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# TESTS OF BOLTED SHEAR WEB CONNECTIONS

by

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Sponsored by
American Institute of Steel Construction

DEPARTMENT OF CIVIL ENGINEERING / Phil M. Ferguson Structural Engineering Laboratory
THE UNIVERSITY OF TEXAS AT AUSTIN

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This work has been carried out as a part of an investigation sponsored by the American Institute of Steel Construction

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A B S T R A C T

Fifteen full-scale tests of single-row bolted shear web connections were conducted. Both coped and uncoped beam ends with 3- and 5-bolt holes (standard and slotted) distributed over the depth of the beam were included in the program. A variety of failure modes were observed from which several design recommendations were developed that predict more closely the actual mode of failure and the approximate failure load.

Comparisons of the test results to the 1978 AISC Specifications suggested that the application of the formulas for edge and end distance and for bearing required some revisions. Recommendations are made for their use, and further, revisions for the block-shear formula are suggested. Design recommendations for slotted-hole connections are presented.

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# NOTATION

| | |
|---|---|
| $A_b$ | area of bolt |
| $A_{t_{net}}$ | net tensile area through the centerline of the bolt hole(s) |
| $A_v$ | gross shear area of the beam web (at cope if coped, net-shear area if slotted holes) |
| $A_{v_{dev}}$ | gross shear area of web below bottom bolt centerline |
| $A_{v_{grs}}$ | gross shear area of web above bolt centerline |
| $A_{v_{net}}$ | net shear area of web above bottom bolt centerline |
| $B_b$ | allowable block shear |
| $d$ | depth of beam |
| $d_c$ | depth of cope |
| $E_r$ | eccentricity reduction factor |
| $e$ | end eccentricity |
| $e_c$ | edge distance to cope |
| $e_e$ | end distance, edge row of bolts to end of beam |
| $F_p$ | allowable bearing stress |
| $F_u$ | ultimate yield stress of steel |
| $F_v$ | allowable web shear stress |
| $F_y$ | yield stress of steel |
| $N_v$ | allowable net shear |
| $P$ | force transmitted by one fastener (AISC edge distance formula) |
| $P_r$ | force transmitted by one fastener (AISC end distance formula) |
| $R$ | connection reaction |

$R_u$ ultimate reaction load

$S_r$ reduction for slotted holes or double-row connections

$T_e$ tensile capacity of net end section

$t_w$ average thickness of beam web

$V_d$ shear capacity of web below bottom bolt centerline

$V_u$ theoretical web shear yield

CHAPTER 1

# INTRODUCTION

The American Institute of Steel Construction (AISC), utilizing criteria developed from tests performed at the University of Toronto,[12] modified their method for the design of bolted beam web connections, as published in the 1978 Edition of the Specifications.[15] They incorporated a strength check for holes placed close to the end or edge of a beam and a "block-shear" failure allowable which involves both tensile and shear fracture paths. These design recommendations were based on very limited data, so a test program was undertaken to study the behavior of bolted beam web shear connections which is reported herein. Before presenting the current design methods for the areas of interest in this paper, some background material will be presented in developing the topic.

Studies of bolted connections were one of the first areas undertaken by the newly formed Research Council on Riveted and Bolted Structural Joints (RCRBSJ) over thirty years ago. The effects of bearing pressures on the strength of riveted joints were studied by the task committee then formed. Utilizing material presented by Jones[7] and Munse,[10] which stated that providing the calculated bearing stress does not exceed 2.25 times the calculated tensile stress, the tensile efficiency will not be reduced as a result of the bearing stress, coupled with the accepted net section allowable stress of 0.6 times the yield stress ($F_y$), an allowable bearing stress of $1.35F_y$ was adopted. (By simple proportions; $1.35 = 0.6 \times 2.25$.) This remained unchanged in North American practice until 1974 when the Canadian Standards Association S16.1--Steel Structures for Buildings--Limit States Design[17] was issued and later the RCRBSJ--1976 Edition of the Specifications for Structural Joints Using ASTM A325 or A490 Bolts[16] in the United States.

The 1976 RCRBSJ Specifications, as the Canadian Standards had done earlier, incorporated the recommendations of Fisher and Struik.[4] These recommendations were based substantially on the findings on tension splice bolted connections by Struik and Wittermans at Delft University of Technology as reported by de Back and de Jong.[3] The allowable bearing stress ($F_p$), as given in the 1976 Edition of the Specifications, is taken as the lesser of

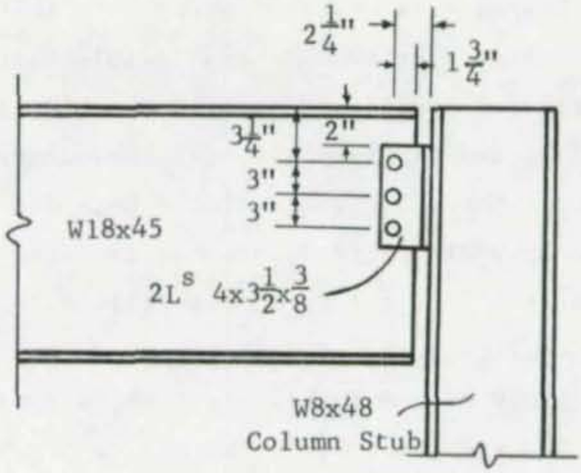
$$\frac{L \cdot F_u}{2d} \tag{1.1}$$

and

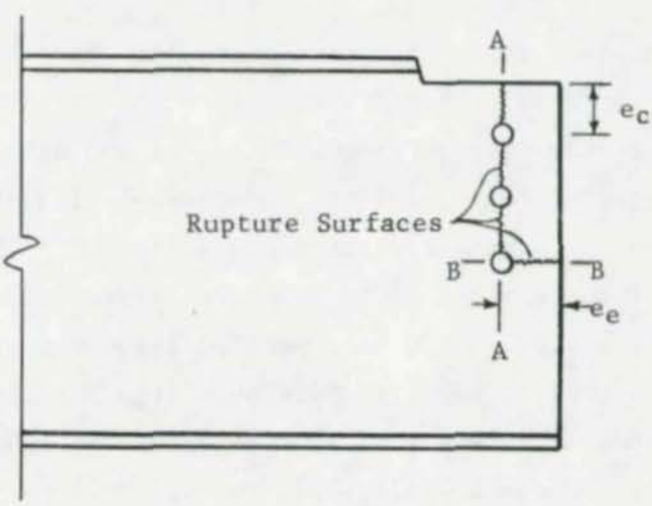
$$1.5F_u, \tag{1.2}$$

where $F_u$ is the specified minimum tensile strength of the connected material, d is the bolt diameter and L is the edge distance (distance from the hole centerline to the edge of the plate in the direction of the force transmitted by the fastener). A factor of safety of 2.0 is incorporated in Eqs. (1.1) and (1.2). Documentation of the corresponding failure modes can be found in Ref. 4. The effect of the new bearing allowable is quite significant for A36 steel with an increase of about 80 percent from the previous specification requirements. The introduction of the edge distance formula as a new requirement for strength was another major change. Previously the allowable bearing strength was independent of edge distance.

The new requirements are easily interpreted in the design of tensile splices, but not as straightforward when used in designing other connections such as common double-angle beam-web connections. Research was initiated at the University of Toronto to test the requirements. Full-scale tests of W18x45 beam ends, connected as shown in Fig. 1.1a with 3/4 in. A325 bolts, were performed. The




(a)




(b)

Fig. 1.1 University of Toronto Connection Tests research cited failure mechanisms previously not anticipated. In these tests bearing was critical under both the old and new specifications. (For these test beams the allowable bearing was 67 percent greater by the new specification over the old one.) For the uncoped beam test, depicted in Fig. 1.1a, failure occurred when the bottom bolt pushed out through the end of the beam due to angle rotation. Using the bearing allowable determined from the 1976 RCRBSJ Specifications,[15] the factor of safety at the failure load was 1.8. (By the 1969 AISC Specifications[14] the factor of safety would have been 3.0.) From observations of the failed specimen, a component of force appeared to be directed towards the end of the beam at the lowest bolt. The 1978 AISC Specifications[15] thus introduced an end distance formula similar to that for edge distance (Eq. (1.1)) in lieu of considerations of eccentricities. This requirement dictated that the maximum allowable load a fastener could develop toward the beam end using Eq. (1.1) would be applied to all of the bolts to obtain the connection allowable load.

Another connection similar to the first, but with a coped top flange (Fig 1.1b), showed a reduction in strength of 24 percent compared to the uncoped beam. This test and similar supplemental tests suggested that a failure model consisting of the simultaneous development of ultimate shear along section A-A (Fig. 1.1b) and tensile ultimate along section B-B was representative of the observed behavior. AISC incorporated these observations and recommendations in the 1978 Edition of the Specifications[15] in the form of nominal stresses on a "block-shear" element (A-A, B-B Fig. 1.1b). The allowable block shear,

$$B_v = 0.3F_u\left(A_{v_{net}}\right) + 0.5F_u\left(A_{t_{net}}\right), \qquad (1.3)$$

where $A_{v_{net}}$ is the net shear area along the rupture surface A-A and

$A_{t_{net}}$ is the net tensile area of section B-B, was presented in the AISC Commentary and further simplified to

$$B_v = 0.3F_u\left(A_{v_{net}} + A_{t_{net}}\right) \tag{1.4}$$

(which is more conservative) in the main body of the AISC Specifications (Section 1.5.1.2.2). The edge distance formula, Eq. (1.1), was applied to the vertical edge distance to the cope and to the horizontal distance to the end of the beam. The block-shear formulas, Eq. (1.3) and (1.4), were to be applied only to coped beams.

Questions regarding the validity and accuracy of the 1978 provisions[15] led to experimental research at The University of Texas, sponsored by AISC, some of which was reported in Ref. 1. The objectives of the study were to determine whether the "block-shear" model for coped beams was applicable for two lines of bolts, the applicability of edge and end distance formulas in general, and if slotted holes presented any problem. Twelve tests were conducted, as reported in Ref. 1, from which it was concluded that the 1978 provisions treated single-row connections with several bolts and a small edge or end distance conservatively. Connections with two rows of holes had less strength than anticipated. More studies were deemed necessary to develop improved design methods for bolted web connections. A second series of tests with a variety of bolt patterns was initiated at The University of Texas. These included tests of double-row connections with 4- and 5-bolt arrangements and single-row connections with 3 bolts (higher bearing stresses) at greater spacings.

In a study of the double-row connections from both of the Texas tests series,[13] important findings were made which greatly improved the overall understanding of the block-shear problem.

These tests indicated that for double-row connections, the block-shear failure was better represented by using the shear yield stress (approximately $0.6F_y$) on the gross vertical shear area ($A_{v_{grs}}$) of section A-A (Fig. 1.1b), and a triangular stress distribution on the net tensile section B-B where the maximum tensile stress would be $F_u$ at the beam end. The equation for determining the allowable block shear load for double-row connections suggested by Ref. 13 was

$$B_v = 0.3F_y\left(A_{v_{grs}}\right) + 0.25F_u\left(A_{t_{net}}\right). \tag{1.5}$$

The changes called for by this equation suggested that the shear section is not at ultimate at the occurrence of fracture of the tensile section. Also, it reduced the strength of the tensile section by one-half for double-rows. Block-shear considerations were found to apply to uncoped beams as well in the double-row connections.

In this study, the results for all of the single-row connections from both test series performed at The University of Texas will be presented and discussed. Utilizing the results of the double-row tests[13] and those of the Toronto tests,[12] a more general method for determining the block-shear allowable will be developed. Bearing problems, dealing with edge and end distances primarily, will be addressed along with numerous other items, such as slotted holes and eccentricity.

C H A P T E R   2

## TEST PROGRAM

### Connection Details

A total of fifteen tests were conducted on connections with a single row of bolts. The connection details can be seen in Fig. 2.1. The first nine connections, labeled 18-1 through 18-9, are from the first series of tests reported in Ref. 1. Connections 18-20 through 18-25 are from the second series. Tests 18-1 through 18-8 dealt with five bolts arranged in a single row where edge distances, coping of the flange and type of bolt hole (standard vs. slotted) were the major variables. Test 18-9 was a pilot test for three-bolt arrangements used in tests 18-20 through 18-25. In all the tests with three bolts, the top flange was coped in order to study edge distance ($e_c$, Fig. 1.1) requirements in the direction of loading. Other major variables were edge distance to the end of the beam (end distance, $e_e$) and bolt hole type. The one-inch edge or end distance is the minimum permitted by the AISC Specifications. The two-inch distance is close to the distance required to develop the maximum bolt bearing strength.

The connections to be tested were fabricated by the beam supplier with one connection detail at each end of a ten-foot W18x60 beam. The framing angles, also prepared by the supplier, were all from 4 in. x 4 in. x 3/8 in. size angles cut 15 in. in length. The beams were flame-cut and web holes were drilled by the fabricator so that the connections represented typical construction practice.

Beams and framing angles were specified A36 steel. Bolts were 3/4-A325 and were tightened by calibrated wrench to 325 ft.-lbs. of torque. Standard bolt holes, 13/16 in. in diameter, were specified

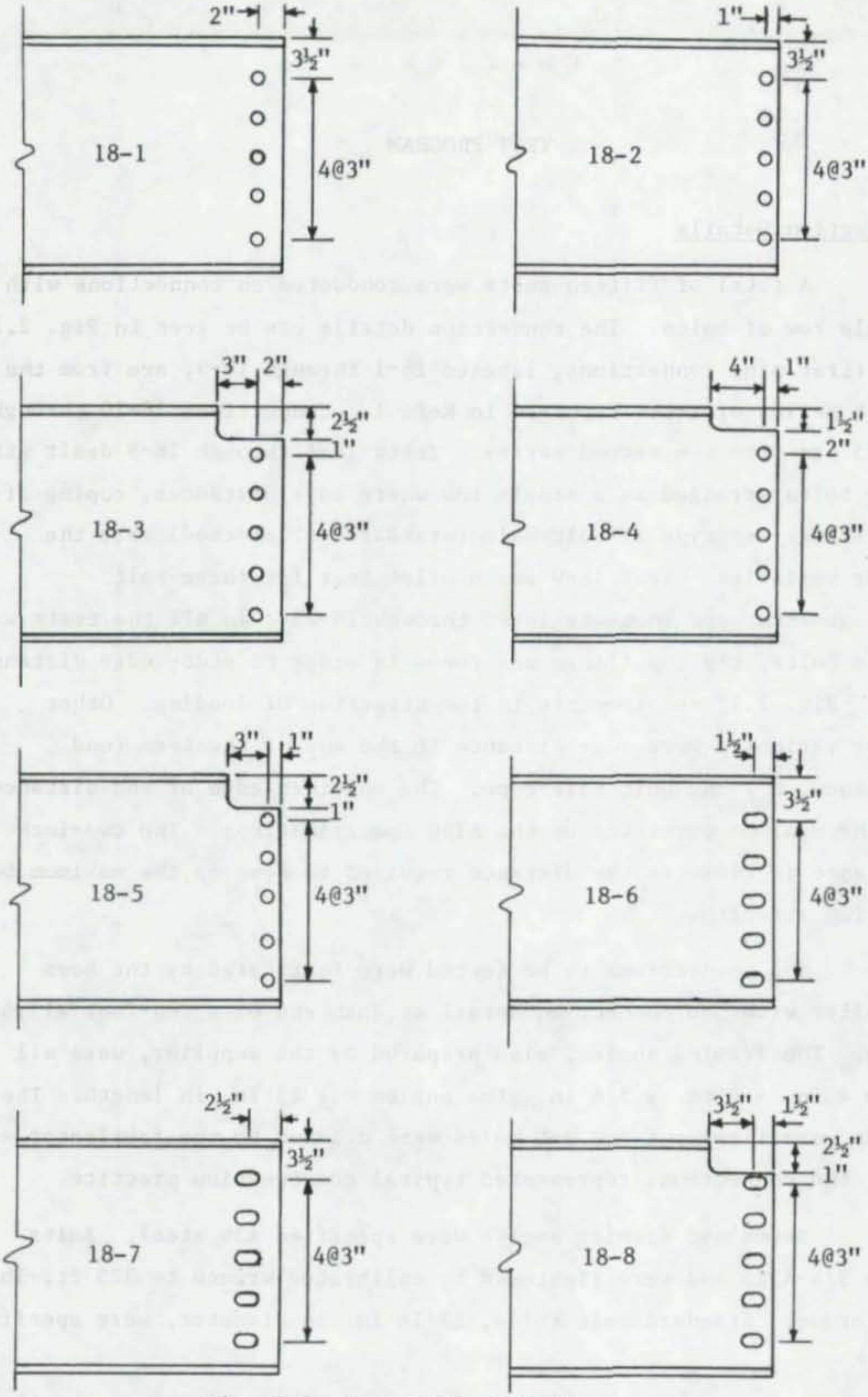



Fig. 2.1 Connection Details

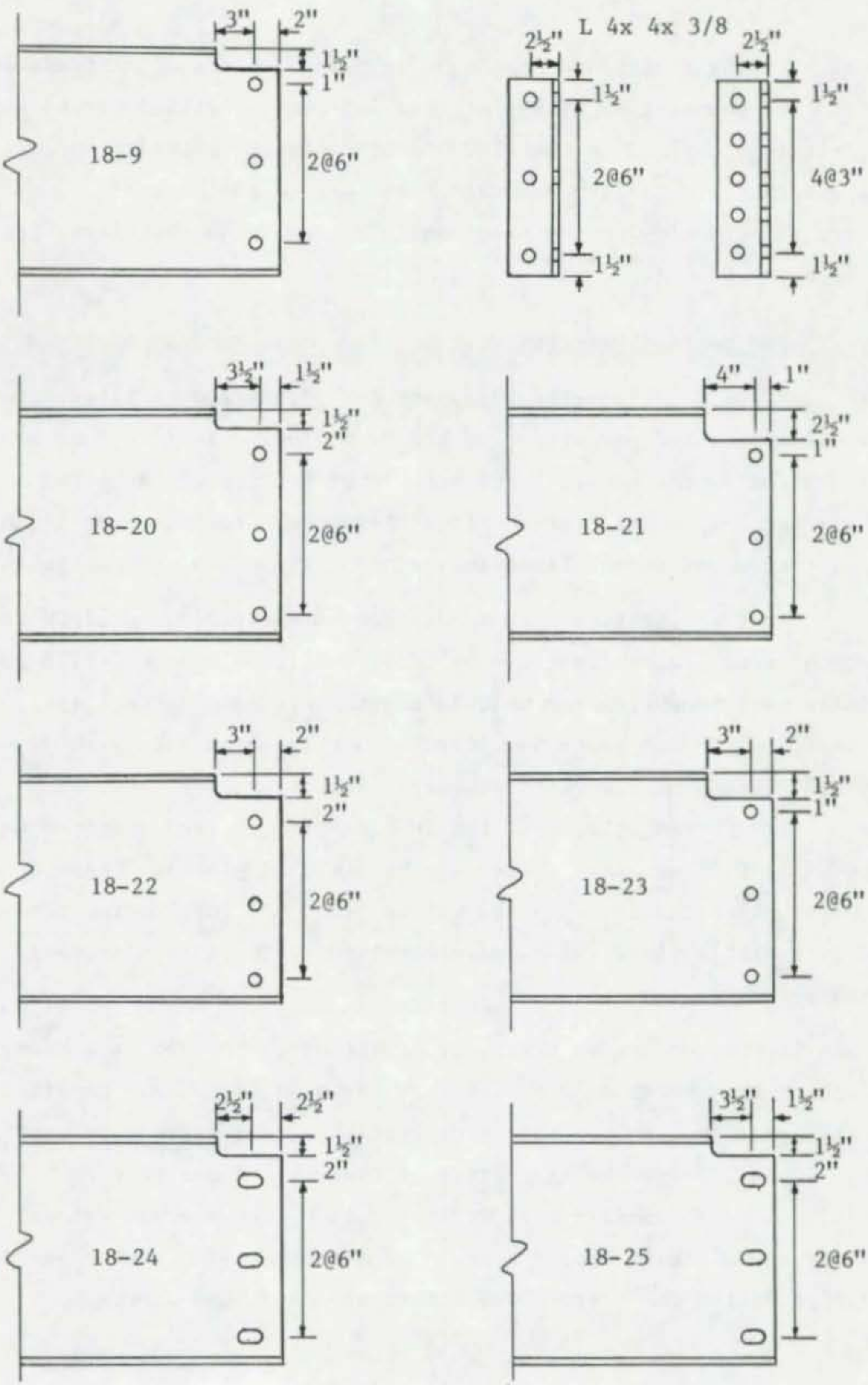

3"
2"
1½"
1"
18-9
2@6"
L 4x 4x 3/8
2½"
1½"
2@6"
1½"
2½"
1½"
4@3"
1½"
3½"
1½"
1½"
2"
18-20
2@6"
4"
1"
2½"
1"
18-21
2@6"
3"
2"
1½"
2"
18-22
2@6"
3"
2"
1½"
1"
18-23
2@6"
2½"
2½"
1½"
2"
18-24
2@6"
3½"
1½"
1½"
2"
18-25
2@6"

Fig. 2.1 (cont.)

for all framing angles and for those test connections with standard holes. The connections with slotted holes were specified to use long slots as defined in the AISC Specifications. Paired framing angles with three or five standard holes were used in each connection depending on the specimen. There were no fastener threads in the shear planes.

Material and Section Properties

All calculations regarding strength, referred to later, are made using measured properties of the beam steel. Table 2.1 is a presentation of the material and section properties of the beams used in each series of tests. Properties under headings Heat 1 and Heat 2 correspond to the first and second test series, respectively.

Shown in the table are values for static yield and ultimate strength as well as percent elongation. Static values for yield and ultimate were essential for calculating the strength of the test connections. Coupon tests were carried out in accordance with ASTM A370 test procedure with standard plate-type specimens. To obtain static strength values, the loading rate on each specimen was stopped for five minutes, thrice on the yielding plateau, and once at ultimate, at which time the load was recorded and the loading resumed. The three static yield values were averaged giving the individual values shown.

In the connection tests, deformations of the web were seen to occur both transverse to and with the length of the beam. Longitudinal and transverse coupon specimens were therefore taken from each heat. In each heat, the capacities of the web coupons in each direction were so similar they were averaged. The average values of yield and ultimate appear beneath each table. Also beneath each table are listed the average web thicknesses and beam depths.

TABLE 2.1 BEAM PROPERTIES

HEAT 1

| Coupon / Property | Tension Flange | Comp. Flange | Long. Web A | Long. Web B | Trans. Web C | Trans. Web D |
|---|---|---|---|---|---|---|
| Static Yield (ksi) | 36.7 | 36.2 | 38.2 | 38.9 | 38.4 | 37.9 |
| Static Ultimate (ksi) | 57.5 | 58.1 | 61.3 | 59.2 | 59.3 | 59.1 |
| Elongation (%) | 28.5 | 29.7 | 28.9 | 28.7 | 19.1 | 23.0 |

Avg. Web Thickness 0.44 in.

Avg. Beam Depth 18.38 in.

Avg. Web Strength (Static)
Yield - 38.3 ksi.
Ultimate - 59.7 ksi.

HEAT 2

| Coupon / Property | Tension Flange | Comp. Flange | Long. Web A | Long. Web B | Trans. Web C | Trans. Web D |
|---|---|---|---|---|---|---|
| Static Yield (ksi) | 33.5 | 34.3 | 39.5 | 34.4 | 35.6 | 36.7 |
| Static Ultimate (ksi) | 56.7 | 57.3 | 58.2 | 56.1 | 58.5 | 59.2 |
| Elongation (%) | 32.6 | 32.6 | 24.8 | 32.9 | 25.0 | 34.4 |

Avg. Web Thickness 0.43 in.

Avg. Beam Depth 18.25 in.

Avg. Web Strength (Static)
Yield - 36.6 ksi.
Ultimate - 58.0 ksi.

Test beams from the first series were of a slightly greater strength than those of the second series. For the first heat, the web static yield was 38.3 ksi and static ultimate was 59.7 ksi. For the second heat static yield was 36.6 ksi and static ultimate was 58.0 ksi.

Test Apparatus

The test apparatus shown schematically in Fig. 2.2 was designed to apply determinate forces to the test connection. The system was set up to load the beam in an inverted sense. This was done for simplicity. The loading frame consisted of two major column units bolted to a reaction slab and a crossbeam. Column spacing was great enough to allow for the ten-foot test beam, a stub-column, and some working room. A photograph of the test set-up is shown in Fig. 2.3.

The stub-column to which a test connection was bolted consisted of a W10x89 section with oversized bolt holes arranged to accommodate all of the test connections. It was heavily bolted to one of the vertical columns.

Two feet from the face of the stub-column a 200-ton compression type hydraulic ram was centered. This location was chosen to force failure at the connection and not in the beam. Channel sections were bolted to the web of the test beam at this position to restrain web crippling. The ram load was transmitted through a plate and roller assembly to the beam permitting longitudinal movement.

