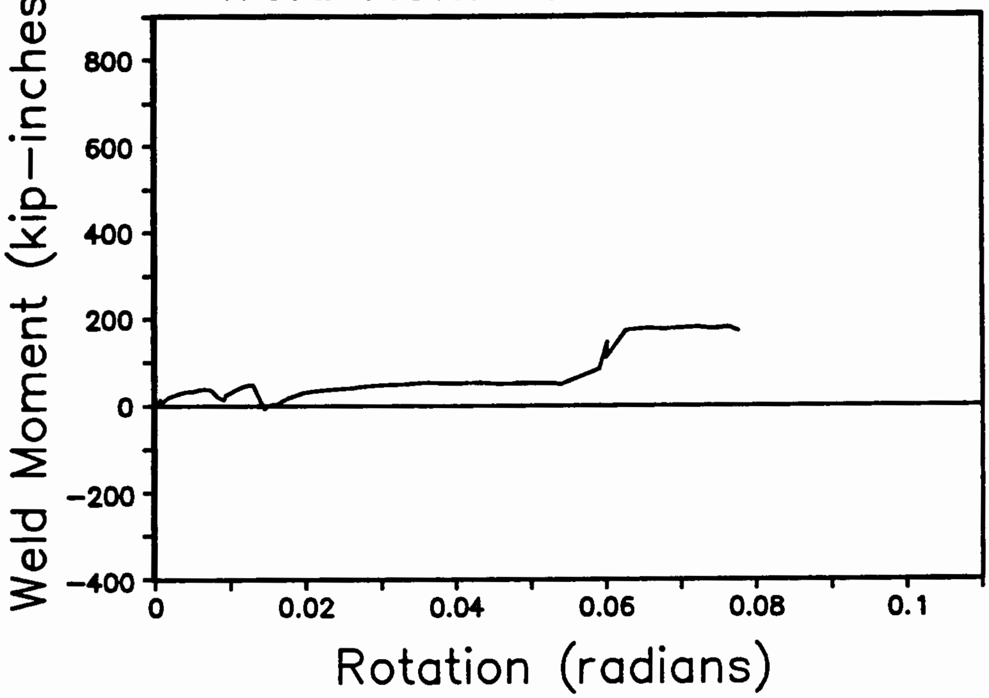


Test #15

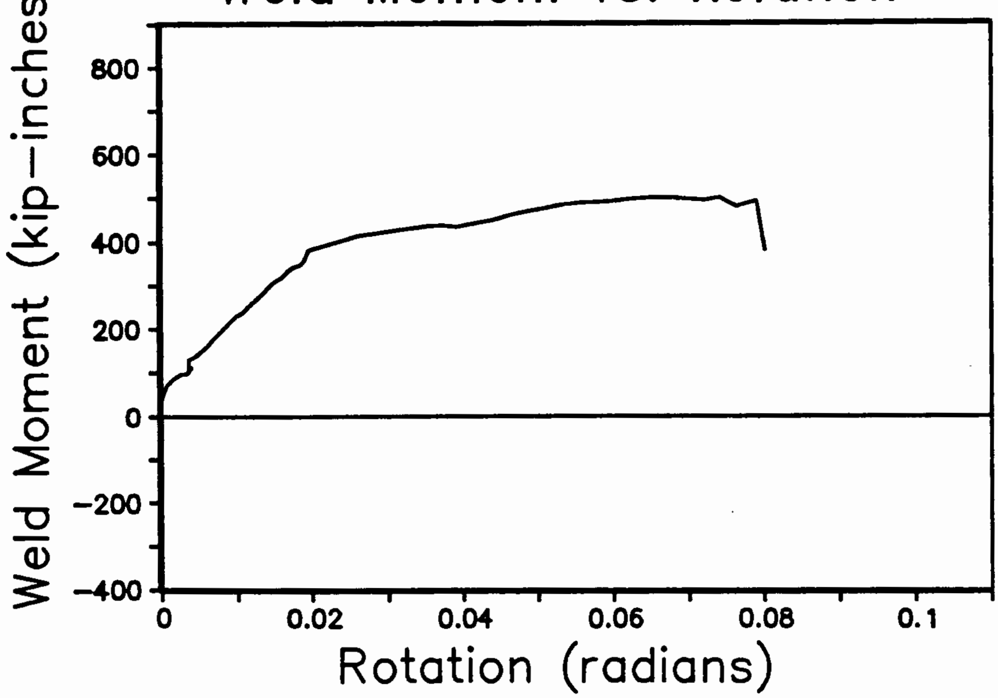
Weld Moment vs. Rotation



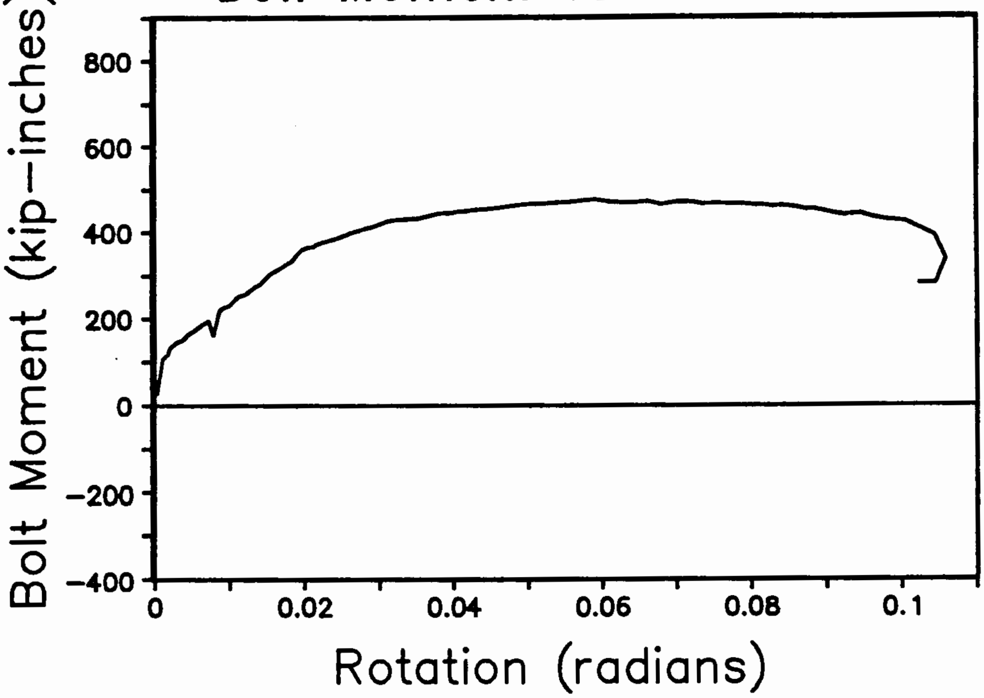
Test #16
Weld Moment vs. Rotation



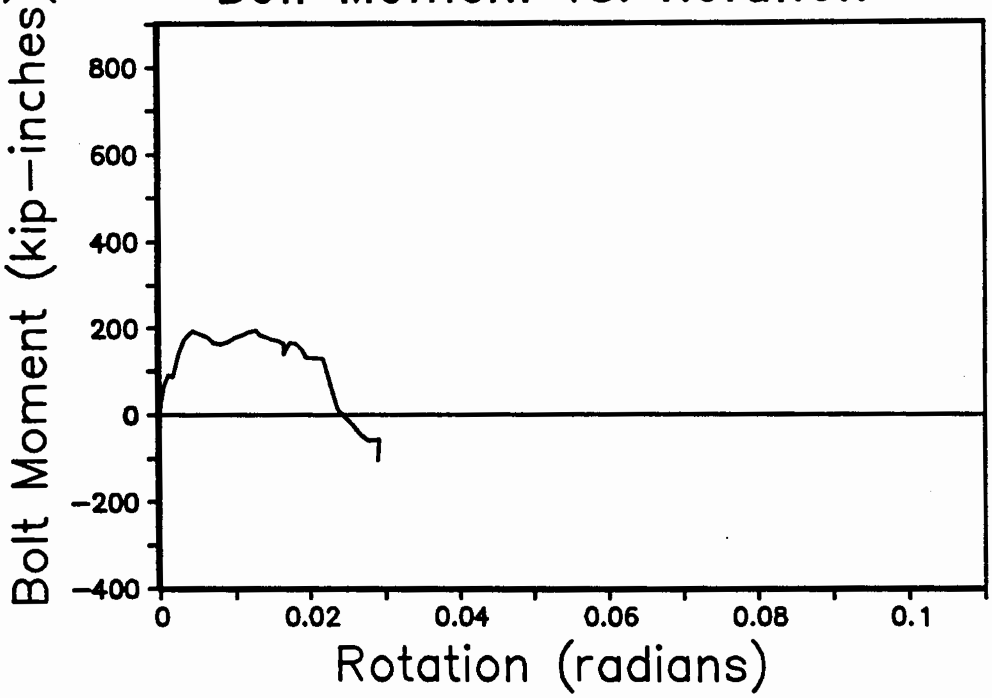
Test #17
Weld Moment vs. Rotation



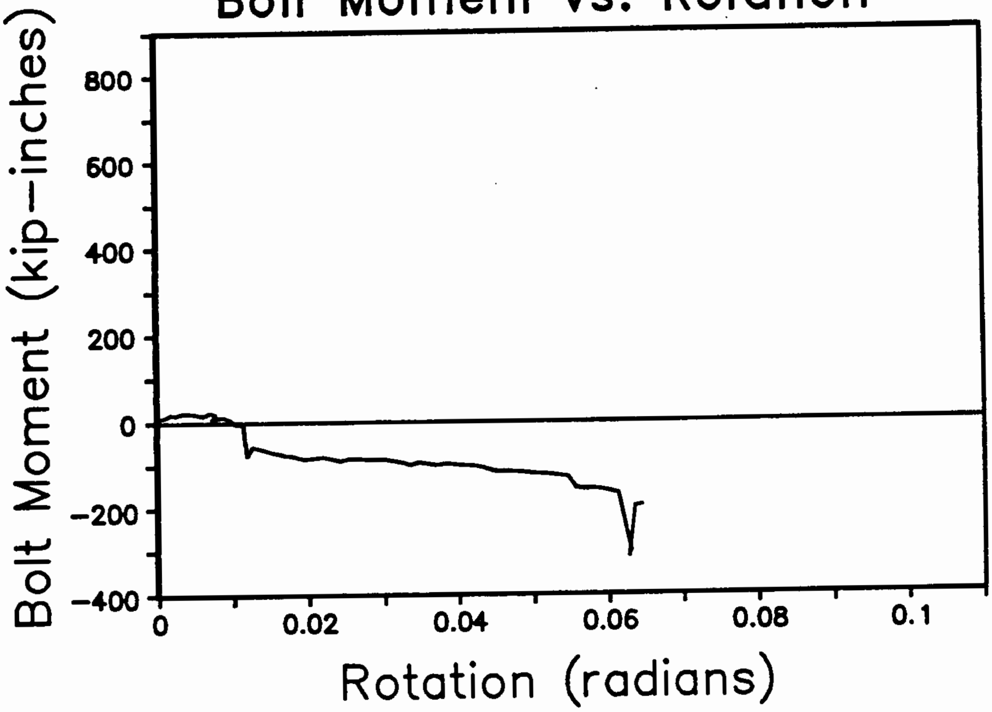
Test #12
Bolt Moment vs. Rotation



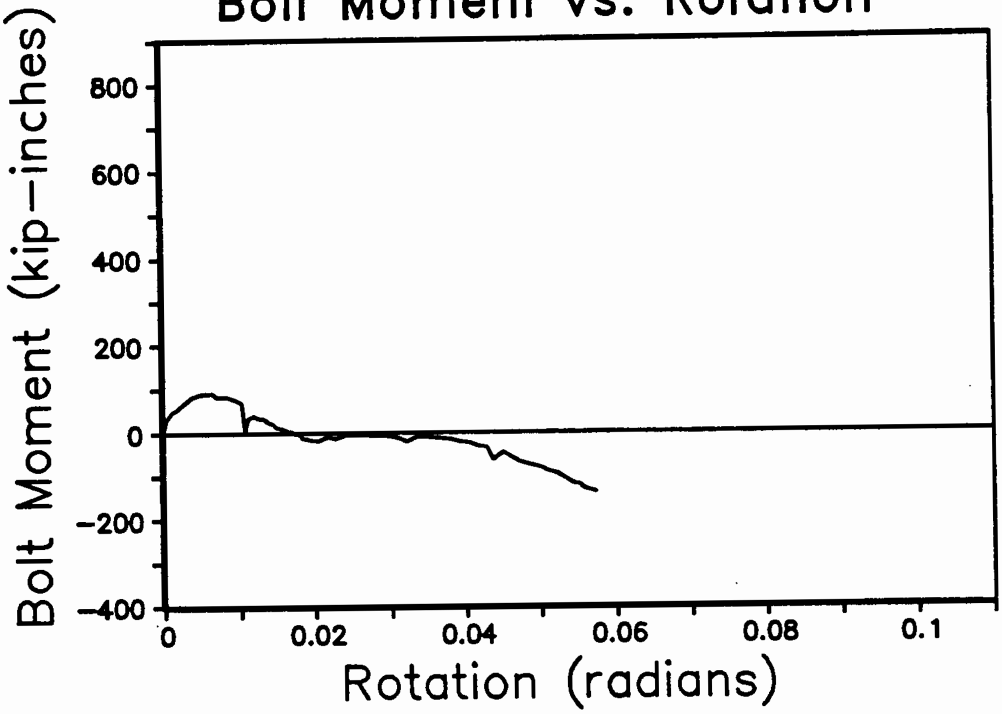
Test #13
Bolt Moment vs. Rotation

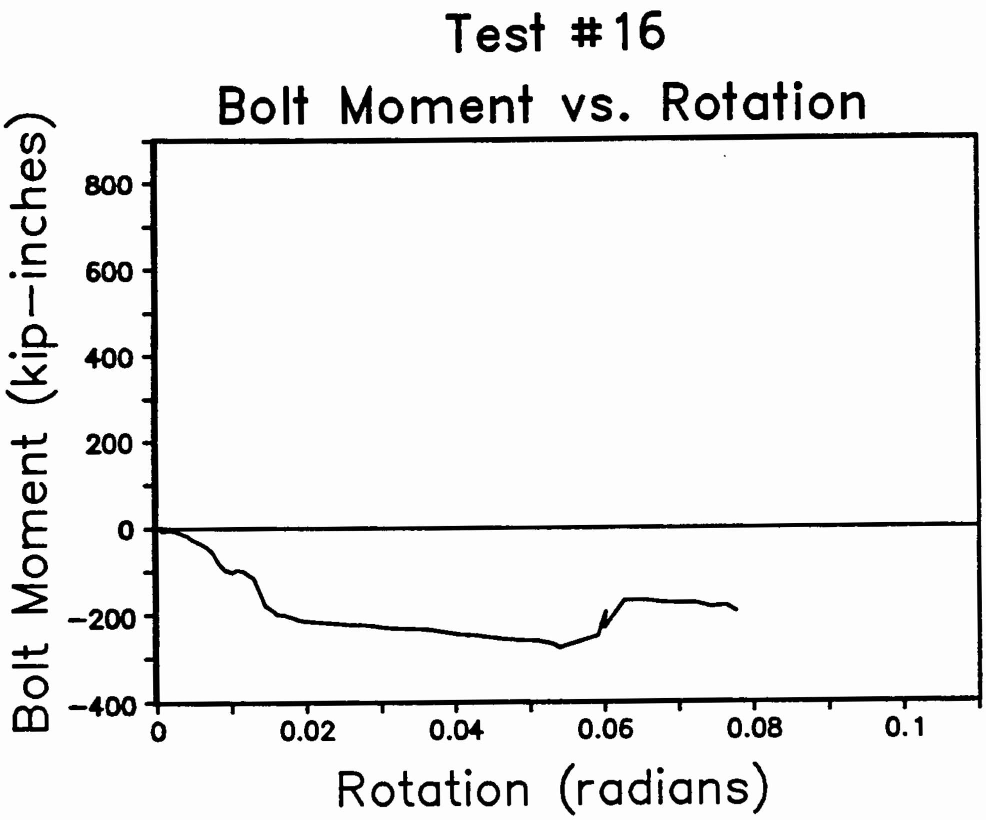


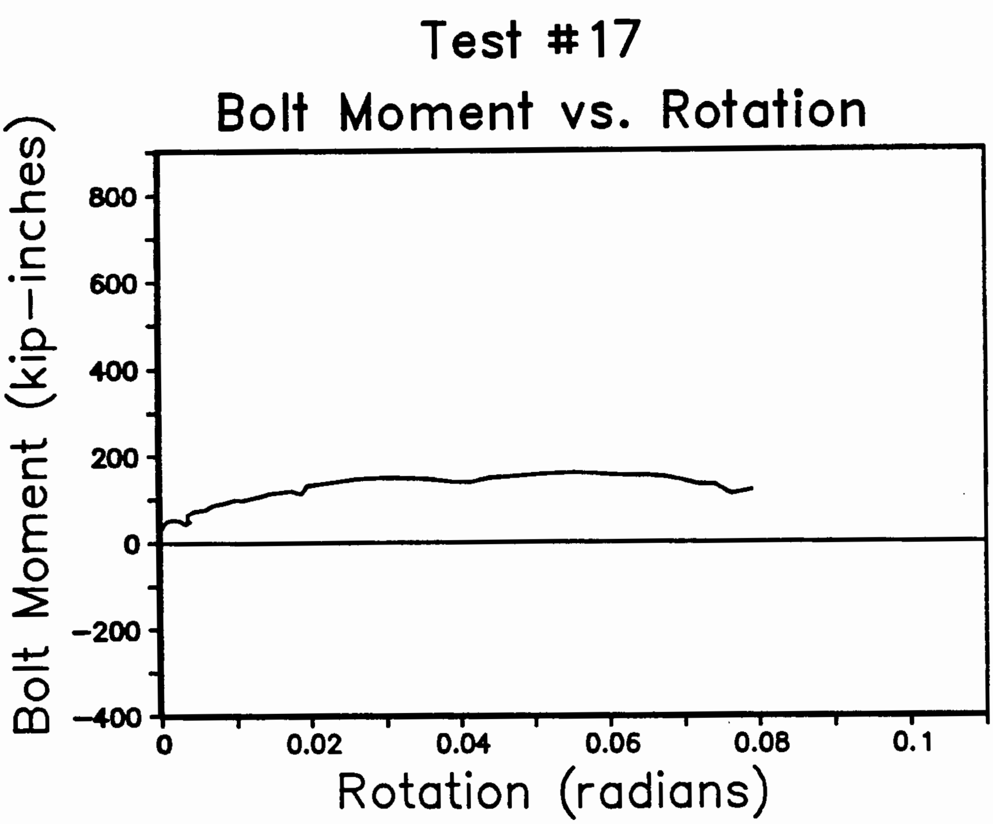
Test #14
Bolt Moment vs. Rotation



Test #15
Bolt Moment vs. Rotation

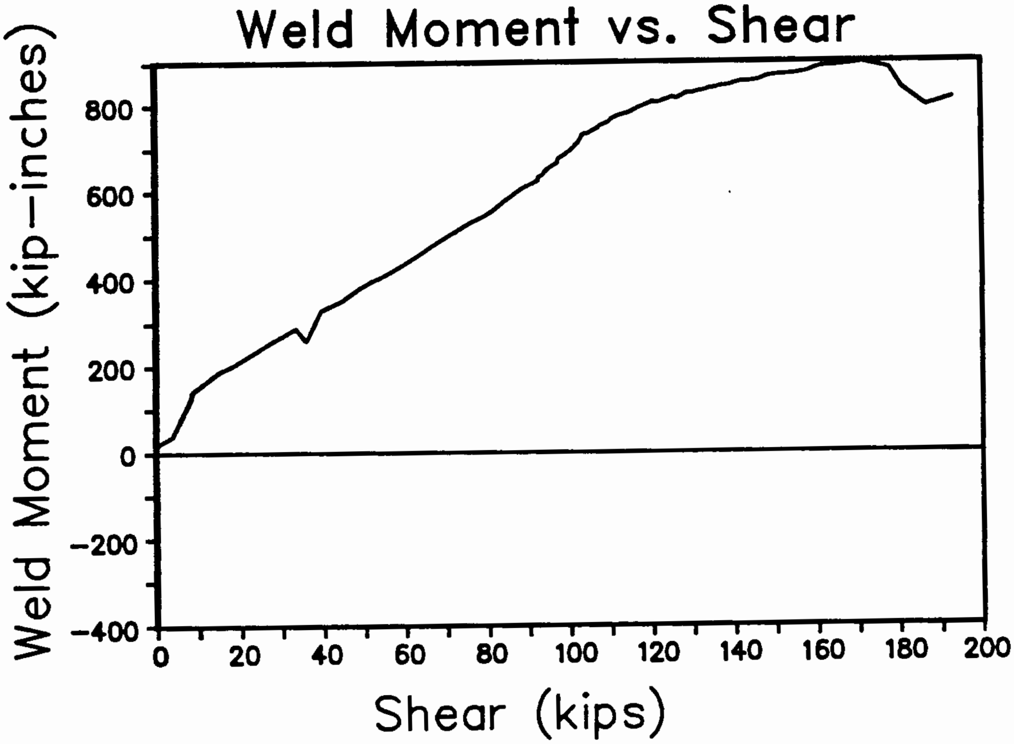



Test #16
Bolt Moment vs. Rotation
Bolt Moment (kip-inches)
800
600
400
200
0
-200
-400
0
0.02
0.04
0.06
0.08
0.1
Rotation (radians)


Test #17
Bolt Moment vs. Rotation
Bolt Moment (kip-inches)
800
600
400
200
0
-200
-400
0
0.02
0.04
0.06
0.08
0.1
Rotation (radians)

