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STRUCTURAL ENGINEERS INC.
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Radford, VA 24141
(703) 231-6074
(703) 731-3330

December 23, 1988

Mr. Robert Q. Disque
American Institute of Steel Construction
The Wrigley Building, 8th Floor
400 North Michigan Avenue
Chicago, IL 60611

Dear Bob:

Enclosed is a draft version of our report on the adequacy of A490 bolts in end-plate moment connections. Our conclusion has not changed: A490 bolts are acceptable in the type of connection tested. Upon receiving your comments, we will prepare the final report.

Also enclosed is an invoice for the work done. I have included the collect freight charges as we agreed in a June telephone conversation.

Sincerely,

Tom

Thomas M. Murray

$3000.00 fee
178.40 Freight
3178.40

7473

DRAFT

Report of

TESTS TO DETERMINE THE ADEQUACY OF A490 BOLTS IN MOMENT END-PLATE CONNECTIONS

Submitted to

American Institute of Steel Construction
800 N. Michigan Avenue
Chicago, Illinois
and
Metal Building Manufacturers Association
1230 Keith Building
Cleveland, Ohio

by

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Rt. 4, Box 148
Radford, Virginia

December 1988

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TESTS TO DETERMINE THE ADEQUACY OF A490 BOLTS IN MOMENT END-PLATE CONNECTIONS

INTRODUCTION

Tests of moment end-plate connections using A490 bolts have not been reported in the literature. To verify the adequacy of A490 bolts in four tension bolt, unstiffened, moment end-plate connection, four tests were conducted by Structural Engineers, Inc. using the facilities of the Prices Fork Structural Engineering Labortory, Virginia Polytechnic Institute and State University. The tests were sponsored by the American Institute of Steel Construction and the Metal Building Manufacturers Association.

To accomplish the objective, two sets of two tests each were conducted. Each set of tests used nominally identical end-plate configurations except for end-plate thickness. One set of test specimens was fabricated using A36 steel (EP36 tests) and the other using A572 Gr50 steel (EP50 tests). One test of each set was conducted using A325 bolts and the second using A490 bolts. Bolt diameter (3/4-in.) was the same for all tests which necessitated using a thicker end-plate for the A490 tests. A hot-rolled W18x50 beam was used for the EP36 tests and a built-up 18x8 beam was used for the EP50 tests. In all tests, the moment end-plate connection was subjected to pure moment (no shear).

The end-plate connections were designed using the procedure in the 8th Edition AISC Manual of Steel Construction. The EP36 end-plates were detailed with a bolt pitch (distance from face of tension flange to centerline of bolt hole) of 2-in. The EP50 end-plates were detailed with a bolt pitch of 1 1/4-in., the minimum recommended pitch for 3/4-in. diameter bolts.

End-plate geometry is shown in Figure 1 and Table 1. The test designations in Table 1 are to be interpreted as follows: EP36-1-3/4 A325-W18x50 designates a test using A36 steel, 1-in. thick end-plate, 3/4-in. diameter A325 bolts and a W18x50 beam. In the third and fourth tests, 18x8 represents an 18-in. deep built-up beam with an 8-in. wide flange. (Flange thickness was nominally 1/2-in. and web thickness 1/4-in.)

The EP36 specimens were fabricated by an AISC member company and the E50 specimens by an MBMA member company.

Test details, test results, comparisons between analytical and experimental results, and between tests using A325 and A490 bolts are found in the following sections.

TESTING DETAILS

Test Set-up

The test set-up was as shown schematically in Figure 2. The end-plates were welded to two beam segments and tested as splice connections under pure moment. The load was applied to the spreader beam (W14x109) using a hydraulic ram powered by an electric pump. The test beams were laterally supported at four locations, approximately 10-in. each side of the splice connections and at the supports. Lateral brace mechanisms which do inhibit vertical movement were used at the splice locations.

The weight of the spreader beam, the pivoting head and the other equipment on top of the test beam produced a moment of approximately 17 ft.-kips at the end-plate connection.