Two rolled sections set at right angles, one with the strong axis vertical and the other attached at its web to the flange of the first (see Fig. 2.4), comprised the crossbeam to which was mounted a calibrated load cell to serve as the far-end reaction. It was situated 8 ft.-6 in. from the stub-column face. This arrangement allowed the far-end connection to be free of any disturbance due to

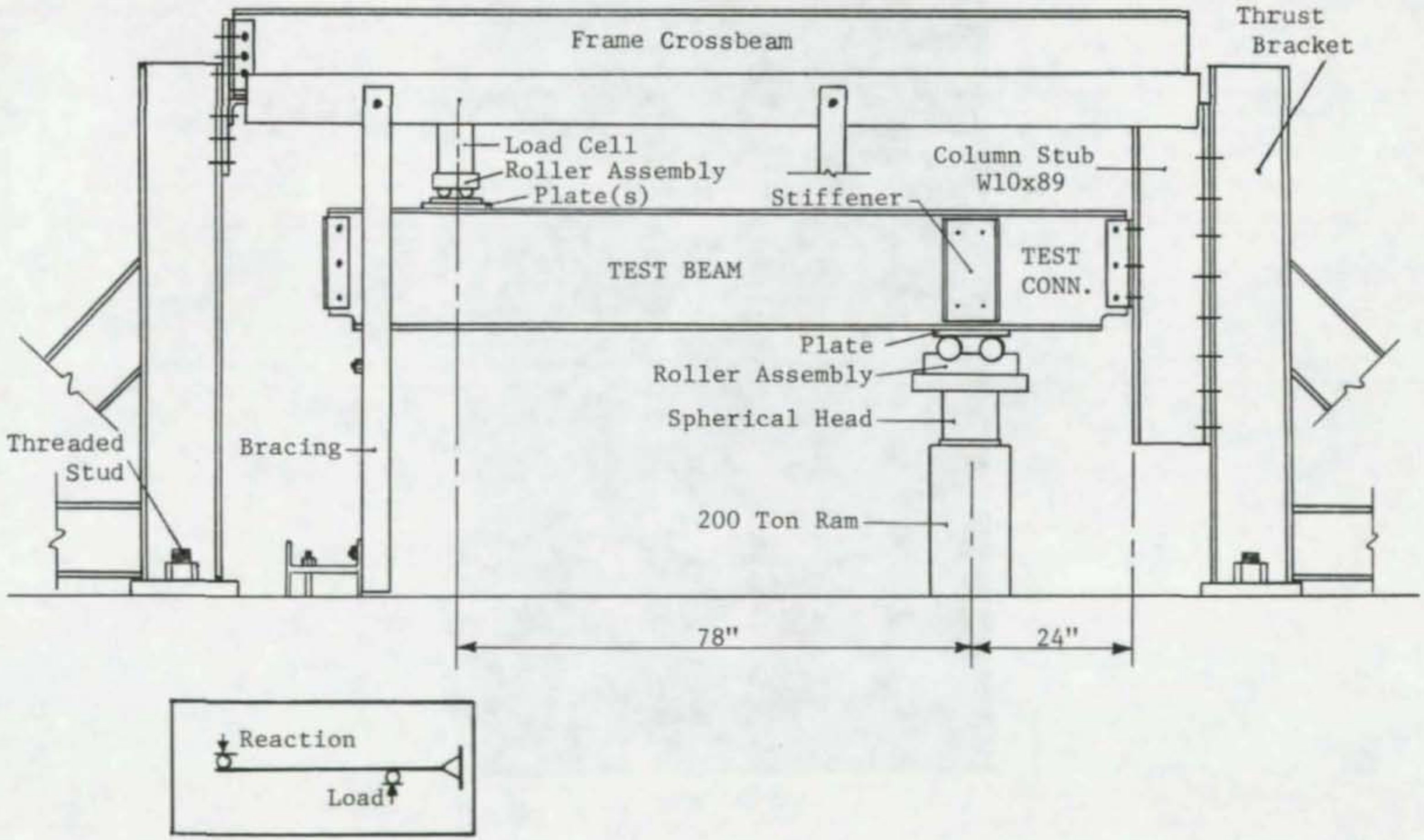



Fig. 2.2 Schematic of the Test Frame

Fig. 2.3 Test Set-Up

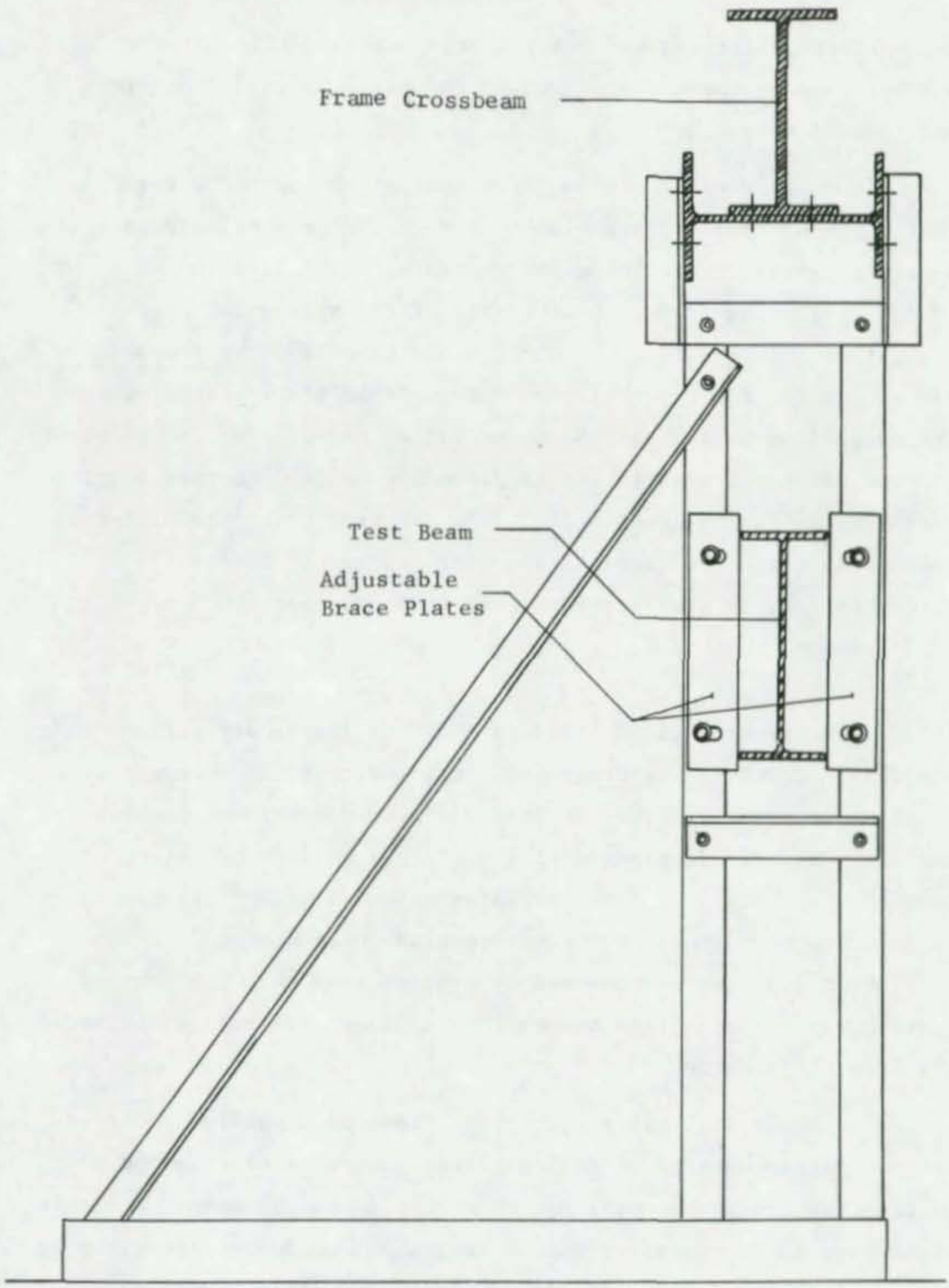



Fig. 2.4 Primary Out-of-Plane Bracing

load when the other end was being tested. The reaction at the load cell was also equipped with a roller and plate assembly to permit longitudinal movement.

Upon concluding a connection test at one end of a beam, bracing was partially dismantled to remove the beam or to rotate it to test the other end. The primary bracing was bolted to the floor and to the loading frame. A schematic of the system can be seen in Fig. 2.4. It provided an out-of-plane brace for the frame as well as bracing for the test beam. Adjustable brace plates, which were snugged up to the test beam, prevented out-of-plane movement of the beam while not restricting in-plane movements. Another bracing system, which can be seen in Fig. 2.5, was used to control lateral movement of the compression flange near the cope. In actual structures a floor slab provides this kind of support.

## Instrumentation

Measurements taken included those of loads, deflections and rotations. Certain data determined to be most critical were recorded redundantly. The ram load was recorded through pressure readings. A pressure transducer linked to a strain indicator provided the basic load data for later calculations. This reading was verified through another pressure transducer hooked up to an x-y plotter and also was checked by a pressure gage. The far-end reaction was recorded from the calibrated load cell output displayed on a strain indicator.

Numerous dial gages with graduations of 0.001 in. recorded vertical displacements. A dial gage was placed at the far-end reaction and two at the ram, one to each side of the beam. Possible twisting of the compression flange at the ram could be determined by relative differences in readings of the paired dials. Dial gages located near the test connection can be seen in Fig. 2.6. A dial

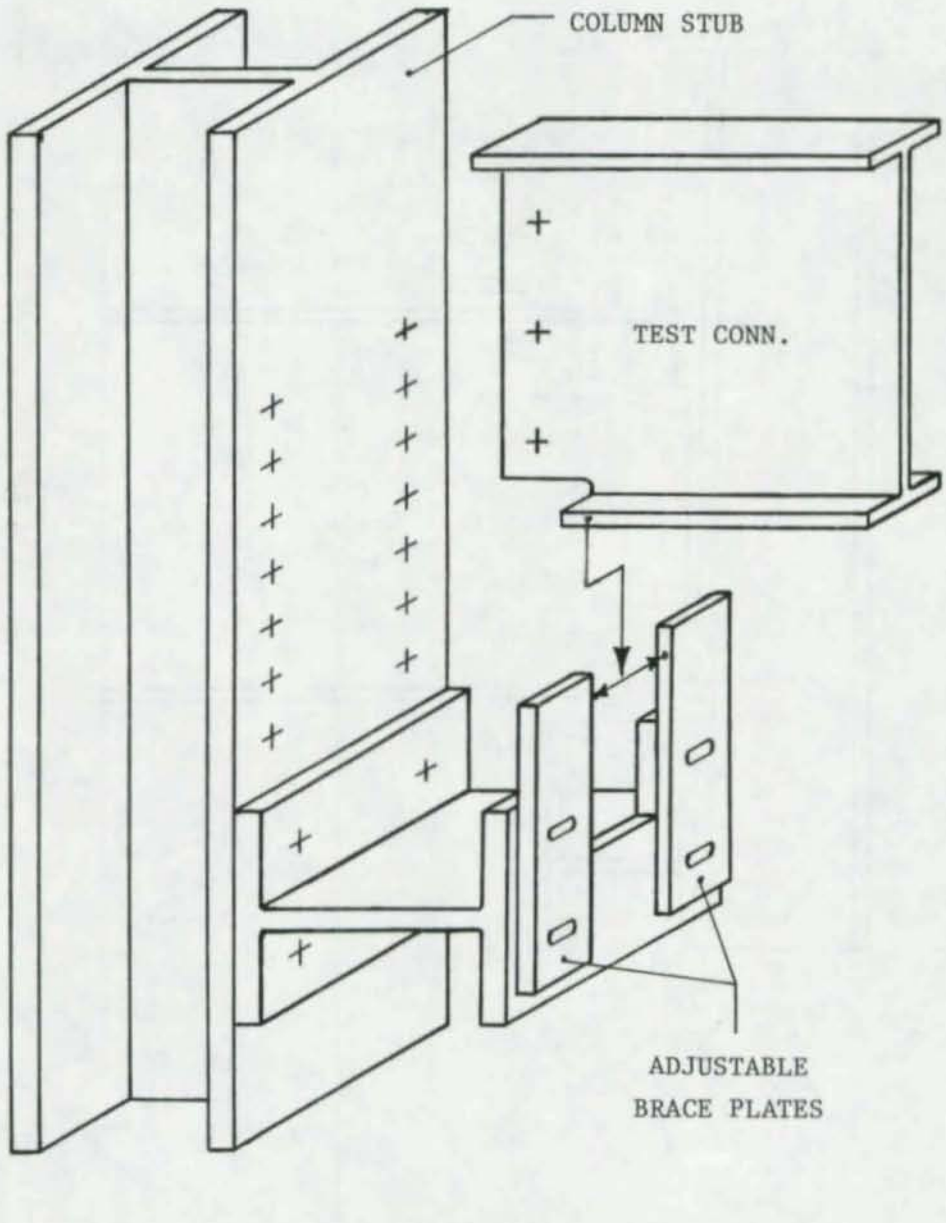



Fig. 2.5 Control of Lateral Movement at the Connection

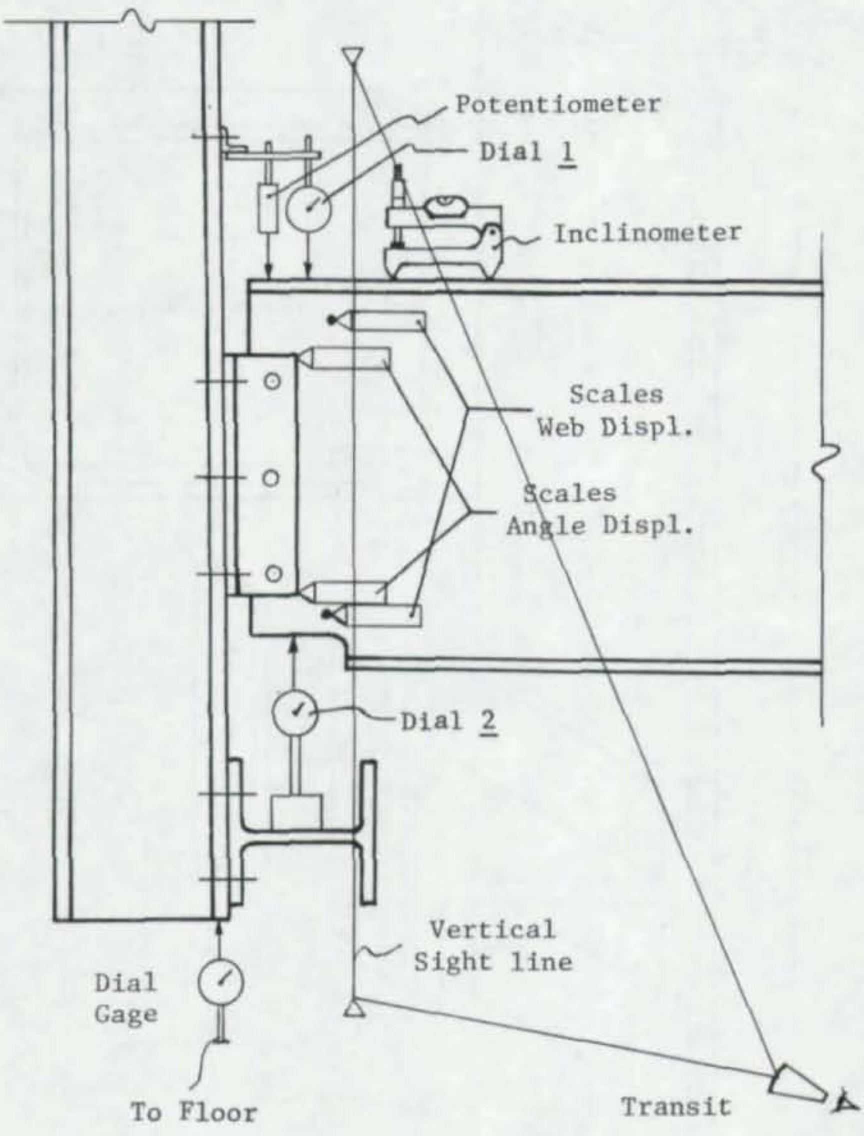



Fig 2.6 Instrumentation at the Connection

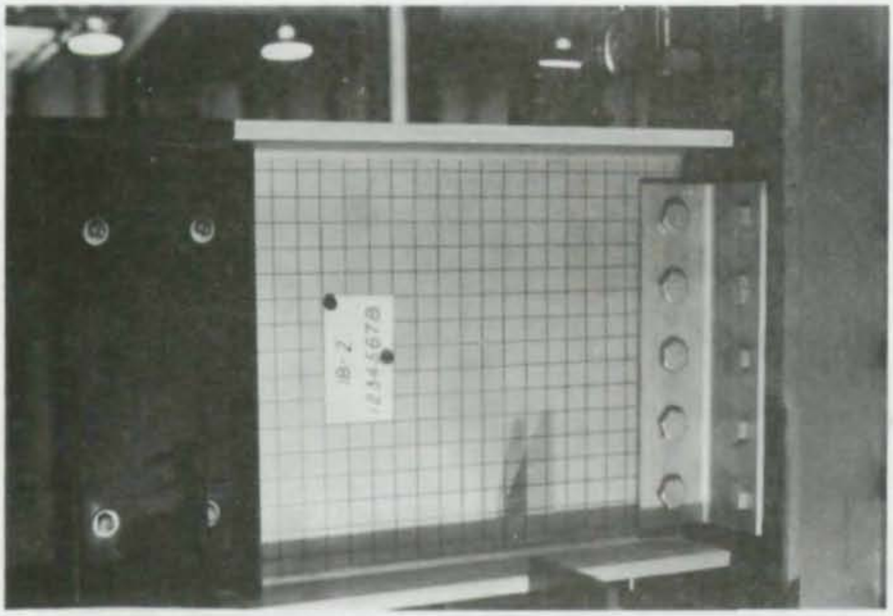
gage measured vertical displacement of the stub-column to which were attached the dials measuring connection movements. In addition to the dial gage measuring deflection of the connection on the tension flange, there was a potentiometer at this location linked to the x-y plotter monitoring the load-deflection response. On the compression flange or coped side of the connection there was also a dial gage, which coupled with the other allowed for measurement of spreading apart and fracture of the connection.

Rotations of the beam web and framing angles in the plane of the frame were measured by sighting scales through a transit along a vertical line.(see Fig. 2.6). Scale graduations were 0.02 in. Markers were glued to the web of the beam at a measured separation and marks were made on the framing angles at a set separation for positioning of the scales. Relative changes in readings of paired scales provided rotation measures. An inclinometer, measuring to 0.0001 in. over a 3-in. distance, was centered on the tension flange next to the dial gage and potentiometer.

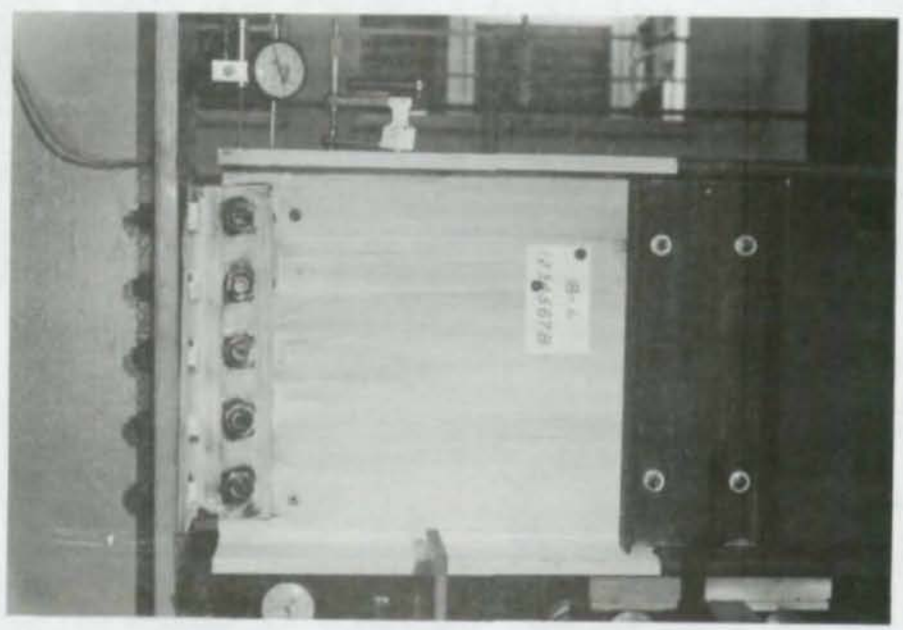
In addition to the instruments for numerical records, there were also provisions made for photographic records of the test. As seen in Fig. 2.7, one side of the connection was prepared with a one-inch square grid pattern while the other was whitewashed. The whitewash shows clearly the formation of yield lines while the grid side provided a reference for items such as the rotation of the framing angles. The grid could also be used to take measurements of deformations in local areas.

Test Procedure

The test procedures were generally repetitious for all of the tests. Pre-test preparations involved the measurement of the thickness of the beam elements and cope dimensions for each connection. Photographs and a silhouette of the beam web at the connection were taken and a grid was drawn on one side of the web. The



(a) 1 in. Square Grid Pattern



(b) Whitewash

Fig. 2.7 Pretest Preparations

silhouette, prepared by placing a sheet of paper on one side of the web and then spraying paint through the bolt holes and along the edges, provided a permanent copy of the exact arrangement of the connection. Upon bolting in the test beam, the clean side of the connection was whitewashed and instrumentation placed and checked.

Loading was done in increments. Pressure was applied by a hand operated hydraulic pump to a desired level. Increments were chosen by observing the load-deflection curve on the x-y plotter and stopping at fairly even increments with particular stops at first yield and at the maximum load. Figure 2.8 shows a typical loading curve. At each increment, after allowing a few minutes for any yielding, readings were taken. A drop in load, designated by "a," was typical as yielding occurred to achieve static equilibrium. The small horizontal line indicates the static load at each increment. An incidental drop in load below the mark occurred when further loading was initiated. Photographs, tape-recordings and written descriptions were also taken at each increment.

In one test the load was removed at certain increments and one framing angle taken off to observe the condition of the beam web beneath the cover of the angles. It is assumed that mid-test removal of the load and the angle does not affect the performance of the connection.

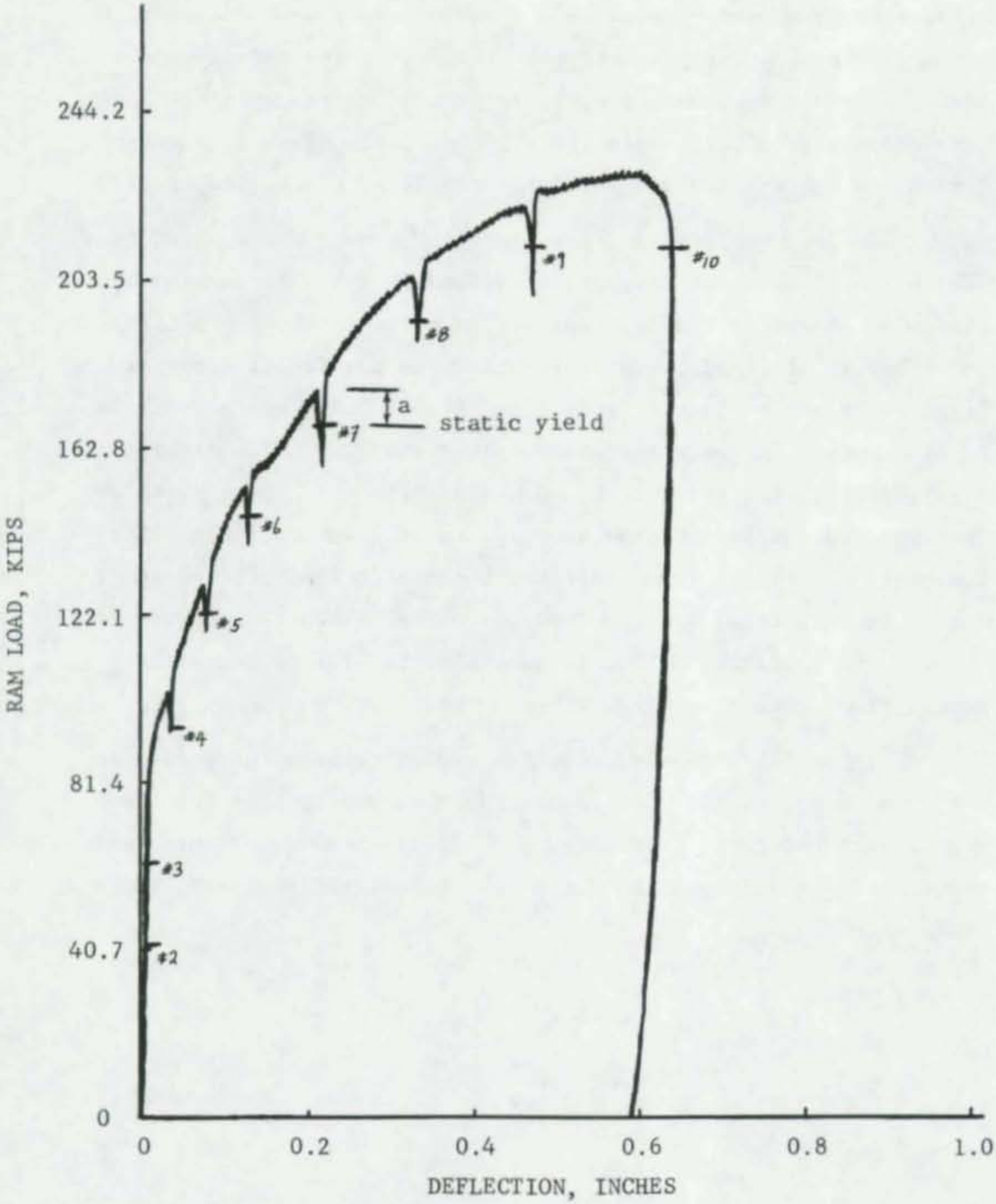



Fig. 2.8 Typical Loading Curve

C H A P T E R 3

# TEST RESULTS

## Introduction

Loads, deflections, and rotations were reduced from the recorded data. Extracted from these were the data that best represented the performance of the connection. A plot of the vertical deflection (dial 1 in Fig. 2.6) at the connection versus the reaction afforded the best description of the connection performances.

In each plot in this chapter several curves are compared. When the difference between the curves was slight, a single line was used to represent the curves until they began separating noticeably. In each plot is given the allowable reaction and the theoretical web shear yield load, $V_u = dt_wF_y/\sqrt{3}$, where d is the beam depth (in coped beams, d is taken as the available depth at the cope), $t_w$ is the web thickness, and $F_y$ is the yield stress. Start of fracture, shown on some of the curves, was determined from the relative differences between the deflection values taken at the top and bottom of the beam at the connection (dials 1 and 2 in Fig. 2.6). In the plots the absence of a major slip may be noticed. This slip was minimized by preloading the connection in the elastic range before fully tightening the bolts. The connection was therefore in bearing throughout the test.

The results of the tests will be divided into groups. The tests with five-bolt hole arrangements will be shown first, followed by the three-bolt arrangements. Photographs of the tested connections will be shown concluding the test descriptions.

Five-Bolt Connections--Uncoped

Shown in Fig. 3.1 are the reaction-deflection response curves for test connections 18-1, 18-2, 18-6 and 18-7 with uncoped beam ends. These were two standard hole connections and two slotted hole connections, respectively. In each pair of connections, there was one with large end distance and one with the minimum required. Photographs of the tested specimens are shown in Fig. 3.2.

Tests 18-1 and 18-7, both with large end distance, performed very nearly identically although test 18-7 had slotted holes. Deflections were small until the web began to yield significantly. First yield lines appeared in the vicinity of the bolt nearest the tension flange (bolt 1), propagating along the length of the beam towards the web stiffener at the load point. Yielding progressed as load was increased with vertical yield lines appearing as well as more longitudinal yield lines. The yielding pattern could be described as a growing triangular zone spreading toward the web stiffener and vertically up the web to the compression flange. Both tests were terminated when the web was thoroughly yielded and deflections at the load point were excessive. The compression flange in test 18-1 began to buckle locally near the loading ram. Ultimate reactions were 205 kips and 206 kips for tests 18-1 and 18-7, respectively. At this load the components of the connections themselves showed no particular signs of distress.

Tests 18-2 and 18-6 both had minimum end distance, but test 18-2 had standard holes while 18-6 had slots. Test 18-2 behaved similarly to test 18-1, reaching an ultimate load of 206 kips, but there was a tensile-type failure across the end distance at the hole nearest the bottom or tension flange. Test 18-6 sustained fracture similar to that of test 18-2, but reached a load of only 161 kips, a 22 percent reduction. In test 18-2, the web was extensively yielded while in test 18-6 the yielding did not propagate as much. In all tests yielding followed the same general pattern.

