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SUBJECTED TO SEISMIC LOADS

by

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ABSTRACT

A number of unanswered questions exist concerning the design of joint panel zones in steel frames subjected to seismic loads. In the city of Los Angeles, California, alone over two million dollars in repairs have been spent for the past two contracted rectifications [1]. Investigators concluded that existing panel zones were inadequate when subjected to cyclic loads. Large shear stresses caused by lateral deformations of columns and couples developed from beam-end moments were suspected contributing factors.

Prior to the mid 1960s, engineers generally assumed that panel zones were not isolated from beam and column webs [1]. Therefore, it was believed that this region was not highly stressed. Consequently, panel zone regions were not considered in design. Today, panel zone regions are considered in design, but very conservatively.

Laboratory testing was conducted at Lehigh University under the supervision of Roger G. Slutter to determine panel zone behavior and eventually to alert engineers of the results.

Chapter 1

INTRODUCTION

1.1 Background

The seismic response of panel zones has been a heated topic of debate since engineers realized its importance in design. Until now, the shear stress failure criteria used to describe this region was defined by von Mises.

Many investigating engineers have diagnosed that the unreinforced panel zone is inadequate in withstanding high shear stresses created by seismic loads. A doubler plate attached to this zone with full penetration welds which meet seismic requirements has been used as a solution to this problem. Installation of such plates is not only time-consuming, but also expensive.

Some argue that column axial loads on the beam-to-column connection alter the behavior of the panel zone. The location of the connection in the structure usually dictates the magnitude of such loads. The dispute centered around the effects of low axial loads must be clarified.

Connection stiffness governs frame drift. A balance must be reached between ductility and drift control without elaborate connections which could prove to be economically unfeasible.

1.2 Investigation of Connection Response

The testing program at Fritz Engineering Laboratory, Lehigh University, was designed to investigate the effect of different parameters imposed on a typical beam-to-column connection. Actual size connections were tested. Beam and column section sizes remained constant throughout the testing program in an attempt to reduce the number of variables. Dimensions of the test specimens are given in Figure 1.

Some measures were taken to reduce the complexity of the testing program. For instance, axial loads were not applied to the columns because these loads in the actual structure would be insignificant when considering the section size and connection location. Also, displacements were applied to beam ends rather than column ends. It was decided that this loading scheme would closely resemble actual conditions. Figure 2 illustrates the testing configuration.

1.3 General Description of Panel Zone

The panel zone is located in the web of a column between connecting beam flanges in a beam-to-column connection. Figure 3 illustrates the location of a typical panel zone region.

During a seismic response this zone is subjected to a number of forces caused by beam-end moments and column displacements. A free body diagram showing the acting external forces from testing is shown in Figure 3.

In many cases, stiffeners are used between beam flanges to transfer loads into the column due to end moment force couples. The stiffeners also provide an increased resistance to the possibility of column flange buckling created by compressive forces or displacements caused by tensile forces.

1.4 Objectives

The primary purpose of this investigation is to develop a correlation between laboratory testing and an analytical computer model. Quantitative and qualitative results were developed to aid the designer and analyzer. Major concepts discussed in this document are listed below.

- Von Mises yield criteria currently used to describe the failure of the panel zone is much too conservative. Alternate formulations are proposed.

- Doubler plate attachment techniques were evaluated. According to American Welding Society (AWS) [2] welding requirements create problems in the panel zone.

- Effects of column axial loads were considered with the use of the finite element model.

- The use of transverse stiffeners as panel zone reinforcement were investigated.

- Welds used to connect column and beam flanges are far more critical than designers may expect. Stress concentrations coupled with residual stresses were studied.

Chapter 2

DESCRIPTION OF LABORATORY TESTS

A number of beam-to-column connections were tested at Lehigh University from April 1981 through January 1984 [3]. Test specimens had typical dimensions but were not representative of any particular frame. Column and beam sections were selected as being very commonly used members in building construction.

W24x62 beam sections and W14x90 column sections were used in all tests. In each case, column and beam flanges were welded and beam webs were connected to column flanges using a connection plate and ASTM A325 bolts [4]. The panel zone was designed to yield before the beams. Also, beams were designed to provide sufficient flexural and shear strength to create severe yielding in the panel zone unless a doubler plate was added.

Four connections, designated Specimens 1 through 4, were tested in the first phase of the program from April 1981 to June 1981. These specimens were assembled by West Coast fabricators in accordance with current specifications. Two additional connections were tested in the second phase of the program from November 1983 to January 1984. These connections, designated Specimens 1A and 3A, were similar to Specimens 1 and 3, but fabrication was done at Lehigh University.

2.1 Specimen Description and Material Properties

Beams, columns, stiffeners and connection plates for all six specimens were fabricated of ASTM A36 steel [4]. Grade 50 steel was used for the 12.7 mm (0.5 in.) thick doubler plate for Specimens 2, 3, and 3A. ASTM A36 steel was used for the 19.1 mm (0.75 in.) thick doubler plate for Specimen 4. Figure 4 further describes the connection components. Material properties for each specimen are given in Table 1.

2.2 Loading Procedure

Load was applied to beam ends simultaneously through four hydraulic jacks for each load increment. Diagonally opposite jacks were connected in parallel to provide the same magnitude and type of loading (tension or compression). Conversely, the other alternate pair of jacks provided the opposite type of loading. A control board consisting of four valves was used to apply the load through the jacks.

The column ends had 50.8 mm (2.0 in.) thick base plates attached. A fillet weld, approximately 254.0 mm (10.0 in.) long, connected both sides of the column web to the base plate. Flanges were not welded so that a shear-type end connection could be simulated. Bearing plates were used to secure the column ends. The top bearing plate was attached to the test frame while the bottom plate was bolted to the floor. The top bearing plate simulated a shear-type connection.

Load increments of 44.5 kN (10.0 kips) per beam were used until the panel zone deformation reached approximately 1.0%. The remaining increments of loadings were limited by 0.5% panel zone deformation. Specimens 1 through 4 were subjected to seven cycles of loading while Specimens 1A and 3A failed at less than seven cycles.

2.3 Instrumentation

The instrumentation used on Specimens 1 through 4 is shown in Figure 5. Ten electrical resistance strain gages were used to monitor jack loads and determine stresses in column flanges, stiffeners, beam webs and flanges. Rotation gages were attached to the column flanges and upper stiffener to monitor panel zone and rigid body rotations. Dial gages were used to measure the column-top deflection, beam-end deflection and the diagonal dimension change of the panel zone.

The instrumentation mounted on Specimens 1A and 3A is shown in Figure 6. Four electrical resistance gages were used to monitor jack loads and determine stresses at various locations. Rotation gages were attached to the column web. Gages were located 304.8 mm (12.0 in.) above and below beam flanges, at beam flanges and in the center of the panel zone. Dial gages monitored column and beam displacements.

Chapter 3

DISCUSSION OF TEST RESULTS

Data recorded during testing of the specimens were presented in beam load versus panel zone deformation relationships. The first and seventh cycles were plotted for Specimens 1 through 4. The first, third and fourth load cycles were plotted for Specimens 1A and 3A. For clarity of presentation, intermediate cycles are not shown for any of the specimens. Beam load versus panel zone deformation hysteresis curves for all specimens are given in Figures 7 through 12.

Beam load versus panel zone deformation curves for the first half cycle of loading are shown in Figure 13.

3.1 Cyclic Response of Specimens 1 through 4--Testing Phase #1

A diagonal gage was used in these tests to monitor the panel zone deformation. This measurement was independent of any rigid body movement at the top of the column and was comparable to the magnitude of deformation obtained from the two rotation gages. A law of cosines computation was used to reduce the gage readings to panel zone deformation.

Maximum panel zone rotations were recorded and are listed below for comparison.

Specimen 1 .... +4.0% and -6.2% (Figure 7)
Specimen 2 .... +2.3% and -1.0% (Figure 8)
Specimen 3 .... +2.7% and -2.7% (Figure 9)
Specimen 4 .... +2.7% and -2.7% (Figure 10)

3.1.1 Cyclic Loading of Specimen 1

The -6.2% rotation of Specimen 1 was limited by the stroke of the hydraulic jacks used to apply load; therefore, the connection may have withstood higher rotation. Specimen 1 exhibited far less load carrying capacity but far more ductility than Specimens 2 through 4. Up to a panel zone deformation of 6.2%, Specimen 1 showed no sign of failure. A plastic hinge formed in the panel zone first rather than in the connecting beams.

The panel zone of Specimen 1 showed inelastic behavior at a beam load of approximately 177.9 kN (40.0 kips). After this loading was exceeded, strain hardening commenced in the panel zone region (see Figure 7).

The first half cycle of loading was carried to 2.5% deformation, far above the minimum 1.5% drift requirement. It was difficult to observe yielding patterns at low beam loads because the connection was blast-cleaned and stripped of mill scale. As loading continued, yielding due to high stress concentrations was present in the center and edges of the panel zone, as well as at flange connection points.

3.1.2 Cyclic loading of Specimens 2 through 4

The remaining three specimens exhibited the effect of the doubler plate. The effects were evident in the elastic and inelastic states of stress in Specimens 2 and 4, and the inelastic state of stress in Specimen 3. Specimen 2, like Specimen 1, was blast-cleaned and stripped of mill scale. Therefore, early yielding could only be observed in Specimens 3 and 4. Yielding common to both Specimens 3 and 4 is shown in Figure 14.

The load carrying capacity of Specimens 2 through 4 was approximately the same, but Specimen 2 had greater stiffness when compared to Specimens 3 and 4. This stiffness was evident in the elastic and inelastic regions of the panel zone. This dictates that full penetration welds produce higher stiffness qualities. Specimen 3, which utilized fillet welds in the doubler plate attachment, exhibited less stiffness but greater ductility. Longitudinal fillet welds of a doubler plate subjected to longitudinal forces have high ductility [5] [6].

Yield line patterns in Specimen 3 suggested that the doubler plate was basically ineffective in the elastic region of the column web, but developed and began to carry load in the inelastic region. The higher yield strength of the doubler plate (Grade 50 steel) had some effect on the difference in yield development. Initial yield line patterns developed in the column web and not the doubler plate at these low beam loads.

Specimen 4 acted somewhat stiffer than Specimen 3 even though the yield strength of the doubler plate used in Specimen 4 was lower. The shear capacity was higher because the thickness of the doubler plate was increased enough to overcome the difference in yield strength. A similar fillet welding procedure was used on both specimens.

The testing of Specimens 2, 3 and 4 was haulted because panel zone strain hardening began to produce plastic hinges outside the panel zone forcing cracks in beam-to-column connection welds.

3.2 Cyclic Response of Specimens 1A and 3A--Testing Phase #2

A six-inch rotation gage was used to monitor panel zone deformation. Calculations showed, however, that the values obtained with the gage were conservative. Therefore, beam displacements were used to quantify the panel zone deformations. Since there was rigid body movement of the column top, a correction term was introduced into the deformation calculation.

Maximum panel zone deformations are listed below.

Specimen 1A .... +3.14% and -2.84% (Figure 11)
Specimen 3A .... +1.42% and -1.30% (Figure 12)

3.2.1 Cyclic Loading of Specimen 1A

Specimen 1A, fabricated similarly to Specimen 1, except stiffeners were not used, exhibited less ductility. One might expect Specimen 1A to be less ductile than Specimen 1 throughout the elastic region, but more ductile in the inelastic region, because elastically, the panel zone of Specimen 1A was not well defined. Consequently, a larger area withstood the shear. As the panel zone became inelastic, the area defining the region became more pronounced. The boundaries of the zone were clearly defined by the beam flange connection points.

The quality of welding and high stress concentrations located at flange connection points in Specimen 1A restricted the specimen from exhibiting ductility. A fracture in the heat-affected weld zone of the column flange took place during the second half of the first cycle. Welds were repaired but the flange again failed in the fourth cycle of loading.

The beam load versus panel zone deformation for Specimen 1A indicated panel zone yielding at a load of 155.7 kN (35.0 kips). Strain hardening of the panel zone soon followed and became more apparent as loading continued.

The cyclic load carrying capacity of Specimen 1A was slightly less than that of Specimen 1 because transverse stiffeners were not used, therefore reducing the stiffness.

Yield lines were first visible in the column flange at beam loads of 89.0 kN (20.0 kips). At 111.2 kN (25.0 kips), yield lines began to form in the column web along a diagonal. These lines originated at the center of the panel zone and grew to the full depth of the beams. At a beam load of 133.4 kN (30.0 kips), yield lines covered the column web over the full depth of the beams and on the column flanges at beam flange connection points. In addition, extensive column flange yielding was seen at beam flange tips and near the tensile beam flanges. Drawings illustrating this sequence of events can be seen in Figure 15.

3.2.2 Cyclic Loading of Specimen 3A

Specimen 3A exhibited far less ductility, but showed a much higher load carrying capacity than Specimen 1A. Specimen 3A was fabricated with the same sections and dimensions as Specimen 3, but did not utilize transverse stiffeners.

One would expect the ductility of Connection 3A to be better than Connection 3 in the inelastic region. This was not the case, however, because welds and high stress concentrations at flange connection points were again the limiting factor. Since Specimen 3A did not utilize transverse stiffeners, a higher demand was placed on welded connection points which initiated early failures.

The load carrying capacity of Specimen 3A was less than that of Specimen 3 because of decreased stiffness. The connection did not have the added reinforcement of the transverse stiffeners.

The doubler plate in Specimen 3A, like that of Specimen 3, had to develop as loads forced the column web to become inelastic. Yield lines originated in the column web and inside the flanges near beam flange connection points. Early yield lines occurred at a load of 266.9 kN (60.0 kips) during the first half cycle of loading. As the beam load increased to 355.9 kN (80.0 kips) the first signs of doubler plate yielding developed. Yielding along fillet welds was also seen. Drawings show these stages of yielding in Figure 16.

Chapter 4

COMPUTER ANALYSIS OF TEST SPECIMENS

A finite element analysis was conducted to simulate the connections tested in program phases #1 and #2. Specimen dimensions and loading were duplicated in the computer model. SAP4, a structural analysis program for static and dynamic response of linear systems [7], was used for the connection simulation. The model was created to illustrate the highly stressed region of the panel zone, local out-of-plane displacement of the beam-to-column flange connection points and areas which exhibit high localized stresses.

4.1 General Description

A number of parametric studies were conducted utilizing the model for comparison to test results. The parametric studies performed are listed below.