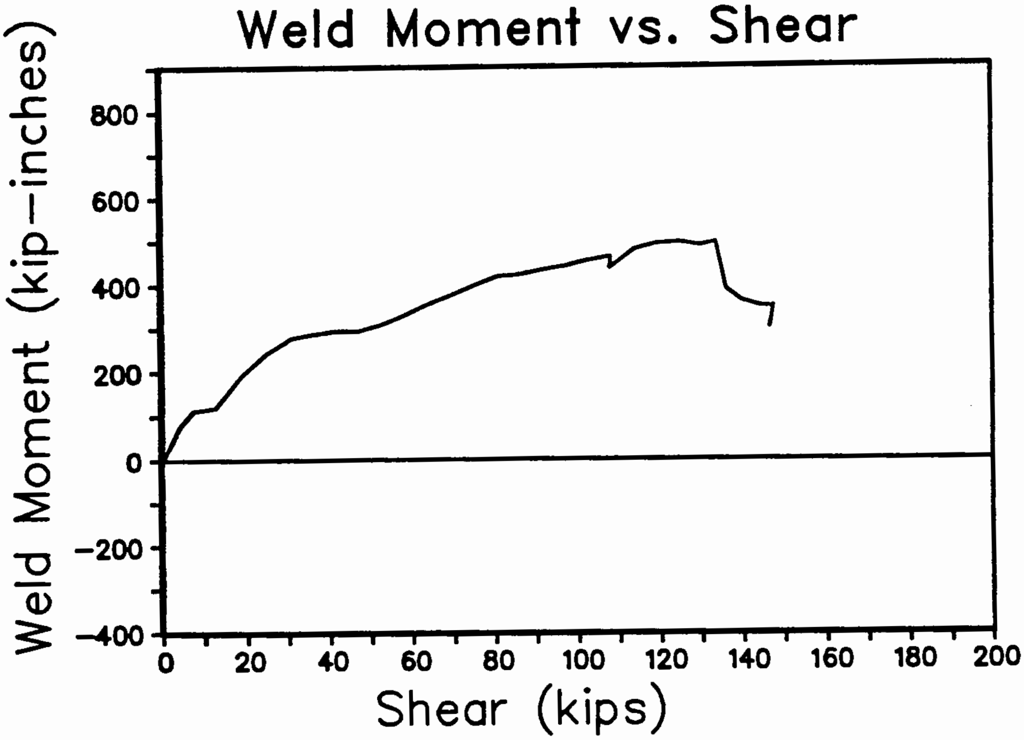
Test #12

Weld Moment vs. Shear

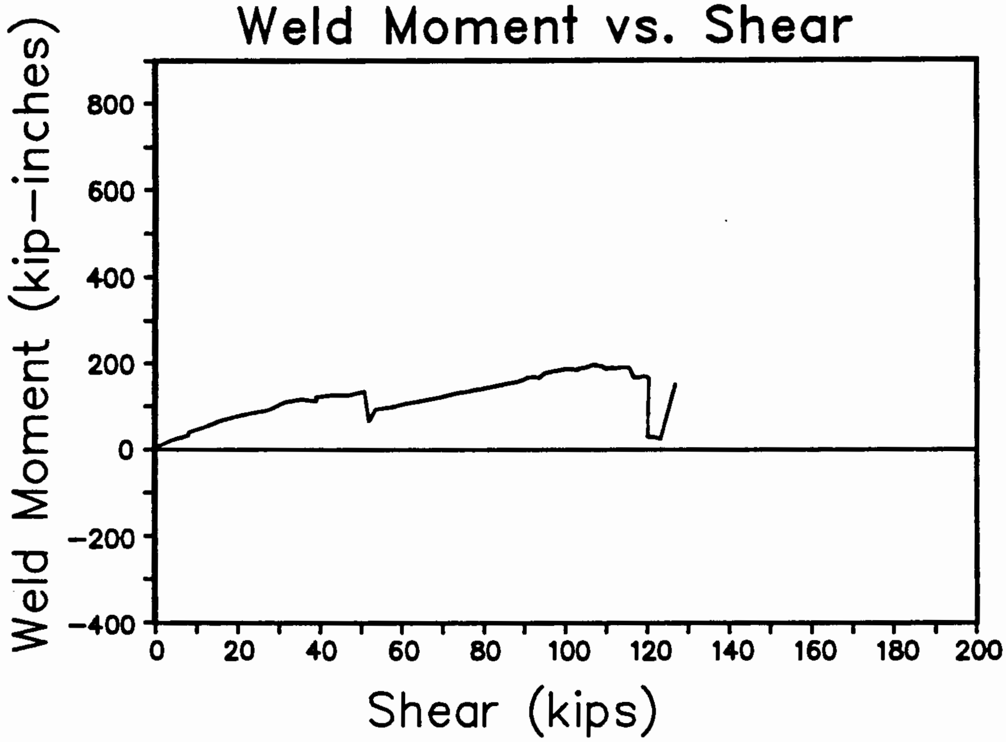


Test #13

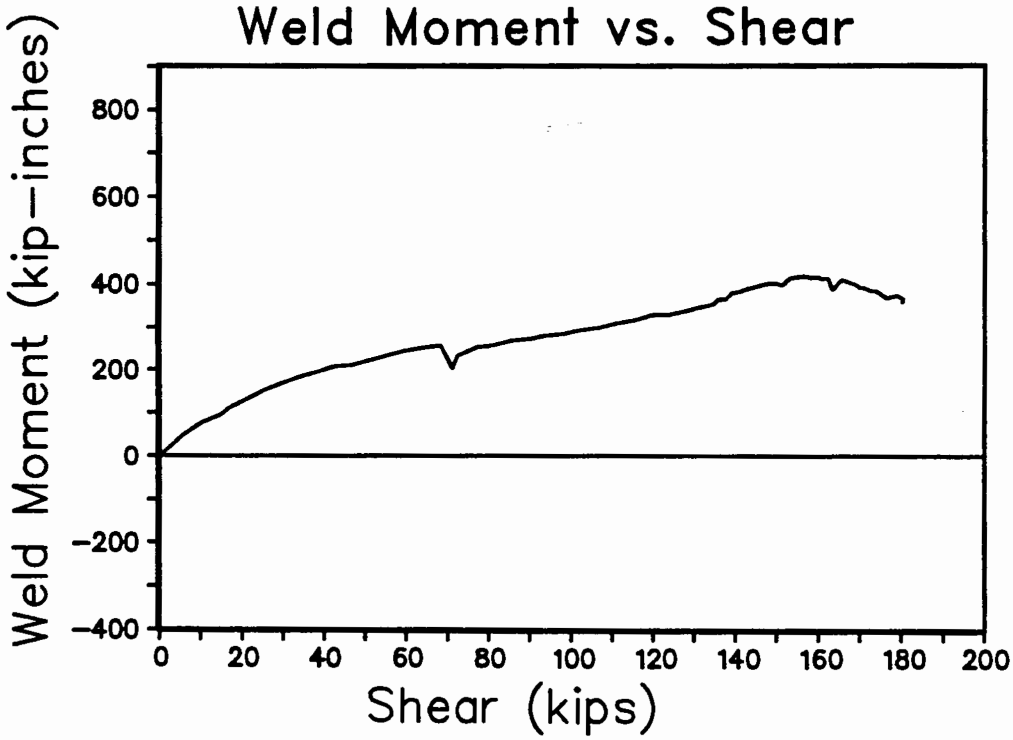
Weld Moment vs. Shear



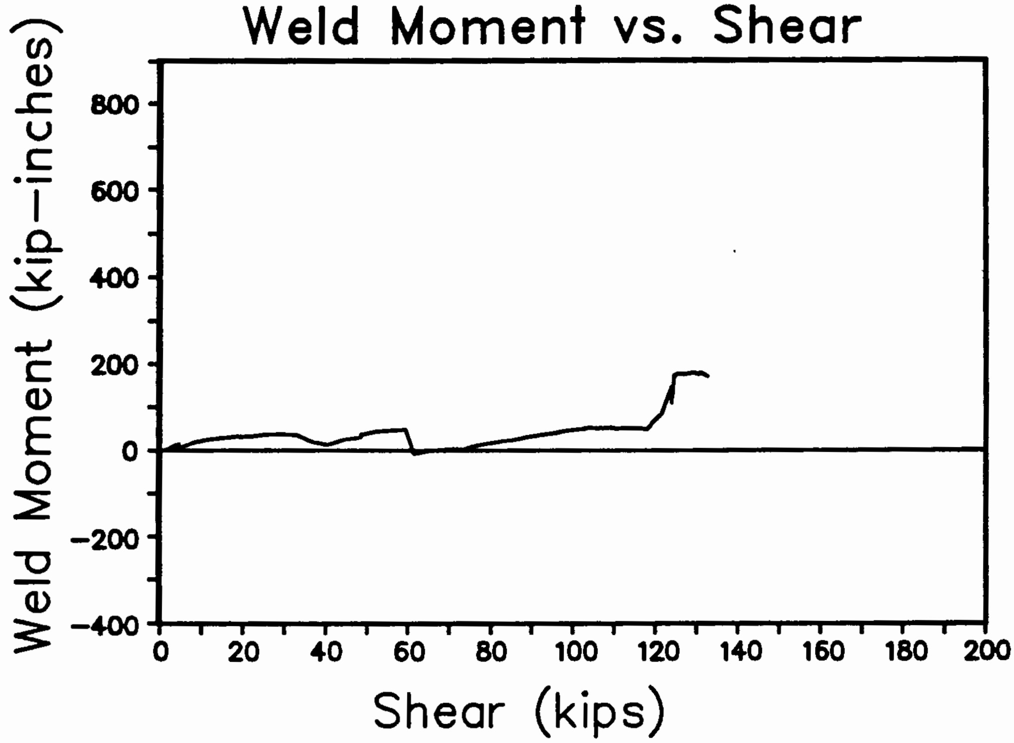
Test #14
Weld Moment vs. Shear



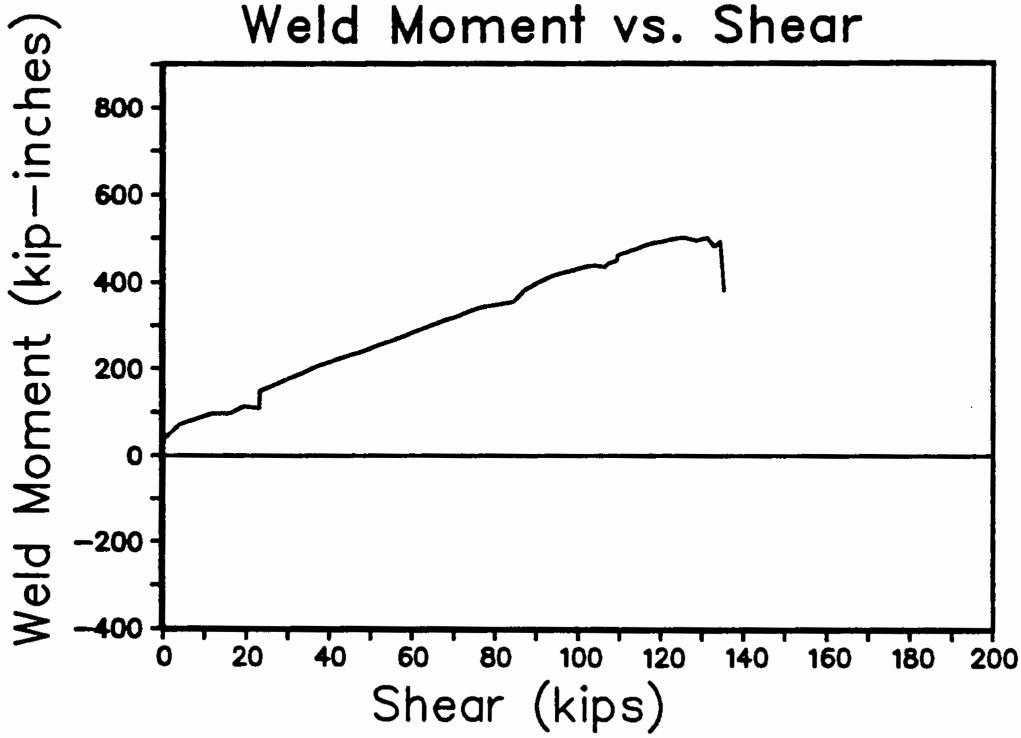
Test #15
Weld Moment vs. Shear

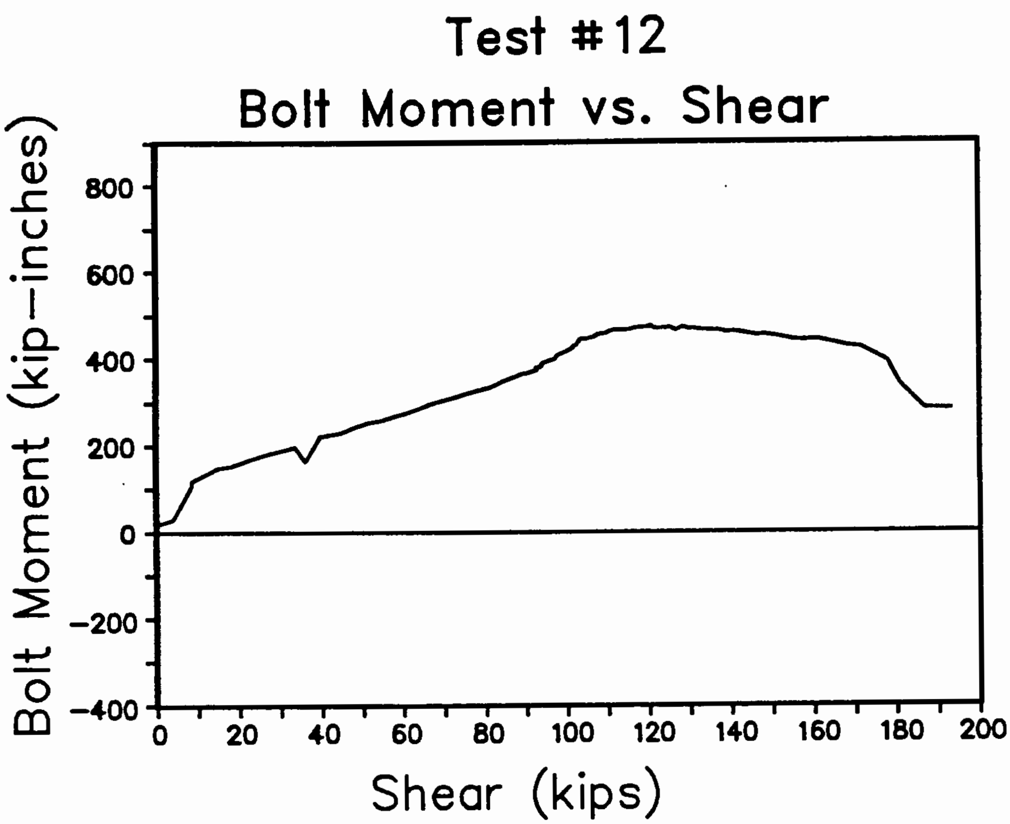


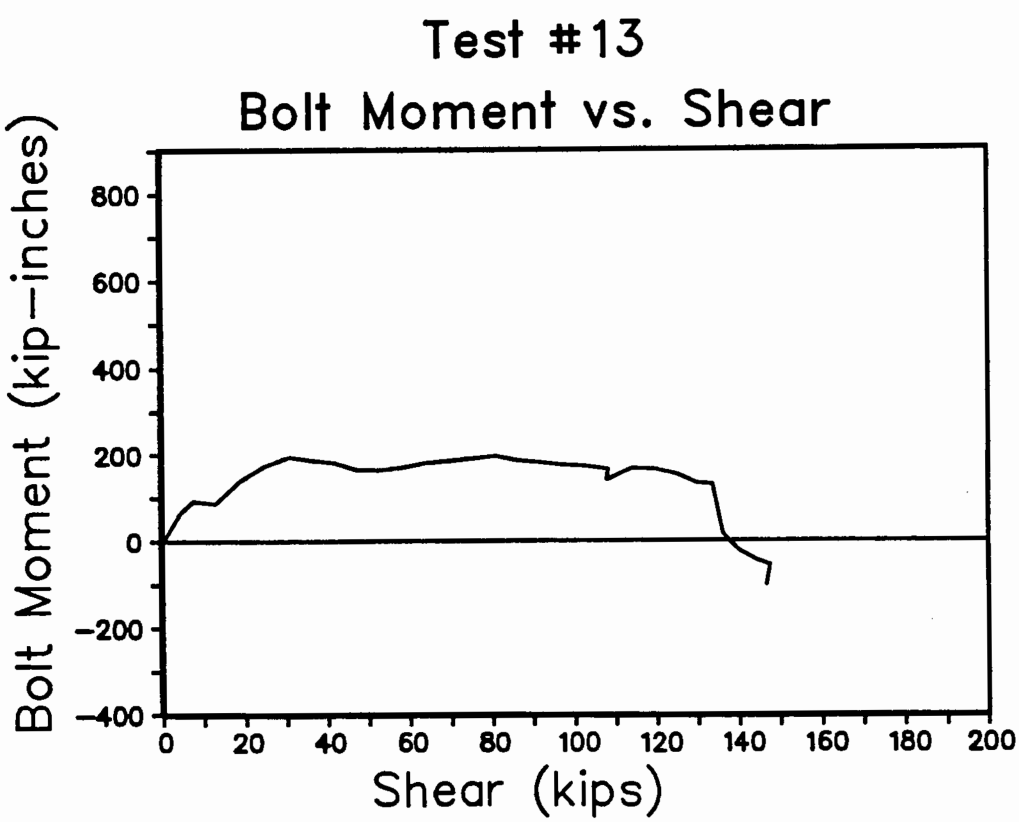
Test #16
Weld Moment vs. Shear

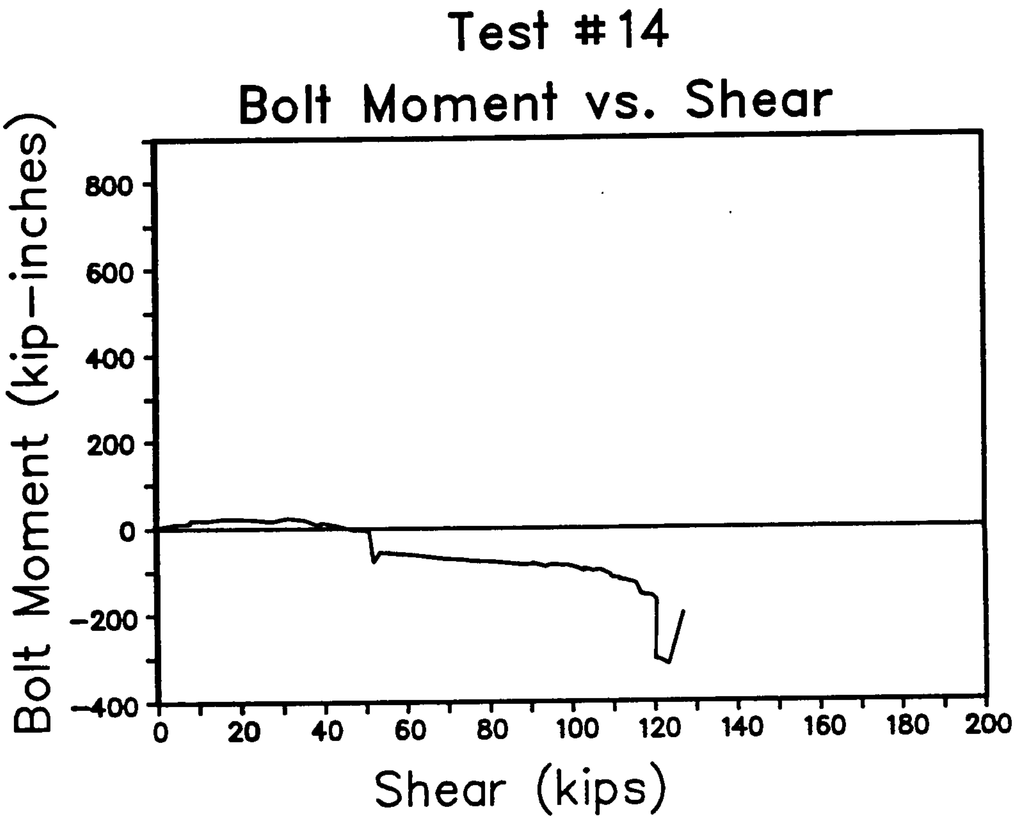


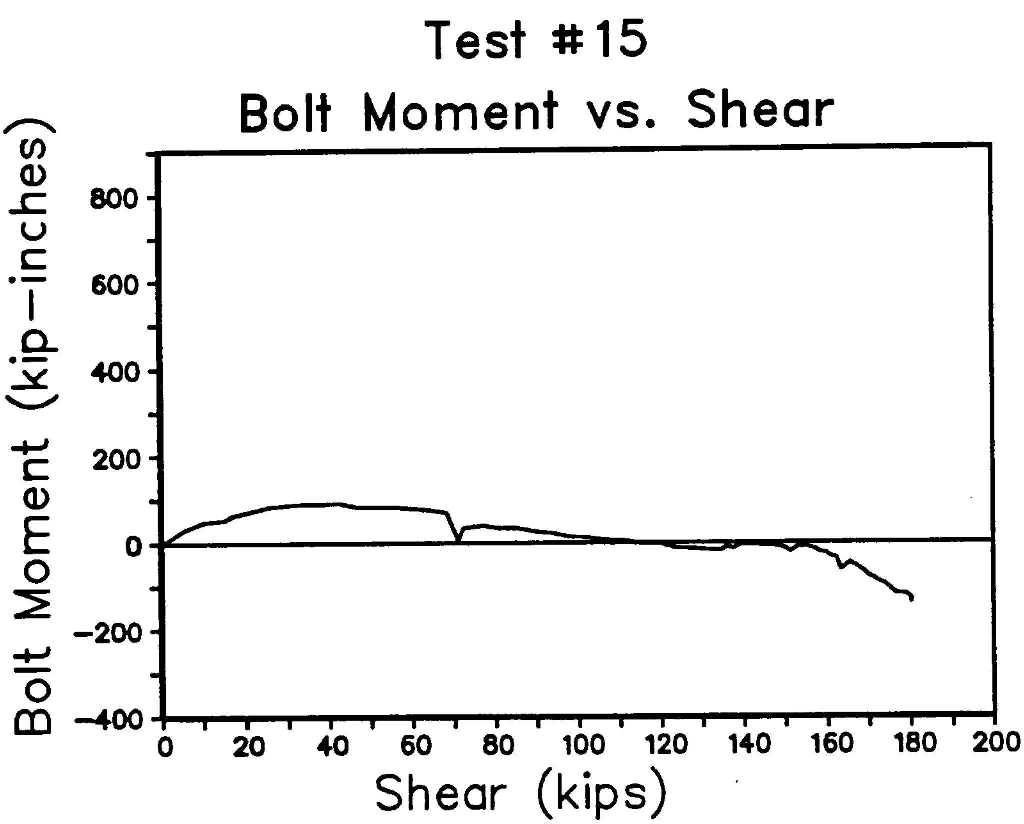
Test #17
Weld Moment vs. Shear



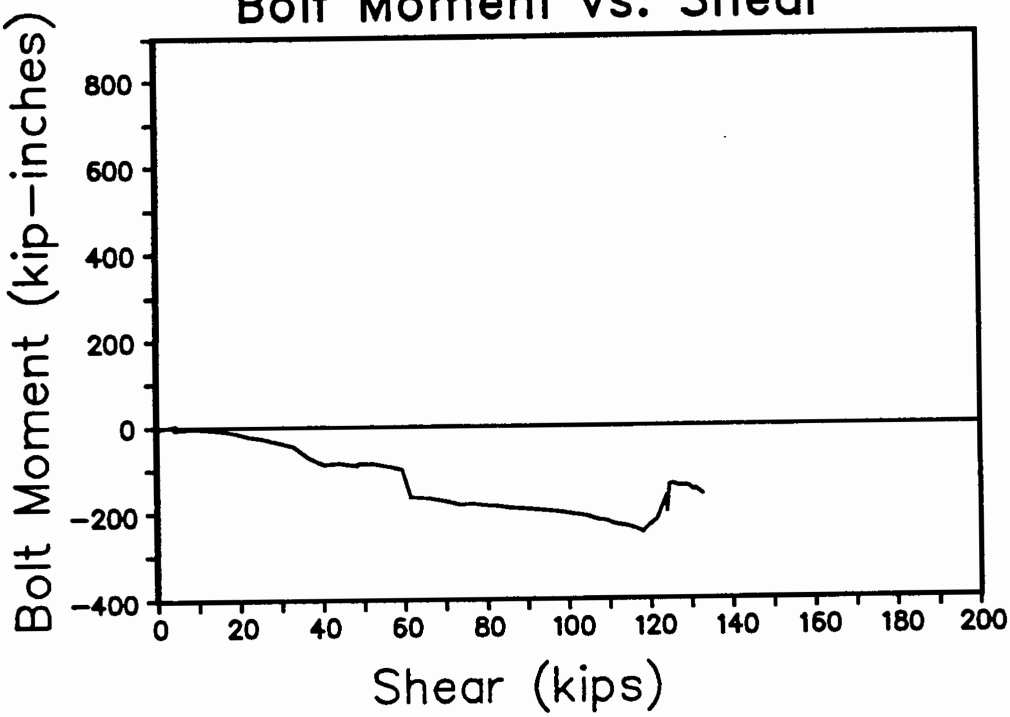

Test #12
Bolt Moment vs. Shear
Bolt Moment (kip-inches)
800
600
400
200
0
-200
-400
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)


Test #13
Bolt Moment vs. Shear
Bolt Moment (kip-inches)
800
600
400
200
0
-200
-400
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)

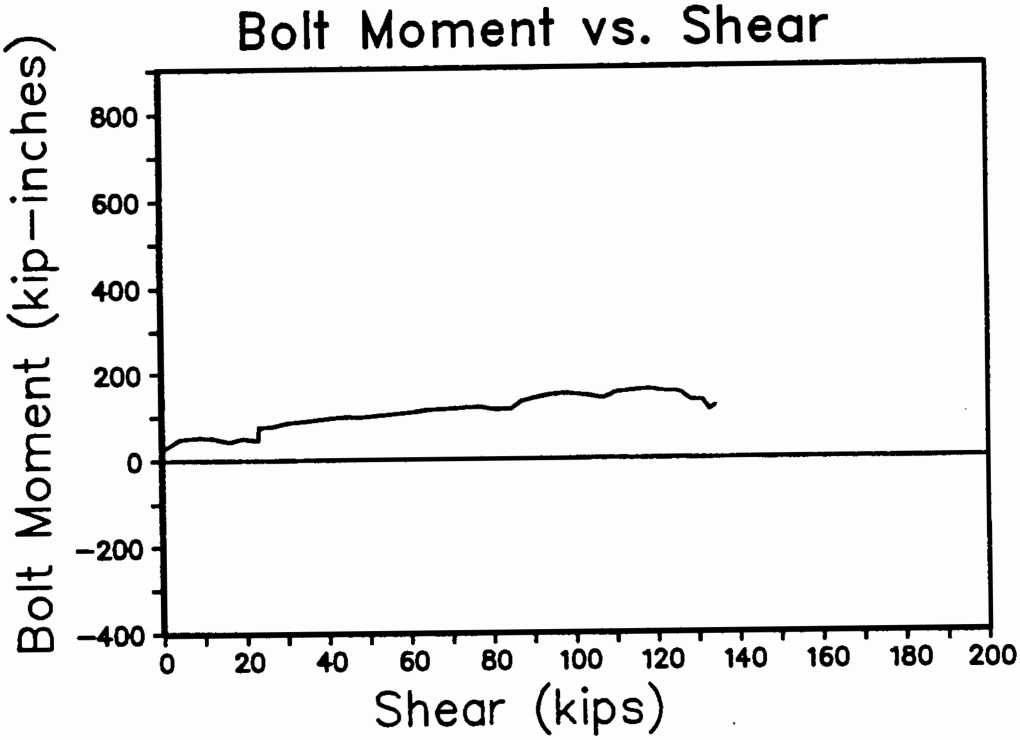

Test #14
Bolt Moment vs. Shear
Bolt Moment (kip-inches)
800
600
400
200
0
-200
-400
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)

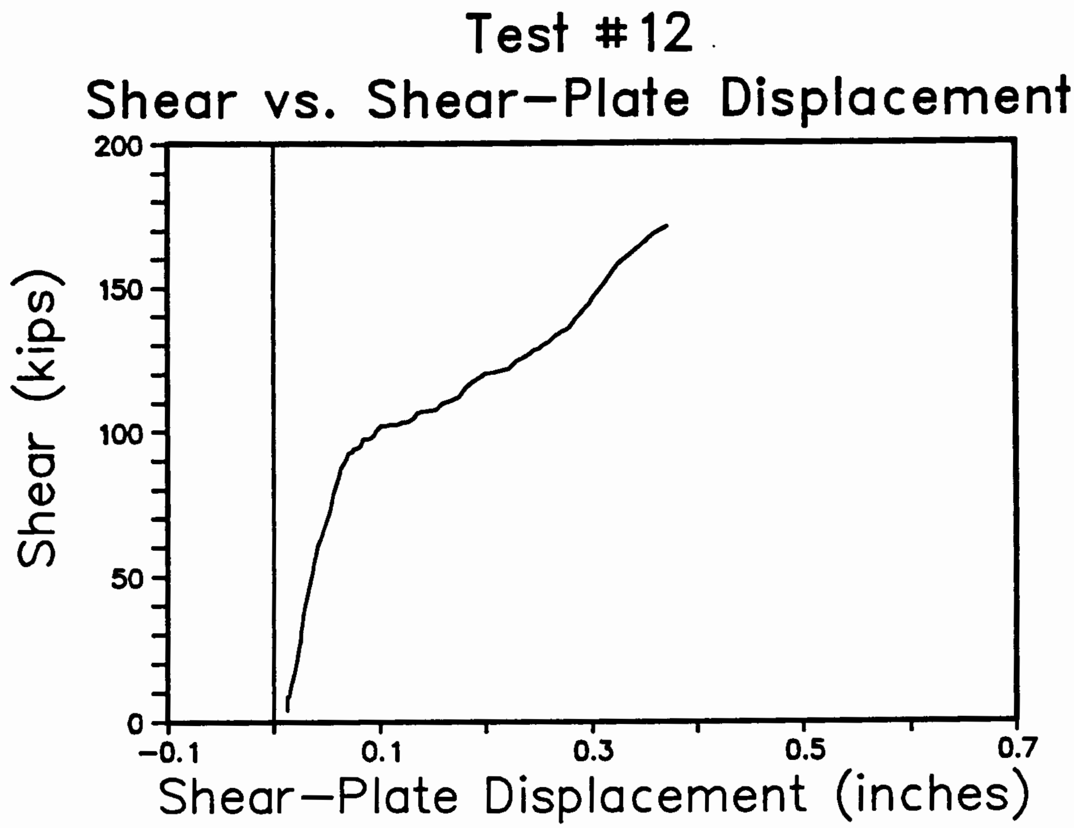

Test #15
Bolt Moment vs. Shear
Bolt Moment (kip-inches)
800
600
400
200
0
-200
-400
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)

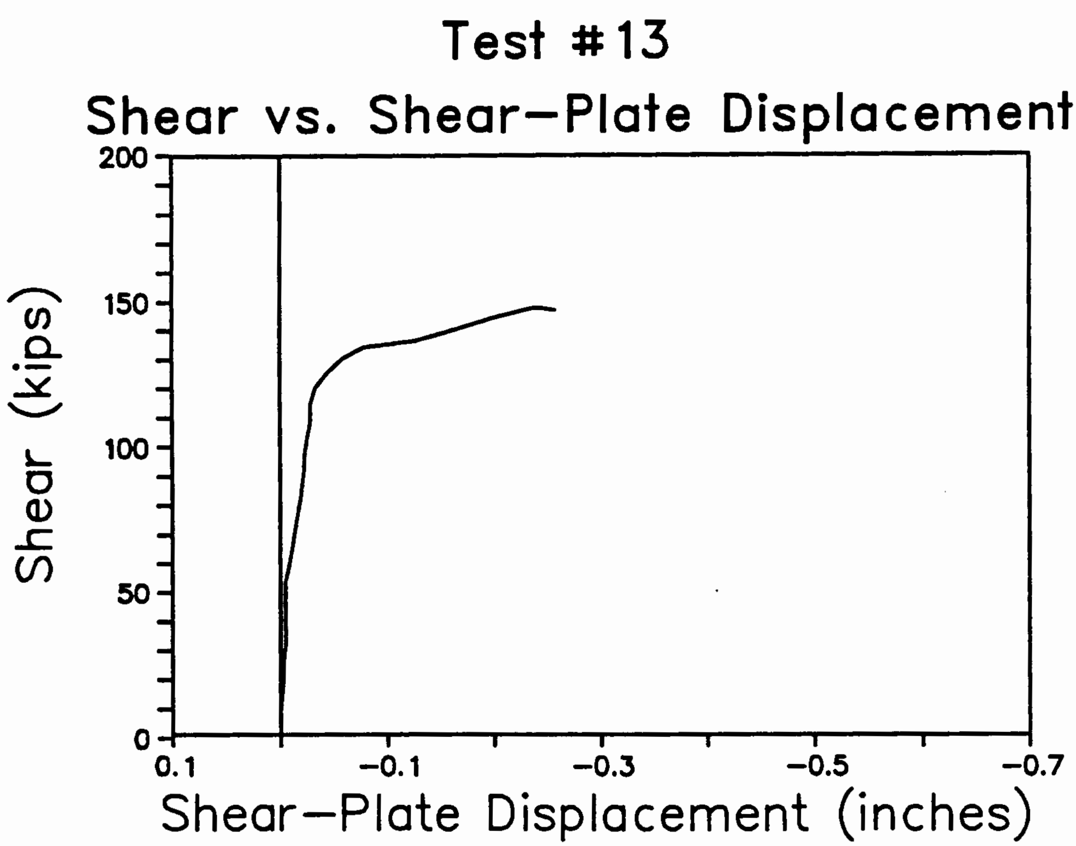
Test #16
Bolt Moment vs. Shear

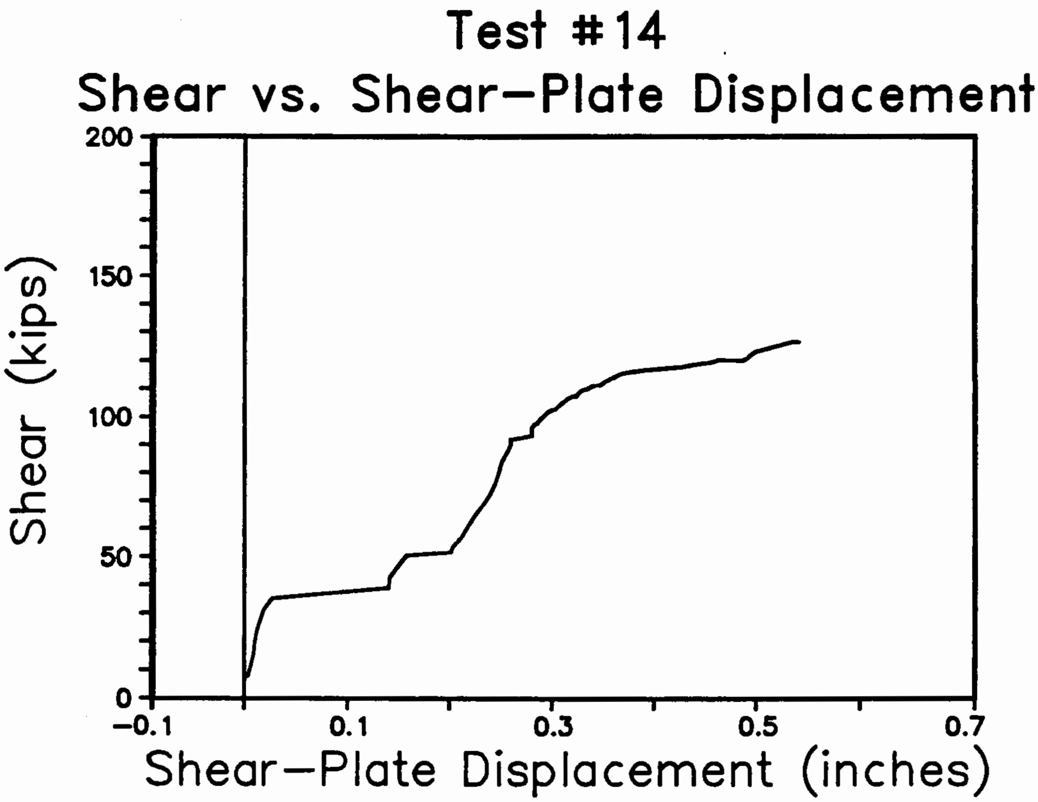


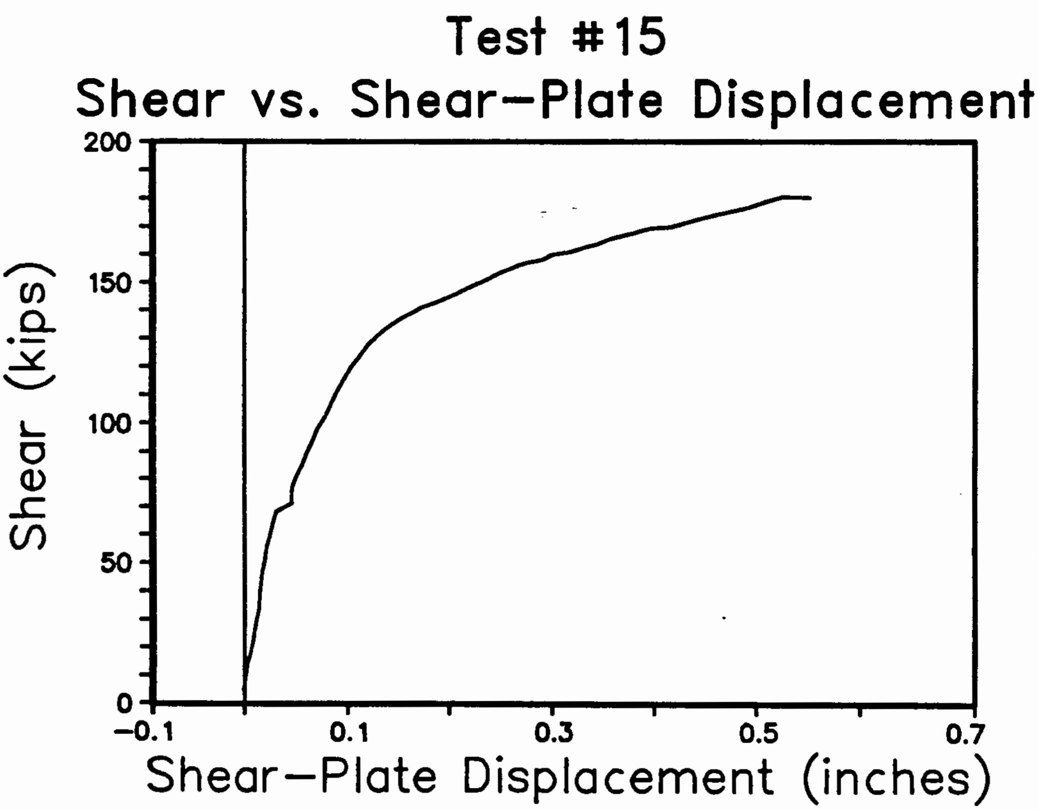
Test #17
Bolt Moment vs. Shear




Test #12
Shear vs. Shear–Plate Displacement
Shear (kips)
200
150
100
50
0
−0.1
0.1
0.3
0.5
0.7
Shear–Plate Displacement (inches)

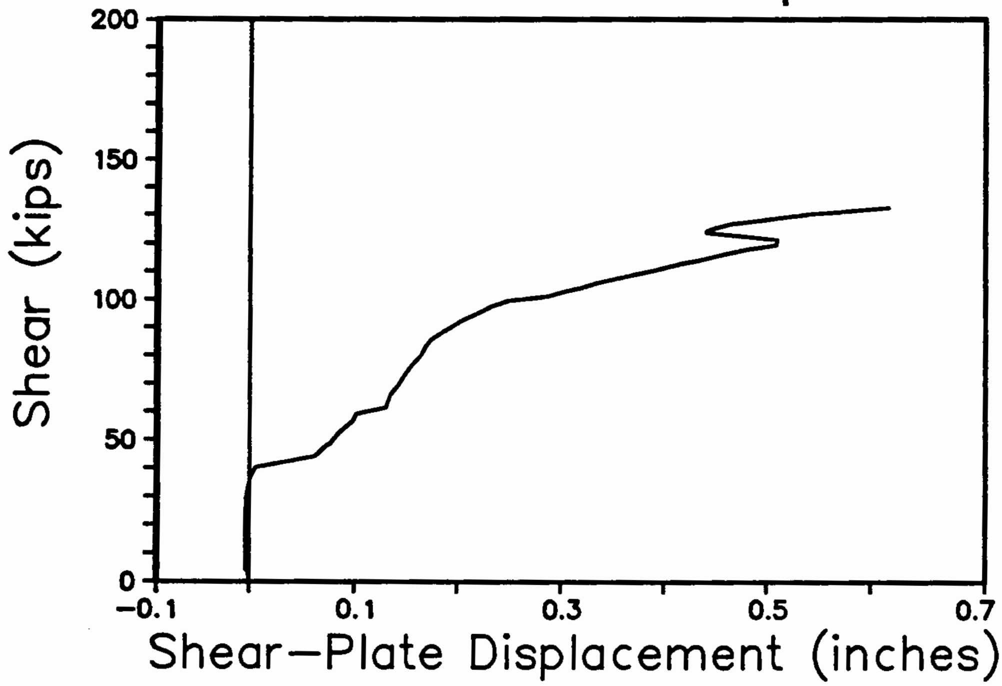

Test #13
Shear vs. Shear–Plate Displacement
Shear (kips)
200
150
100
50
0
0.1
−0.1
−0.3
−0.5
−0.7
Shear–Plate Displacement (inches)


Test #14
Shear vs. Shear-Plate Displacement
200
150
100
50
0
Shear (kips)
-0.1
0.1
0.3
0.5
0.7
Shear-Plate Displacement (inches)


Test #15
Shear vs. Shear-Plate Displacement
200
150
100
50
0
Shear (kips)
-0.1
0.1
0.3
0.5
0.7
Shear-Plate Displacement (inches)