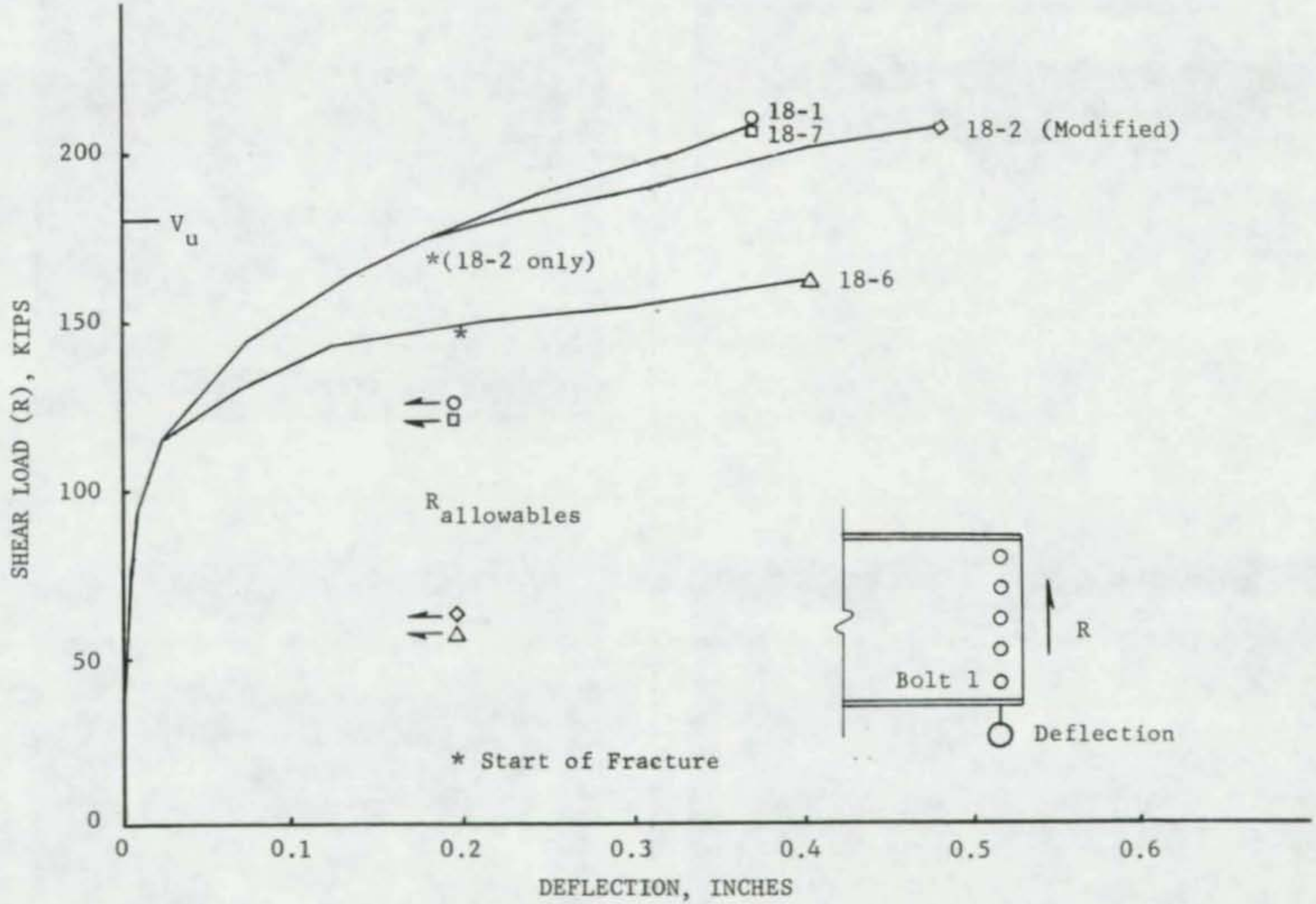
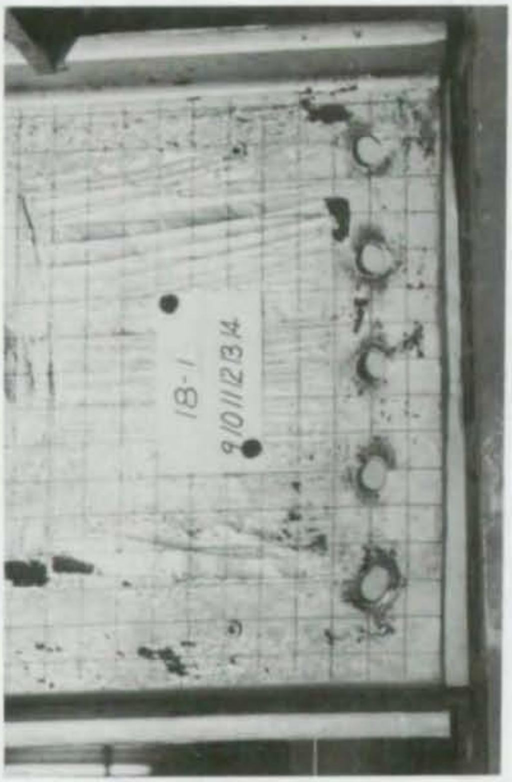
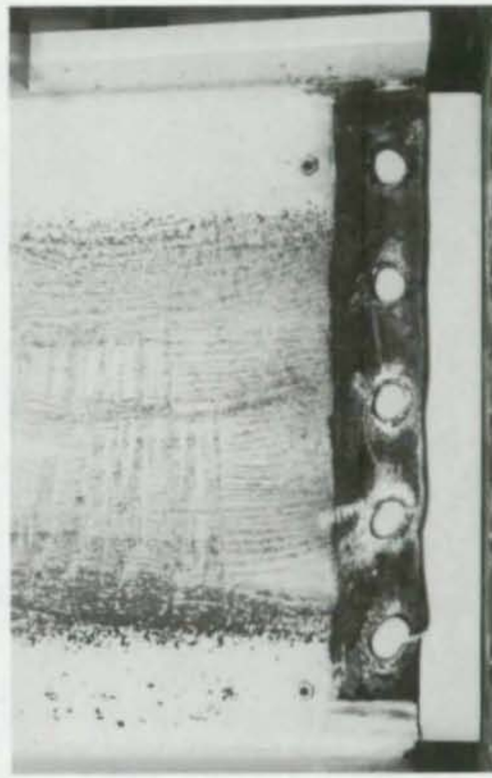


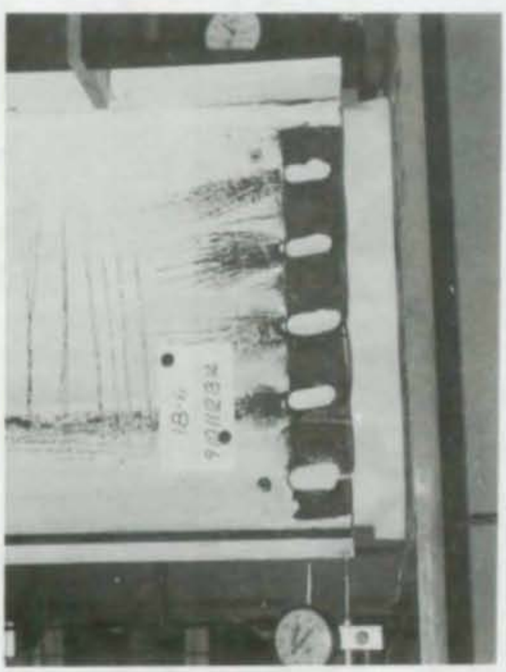

Fig. 3.1 Reaction-Deflection Response Curves



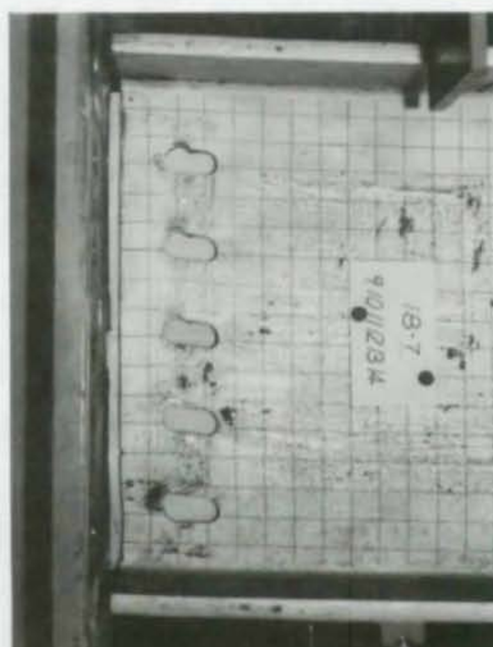
Test 18-1



Test 18-2



Test 18-6



Test 18-7

Fig. 3.2 Five-Bolt Uncoped Ends, Tested

In test 18-6 the bolts were installed as near as possible to the end of the beam. This was done for all tests with slotted-hole arrangements to ensure a "worst care" situation. The effect of the slots was to produce a number of small cantilevers which were more readily deformed. A fabrication error left the middle bolt-hole with less than minimum end distance, but this did not affect the connection's performance.

The load-deflection curves generally followed the same initial slope except for test 18-2. In this test, the load deflection curve, given in Appendix A, showed larger initial deformations than the other specimens. This was probably due to lack of equal bearing contact at all the holes. It was reasoned that the actual stiffness of test 18-2 should be similar to its slotted counterpart, test 18-6. Therefore, the load-deflection response curve shown in Fig. 3.1 was adjusted 0.05 in. to the left so that the response could be compared with the other tests.

Five-Bolt Connections--Coped

This group of connections, tests 18-3, 18-4, 18-5, and 18-8, all had fairly equal allowable load capacities based on the AISC Specifications, yet they carried a range of loads such that the factors of safety varied from 2.54 to 3.93. The reaction-deflection response curves for these tests are shown in Fig. 3.3. These tests provide a good insight to some effects of the cope. Figure 3.4 shows photographs of the tested connections.

Test 18-3, although having a coped end and minimum edge distance to the cope, performed very well reaching a maximum load of 212 kips. At this point the bracing system to the coped compression flange at the far end proved inadequate and slipped to one side contributing to a web-buckle forming near the cope. The test was then stopped. The bracing was corrected for the rest of the tests. This occurrence, however, did not affect the data on the connection

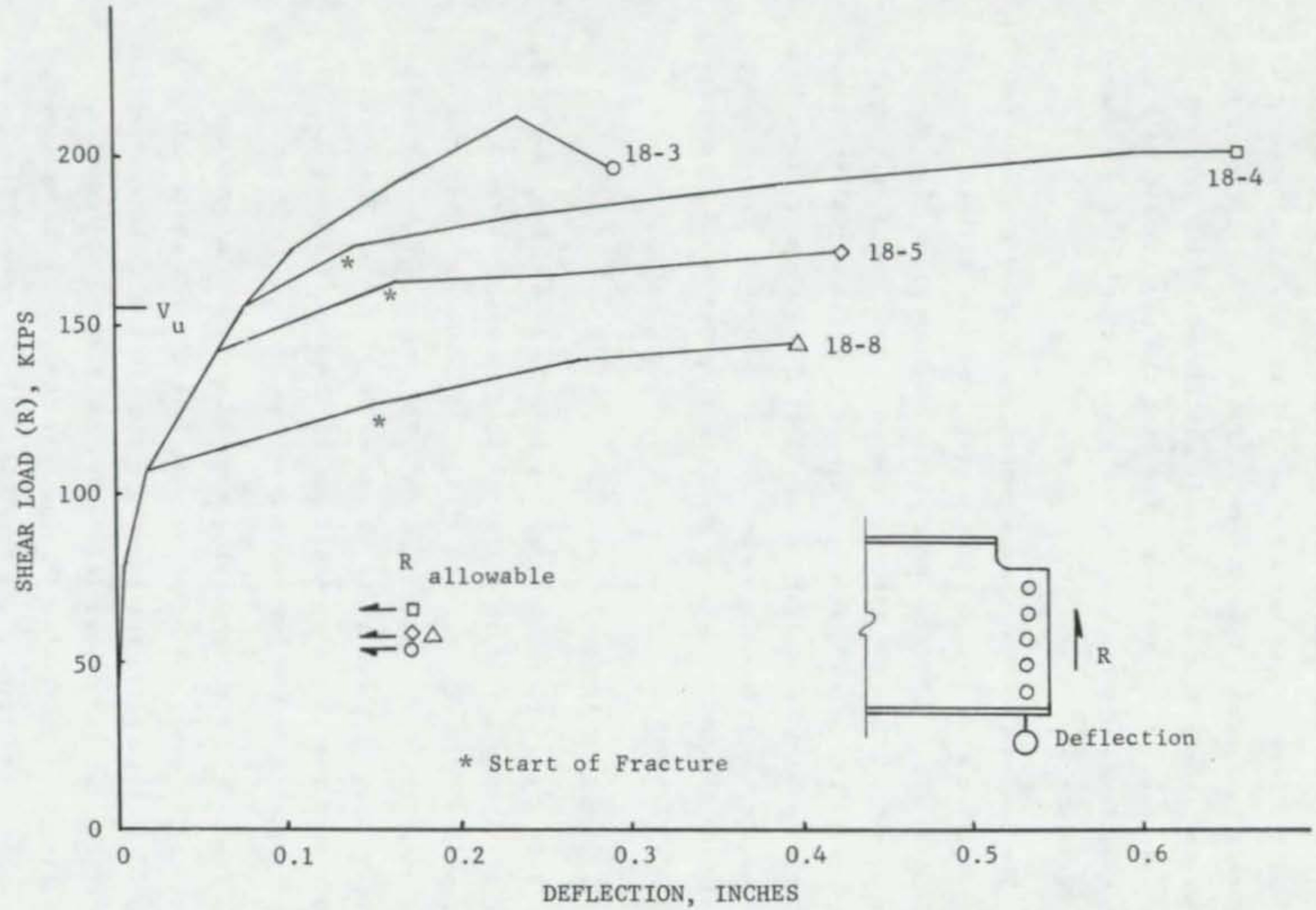
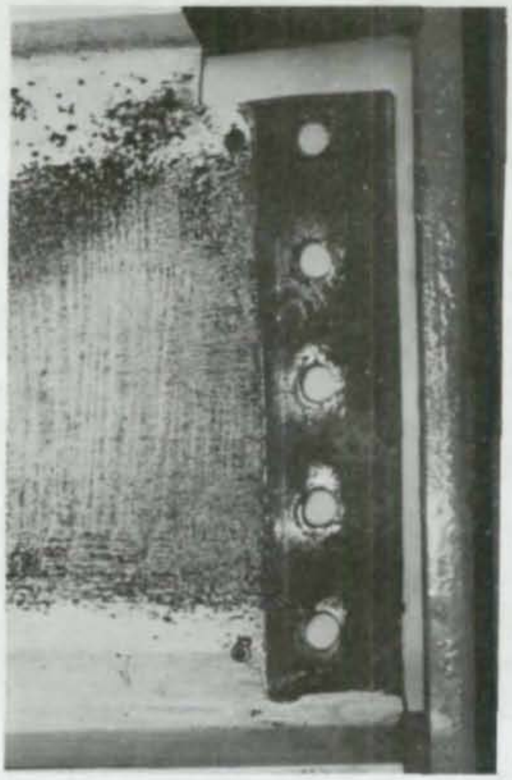
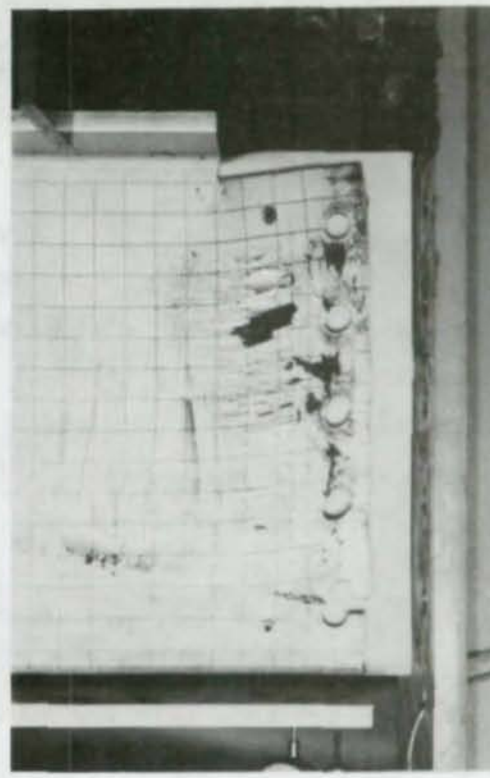


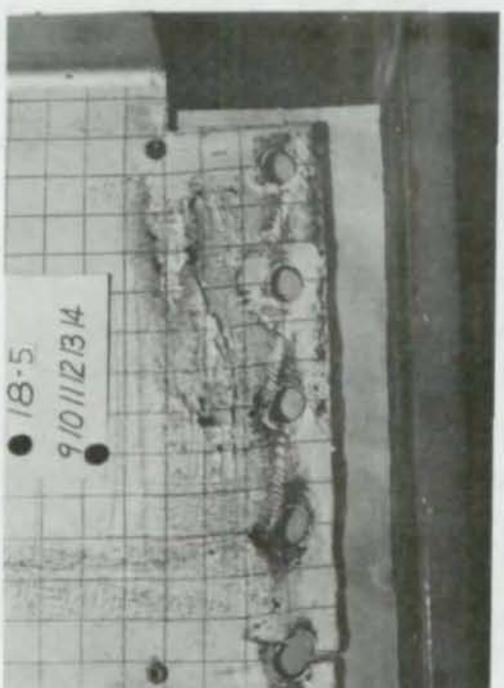

Fig. 3.3 Reaction - Deflection Response Curves



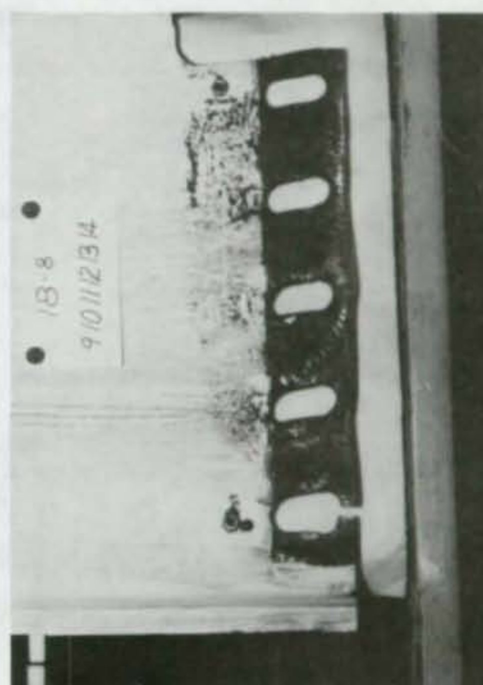
Test 18-3



Test 18-4




Test 18-5




Test 18-8

Fig. 3.4 Five-Bolt Coped Ends, Tested

performance. Deflections at the load point were great and the beam was reaching its ultimate capacity. Comparing this connection to test 18-1 with no cope, the cope did not affect this connection's performance.

Tests 18-4 and 18-5 each had minimum end distance, but test 18-4 had a large edge distance to the cope while test 18-5 had the minimum. The bolt-holes on each of these test ends were located identically, but the cope was cut to a greater depth in the fabrication of the connection for test 18-5 to obtain the minimum edge distance to the cope. Both test connections sustained a fracture through the end distance of the lowest bolt-hole near the uncoped flange. Test 18-4 reached a maximum reaction of 201 kips while test 18-5 reached only 173 kips, a 14 percent reduction, part of which can be attributed to the reduced web depth. Here as before, the bolts nearest the tension flange appear to carry a larger portion of the load as evidenced by greater yielding in their vicinity and greater bearing deformations or fracture at these holes. Once fracture of the end distance of the bottom bolt-hole had occurred, in each case the connections continued to deflect at a small increase in load. The loading was then discontinued. In each of these tests the yielding of the web was much more pronounced within the zone where the beam depth was reduced by the cope.

Test 18-8, the slotted hole companion to test 18-5 with minimum edge and end distance, reached a maximum reaction of 145 kips, 16 percent less than test 18-5. Testing was terminated when the load no longer increased as there was fracture through bottom bolt-hole end distance. Much like the behavior of specimen 18-6, the zones between the slotted holes acted as small cantilevers. With the removal of web shear area through coping, test 18-8 carried about 10 percent less load than test 18-6.

Three-Bolt Connections--Minimum Edge Distance, Large End Distance

There are two test connections with similar bolt arrangements, one from each heat, discussed in this section. The reaction-deflection response curves for tests 18-9 (heat 1) and 18-23 (heat 2) are shown in Fig. 3.5. Shown also on the plot is the response curve of test 18-3 at a 3/5 vertical scale. These connections were similar to test 18-3, except these had three bolts at 6-in. spacings as opposed to five bolts at 3-in. spacings for test 18-3. (Photographs are shown in Fig. 3.6.)

Test 18-9 reached a maximum reaction of 152 kips at which point the top bolt nearest the cope ripped through the edge. The average bolt bearing stress was 153.5 ksi, which is 86 percent of the theoretical maximum of $3F_u$.[15] Test 18-23 reached a maximum load of 157 kips with the bolt near the cope also ripping out. The average bolt bearing stress was 162 ksi which is 93 percent of the theoretical maximum. The actual edge distance of test 18-23 was slightly greater than minimum which accounts for the greater capacity. Bolt strength, hole deformation and bearing will be discussed later in Chapter 4. Test 18-23 duplicated test 18-9 extremely well and the scaled curve of test 18-3 correlated closely to these.

Three-Bolt Connections--Large Edge Distance

Tests 18-20, 18-21, and 18-25 are included in this category. Although the end distance to the holes in test 18-20 was greater than the 1 in. minimum by nearly 1/2 in., its mode of failure still provides us useful information. The reaction-deflection response curves for these tests can be seen in Fig. 3.7. Photographs of the tested ends are shown in Fig. 3.8.

In test 18-20 the bottom bolt began to bear noticeably into the web at a reaction of about 110 kips as evidenced by the relative displacement between the clip angles and the web. The connection

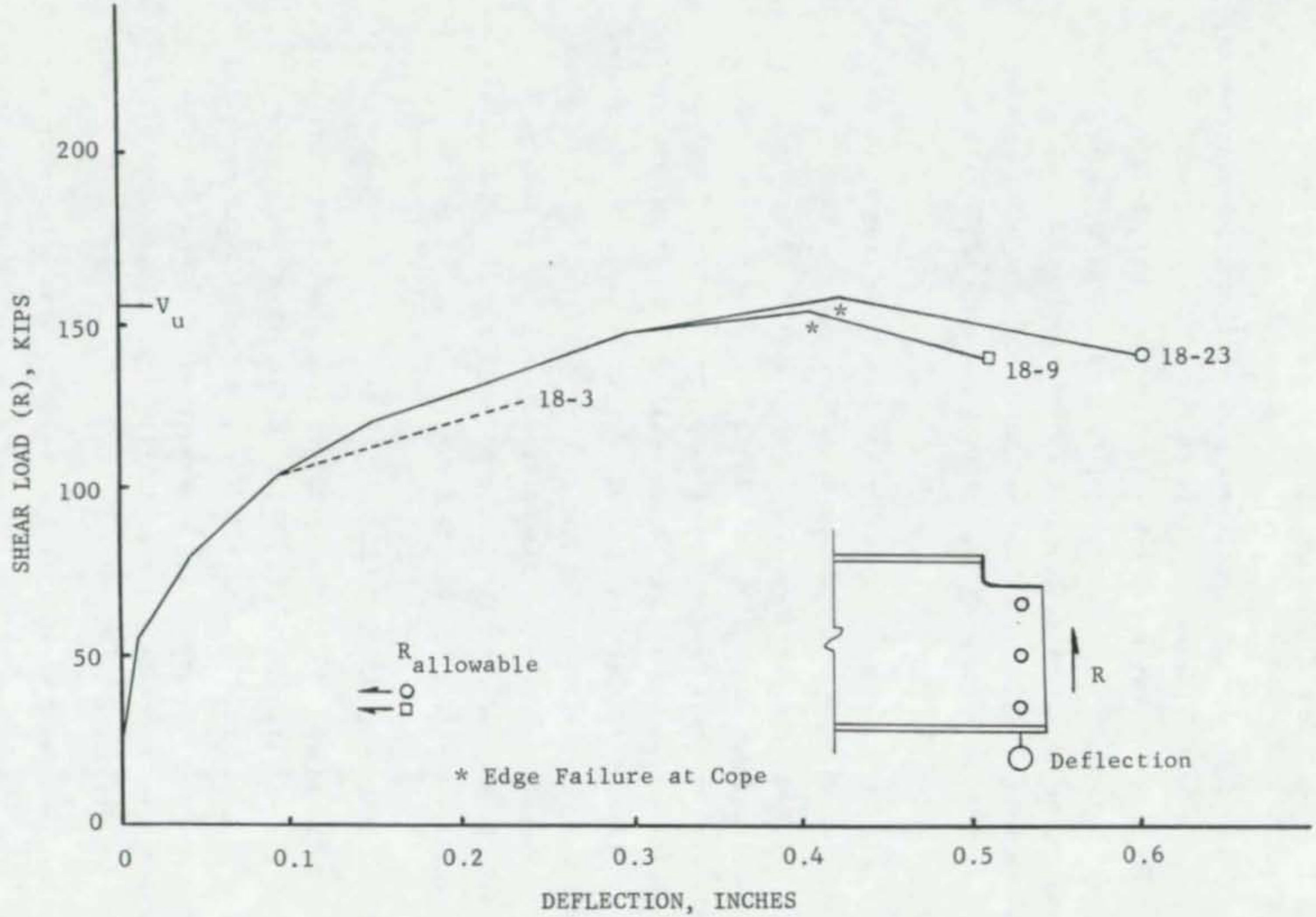
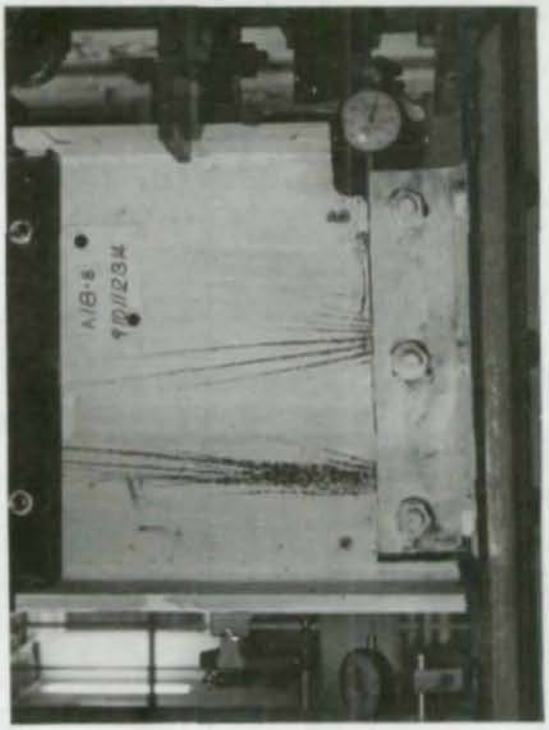
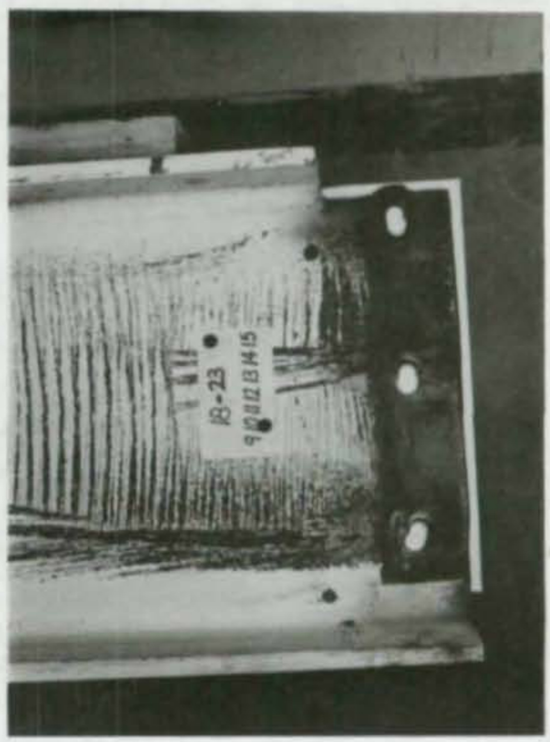



Fig. 3.5 Reaction-Deflection Response Curves



Test 18-9



Test 18-23

Fig. 3.6 Three-Bolt Coped Ends with Minimum Edge Distance and Large End Distance, Tested

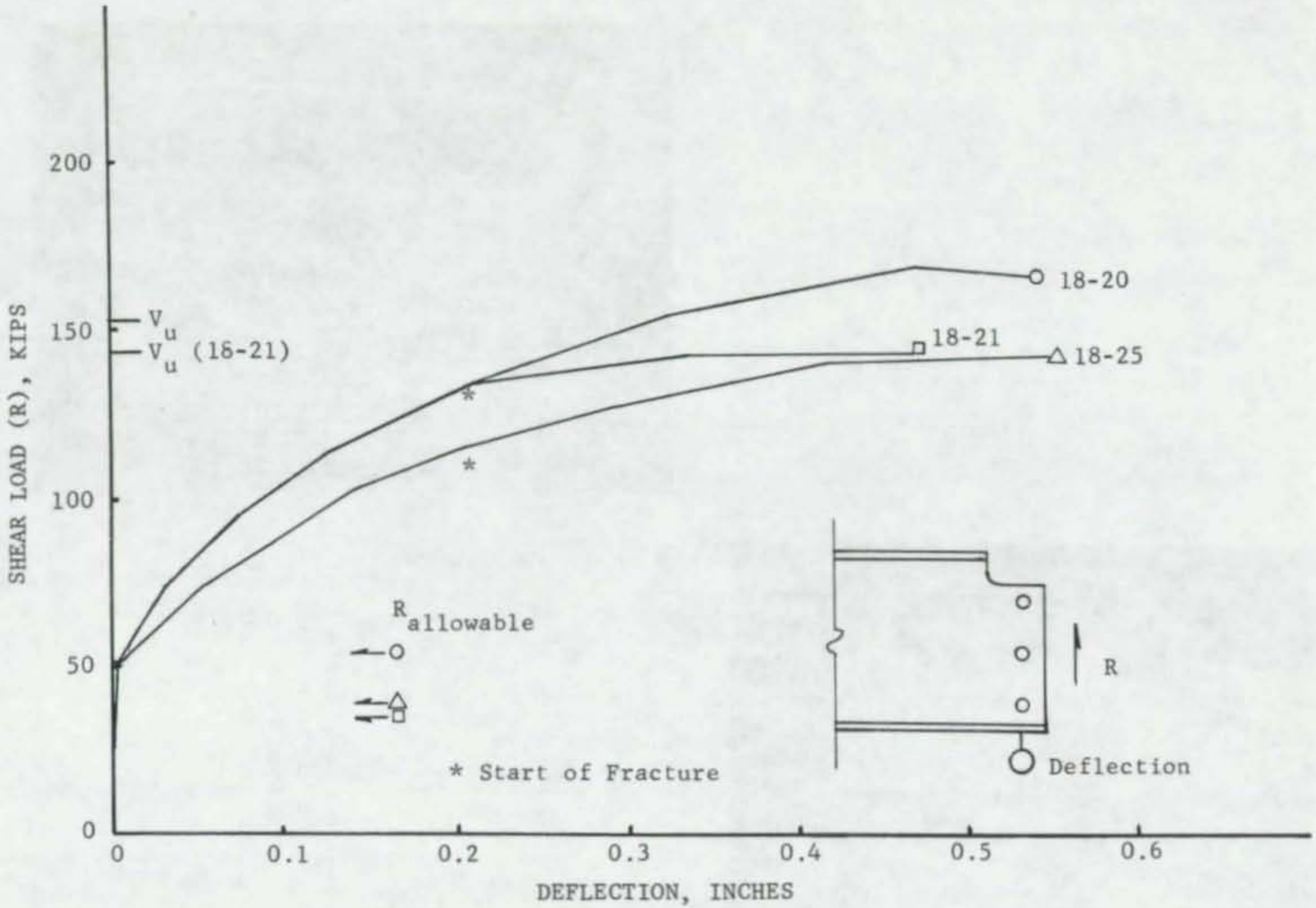
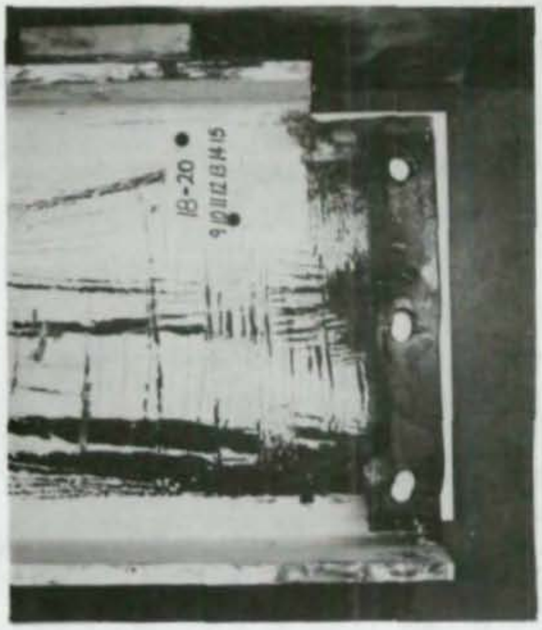
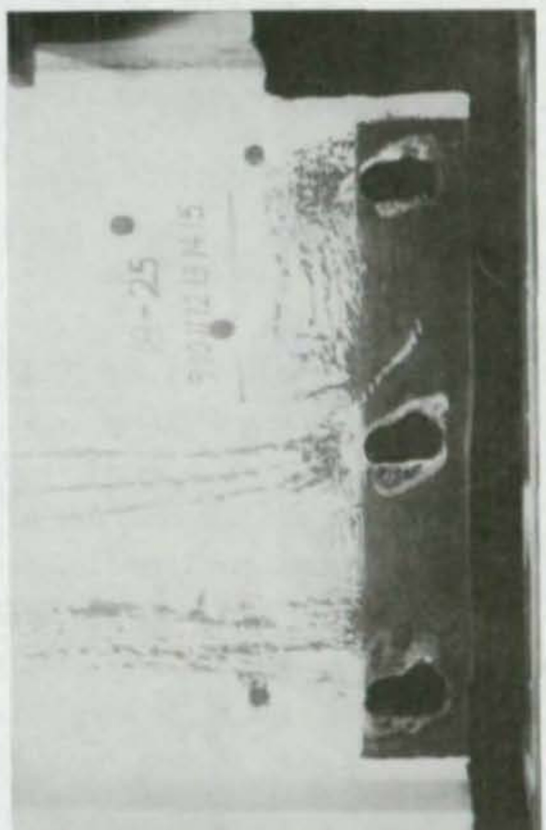



Fig. 3.7 Reaction-Deflection Response Curves




Test 18-20


Test 18-21



Test 18-25

Fig. 3.8 Three-Bolt Coped Ends with Large Edge Distance, Tested reached a maximum reaction of 167 kips at which point a fracture occurred at the bottom bolt-hole. There was considerable bearing deformation of the middle bolt-hole with the end distance area beginning to neck down. There was only a small amount of bearing deformation at the bolt-hole nearest the cope. Yield patterns were well developed across the web indicating that the web shear capacity was reaching full development. The plot shows that the reaction did exceed the theoretical ultimate web shear capacity.