1. Connection without any panel zone reinforcement.

2. Connection with doubler plate only.

3. Connection with transverse stiffeners connected to column flanges only.

4. Connection with transverse stiffeners connected to column flanges and web.

5. Connection with large transverse stiffeners connected to column flanges and web.

6. Non-linearization of center panel zone region with stiffeners fully connected to column.

7. Non-linearization of center panel zone region with a column load applied and stiffeners fully connected to column.

Sap4 is strictly a linear finite element program, but modifications were made to simulate the inelastic behavior of the connection. Also, the computer model analyzes the first half of the first loading cycle only.

4.2 Discretization

The mesh used in the analysis along with representative nodal point numbering is shown in Figure 17. It is apparent by the discretization that the areas of interest center around the panel zone, beam webs near column flanges and beam-to-column flange connection points.

Aspect ratios of these regions are:

PANEL ZONE: 1 to 1.035

BEAM WEBS NEAR COLUMN FLANGES: 1 to 1.038

BEAM-TO-COLUMN FLANGE CONNECTION AREAS: 1 to 2.416

The model is composed of two elements:

- Plane stress elements (Type 4)--used for both column and beam webs.

- Plate bending elements (Type 6)--used for column and beam flanges.

4.3 Idealization

The following assumptions were made in modeling the beam-to-column connection.

1. All eccentricities in the connection were ignored. The beams have the same depth and length on either side of the column flanges and distances are identical from the top beam flanges to the top column support and the bottom beam flanges to the base column support.

2. The bolted connection of the beams to the column via the connection plate was assumed to be a friction-type connection in the elastic range and throughout the early stages of inelastic panel zone behavior. Therefore, plane stress elements of the beam webs share nodal points of the column flange as well as the column web.

3. Loads are applied at the centerline of the beam webs. Also, loads are placed at extreme nodes of the beams.

4. A line of symmetry was used to reduce computation complexity and time. This line passes through the webs of both the column and beams.

A drawing of the idealized connection and orientation of global coordinates shown in Figure 18 may be used as reference.

4.4 Boundary Conditions

Boundary conditions used in the finite element model are listed below.

1. Nodal points located at supports of the column (web only) were modeled to simulate a shear-type connection. The boundary conditions at these points were modeled with testing procedures in mind. The weld used in the test specimens to connect the column web to supports was approximately 254.0 mm (10.0 in.) long. Therefore, nodes in this region are free to rotate, but translation is suppressed. The nodes at all other points on the column ends are free to rotate and translate.

2. Boundary conditions along the line of symmetry favor the conditions of that cut. Out-of-plane translation is prohibited (global x-direction) while the horizontal and vertical in-plane translations are possible. Rotation of the entire connection about the global x-axis is allowed to simulate the

rotation of the entire structure during testing.

3. Boundary conditions at interior nodal points (flanges of the column and beams) are imposed considering out-of-plane action. Conditions are dependent on element location and freedom of nodal points favor plate bending elements rather than plane stress elements at shared nodal points. Two types of boundary conditions were used for these elements. For the beam flanges, all translations and rotations are free except the rotation perpendicular to the plate bending elements. The perpendicular axis for these elements is the global z-axis; therefore, this rotation is suppressed. For the column flanges, all translations and rotations are free except the rotation about the global y-axis which is perpendicular to the plate bending elements.

4.5 Coordinate Axes of Elements

Local coordinate axes for plane stress (Type 4) elements and plate bending (Type 6) elements are listed below.

- Plane stress elements--Local u,v axes coorespond to global y,z axes respectively. The IJKL numbering scheme of quadrilaterals (IJK for triangles) utilizes the right hand rule. All elements are in the y-z plane and the direction of IJK(L) create a perpendicular axis in the positive x-direction.

- Plate bending elements--The plate bending elements (described by IJKL) which simulate the beam flanges correspond the local x,y,z axes to the global y,x,z axes where the local z-axis is perpendicular to the element. The elements which describe the column flanges are numbered so that the local x,y,z axes correspond to the global x,z,y axes. The perpendicular local axis is in the z-direction.

Element coordinate axes are illustrated in Figure 19. The SAP4 output sign convention is shown in Figures 20 and 21. These conventions are given to aid in result interpretation.

Chapter 5

DISCUSSION OF COMPUTER ANALYSIS AND COMPARISON WITH LABORATORY TEST RESULTS

The finite element model reinforces the conclusions drawn during the testing of the six specimens. The model clearly illustrates the high shear stresses in the panel zone, the local out-of-plane displacements of the beam-to-column flange connections and the high localized stresses in the column flanges, beam flanges and stiffener plates.

5.1 Beam-to-Column Connection Without Reinforcement

Specimen 1A, tested in November 1983 is the connection that was modeled in this segment of the computer analysis. The finite element model results closely coincide with observations and theories developed through testing which showed that the panel zone was not well defined under low beam loads. As loading increased and the panel zone began to yield, the geometry of this region became distinct. A high shear stress region in the column web between beam flange connection points defines the boundaries.

Shear stresses in the center of the panel zone are highest. The current AISC specification [8] states that von Mises yield stress criteria predicts failure due to yielding at:

$$V=0.58F_y$$

with F_y=288.9 MPa (41.9 ksi)

therefore,

V=167.6 MPa (24.3 ksi) at failure

From the model, this shear stress would require a beam load of 130.8 kN (29.4 kips). From Figure 11, the onset of panel zone yielding occurred at approximately 155.7 kN (35.0 kips) load per beam which is significantly greater than 130.8 kN (29.4 kips).

The tensile and compressive stresses in the center of the panel zone were essentially zero. Therefore, principal compressive and tensile stresses are equal in magnitude to the shear stresses in this area.

Shear stresses away from the center region were highest on the diagonals from one corner of the panel zone to the other. Maximum stresses followed the principal tensile stresses along the diagonal. The zone qualitatively resembles tensile field action seen in plate girders subjected to transverse loads [9]. Figure 22 illustrates the tension field concept.

Stress concentrations observed during testing were found in the model. Critical areas in the connection centered around beam-to-column flange attachment points. The drawing in Figure 23 visually describes these areas of high stress concentrations. Heat-affected zones created by welding are critical. Tensile residual stresses caused by welding imposed on fabricated tensile residual stresses near section webs prove to be extremely critical when tensile loads are applied. Also, in test cases, beam flanges were smaller than column flanges, creating an additional stress concentration effect at the beam flange tips imposed on the column flanges. Failure did occur during the second half of the first loading cycle in Specimen 1A due to a fracture at the weld toe in the column flange.

In addition, results from the model indicated areas of highly concentrated longitudinal stresses in beam-to-column flange connection points. Beam stresses were largest in central extreme fibers of the flange. Also, longitudinal stresses in the column flange were highest near the web in the immediate vicinity of the beam flange.

The stresses in the beam flange approached the yield strength of the material at a beam-end load of 311.4 kN (70.0 kips) (based on results from the model and a beam flange yield strength of 308.9 MPa (44.8 ksi)). During testing, yielding of the beam flange was observed far below a beam load of 311.4 kN (70.0 kips) indicating the effect of residual stresses. It is difficult to quantify the magnitudes of the residual stresses; therefore, any attempt to introduce synthetic stresses in the model would be difficult to justify.

Values obtained from the model show that the column flange will begin to yield at a beam load of 349.2 kN (78.5 kips) (based on F_y=259.3 MPa (37.6 ksi)). Again, yielding occurred in the column flange at lower loads during testing. Tensile residual stress from fabrication and welding was the primary reason for this.

Finite element model displacements describe the deformation of the panel zone under loading. Figure 24 shows the elastic response of the column flanges and panel zone when loads are applied. This illustrates that shear deformation truly dominates the panel zone region.

Comparison of beam-end deflection in the model and in the test is a strong indication of the model accuracy. Results from the computer model and the test for the elastic region of the panel zone are listed below.

LOAD kN (kips)	END BEAM DEFLECTION mm (in.) Model	Test Specimen 1A
44.5 (10.0)	2.27 (0.0894)	3.15 (0.124)
89.0 (20.0)	4.54 (0.1788)	6.10 (0.240)
133.5 (30.0)	6.81 (0.2682)	11.07 (0.436)

These values represent differences of 28%, 25% and 39% for 44.5 kN (10.0 kip), 89.0 kN (20.0 kip) and 133.5 kN (30.0 kip) loadings. The values obtained from the computer model are lower than the values from the test because the finite element technique is an upper bound analysis. Therefore, the model is much stiffer than the actual connection. Also, the percent difference in values increases as beam loads force the initiation of panel zone plastification. For instance, at a load of 133.5 kN (30.0 kips) some local areas of the connection have yielded creating larger beam-end deflections and, therefore, a greater discrepancy of results.

5.2 Beam-to-Column Connection with Doubler Plate Attached to Column Web

Specimen 3A, tested in December 1983 most closely resembles the computer model. The model was developed assuming the doubler plate utilizes full penatration welds for web attachment; therefore, the doubler plate was fixed to the web. In the test, however, the doubler plate was fillet welded to the column web and flanges. The model, like the test specimen, did not have transverse stiffeners used for reinforcement. The doubler plate, which covers one side of the column web in the panel zone region, extends approximately 63.5 mm (2.5 in.) beyond the beam-column connection points.

Test results and the finite element model confirm that the doubler plate increases the stiffness of the connection while decreasing the shear stresses in the panel zone. The doubler plate develops under early loading in the finite element model because the plate is fixed to the column web and flanges. In the test, the fillet welds forced the doubler plate to develop in the inelastic region. In spite of this, some result comparisons are still possible.

Results from the model indicate that the shear stresses in the panel zone decrease by approximately 50% with a 12.7 mm (0.5 in.) thick doubler plate. However, column and beam flange stresses are not reduced and in some cases are higher than stresses without the doubler plate.

For instance, the computer model indicates yielding of the beam flange near the column flange connection point at a beam load of 311.4 kN (70.0 kips). Tensile residual stresses due to fabrication and welding in the actual connection may have been enough to yield the sections prior to loading. Also, longitudinal stresses in the column flange are similar to the stresses found in the connection without the doubler plate in place. Here again, yielding commences soon after load has been applied. Figure 25 shows areas in which yielding at low beam loads is present.

This connection is far stiffer than the model or test connection, Specimen 1A. Maximum beam end loads nearly doubled when the doubler plate was used.

Displacements developed in the finite element model illustrate the panel zone deformation. The panel zone deforms predominantly because of shear forces, but only reaches deformations comparable to Specimen 1A when loads nearly double. Figure 24 shows the panel zone deformation of Specimens 1A and 3A.

The most accurate method of comparing relative stiffness and its effect on panel zone deformation is through beam-end displacements. Beam deflections in the elastic region are listed below. A comparison is drawn between the model and test results for Specimen 1A (no panel zone reinforcement) and Specimen 3A (doubler plate used for reinforcement).

LOAD kN (kips)	BEAM-END DEFLECTION mm (in.) Specimen 1A (w/o d. plate) Model	Test
44.5 (10.0)	2.27 (0.0894)	3.15 (0.124)
89.0 (20.0)	4.54 (0.1788)	6.10 (0.240)
133.5 (30.0)	6.81 (0.2682)	11.07 (0.436)
	Specimen 3A (w/ d. plate) Model	Test
44.5 (10.0)	1.82 (0.0717)	2.84 (0.112)
89.0 (20.0)	3.64 (0.1434)	5.31 (0.209)
133.5 (30.0)	5.46 (0.2151)	8.00 (0.315)

* Results based on first half of first load cycle. Correction for frame seating used for test results of Specimen 3A.

The differences between various results are shown below.

LOAD kN (kips)	PERCENT DIFFERENCE (%) Model w/o d.p. / Model w/ d.p.	Test w/o d.p. / Test w/ d.p.	Test w/ d.p. / Model w/ d.p.
44.5 (10.0)	19.0	10.0	36.0
89.0 (20.0)	20.0	13.0	31.0
133.5 (30.0)	20.0	28.0	32.0

where,
- w/ - with
- w/o - without
- d. - doubler
- d.p. - doubler plate

Model and test results listed above prove that the average percent difference in stiffness between specimens with and without doubler plates is 20% and 17% (model and test results respectively). The difference between the model and test of Specimen 3A is high in the early stages of inelastic behavior, a percentage of 33%. The values of beam deflections from the test are consistently higher indicating the upper bound finite element analysis. Also, the doubler in Specimen 3A does not develop in elastic loading; therefore, the connection has less stiffness in the test when compared to the model under these beam loadings.

5.3 Beam-to-Column Connection with Transverse Stiffeners

The computer model was designed to consider the effects of transverse stiffeners in the connection when attached to the column flanges only and when attached to both the column flanges and web.

Specimen 1, tested in April, 1981, is the connection which resembles the finite element model the closest. The specimen was fabricated with 9.5 mm (0.375 in.) thick transverse stiffeners connected to the column flanges and web.

The transverse stiffeners created a well-defined panel zone boundary. This boundary existed at low beam loads (elastic panel zone) in both the model and the test. Shear stresses within the panel zone are at least twice the magnitude of stresses in the column web outside the stiffeners (beam loads less than 133.4 kN (30.0 kips)).

Shear stresses are highest in the center of the panel zone. The diagonals of this region have large, yet slightly lower, shear stresses. During testing, early yielding in the specimen originated in the center of the panel zone, then developed along one diagonal. This response closely resembles tensile field action seen in plate girders [9]. The tensile field action was more pronounced in Specimen 1 than Specimen 1A because the boundaries of the panel zone are more restricted by the stiffeners.

Von Mises shear stress criteria predicts shear stress failure at V=0.58Fy which is a shear stress of V=168.9 MPa (24.5 ksi) (based on Fy=291.0 MPa (42.2 ksi) for the column web of Specimen 1). According to results from the finite element model, a beam force of 139.0 kN (35.3 kips) would be required to fail the panel zone in shear. From Figure 7, the onset of yielding occurs at a beam force of approximately 155.7 kN (35.0 kips) which indicates a shear stress at first yielding to be 188.9 MPa (27.4 ksi). This stress is significantly higher than 168.9 MPa (24.5 ksi).

Stress concentrations observed during testing were chiefly at flange connection points. The computer model also indicates these areas of high local stress. Again, high residual stress combined with tensile loads force early yielding in these flange areas. A drawing showing areas of high local stress is given in Figure 23.

Comparisons of the connection stiffness, including the introduction of varied parameters, are listed below. These parameters are: (1) no stiffeners, (2) stiffeners - connected to column flanges only, and (3) stiffeners - connected to column web and flanges. An accurate method of comparison is by beam-end displacement.