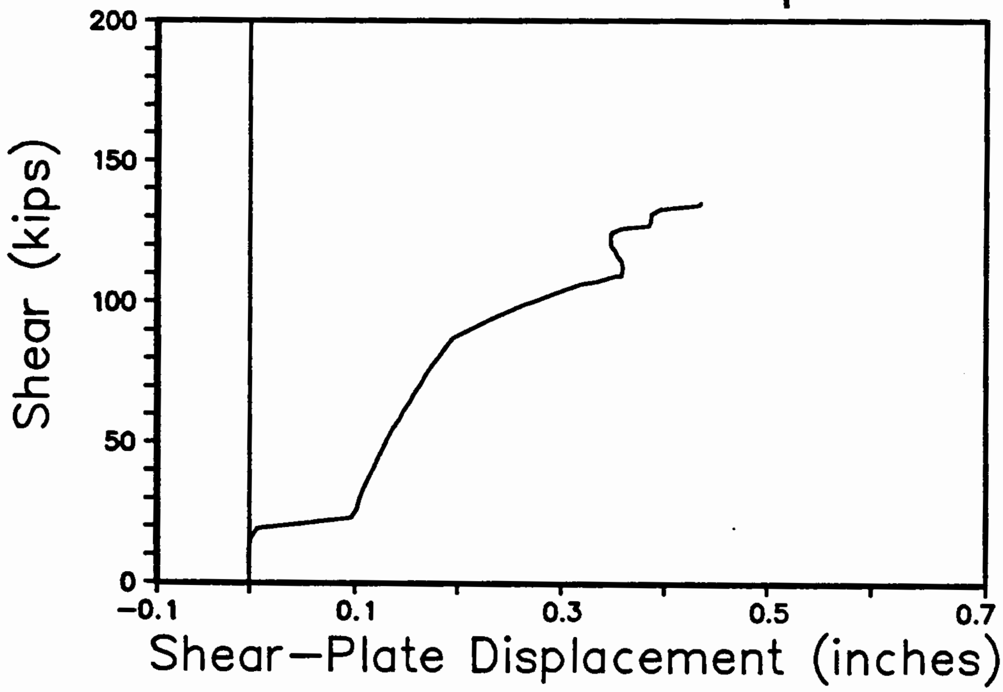
Test #16

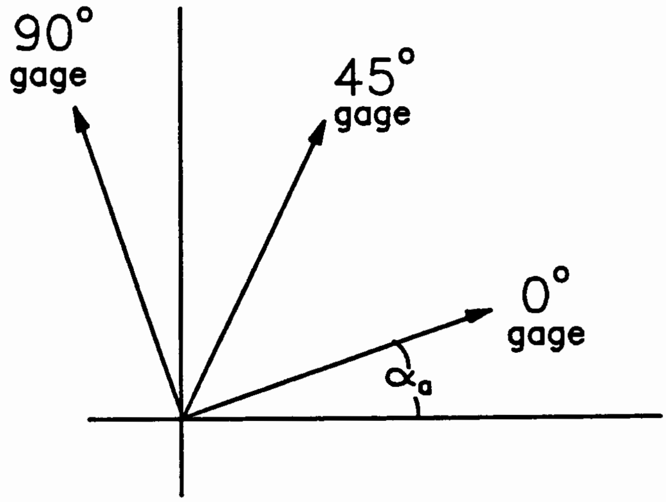
Shear vs. Shear-Plate Displacement



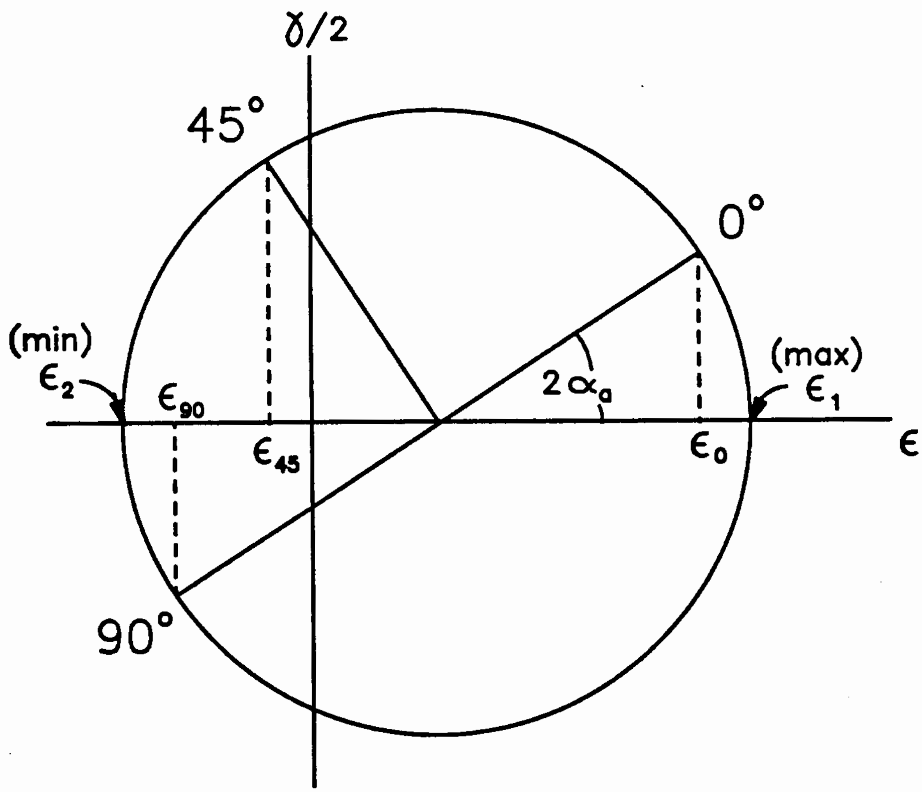
Test #17

Shear vs. Shear-Plate Displacement

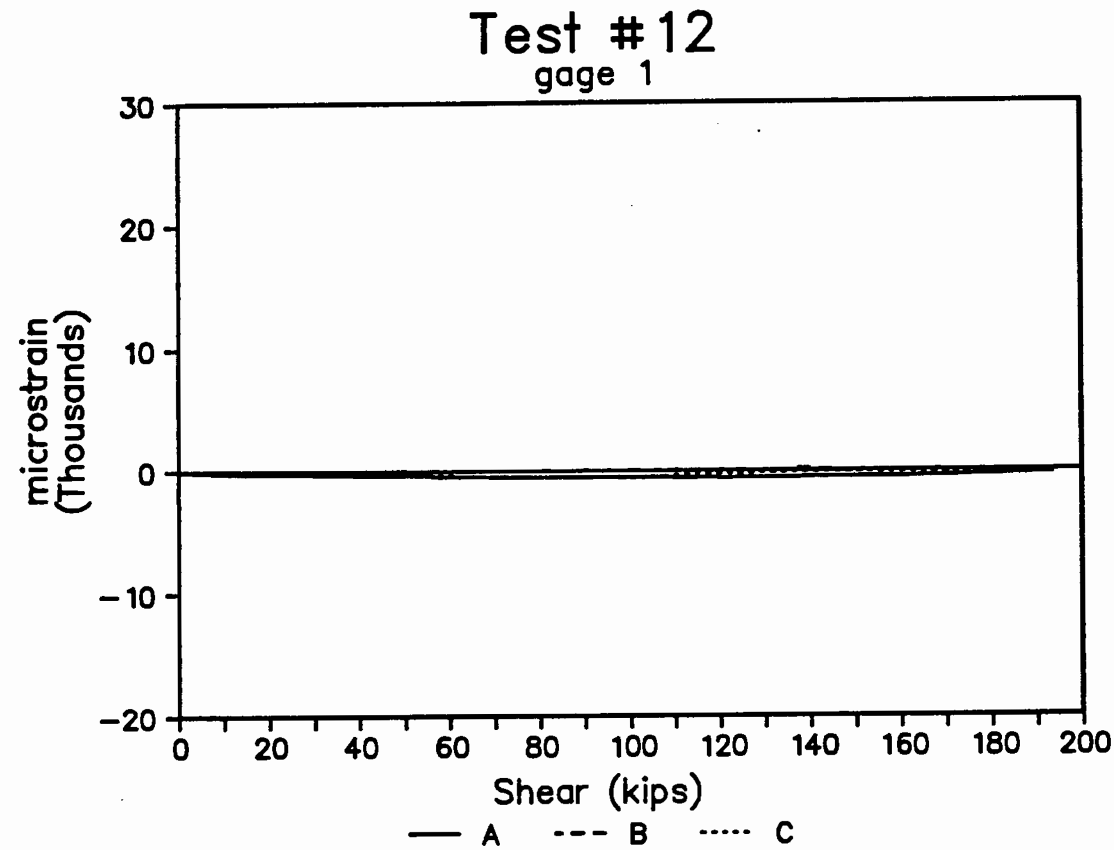


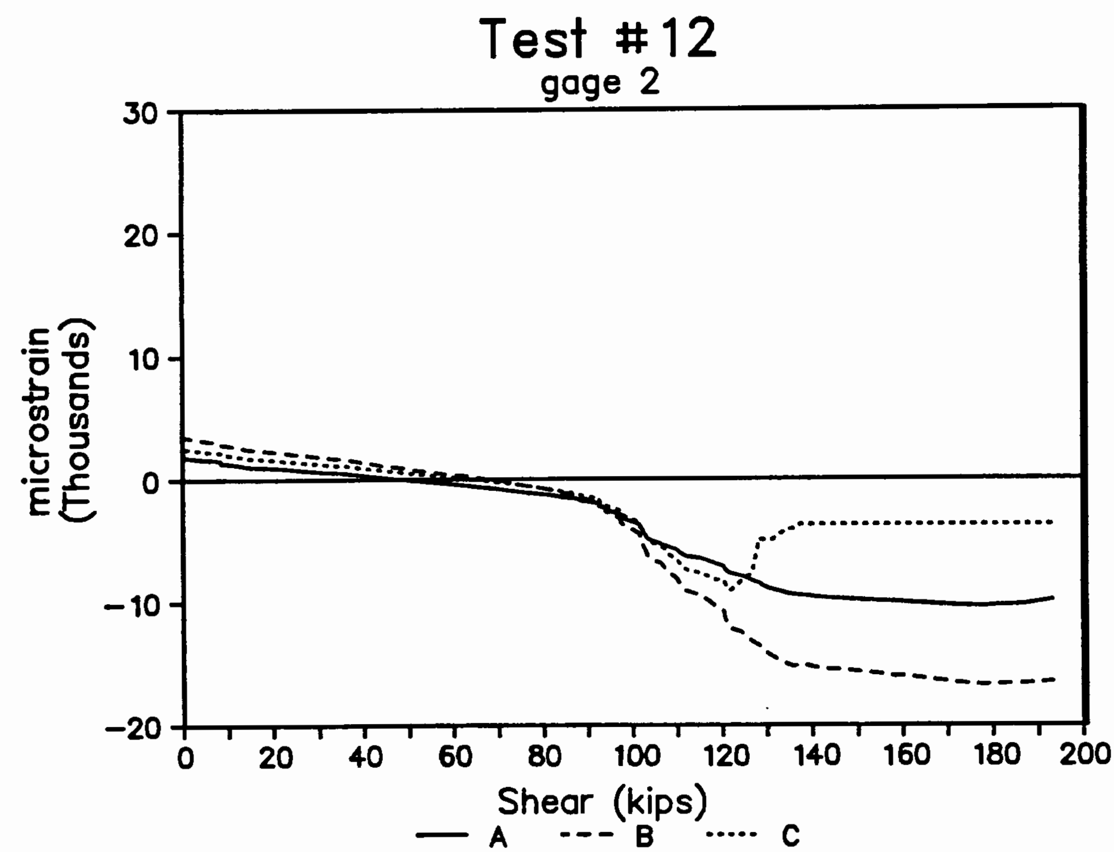


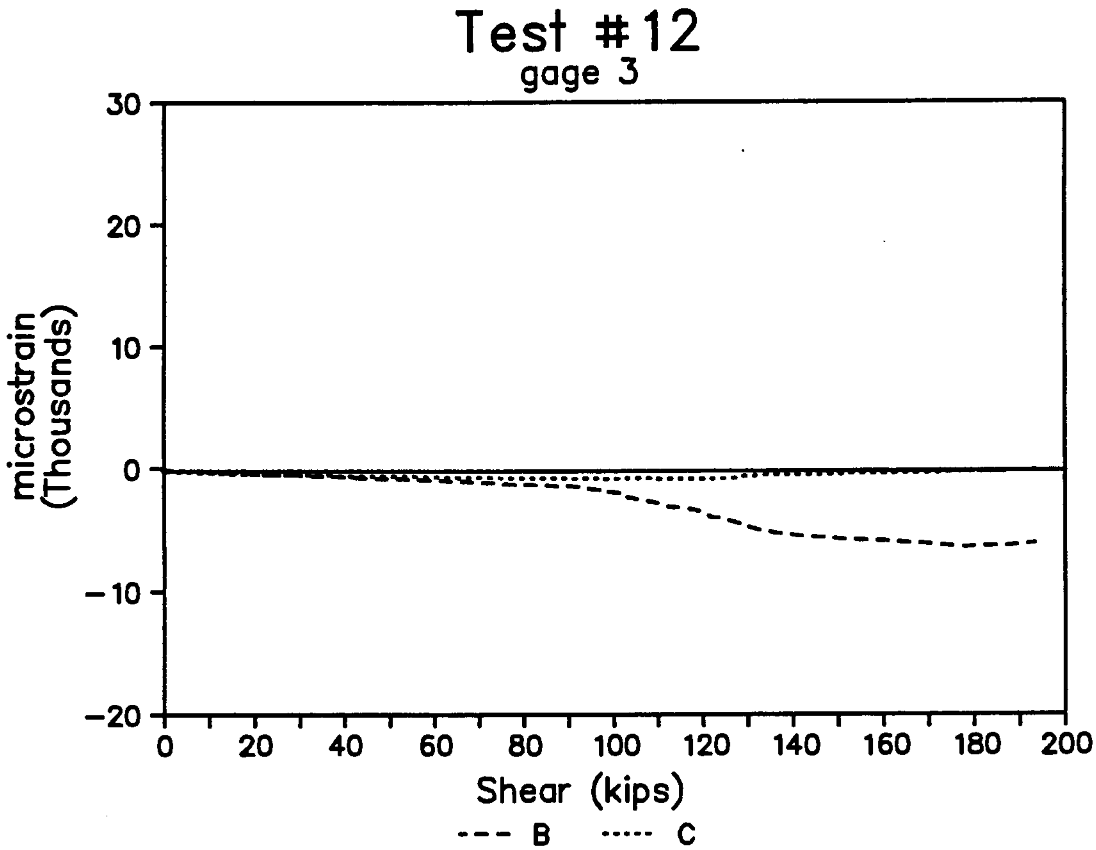
Rosette Strain Gage



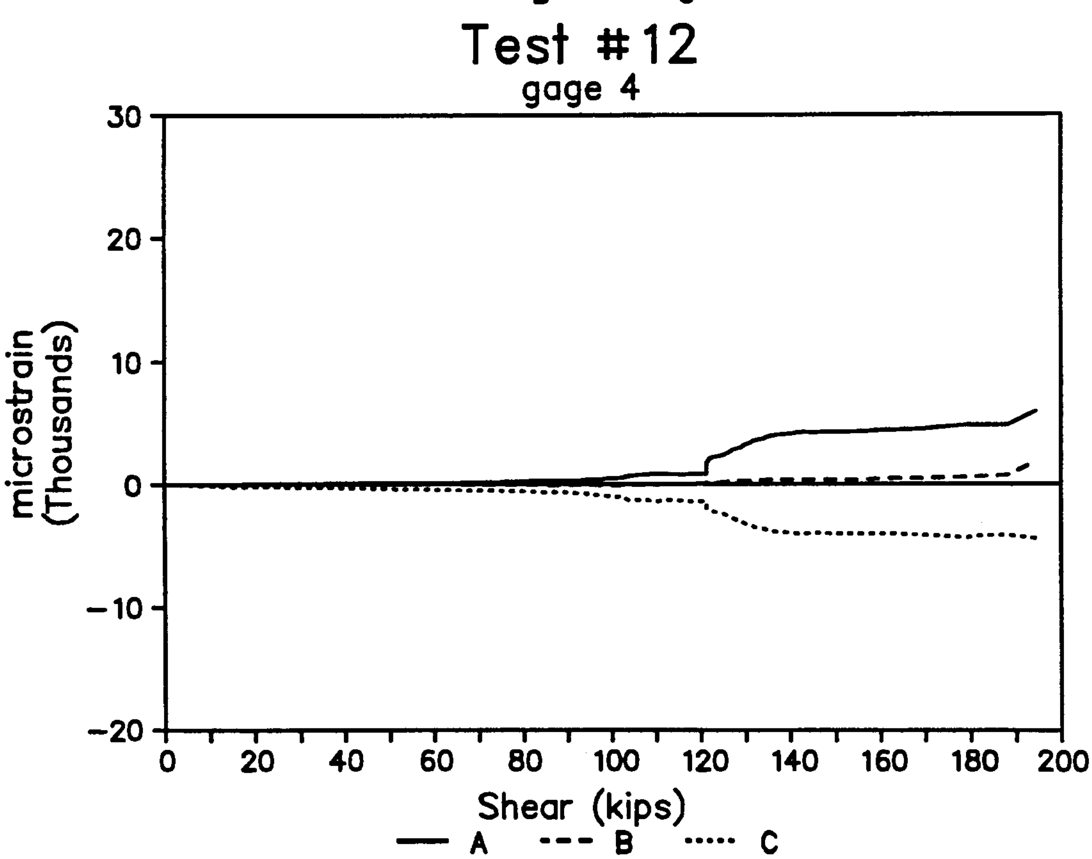
Corresponding Strains


Test #12
gage 1
30
20
10
0
−10
−20
microstrain
(Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
A
B
C


Test #12
gage 2
30
20
10
0
−10
−20
microstrain
(Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
A
B
C

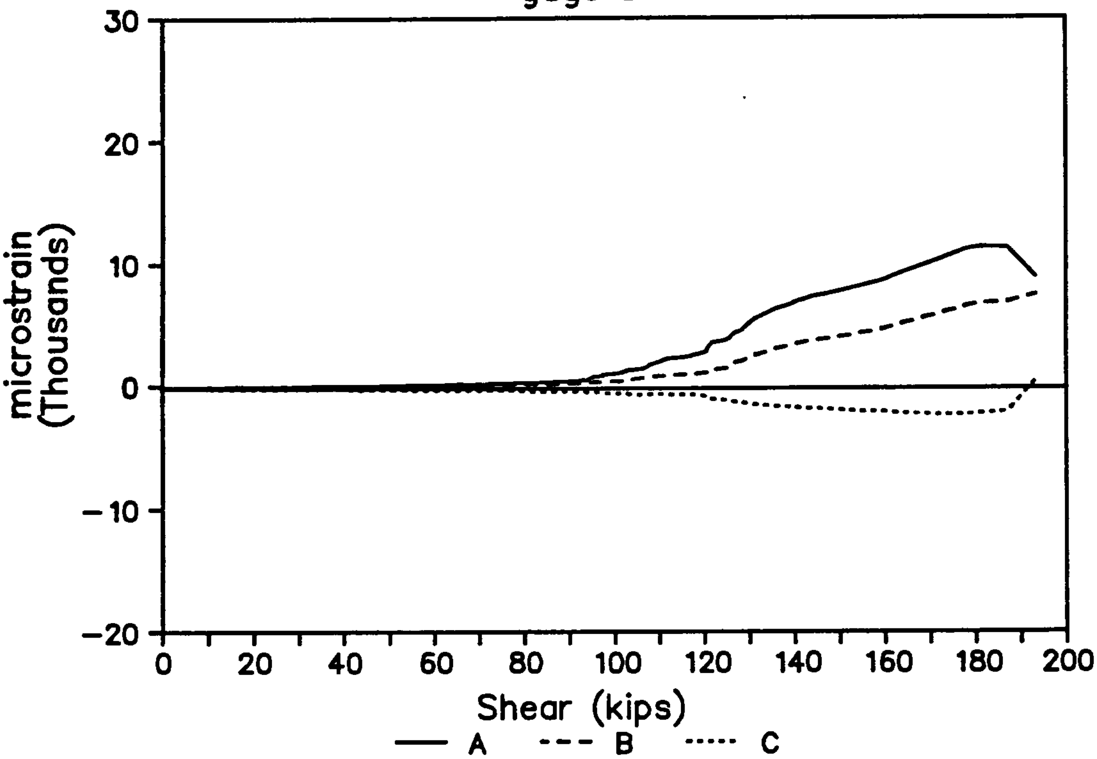


Test #12
gage 3
30
20
10
0
−10
−20
microstrain (Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
B
C

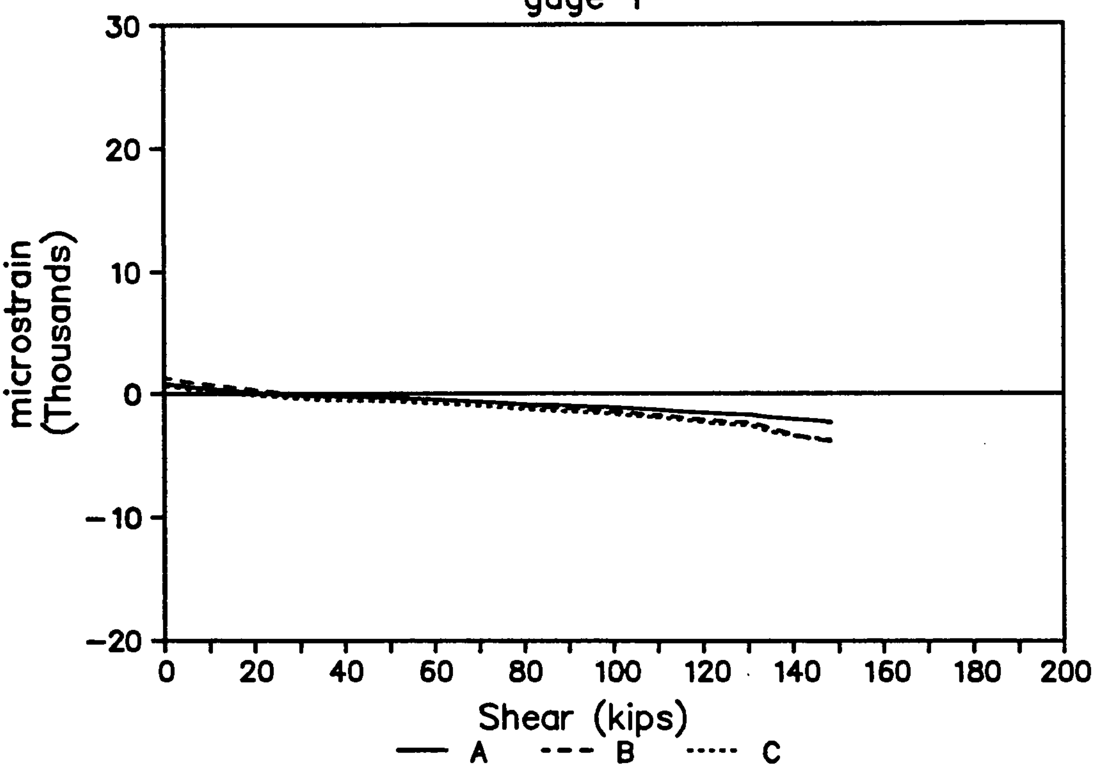


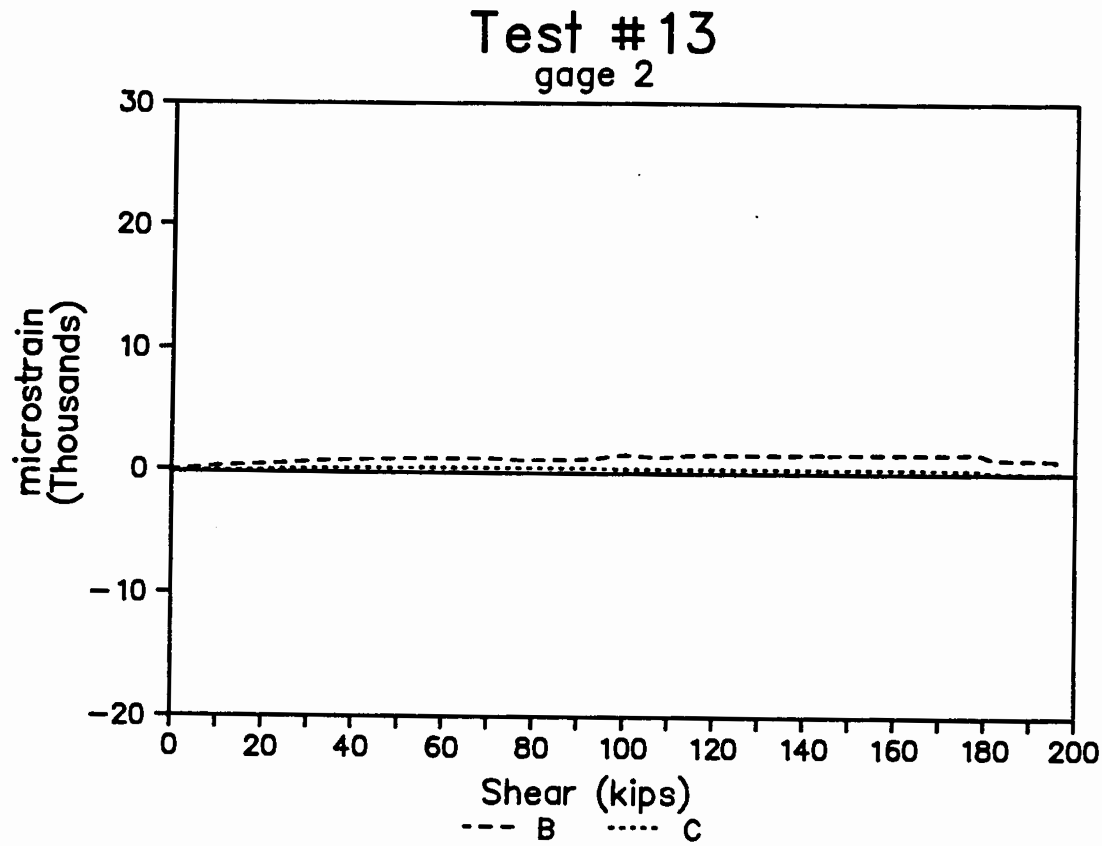
Test #12
gage 4
30
20
10
0
−10
−20
microstrain (Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
A
B
C

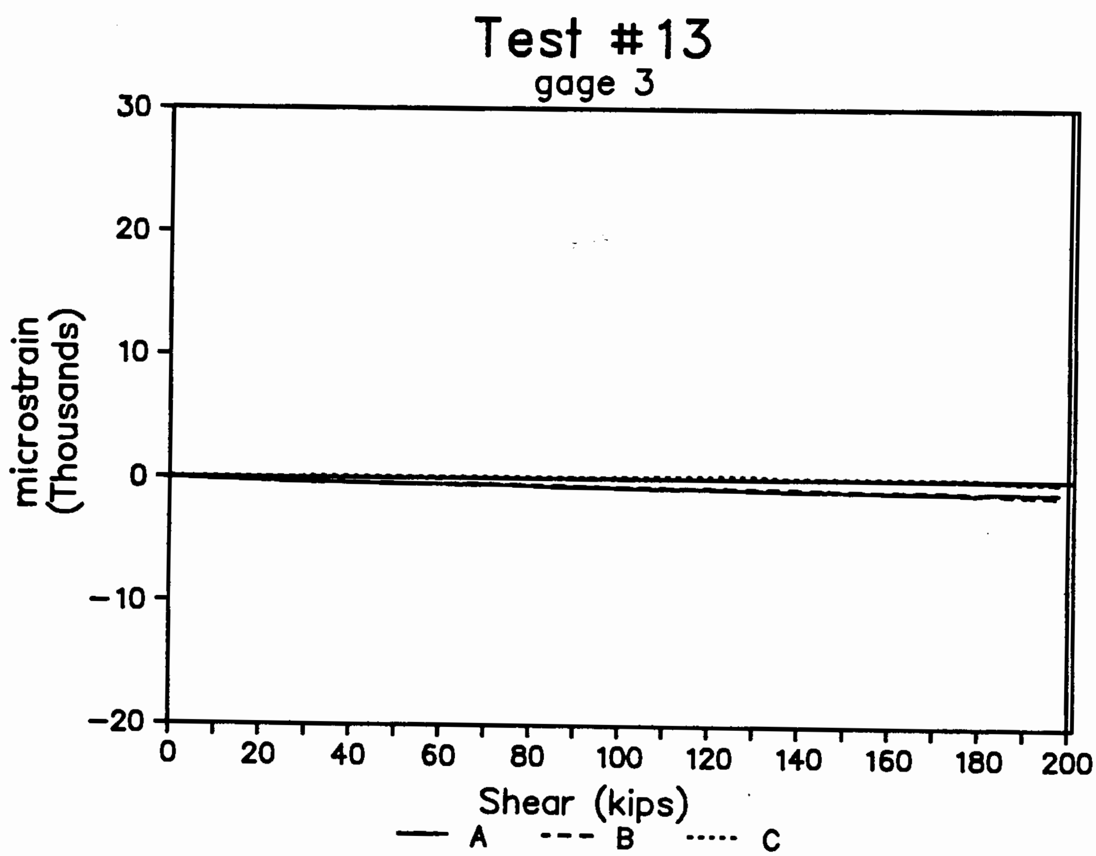
Test #12
gage 5

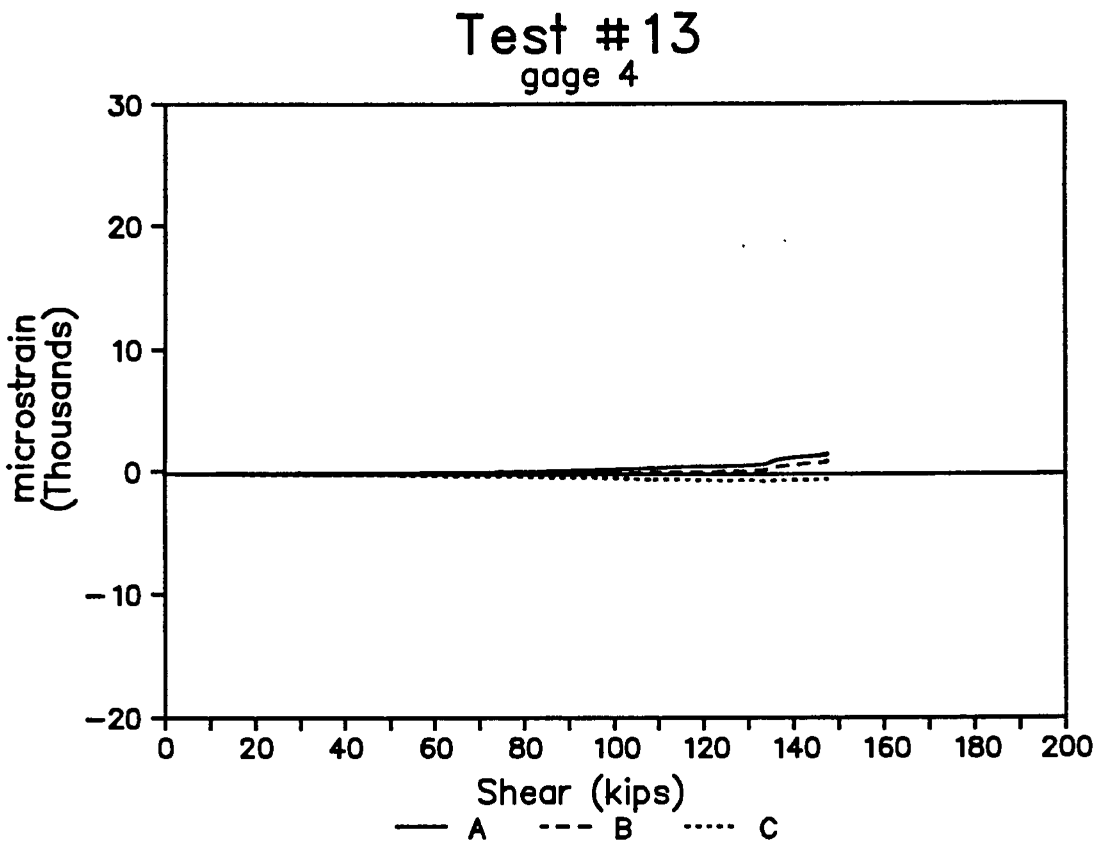


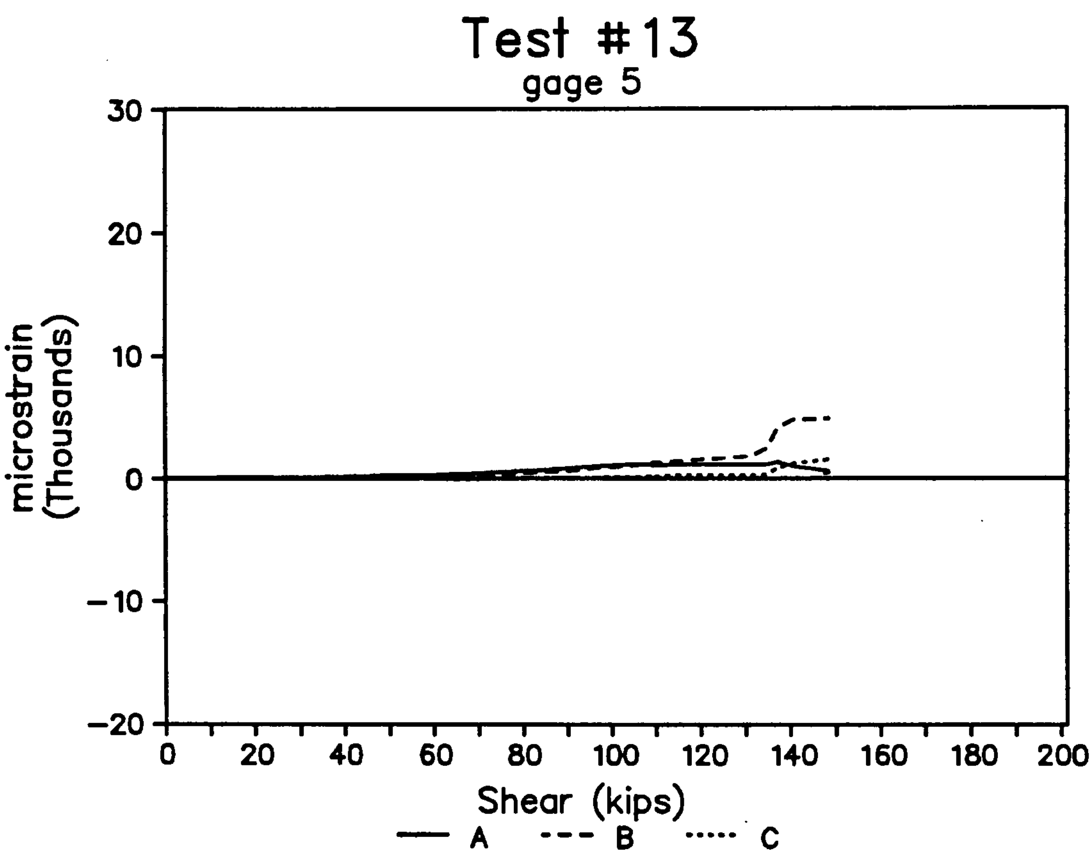
Test #13
gage 1



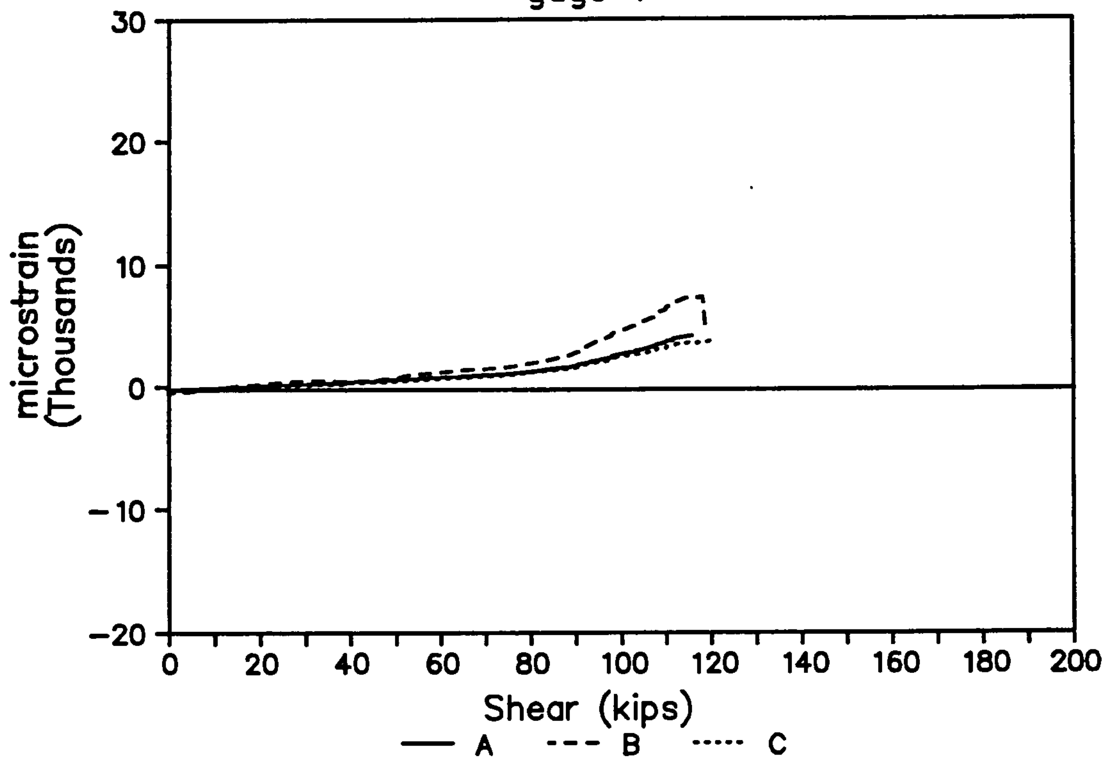

Test #13
gage 2
30
20
10
0
-10
-20
microstrain
(Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
B
C