Test 18-21, with minimum edge and end distance, was unique in that all the bolt-holes experienced necking in the end distance area and were all near fracture when the bottom hole fractured through and the test was stopped. The maximum reaction was 142 kips and the elongation of the bolt-holes averaged 1/4 in. The yielding of the reduced web area beneath the cope was thorough and the reaction was very close to the theoretical yield load in shear for the reduced web area.

In test 18-25 there was a slight evidence of cantilever action between the slotted holes at the maximum reaction of 142 kips. This reaction matched that of test 18-21. The lower stiffness and slightly greater deflection could be attributed to the cantilever action. The amount of web yielding was comparable to tests 18-20 and 18-21. The end distances at each hole did not reach the same levels of yielding as evenly as in test 18-21, partly because the end distance was not constant for all holes. The bottom hole was at minimum, the middle slightly more, and the hole near the cope had about 3/16 in. extra end distance. The bottom hole end distance underwent necking and fracture, the middle some necking, and the top was yielding, but no necking. The reduced stiffness due to the slots could also account for differences.

Three-Bolt Connections--Large Edge and End Distance

Plotted in Fig. 3.9 are the reaction-deflection response curves for tests 18-22 and 18-24. Each of these connections performed extremely well, thoroughly yielding the web of the beam, reaching maximum reactions of 185 kips and 178 kips, respectively. Both beams failed from buckling of the web near the cope. Bearing deformations in all of the bolt-holes were extensive. Figure 3.10 shows photographs of the tested ends. The average bearing stresses for each test were 191 ksi and 184 ksi, respectively, 110 percent and 106 percent of the theoretical maximum of $3F_u$. A slight adjustment for slip was made when plotting the response curve for test 18-24. This was done for improved comparisons as discussed earlier in reference to test 18-2.

In these tests as the web began to buckle there was a slipping of the secondary bracing supporting the compression flange just behind the coped section. With the web buckling and the flange slipping sideways at no increase in load, the tests were terminated. Web yielding was extensive for both tests. There was also much yielding of the tension flange above and to each side of the load point in both tests. The compression flange also had yield zones around the loading ram location.

Through the course of the test of connection 18-22, the ram pressure was released at certain load increments and one of the clip angles removed so the hole deformation in the webs could be observed. Measurements of bearing deformations at the bolt-holes due to the previous load were taken and observations of the condition of the web beneath the cover of the angles were made. The information collected about bearing deformation is discussed in the next chapter.

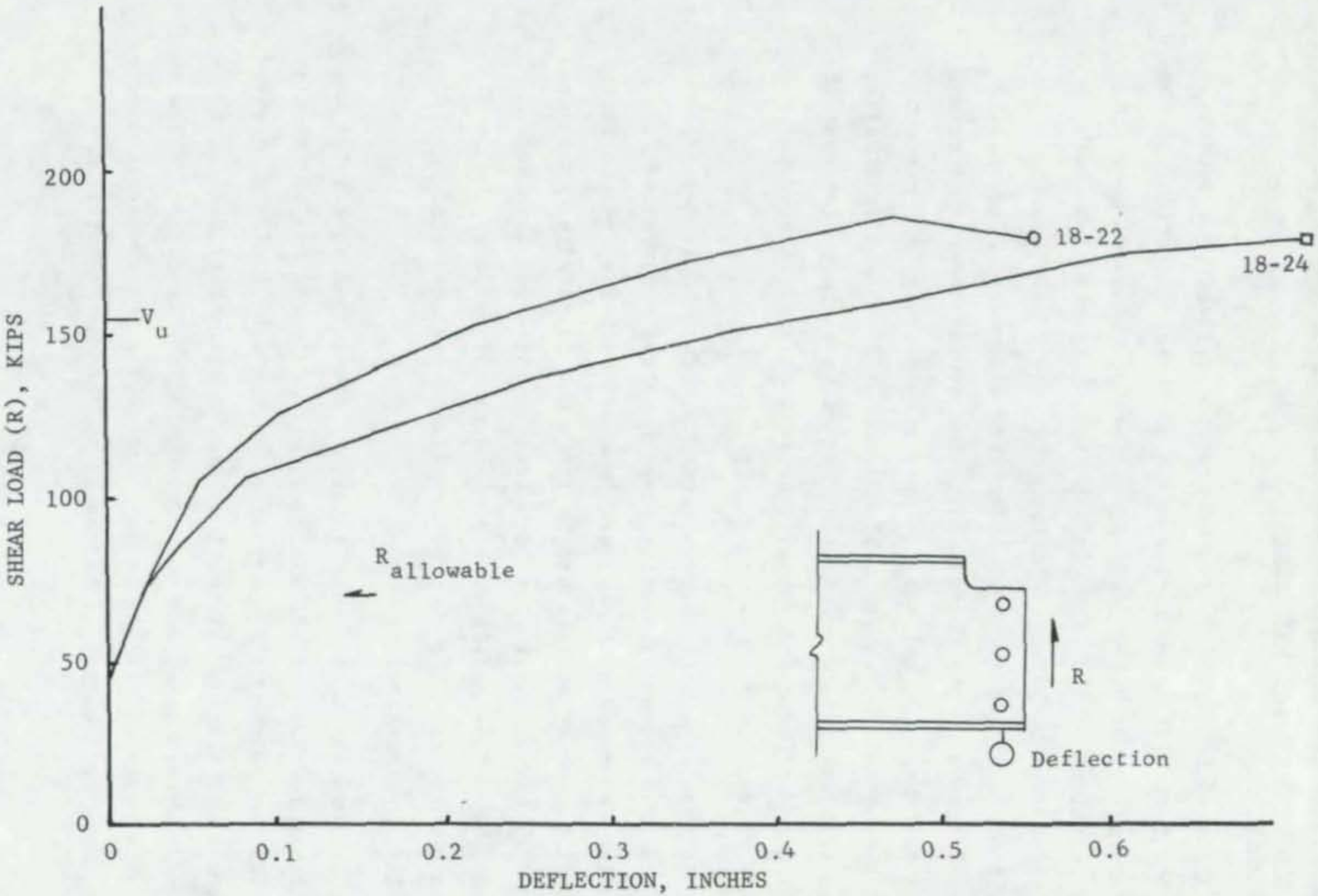
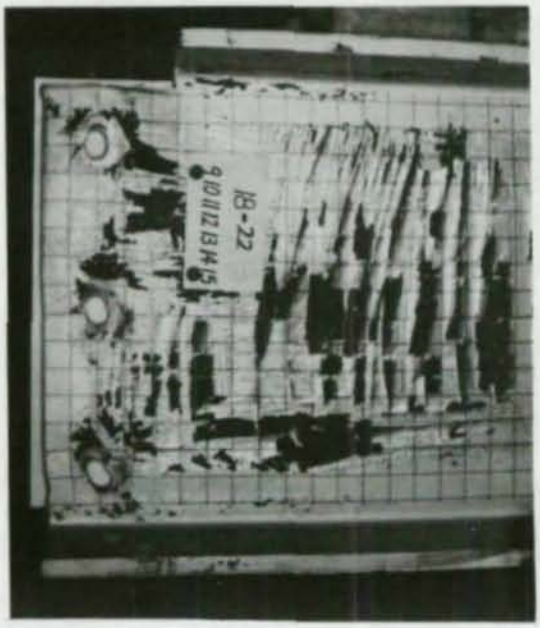
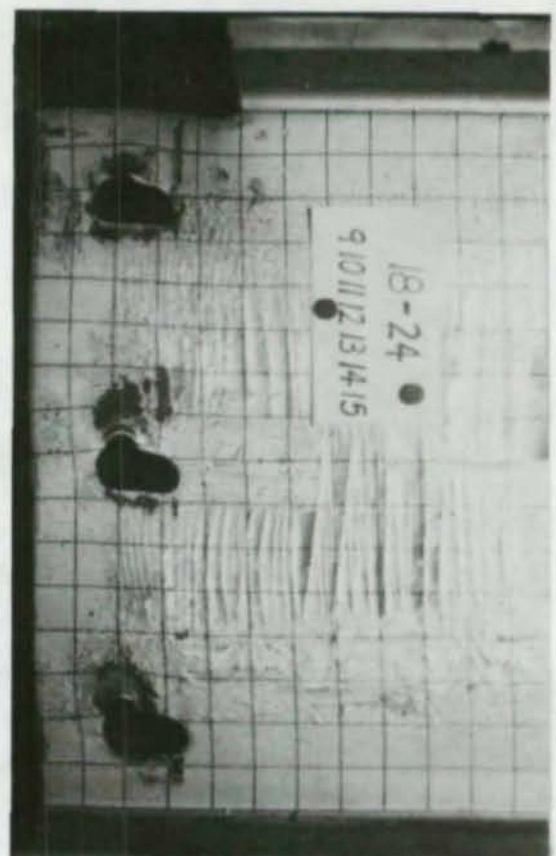



Fig. 3.9 Reaction-Deflection Response Curves




Test 18-22

Fig. 3.10 Three-Bolt Coped Ends with Large Edge and End Distance, Tested




Test 18-24

Observations

Generally all of the beam webs began yielding in the vicinity of the hole nearest the tension flange. In test 18-25 there was some early yielding at the middle bolt-hole. A check of the exact hole location showed that it was off center (a little low) and was bearing before the others. When the holes were all evenly spaced, yielding usually initiated near the lowest bolt line.

In the five-bolt connections, as the average bolt bearing stresses were low (ranging between 50 and 70 percent of ultimate bearing capacity), bearing deformations at the top bolt were negligible and the edge distance to the cope never controlled. For the three-bolt connections where the bearing stresses were fairly high (at or greater than theoretical ultimate), the edge distance was more critical.

For the slotted hole connections where the spacing of the holes was 3 in., the stiffness of the connection was generally reduced as the slots created small cantilevers which deformed more easily. This cantilever effect was not as noticeable for the connections with 6-in. spacings.

Comparison to Current Design

The November 1978 AISC Specifications[15] present a number of factors that must be checked when designing a bolted shear web connection. In Table 3.1 the allowable values of load as determined by each factor for the tests are shown. A factor of safety based on the smallest allowable load is given on the last column. The allowable loads were calculated using measured dimensions and variables of the beam ends and the corresponding material properties. These can be seen in Table A.1 of Appendix A.

TABLE 3.1 SUMMARY OF RESULTS AND ALLOWABLE LOADS

| Test Number (1) | Allow. Web Shear (K) (2) | AISC Allowable Loads, Connections: Bolt Shear (K) (3) | Bolt Bearing (K) (4) | Top Edge Dist. (K) (5) | Eccen. Factor (6) | End Dist. (K) (7) | Block Shear (K) (8) | Ru (K) (9) | F.S. (10) |
|---|---|---|---|---|---|---|---|---|---|
| Heat 1 | | | | | | | | | |
| 18-1 | 124 | 133 | 148 | - | 0.83 | 128 | - | 205 | 1.65* |
| 18-2 | 124 | 133 | 148 | - | 0.83 | 61 | - | 206 | 3.38 |
| 18-3 | 107 | 133 | 148 | 65 | 0.83 | 132 | 92 | 212 | 3.93 |
| 18-4 | 113 | 133 | 148 | 123 | 0.83 | 65 | 86 | 201 | 3.09 |
| 18-5 | 107 | 133 | 148 | 70 | 0.83 | 65 | 79 | 173 | 2.98 |
| $18\text{-}6_s$ | 124 | 133 | 148 | - | 0.83 | 57 | - | 161 | 2.82 |
| $18\text{-}7_s$ | 124 | 133 | 148 | - | 0.83 | 119 | - | 206 | 1.73 |
| $18\text{-}8_s$ | 107 | 133 | 148 | 70 | 0.83 | 57 | 78 | 145 | 2.54 |
| 18-9 | 107 | 80 | 89 | 39 | 0.87 | 79 | 104 | 152 | 4.48 |
| Heat 2 | | | | | | | | | |
| 18-20 | 105 | 80 | 83 | 70 | 0.87 | 54 | 103 | 167 | 3.09 |
| 18-21 | 99 | 80 | 83 | 40 | 0.87 | 40 | 90 | 142 | 4.08 |
| 18-22 | 104 | 80 | 83 | 81 | 0.87 | 73 | 109 | 185 | 2.66 |
| 18-23 | 105 | 80 | 83 | 46 | 0.87 | 76 | 102 | 157 | 3.92 |
| $18\text{-}24_s$ | 105 | 80 | 83 | 80 | 0.87 | 70 | 109 | 178 | 2.56 |
| $18\text{-}25_s$ | 104 | 80 | 83 | 75 | 0.87 | 39 | 97 | 142 | 3.64 |

PROPERTIES:

Heat 1 $t_w$ = 0.44 in. Fy = 38.3 ksi.
d = 18.38 in. Fu = 59.7 ksi.

Heat 2 $t_w$ = 0.43 in. Fy = 36.6 ksi.
d = 18.25 in. Fu = 58.0 ksi.

* F.S. Based on Allow. Web Shear
F.S. = 1.44 (AISC)

s Slotted Hole Connections

_ Controlling Allow. Underlined

In column 2 of Table 3.1 are the allowable web shear values for the beam end. They were based on Section 1.5.1.2.1 of the AISC Specifications. On the cross-sectional area effective in resisting shear, which for rolled W-shapes is taken as the overall depth of the beam (depth to the cope for coped beams) times the web thickness, the allowable shear stress is taken as

$$F_v = 0.4F_y \tag{3.1}$$

Columns 3 and 4 show the allowable loads for bolt shear capacity and web bearing capacity, respectively. Bolt shear was obtained by summing the shear capacities of each bolt shear plane. (From Table 1.5.2.1 of the AISC Specifications, the allowable shear stress for an A325 bolt, threads excluded, is 30.0 ksi.) Bearing was calculated following Section 1.5.1.5.3 where the allowable bearing stress of the web,

$$F_p = 1.5F_u, \tag{3.2}$$

is multiplied by the sum of the projected areas of the bolts.

The AISC allowable loads which almost exclusively controlled the design of these connections were those dealing with edge and end distance, columns 5 and 7, respectively. Section 1.16.5.2 of AISC requires a minimum edge distance ($e_c$) along the line of force, where

$$P = 0.5F_u(e_c \cdot t_w). \tag{3.3}$$

P is the force transmitted by one fastener. The edge distance values used were the measured values. It should be noted that spacing should be checked as well, but for these tests the spacing was not a controlling factor. The allowable was determined by applying the minimum value of P to all of the fasteners. The end distance ($e_e$) requirements of Section 1.16.5.3 of AISC were checked in a similar fashion following the equation

$$P_r = 0.5F_u(e_e \cdot t_w), \tag{3.4}$$

where $P_r$, like P, is the force per fastener.

Column 6 contains an eccentricity factor derived using the "instantaneous center of rotation" method developed by Kulak.[8] These factors were applied to the allowable loads derived from bolt shear, bearing and top edge distance requirements, columns 3, 4, and 5, respectively. Eccentricities do not have to be taken into account for end distance allowables from Eq. (3.4) or for block shear (Eq. (3.6)).

The AISC Specifications, Section 1.5.1.2.2, requires an additional check for coped beams. Column 8 of Table 3.1 shows the allowable load values for the block shear failure mode. AISC allows the designer to use an allowable stress of

$$F_v = 0.30F_u \tag{3.5}$$

on a minimum net failure surface composed of the net vertical shear area from the bottom bolt-line to the cope and a net horizontal tensile area along the bottom bolt-line (Fig. 3.11). The Specifications alternatively allow the designer to use a two part equation, which has been discussed in the introduction (Eq. (1.4)),

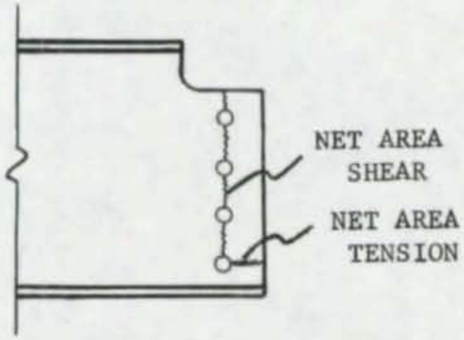



Fig. 3.11 Block-Shear Model

$$B_v = 0.3F_u\left(A_{v_{net}}\right) + 0.5F_u\left(A_{t_{net}}\right), \tag{3.6}$$

to obtain the allowable load. This equation gives a larger load value which is less conservative. In column 8 the values shown were calculated using the less conservative equation, Eq. (3.6).

In column 9 the ultimate loads are given for each test followed by the factors of safety in column 10 obtained by dividing the lowest allowable into the ultimate load. Ideally, the factor of safety should be approximately 2.0. Although in some instances the factors of safety appeared quite good and provided an adequate, yet not too conservative value, they generally did not reflect the actual mode of failure that should have controlled. This necessitated a careful analysis of the test results to arrive at a better understanding of the behavior of the connections and an improved design approach as given in the next chapter.

CHAPTER 4

# DEVELOPMENT OF A DESIGN METHOD

## Approach

In these series of tests there were a variety of failure modes that terminated the useful performance of the connection. In the actual testing procedure the tests were typically stopped when the connection could no longer develop load due to fracture, bearing failure or buckling, or simply were stopped when deflections became excessive. Careful observations were made of the test results, paying special attention to those tests which truly failed at the connection.

The tests that were stopped from excessive deflections often were failing in web shear or sustained large bearing deformations. Occasionally there was buckling in the final stages of loading. When a fracture of the net tensile area (Fig. 3.11) occurred, it started early at relatively low deflections. A fracture of this sort did not always result in total failure of the connection, but generally marked a change in the stiffness of the connection (see the plots in Chapter 3). All developing failures were accompanied by a decreasing stiffness, but the change in stiffness was more clearly defined at the start of fracture.

In the following sections, three basic failure criteria, each involving several items, are reviewed. The first phenomenon is occurrence of fracture with the accompanying possibility of a block-shear type failure. Second, bolt or bolt-hole bearing capacity, involving the edge and end distances and the distribution of load to the bolts, are studied. Third, deflection, as a definition of failure, is studied through the development of ideas involving slotted holes versus standard holes and further observations of the double-row connection tests. The loads at the deflection chosen to define failure are compared to the allowables developed in this chapter and the comparisons are discussed.

Fracture

In developing criteria for the prediction of the fracture load it was noted that connection details with a large end distance apparently had sufficient tensile strength, $T_e$, (Fig. 4.1) to develop the vertical shear $V_d$ in the web beneath the probable fracture plane (section B-B), so no fracture was sustained. For those tests in this study which had minimum end distances, the available web shear force, $V_d$, of section C-C, was sufficient to develop fracture. The results of the tests performed at The University of Toronto[12] and the double-row tests from The University of Texas[13] indicated that fracture could occur with fairly large end distances if the web shear area (section C-C) could develop the reaction, $V_d$, necessary for fracture. The question arising is what constitutes sufficient web capacity below the bottom hole to develop fracture.

Recalling there must be elongation at section B-B for fracture to occur, it was obvious that the shear area (Fig. 3.11) of the "web block" must yield to allow for movement. Observations from these and other tests (Ref. 13) strongly support the suggestion that for the occurrence of fracture the web must reach at least shear yield. (This is supported by the marked change in slope of the curves in Chapter 3 for those tests sustaining fractures.) If the web shear area below the bottom hole (section C-C) also yields, it will move along with the upper block and not allow for elongations to occur. At this point the connection is controlled by general web yielding.

The occurrence of fracture, which permits the possibility of a block-shear type failure, depends upon the relationship between

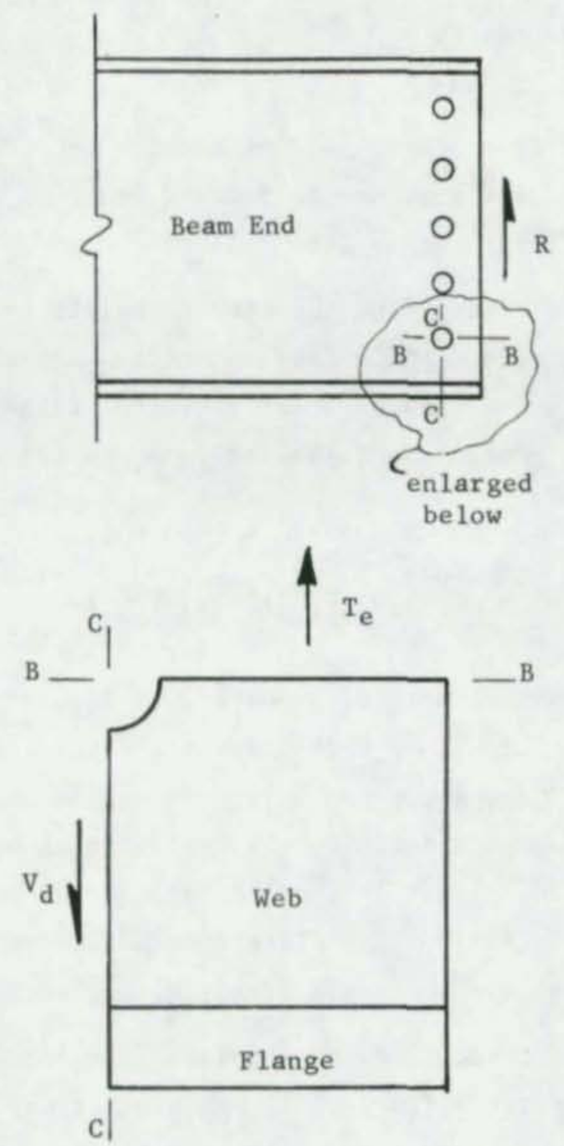



Fig. 4.1 Freebody Forces - Development of Fracture

$T_e$ and $V_d$ (Fig. 4.1). Defining the web shear strength of section section C-C as

$$F_d = 0.6F_y\left(A_{v_{dev}}\right), \tag{4.1}$$

where $A_{v_{dev}}$ is the gross shear area of section C-C (Fig. 4.1), it was recognized that for fracture to occur $V_d$ must be greater than $T_e$ (the tensile strength at section B-B).

The Toronto tests[12] and the results herein indicate a probably uniform stress distribution on section B-B with a value of $F_u$ at fracture and a general shear yielding of the gross area above the fracture giving the following equation for allowable block shear,

$$B_v = 0.3F_y\left(A_{v_{grs}}\right) + 0.5F_u\left(A_{t_{net}}\right), \tag{4.2}$$

using a factor of safety of 2.0. However, the study on double-row connections[13] had indicated a triangular stress distribution which reduced the force $T_e$ by about one-half. It was necessary to develop a correlating factor, indicated by the similarities between the block-shear equation of Ref. 13 (Eq. (1.5)) and the one being developed here (Eq. (4.2)). The difference between these block-shear equations lay in the tensile stress distribution.

It was soon recognized, by observing the tabulations of the test results in Chapter 3 (Table 3.1) and similar tabulations for the double-row connections of Ref. 13, that the eccentricity factor, $E_r$, from the instantaneous center of rotation method developed by Kulak and presented in the 1980 edition of the AISC Manual,[9] afforded a reasonable reduction factor that could be used to adjust the stress distribution in the tensile section. For the double-row connections the eccentricity factor, which ranged from 0.51 to 0.60, when applied to the uniform tensile stress, $0.5F_u$, on the net section, reduced the stress to $0.26F_u$ and $0.3F_u$, respectively. These values are fairly close to $0.25F_u$, from the triangular distribution of Ref. 13. Thus, fracture is possible if

$$0.3F_y\left(A_{v_{dev}}\right) \geq 0.5(E_r)F_u\left(A_{t_{net}}\right), \tag{4.3}$$

where $E_r$ is the eccentricity factor. The allowable block-shear formula applicable to both single and double rows of bolts becomes

$$B_v = 0.3F_y\left(A_{v_{grs}}\right) + 0.5(E_r)F_u\left(A_{t_{net}}\right). \tag{4.4}$$

(Further notes on eccentricity can be found in Appendix C.)

Tables 4.1 and 4.2, given at the end of this chapter, summarizing the results of this study include among other things the eccentricity factors for each connection, the results of the fracture check (a "+" indicating possible fracture) and the block-shear values for those cases where fracture is possible. A reduction factor developed for slotted-hole connections ($S_r$) and also adopted for double-row connections is included where applicable. In a comparison of the slotted-hole tests to the standard hole tests, it was found that typically the slotted-hole connections carried lower load than their standard hole counterparts for the same level of deflection: a reduction of about 15 percent or $S_r = 0.85$. From observations of the load-deflection curves of the double-row connections of Ref. 13 (shown later in Fig. 4.6 of the deflection section), the same reduction is applied to double-row connections. The block-shear equation with this reduction factor becomes

$$B_v = \left[0.3F_y\left(A_{v_{grs}}\right) + 0.5(E_r)F_u\left(A_{t_{net}}\right)\right](S_r) \tag{4.5}$$

(For single-row standard hole connections $S_r$ is taken as 1.0.) Tables 4.1 and 4.2 reflect values calculated using Eq. (4.5).

Bolt and Bolt-Hole Capacity

Using primarily those tests for which the load on an individual bolt was high, it was found that the holes were necessarily reaching or going beyond individual accepted bearing capacities to develop the load, as was indicated in Chapter 3. The results of test 18-22 and of the duplicate tests, 18-9 and 18-23, were studied. Each had a large end distance with no fracture occurring when tested, which allowed bearing to be studied more effectively.

Test 18-22, where loading was stopped at intervals and the clip angles removed to study the web, provided important facts about load distribution and bearing. In test 18-22, measurements of hole elongations were taken corresponding to the permanent deformation for the load level reached. The deformations of each hole are plotted against the corresponding load level in Fig. 4.2 which show that the bottom hole had the largest deformation. Shown also on the plot is the maximum bearing load following a summation of individual hole bearing capacities in the direction of fastener loads (based on a bearing stress of $3F_u$). The plotted information coupled with observations from other tests indicated that the bottom bolt-hole probably had high tensile stress on the net section in addition to the high bearing stresses resulting in greater hole deformations. A discussion of this can be found in Appendix C.

The effects of the edge distance to the cope was effectively covered by tests 18-9 and 18-23, both of which had minimum edge distance and sustained failures of that edge. These tests supported the use of the edge distance strength formula (Eq. (3.3)) in conjunction with other bolt or hole capacities to obtain the allowable.