LOAD kN (kips)	BEAM-END DEFLECTION Model mm (in.) w/o st.	w/ st. f.c.o.	w/ st. w.&f.c.
44.5 (10.0)	2.27(0.0894)	2.25(0.0887)	2.19(0.0863)
89.0 (20.0)	4.54(0.0894)	4.51(0.1788)	4.38(0.1726)
133.5(30.0)	6.81(0.2682)	6.76(0.2661)	6.58(0.2589)

LOAD kN (kips)	Test mm (in.) w/o st. Specimen 1A	w/ st. w.&f.c. Specimen 1
44.5 (10.0)	3.15 (0.124)	3.48 (0.127)
89.0 (20.0)	6.10 (0.240)	6.96 (0.274)
133.5(30.0)	11.07 (0.436)	14.71 (0.579)

where,

w/ - with
w/o - without
st. - stiffeners
f.c.o. - flange connection only
w.&f.c. - web and flange connection

The computer model shows the increased stiffness (based on beam-end displacement) of the connection when transverse stiffeners are used; however, in most cases the increases are 5% or less.

The localized stresses at flange connection points decrease when stiffeners are used. These stresses were reduced the most with stiffeners connected to the column flanges only. The magnitude of the stresses is decreased by approximately 14% when compared to the connection without any stiffeners. Stiffeners attached to the column flanges and web also decrease the localized flange stress, but by 10%.

Shear stresses as well as horizontal stresses [10] in the panel zone are reduced by the introduction of transverse stiffeners. Shear stress in the center of the panel zone is used for comparison. Stiffeners connected to the column flanges decrease the stress by less than 1%, while stiffeners connected to both the flanges and web decrease the stress by 5% (based on a stiffener thickness of 9.4 mm (0.3693 in.)).

5.4 Beam-to-Column Connection with Portion of Panel Zone Plastic

The connection modeled in this study has transverse stiffeners between the beam flange connection points. This model simulates test Specimen 1.

Since SAP4 is strictly a linear finite element program, some modifications were made. Highly stressed regions were piecewise non-linearized. Properties of elements were altered in these regions. Both the modulus of elasticity and shear modulus were reduced to one tenth the original carbon steel values.

Specimen 1 test results indicated panel zone non-linearization began to occur at a beam load of 177.9 kN (40.0 kips). In accordance with the test results, the highest elastically stressed region was non-linearized to simulate the onset of yielding. The center 18 plane stress elements of the panel zone were plasticized.

In the model, stress redistribution is the most significant change in the specimen response with a yielded panel zone. Shear stress in bordering elements of the plastic zone consequently approach yield in shear. Highest shear stresses border the tranverse stiffeners in the column web. The longitudinal stresses in the beam flange are greatly increased and rapidly approach the yield strength of the material. Column flange longitudinal stresses also increase, but at a slightly slower rate than in the beam flange.

In addition, the computer model and test results show that stress redistribution increases the stresses on the diagonals of the panel zone. Tension field action is further noticed. As loads are increased, yielding along the diagonals is completed and full plastification is observed.

Panel zone deformation increases significantly with the center of the panel zone yielding. Computer model beam-end deflections increase by 43% (comparison of deflections at 177.9 kN (40.0 kips) before and after non-linearization). The increased beam deflection is similar to the response of the connection during actual testing.

5.5 Beam-to-Column Connection with Portion of Panel Zone Yielded and Column Axial Load Applied

Axial load was applied to the finite element model only to study its effects on the connection stresses. No tests were performed at Lehigh with axial load on the columns.

Stresses obtained from the model show that axial load has a minimal effect on the shear stresses in the panel zone (except when loads approach Py). The column flanges take most of the axial load. Stresses are longitudinal and compressive in the flanges.

The compressive stresses tend to decrease existing tensile stresses caused by beam loads (column flanges). This reduction in tensile stress is advantageous in the region of beam-column flange connection points. At these points, tensile residual stress is high, and a reduced applied tensile load minimizes the possibility of failure.

Chapter 6

SUMMARY OF RESULTS FROM TESTING AND MODELING

Confusion surrounding beam-to-column connections subjected to seismic loads has been reduced as a result of the testing program and the finite element computer model. A number of conclusions have been developed to aid the designer in considering this type of connection problem. For the most part, these conclusions create a greater awareness of the problems associated with the connection and loading. These conclusions are intended to provide the designer with a qualitative feel for the manner in which stresses are transferred in the connection. Therefore, engineering judgment must be exercised in each special case.

6.1 The Effects of Welding

Welds and welding procedures used throughout a moment connection subjected to seismic loading are probably the most critical in design.

Numerous specimens required weld repairs in early test cycles. Fracture at welds dominated these failures. Laboratory repairs were relatively easy and inexpensive, but this would not be the case in field repairs.

Martensitic zones created by rapid cooling in weld areas give brittle properties to an otherwise ductile material. High tensile residual stresses develop in and near heat-affected zones. High residual stresses due to welding combined with residual stresses from fabrication usually force the material to yield prior to load application.

Repeated loading (compressive or tensile) applied to a brittle area usually creates severe conditions. The slightest welding flaws caused by hot cracking, cold cracking, or inclusions can initiate early failure.

Eliminating welding completely would be unwise because of the economy associated with this fastening technique. Therefore, local applied stresses must be minimized in weld regions.

The ductility of a connection subjected to seismic loads is critical. The panel zone must be ductile enough to withstand the cyclic loads while providing satisfactory stiffness to control drift. Welded connections must have the strength and ductility to transfer loads to the panel zone without premature failure.

6.2 Design of the Panel Zone for Shear

The current AISC Code design procedure describes failure of the panel zone with the following equation.

$$Vu = Fy/\sqrt{3} \times 0.95dt = 0.55Fydt$$

where,

Fy - Yield strength of column web

$Fy/\sqrt{3} = 0.58Fy$ Based on von Mises Yield Stress Failure Criteria

d - Depth of the column
(95% of depth used)
t - Thickness of the column web

Edward J. Teal in "Tests of Panel Zone Behavior in Beam-Column Connections" [1], his analysis of a Lehigh report, states, "There are presently several bases for computing allowable shear stress. The first, of course, is the von Mises shear stress of $Fy/\sqrt{3}$ or 0.58Fy, which is often referred to as the ultimate shear. This is theoretically correct if the word ultimate is changed to yield. There seems to be no doubt that any part of a panel zone which reaches this stress will start to yield."

The code also allows a working stress of 1.33 x 0.4Fy or 0.53Fy for wind and seismic design.

Failure never occurred in the panel zone in any test case. The zone was severely deformed, but with no fracture in the region. The shear capacities of the specimens tested are listed below (governed by failure outside of the panel zone).

Specimen 1 .... 2.54x0.58Fydt at Pbu=311 kN (70 kips)
Specimen 2 .... 3.20x0.58Fydt at Pbu=498 kN (112 kips)
Specimen 3 .... 3.23x0.58Fydt at Pbu=494 kN (111 kips)
Specimen 4 .... 3.95x0.58Fydt at Pbu=494 kN (111 kips)

Specimen 1A .... 2.75x0.58Fydt at Pbu=245 kN (55 kips)
Specimen 3A .... 2.75x0.58Fydt at Pbu=378 kN (85 kips)

All values based on a moment arm of 1651.0 mm (65.0 in.)
and a beam depth of 603.0 mm (23.74 in.).

d - Depth of the column
(95% of depth used)
t - Thickness of the column web

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Specimen 4 .... 3.95x0.58Fydt at Pbu=494 kN (111 kips)

Specimen 1A .... 2.75x0.58Fydt at Pbu=245 kN (55 kips)
Specimen 3A .... 2.75x0.58Fydt at Pbu=378 kN (85 kips)

All values based on a moment arm of 1651.0 mm (65.0 in.)
and a beam depth of 603.0 mm (23.74 in.).

Also, an average material yield strength is used for specimens with doubler plates.

Pbu - Maximum beam load

It is obvious that von Mises yield stress failure criteria is too conservative for panel zone design. The general load and resistance design safety factor applied to steel is 1.7. This value can be applied to 0.4Fy for shear which defines the ultimate shear strength as 0.68Fy, exceeding von Mises failure criteria. Strain hardening of the panel zone is a valid qualitative justification for the increase in shear capacity, but there should be further reasoning.

Column sections are the governing factor in the strength of the panel zone. As the section sizes increase, so does the residual stress due to fabrication. Therefore, ductility of the section decreases. Light column sections are affected less by residual stresses and consequently have greater ductility. In addition, as the yield and ultimate strengths of the material increase, the ductility generally decreases. A yield stress criteria must consider this ductility.

Based on tests and the computer model, it is clear that an upper and lower bound for analysis must be used. These bounds define the design procedure and allow for full development of the column web. However, an equation must be used to define the shear strength between the upper and lower bounds. This equation is:

$$Vu=0.5FuTt$$

where,
Vu - Ultimate shear strength of col. web
Fu - Ult. tensile strength of col. web
T - Distance between flange fillet welds
t - Thickness of column web

This formula applies to wide flange shapes only.

This equation is based on a reduced area (area of column web only) and half of the material ultimate strength. The equation is effective because it is governed by the column web only. The ratio of T/d (comparing parameters of proposed equation to von Mises equation) decreases with increased column sizes, therefore, decreasing the shear strength capacity. As the column size decreases, the ductility increases along with the ratio of T/d, thus, increasing the shear strength capabilities. Also, higher strength materials have a Fy/Fu ratio which approaches 1.0 formulating a value for Vu that is less than ultimate values based on Fy.

The lower bound of the equation is von Mises yield stress criteria which has been proven to be correct. The upper bound of the equation is the allowable shear stress multiplied by the load and resistance design safety factor. The two equations are listed below.

Lower Bound: $Vu=Fy/\sqrt{3}\ xdt=0.58Fydt$

Upper Bound: $Vu=1.7(0.4)Fydt=0.68Fydt$

where,
Vu - Shear strength of column web
Fy - Yield strength of column web

d - Full column depth
t - Thickness of column web

Transverse stiffeners introduced into the connection reduce the shear stress in the panel zone. This reduction is based on the stiffener thickness. The larger the thickness, the greater the reduction. There is an upper bound to the shear stress reduction. As the stiffener thickness increases to large values, shear stress reduction becomes minimal. Figure 26 illustrates the relationship between percent panel zone shear stress reduction and the non-dimensional ratio of stiffener thickness to web thickness for the specific connection analyzed in this document.

The relationship shown in Figure 26 suggests that a 5% increase in the panel zone shear stress capacity should be considered in design (based on 9.5 mm (0.375 in.) stiffeners). Therefore, the following adjustments are made on the equations dictating shear stress capacity of the panel zone.

Lower bound:
$$Vu=1.05(0.58)Fydt = 0.61Fydt$$

Design equation:
$$Vu=1.05(0.5)FuTt = 0.53FuTt$$

Upper bound:
$$Vu=1.05(0.68)Fydt = 0.71Fydt$$

It should be noted that the increased capacity of the panel zone when stiffeners are used is based on results from the finite element model.

6.3 Panel Zone Reinforcement

Doubler plates and transverse stiffeners are two means of panel zone reinforcement used to accomodate panel shear and column flange buckling respectively.

Doubler plate design is simple, but web attachment techniques should be evaluated further. The thickness of doubler plates under the current AISC design code is calculated as follows.

$$tdp = [V/0.95(0.58)Fyd] - t$$

where,

tdp - Thickness of doubler plate
V - Applied shear force
Fy - Yield strength of column web
d - Depth of column web
t - Thickness of column web

This calculation is only done when the column web does not have the capacity to carry the shear.

Utilizing full penetration welds for doubler plate attachment increases the heat-affected zones, and consequently creates large brittle regions. Since ductility is vital to the cyclic capacity of the panel zone, other methods of welding must be used.

Fillet welding increases ductility while decreasing costs. Since panel zones are subjected to tension field action, buckling of the doubler plate is unlikely. Even in large plates, intermediate connection points using plug welds are not needed. Plug welds create additional residual stresses and stress concentrations, decreasing the panel zone ductility.

Transverse stiffeners used to restrain the column flanges in compression and tension are designed by the following procedure.

$$Fyc(twc)[tfb+5kc] \leq Afb(Fyb) \quad \text{Compression}$$

or

$$d > [4100(twc)^3\sqrt{Fyc}] \,/\, Pfb \quad \text{Compression}$$

and,

$$tfc < 0.4\sqrt{Pfb/Fyc} \quad \text{Tension}$$

If any of the above relationships are true, the following stiffener area is required.

$$Ast=[Pfb-Fyc(twc)[tfb+5kc]] \,/\, Fyst \quad \text{Compression or Tension}$$

where,

Fyc - Yield stress of column
twc - Thickness of column web
tfb - Thickness of beam flange
kc - Fillet size of column
Afb - Area of beam flange
Fyb - Yield stress of beam
Pfb - Force delivered by beam flange (x4/3 for earthquake forces)
d - Depth of column web
tfc - Thickness of column flange
Ast - Cross-Sectional area of stiffener
Fyst - Yield stress of stiffener

Stiffeners should be connected to both the column web and flanges with fillet welds. This provides ductility and decreases panel zone shear stress which was proven with the finite element model.

6.4 Plastic Hinge Development

Development and location of plastic hinges are critical in beam-to-column connections subjected to seismic loads. For proper hinge development, the overall frame design along with member design must be considered.

Stiff panel zone construction forces a plastic hinge to develop in the reduced beam section adjacent to the column flange. Qualitatively, the loading response of the beam resembles the curve in Figure 27. The beam has no reserved strength after the hinge is formed. In the structure, load will be redistributed after hinges form. But, hinges forming in beams near column flanges may accelerate mechanism formation. Collapse of the structure may be rapid.

The designer must allow the first plastic hinge to form in the panel zone. Once the hinge is formed, both the strength and the deformation of the zone will continue to increase, only the deformation will increase at a faster pace than before the hinge formed. Specimens 1, 3 and 1A exhibit this response the best (see hysteresis curves in Figures 7, 9 and 11). The idealized qualitative response of the panel zone needed in design is shown in Figure 27.

Strain hardening of the panel zone occurs with large zone deformations. These deformations rotate beam ends without forcing the beam ends to reach plastic moment capacity while creating plastic hinges in column flanges. As rotations increase, the plastic moment is forced to form at the beam midspan. Maximum structural drift capacity is exhausted after all panel zones have deformed and beam midspans have reached plastic moment capacity. A schematic illustrating the sequence of hinge formation is shown in Figure 28.

6.5 Stress Concentrations

Stress concentrations have a significant effect on connection vitality during a seismic response. Local stresses at flange connection points, undersized beam flanges and flame cut edges deserve close attention from the designer.

Stress concentrations in the finite element model increase substantially at the onset of panel zone yielding. As the panel zone forms a plastic hinge, the condition intensifies. The stresses from the model are independent of residual stresses which are difficult to quantify, but create even worse conditions.