Test #13
gage 3
30
20
10
0
-10
-20
microstrain
(Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
A
B
C

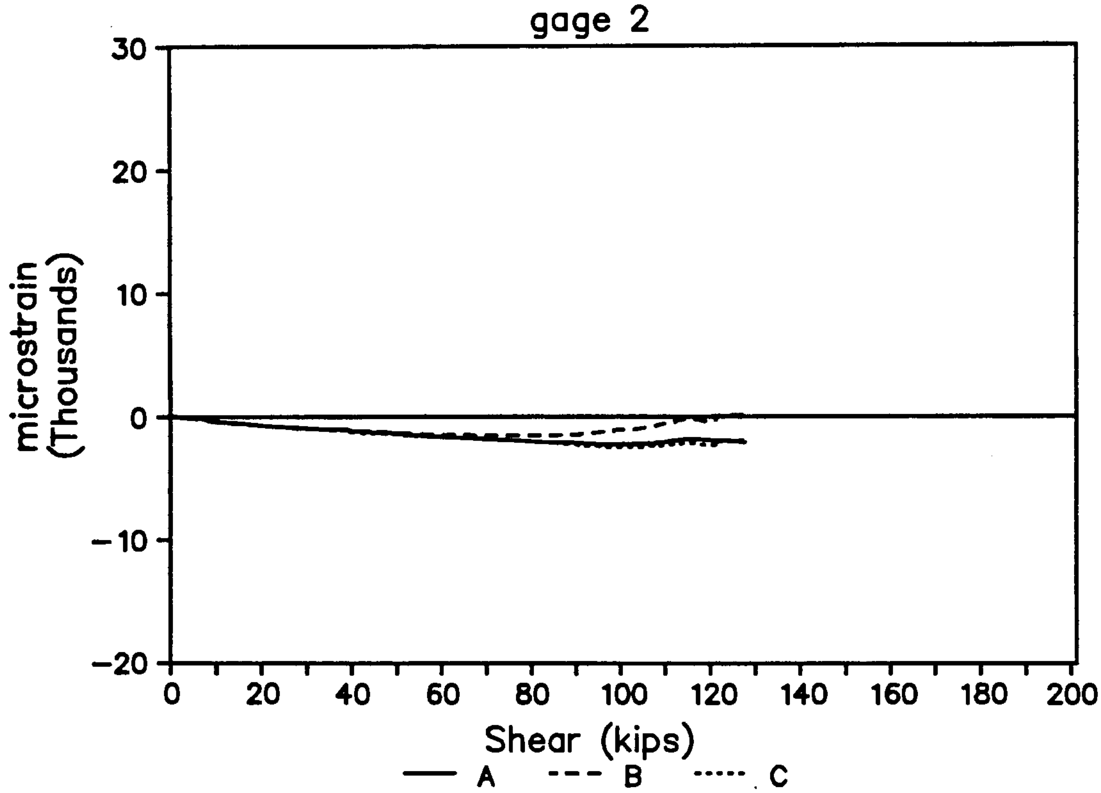

Test #13
gage 4
30
20
10
0
-10
-20
microstrain (Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
A
B
C

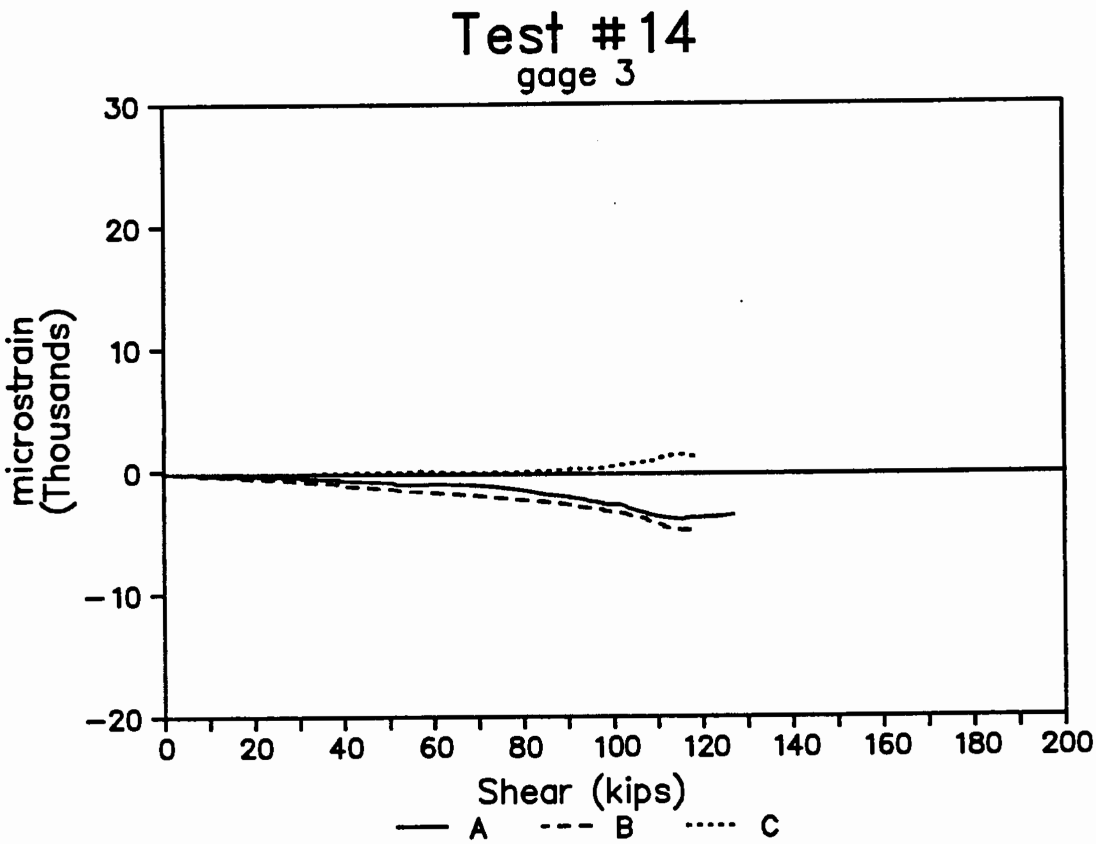

Test #13
gage 5
30
20
10
0
-10
-20
microstrain (Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
A
B
C

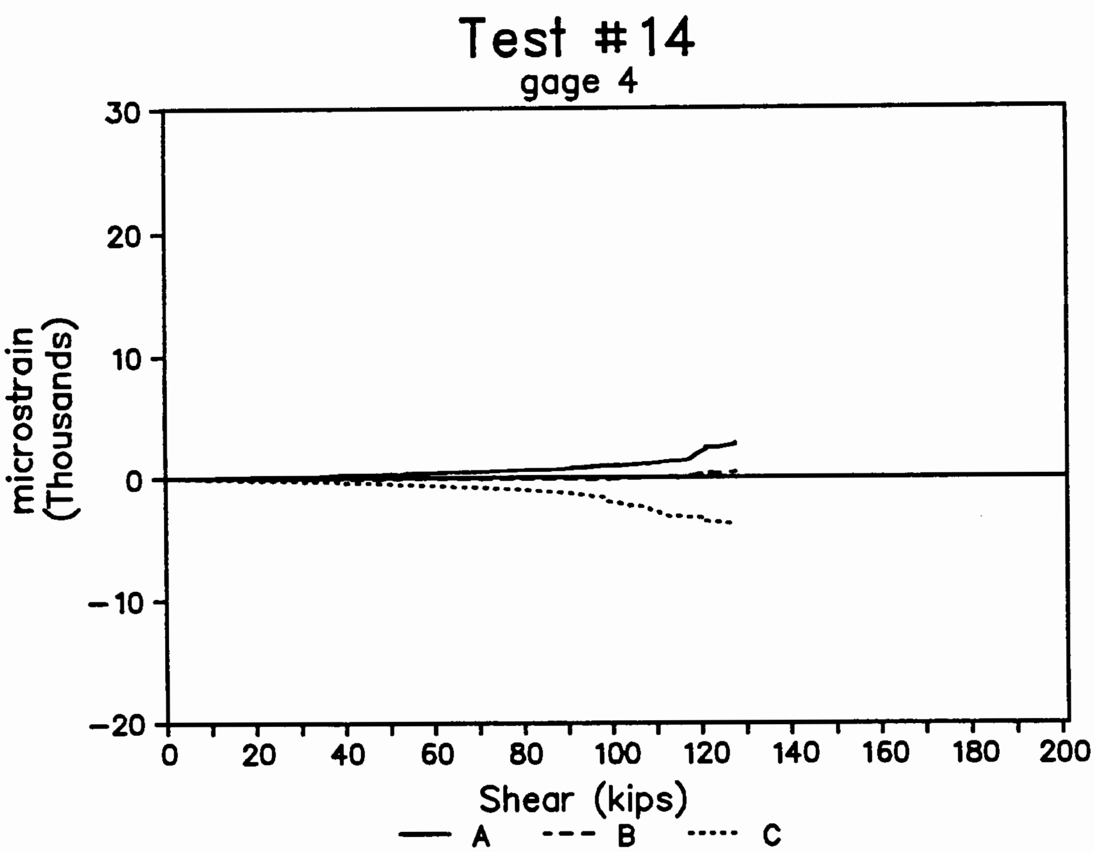
Test #14
gage 1

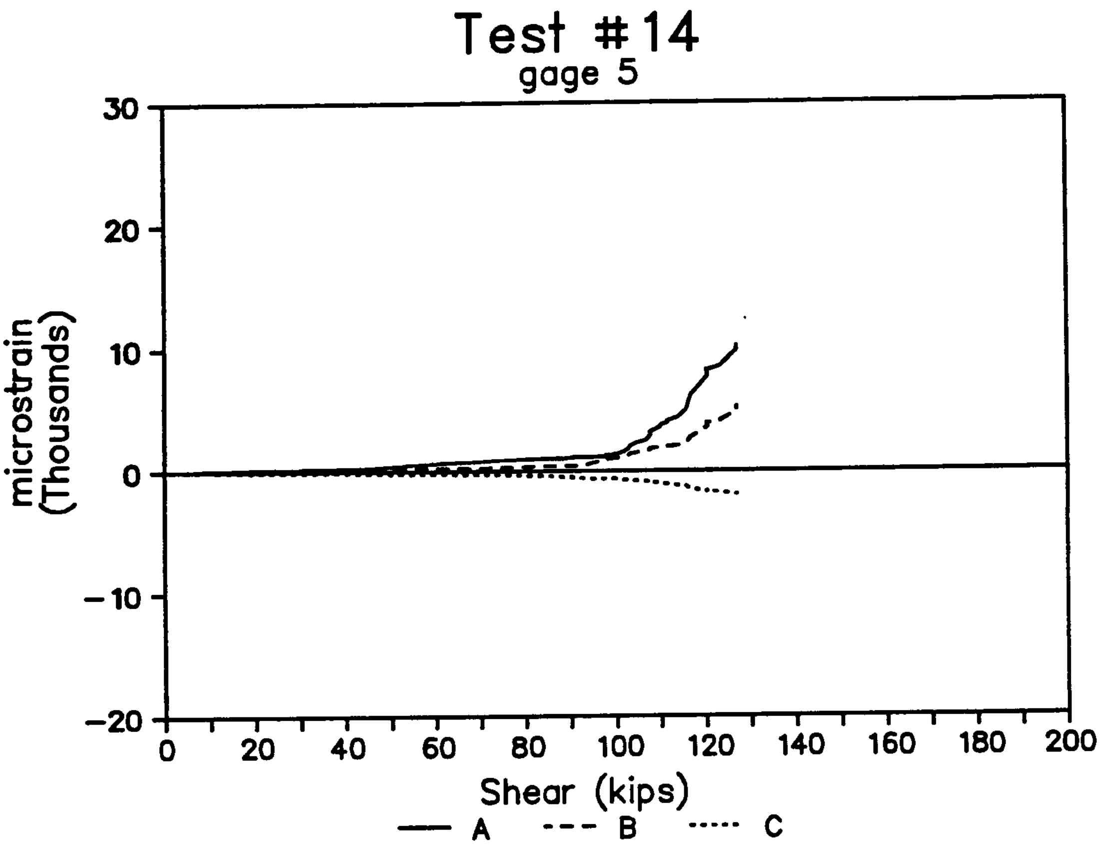


Test #14
gage 2

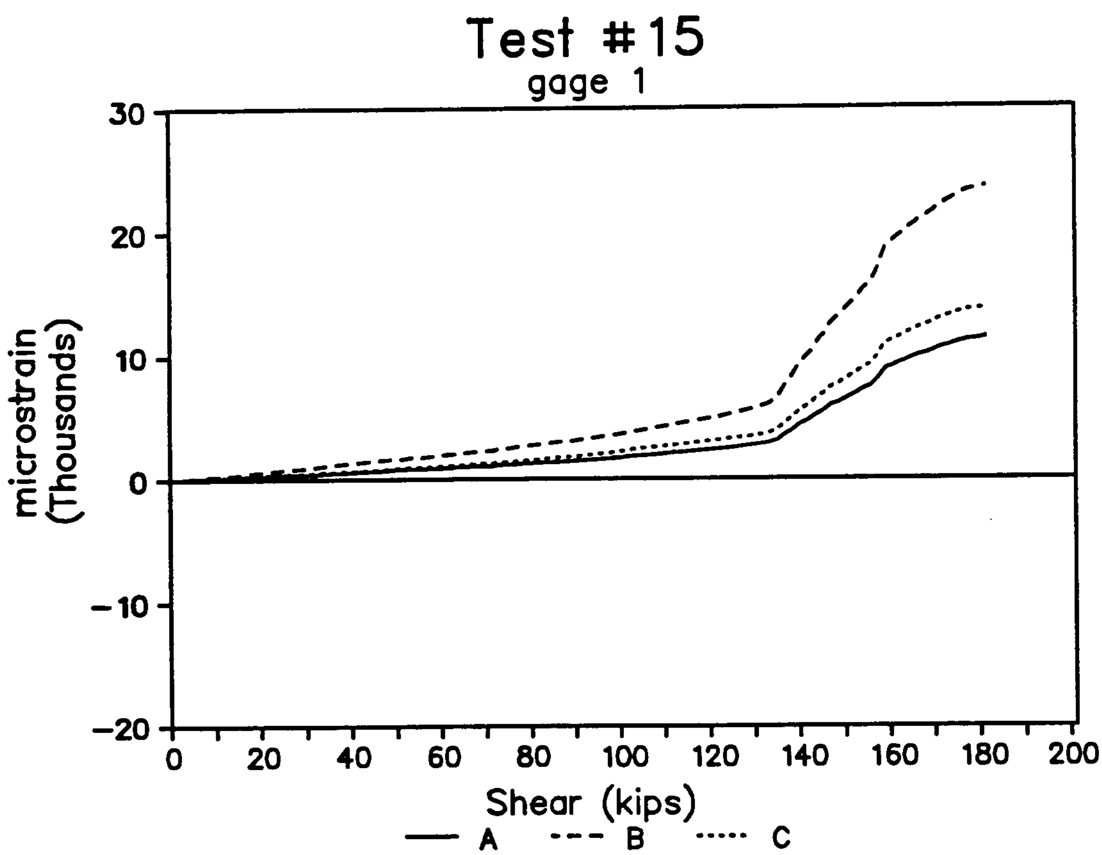



Test #14
gage 3
microstrain (Thousands)
30
20
10
0
−10
−20
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
A
B
C

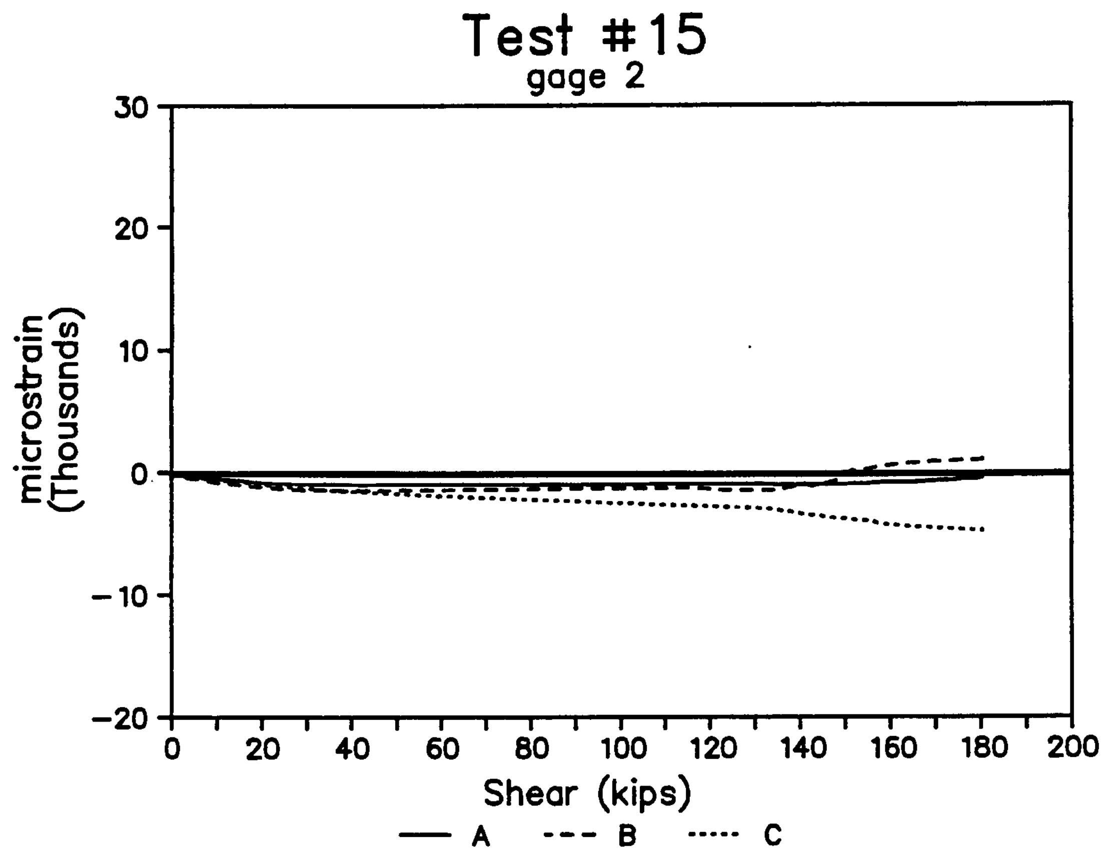

Test #14
gage 4
microstrain (Thousands)
30
20
10
0
−10
−20
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
A
B
C

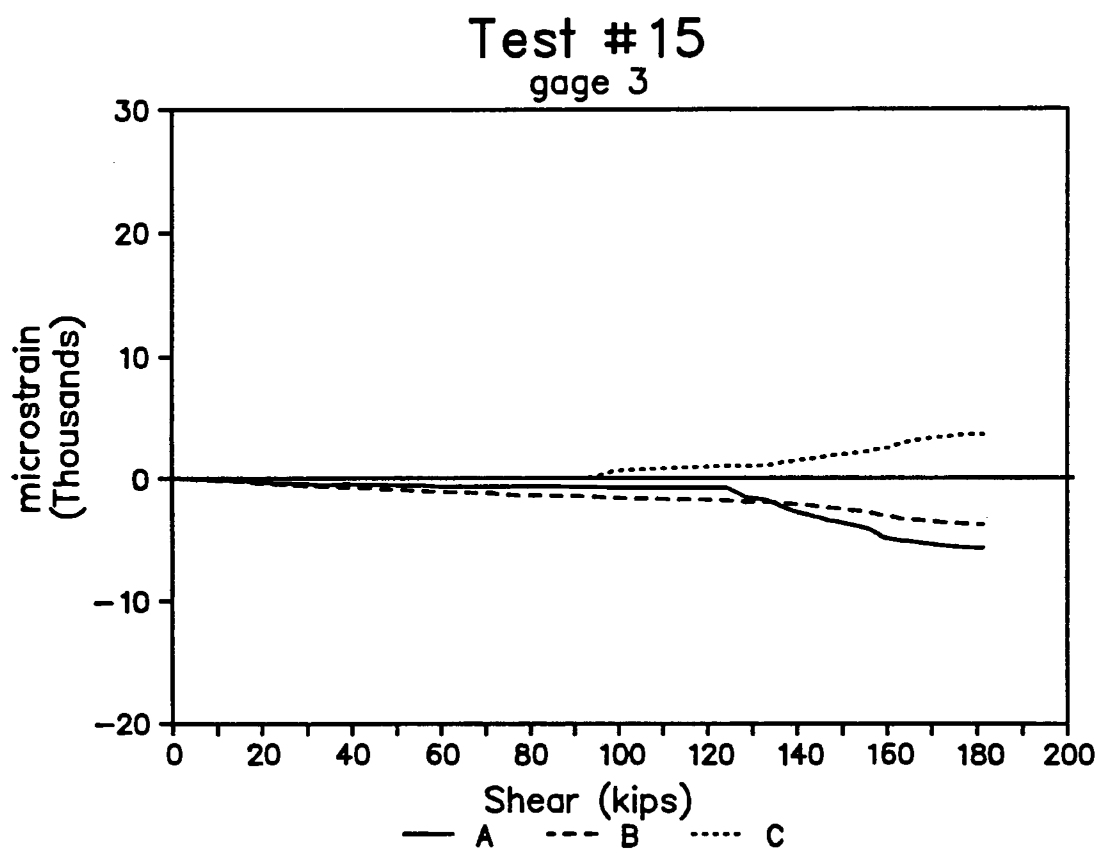


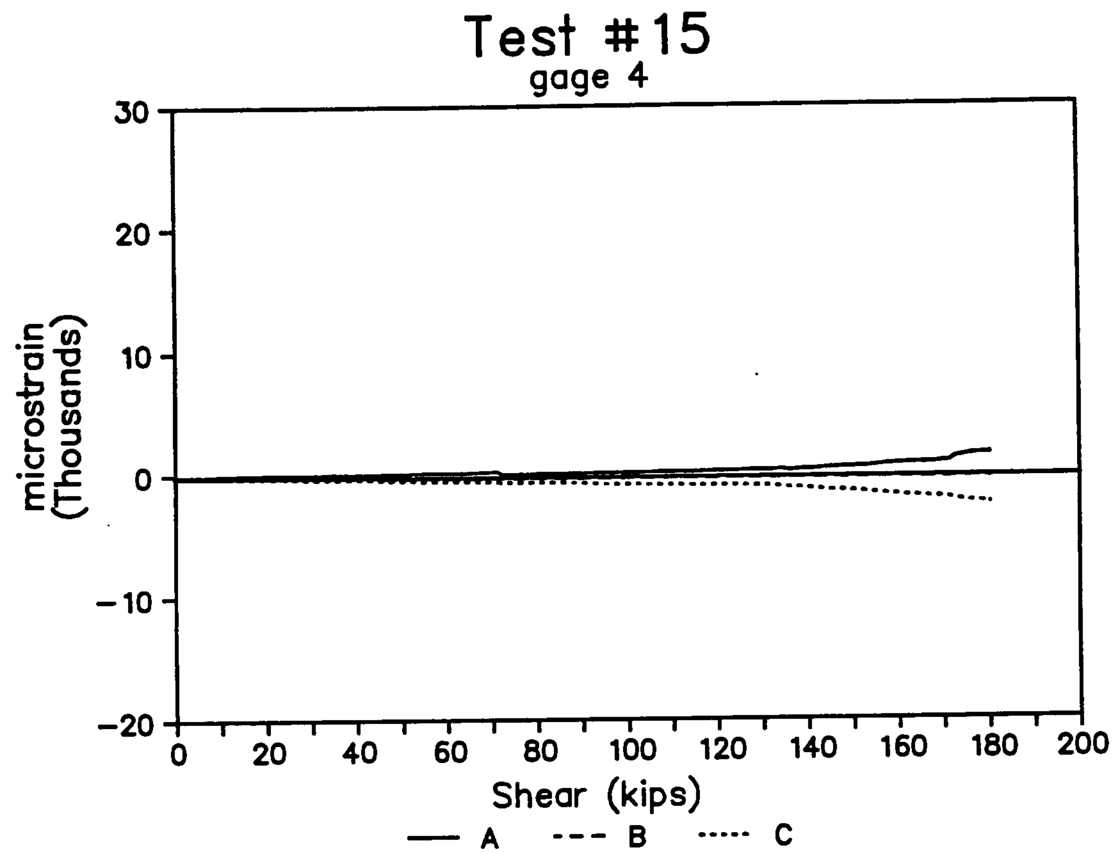
Test #14
gage 5
microstrain (Thousands)
30
20
10
0
−10
−20
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
A
B
C

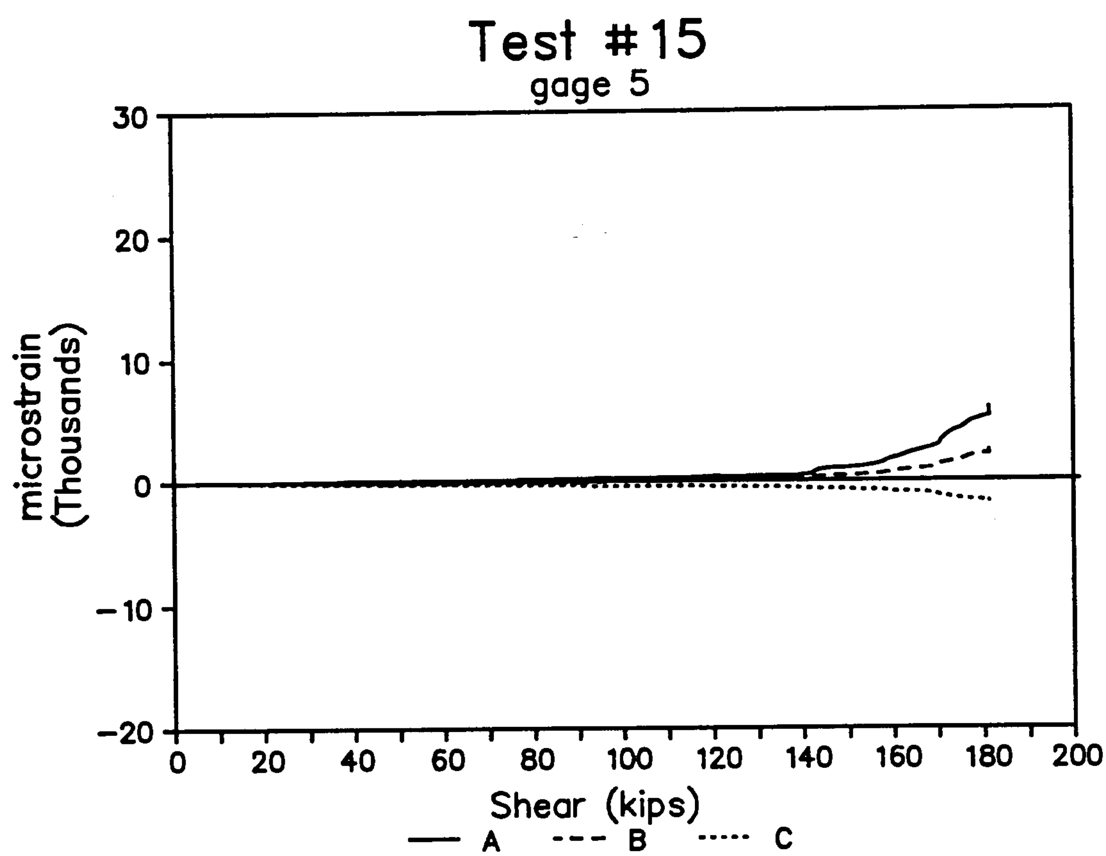


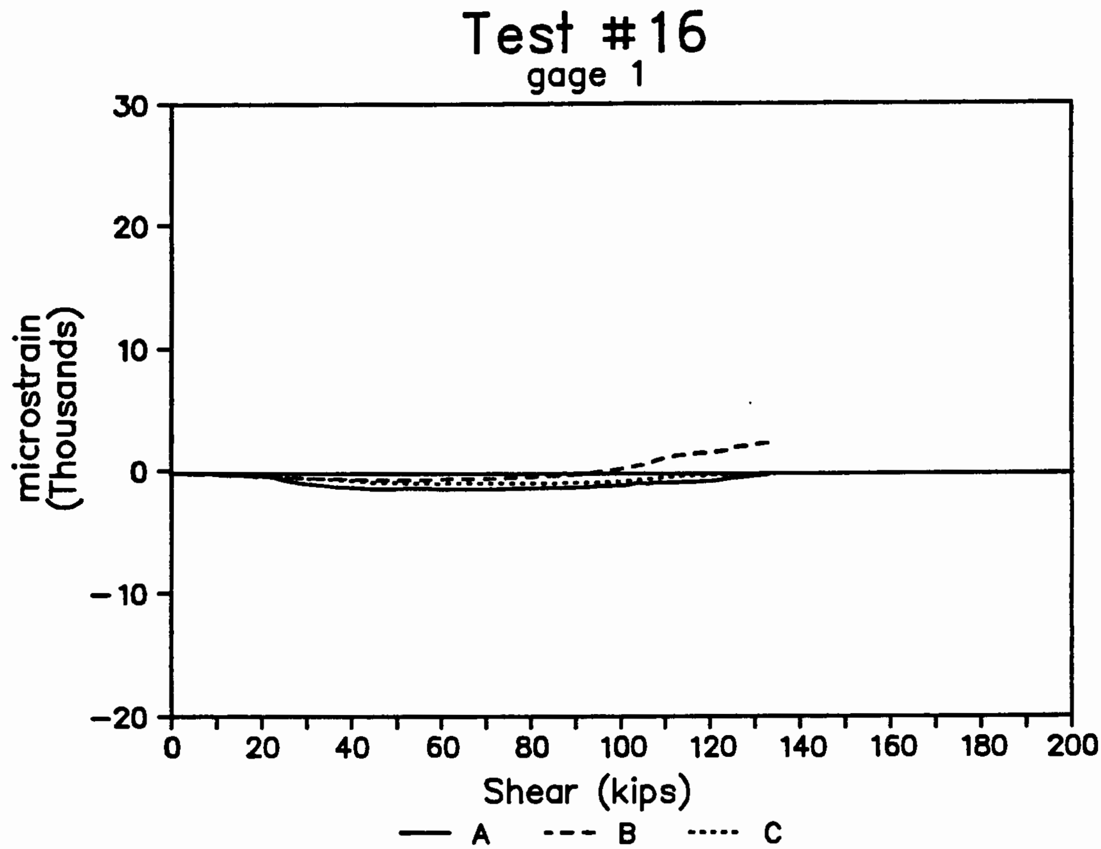
Test #15
gage 1
microstrain (Thousands)
30
20
10
0
−10
−20
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
A
B
C