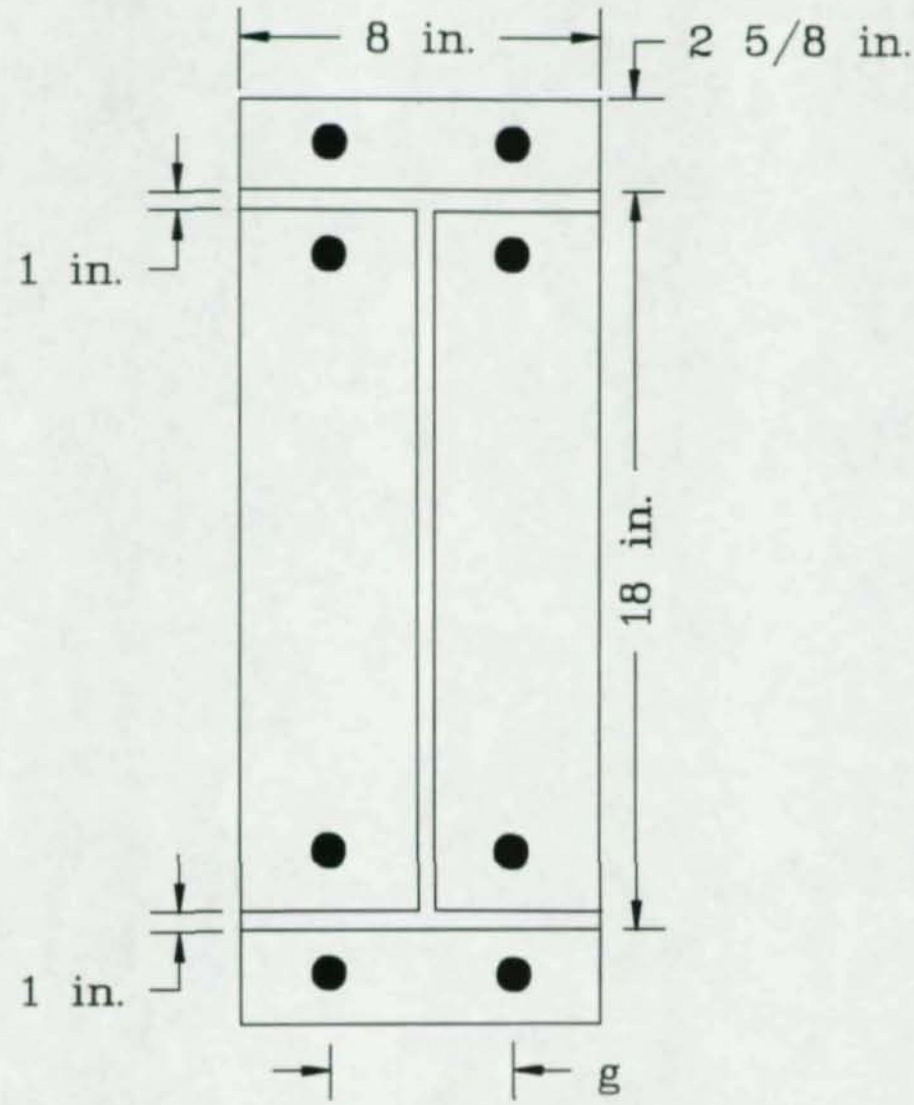

Figure 1 Geometry of End Plates Tested

Table 1

End-Plate Geometry

Test Designation	End-Plate Thickness (in.)*	End-Plate Width (in.)*	Bolt Diameter (in.)	Bolt Pitch (in.)*	Bolt Gage (in.)	Beam Depth (in.)	Flange Width (in.)
EP36-1-3/4 A325-W18x50	1.011	8 1/16	3/4	2	5	18	7 1/2
EP35-1 1/4-3/4 A490-W18x50	1.285	8 1/16	3/4	2	5	18	7 1/2
EP50-5/8-3/4 A325-18x8	0.630	8 1/16	3/4	1 3/16	3	18	8
EP50-3/4-3/4 A490-18x8	0.752	8 1/16	3/4	1 5/16	3	18	8

*Measured dimensions

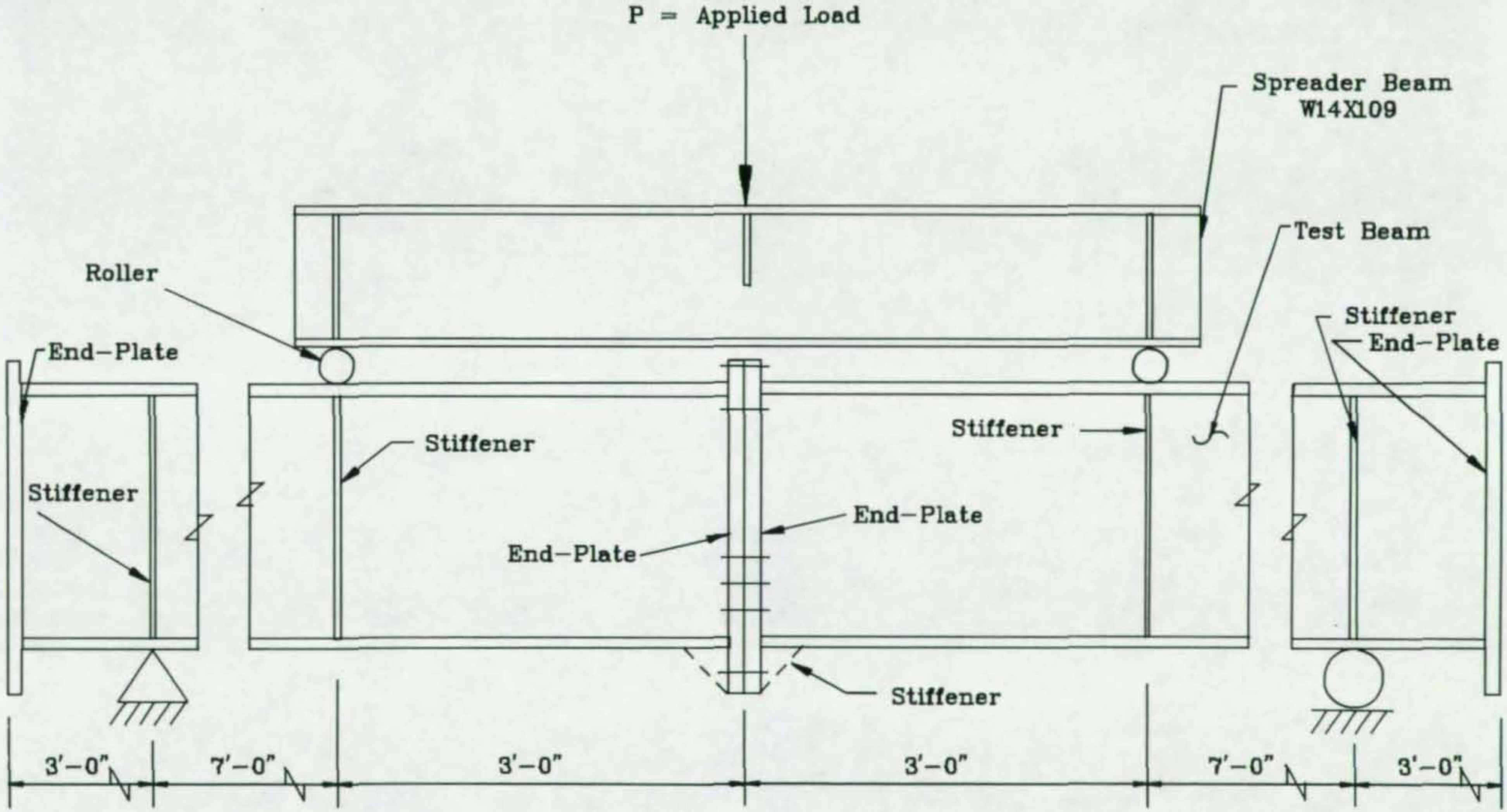

Figure Elevation of Test Set-Up

Instrumentation

Instrumentation consisted of a load cell, wire displacement transducers and instrumented bolts. A Measurements Group System 4000 data acquistion system was used to collect and record data. The 500 kips capacity load cell was used to measure the force applied by the hydraulic ram. One wire displacement transducer was placed at the midspan to measure vertical displacement directly under the splice correction. Two more were placed under the test beam reaction supports so that connections for support setttlement could be calculated.

Two instrumented bolts, one outside the tension flange and one inside the tension flange were used to monitor bolt strains resulting from the applied load. The bolts were instrumented by installing a strain gage in a small hole which had been previously drilled through the bolt head and into the unthreaded portion of the bolt shank. (The material removed was less than the difference between the gross and tensile areas of the bolt shank.) After installation of the strain gage, the bolts were calibrated using a universal testing machine.

Testing Procedures

After the test and spreader beams were placed in the loading frame, the tension flange bolts were installed and pretensioned. The instrumented bolts were connected to the data acquistion system and, by using the calibration curves, these bolts were tighten to the exact pretension level specified by AISC. The other two bolts were tightened by "feel" to the same level as that of the instrumented bolts.

At the beginning of each test, the specimen was loaded to approximately 20 percent of the expected maximum load to check the test setup and instrumentation. The load was then removed and initial readings recorded at zero load. The specimens were then loaded in varying increments, depending on the expected failure load of each test, until failure of the connection occurred. Data from all instrumentation was recorded at each increment. Failure in all tests was rupture of the test bolts.

TEST RESULTS

The test results consist of load versus vertical deflection, and load versus bolt forces. Failure moment, failure mode and a photographic record were also recorded during the tests.