Flanges connected by welds should have comparable widths. Beam flanges with widths smaller than column flanges create additional stress concentrations at beam flange tips. Also, stress transferral is not evenly distributed across the column flange.

Flame cut edges used in welding flanges together also create the threat of premature failure. Sharp edges, high in residual stresses, can cause fracture under low fatigue cycles.

6.6 Panel Zone Deformation and Yielding Patterns

The Uniform Building Code (UBC) [11] states that connections subjected to seismic forces must withstand 0.5% drift. In a severe seismic event, the drift could be at least three times this amount, or 1.5%. In testing the specimens, it was assumed that drift would be caused by the deformation of the panel zone. In an actual structural frame, the drift consists of four components:

1. Column bending due to flexure and shear.

2. Beam bending due to flexure and shear.

3. Bending of the frame as a whole due to column axial strains.

4. Panel zone distortion due to shear and bending stresses.

Panel zone deformation, in every test case, equaled or exceeded 1.5% drift. The maximum percent rotations (drift) for all specimens are listed below.

CONNECTION	MAXIMUM ROTATION	# OF CYCLES
Specimen 1	6.2%	7
Specimen 2	2.3%	7
Specimen 3	2.7%	7
Specimen 4	2.7%	7

Specimen 1A	3.1%	4
Specimen 3A	1.4%	4

The maximum rotation of Specimen 3A is considered to be within reasonable error of 1.5%.

The panel zones were deformed to these rotations without any zone failure. Failure did occur near welded flange connection points prior to the maximum recorded cycles in all specimens except Specimen 1.

Initial yielding in the panel zone closely resembles tension field action seen in plate girders. However, conditions are slightly different. Panel zone boundaries are closer to being fixed than simple, as in the case of plate girder design. Therefore, quantative estimation is difficult to formulate, and further research is needed. Regardless, the initial yield lines in all test specimens formed along a diagonal of the zone. Figure 14 shows an example of a typical yield pattern. In addition, Figure 22 illustrates the theory behind the tension field concept.

Questions may be raised as to which diagonal yields first because in most engineering applications materials are assumed to have the same yield strength in both tension and compression. Residual stresses provide the answer. These stresses tend to shift the neutral axis of the beams, causing higher applied stresses at certain flanges. If the axis shifts upward in one beam, the axis shifts downward in the other, and visa versa. Column flanges subjected to higher stress initiate yielding and cause one diagonal to yield before the other. Figure 29 illustrates this concept.

Chapter 7

POSSIBILITIES FOR FURTHER RESEARCH

In some segments of this document, more questions are raised than answered. Research is needed to quantify qualified aspects of this connection problem. Early assumptions about the panel zone behavior were conservative, and other portions of the connection were misunderstood. The following sections attempt to make researchers aware of possible testing procedures which may lead to simplified connection descriptions, and ultimately, to aids for designers.

7.1 Severity of Connection Tests

The connection tests conducted to date have clearly indicated a severe loading situation. The cruciform connection is basically tested as a bracket. Beams are not allowed to deform as they would in an actual structure. Also, stress concentrations at flange connection points are higher in the tests than in an actual frame.

The connection should be made with full-span simply supported beams or half-span cantilevered beams. This connection test would more closely resemble field conditions. Figure 30 describes the possible test specimen loading procedures. The results from future tests could then be compared to the existing test results to show similarities or contrasts.

7.2 Plastic Hinge Development

Restricting hinge development to certain locations within a frame is another area which needs further research. Connections should be designed to allow a plastic hinge to develop in the panel zone, column flanges and beam midspan rather than forcing hinges to develop near column flanges where reduced beam sections are located.

This hinge formation will provide ductility for the overall frame while reducing the possibility for rapid failure mechanisms.

7.3 Description of Column Load Effects

Some controversy has centered on the effects of axial load on panel zones. A non-dimensionalized relationship between axial load and panel zone shear stress must be developed.

Low axial loads have proven to be insignificant in affecting panel zone behavior (finite element model results have shown this). The magnitude of axial loads that change panel zone behavior must be quantified.

7.4 Panel Zone Reinforcement

Further research should be conducted into the effects of fillet welding doubler plates to column webs. Designers must be convinced that this technique of welding provides an adequate, ductile, economical answer to doubler plate design.

Increasing the stiffness of the panel zone forces some welded areas to become critical. High stiffness usually causes failure to occur outside of the column web. If this stiff design is used, tests are needed to determine the adequacy of full-bolted moment connections.

Positive moment rotation of neighboring beams is essential. All connection reinforcement must be designed to create this response.

7.5 Sections Fabricated for Seismic Design

Rules should be developed for beam and column sections used in seismic design. For instance, beam flange sizes might be increased to become compatible with column flanges. Also, hybrid column sections should be investigated for possible use.

Designers must consider overall structure design rather than the design of a group of connecting members. Drift and inelastic seismic response of the entire structure play a vital role in seismic design.

7.6 Panel Zone Region Designed as a Frame

Researchers should investigate the possibility of designing the panel zone as a frame. If the surrounding boundary would be reinforced with heavy stiffeners, the column web would then act as a stiffening agent for the boxed frame.

The frame must be designed in accordance with requirements for connection ductility and structural drift control. Plastic hinges (four) must be forced to develop in the column flanges. The frame must withstand shear and axial forces. Results from the computer model prove that most of axial forces are carried by the column flanges.

Figure 31 shows the frame analogy concept. This figure also illustrates the forces acting on the frame which must be considered in design.

Chapter 8

CONCLUSIONS

Concepts regarding the response of connections subjected to cyclic loads have been studied and clarified in this thesis. It is important that designers use the results and the following conclusions when considering the connections in structural applications.

1. Von Mises shear stress yield criteria has been proven to be excessively conservative in describing failure of the panel zone due to shear. The design equation (based on the column web ultimate strength) should be used as the formulation, considering upper and lower bounds.

2. Transverse stiffeners based on static design should be used. These stiffeners prevent column flange deformation while reducing critical stress concentrations at flange connection points. These stiffeners also reduce shear stresses in the panel zone. In addition, the adverse effects of flange welding are reduced with the reduction of stress concentrations.

3. Doubler plates should be used in design when column webs are inadequate in withstanding shear forces. These plates should only be used after transverse stiffeners and the panel zone shear stress equation have been considered. Also, fillet welds should be used to connect the plates to the column web

and flanges.

These general design conclusions were formulated from a specific connection and loading but can be applied to other connections with similar configurations and loadings. Each concept has been carefully researched and the author is confident that applications of the results and conclusions will aid designers when considering beam-to-column connections subjected to seismic loads.

TABLE

SPECIMEN	YIELD STRENGTH MPa(ksi)	TENSILE STRENGTH MPa(ksi)	% ELONGATION %
SPECIMEN 1 & 2			
Beam Flange	288.3(41.8)	442.1(64.1)	27.4
Beam Web	288.3(41.8)	455.2(66.0)	32.9
Column Flange	260.0(37.7)	515.8(74.8)	31.7
Column Web	291.0(42.2)	511.7(74.2)	29.9
Doubler Plate 2	437.9(63.5)	573.1(83.1)	21.8
Transverse Stiff.	273.1(39.6)	424.8(61.6)	26.5
SPECIMENS 3 & 4			
Beam Flange	282.8(41.0)	450.3(65.3)	29.8
Beam Web	326.9(47.4)	465.5(67.5)	31.8
Column Flange	269.6(39.1)	420.7(61.0)	33.3
Column Web	291.7(42.3)	443.3(64.3)	29.9
Doubler Plate 3	424.8(61.6)	560.0(81.2)	34.8
Doubler Plate 4	300.0(43.5)	482.7(70.0)	29.1
Transverse Stiff.	277.2(40.2)	437.9(63.5)	28.3
Connection Plate	293.1(42.5)	465.5(67.5)	29.9
SPECIMENS 1A & 3A			
Beam Flange	309.0(44.8)	470.3(68.2)	28.4
Column Flange	259.3(37.6)	437.2(63.4)	33.1
Column Web	289.0(41.9)	463.4(67.2)	31.4
Doubler Plate 3A*	358.6(52.0)		