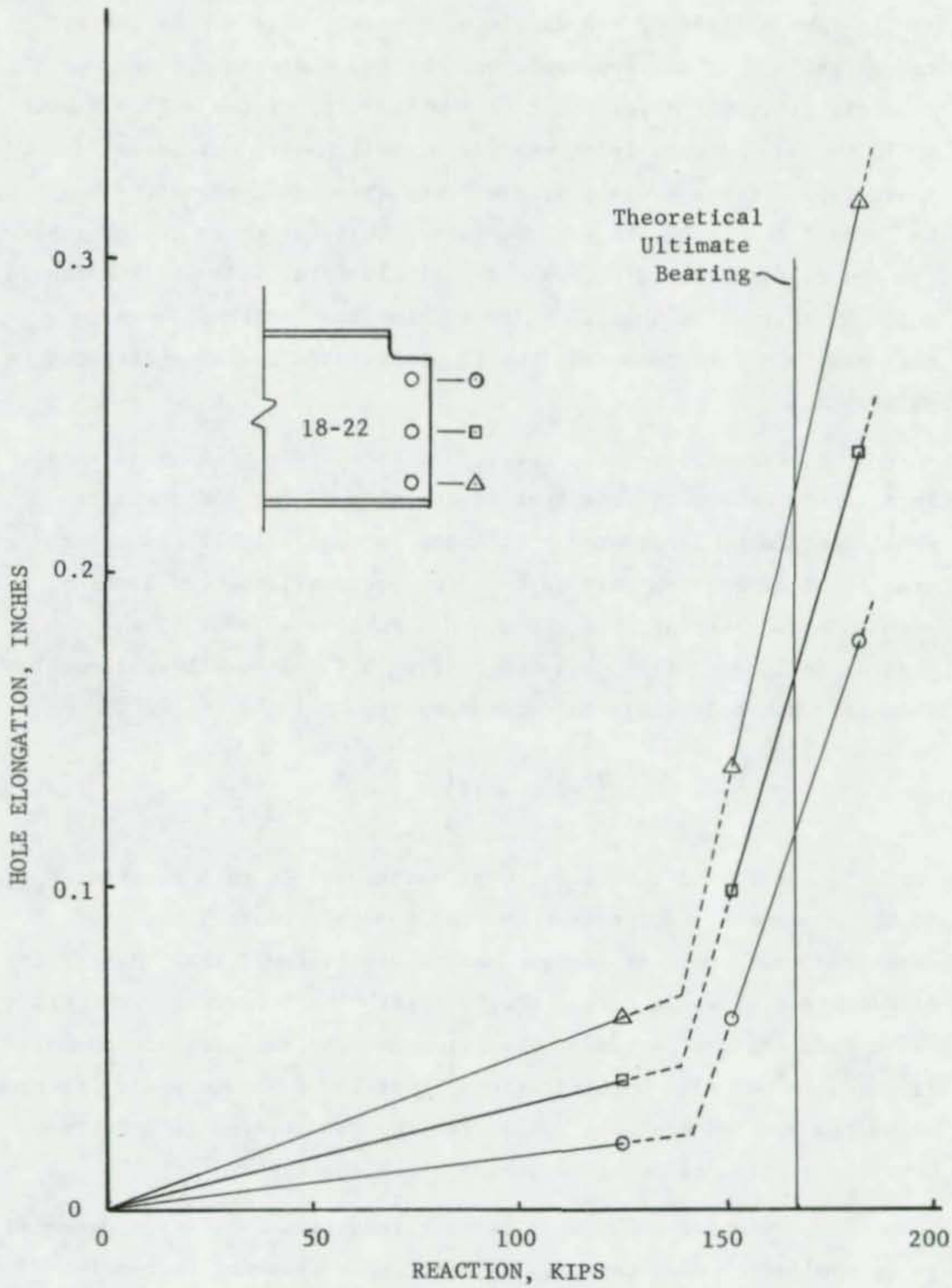



Fig. 4.2 Bolt Bearing Deformations

The effects of end distance, however, were not as easily recognizable. It was apparent that the end distance did not directly limit the development of the capacity of the bolt. A bolt could be developed fully in bearing or bolt shear, whichever controlled, if there was sufficient web shear area above the bolt to develop the load. It was recognized that for these connections the end distance could be removed entirely (Fig. 4.3) and load could still be carried in shear. (A connection such as that shown in Fig. 4.3 is not recommended, but is simply used to illustrate the point.)

In order to develop the top bolt fully in bearing or shear in a coped connection, the tensile strength of the end distance would need to be considered. As there is typically little shear area ahead of the top bolt in coped connections, the end section becomes more important. A check would have to be made of the minimum failure surface depicted in Fig. 4.4. A possible allowable load for that bolt would be determined by

$$F_{min} = \left[0.3F_y\left(A_{v_{grs}}\right) + 0.5F_u\left(A_{t_{net}}\right)\right](S_r) \qquad (4.6)$$

which is similar to the block shear equation. In this equation $A_{v_{grs}}$ is the gross web shear area above the top bolt centerline. No eccentricity factor was included as it was observed that the effects of eccentricity would direct the bolt into the beam away from this end and do not act to reduce its strength. An end distance formula as given in the AISC Specifications[15] was found not to apply for these connections. End distance considerations are covered indirectly by other connection allowables such as block shear.

For slotted holes a 15 percent reduction ($S_r$) is recommended to be applied to each capacity calculation concerning bolt-hole strength, i.e. bearing, spacing, edge, and $F_{min}$ (Eq. (4.6)). Referring to the plots of the results of tests 18-24 and 18-25 in

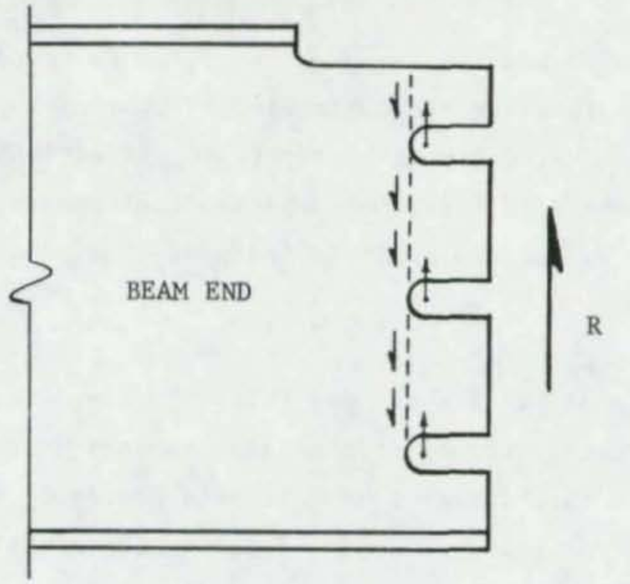



Fig. 4.3 End Distance Removal

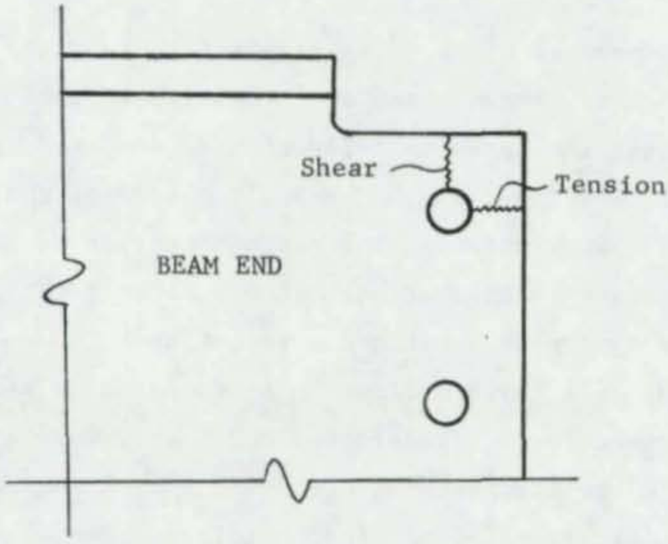



Fig. 4.4 Top Bolt--Minimum Failure Plane

Chapter 3 (Figs. 3.7 and 3.9, respectively), it is noted that the plots, although both tests are controlled by bearing type failures, show a reduction in strength with respect to deflections when compared to the standard hole counterparts. Bolt shear strength does not need to be reduced as it is independent of the hole type.

Deflections

The study of deflections was initially approached by observing differences between slotted and standard hole connections. It was recognized that deflections, although generally not regarded in the design of connections, could be quite substantial at the ultimate load, which is the load used in calculating factors of safety upon which designs are based. Deflections are considered in the design of the beam and as connection deflections do occur recommendations are included herein to account for them. Following a review of other literature[11] and observations made of these tests, a deflection defining "failure" of 1/4 in. was chosen and its validity checked by comparisons to the test results.

About 50 percent of the connections in this study sustained a fracture. Of these several went on to develop somewhat higher loads, but at large increases in deflection as compared to earlier portions of their loading curve. Carefully observing the load-deflection plots and associating the response curves to the respective connection configurations, it was noticed that slotted-hole connections typically exhibited a reduction in load of approximately 15 percent at a given deflection as compared to similar standard hole connections. The slotted holes that were used in these tests were long slots (dimensions: $(d + \frac{1}{16}) \times 2\frac{1}{2}d$, see Table 1.23.4 of the AISC Specifications[15]). Unfortunately, no short slots were used and therefore data are not available to study such a variation. Five slotted-hole connections were tested. They are labelled with "s" after the test number, column 1 in Table 4.1.

Reviewing the connections with 3-in. spacings between the holes, it was found that the load curves for the slotted connections (tests 18-6 and 18-8, Figs. 3.1 and 3 3, respectively) were nearly flat after fracture. In the standard hole tests, particularly tests 18-2 and 18-4, after fracture the connections were able to develop considerably more load although at relatively large deflection. The standard hole connections were better able to pick up the transfer of load at fracture and continue to develop more load. From these observations a theory was developed to explain the differences. In standard hole connections the net shear area is concentrated in a small zone along the vertical centerline of the single row of holes. For slotted holes yielding can occur along the entire slot length. The deflections accompanying yielding on the net shear section are much more significant for slotted holes than for standard holes before strain hardening begins. Single-row standard hole connections could reach strain hardening relatively quickly after end section fracture providing a "reserve strength." Fracture did not necessarily impair the connection.

As some standard hole tests did develop a reserve strength, it was decided to include an equation to check for strain hardening effects after fracture applicable only to standard hole connections. The allowable net shear capacity,

$$N_v = 0.3F_u\left(A_{v_{net}}\right), \tag{4.7}$$

based on the ultimate shear stress taken on the net web shear area ($A_{v_{net}}$) above the fracture, was chosen. For standard hole single-row connections the larger of net shear ($N_v$, Eq. (4.7)) or block-shear ($B_v$, Eq. (4.5)) would be chosen as the allowable taking full advantage of the available capacity. Values from the net shear allowable can be seen in column 7 of Table 4.1.

Reviewing the slotted-hole connection results, some apparent discrepancies with the 15 percent reduction theory were noted. In the plot of test 18-7 (Fig. 3.1) a reduction was not seen. That test did not sustain a failure at the connection and is therefore not a representative example. In the case of test 18-25 (Fig. 3.7), there was some increase in load after fracture. In that test the spacing was 6 in. between holes enabling the connection to perform better after fracture. The greater spacing, however, did not affect the 15 percent reduction in load at a given deflection (compare test 18-25 to the standard hole test 18-20 in Fig. 3.7). Test 18-24 (Fig. 3.9), also with slotted-holes at 6 in. spacings, exhibited a reduction of 15 percent as compared to test 18-22. Test 18-24 did not sustain a fracture and did go on to develop loads comparable to test 18-22, but at greater deflections. It is noted here that the 15 percent reduction could be in some way analagous to a similar reduction for slots in friction type connections, where the slots affect the clamping force.[4]

The comparison of test 18-24 to 18-22 (Fig. 3.9) provided a good example for the need to control deflection. Connection 18-22 deflected about 1/2 in. at ultimate and its slotted-hole counterpart, connection 18-24, deflected about 50 percent further, both fairly large concentrated deflections. Some limit of usefulness needed to be placed on these deflections. Previously a deflection of 1/4 in. was suggested as a definition of failure for bolted member-end connections.[11] For comparison of the 1/4 in. deflection criteria, the load-deflection curves for all of the tests discussed were examined and the loads corresponding to a deflection of 1/4 in. were taken. (The load-deflection curves used were those from Chapter 3 and those shown in Figs. 4.5 and 4.6, the Toronto test curves and the Texas double-row test curves, respectively. In preparing the plots for Figs. 4.5 and 4.6 three of the curves were modified slightly. Adjustments were made to eliminate deflection response distortions

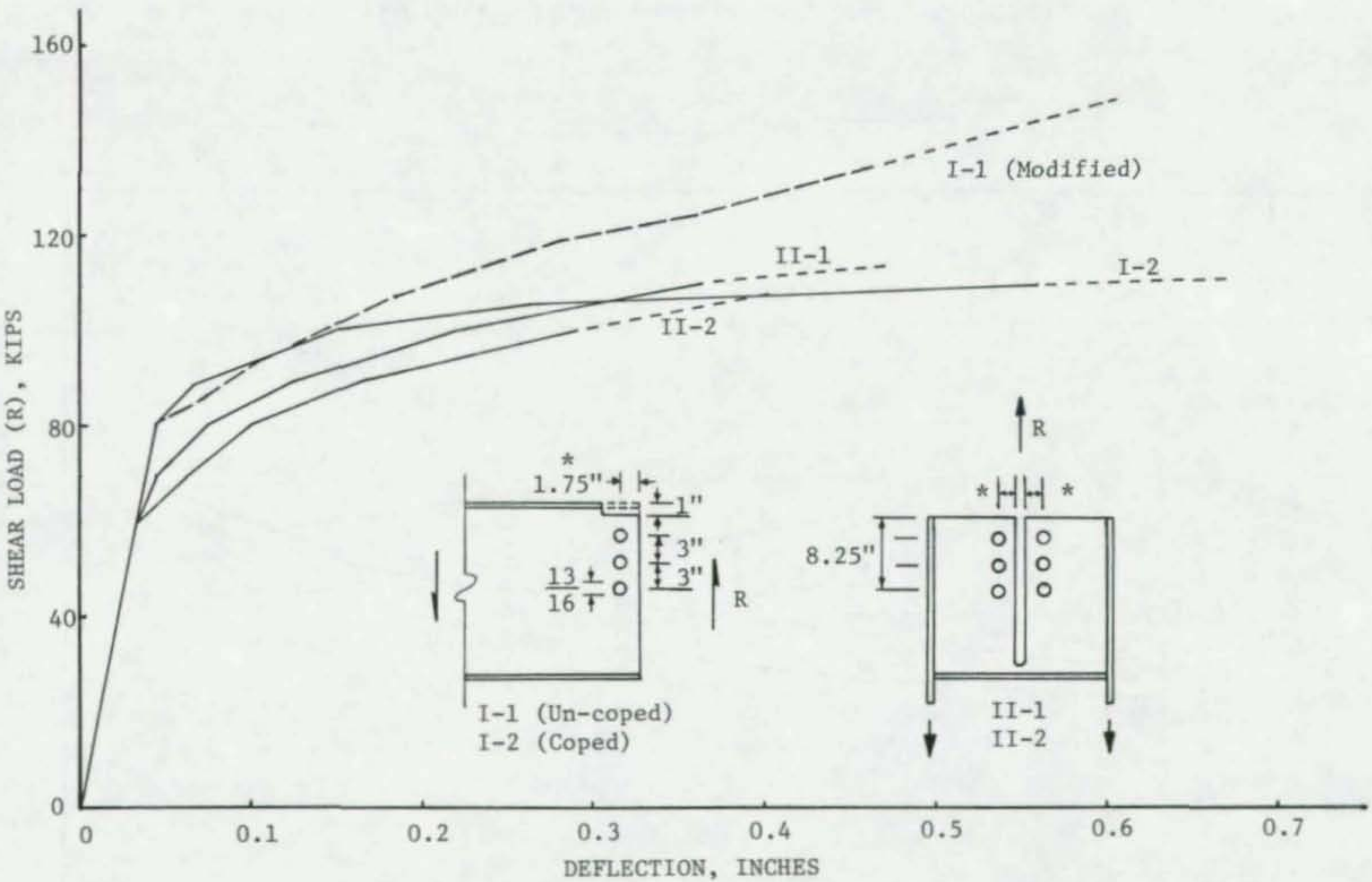



Fig. 4.5 Reaction-Deflection Response Curves, University of Toronto Tests

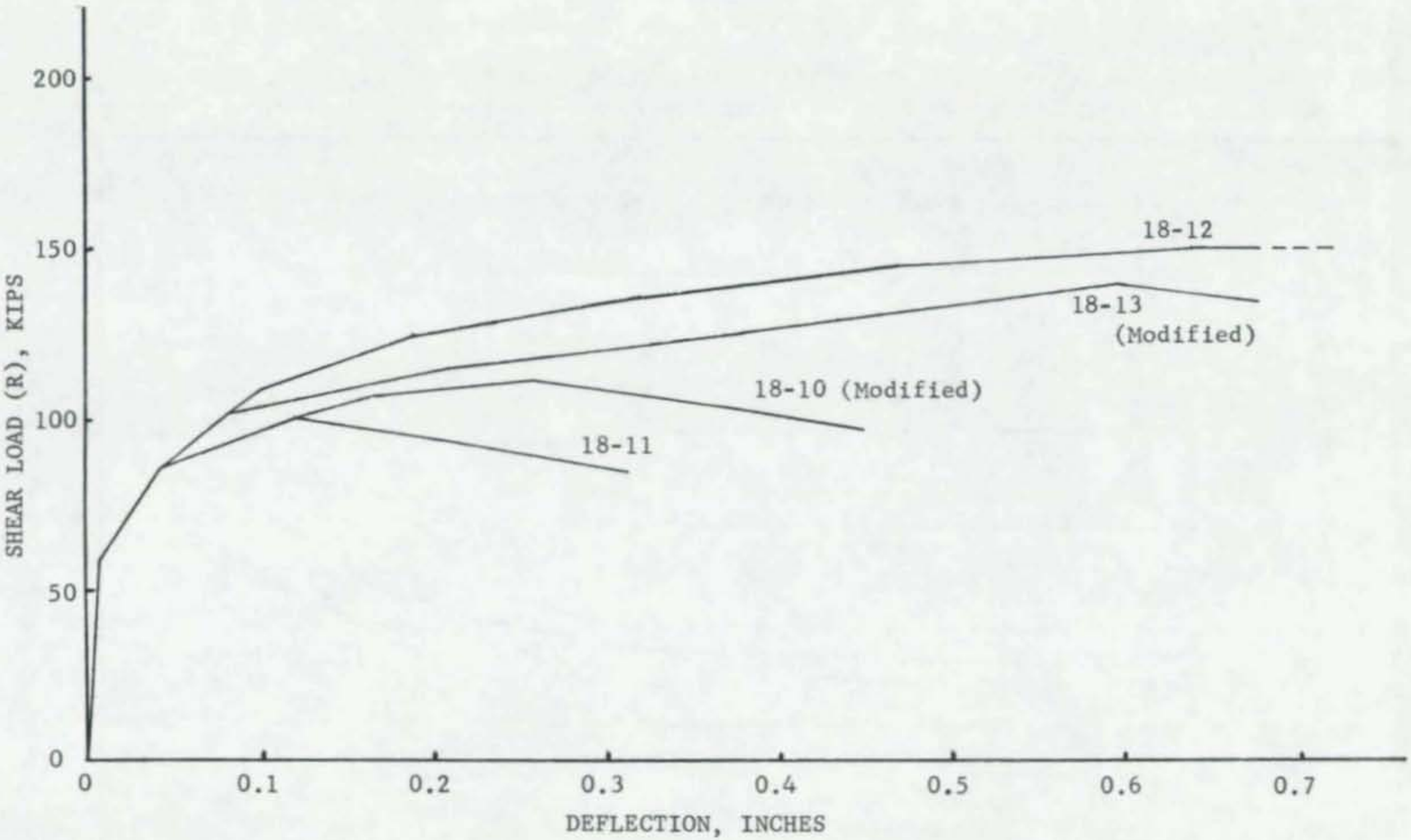



Fig. 4.6 Reaction-Deflection Response Curves, Double-Row Connections (Texas)

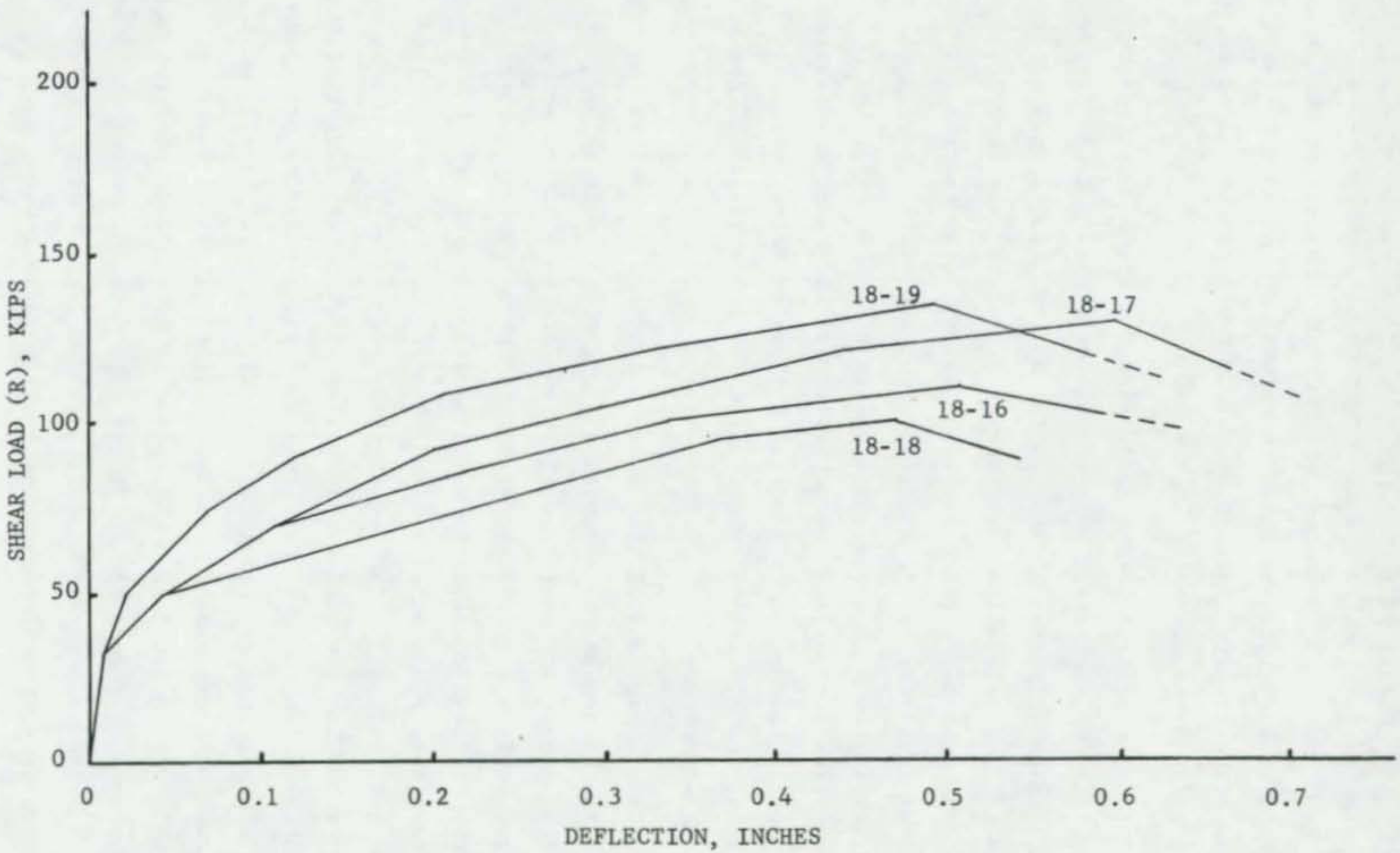

SHEAR LOAD (R), KIPS
200
150
100
50
0
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
DEFLECTION, INCHES
18-19
18-17
18-16
18-18

Fig. 4.6 (cont.)

from unplanned fabrication and construction differences as was discussed in Chapter 3. The modified curves are labelled on the plots.) The 1/4 in. load values are shown in columns 10 and 8 of Tables 4.1 and 4.2, respectively. Factors of safety based on these load values were calculated and are shown in the tables.

The comparisons of the deflection criterion and strength criteria for the single-row connections are shown in Table 4.1. Using the deflection criterion of 1/4 in. for $R_u$ (reaction at ultimate), the controlling allowables gave factors of safety very near 2.0 except for those cases where overall shear yielding controlled. This level of safety is the generally accepted minimum margin of safety for fracture in connections and greater than 1.67, the accepted value for deflection controlled failures such as yielding. When general yielding controlled the factors of safety were all above 1.44, the minimum allowed by AISC for shear yielding, and therefore acceptable.

The comparisons of the deflection criterion against the test results of the double-row connections were not as successful suggesting a need for more in-depth study of those connections. In preparing Table 4.2 (which contains the double-row test comparisons), the plots of these connections were carefully examined. It was recognized that the general configuration of the curves described a reduced stiffness much like for slotted holes in single-row connections. Recalling the earlier discussion where it was suggested that for slotted holes the web shear area over which yielding occurred was not as concentrated as for standard holes, but that yielding must occur over the length of the slot, a similar situation was recognized for double-row connections. The spreading of the load over two bolt-rows appeared to reduce the stiffness much like for slots. A 15 percent reduction was therefore applied to double-rows also. (Double-row connections with slotted holes require to be reduced only once as observed from the results of test 18-11.) Table 4.2 includes that reduction ($S_r$ = 0.85). The reduction produced very satisfactory results when compared to ultimate load levels (see F.S.($R_u$), Table 4.2, which are greater than 2.0). However, factors of safety for the 1/4-in. deformation loads were in some cases up to 16 percent below the F.S. of 2 0 (see F.S.($R_u$ 1/4), Table 4.2), but were greater than or equal to 1.67, the factor of safety related to large deformations. The 15 percent reduction and the eccentricity considerations for double-row connections appear to have some merit, but need further study.

Design Recommendations

Throughout this study a great number of variables and other criteria were considered. Some of them appeared to be independent, but mostly they were interrelated, each affecting the other to some degree. An effort was made to recognize many of the variables and to explain their effects and wherever possible, to include some method to account for their effects in a design recommendation. It is re-emphasized that a number of points in the design method merit further investigation. These and other points will be mentioned in the next chapter.

Table 4.1 contains values developed from the design criteria and the factors of safety for the single-row connections discussed earlier. Figure 4.7 is a comparison of the theoretical strength (from the controlling allowable in Table 4.1) to the ultimate load capacity for each test. (Table 4.2, containing the allowables for the double-row connections, shows only the block-shear allowables as they were of paramount importance in those connections.) The design recommendations are based on the allowable load approach used in the present AISC Specifications.[15] Design example 1 on page 4-16 of the AISC Manual[9] is done using the method recommended herein and is shown at the end of this section.