The load versus vertical deflection data includes a theoretical line obtained using the following equation

$$\delta_{max} = \frac{Pa}{24EI}(3L^2 - 4a^2) \tag{1}$$

where a is the distance from the beam support to the point of load application.

EP36 Tests

Test EP36-1-3/4 A325-W18x50. Figure 3 shows a plot of applied load (hydraulic ram load in Figure 2) versus vertical deflection at the test beam midspan. (Deflections were corrected for support beam movement.) The measured load-deflection curve is nearly identical to the predicted elastic response curve to an applied load of approximately 35 kips, whereupon deviation occurs at an increasing rate. The maximum applied load was 59.0 kips which corresponds to an end-plate moment of 206.5 ft.-kips. After the maximum load was reached, the applied load decreased to 58.4 kips when the four tension bolts ruptured.

Figure 4 shows a plot of bolt force versus applied load. Bolt strains were measured in two of the four tension bolts; one outside the tension flange (outer) and one diagonally opposite between the beam flanges (inner). Also shown on the plot is the bolt pretension force required by the AISC Specification, 28 kips for 3/4-in.

diameter A325 bolts. Both bolt forces remained close to their pretension level until an applied load of approximately 45 kips was reached. Increased load caused the outer bolt force to increase at an extremely rapid rate as shown in Figure 4. The maximum strain recorded for the outer bolt was 4700 micro-in./in. (It is noted that the "bolt force" in Figure 4 is calculated assuming linear material behavior and, therefore, does not represent true force after the material yield stress is reached.)

Test EP36-1 1/4-3/4 A490-W18x50. Figure 5 shows a plot of applied ram load versus corrected vertical deflection at the test beam midspan. Again, the measured load-deflection curve is nearly identical to the predicted elastic response curve to an applied load of approximately 45 kips, whereupon deviation occurs at an increasing rate. The maximum applied load was 72.8 kips which corresponds to an end-plate moment of 254.8 ft.-kips. After the maximum load was reached, a slight decrease occurred, followed by rupture of the four tension bolts.

Figure 6 shows a plot of bolt force versus applied load for the outer and inner instrumented bolts, along with the specified pretension level (35 kips). In this test, both measured bolt forces increased above the pretension level as soon as load was applied to the connection. The forces began to increase at an increasing rate when the applied load reached approximately 45 kips. However, rupture occurred without the extremely rapid increase as occurred in the companion A325 test.