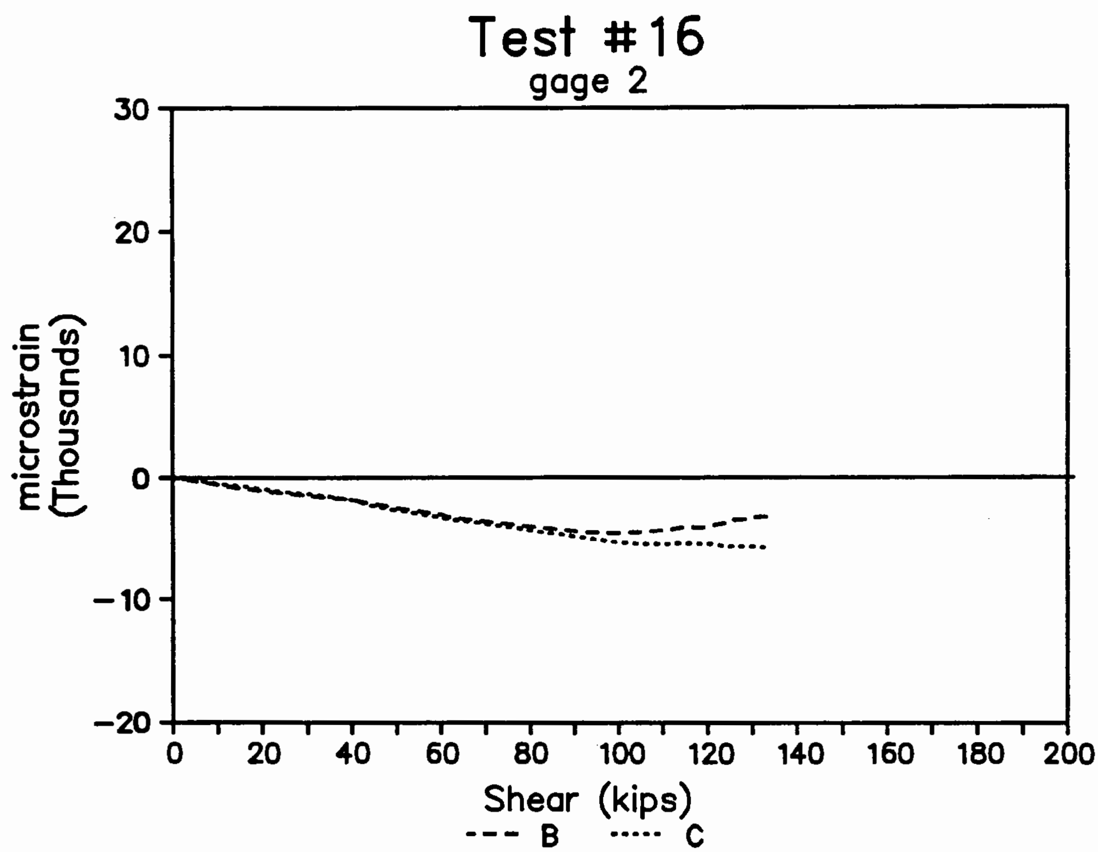

Test #15
gage 2
30
20
10
0
−10
−20
microstrain (Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
A
B
C

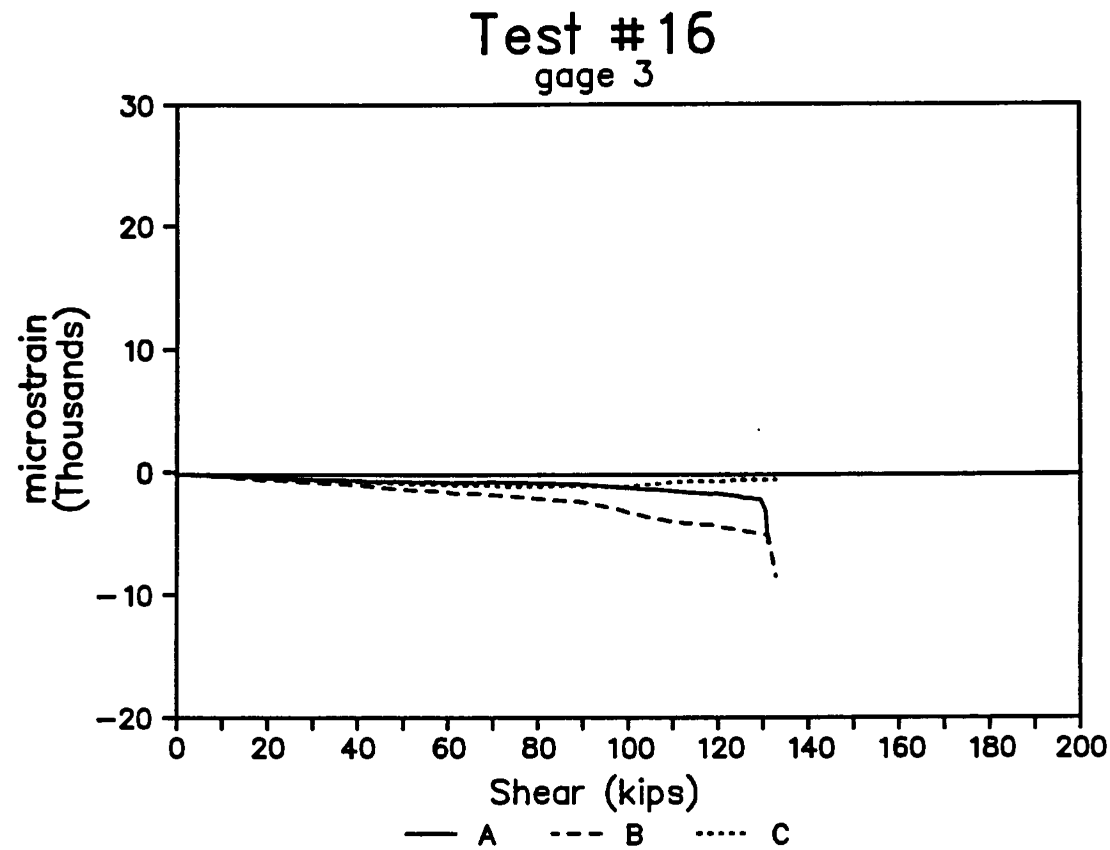

Test #15
gage 3
30
20
10
0
−10
−20
microstrain (Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
A
B
C


Test #15
gage 4
microstrain
(Thousands)
30
20
10
0
−10
−20
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
A
B
C

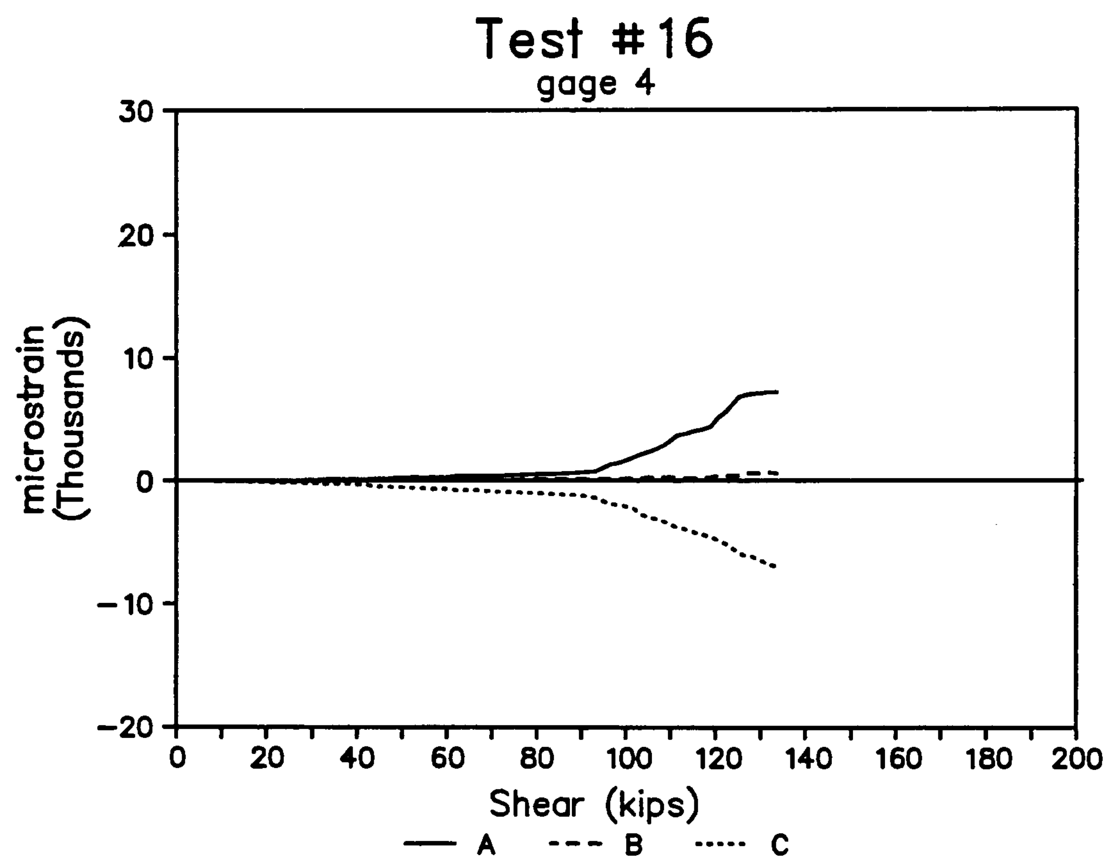

Test #15
gage 5
microstrain
(Thousands)
30
20
10
0
−10
−20
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
A
B
C


Test #16
gage 1
30
20
10
0
−10
−20
microstrain
(Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
A
B
C

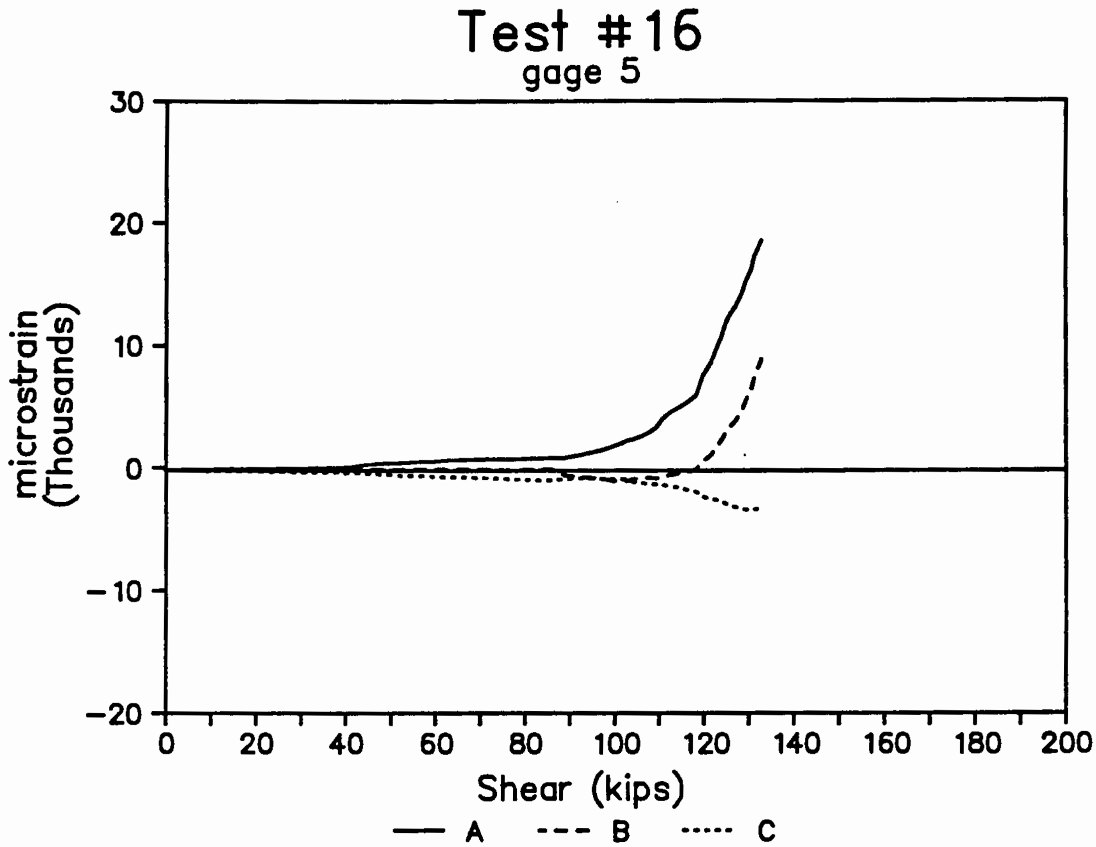

Test #16
gage 2
30
20
10
0
−10
−20
microstrain
(Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
B
C

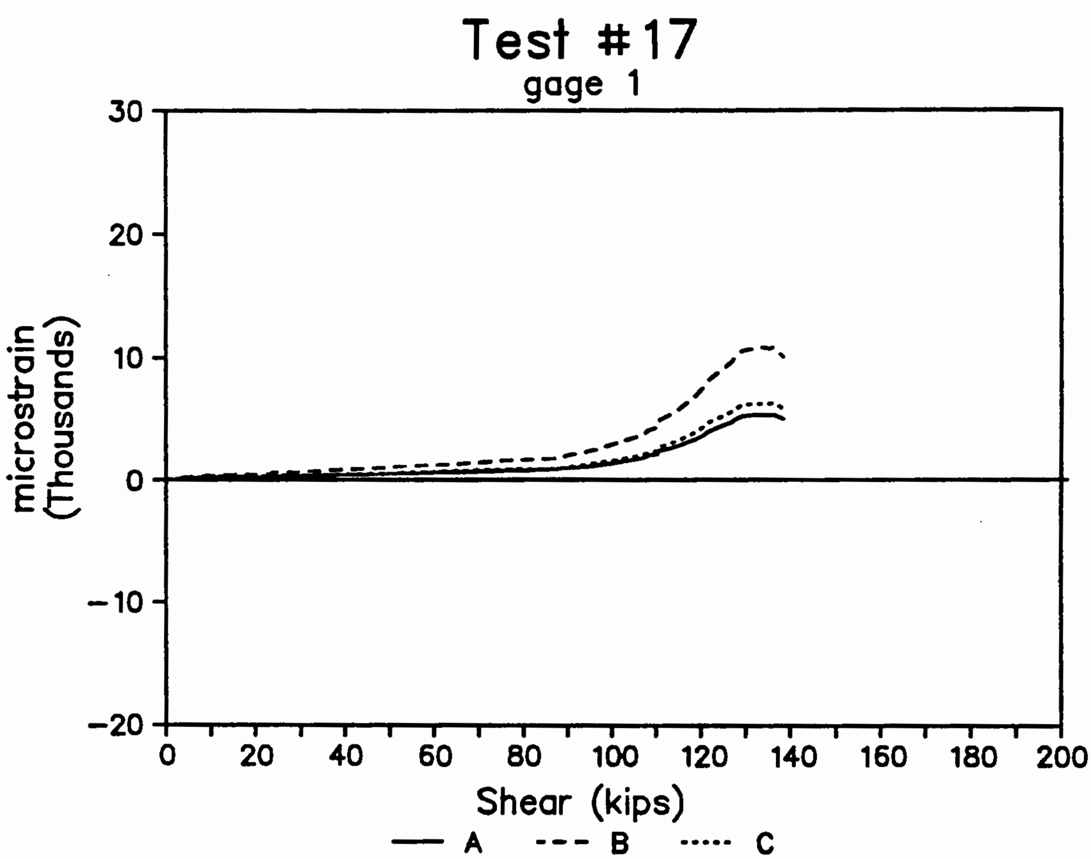


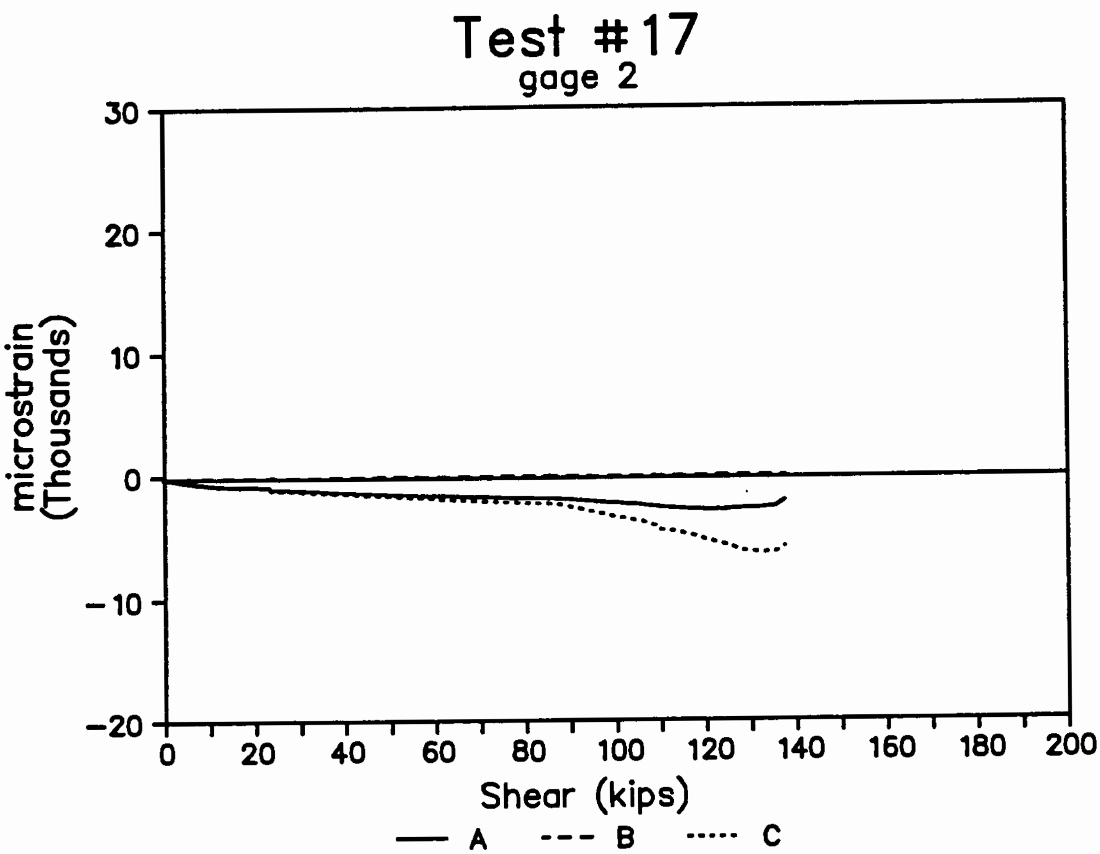
Test #16
gage 3
30
20
10
0
−10
−20
microstrain
(Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
A
B
C

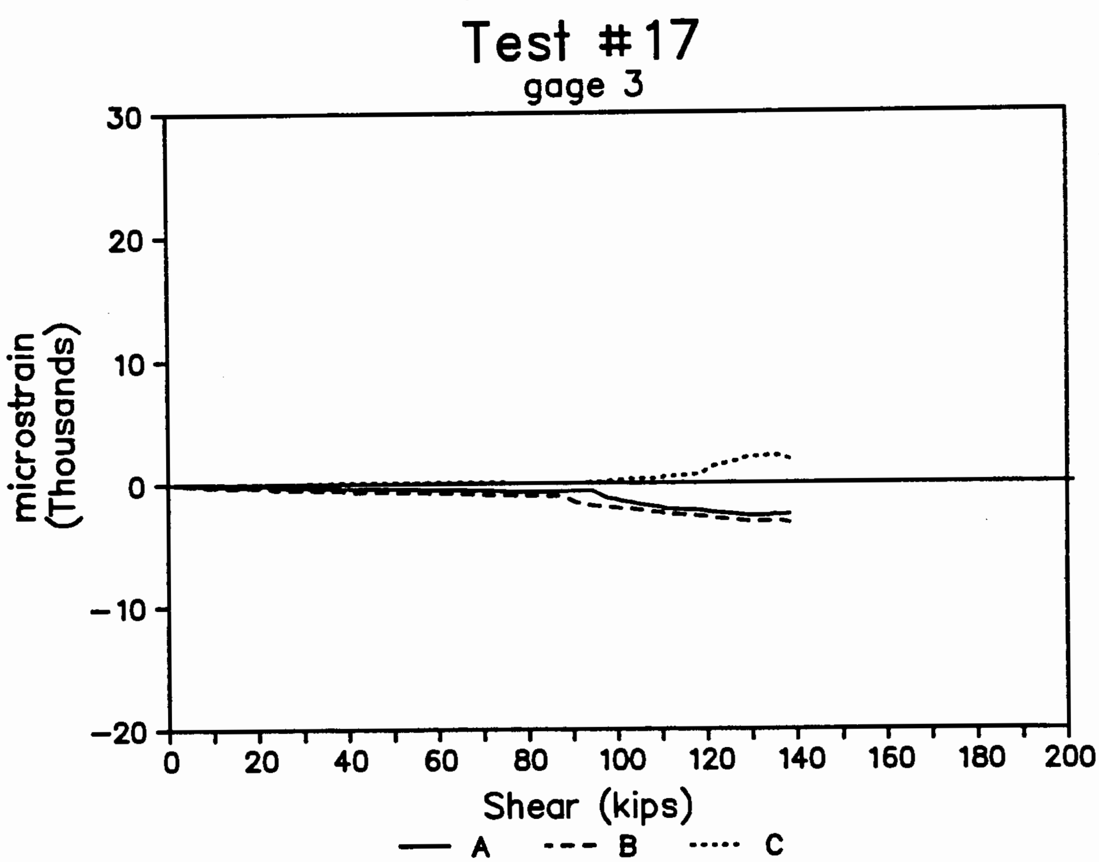


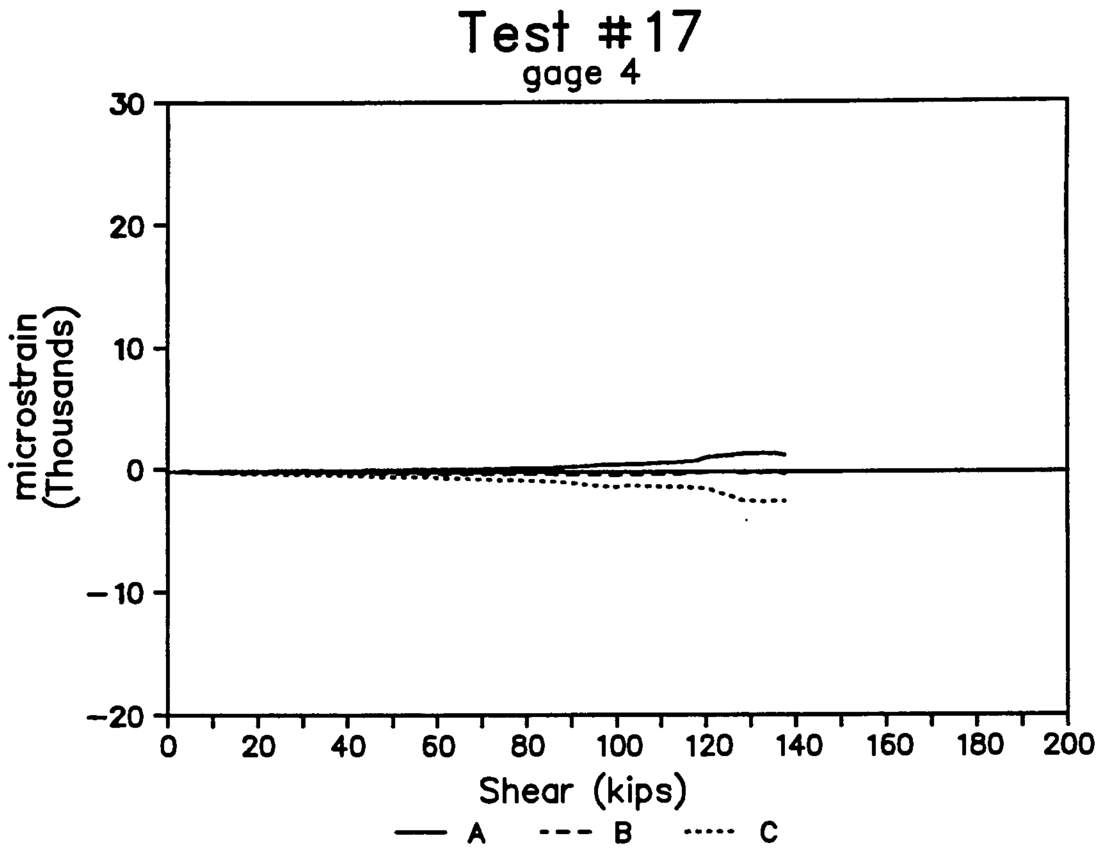
Test #16
gage 4
30
20
10
0
−10
−20
microstrain
(Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
A
B
C

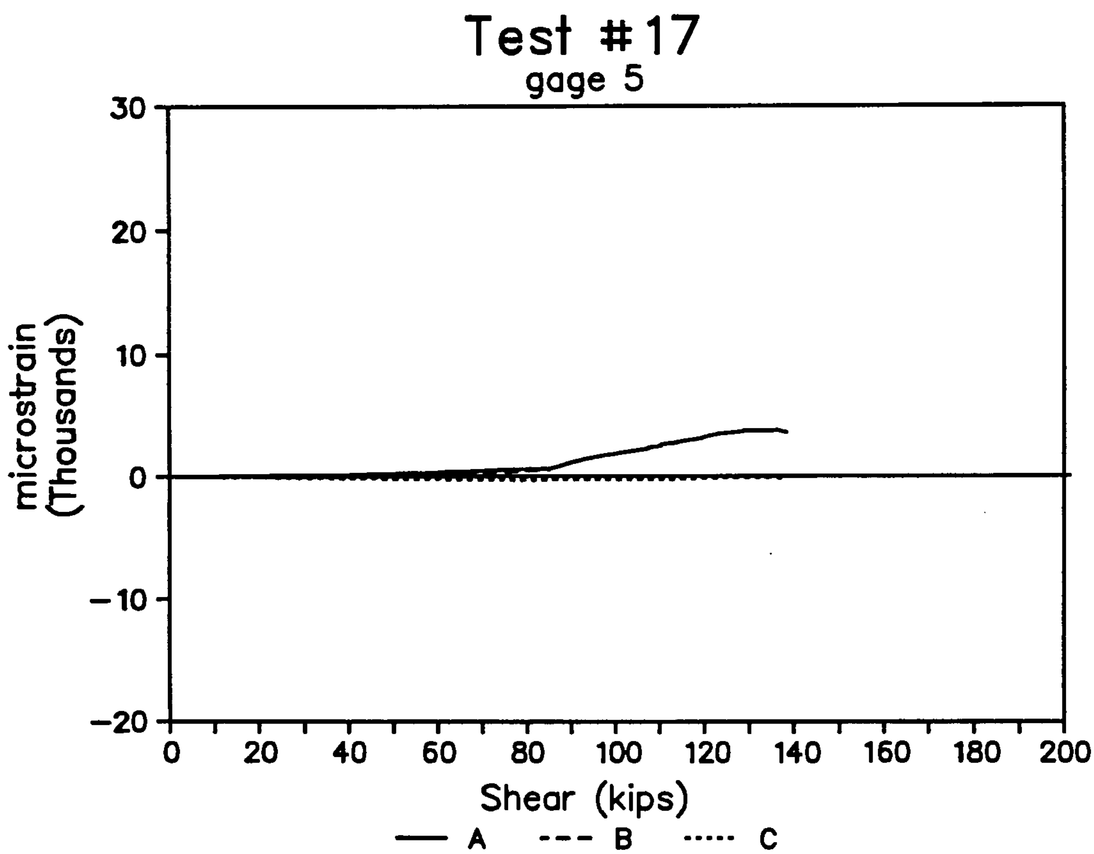

Test #16
gage 5
30
20
10
0
-10
-20
microstrain
(Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
A
B
C


Test #17
gage 1
30
20
10
0
-10
-20
microstrain
(Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
A
B
C


Test #17
gage 2
30
20
10
0
-10
-20
microstrain
(Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
A
B
C


Test #17
gage 3
30
20
10
0
-10
-20
microstrain
(Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
A
B
C

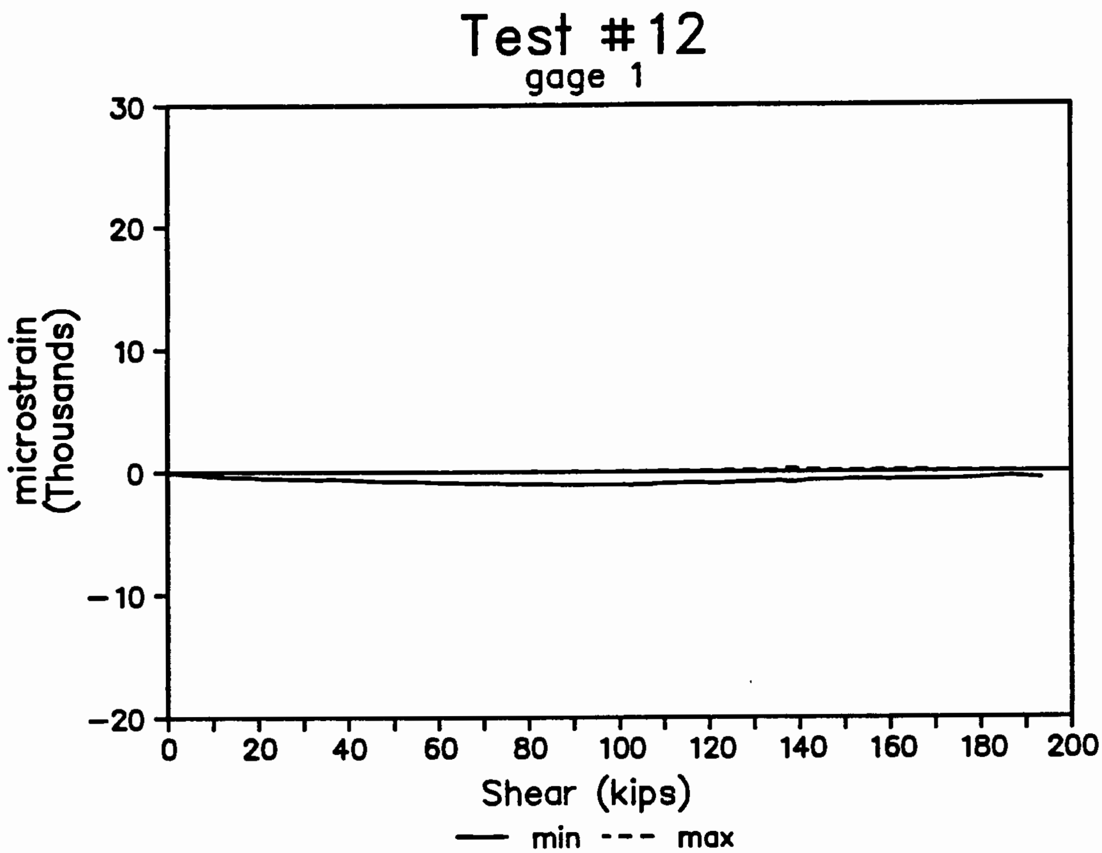

Test #17
gage 4
30
20
10
0
−10
−20
microstrain
(Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
A
B
C