*Approximate yield strength based on mill report

Table 1: Material Properties of Test Specimens

FIGURES

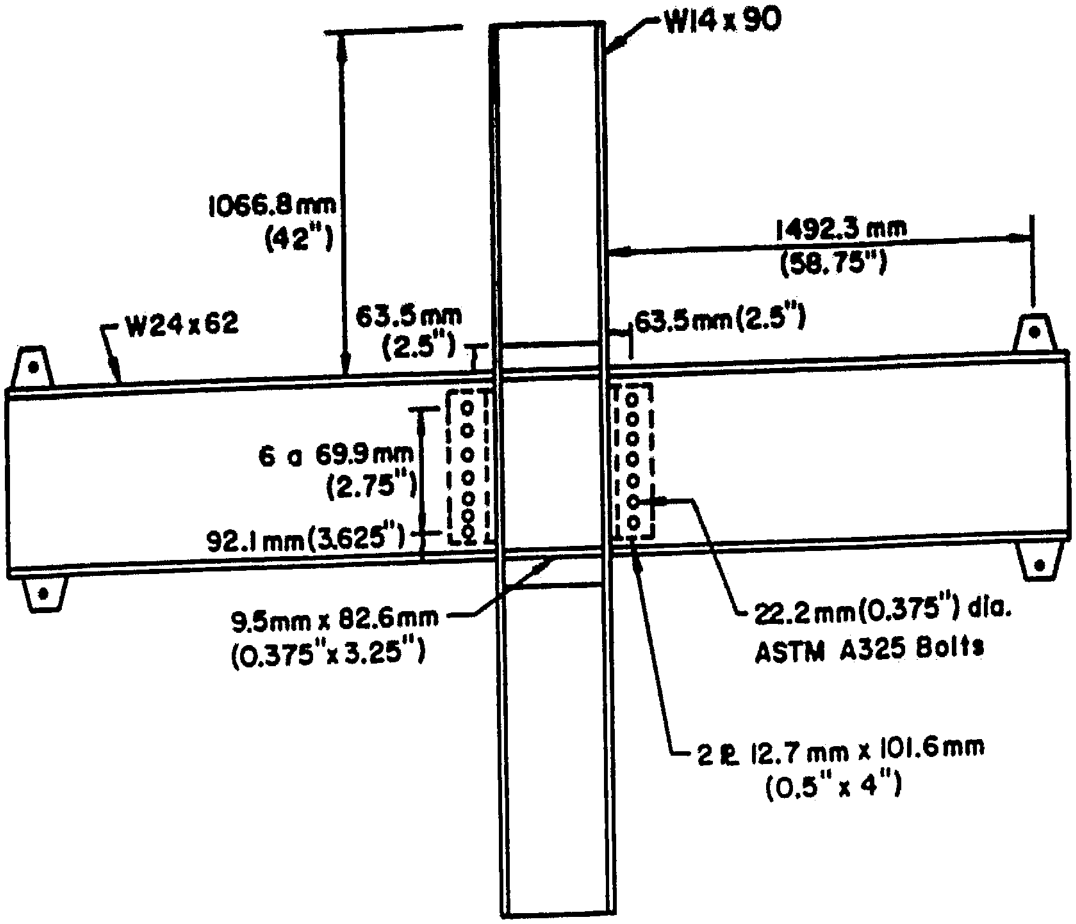


Figure 1: Dimensions of Test Specimens

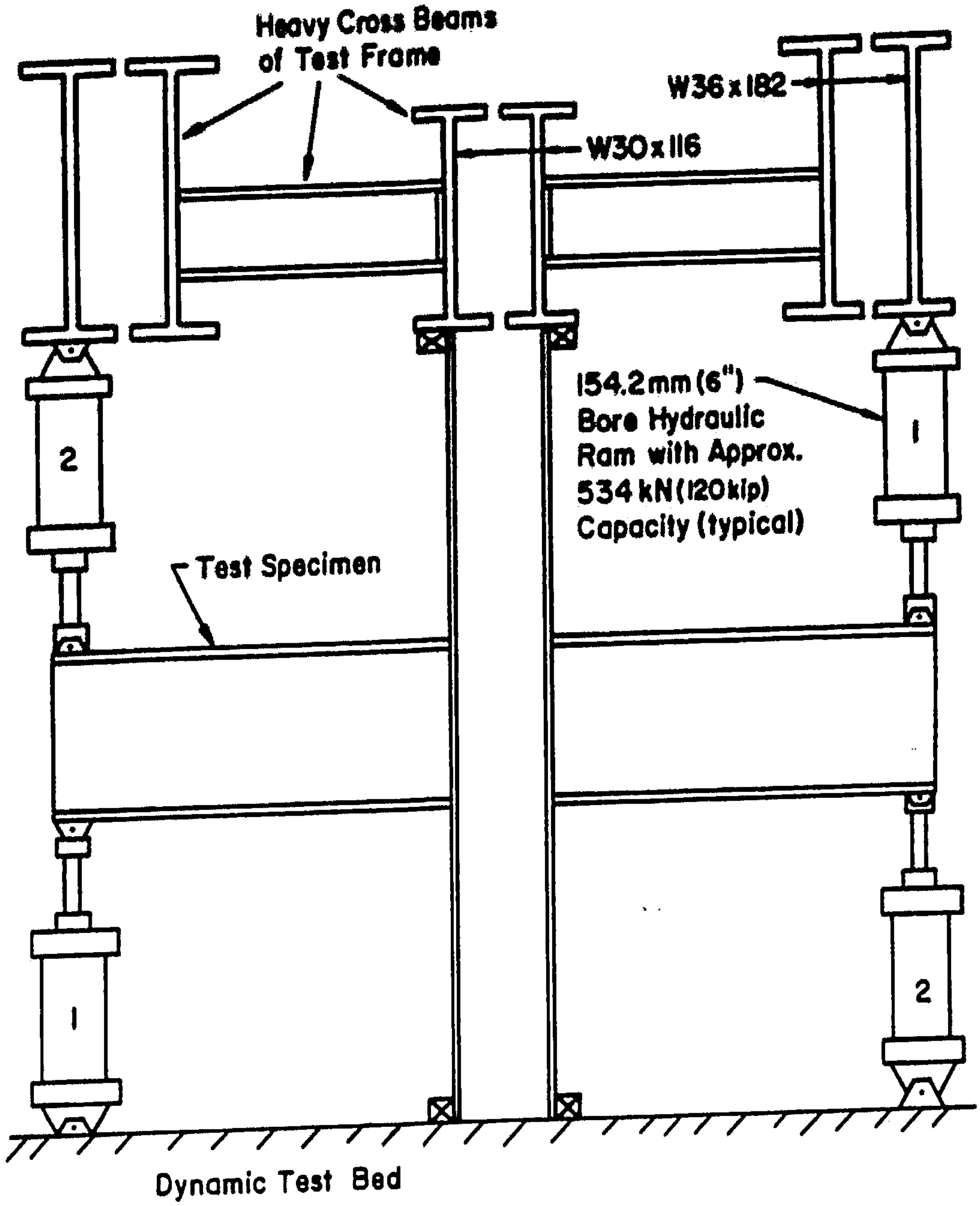


Figure 2: Loading Configuration of Test Specimens

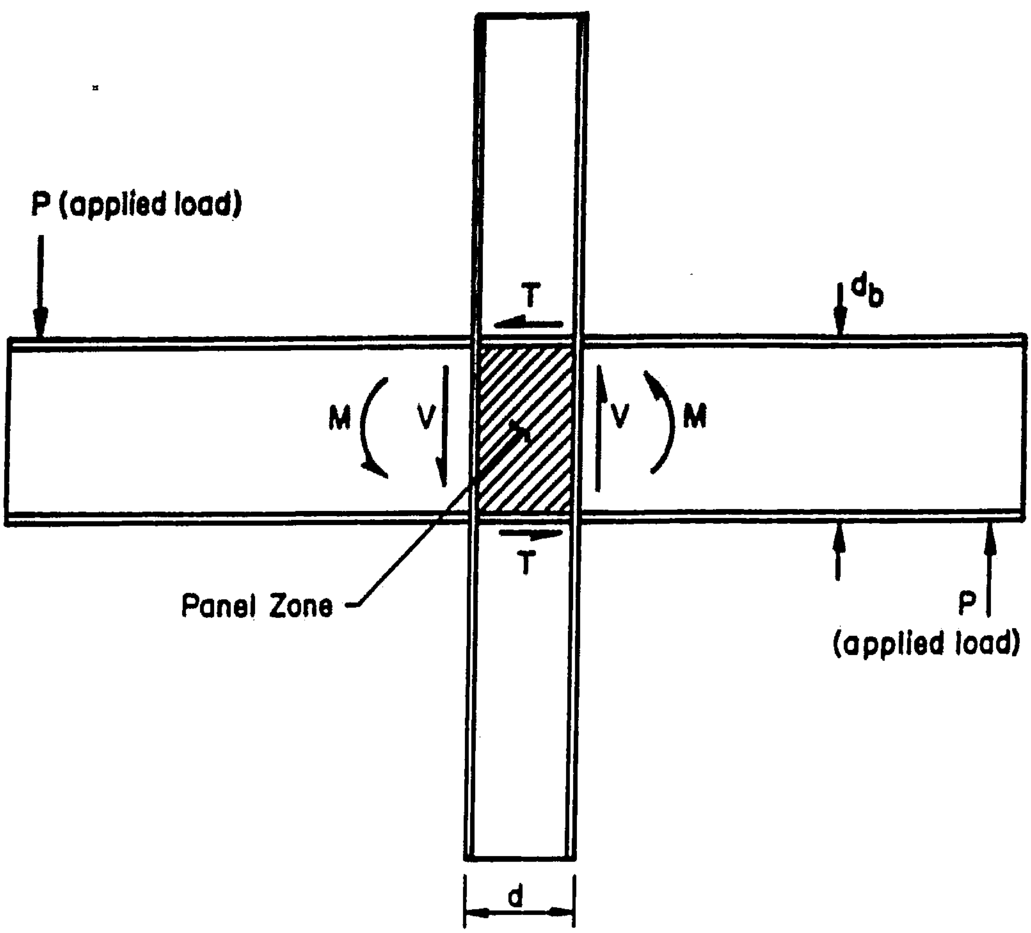


Figure 3: Forces Acting on and Location of Panel Zone

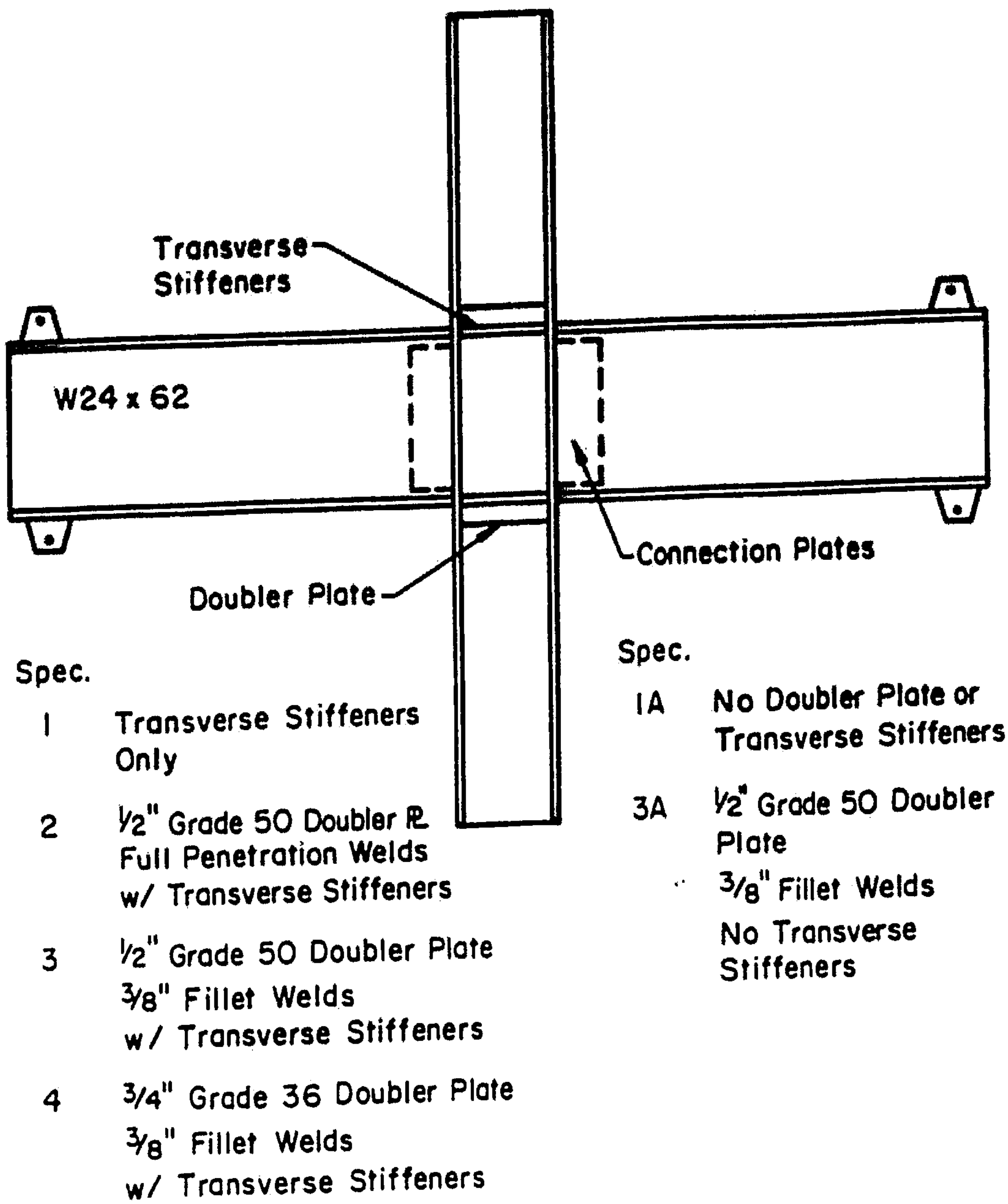


Figure 4: Test Specimen Component Description