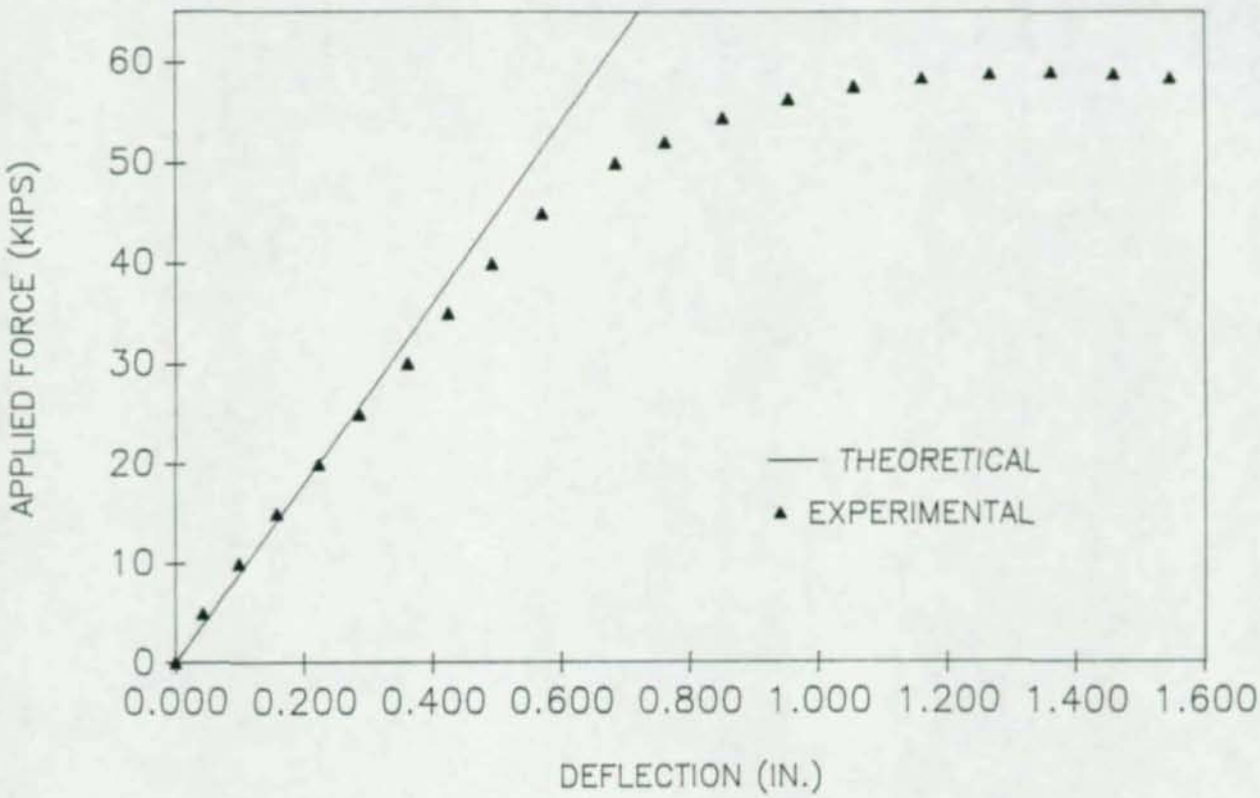

Figure 3. Load vs. Midspan Deflection, Test EP36-1-3/4 A325-W18X50

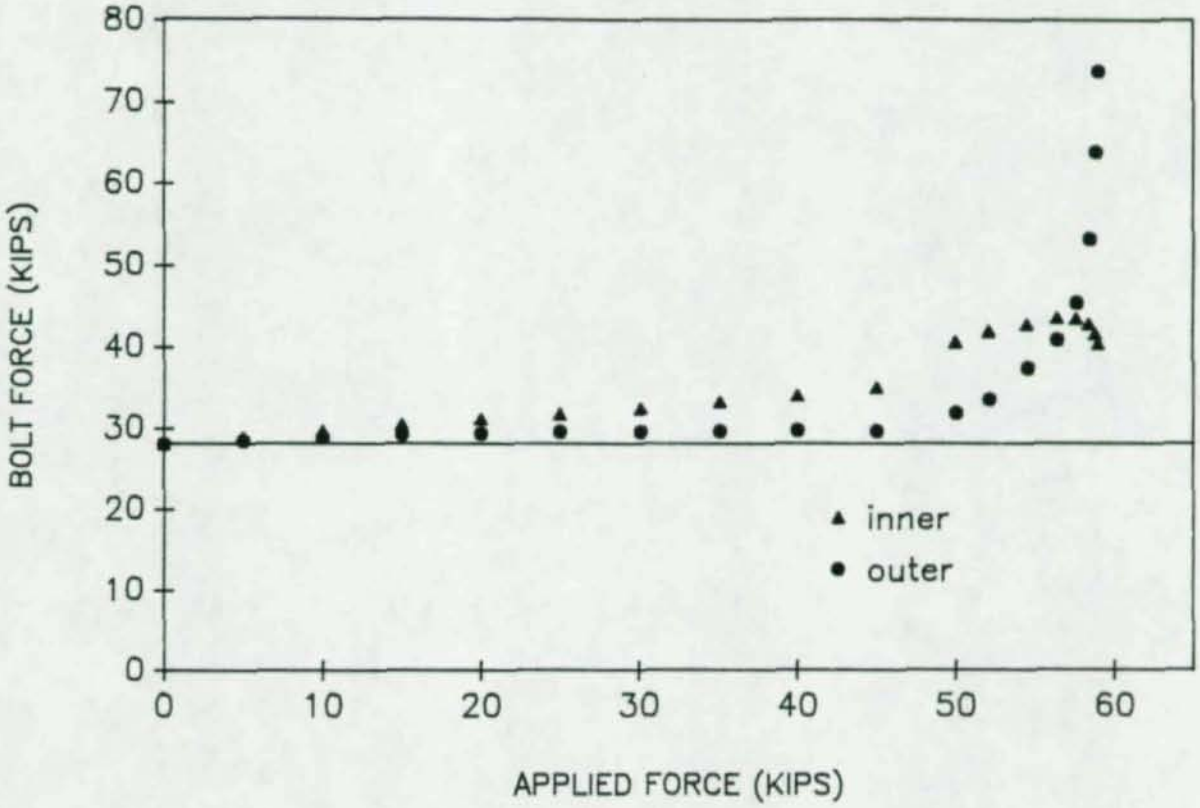

Figure 4. Bolt Force vs. Applied Load,
Test EP36-1-3/4 A325-W18X50

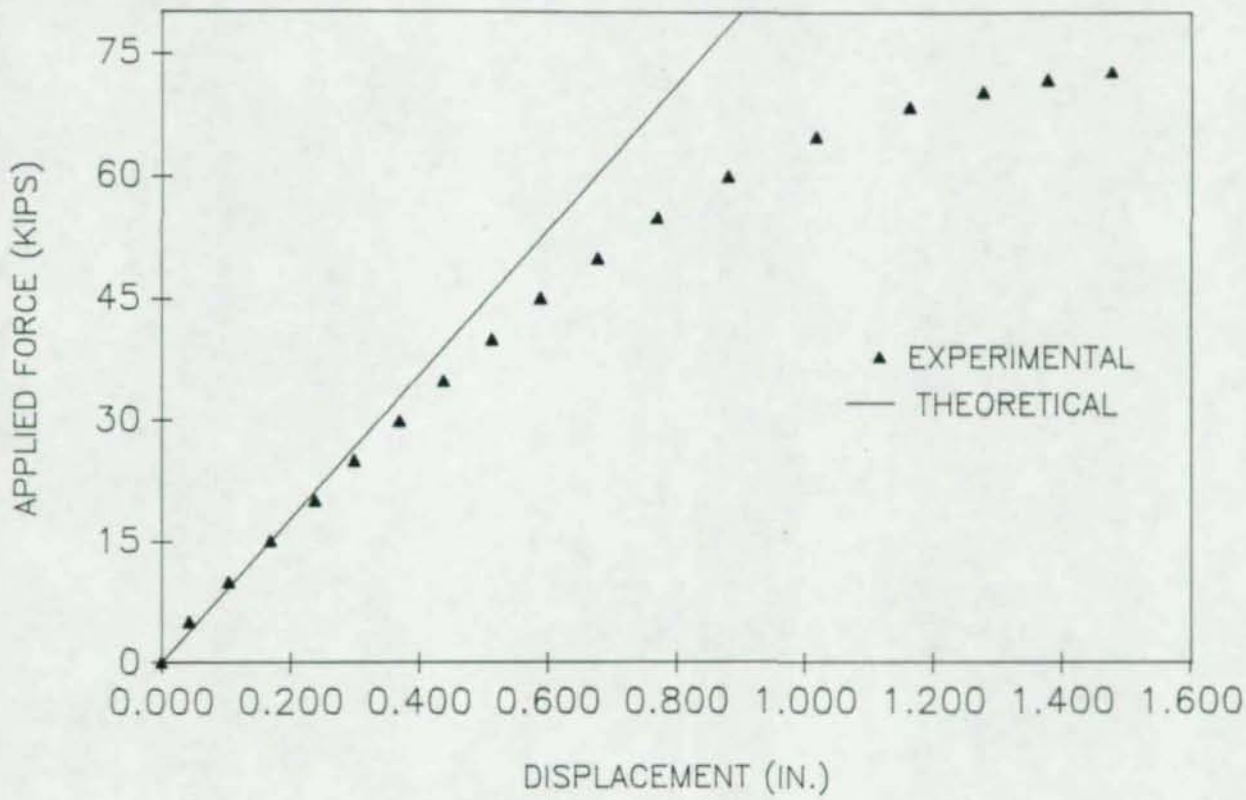

Figure 5. Load vs. Midspan Deflection,
Test EP36-1 1/4-3/4 A325-W18X50

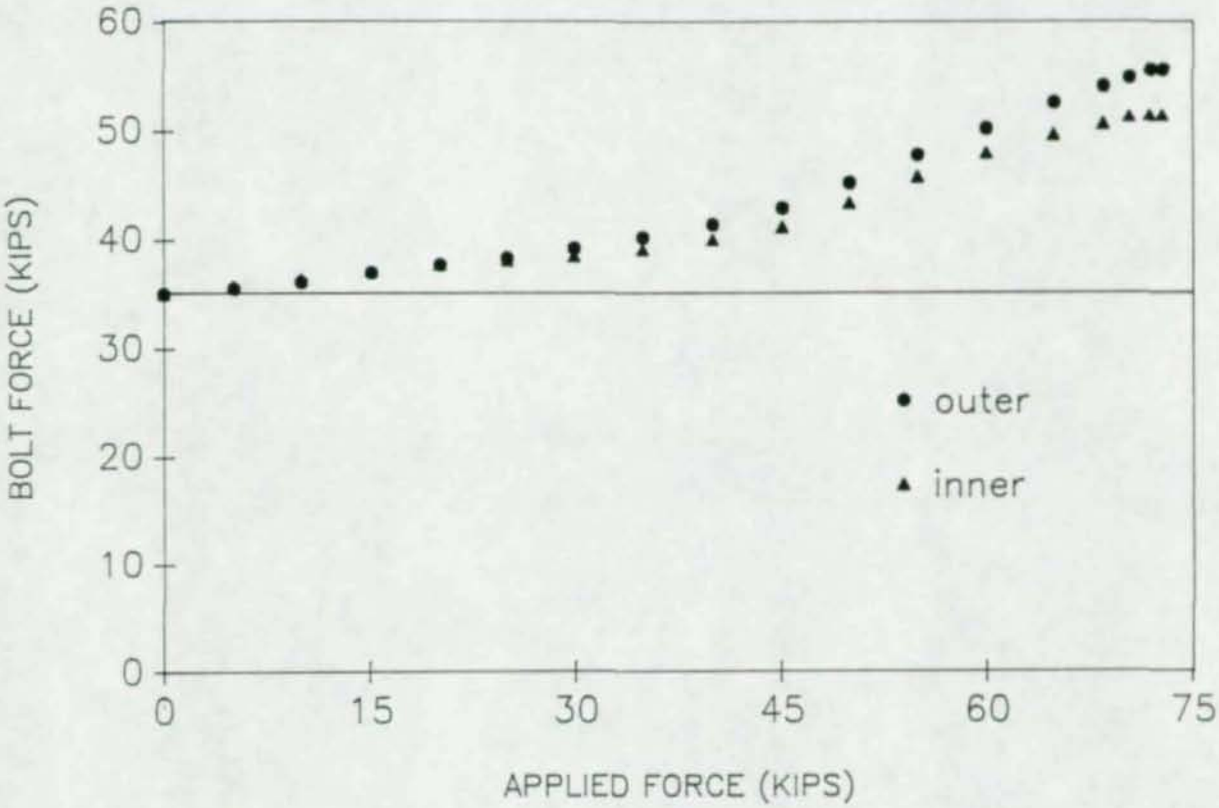

Figure 6. Bolt Force vs. Applied Load, Test EP36-1 1/4-3/4-W18X50

EP50 Tests

Test EP50-5/8-3/4 A325-18x8. A plot of applied load versus midspan vertical deflection for this test is shown in Figure 7. The measured response is elastic and in agreement with the predicted curve to an applied load of approximately 60 kips, whereupon significant departure occurred. The maximum applied load was 71.9 kips corresponding to an end-plate moment of 251.6 ft.-kips. Failure was rupture of the bolts after a slight drop in applied load.

Measured bolt force versus applied load is shown in Figure 8. The measured forces slowly increased above the pretension level (28 kips) until the applied load reached the 60 kips level, whereupon the outer bolt force increased at an extremely fast rate. The maximum recorded strain in the outer bolt was 7391 micro-in./in.

Test EP50-3/4-3/4 A490-18x8. Figure 9 shows the measured and elastic applied load versus midspan deflection curves. The measured response was close to the predicted elastic response to a load of approximately 60 kips. Yielding then occurred before the maximum load of 77.1 kips (end-plate moments of 270 ft.-kips) was reached. The four tension bolts ruptured without decrease in the applied load.

Bolt force versus applied load for this test is shown in Figure 10. The measured bolt forces increased above the pretension level (35 kips) at a slow rate until the 55 kips level was reached. Above this level, the outer bolt force increased dramatically before rupture. The maximum measured strain was 1174 micro-in./in.