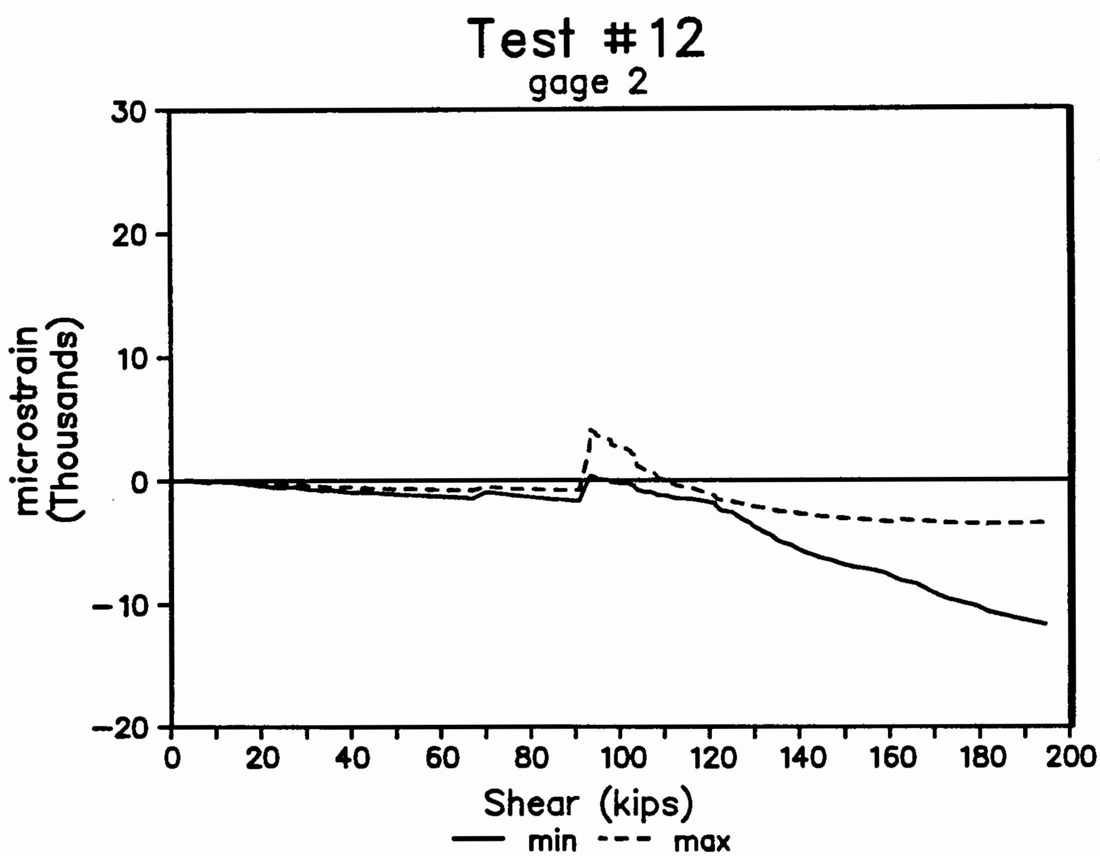

Test #17
gage 5
30
20
10
0
−10
−20
microstrain
(Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
A
B
C

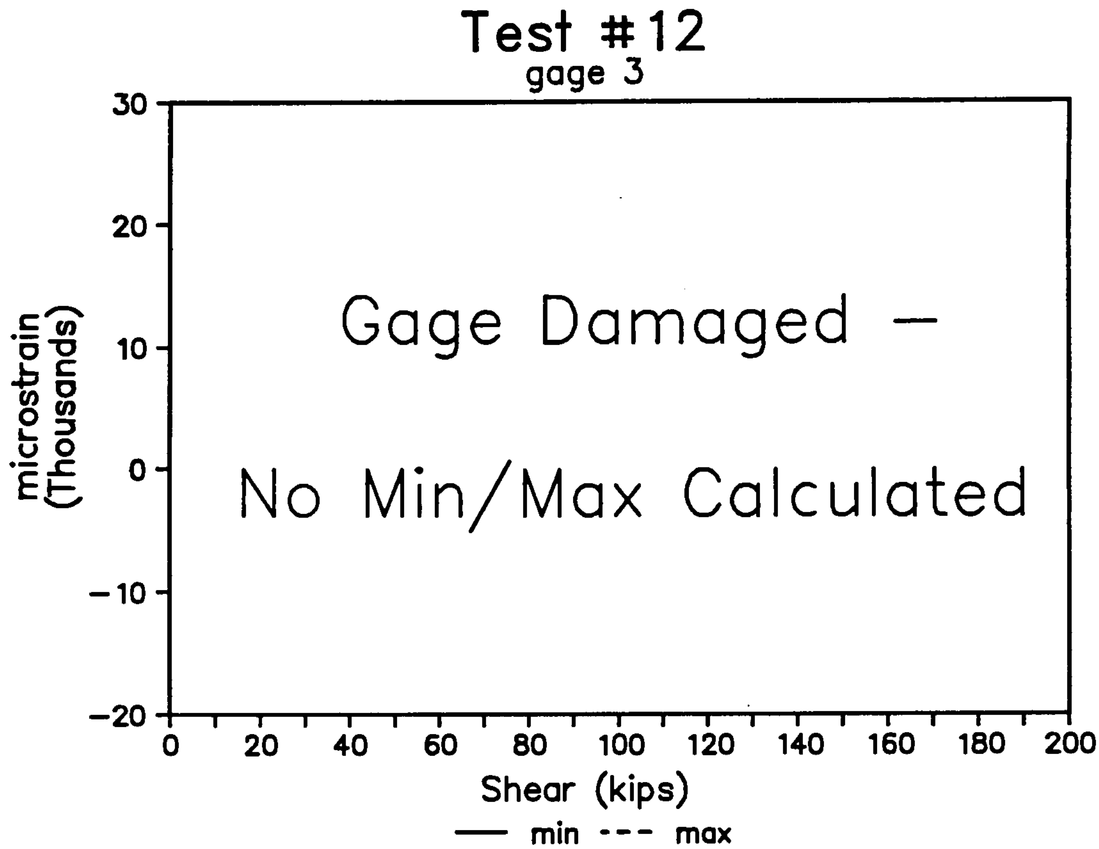
Appendix B

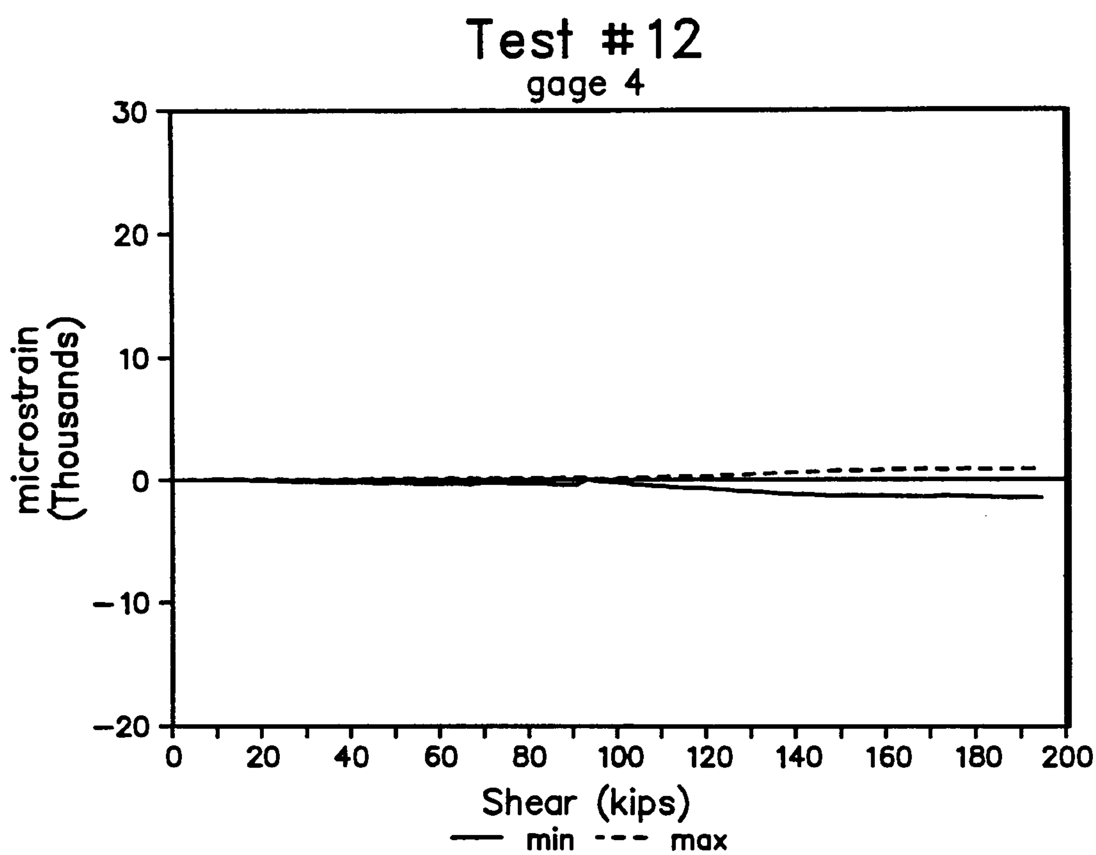
Appendix E

Test Summaries


Test #12
gage 1
30
20
10
0
−10
−20
microstrain
(Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
min
max

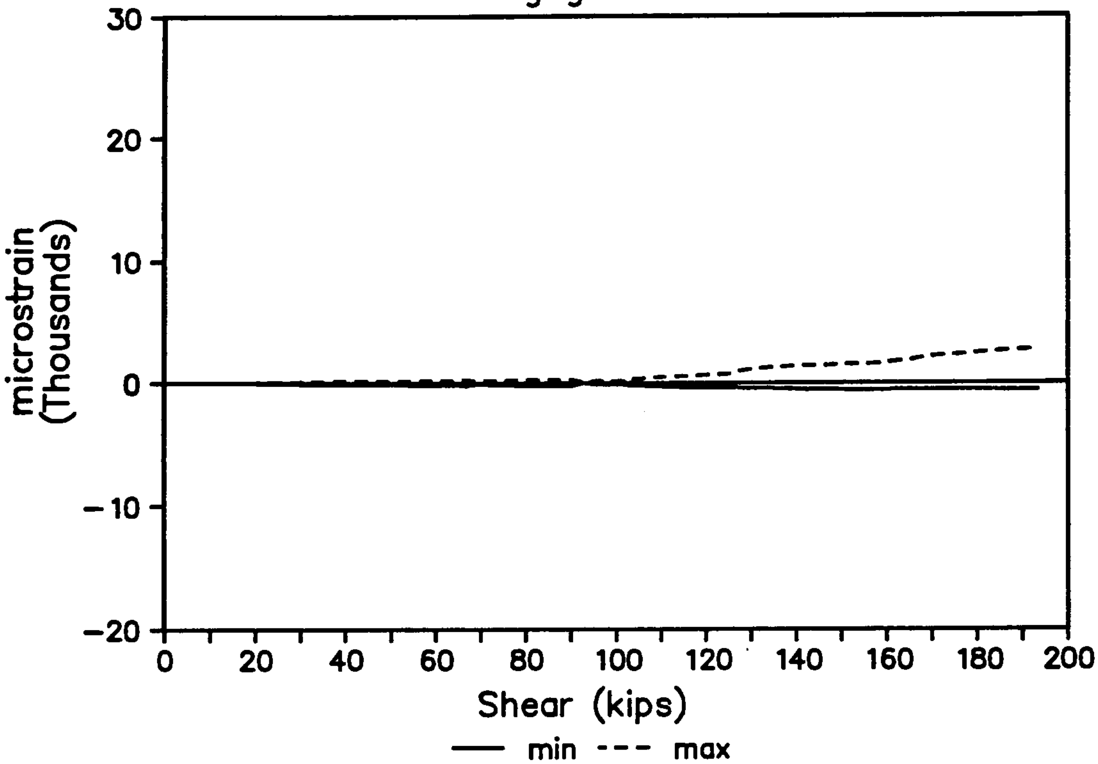

Test #12
gage 2
30
20
10
0
−10
−20
microstrain
(Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
min
max


Test #12
gage 3
microstrain (Thousands)
30
20
10
0
-10
-20
Gage Damaged -
No Min/Max Calculated
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
min
max


Test #12
gage 4
microstrain (Thousands)
30
20
10
0
-10
-20
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
min
max

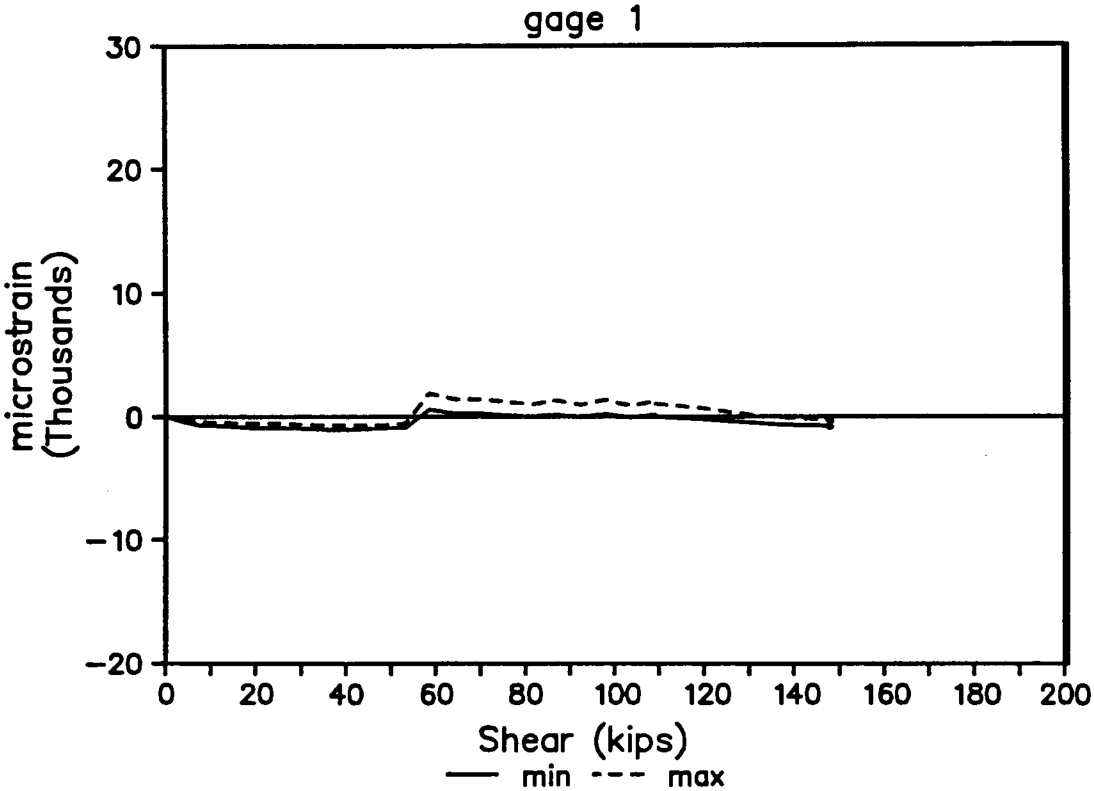
Test #12

gage 5



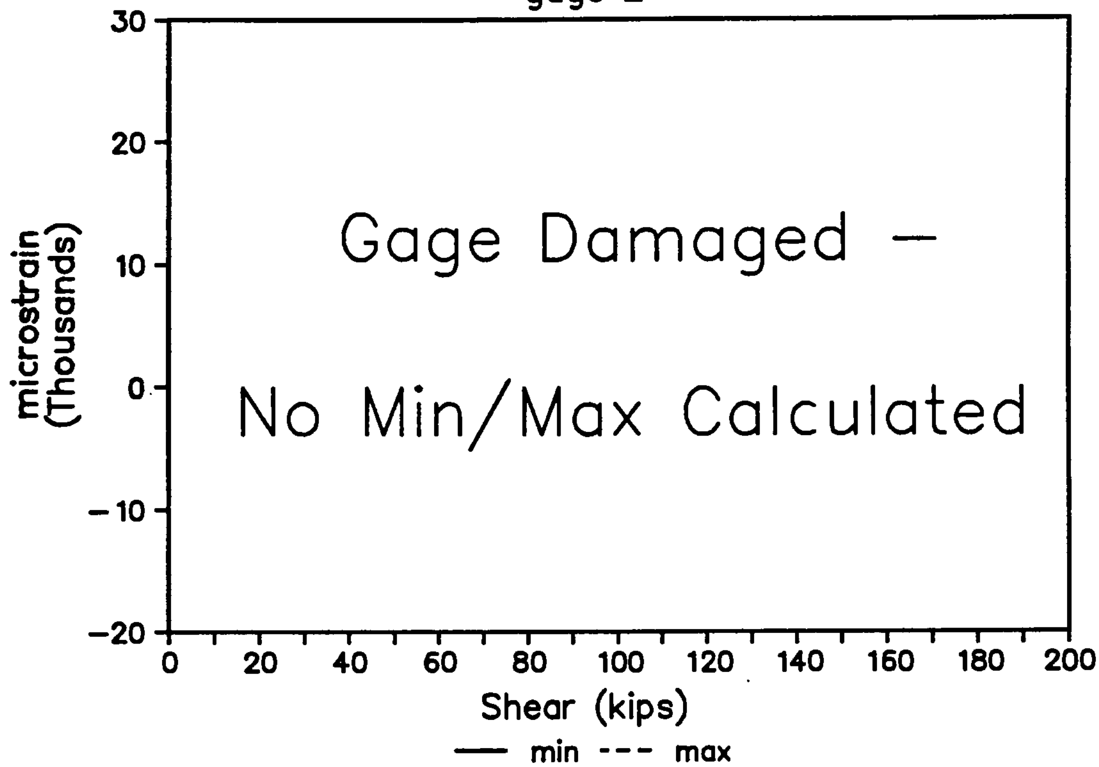
Test #13

gage 1



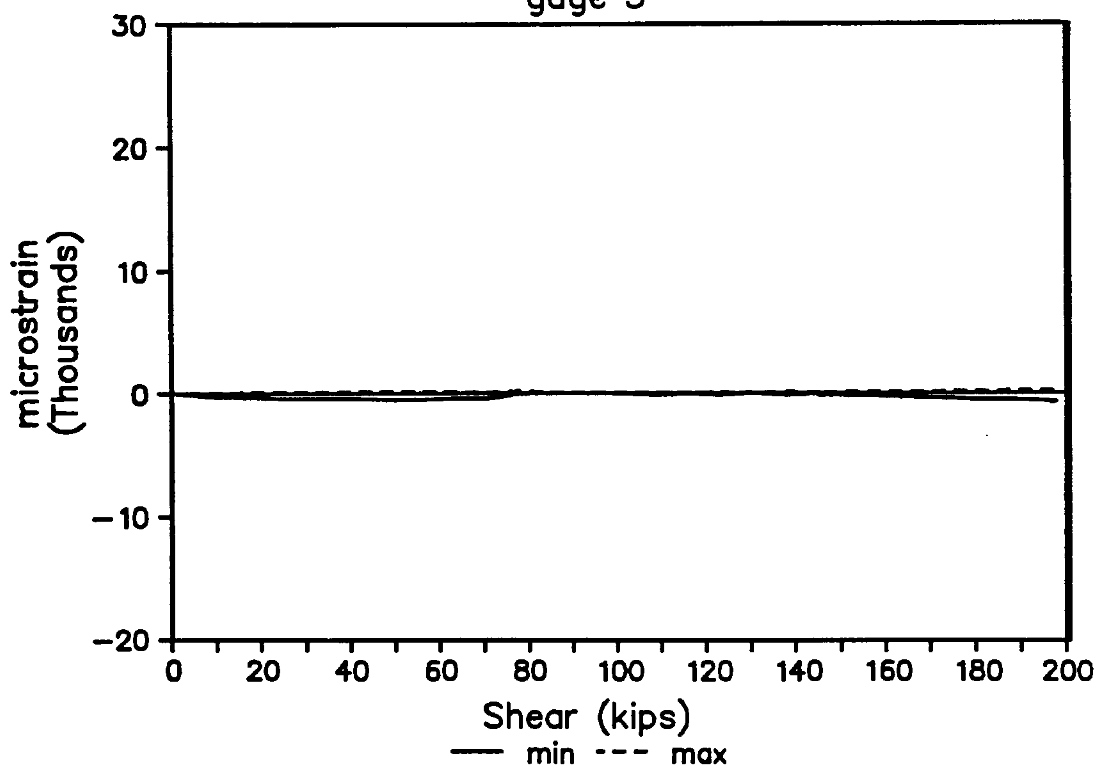
Test #13

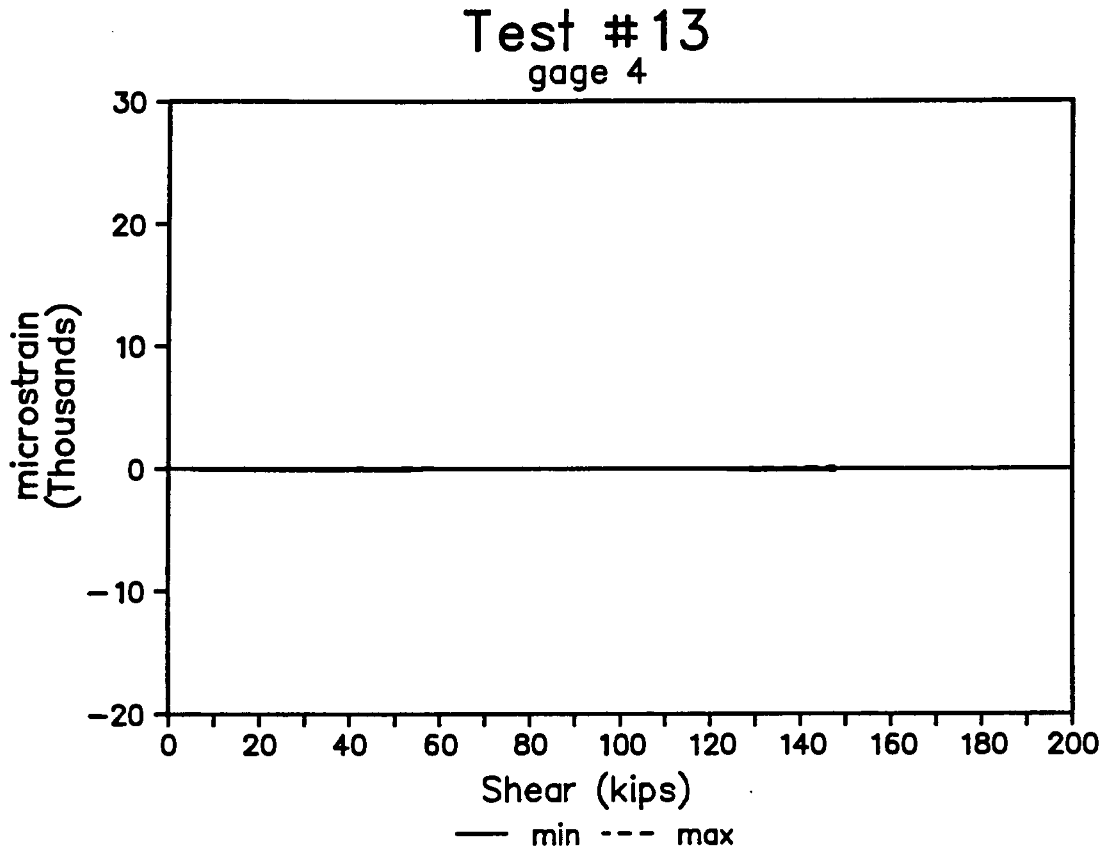
gage 2



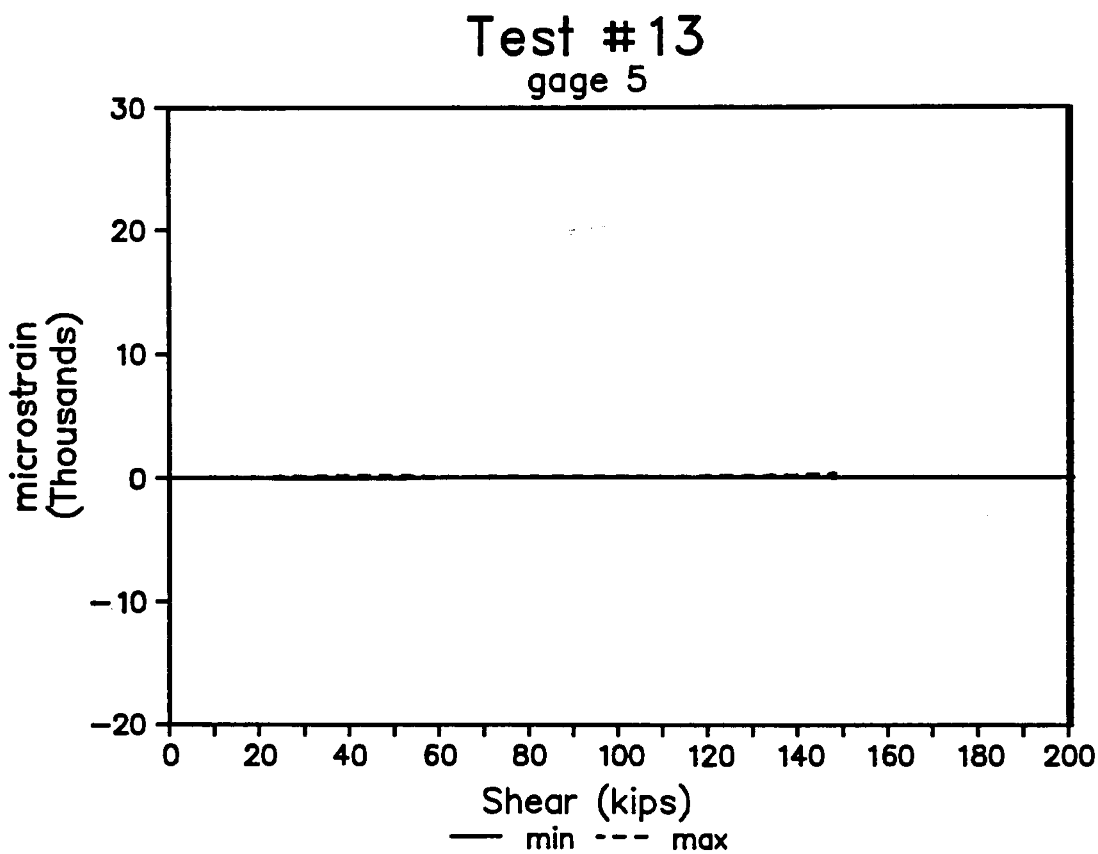
Test #13

gage 3



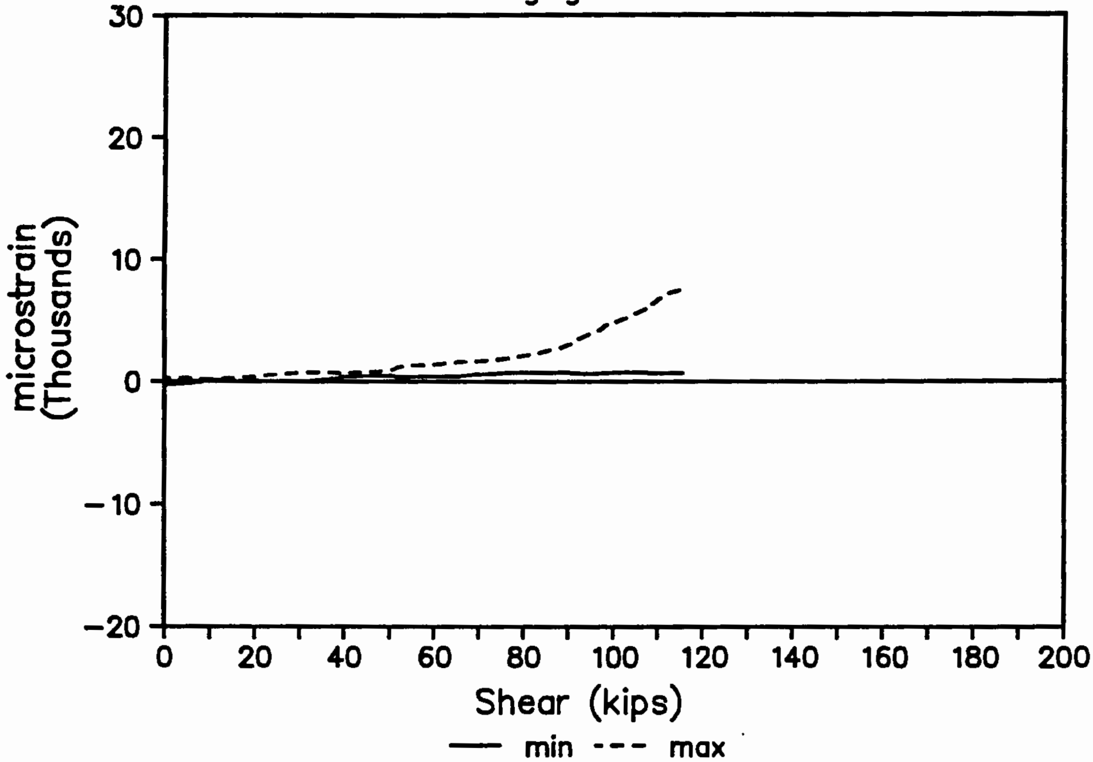


Test #13
gage 4
30
20
10
0
-10
-20
microstrain
(Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
min
max