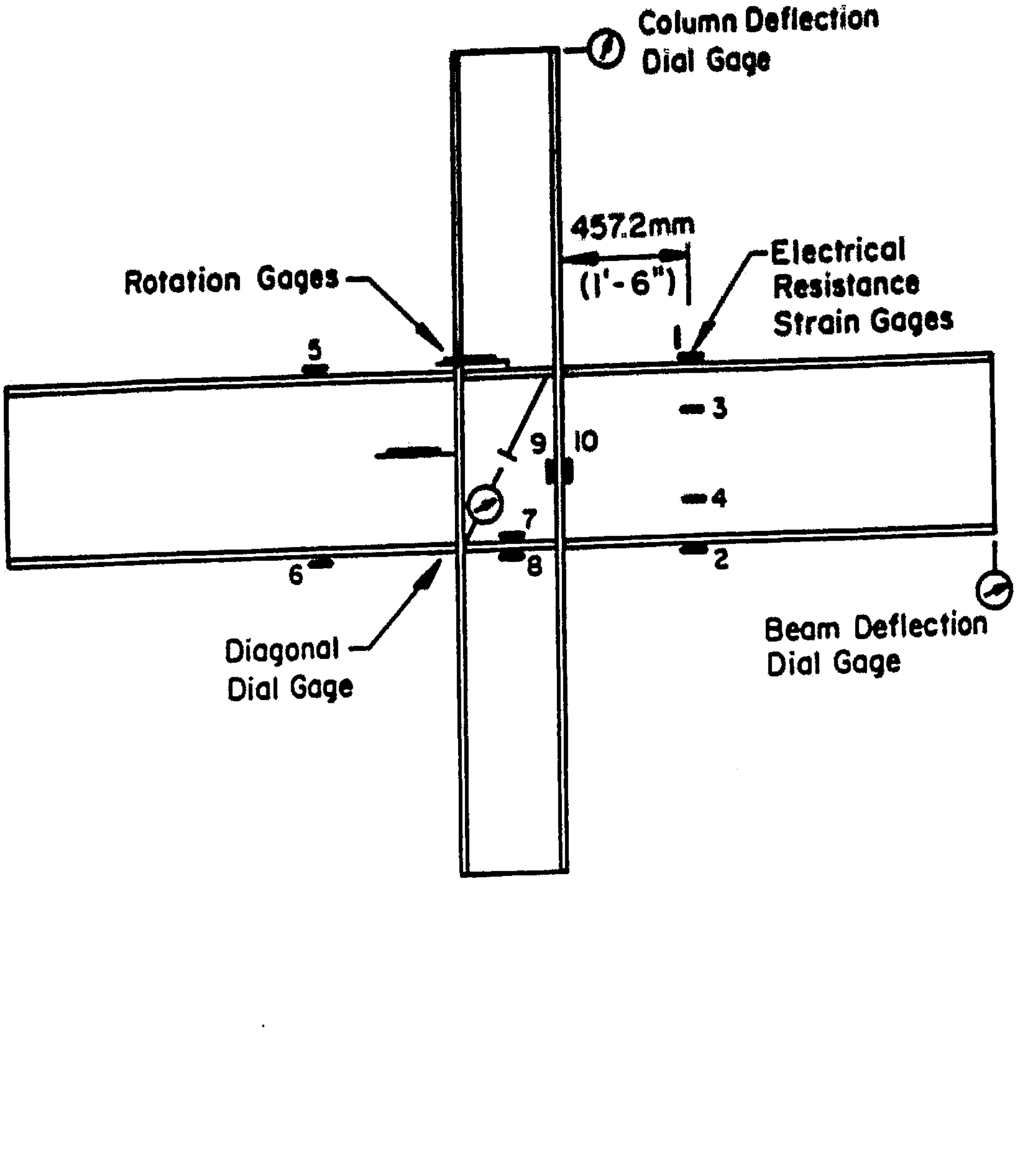


Figure 5: Instrumentation for Test Specimens 1-4

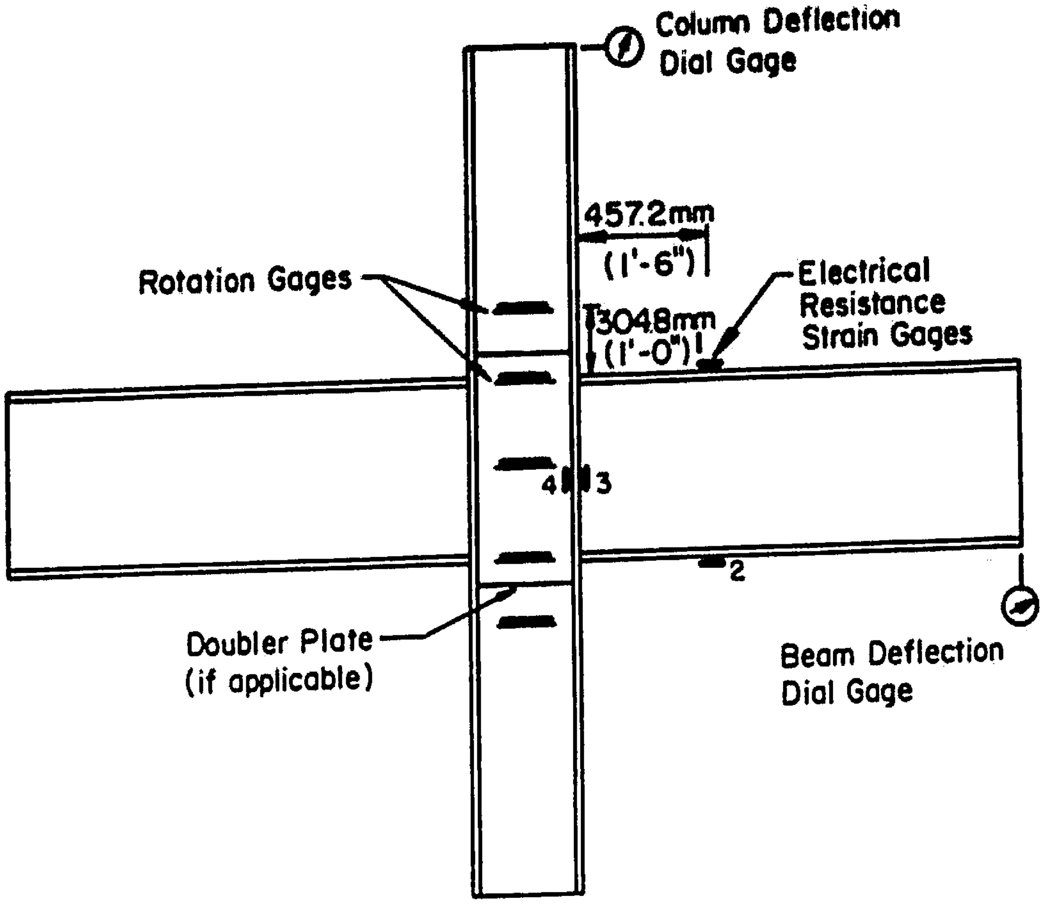


Figure 6: Instrumentation for Test Specimens 1A and 3A

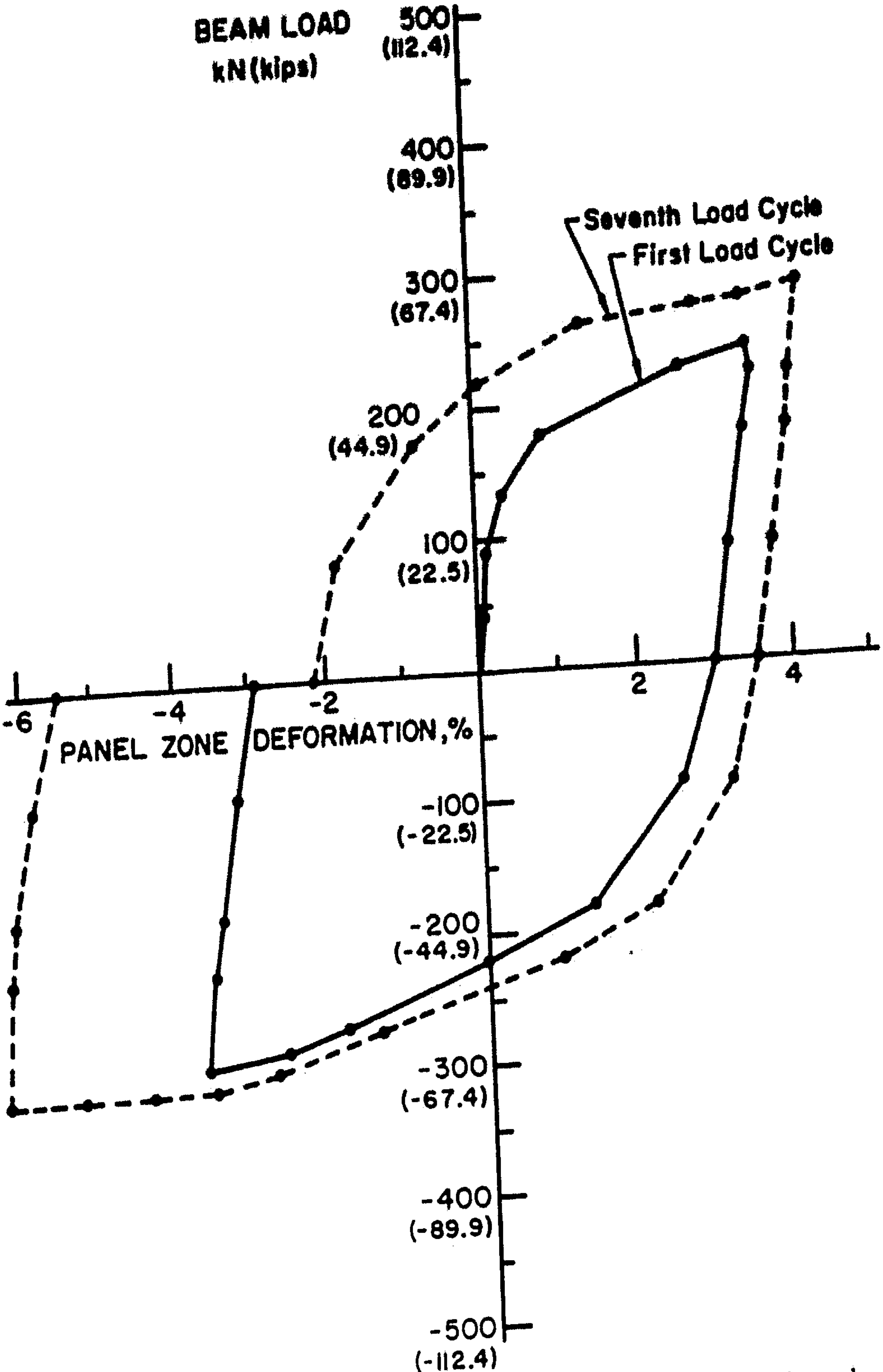


Figure 7: Beam Load vs. Panel Zone Deformation - Specimen 1

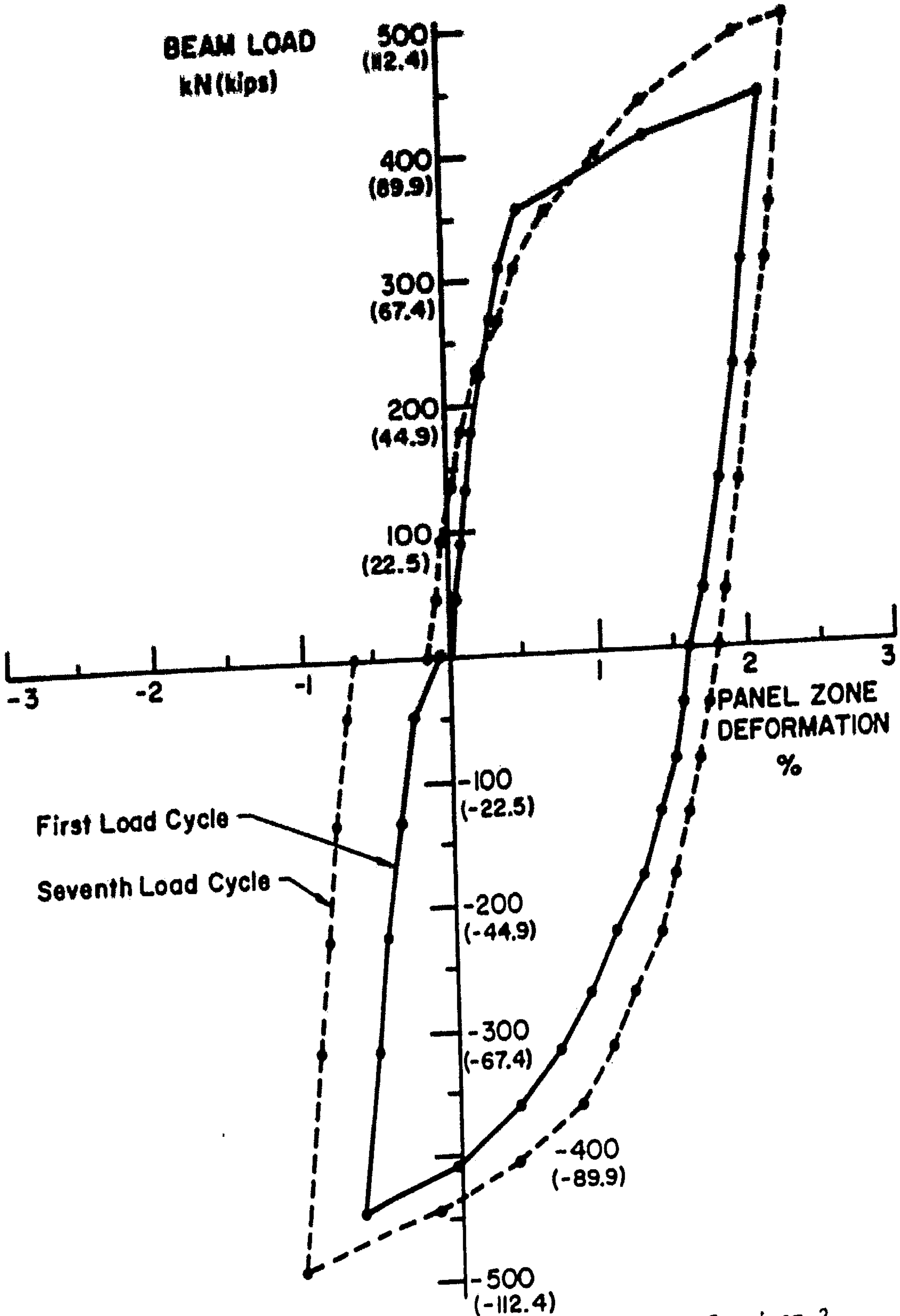


Figure 8: Beam Load vs. Panel Zone Deformation - Specimen 2

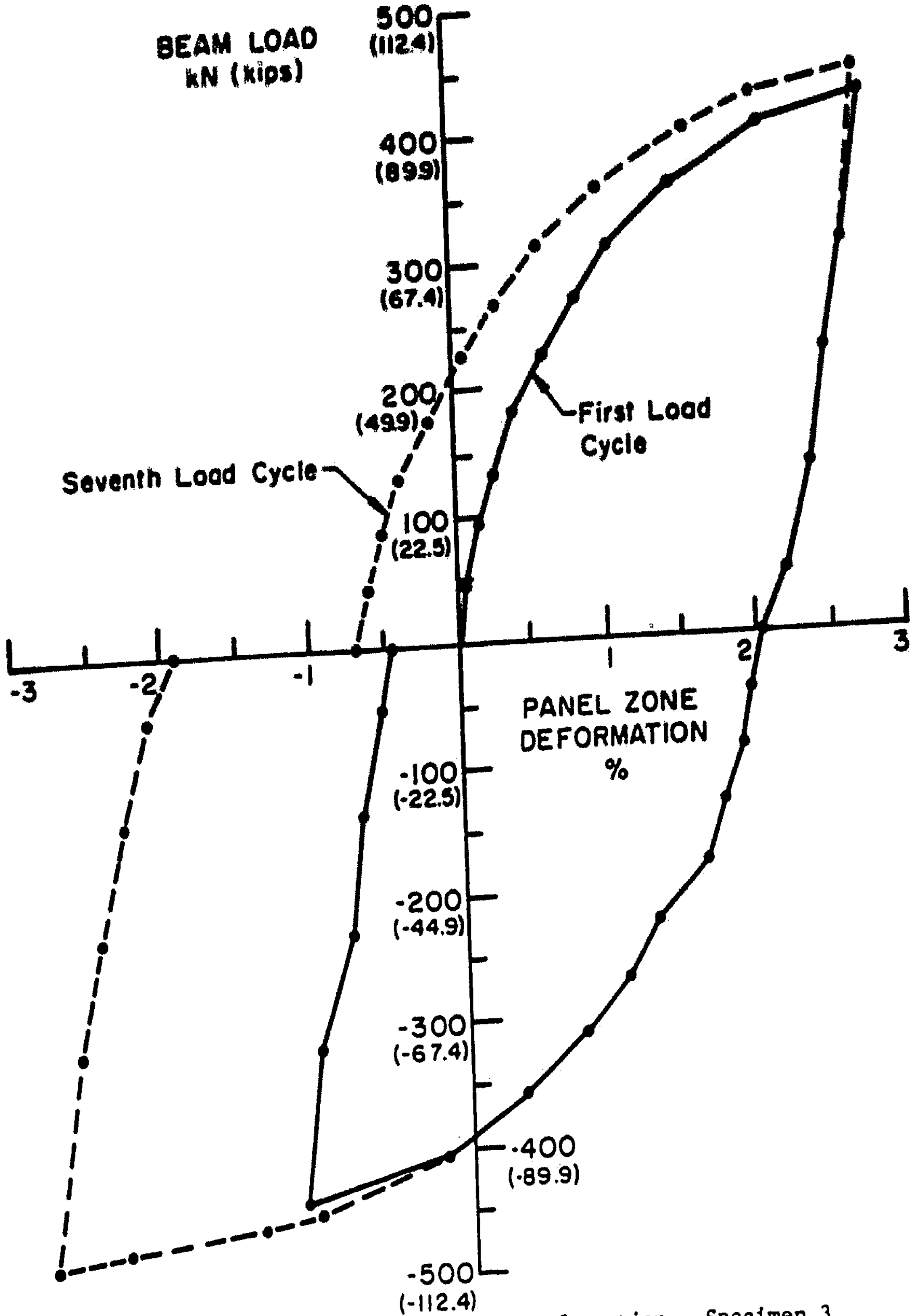