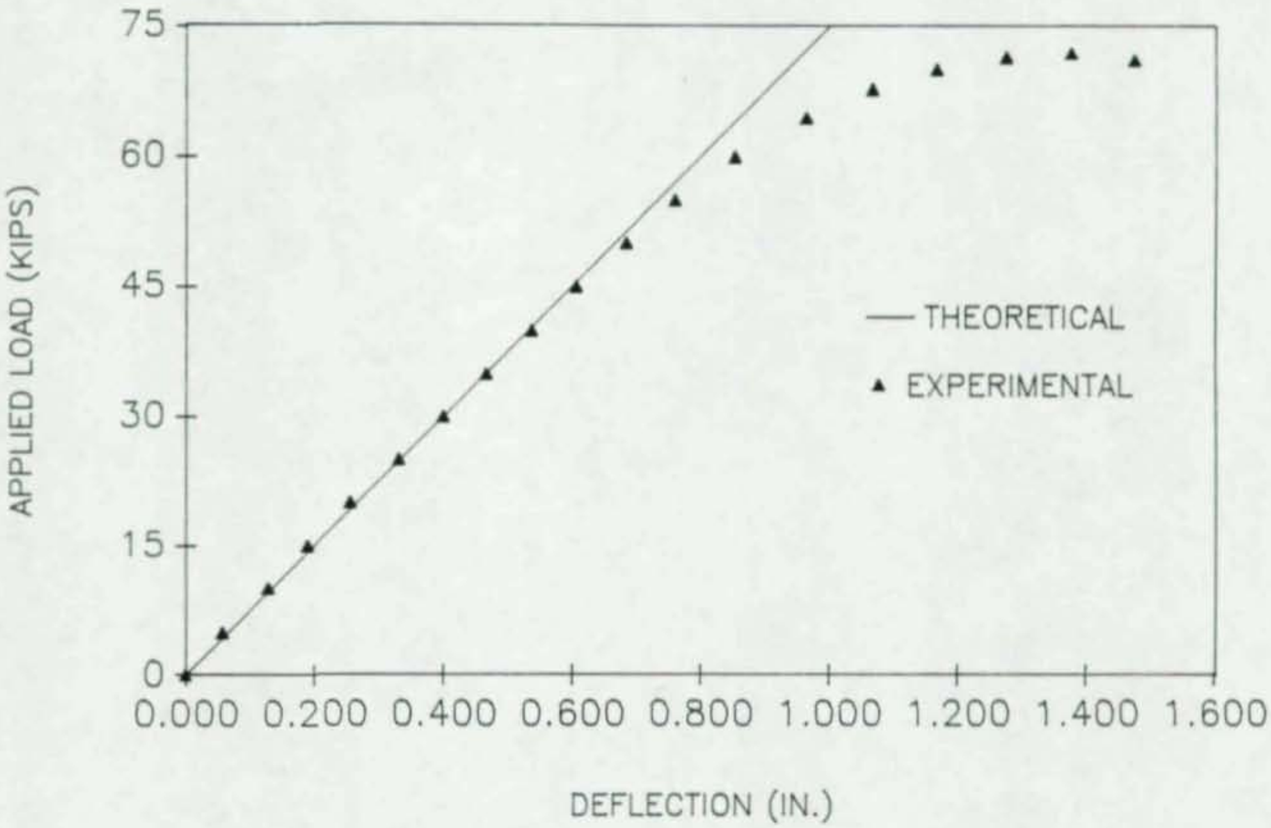

Figure 7. Load vs. Midspan Deflection, Test EP50-5/8-3/4 A325-W18X8

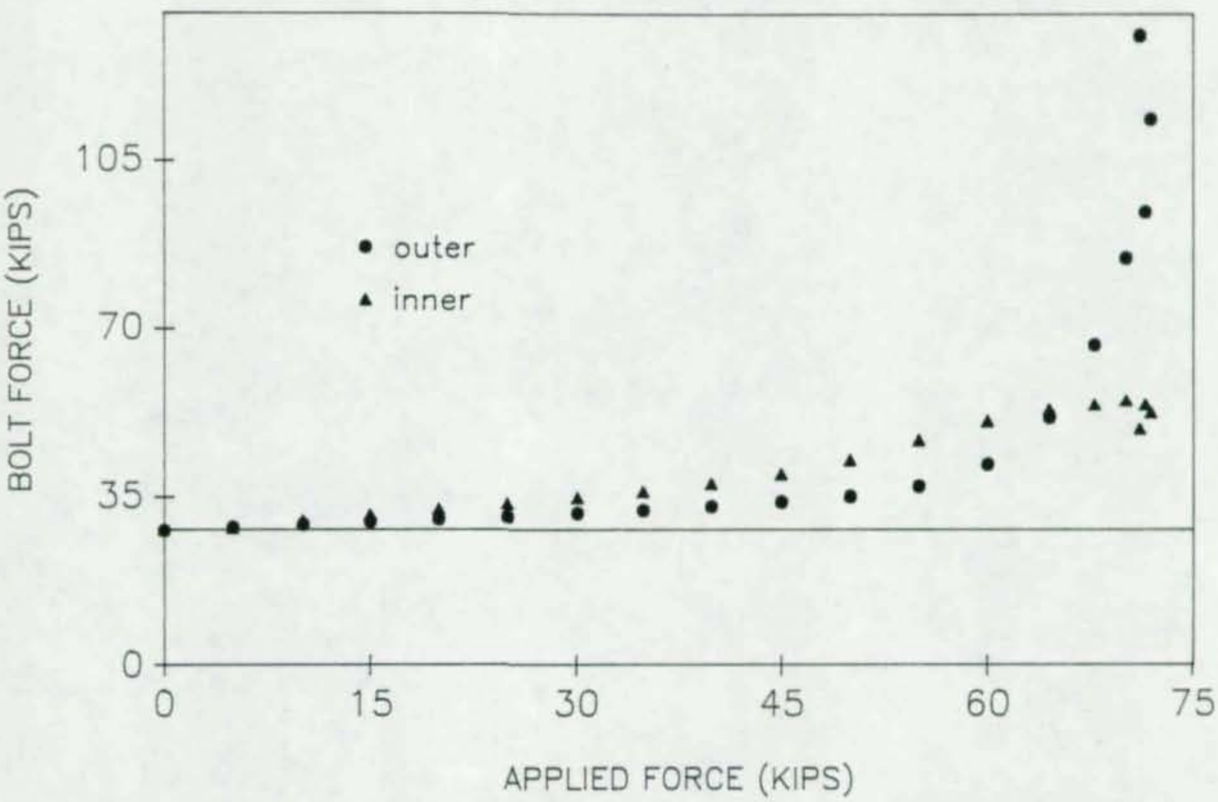

Figure 8. Bolt Force vs. Applied Load, Test EP50-5/8-3/4 A325-W18X8

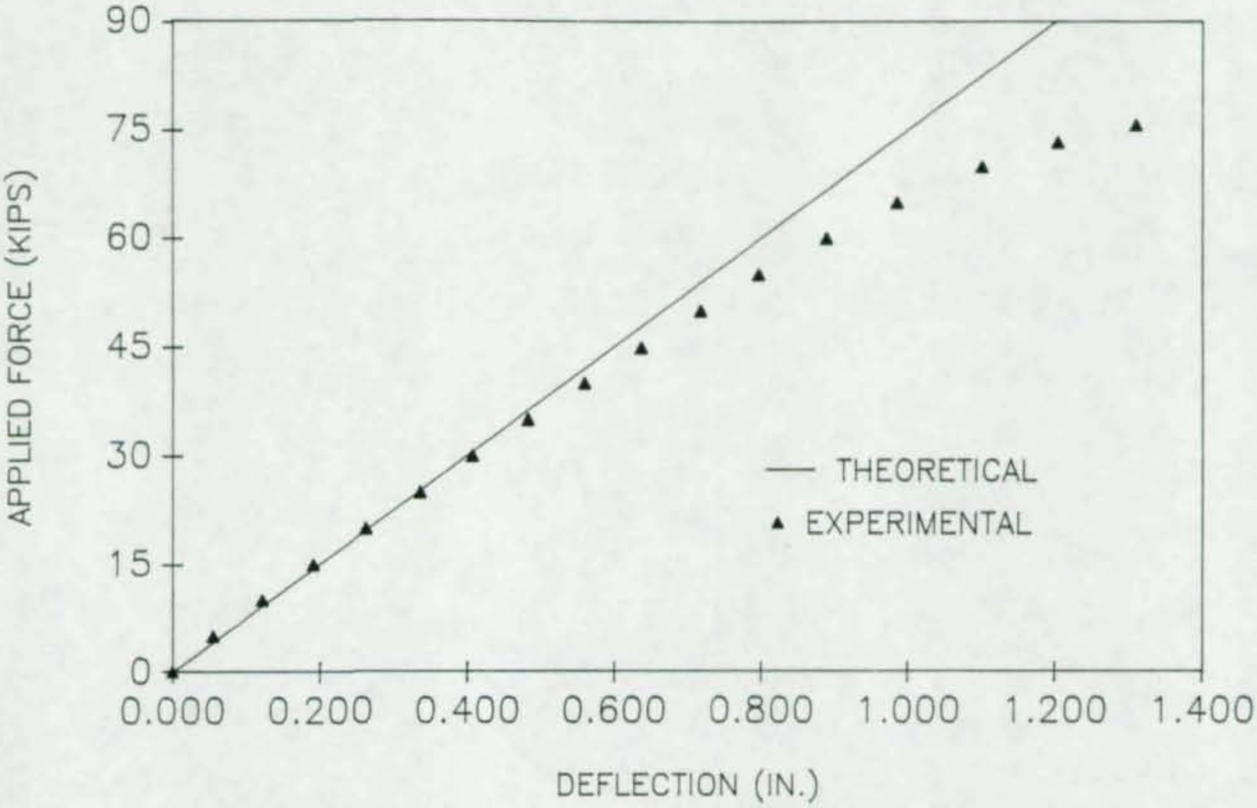

Figure 9. Load vs. Midspan Deflection, Test EP50-3/4-3/4 A490-18X8

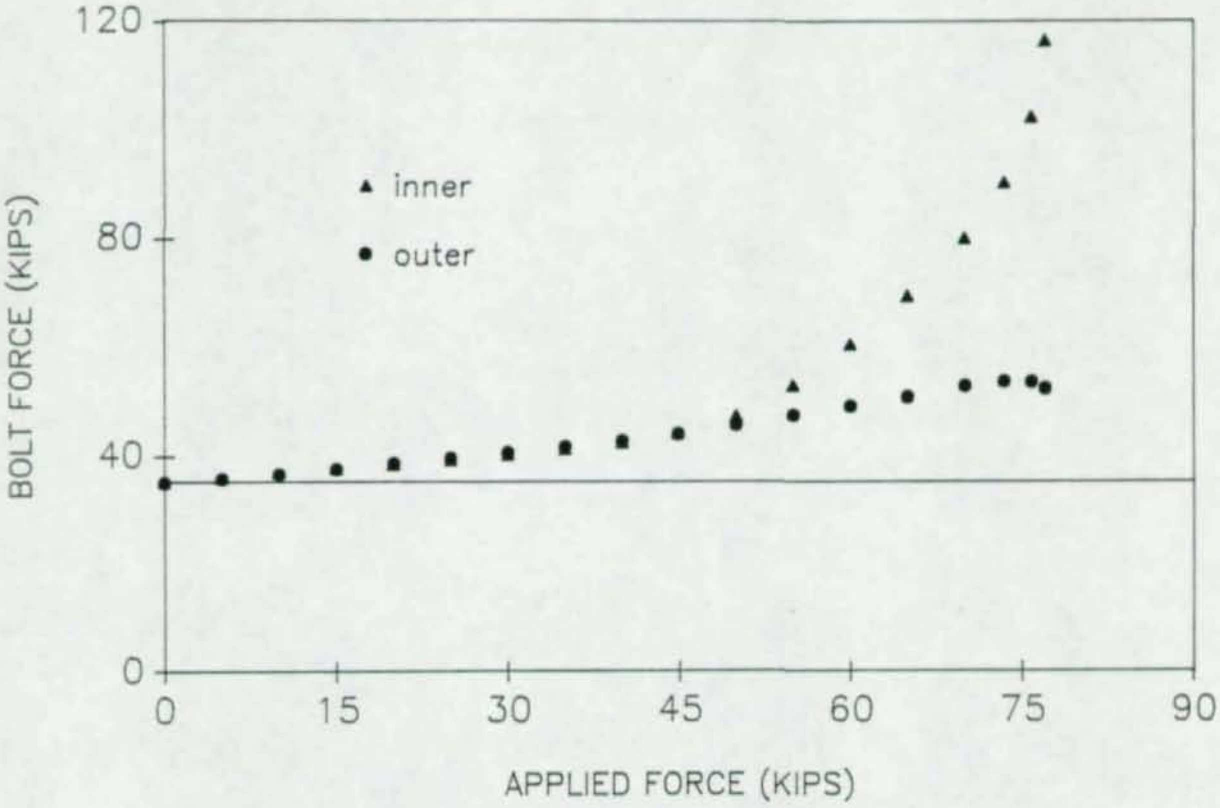

Figure 10. Bolt Force vs. Applied Load,
Test EP50-3/4-3/4 A490-W18X8

THEORICAL ANALYSES AND COMPARISONS

First Yield and Failure Load Predictions

To evaluate the test results four criteria were calculated:

a) The first yield moment of the beam, M_r, where, from the AISC LRFD Specification, is

Fyf ?

$$M_r = (F_{yw} - F_r)\, S_x \qquad (2)$$

flange

with F_{yw} = yield stress of the web material, F_r = compressive residual stress; 10 ksi for rolled shapes, 16.5 ksi for welded shapes and S_x = elastic section modulus.

b) The plastic moment capacity of the beam

$$M_p = F_y Z_x \qquad (3)$$

where F_y = material yield stress and Z_x = plastic section modulus.

c) The connection moment capacity as limited by the capacity of the end-plate to resist the beam flange force. The procedure on pages 4-11 through 4-114 of the 8th Edition AISC Manual of Steel Construction was used to calculate the service load moment. An assumed factor of safety of 1.67 was then removed to determine the predicted capacity of the connection for this limit state.

d) The connection moment capacity as limited by the tensile strength of four tension bolts with prying forces neglected. The moment capacity of the connection for this criterion is then

$$M_{bolts} = 4\, F_t\, A_b\, (d - t_f) \qquad (4)$$

where F_t = bolt material tension strength; 90 ksi for A325 bolts, 112.5 ksi for A490 bolts, A_b = nominal bolt area, d = beam depth, and t_f = beam flange thickness.