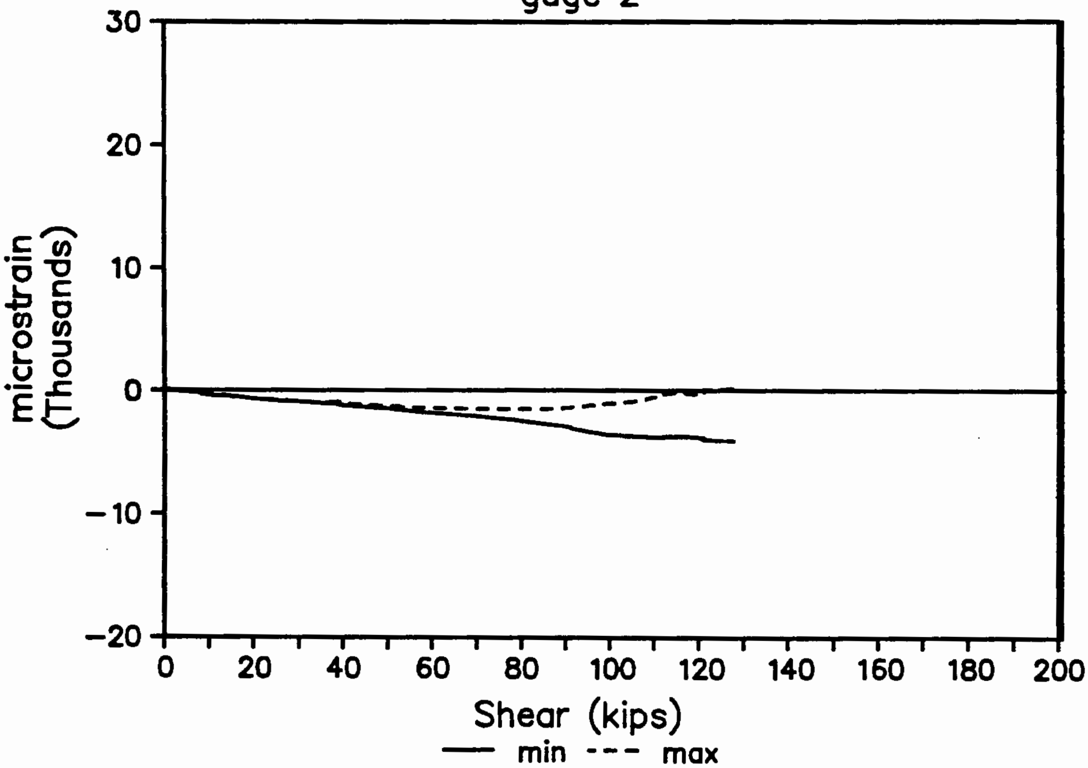


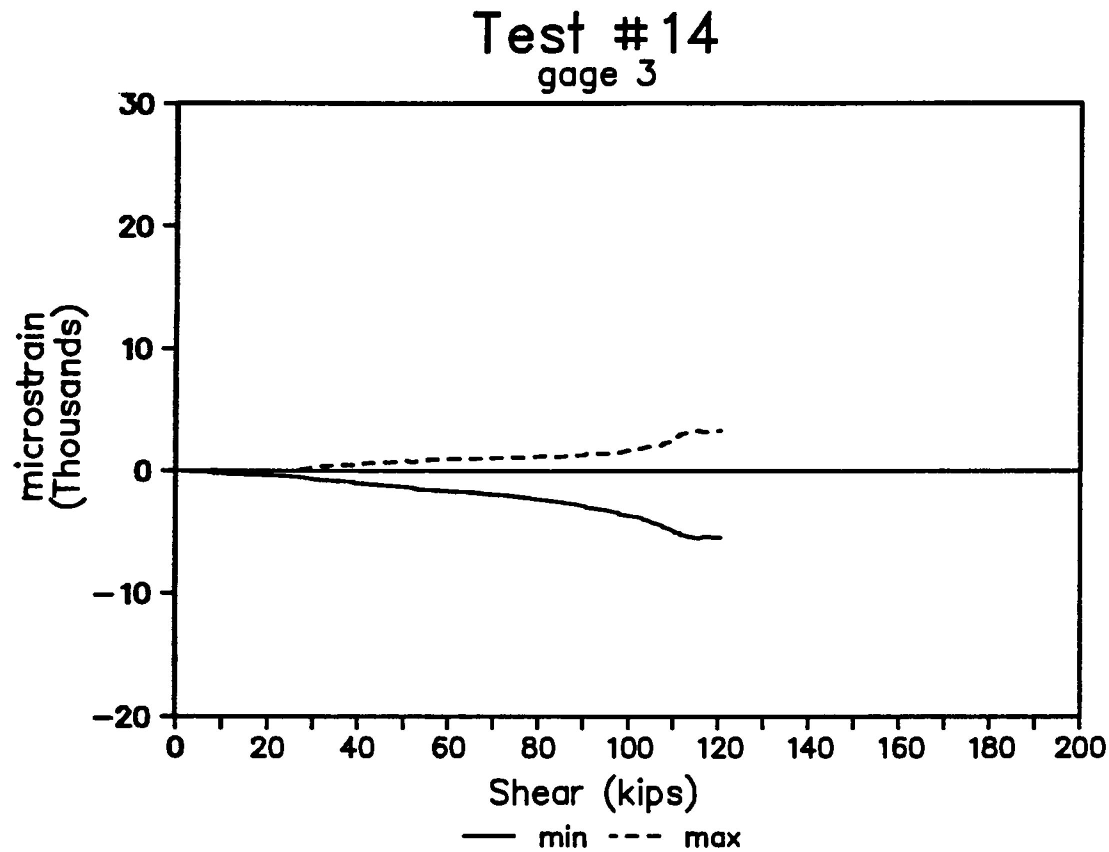
Test #13
gage 5
30
20
10
0
-10
-20
microstrain
(Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
min
max

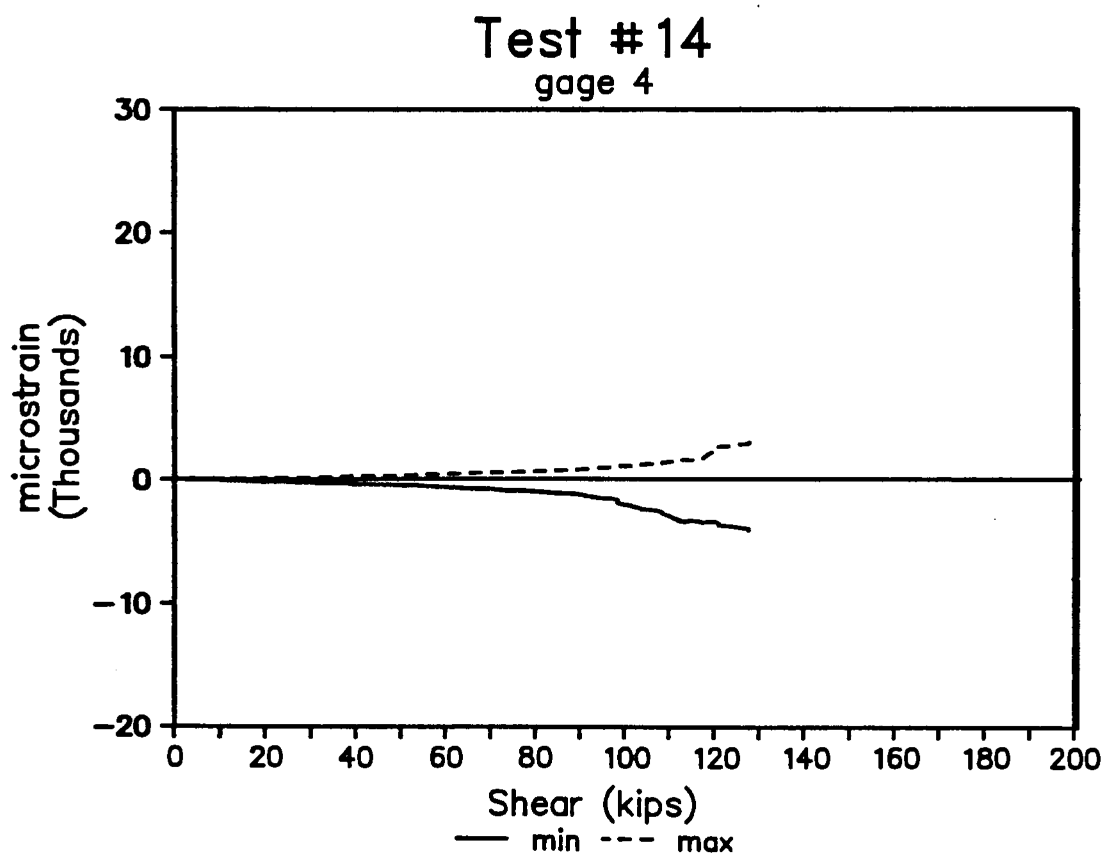
Test #14
gage 1



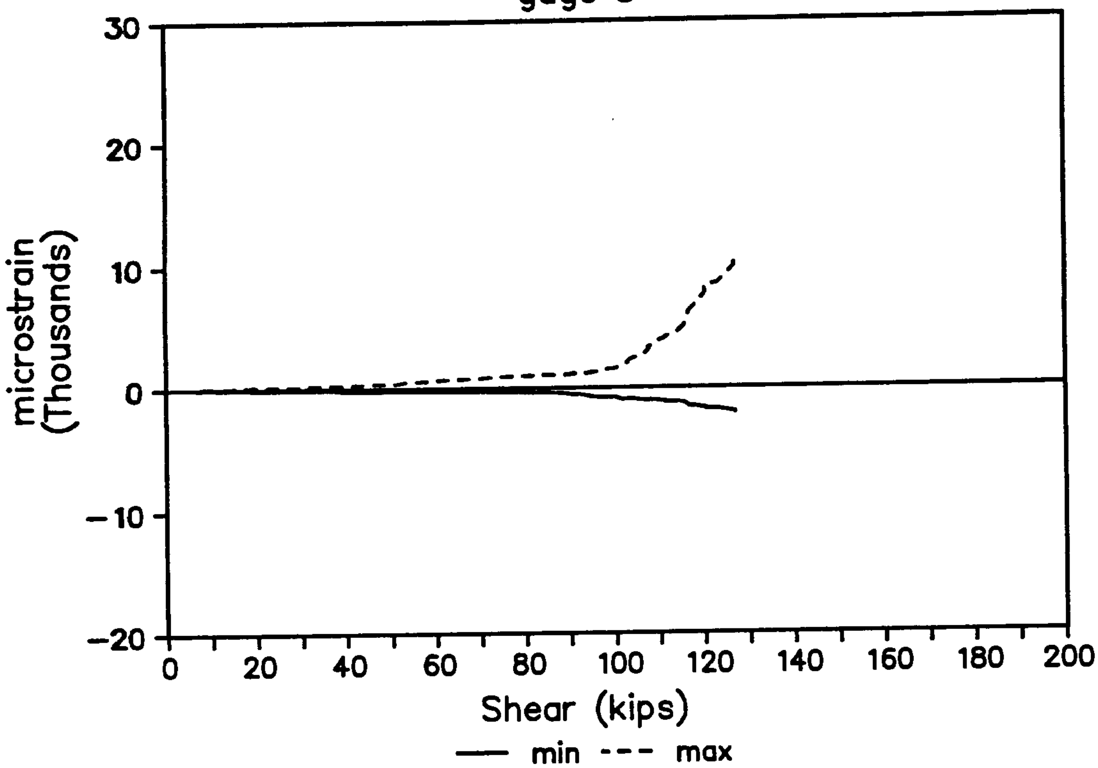
Test #14
gage 2



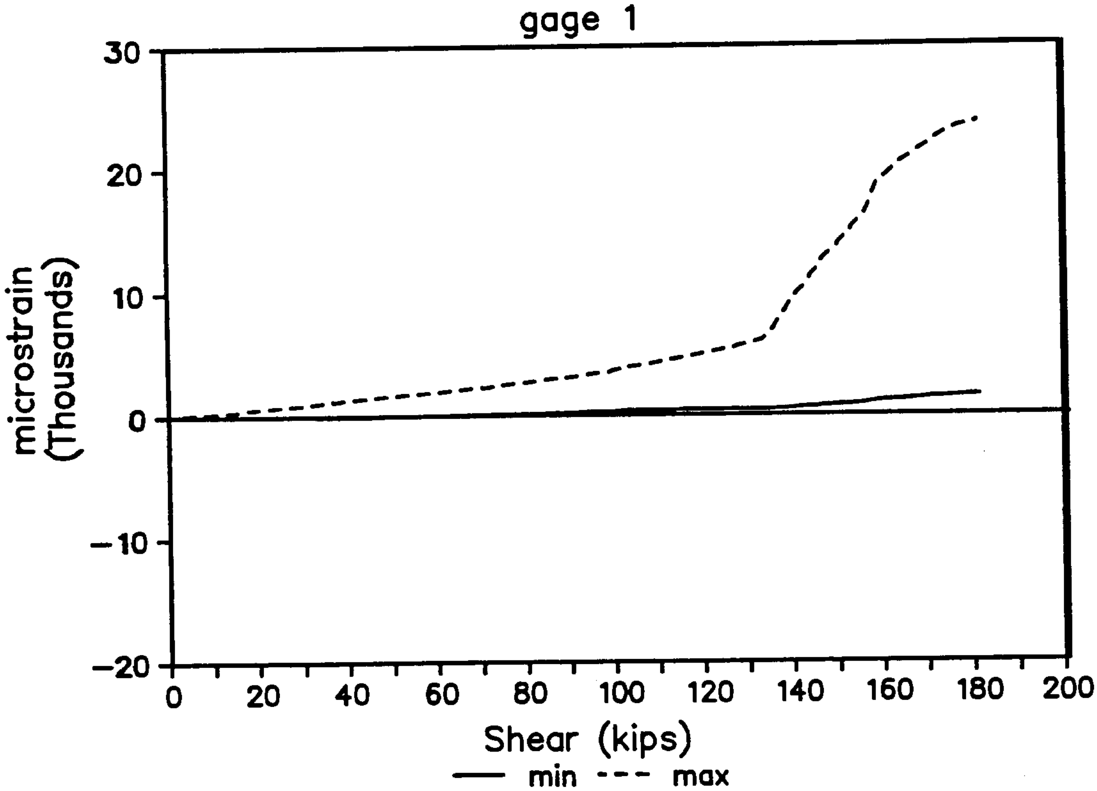

Test #14
gage 3
30
20
10
0
−10
−20
microstrain
(Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
min
max

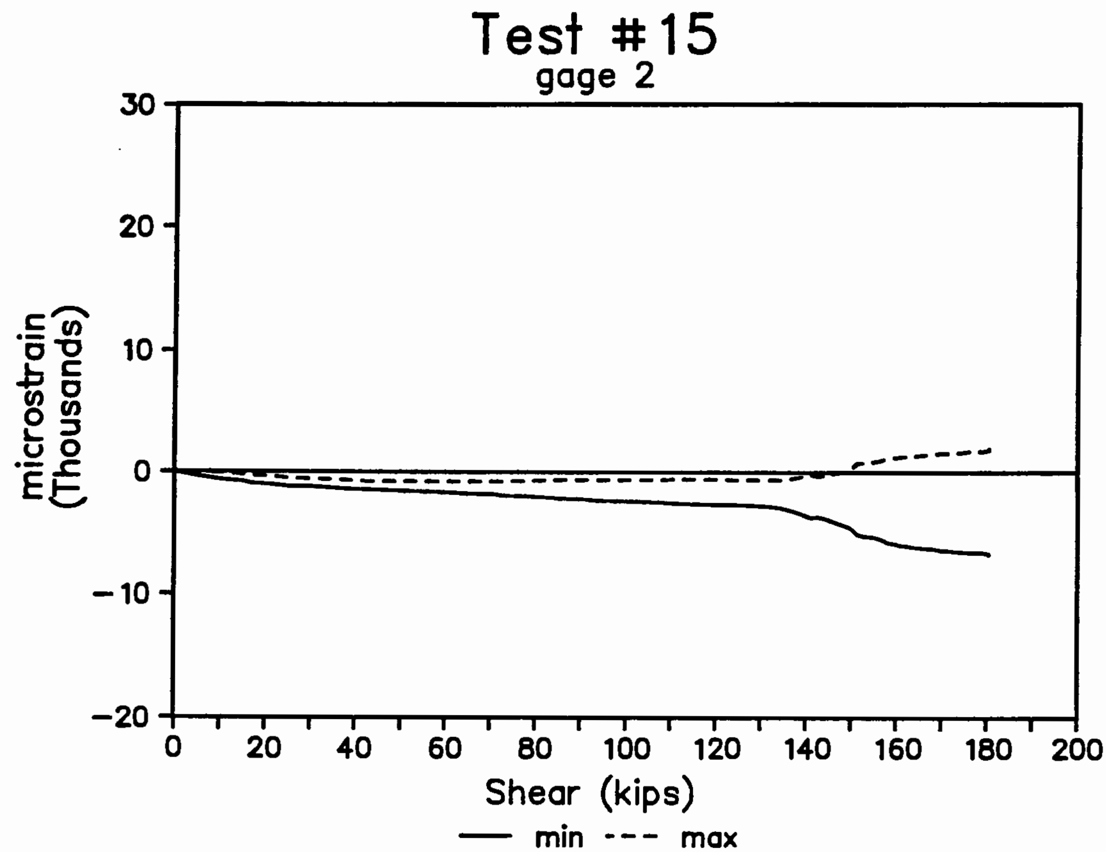

Test #14
gage 4
30
20
10
0
−10
−20
microstrain
(Thousands)
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
min
max

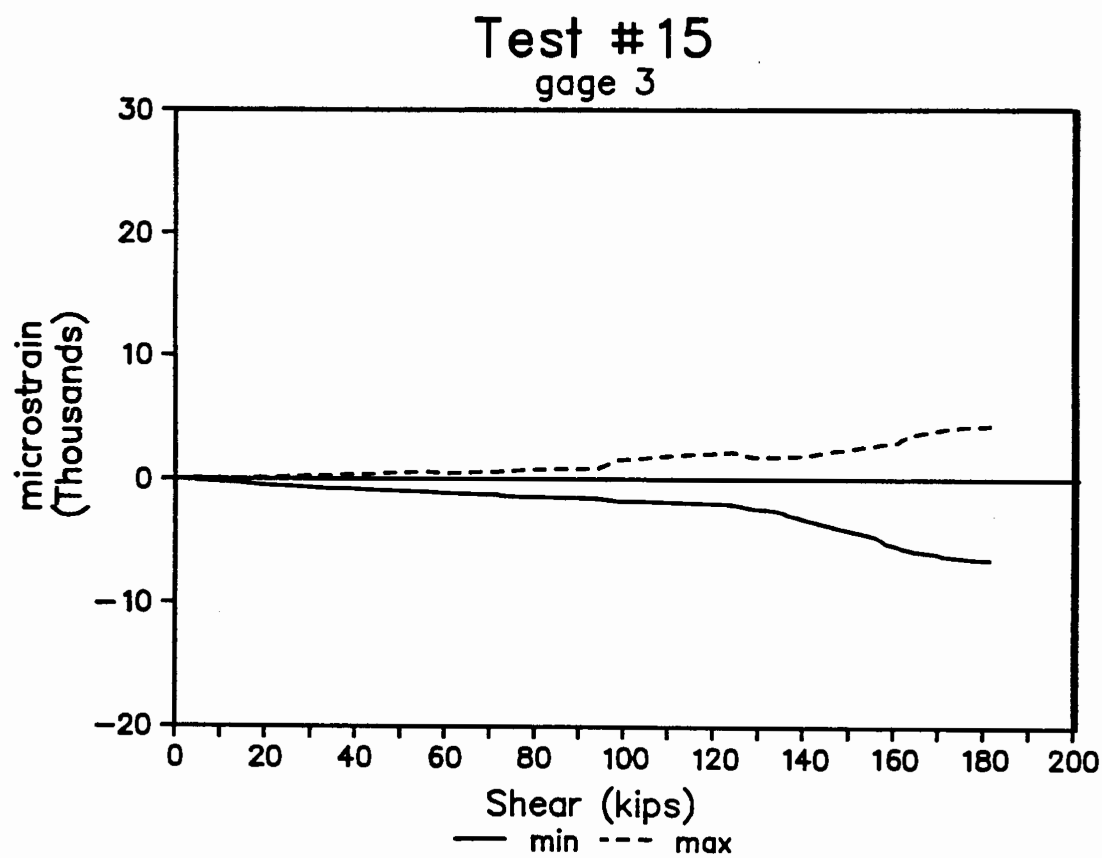
Test #14
gage 5

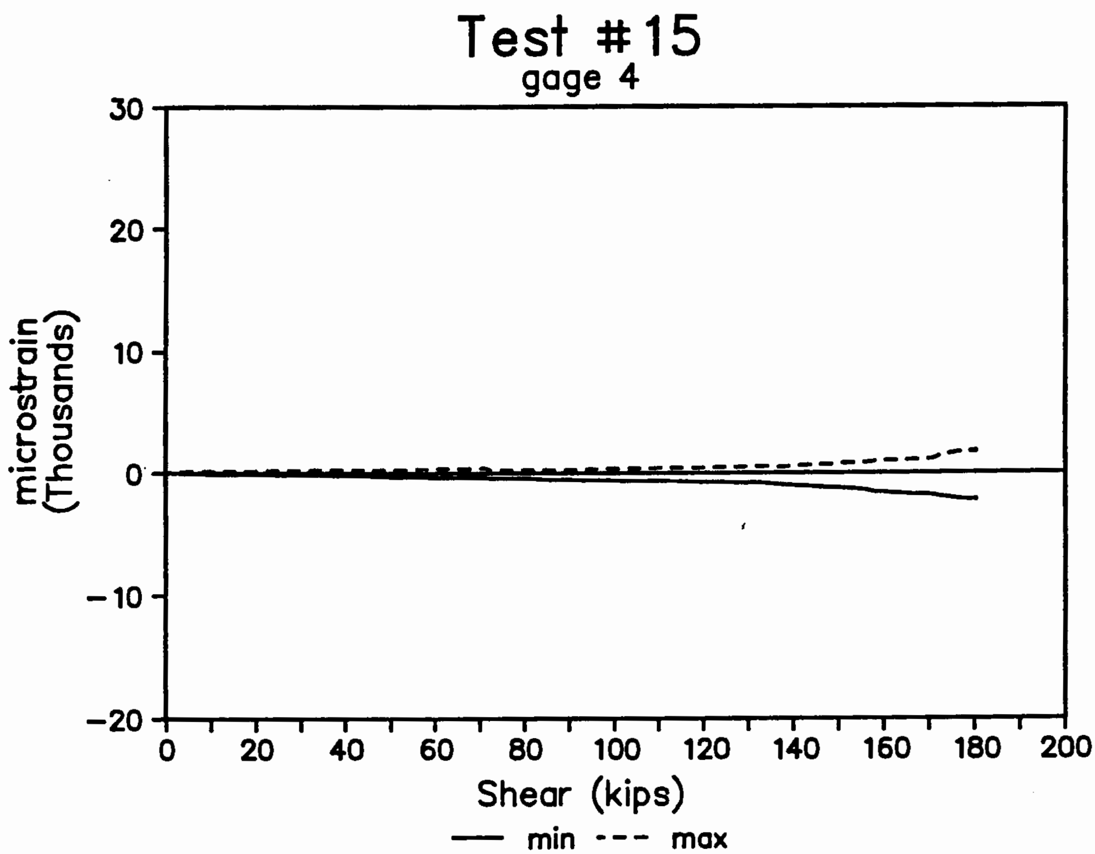


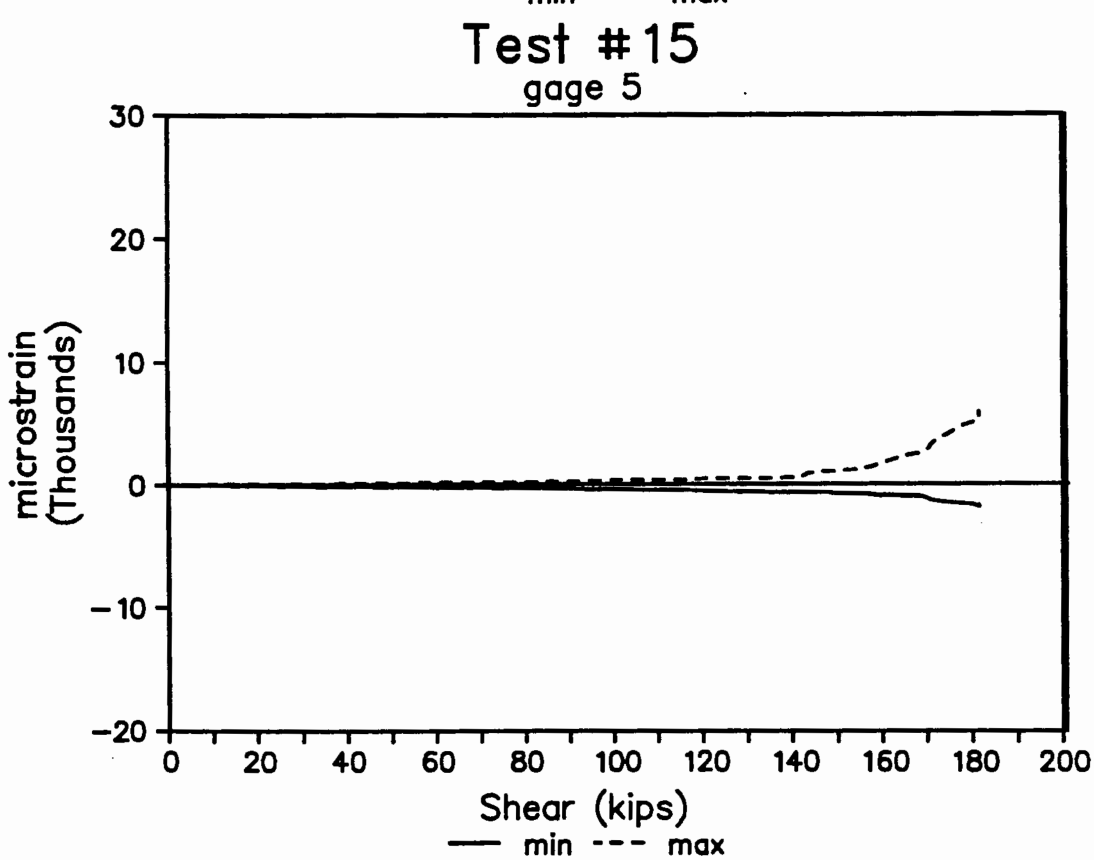
Test #15
gage 1

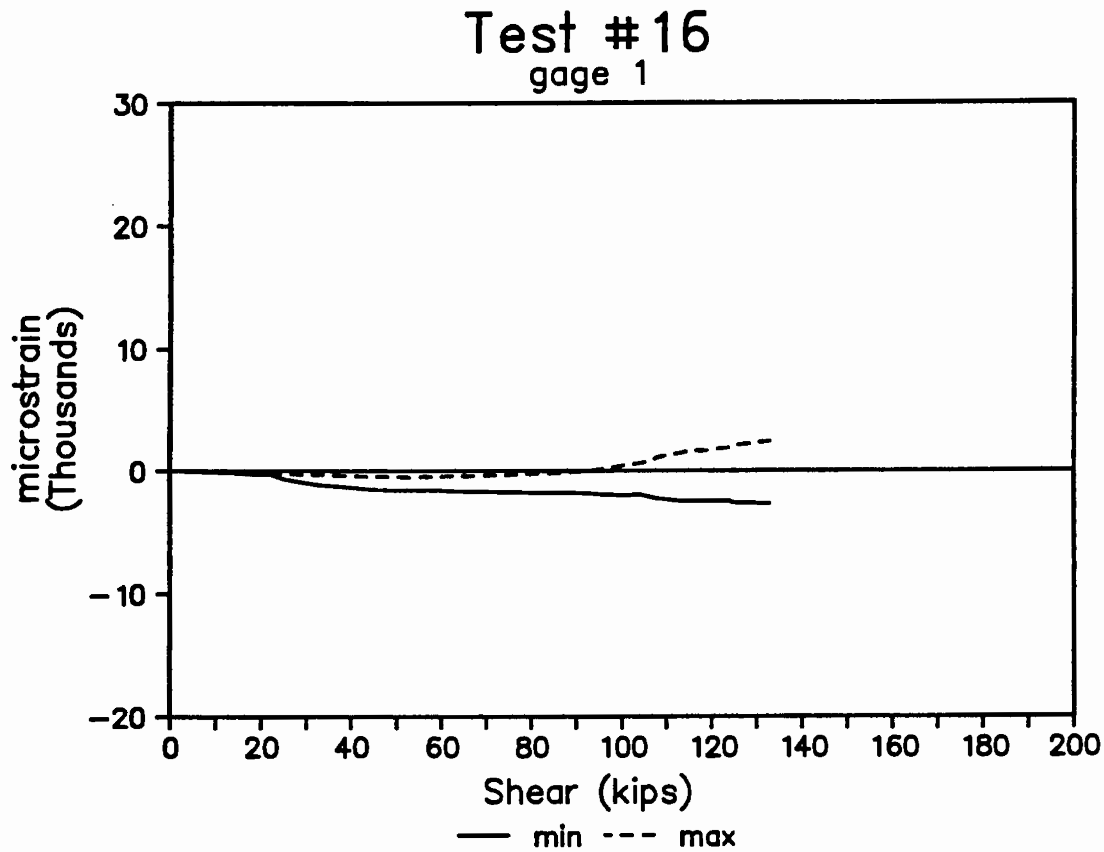


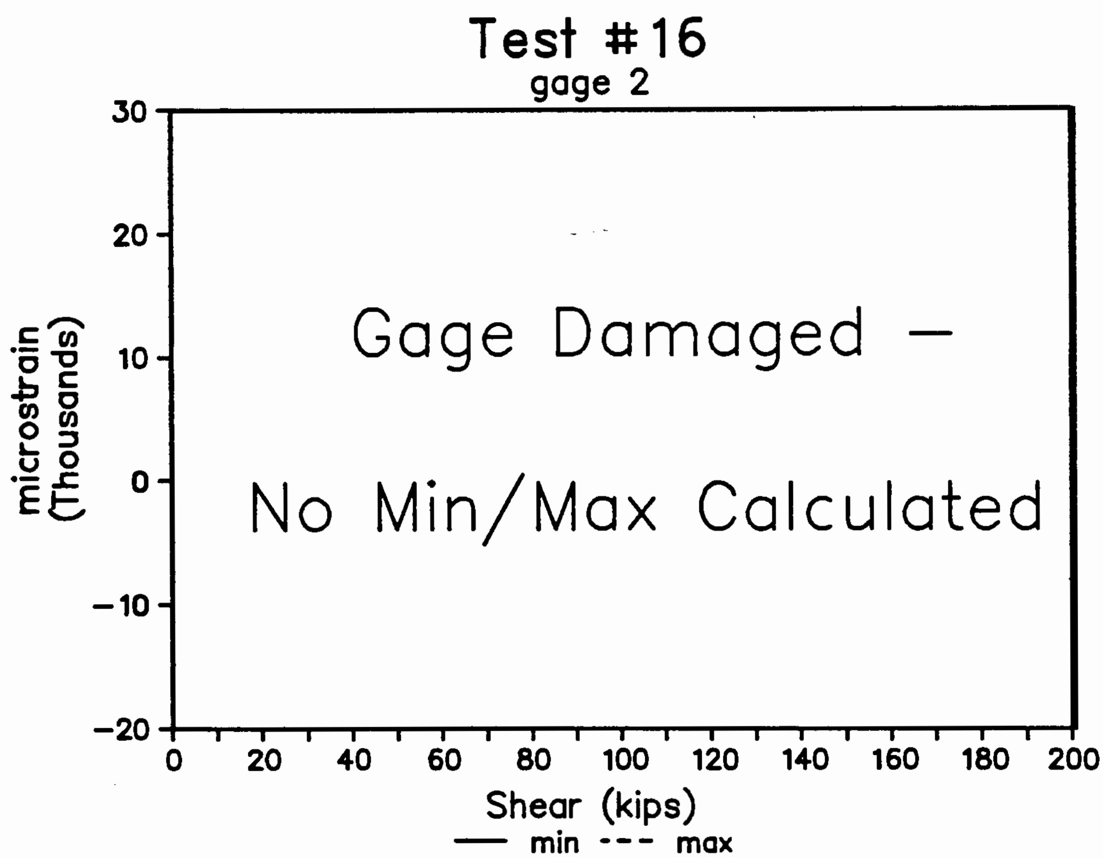

Test #15
gage 2
microstrain (Thousands)
30
20
10
0
−10
−20
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
min
max

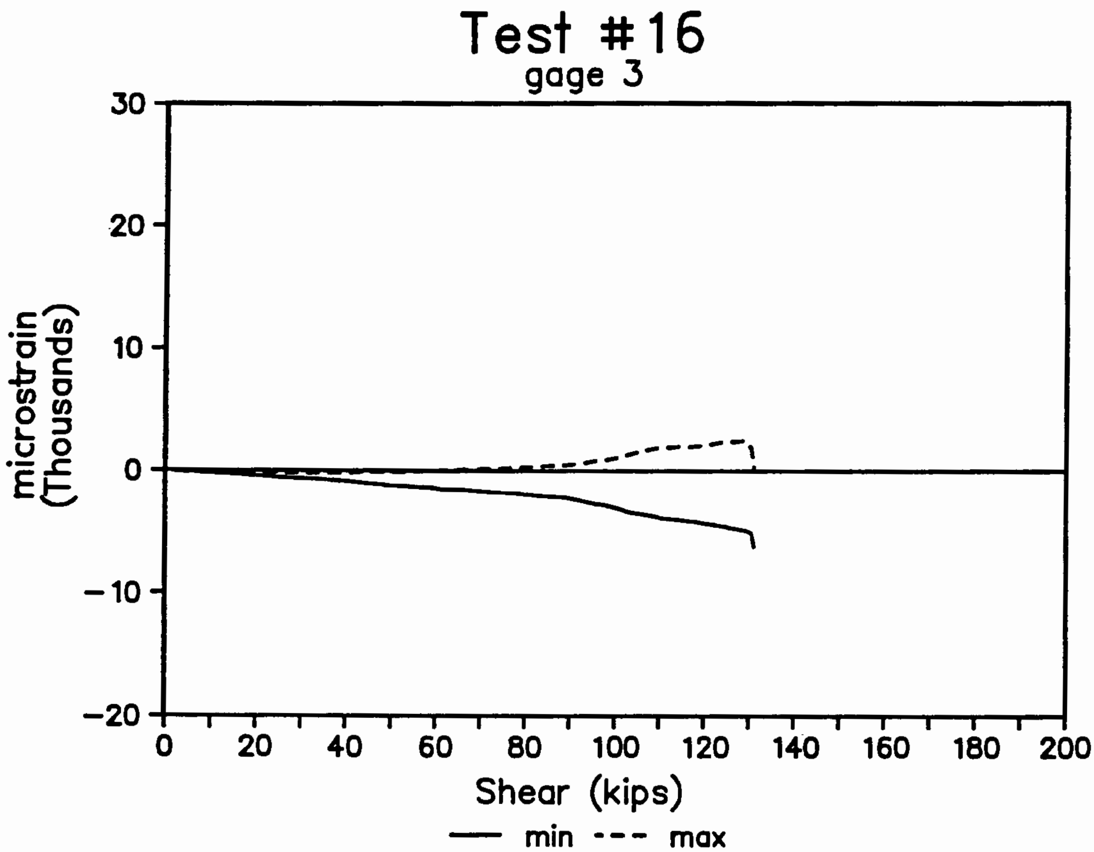

Test #15
gage 3
microstrain (Thousands)
30
20
10
0
−10
−20
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
min
max