Figure 9: Beam Load vs. Panel Zone Deformation - Specimen 3

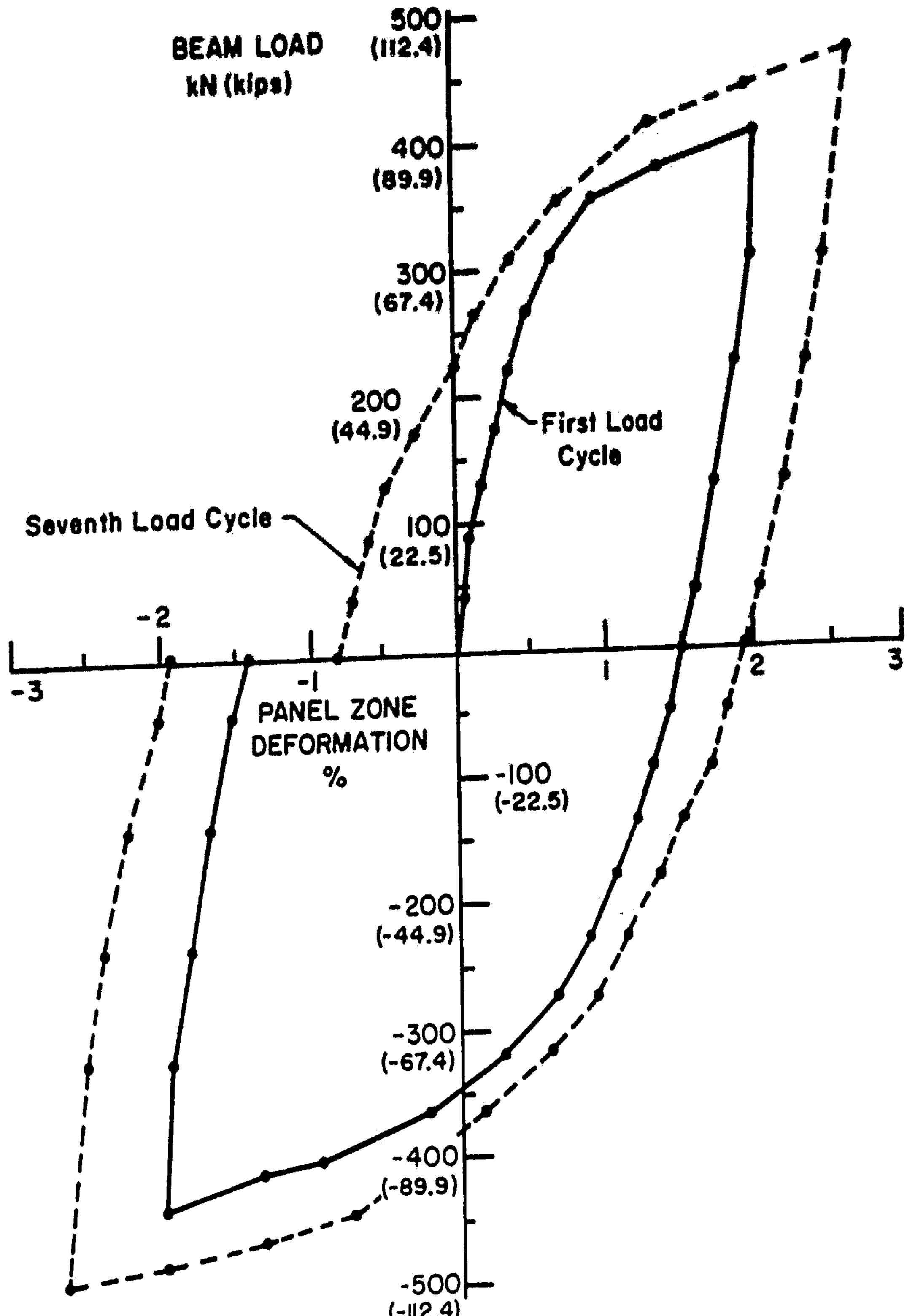


Figure 10: Beam Load vs. Panel Zone Deformation - Specimen 4

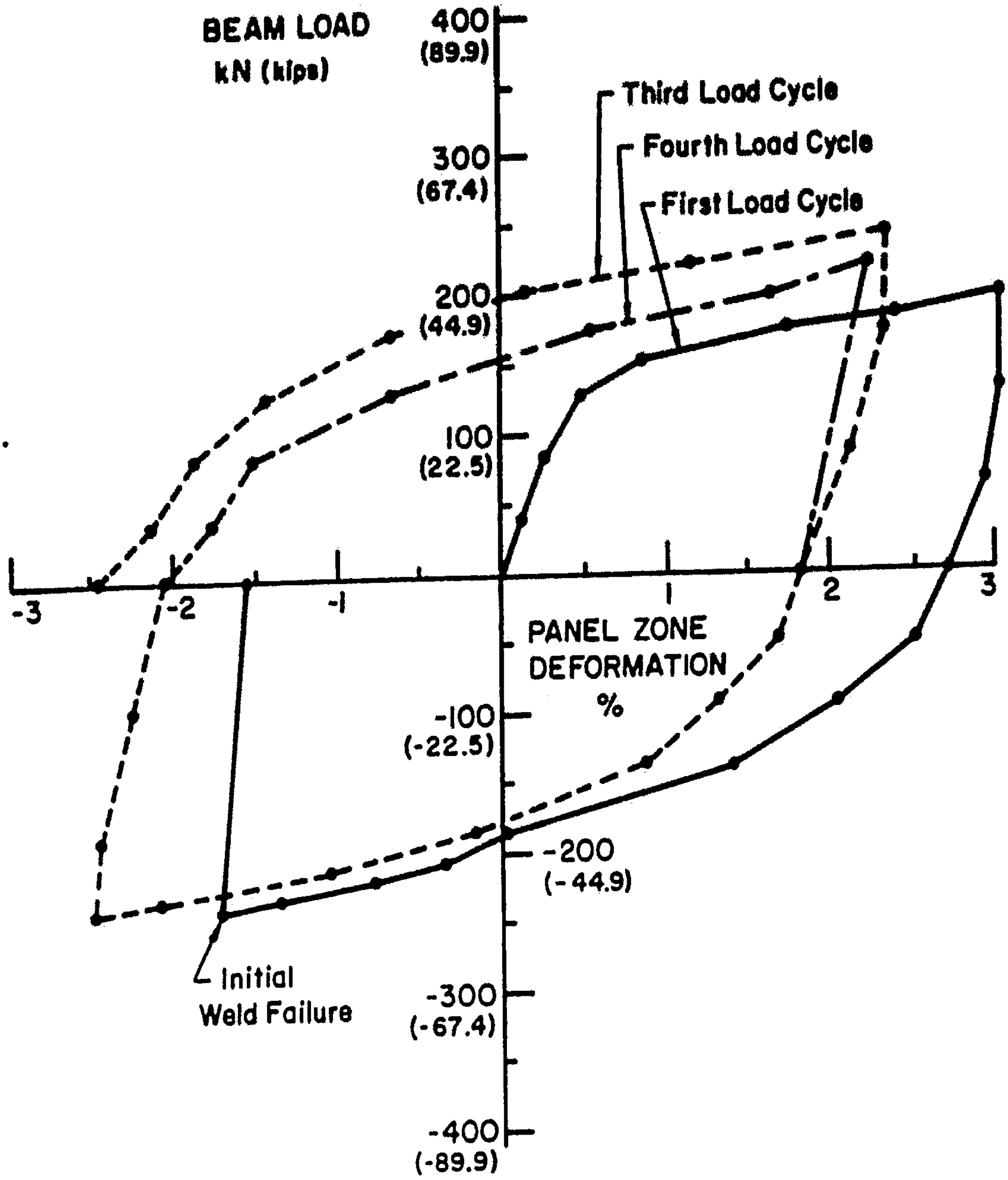


Figure 11: Beam Load vs. Panel Zone Deformation - Specimen 1A

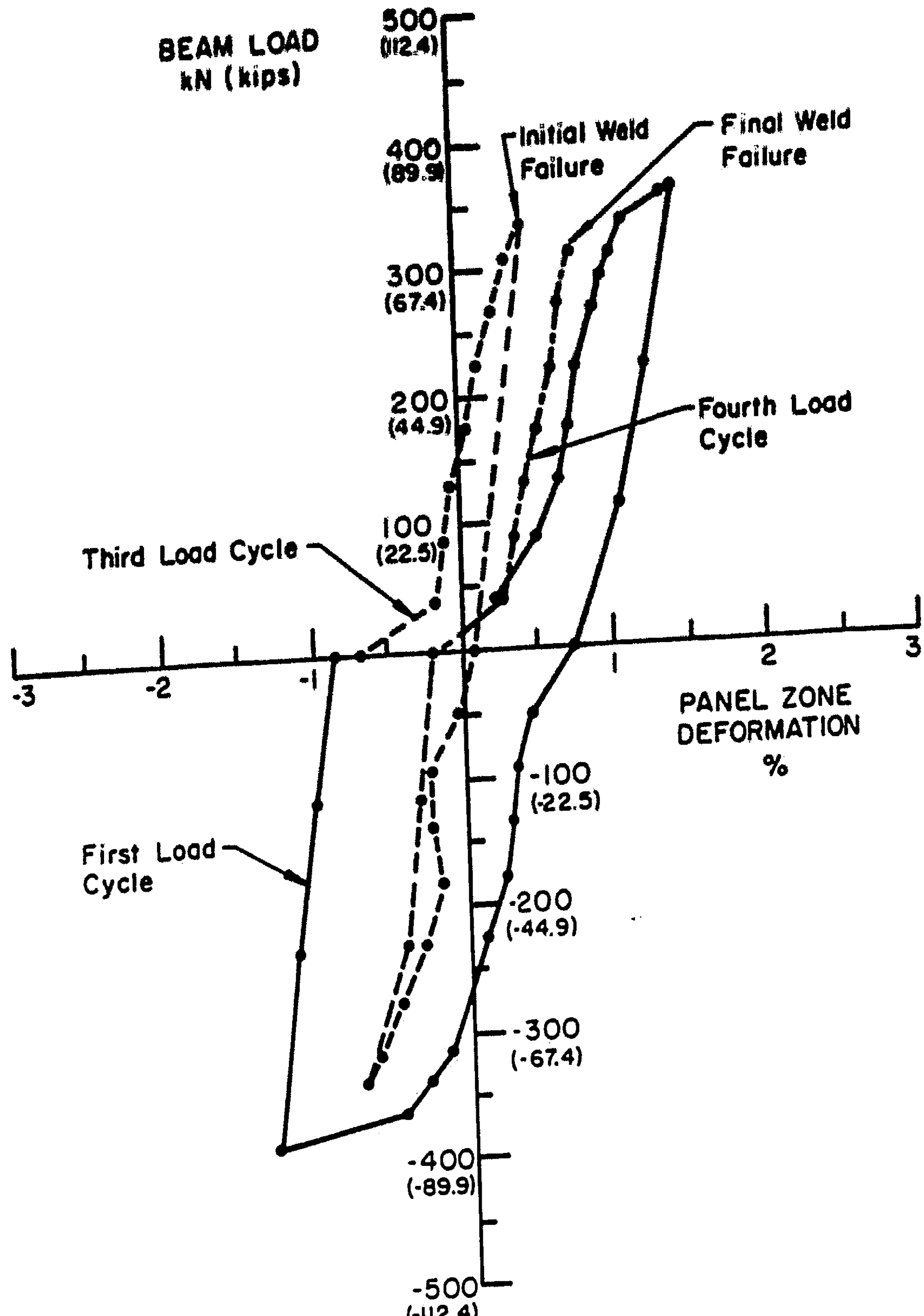


Figure 12: Beam Load vs. Panel Zone Deformation - Specimen 3A

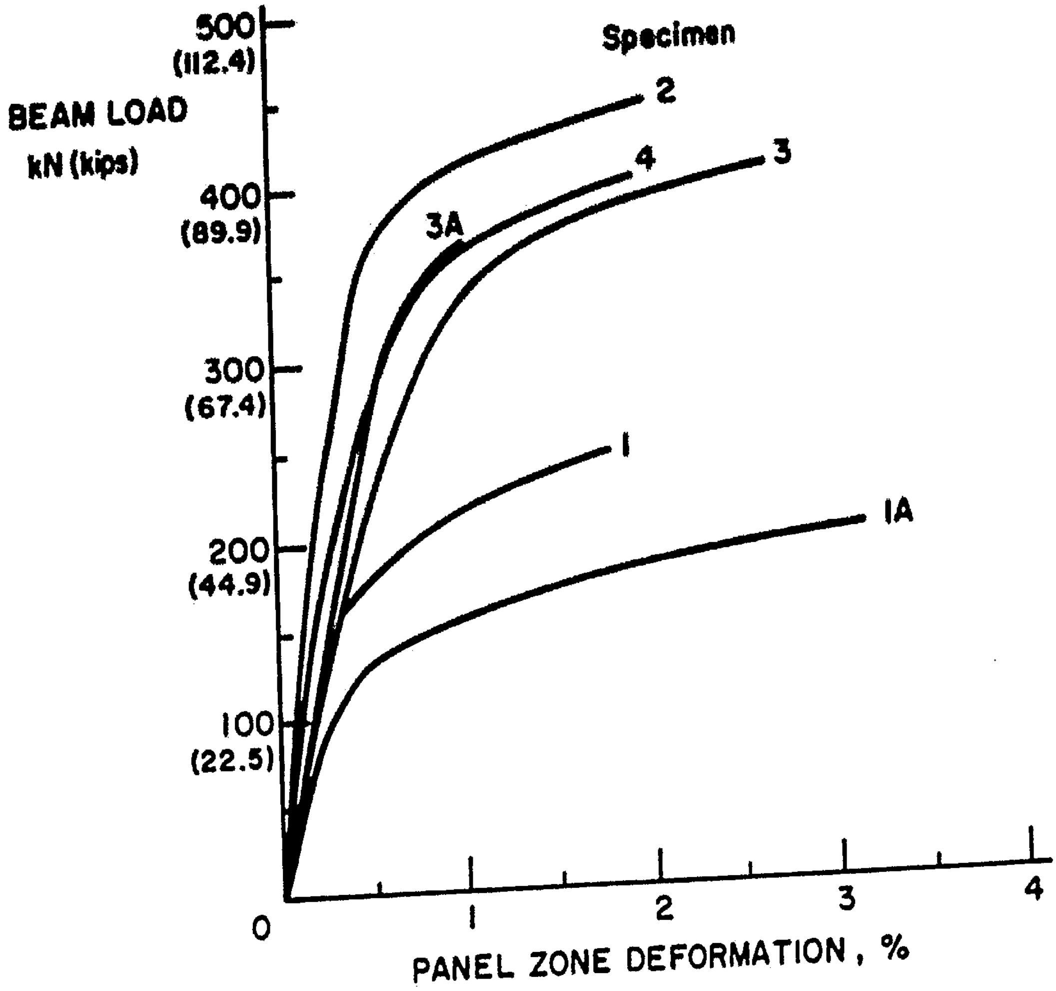


Figure 13: Beam Load vs. Panel Zone Deformation - First Half Cycle