All calculations were based on nominal yield and tensile stresses and nominal dimensions except as noted in Table 1.

Comparison of Results

Table 2 is a summary of the analytical and experimental results. It is evident from this data that there is correlation between the beam yield moments, M_r, and the test yield moments, M_y. It is also evident that the test ultimate moment equaled or exceeded the end-plate capacity moment in all tests. Although bolt rupture was the failure mode in all four tests, the predicted tension bolt capacity was exceeded in only one test, EP50-5/8-3/4 A325-18x8. The tests using A325 bolts reached 91.6 percent (EP36) and 113 percent (EP50) of the predicted tension bolt capacity moment. The tests using A490 bolts reached 92.8 percent (EP36) and 96.8 percnet (EP50) of the predicted tension bolt capacity moment.

Figures 11 and 12 are comparison plots of the tests using A36 and A572 Gr50 steels, respectively. From both figures it is evident that the bolt material used in the connections did not significantly effect the load versus midspan deflection (and consequently the moment-curvature response curve) of the extended, unstiffened, moment end-plate connections tested.

CONCLUSION

Based on the limited data developed in this study, it is concluded the A490 bolts can be safely used in four bolt, extended, unstiffened moment end-plate connections.

Table 2

Summary of Analytical and Experimental Results

Test Designation	Beam Yield Moment M_r (ft-kips)	Beam Plastic Moment M_p (ft-kips)	End-Plate Capacity Moment M_{pl} (ft-kips)	Tension Bolt Capacity M_{bolt} (ft-kips)	Test Yield Moment M_y (ft-kips)	Test Ultimate Moment M_u (ft-kips)
EP36-1-3/4 A325-W18x50	192.6	303.0	206.5	225.2	175.0	206.5
EP36-1 1/4-3/4 A325-W18x50	192.6	303.0	331.0	277.6	217.0*	254.8
EP50-5/8-3/4 A325-18x8	221.7	366.9	203.4	226.4	227.5*	254.8
EP50-3/4-3/4 A325-18x8	221.7	366.9	244.5	278.8	227.5	270.0