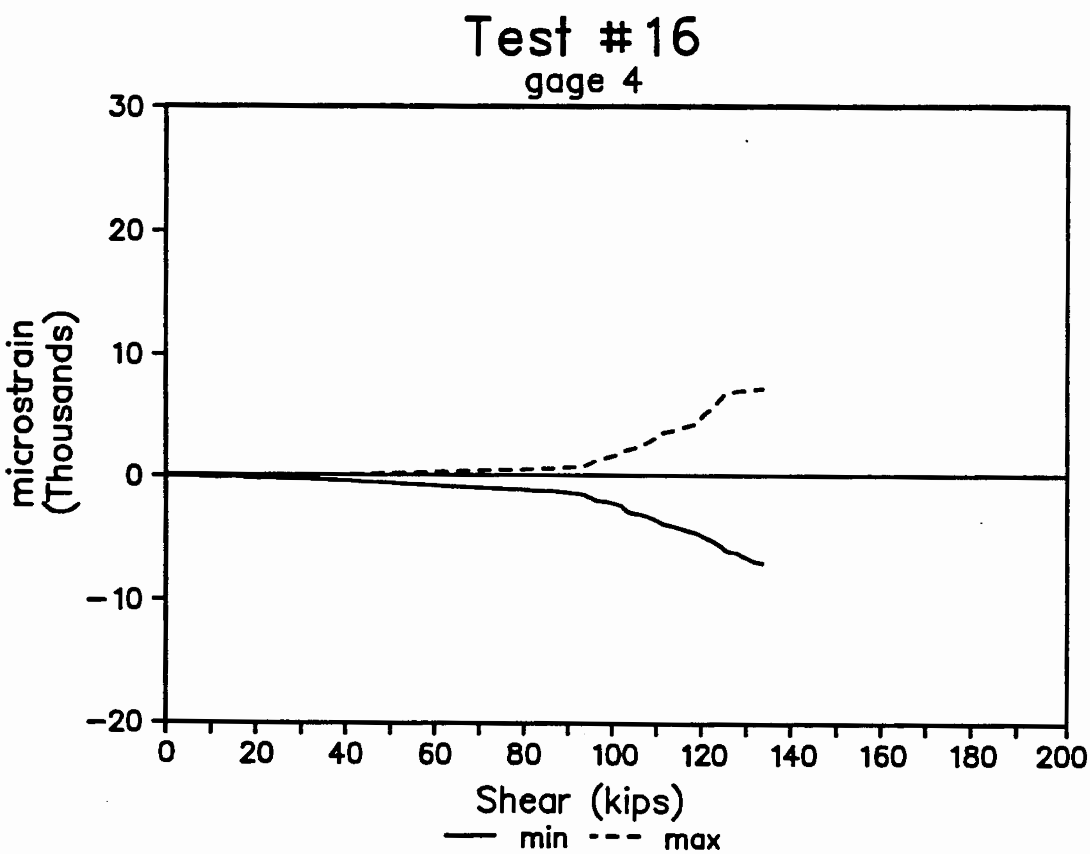

Test #15
gage 4
microstrain (Thousands)
30
20
10
0
−10
−20
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
min
max

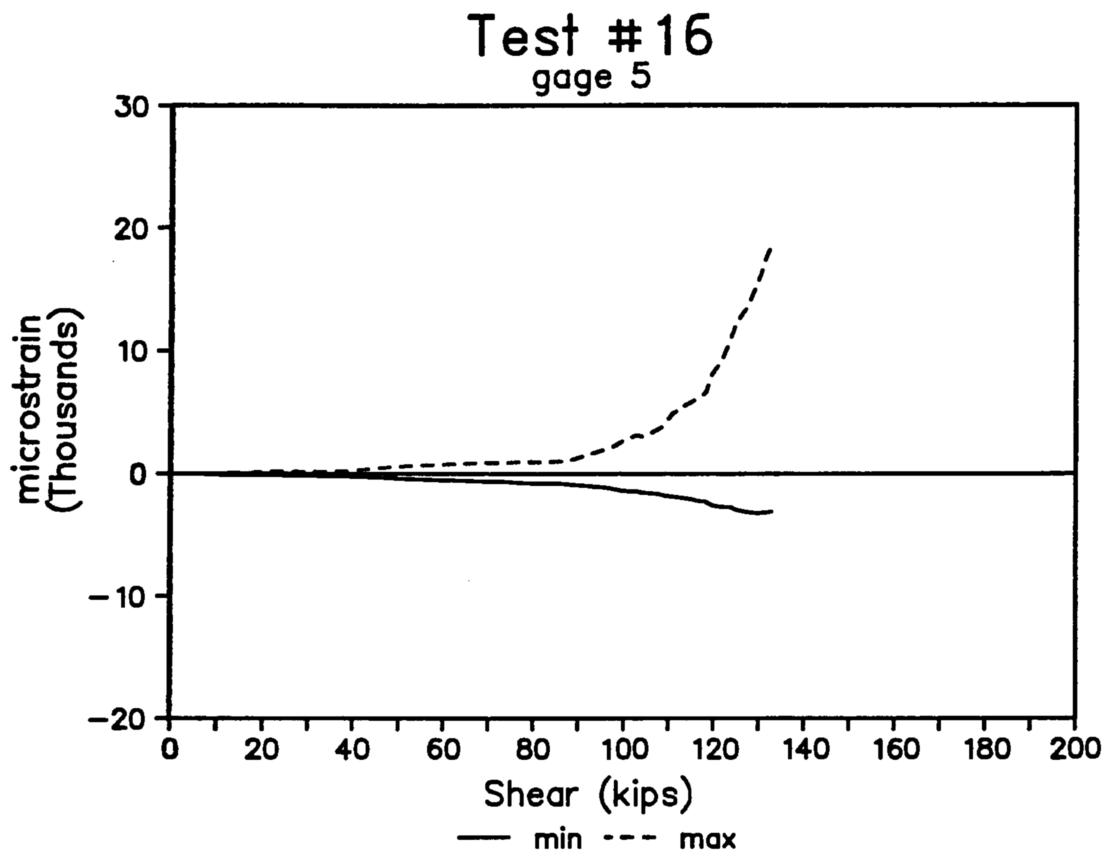

Test #15
gage 5
microstrain (Thousands)
30
20
10
0
−10
−20
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
min
max

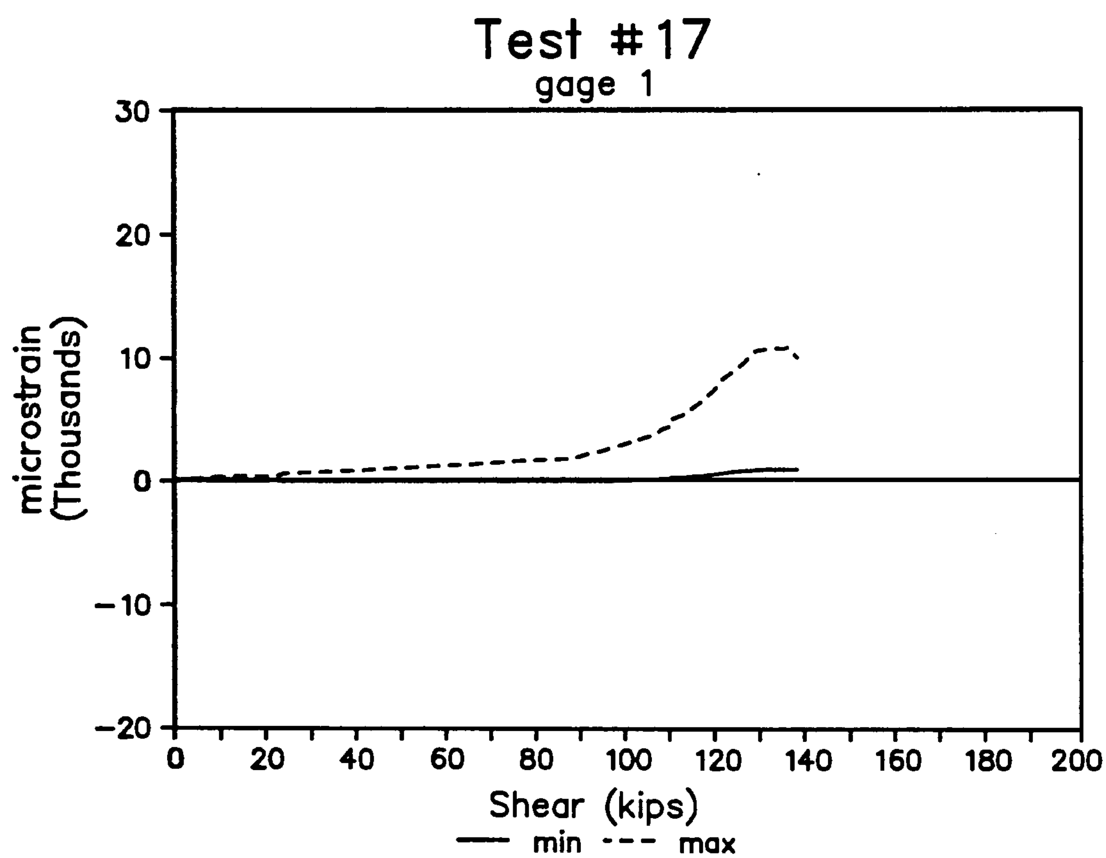

Test #16
gage 1
30
20
10
0
-10
-20
microstrain (Thousands)
0 20 40 60 80 100 120 140 160 180 200
Shear (kips)
min
max

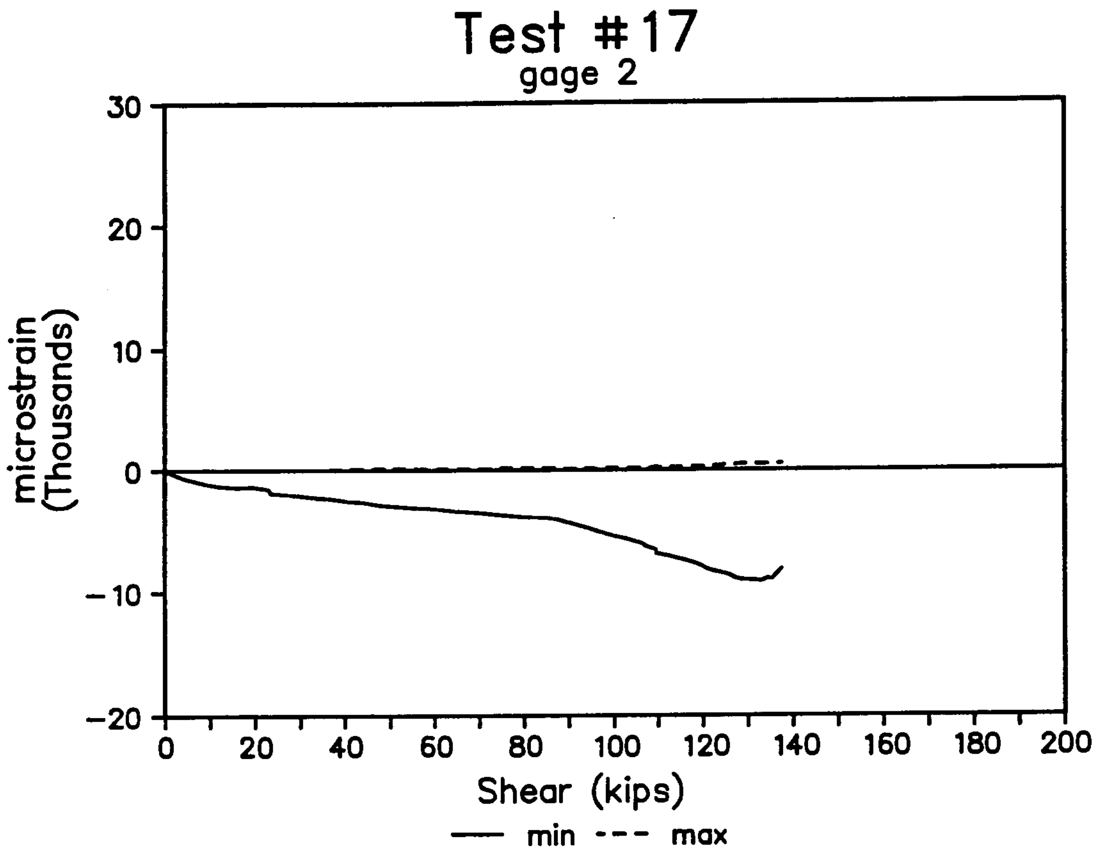

Test #16
gage 2
30
20
10
0
-10
-20
microstrain (Thousands)
Gage Damaged —
No Min/Max Calculated
0 20 40 60 80 100 120 140 160 180 200
Shear (kips)
min
max

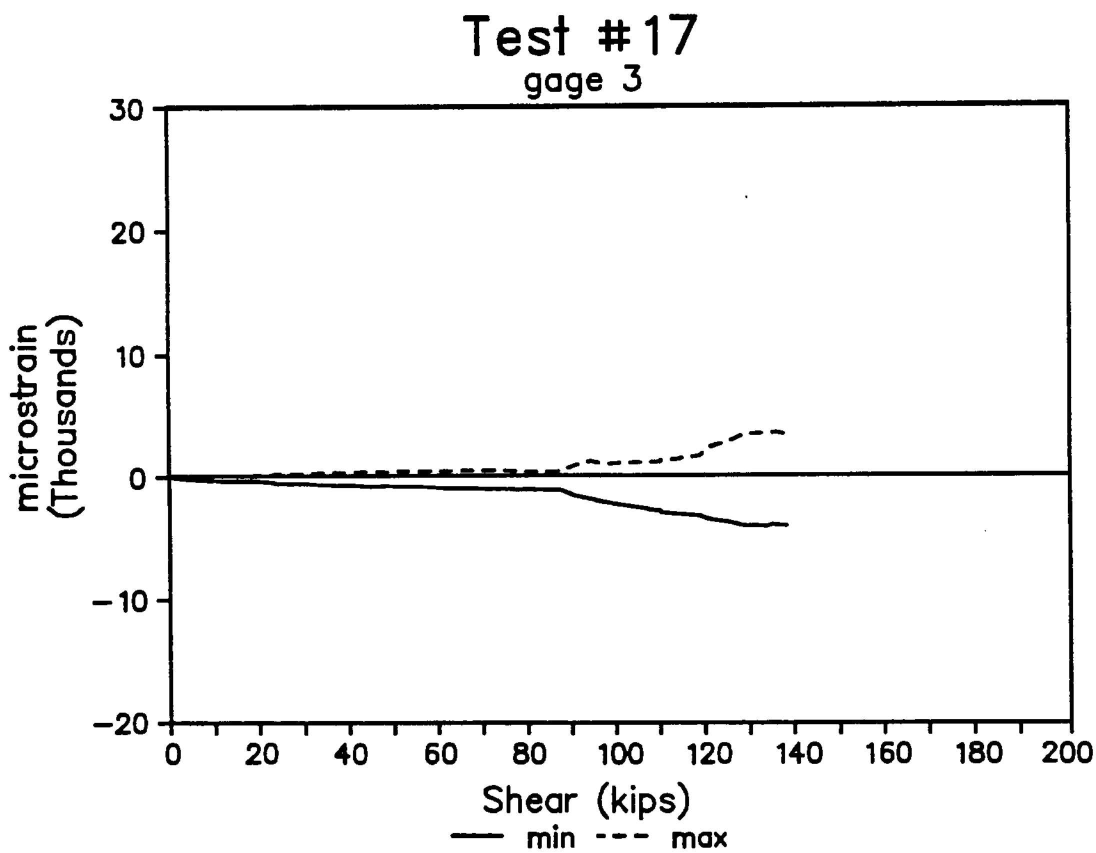

Test #16
gage 3
microstrain
(Thousands)
30
20
10
0
−10
−20
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
min
max

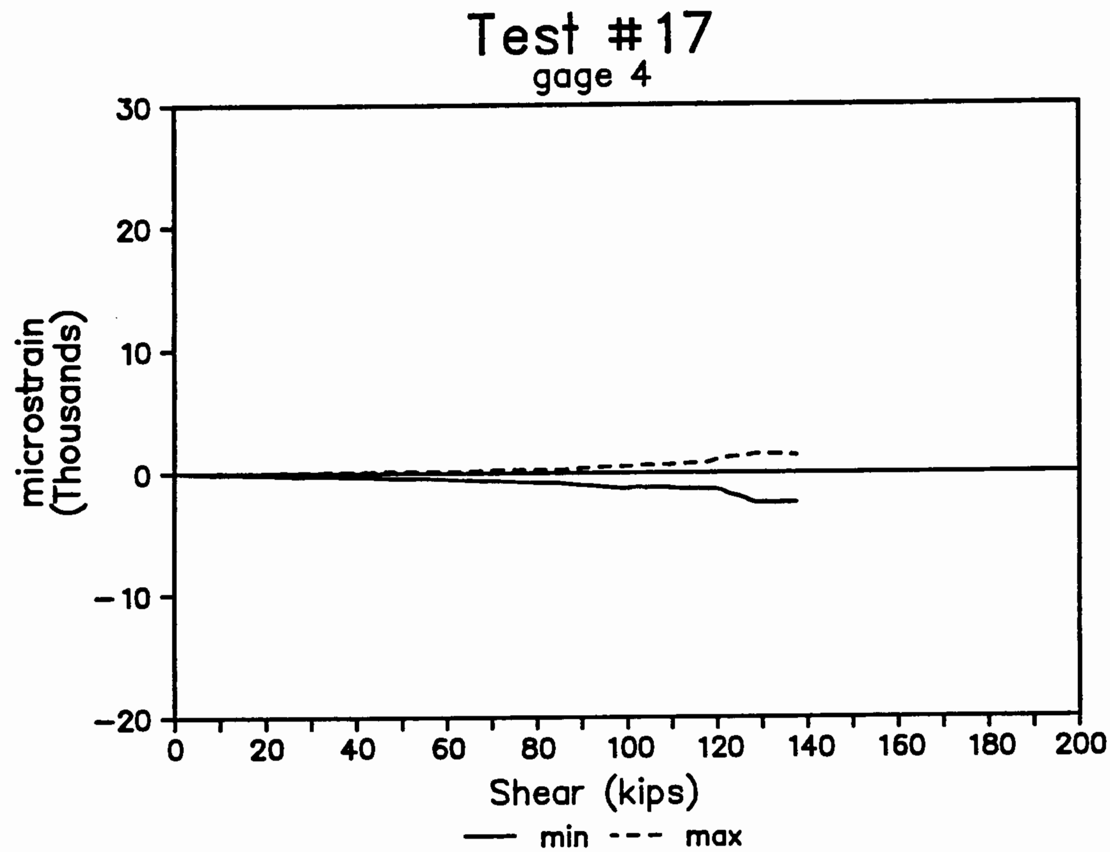

Test #16
gage 4
microstrain
(Thousands)
30
20
10
0
−10
−20
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
min
max

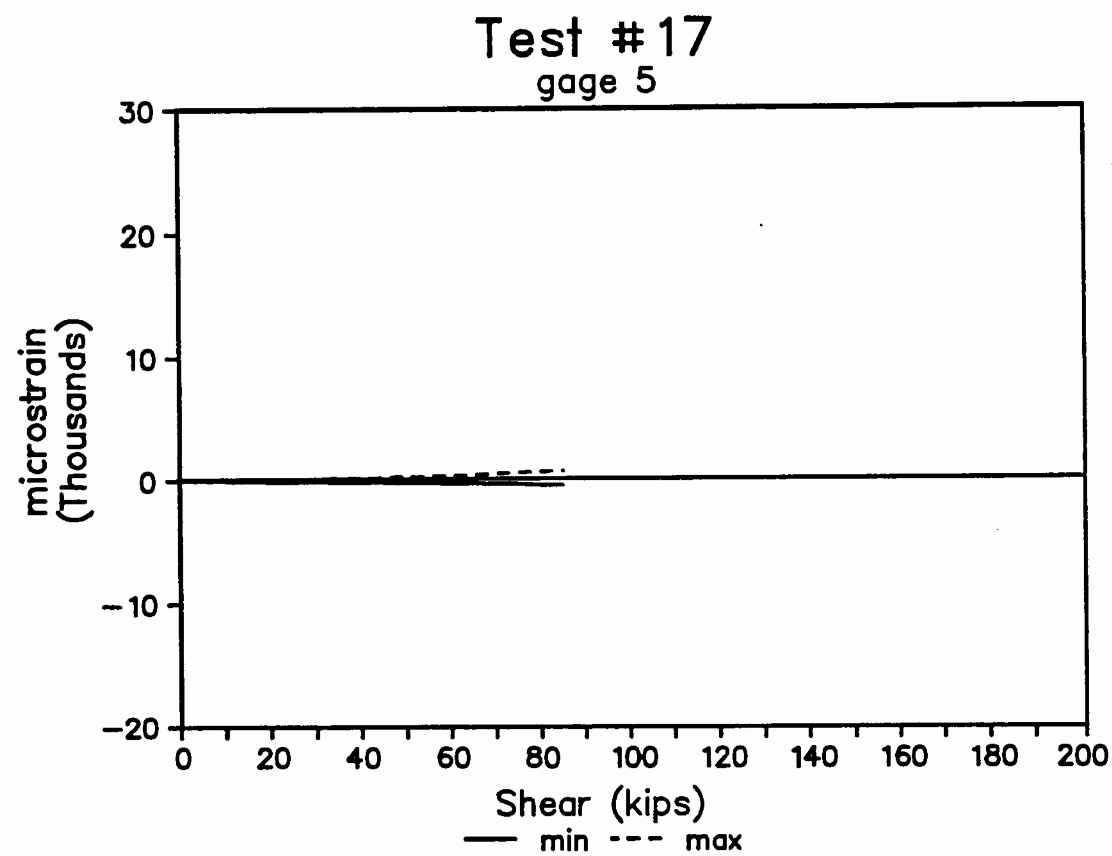

Test #16
gage 5
microstrain
(Thousands)
30
20
10
0
−10
−20
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
min
max


Test #17
gage 1
microstrain
(Thousands)
30
20
10
0
−10
−20
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
min
max


Test #17
gage 2
microstrain (Thousands)
30
20
10
0
-10
-20
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
min
max


Test #17
gage 3
microstrain (Thousands)
30
20
10
0
-10
-20
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
min
max


Test #17
gage 4
microstrain (Thousands)
30
20
10
0
−10
−20
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
min
max


Test #17
gage 5
microstrain (Thousands)
30
20
10
0
−10
−20
0
20
40
60
80
100
120
140
160
180
200
Shear (kips)
min
max

PROJECT ON

AISI SINGLE PLATE FRAMING CONNECTION

SUMMARY OF TEST ON SPECIMEN NO. 12

OBJECTIVE: To study actual behavior of single plate framing connection used to transfer shear from a beam to a girder.

TEST DATE: 3/4/91

CONDUCTED BY: A. Shaw, R. Stephen, and A. Astaneh-Asl

LABORATORY: 200 Davis Hall, University of California, Berkeley

PROPERTIES OF TEST

PLATE DEPTH: 12 in. PL. WIDTH: 4.5 in. PL. THICKNESS: 3/8 in.

PLATE F_y: 34 ksi PL. F_u: 58 ksi PL. MATERIAL: A36

NUMBER OF BOLTS: 4 BOLT DIAM: 3/4 in. BOLT TYPE: A490N

HOLE DIAM: 13/16 in. EDGE DIST: 1.5 in. HOLE TYPE: Standard

WELD SIZE: 9/32 in. WELD LENGTH: 12 in. WELD ELECTRODE: E70XX

EXPECTED SHEAR CAPACITY R_y: 120 kips GIRDER SECTION: W24x146

BEAM SECTION: W16x50

TEST RESULTS:

MAXIMUM SHEAR: 193 kips AT ROTATION: .103 rad.

FAILURE MODE: Weld Fracture

GENERAL COMMENTS AND DISCUSSION:

- Yielding in girder web as early as 60k shear, even though thick girder web (0.650 in.).

- Weld crack first at 127k and 0.068 rad rotation.

- Doubled load/rotation slope at 0.08 rotation to account for beam plasticity.

PROJECT ON

AISI SINGLE PLATE FRAMING CONNECTION

SUMMARY OF TEST ON SPECIMEN NO. 13

OBJECTIVE: To study actual behavior of single plate framing connection used to transfer shear from a beam to a girder.

TEST DATE: 3/27/91

CONDUCTED BY: A. Shaw, R. Stephen, and A. Astaneh-Asl

LABORATORY: 200 Davis Hall, University of California, Berkeley

PROPERTIES OF TEST

PLATE DEPTH: 18 in. PL. WIDTH: 4.5 in. PL. THICKNESS: 3/8 in.

PLATE F_y: 34 ksi PL. F_u: 58 ksi PL. MATERIAL: A36

NUMBER OF BOLTS: 6 BOLT DIAM: 3/4 in. BOLT TYPE: A490N

HOLE DIAM: 13/16 in. EDGE DIST: 1.5 in. HOLE TYPE: Standard

WELD SIZE: 9/32 in. WELD LENGTH: 18 in. WELD ELECTRODE: E70XX

EXPECTED SHEAR CAPACITY R_y: 180 kips GIRDER SECTION: W24x146

BEAM SECTION: W21x50

TEST RESULTS:

MAXIMUM SHEAR: 148 kips AT ROTATION: .029 rad.

FAILURE MODE: Weld Fracture (note: poor weld quality)

GENERAL COMMENTS AND DISCUSSION:

- Weld crack very early, around 100k shear. It became evident early on that the weld quality was poor and therefore cracked earlier than expected.

- Examining weld shows poor penetration into base metal.

PROJECT ON

AISI SINGLE PLATE FRAMING CONNECTION

SUMMARY OF TEST ON SPECIMEN NO. 14

OBJECTIVE: To study actual behavior of single plate framing connection used to transfer shear from a beam to a girder.

TEST DATE: 4/12/91

CONDUCTED BY: A. Shaw, R. Stephen, and A. Astaneh-Asl

LABORATORY: 200 Davis Hall, University of California, Berkeley

PROPERTIES OF TEST

PLATE DEPTH: 12 in. PL. WIDTH: 4.5 in. PL. THICKNESS: 3/8 in.

PLATE F_y: 34 ksi PL. F_u: 58 ksi PL. MATERIAL: A36

NUMBER OF BOLTS: 4 BOLT DIAM: 3/4 in. BOLT TYPE: A490N

HOLE DIAM: 13/16 in. EDGE DIST: 1.5 in. HOLE TYPE: Standard

WELD SIZE: 9/32 in. WELD LENGTH: 12 in. WELD ELECTRODE: E70XX

EXPECTED SHEAR CAPACITY R_y: 120 kips GIRDER SECTION: W18x35

BEAM SECTION: W16x50

TEST RESULTS:

MAXIMUM SHEAR: 122 kips AT ROTATION: .062 rad.

FAILURE MODE: Weld fracture; Beam flange ran into girder flange.

GENERAL COMMENTS AND DISCUSSION:

- Bolt slip at 35k and again at 53k.
- Some yielding of girder web at 88k, 0.02 rad.
- By 92k, S-shape deformation has max relative deflection of 1/4 in.

- At 111k weld crack appears.
- Very noticeable S-shape in failed specimen.
- Large deflections in girder web led to twisting in beam.

PROJECT ON

AISI SINGLE PLATE FRAMING CONNECTION

SUMMARY OF TEST ON SPECIMEN NO. 15

OBJECTIVE: To study actual behavior of single plate framing connection used to transfer shear from a beam to a girder.

TEST DATE: 5/30/91

CONDUCTED BY: A. Shaw, R. Stephen, and A. Astaneh-Asl

LABORATORY: 200 Davis Hall, University of California, Berkeley

PROPERTIES OF TEST

PLATE DEPTH: 18 in. PL. WIDTH: 4.5 in. PL. THICKNESS: 3/8 in.

PLATE F_y: 34 ksi PL. F_u: 58 ksi PL. MATERIAL: A36

NUMBER OF BOLTS: 6 BOLT DIAM: 3/4 in. BOLT TYPE: A490N

HOLE DIAM: 13/16 in. EDGE DIST: 1.5 in. HOLE TYPE: Standard

WELD SIZE: 9/32 in. WELD LENGTH: 18 in. WELD ELECTRODE: E70XX

EXPECTED SHEAR CAPACITY R_y: 180 kips GIRDER SECTION: W24x55

BEAM SECTION: W21x50

TEST RESULTS:

MAXIMUM SHEAR: 180 kips AT ROTATION: .058 rad.

FAILURE MODE: Weld fracture; Lateral-torsional buckling of beam.

GENERAL COMMENTS AND DISCUSSION:

- Begin yielding (paint chipping) directly under shear tab at 125k.

- Begin weld fracture at 150k.

- By 175k, weld advanced to 1/4 in.; yielding in cope region of beam.

PROJECT ON

AISI SINGLE PLATE FRAMING CONNECTION

SUMMARY OF TEST ON SPECIMEN NO. 16

OBJECTIVE: To study actual behavior of single plate framing connection used to transfer shear from a beam to a girder.

TEST DATE: 6/5/91

CONDUCTED BY: A. Shaw, R. Stephen, and A. Astaneh-Asl

LABORATORY: 200 Davis Hall, University of California, Berkeley

PROPERTIES OF TEST

PLATE DEPTH: 12 in. PL. WIDTH: 4.5 in. PL. THICKNESS: 3/8 in.

PLATE F_y: 34 ksi PL. F_u: 58 ksi PL. MATERIAL: A36

NUMBER OF BOLTS: 4 BOLT DIAM: 3/4 in. BOLT TYPE: A490N

HOLE DIAM: 13/16 in. EDGE DIST: 1.5 in. HOLE TYPE: Standard

WELD SIZE: 9/32 in. WELD LENGTH: 12 in. WELD ELECTRODE: E70XX

EXPECTED SHEAR CAPACITY R_y: 120 kips GIRDER SECTION: 24x55

BEAM SECTION: W16x50

TEST RESULTS:

MAXIMUM SHEAR: 133 kips AT ROTATION: 0.078 rad.

FAILURE MODE: Bolt fracture; Lateral-torsional buckling of beam; weld fracture; Severe yielding of girder web.

GENERAL COMMENTS AND DISCUSSION:

- Bolt slip at 40 kips load.
- Beam begins to twist at 62 kips.
- Weld begins to crack around 118 kips.

PROJECT ON

AISI SINGLE PLATE FRAMING CONNECTION

SUMMARY OF TEST ON SPECIMEN NO. 17

OBJECTIVE: To study actual behavior of single plate framing connection used to transfer shear from a beam to a girder.

TEST DATE: 6/11/91

CONDUCTED BY: A. Shaw, R. Stephen, and A. Astaneh-Asl

LABORATORY: 200 Davis Hall, University of California, Berkeley

PROPERTIES OF TEST

PLATE DEPTH: 12 in. PL. WIDTH: 4.5 in. PL. THICKNESS: 3/8 in.

PLATE F_y: 34 ksi PL. F_u: 58 ksi PL. MATERIAL: A36

NUMBER OF BOLTS: 4 BOLT DIAM: 3/4 in. BOLT TYPE: A490N

HOLE DIAM: 13/16 in. EDGE DIST: 1.5 in. HOLE TYPE: Standard

WELD SIZE: 9/32 in. WELD LENGTH: 12 in. WELD ELECTRODE: E70XX

EXPECTED SHEAR CAPACITY R_y: 120 kips GIRDER SECTION: 18x97

BEAM SECTION: W16x50

TEST RESULTS:

MAXIMUM SHEAR: 138 kips AT ROTATION: 0.079 rad.

FAILURE MODE: Beam flange hit girder flange; girder web yielding; bolt twisting.

GENERAL COMMENTS AND DISCUSSION:

- Bolt slip at 23 kips load.
- Begin to notice yielding (i.e. paint chipping) at 84 kips.
- Noticeable bolt twisting by 121 kips.