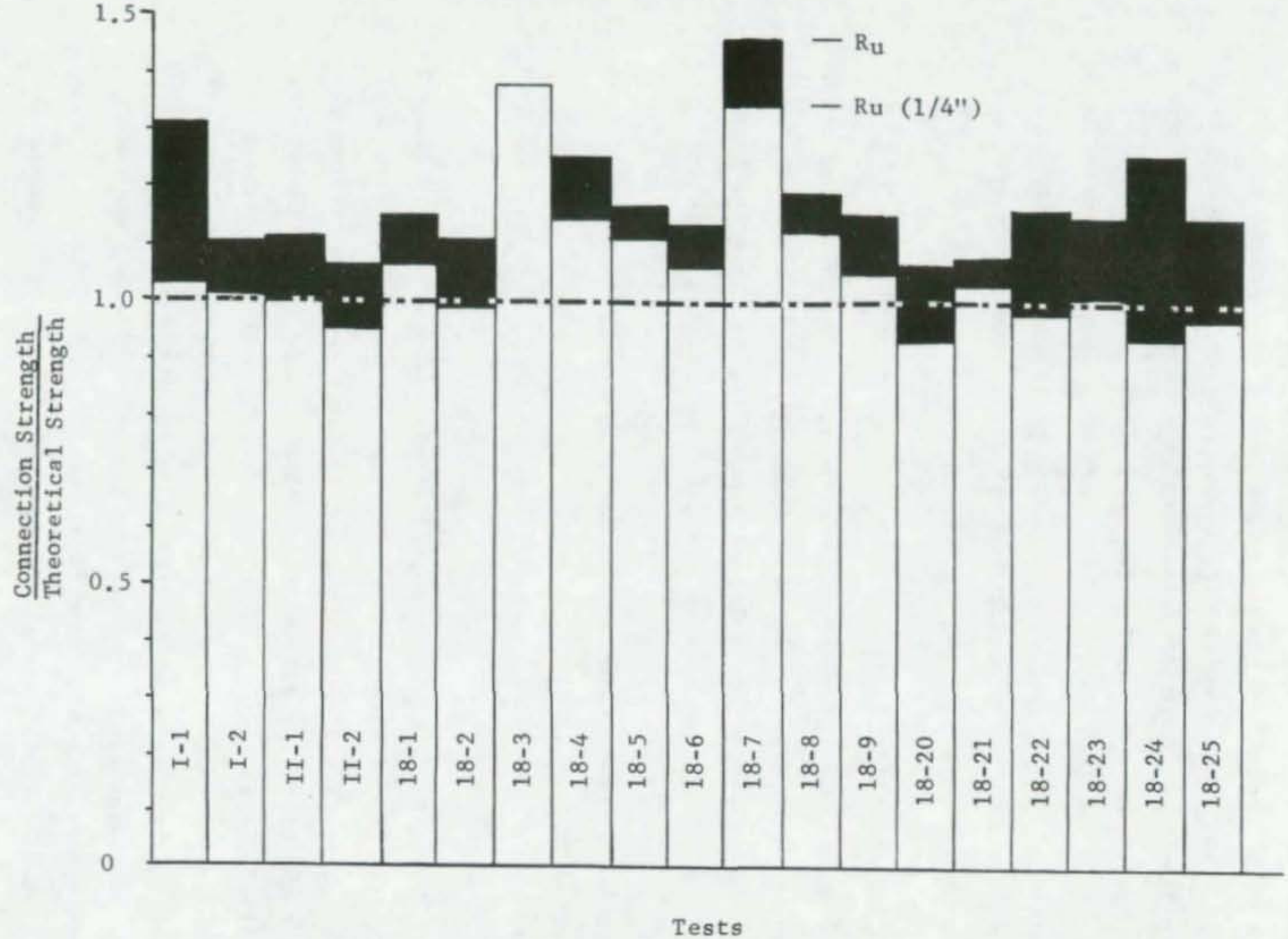



Fig. 4.7 Comparison of Theoretical Strength to Connection Ultimates

(1) For all cases the allowable web shear must be checked following the equation

$$0.4F_y(A_v). \tag{4.8}$$

This equation is that given for web shear in the AISC Specifications (Section 1.5.1.2) except that for long slotted holes it is recommended that $A_v$ be based on the net area, because the reduced area through the slots can cause increased deflections. For short slots perhaps it may be acceptable to use the gross area, but no data are currently available to define the precise variations. Until tests on short slots are conducted, it is recommended that the reduced area also apply to short slots. Further tests are recommended to better understand the effects of slots especially for this section.

(2) Allowable bolt or hole capacity must also be checked. For this type of connection, the allowable can be calculated by summing the lowest individual bolt or hole allowables. The AISC Specifications provide charts and tables for the determination of bolt-shear allowables and with yield and ultimate values for various grades of structural steels. Bolt shear allowables are covered in Section 1.5.2 while allowable bearing is covered in Section 1.5.1.5 of AISC. (Special note: a distinction is suggested to be made between friction and bearing bolt shear allowables and is demonstrated in the design example.)

Also to be included in this check are spacing and edge distance in the direction of the load from the bolts (the edge to the cope). Sections 1.16.4 and 1.16.5.2 of AISC deal with these requirements and need to be checked. A block-shear type check at the top bolt hole as shown in Fig. 4.4 for the minimum failure surface,

$$F_{min} = \left[0.3F_y\left(A_{v_{grs}}\right) + 0.5F_u\left(A_{t_{net}}\right)\right](S_r), \tag{4.6}$$

should be made in addition to other applicable checks. No reduction in bolt or hole strength due to eccentricity is recommended for single-row connections. The factors of safety for tests 18-9 and 18-20 through 18-25 as well as the Toronto tests are satisfactory without this consideration. Elimination of the end distance formula (Section 1 16.5.3, AISC) is suggested, but as the edge and end distances used in the tests were at minimum or better (Table 1.16.5.1, AISC), it is recommended to maintain these minimum limits. For slotted holes, it is recommended that the bearing capacity and the edge distance of $F_{min}$ calculation be reduced by 15 percent because of the increased deflections observed. (The bolt shear strength is not affected by hole type.)

(3) A fracture check, the key to determining if a block-shear or net-shear failure check is applicable, should be made. To determine whether or not the end distance at the lowest bolt can fracture (recall Fig. 4.1), the following check must be made:

$$0.3F_y\left(A_{v_{dev}}\right) \geq 0.5(E_r)F_u\left(A_{t_{net}}\right). \tag{4.3}$$

In this check the effects of eccentricity were included as it was found to exhibit the greatest influence on the tensile section. The eccentricity factor ($E_r$) values, tabulated in the AISC Manual,[9] are used. The factor accounts for uneven distributions of stress along the tensile section. No reduction for slotted-hole or double-row connections are made in this check. The reduction factor, $S_r$, was developed to account for the lower stiffness of slotted-hole and double-row connections and to be used with the various applicable allowable load values. It would be unjustified to apply this factor to one side of the relation and not the other. Further studies of the occurrence of fracture should be undertaken.

(4) Block shear is checked if there is the possibility of fracture following the equation:

$$B_v = \left[0.3F_y\left(A_{v_{grs}}\right) + 0.5(E_r)F_u\left(A_{t_{net}}\right)\right](S_r) \tag{4.5}$$

In this equation a reduction for eccentricity, $E_r$, on the net tensile section is considered as well as the reduction for slotted holes, $S_r$.

A change in the calculation of the capacity of the shear portion of the equation was suggested by carefully reviewing the tests It was found that substantial displacements began to occur when a stress of $0.6F_y$ on the gross shear area of the web above the fracture line was reached. The slotted-hole tests provided evidence that a stress of $0.6F_y$ can be the most the section will achieve in certain situations, thus it was chosen to replace the $0.6F_u$ shear stress level used in the AISC Specification block-shear formula. In Table B.1 of Appendix B a comparison is made of the allowable capacity of the shear areas using $0.3F_u$ (from Section 1.5.1.2.2 of of AISC) on the net shear area and $0.3F_y$ on the gross shear area.

(5) For standard holes in single-row connections, if there can be a fracture at the lowest hole, there may be reserve strength at small deflections as the web develops its ultimate shear capacity. The strength of the remaining shear section can be taken as:

$$N_v = 0.3F_u\left(A_{v_{net}}\right). \tag{4.7}$$

For these connections the larger of block shear ($B_v$) or <u>net shear</u> ($N_v$) should be taken and compared against the other allowables. Net shear does not apply to slotted-hole connections as the deflections are much greater and the connection effectively fails before achieving the stress increase. Neither does it apply to double-row connections for similar reasons.

Example

Beam W18x50, $t_w = 0.355$ in.

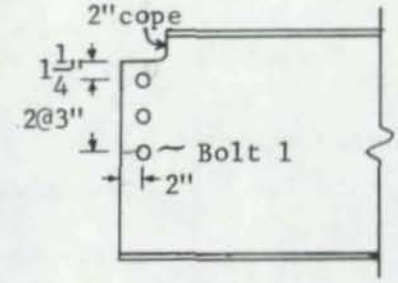
ASTM A36 ($F_y$ = 36 ksi., $F_u$ = 58 ksi.)

A325-F Bolts, 3/4 in. Diam.

Holes, 13/16 in. Diam. ($S_r = 1.0$)

Eccentricity - e = 2.5 in. ($E_r = 0.66$)

Beam Reaction = 38 kips




1. Web Shear - $0.4F_yA_v = 0.4F_y(18" - 2")(t_w)$ = 82 kips ok [Eq. (4.8)]

2. Bolt or Bolt-Hole Capacity

   Friction Capacity = 3x(17.5ksi)($2A_b$) = 46.5 kips ok

   Capacity After Slip (Note: $S_r$ is shown where applicable)

   Bolt Shear; (30.0ksi)($2A_b$) = 26.5 kips

   Bearing; $3F_u(d \cdot t_w)(S_r)$ = 46.3 kips

   Spacing; $0.5F_u(L - d/2)(t_w)(S_r)$ = 27 kips

   * Edge Distance; $0.5F_u(e_c \cdot t_w)(S_r) = 0.5F_u(1.25")(t_w)(S_r)$ = 12.9 kips

   * $F_{min} = (0.3F_yA_{v_{grs}} + 0.5F_uA_{t_{net}})(S_r)$ = 20.9 kips [Eq. (4.6)]

   where $A_{v_{grs}} = (e_c \cdot t_w) = (1.25")(t_w) = 0.444$ in.$^2$

   and $A_{t_{net}} = (e_e - \frac{\text{Hole Diam.}}{2})(t_w) = (2" - 7/16)(t_w) = 0.555$ in.$^2$

   Capacity After Slip = top bolt + other bolts

   = 12.9 kips + 2(26.5 kips) = 65.9 kips ok

3. Fracture Check - $0.3F_yA_{v_{dev}} \geq 0.5E_rF_uA_{t_{net}}$ [Eq. (4.3)]

   where $A_{v_{dev}}$ = web area below bolt 1 = $(8.75")(t_w)$ = 3.106 in.$^2$

   and $A_{t_{net}}$ = 0.555 in.$^2$ (same as for $F_{min}$)

   $33.5 \geq 10.6$ can sustain fracture, therefore, continue

4. Block Shear - $(0.3F_yA_{v_{grs}} + 0.5E_rF_uA_{t_{net}})(S_r)$ = 38.4 kips ok [Eq. (4.5)]

   where $A_{v_{grs}}$ = web area above bolt 1 = $(7.25")(t_w)$ = 2.574 in.$^2$

   and $A_{t_{net}}$ = 0.555 in.$^2$ (same as for $F_{min}$)

5. Net Shear need not be checked as block shear was ok.

This connection is controlled by its block shear capacity and is ok.

* Edge Distance is applied only to the top bolt(s) in coped connections.
$F_{min}$ is applied to the top bolt or bolt line of all end connections.

TABLE 4.1 SUMMARY OF RESULTS AND PROPOSED ALLOWABLE LOADS

| | Test Number (1) | Allowable Loads: Web Shear (K) (2) | Bolt or Hole Strength (K) (3) | Eccen. Factor (4) | Fracture Check (5) | Block Shear (K) (6) | Net Shear (K) (7) | $R_u$ (K) (8) | F.S. ($R_u$) (9) | $R_u$ 1/4" (K) (10) | F.S. ($R_u$ 1/4") (11) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Toronto | I-1 | 115 | 80 | 0.70 | + | 56 | 52 | 147 | 2.63 | 115 | 2.05 |
| | I-2 | 109 | 80 | 0.70 | + | 51 | 45 | 112 | 2.20 | 103 | 2.02 |
| | II-1 | 109 | 80 | 0.70 | + | 51 | 45 | 113 | 2.22 | 102 | 2.00 |
| | II-2 | 109 | 80 | 0.70 | + | 51 | 45 | 108 | 2.12 | 97 | 1.90 |
| Texas: Heat 1 | 18-1 | 124 | 133 | 0.83 | - | - | - | 205 | 1.65* | 188 | 1.52* |
| | 18-2 | 124 | 131 | 0.83 | + | 84 | 93 | 206 | 2.22 | 184 | 1.98 |
| | 18-3 | 107 | 120 | 0.83 | - | - | - | 212 | 1.98* | 212 | 1.98* |
| | 18-4 | 113 | 124 | 0.83 | + | 76 | 80 | 201 | 2.51 | 182 | 2.28 |
| | 18-5 | 107 | 120 | 0.83 | + | 72 | 74 | 173 | 2.33 | 164 | 2.22 |
| | 18-6s | 97 | 122 | 0.83 | + | 71 | - | 161 | 2.27 | 150 | 2.11 |
| | 18-7s | 97 | 126 | 0.83 | - | - | - | 206 | 2.12* | 188 | 1.94* |
| | 18-8s | 80 | 111 | 0.83 | + | 61 | - | 145 | 2.38 | 137 | 2.24 |
| | 18-9 | 107 | 66 | 0.87 | - | - | - | 152 | 2.30 | 139 | 2.11 |
| Heat 2 | 18-20 | 105 | 76 | 0.87 | + | 78 | 91 | 167 | 2.14 | 141 | 1.86 |
| | 18-21 | 99 | 66 | 0.87 | + | 69 | 83 | 142 | 2.15 | 136 | 2.06 |
| | 18-22 | 104 | 80 | 0.87 | - | - | - | 185 | 2.31 | 156 | 1.95 |
| | 18-23 | 105 | 68 | 0.87 | - | - | - | 157 | 2.31 | 138 | 2.03 |
| | 18-24s | 83 | 70 | 0.87 | - | - | - | 178 | 2.54 | 130 | 1.86 |
| | 18-25s | 82 | 62 | 0.87 | + | 62 | - | 142 | 2.29 | 120 | 1.94 |

* Compare to F.S. = 1.44; AISC Minimum for Web Shear
s Slotted Hole Connections
_ Controlling Allowable is Underlined

PROPERTIES:

| | $t_w$ | d | $F_y$ | $F_u$ |
|---|---|---|---|---|
| Toronto | 0.305 in. | 18.00 in. | 52.5 ksi. | 79.0 ksi. |
| Texas: Heat 1 | 0.44 in | 18.38 in. | 38.3 ksi. | 59.7 ksi. |
| Heat 2 | 0.43 in. | 18.25 in. | 36.6 ksi. | 58.0 ksi. |

TABLE 4.2 DOUBLE-ROW CONNECTION BLOCK SHEAR COMPARISON

| | Test Number | Eccen. Factor | Block Shear Proposals: $0.85[0.3F_y(Av_{grs})$ + $0.5F_u(At_{net})E_r$ | or $0.25F_u(At_{net})]$ | $R_u$ (K) | F.S. ($R_u$) col. (3) | F.S. ($R_u$) col. (4) | $R_u$ 1/4" (K) | F.S. ($R_u$ 1/4") col. (3) | F.S. ($R_u$ 1/4") col. (4) |
|---|---|---|---|---|---|---|---|---|---|---|
| | (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) | (10) |
| Heat 1 | 18-10 | 0.60 | 53 | 50 | 111 | 2.09 | 2.22 | 111 | 2.09 | 2.22 |
| Heat 1 | 18-11s | 0.60 | 46 | 44 | 101 | 2.20 | 2.30 | 101 | 2.20 | 2.30 |
| Heat 1 | 18-12 | 0.60 | 60 | 55 | 152 | 2.53 | 2.76 | 130 | 2.17 | 2.36 |
| Heat 1 | 18-13 | 0.60 | 59 | 56 | 140 | 2.37 | 2.50 | 120 | 2.03 | 2.14 |
| Heat 2 | 18-16 | 0.60 | 54 | 50 | 111 | 2.06 | 2.22 | 90 | 1.67 | 1.80 |
| Heat 2 | 18-17 | 0.60 | 60 | 55 | 131 | 2.18 | 2.38 | 100 | 1.67 | 1.82 |
| Heat 2 | 18-18 | 0.51 | 47 | 46 | 101 | 2.15 | 2.20 | 80 | 1.70 | 1.74 |
| Heat 2 | 18-19 | 0.51 | 56 | 55 | 134 | 2.39 | 2.44 | 115 | 2.05 | 2.09 |

s Slotted Hole Connection

PROPERTIES:

| | $t_w$ | $d$ | $F_y$ | $F_u$ |
|---|---|---|---|---|
| Heat 1 | 0.44 in. | 18.38 in. | 38.3 ksi. | 59.7 ksi. |
| Heat 2 | 0.43 in. | 18.25 in. | 36.6 ksi. | 58.0 ksi. |

# CHAPTER 5

## CONCLUSIONS

The main objective of this study was to investigate problem areas that had arisen in the design of bolted shear web connections and to arrive at a more comprehensive approach for their design. The tests of end connections that were performed provided a wealth of data, from instrumentations and observations alike, much of which was used in the development of the design approach presented. Some of the information generated could not be addressed in the main body of this report, but can be found in Appendix C. (Appendix A contains beam end measurements and load-deflection curves of the connections tested. Within the information covered in Appendix C are observations about flange effects, eccentricity, bearing, and moment-rotation characteristics. A comparison of the proposed allowable values for the shear portion of the block-shear equation to those in the AISC code[15] is given in Appendix B.)

In developing a design method, it was recognized that the beam end connection problem was like a puzzle with some pieces out of place and several missing. In the new approach for design which was developed, it is recognized that all of the pieces have not yet been found and that some may still be out of place, but the concepts of the approach seem promising.

As shown in Appendix C, in these single-row connections, there was only a small amount of rotational restraint. (The inflection point was located between 1-1/2 and 3 in. from the face of the column stub.) Even though eccentricity was practically negligible for these connections, the use of the eccentricity factor, placed in the tensile end section strength calculation, was the key to a new generalized block-shear model given by Eq. (4.5). Its use in this manner appears to be promising. Observations of the double-row connection tests with respect to the eccentricity factors and other reductions suggest strongly the need for more in-depth observations. The double-row connections tested were relatively compact, sustaining large rotations of the connected parts. Rotations and eccentricity of double-row connections merit further investigation. The effects of variations in the length of slotted holes and the use of slots in double-row connections require further study. (Only one double-row slotted-hole test was made.)

On the question of bolt and bolt-hole capacity, where the AISC Specification requirements dealt very harshly with edge and end distance, it was found that the end distance formula (Eq. (3.4)) did not provide an appropriate check. One consideration for the end distance in tension was made at the top hole location where a minimum failure plane could control as given by Eq. (4.6). The edge distance formula (Eq. (3.3)) was found to be satisfactory when used to determine the top bolt-hole capacity. It can be concluded that the sum of the individual bolt or bolt-hole capacities provide a more realistic value for the allowable load than using the lowest single-bolt value for all bolts as currently suggested in the AISC Manual.[9] It was found that the allowables for bolts and holes (other than end distance), shear, bearing, spacing and edge distance, were applicable checks. Generally, the end distance factor was accounted for in the fracture and block-shear formulas.

The Fracture Check (Eq. (4.3)) provides a method by which to determine if there can be a block-type failure of the connection. It eliminates calculations of additional checks when they are not necessary. The Block-Shear Equation developed provides an improved approach to account for such a failure mode. Finally, the Net-Shear Equation allows the designer to take advantage of reserve capacity in web shear for standard hole connections after fracture has occurred. It is noted here that the Net-Shear Equation was developed from static tests under temperate conditions. Although the fracture occurred in the end distance, occasionally there was some fracturing further into the web. Under severe climactic conditions or cyclic loadings there could be some problems. Further study is recommended.

The design recommendations for bolted shear web connections are summarized as follows:

(1) Web Shear

$$0.4F_y(A_v) \tag{4.8}$$

(2) Bolt or Bolt-Hole Capacity

Friction Capacity

All Bolts (shear)

Capacity After Slip

Top Bolt

(a) Shear

(b) Bearing $(S_r)$

(c) Edge Distance $(S_r)$

(d) $$F_{min} = \left[0.3F_y\left(A_{v_{grs}}\right) + 0.5F_u\left(A_{t_{net}}\right)\right](S_r) \tag{4.6}$$

Other Bolts

(a) Shear

(b) Bearing $(S_r)$

(c) Spacing $(S_r)$

(3) Fracture Check

$$0.3F_y\left(A_{v_{dev}}\right) \geq 0.5(E_r)F_u\left(A_{t_{net}}\right) \tag{4.3}$$

If the relation (Eq. (4.3)) is true and fracture can occur, check the following:

(4) Block Shear

$$\left[0.3F_y\left(A_{v_{grs}}\right) + 0.5(E_r)F_u\left(A_{t_{net}}\right)\right](S_r) \tag{4.5}$$

(5) Net Shear

$$0.3F_u\left(A_{v_{net}}\right),\ \text{standard holes only} \tag{4.7}$$

$E_r$ is an eccentricity reduction factor (instantaneous-center of rotation-method).

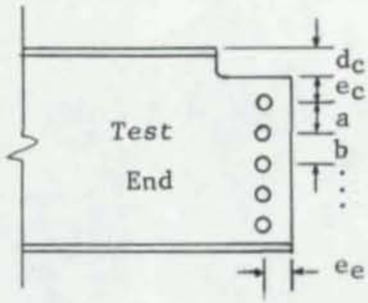
$S_r = 0.85$, reduction factor applied to slotted holes and double-row connections.

A P P E N D I X   A

CONNECTION PROPERTIES AND LOAD-DEFORMATION BEHAVIOR

TABLE A.1

MEASURED DIMENSIONS




Heat 1: $t_w$ = 0.44 in. $F_y$ = 38.3 ksi.
d = 18.38 in. $F_u$ = 59.7 ksi.

| Test Specimen Number | $d_c$ (in.) | $e_c$ (in.) | Bolt Centerline Spacing a (in.) | b (in.) | c (in.) | d (in.) | $e_e$ (in.) | $R_u$ (K) |
|---|---|---|---|---|---|---|---|---|
| 18-1 | - | 3.50 | 3.00 | 3.00 | 3.00 | 3.00 | 1.94 | 205 |
| 18-2 | - | 3.50 | 3.00 | 2.94 | 3.00 | 3.00 | 0.94 | 206 |
| 18-3 | 2.44 | 1.00 | 3.00 | 2.94 | 3.06 | 3.00 | 1.94 | 212 |
| 18-4 | 1.50 | 1.88 | 3.00 | 2.94 | 3.00 | 3.00 | 1.00 | 201 |
| 18-5 | 2.44 | 1.06 | 3.00 | 3.00 | 3.00 | 2.94 | 1.00 | 173 |
| 18-6s | - | 3.50 | 3.00 | 3.00 | 2.94 | 2.94 | 0.94 | 161 |
| 18-7s | - | 3.50 | 3.00 | 2.97 | 3.00 | 2.97 | 1.88 | 206 |
| 18-8s | 2.50 | 1.06 | 3.06 | 2.94 | 3.00 | 3.00 | 0.94 | 145 |
| 18-9 | 2.50 | 1.00 | 5.94 | 5.94 | - | - | 1.94 | 152 |

Heat 2: $t_w$ = 0.43 in. $F_y$ = 36.6 ksi.
d = 18.25 in. $F_u$ = 58.0 ksi.

| Test Specimen Number | $d_c$ (in.) | $e_c$ (in.) | Bolt Centerline Spacing a (in.) | b (in.) | $e_e$ (in.) | $R_u$ (K) |
|---|---|---|---|---|---|---|
| 18-20 | 1.38 | 2.06 | 5.94 | 6.19 | 1.44 | 167 |
| 18-21 | 2.38 | 1.06 | 5.94 | 6.06 | 1.06 | 142 |
| 18-22 | 1.44 | 2.16 | 6.00 | 6.06 | 1.94 | 185 |
| 18-23 | 1.38 | 1.22 | 5.94 | 6.00 | 2.03 | 157 |
| 18-24s | 1.38 | 2.13 | 6.00 | 6.13 | 1.88 | 178 |
| 18-25s | 1.50 | 2.00 | 6.00 | 6.13 | 1.03 | 142 |