*Not well defined point on load-deflection response curve

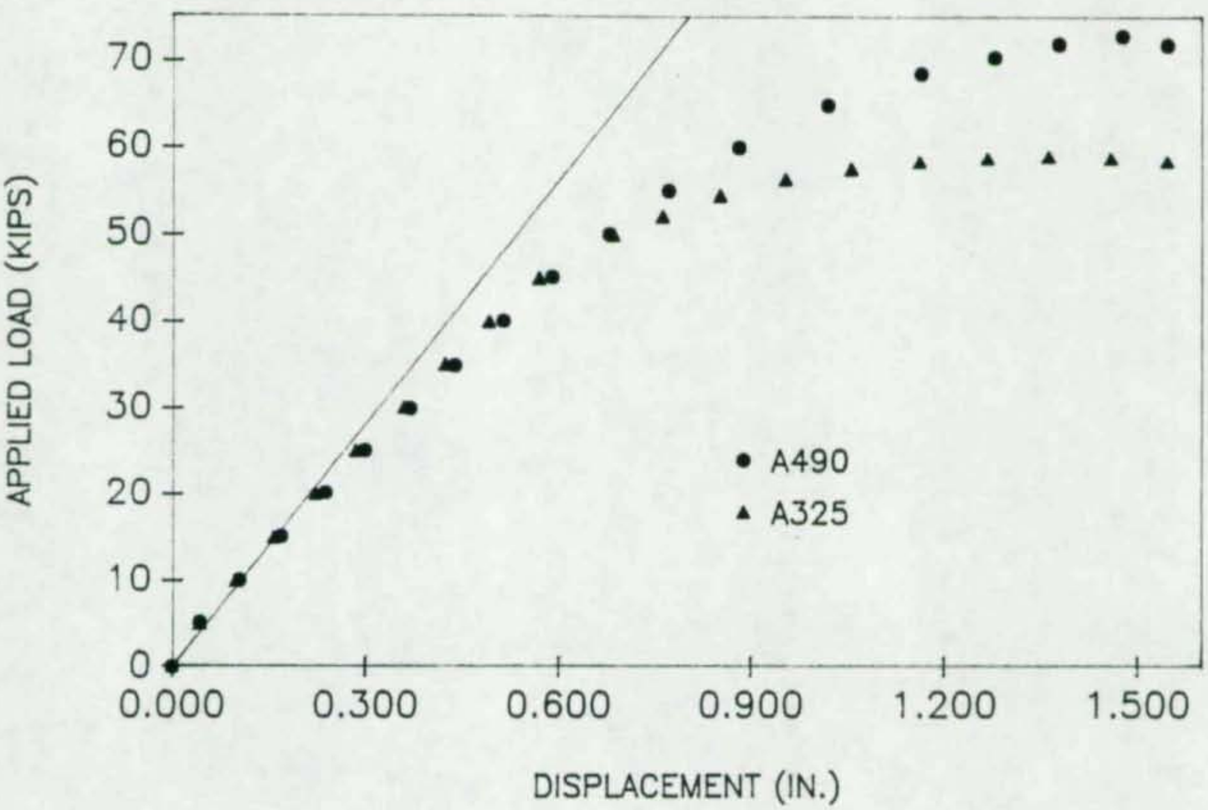

Figure 11. Comparison of EP36 Tests Applied Load vs. Midspan Displacement Curves

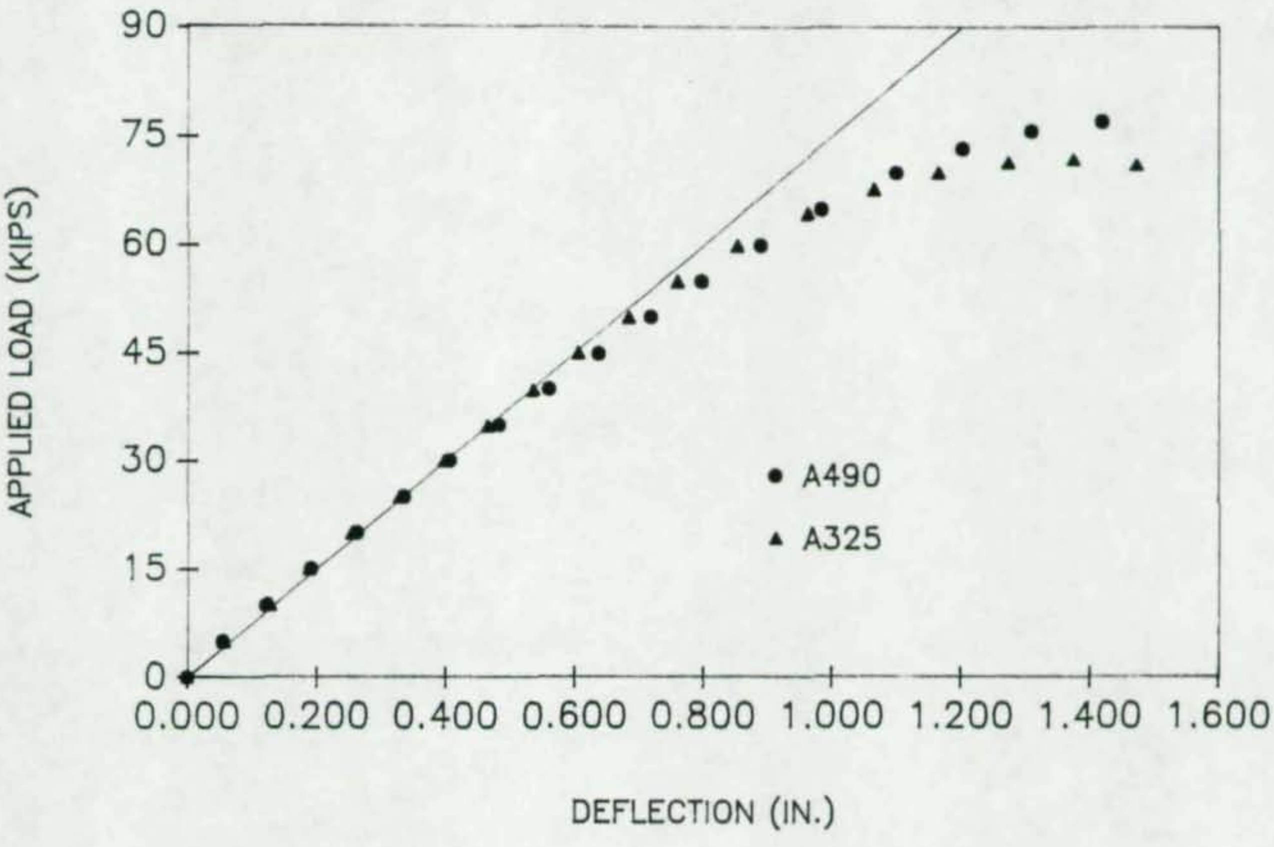

Figure 12. Comparison of EP50 Tests Applied Load vs. Midspan Displacement Curves

F. Research Reports
"End Plate Connections"