s Slotted Hole Connections

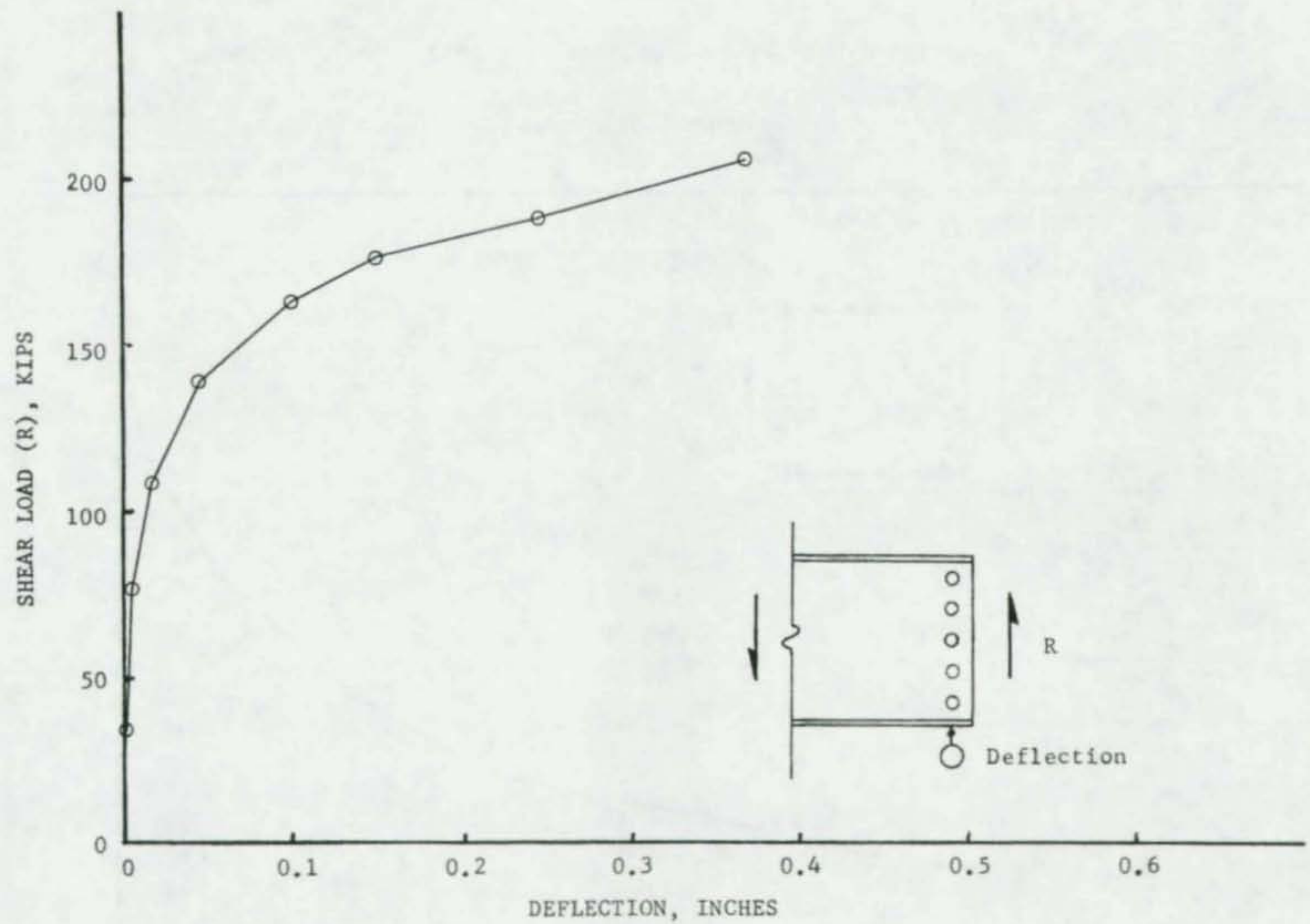



Fig. A.1 Test 18-1; 2 in. End Distance

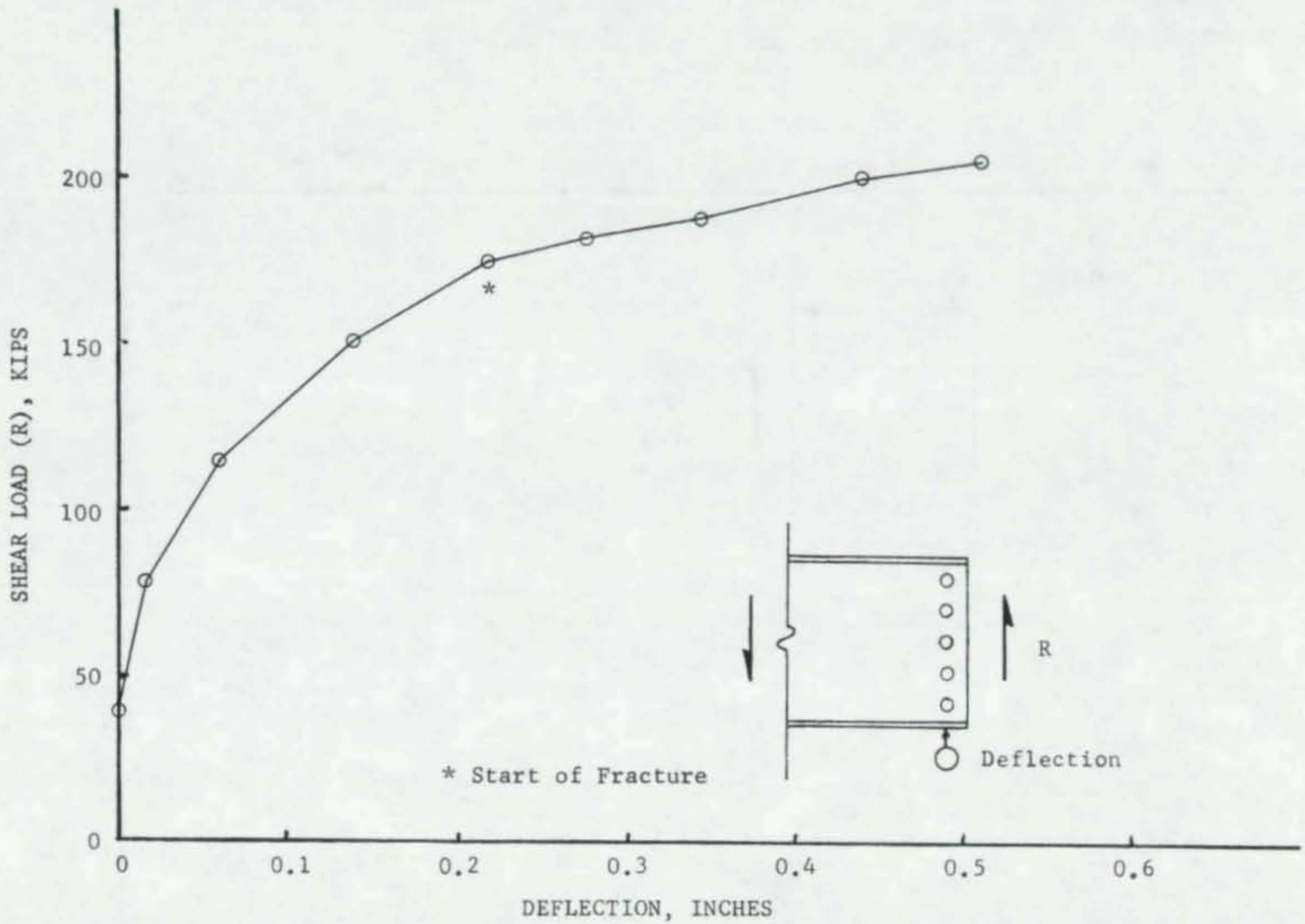



Fig. A.2 Test 18-2; 1 in. End Distance

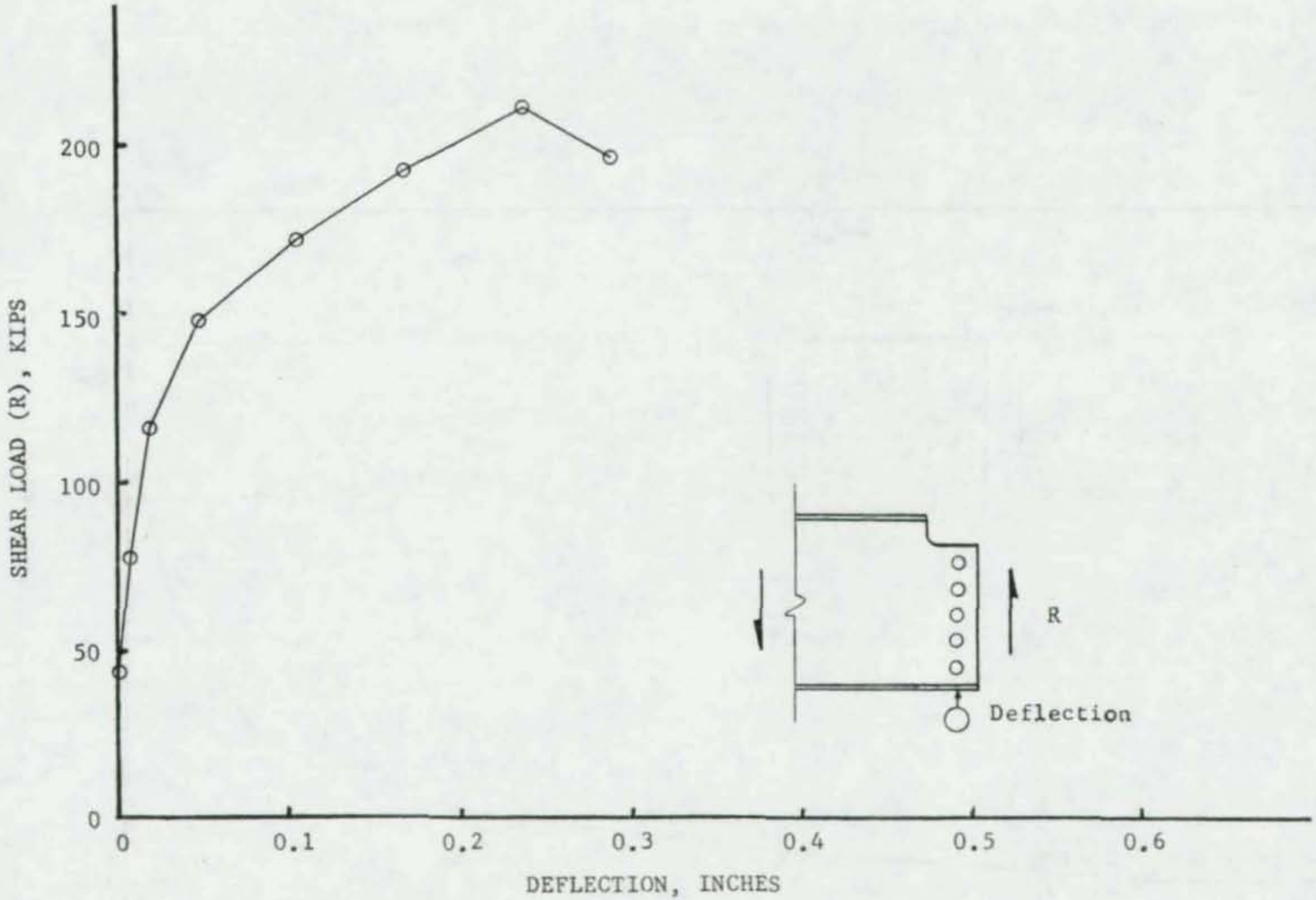



Fig. A.3 Test 18-3; 1 in. Edge, 2 in. End Distance

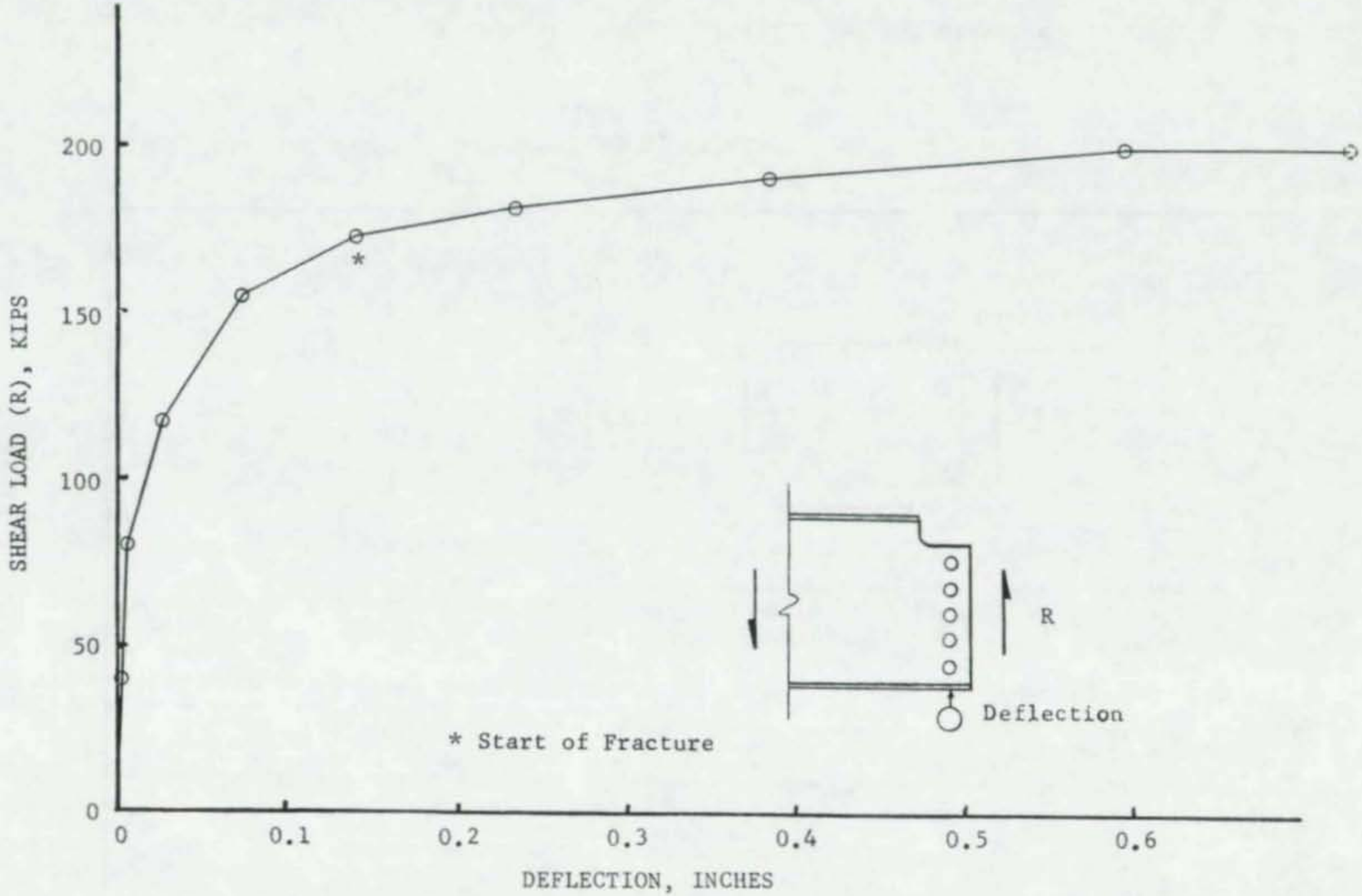



Fig. A.4 Test 18-4; 2 in. Edge, 1 in. End Distance

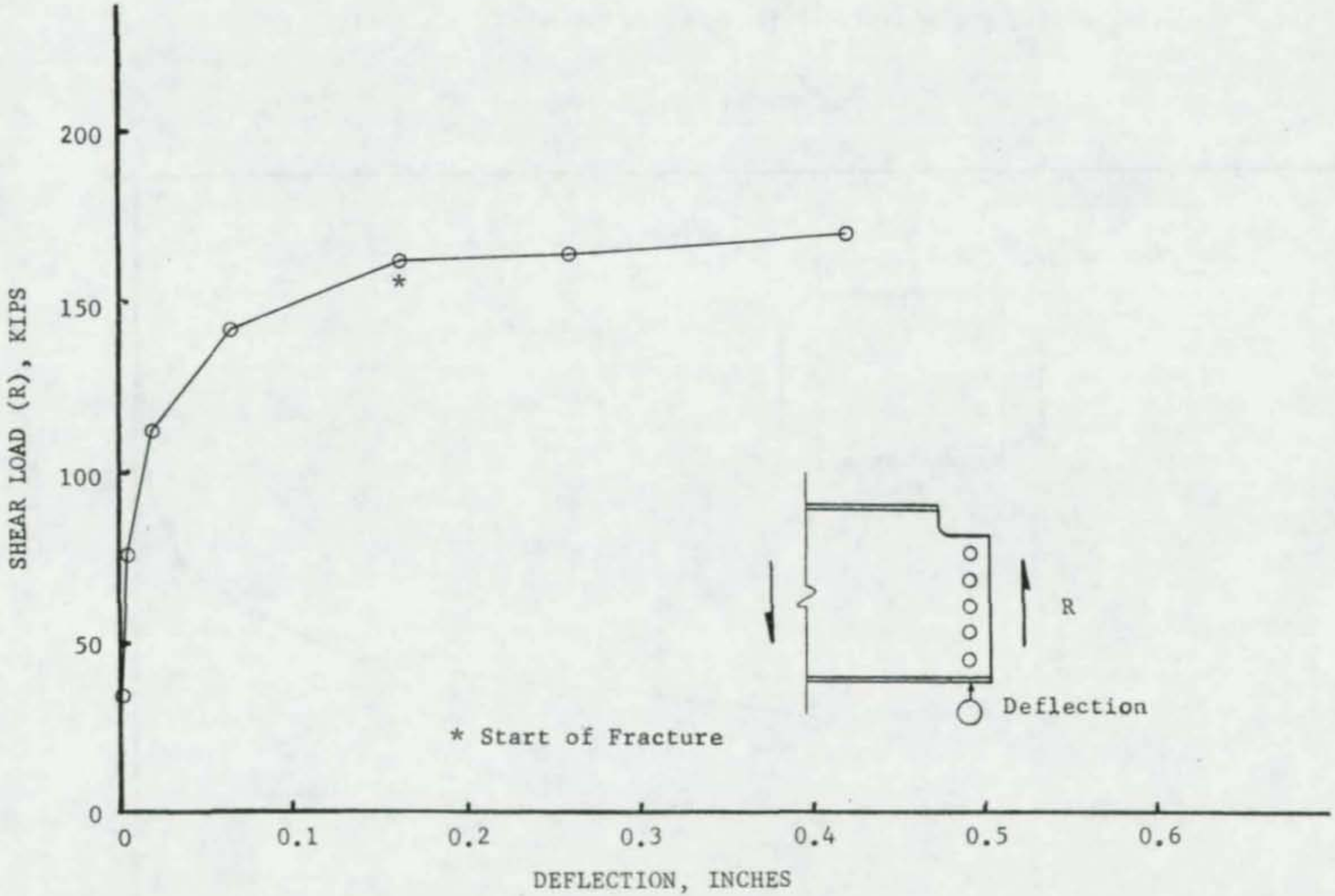



Fig. A.5 Test 18-5; 1 in. Edge, 1 in. End Distance

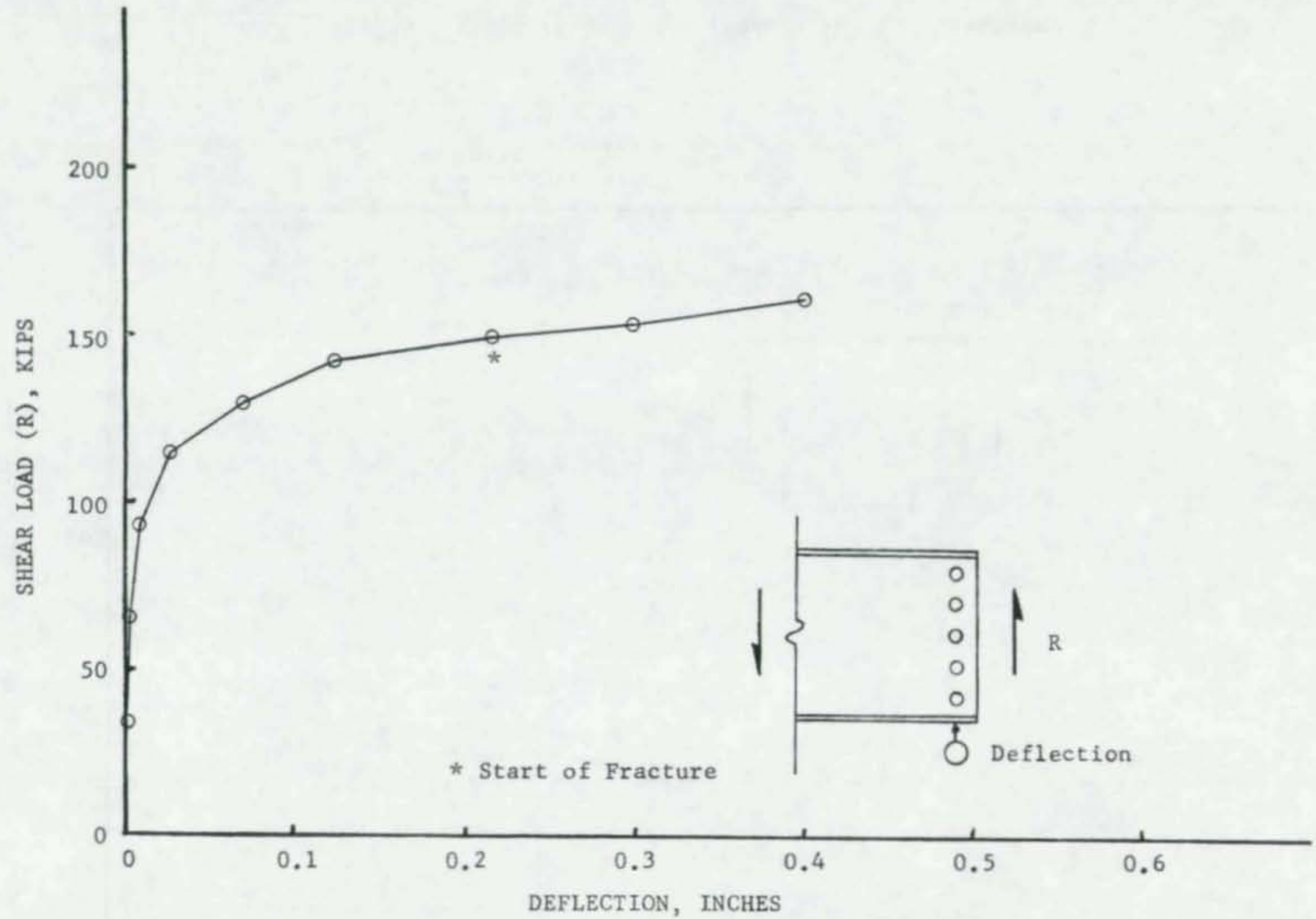



Fig. A.6 Test 18-6; Slotted Holes, Minimum End Distance

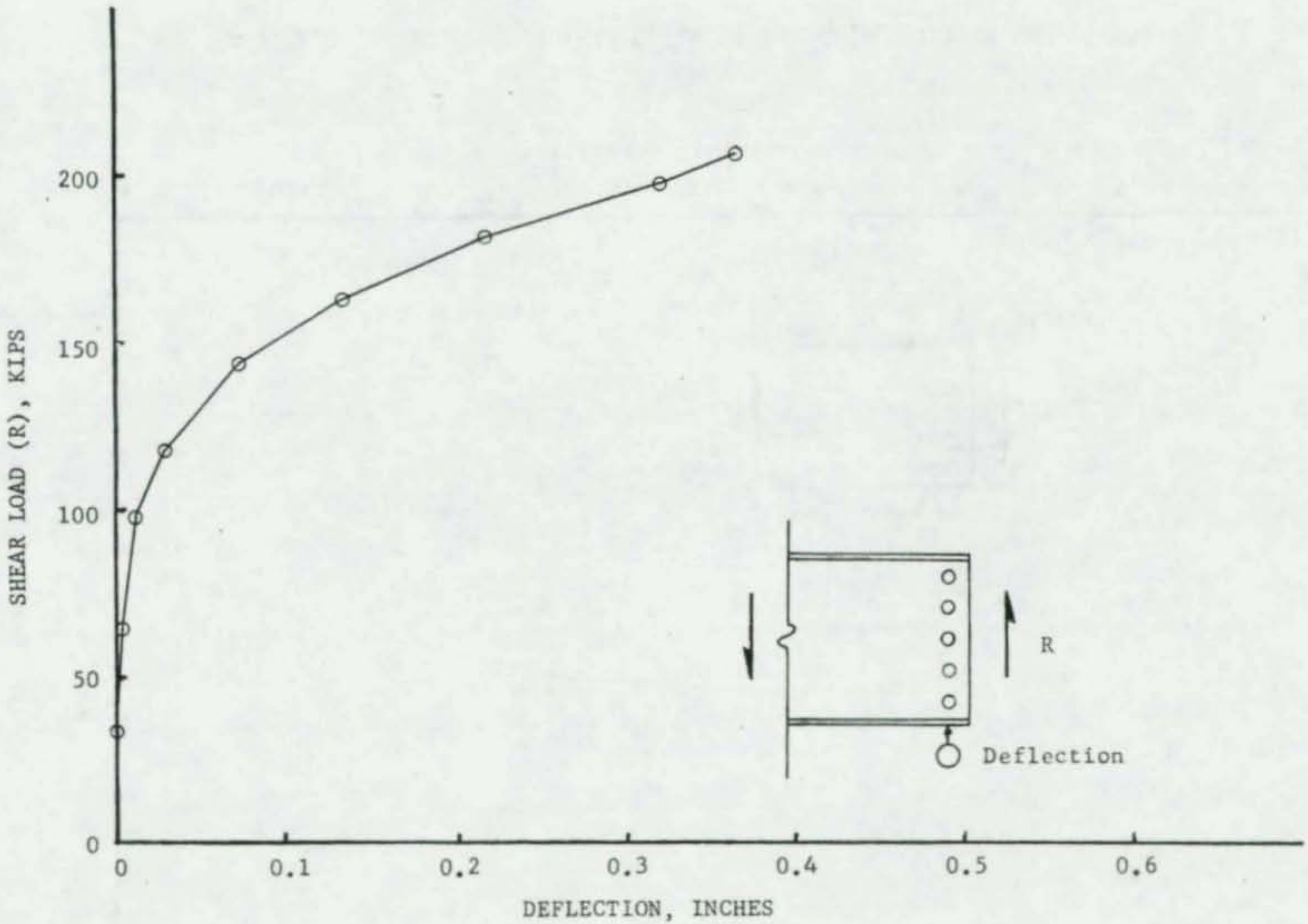



Fig. A.7 Test 18-7; Slotted Holes, Large End Distance

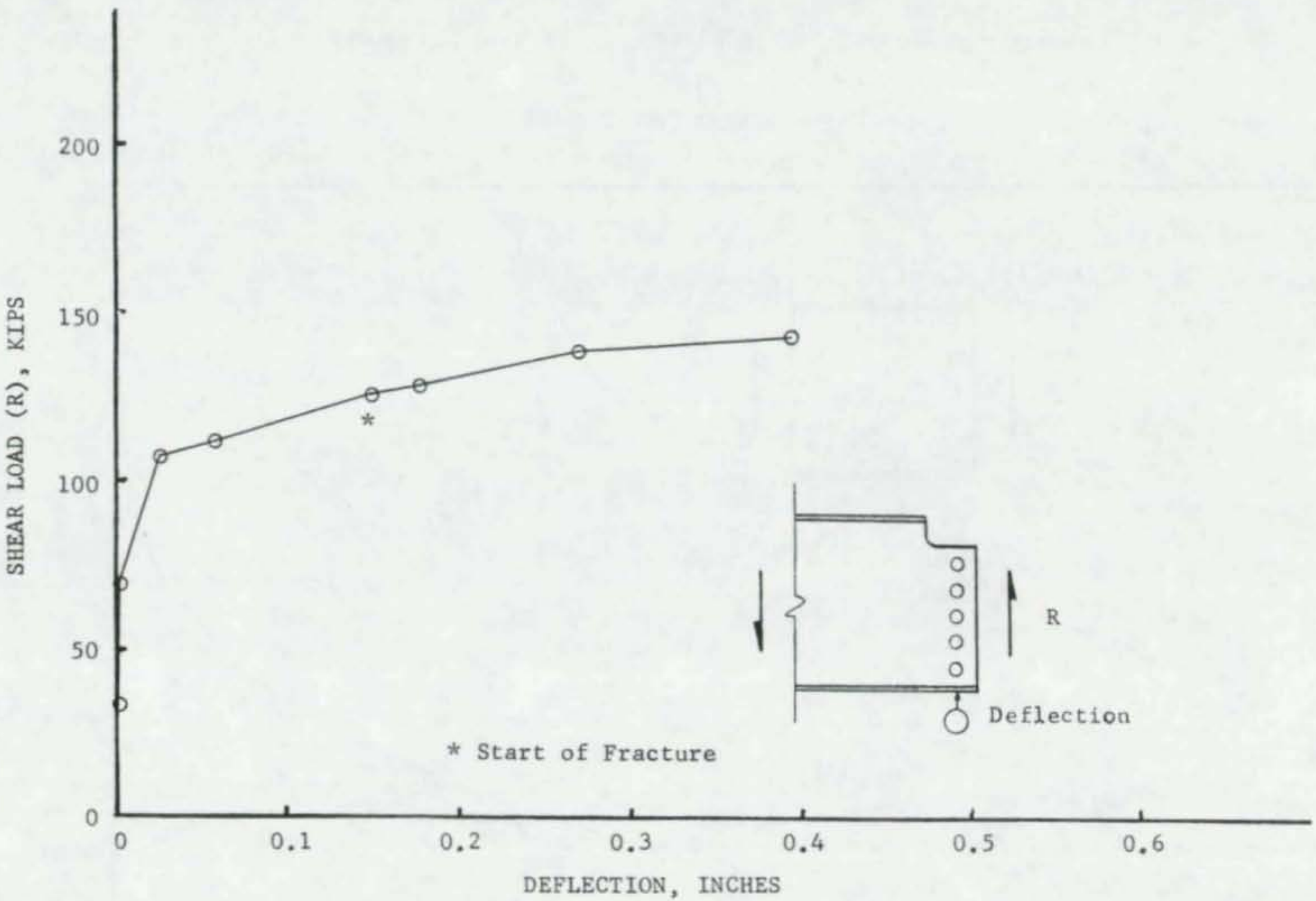



Fig. A.8 Test 18-8; Slotted Holes, 1 in. Edge, Minimum End Distance

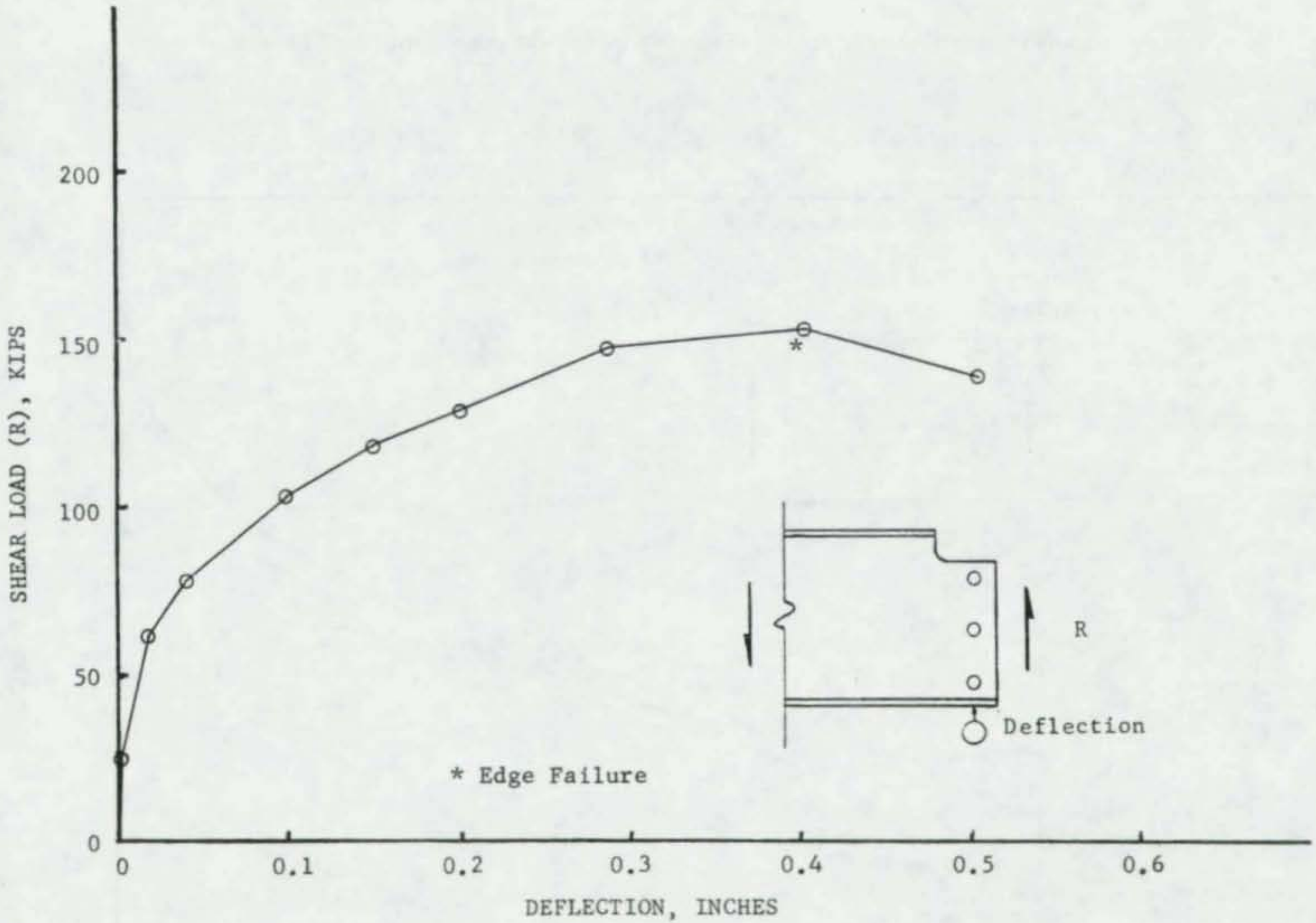



Fig. A.9 Test 18-9; 1 in. Edge, 2 in. End Distance

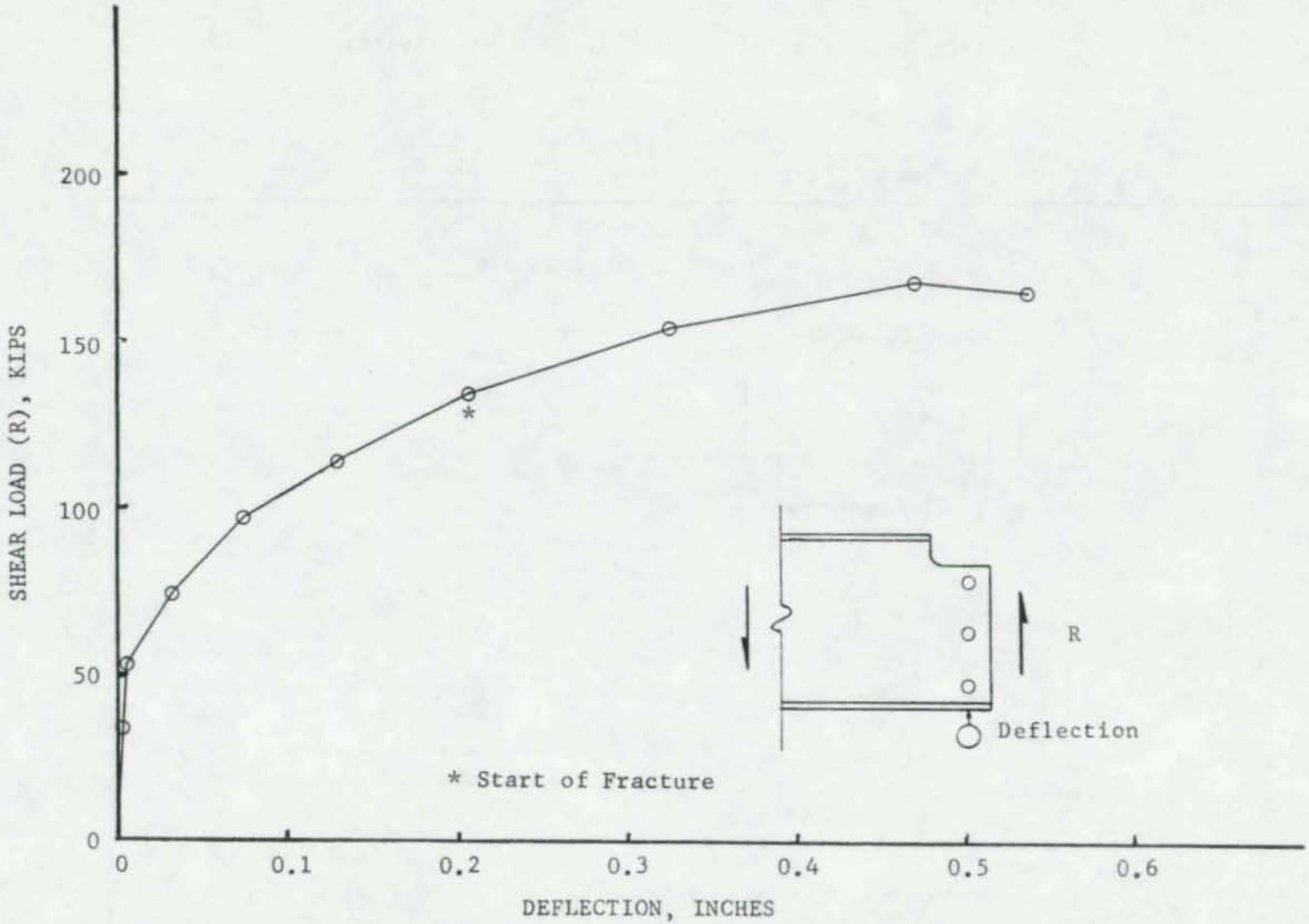



Fig. A.10 Test 18-20; 2 in. Edge, 1-1/2 in. End Distance

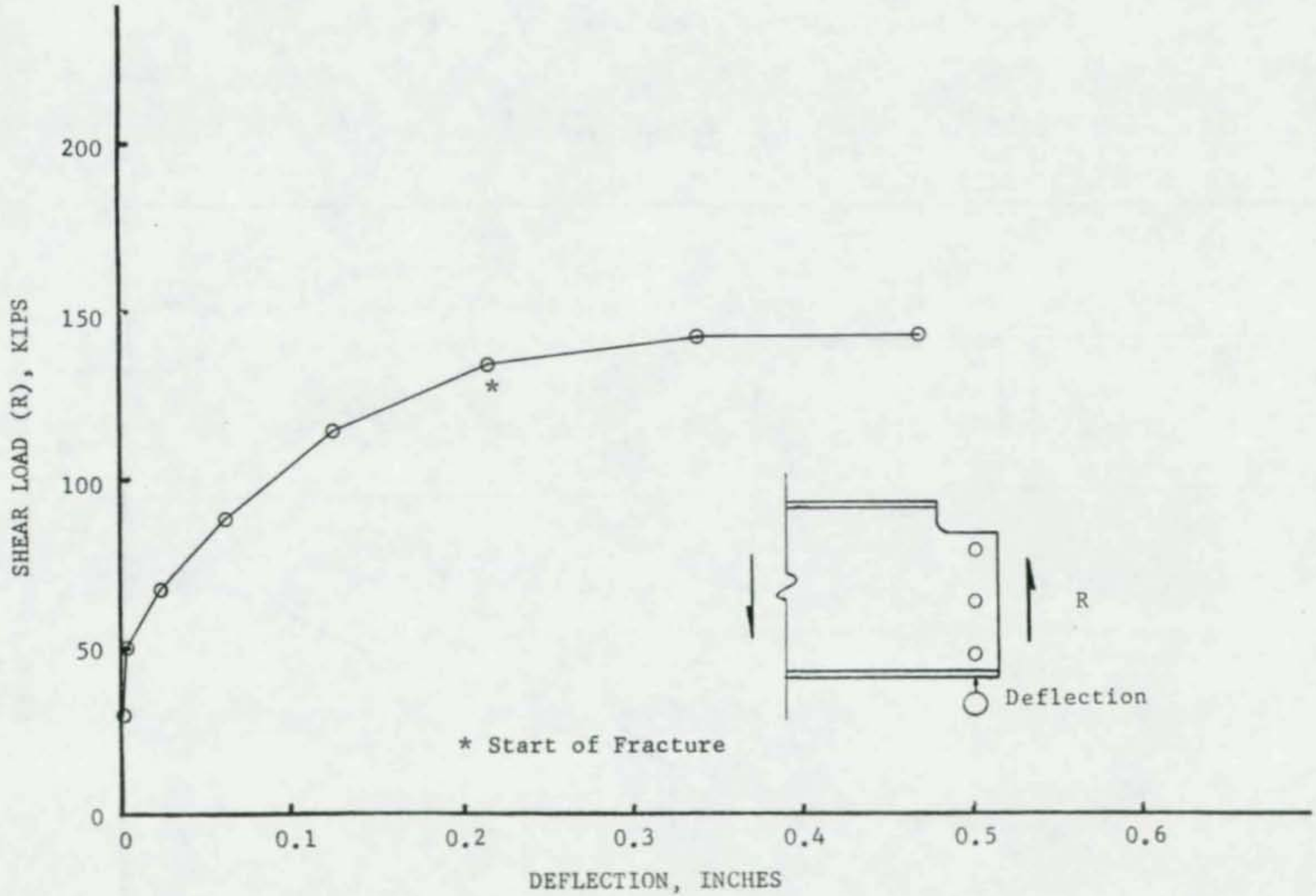



Fig. A.11 Test 18-21; 1 in. Edge, 1 in. End Distance

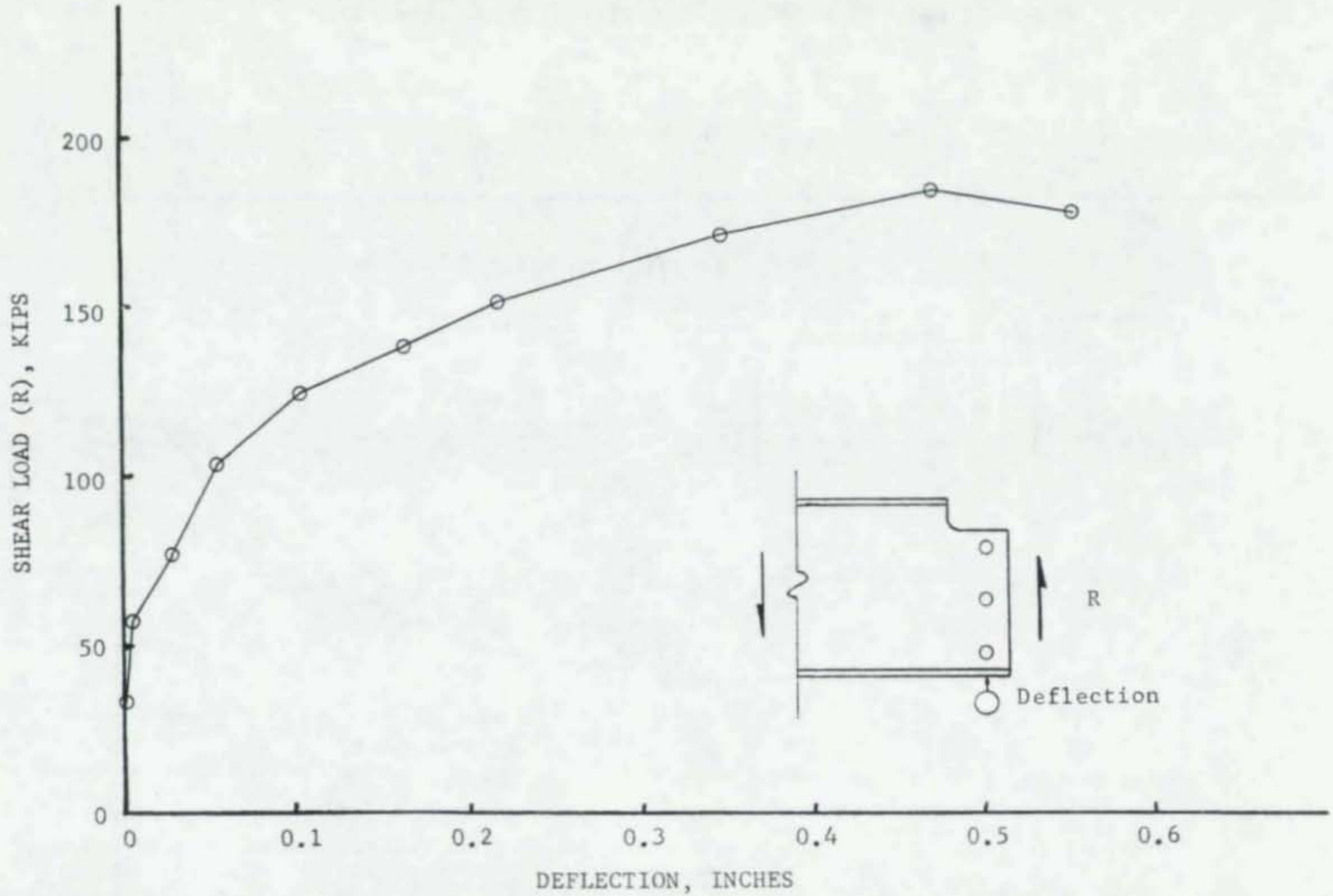



Fig. A.12 Test 18-22; 2 in. Edge, 2 in. End Distance

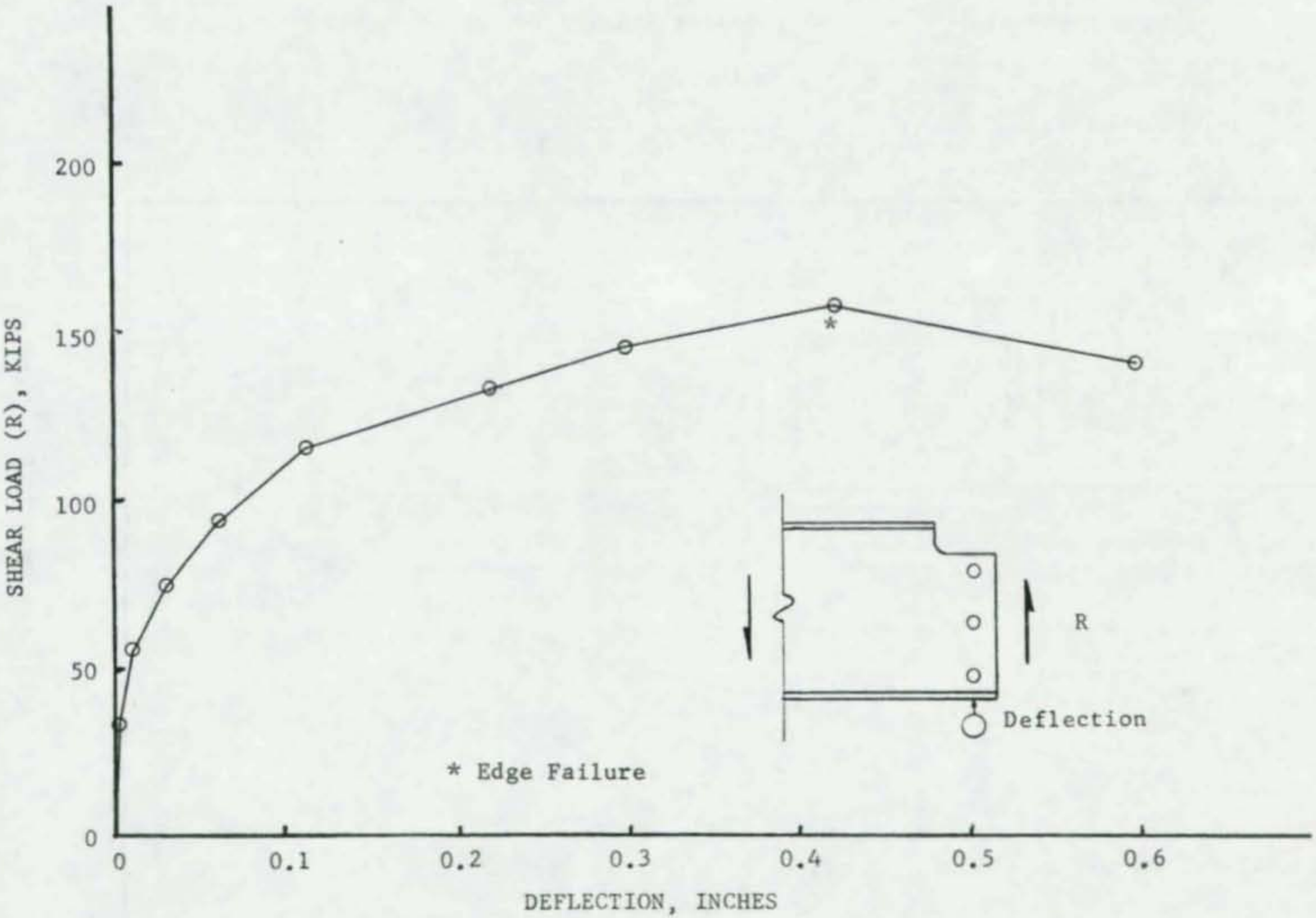



Fig. A.13 Test 18-23; 1 in. Edge, 2 in. End Distance

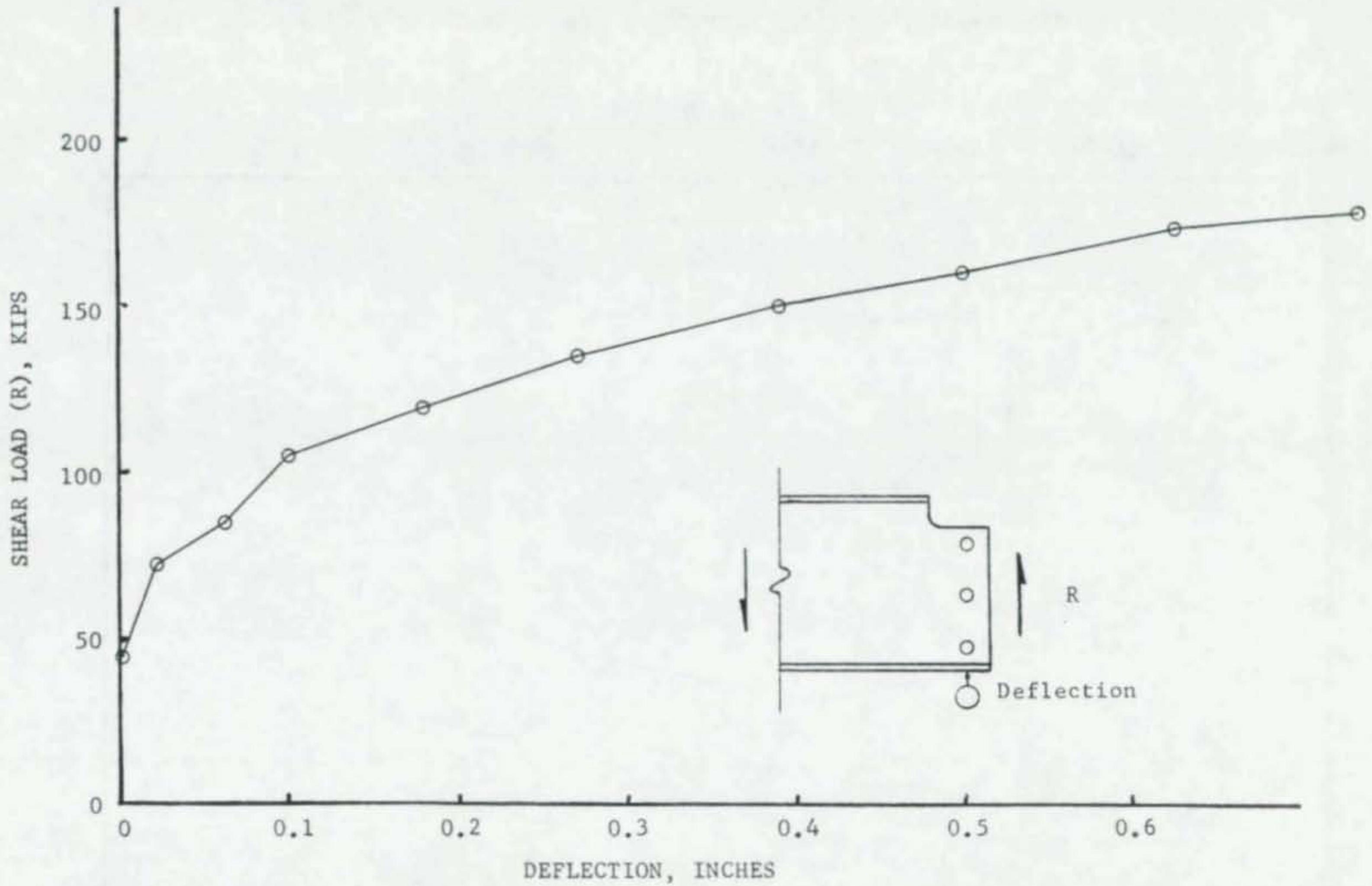



Fig. A.14 Test 18-24; Slotted Holes, 2 in. Edge, Large End Distance

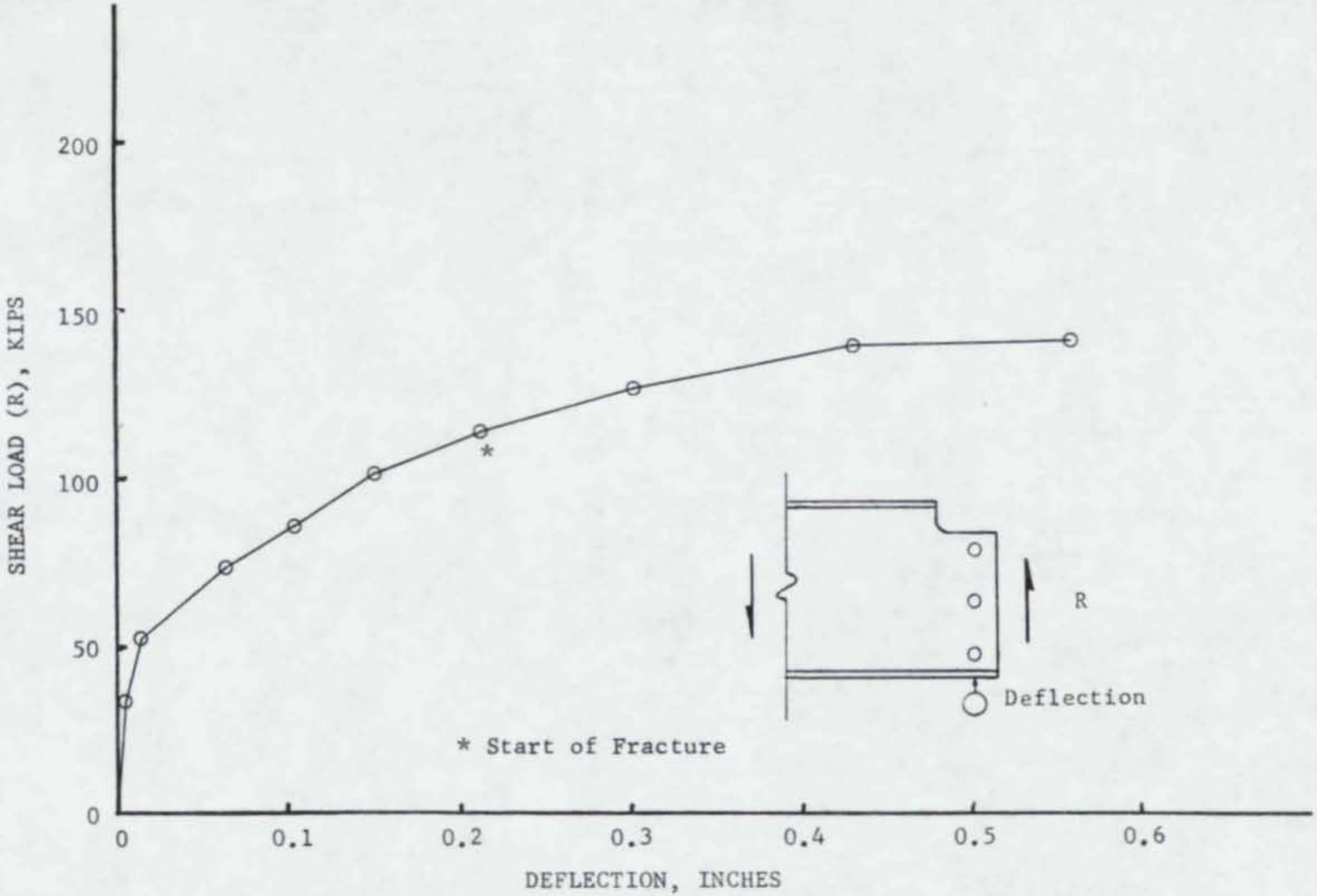



Fig. A.15 Test 18-25; Slotted Holes, 2 in. Edge, Minimum End Distance

# A P P E N D I X   B

## SHEAR COMPARISON

Shear Comparison

The values for the allowable shear strength of the web in the block-shear requirements of the current AISC method[9] (based on $F_u$) and of the proposal herein (based on $F_y$, Eq. (4.4)) are tabulated below (Table B.1) for comparison.

TABLE B.1 SHEAR COMPARISON

| | Test Number | Shear Comparison $0.3F_u(A_{v_{net}})$ (K) | $0.3F_y(A_{v_{grs}})$ (K) | $R_u$ (K) |
|---|---|---|---|---|
| | (1) | (2) | (3) | (4) |
| Toronto | I-1 | 52 | 44 | 147 |
| | I-2 | 45 | 40 | 112 |
| | II-1 | 45 | 40 | 113 |
| | II-2 | 45 | 40 | 108 |
| Texas: Heat 1 | 18-1 | 93 | 79 | 205 |
| | 18-2 | 93 | 79 | 206 |
| | 18-3 | 74 | 67 | 212 |
| | 18-4 | 80 | 71 | 201 |
| | 18-5 | 74 | 67 | 173 |
| | 18-6s | 92 | 79 | 161 |
| | 18-7s | 93 | 79 | 206 |
| | 18-8s | 74 | 67 | 145 |
| | 18-9 | 85 | 66 | 152 |
| Heat 2 | 18-20 | 91 | 66 | 167 |
| | 18-21 | 83 | 62 | 142 |
| | 18-22 | 91 | 67 | 185 |
| | 18-23 | 83 | 62 | 157 |
| | 18-24s | 91 | 67 | 178 |
| | 18-25s | 91 | 67 | 142 |

s Slotted Hole Connections

A P P E N D I X   C

ADDITIONAL REMARKS

Flange Effects

Paired specimens, which were identical in all respects except for one having the flange coped and the other not, were chosen for comparison. For several of these sets no significant comparisons could be made for the effects of the flange as either one or both beam specimens sustained failure by general yielding of the web rather than a failure at the connection. One set of tests, however, did fail at the connection. Slotted hole tests 18-6 and 18-8 (the latter one coped) could be used to determine the significance of coping. Both fractured at the bottom hole maintaining load after fracture, testing being stopped later due to excessive deflections at the connection.

Using the loads at 1/4 in. deflection, the uncoped beam supported 10 percent more load than the coped beam. The additional web area suggests that the uncoped beam would carry 20 percent more load This indicates that the additional web and flange area at the top of the connection is not as effective as anticipated. Additional tests directed at this factor would be needed for more conclusive results.

Fracture Location and Hole Deformations

The model in Fig. C.1 demonstrates in a simplified though not completely accurate fashion why fracture always occurred at the lowest bolt (bolt 1). If the connection acted as shown in the link model, with each bolt assumed to exert equal loads (P) at the mid-points of the links above, half of each bolt load would go to the end links. The end link at bolt 1 would carry $\frac{3}{2}$P which is less than for the other links, therefore, this link would be expected to fail first.

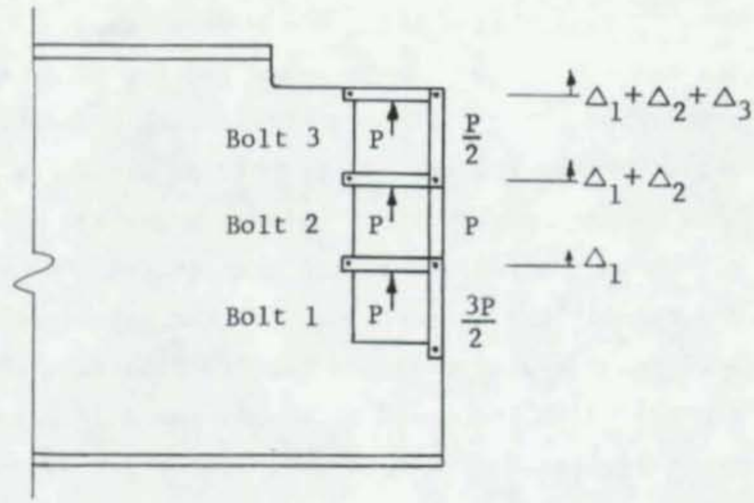



Fig. C.1 Link Model

There are many variables which alter the simple model (Fig. C.1). There must be moments at each horizontal link. The effects of coping vs. not coping could alter the distribution of the forces. Another major variable would be the stiffness of the angles. The relative position of the bolts must change to maintain an equal load P at each hole. Angles with sufficient stiffness to maintain the distance between bolts constant would cause more load to be picked up at the bottom bolt as suggested by Fig. 4.2. Further studies of the deformation patterns and load distribution are suggested.

Moment-Rotation Characteristics

From the recorded load and far-end reaction, moments at the connection were calculated and plotted against the rotations. In these tests, even though the depth of the connection angles was large, the moments developed were negligible. The sizing of the angles made the joints flexible in view of the rotations which occurred. The moments developed by the connections were generally in the range of 10 percent of the plastic moment capacity of the beam. Even for a case where the rotations of the angles and the beam end were very large (around 0.02 radians), the moments developed reached only 16 percent of the plastic moment. Shear and bearing did not present any problem in the connection angles.

For the design of simple-beam type connections, the end restraint is low so that large moments are not developed. Generally, the distance to the inflection point (e), taken as the moment divided by the reaction at the connection (M/R), ranged between 2 to 3 in. for the single-row 5-bolt connections and 1.5 to 2.5 in. for the 3-bolt connections. The single-row of bolts were located at 2.5 in. from the column face. For the 5-bolt connections, the eccentricity effects on the bolts is negligible, but for the 3-bolt connections, there is a slightly greater indication of eccentric effects.

Eccentricity

The eccentricities of the connections could not be studied effectively herein as eccentricity was not a variable. A careful observation of the hole deformations of the tested ends provided some indication about the eccentric characteristics of these connections. It was found, by estimating the direction of bearing at each hole, that the initial bearing directions indicated a center of rotation located very near that determined from the instantaneous center of rotation method.[8] As loads increased and bearing deformations increased, the center of rotation appeared to move away from the connection along the length of the beam. These general trends held true for all of the tests suggesting that the method for handling eccentricities has some merit, but further studies would have to be undertaken for an improved understanding of eccentricities.

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