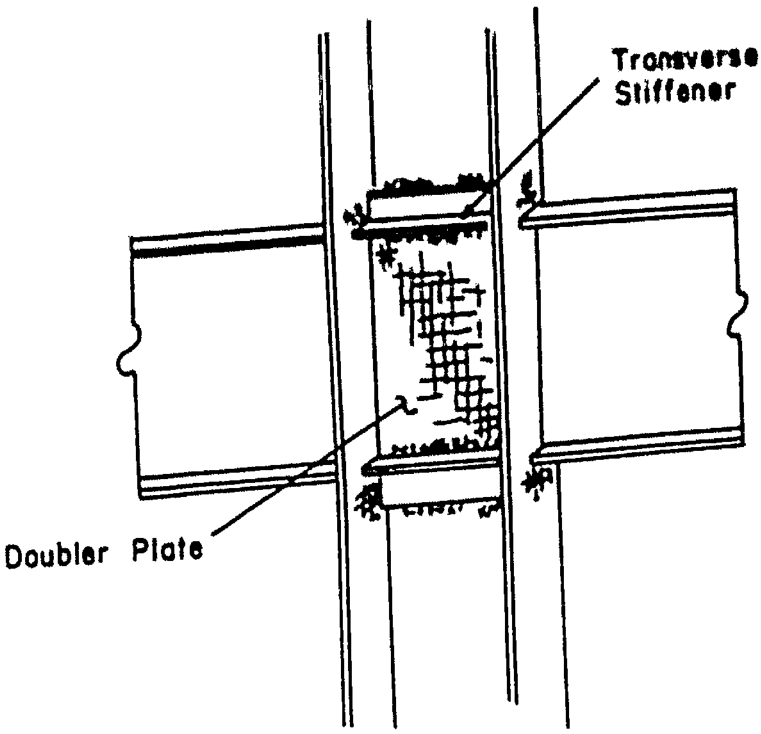


Figure 14: Yield Lines in Specimens 3 & 4

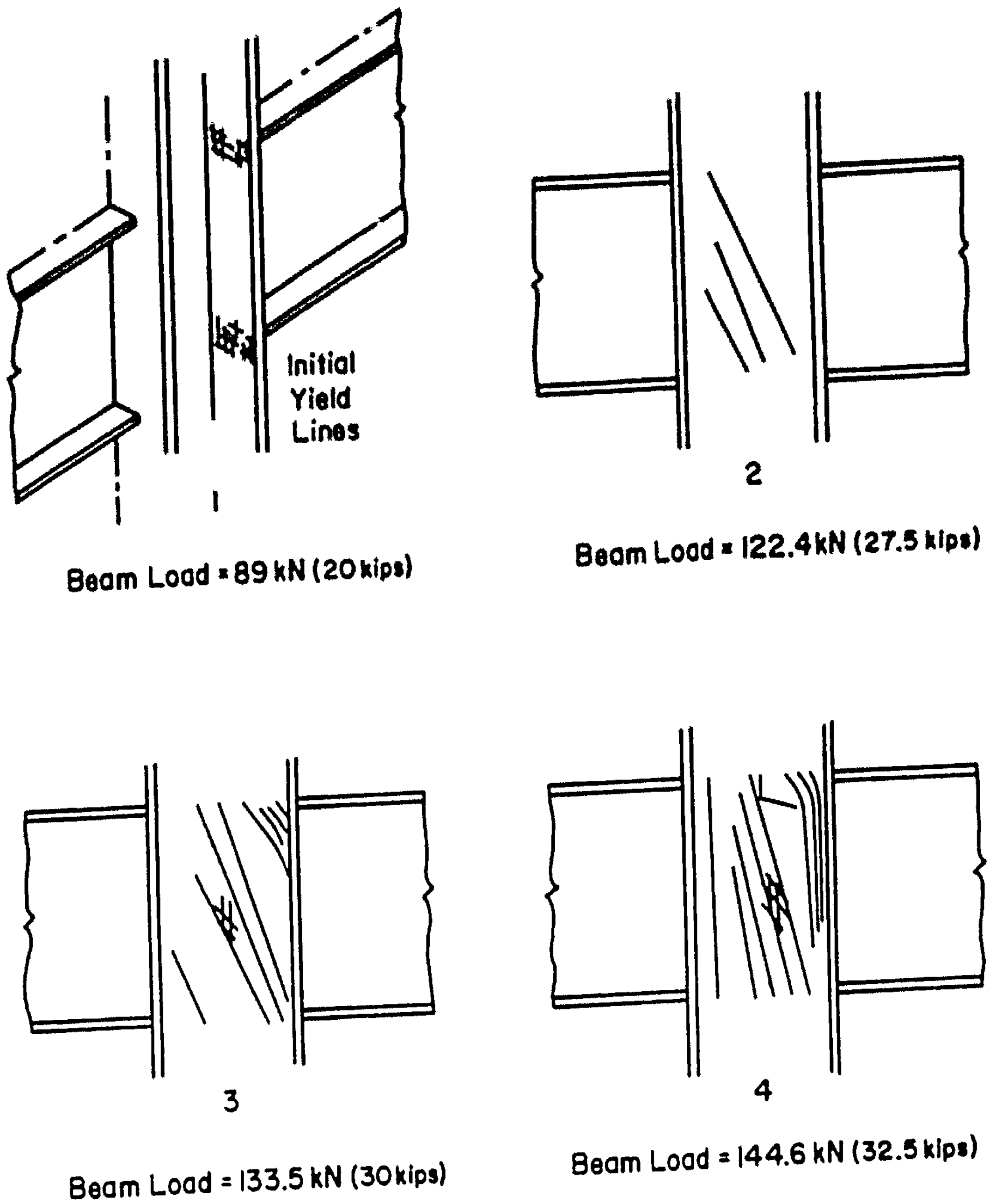


Figure 15: Yield Line Development in Specimen 1A

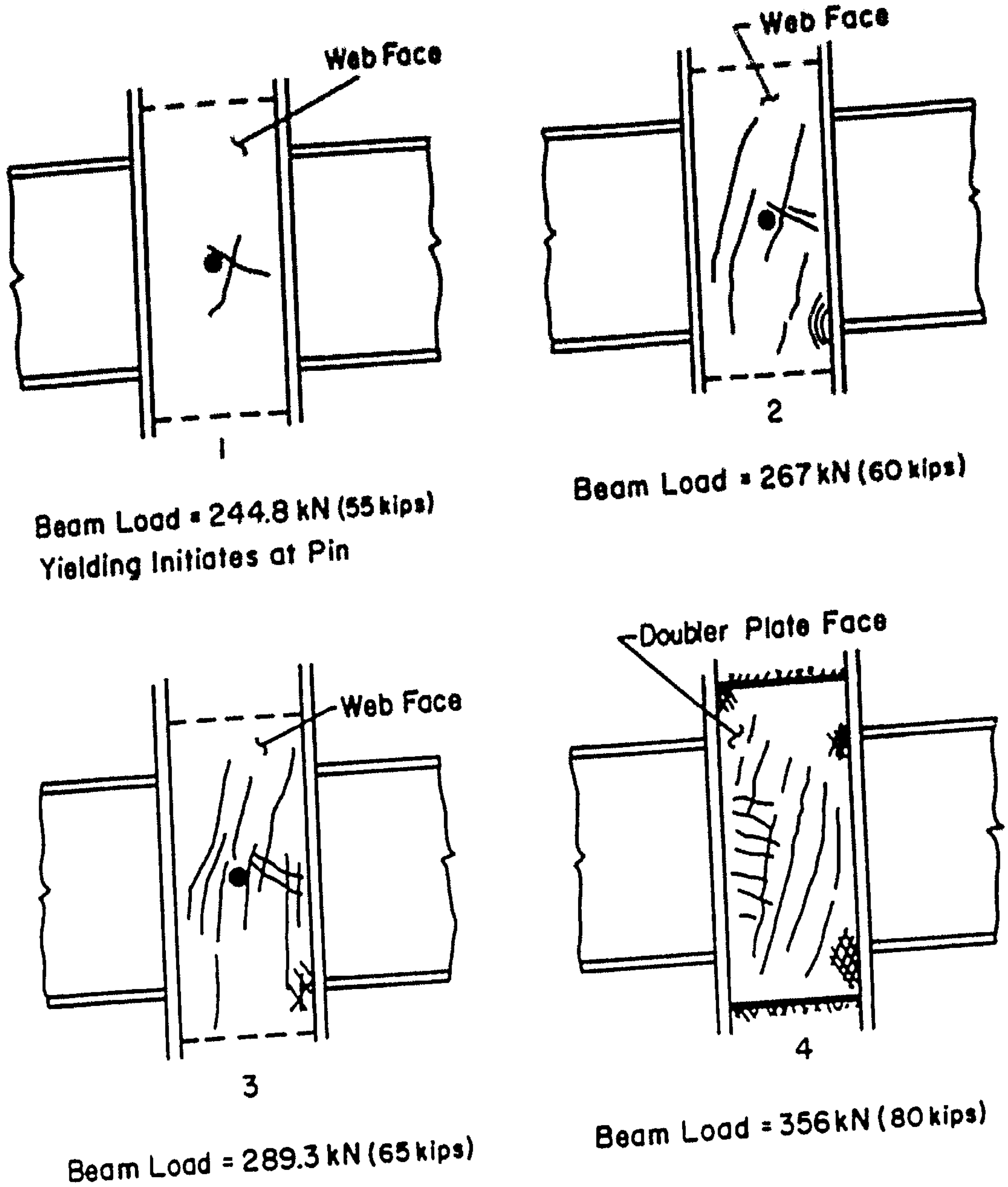


Figure 16: Yield Line Development in Specimen 3A

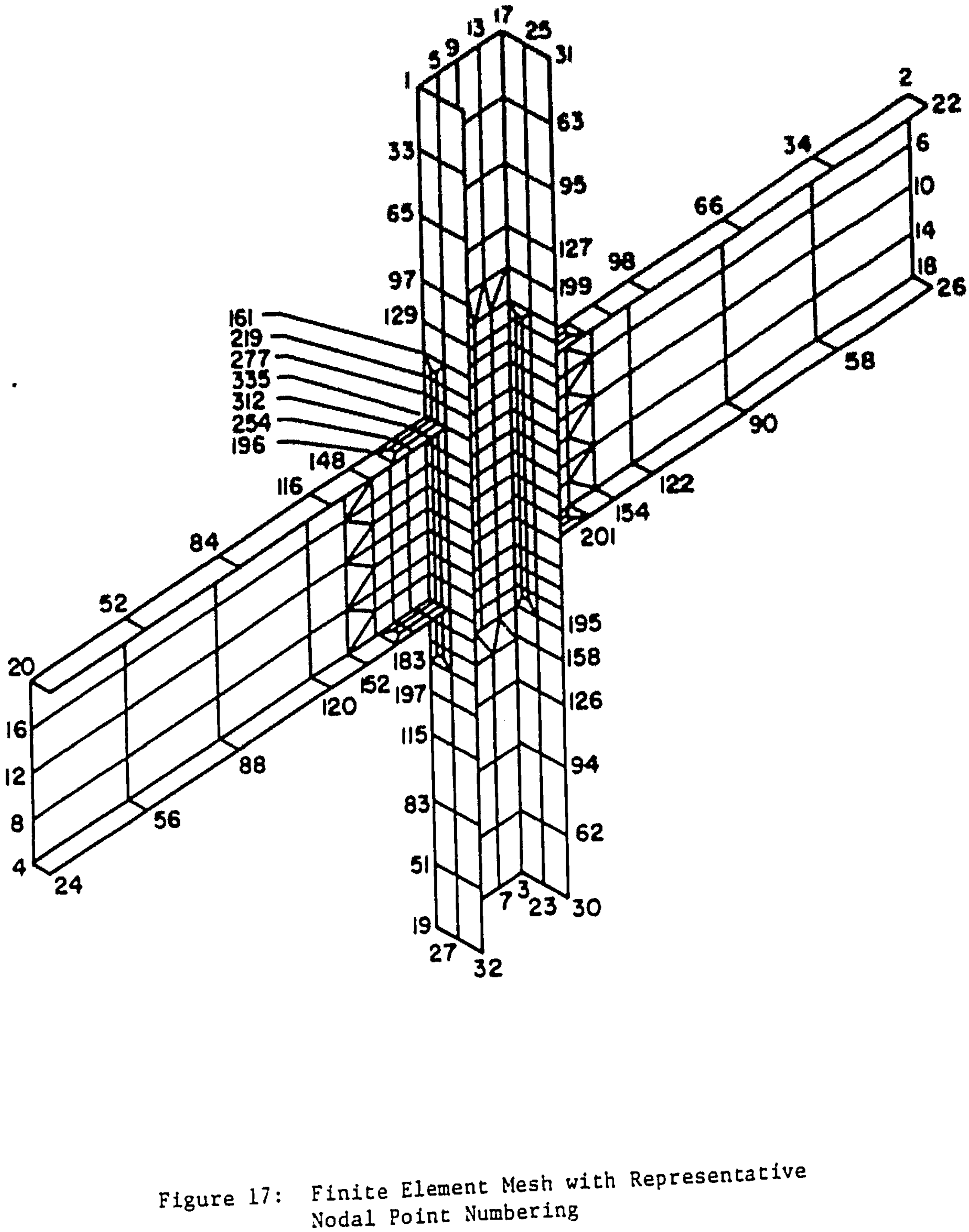


Figure 17: Finite Element Mesh with Representative Nodal Point Numbering

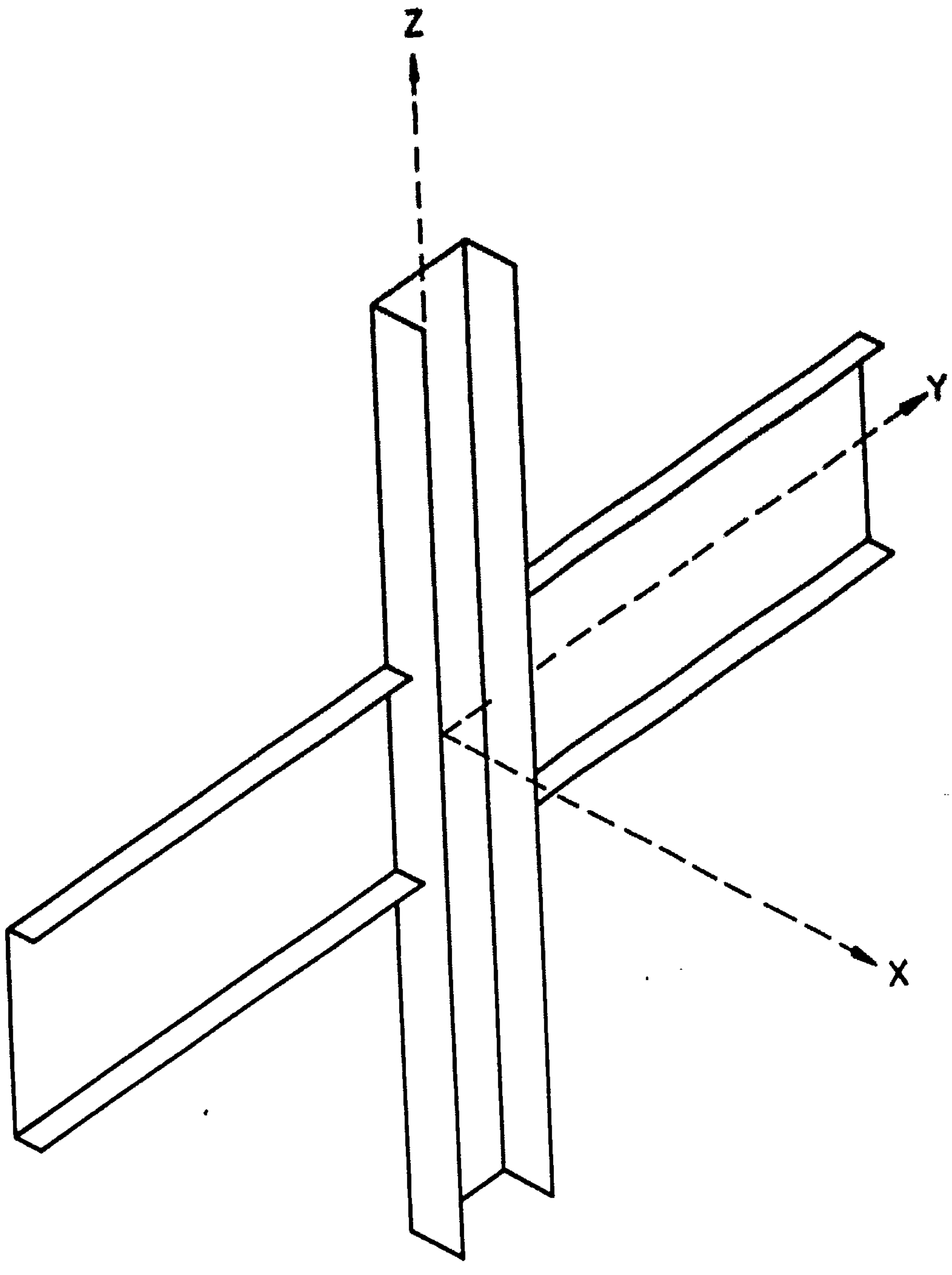


Figure 18: Idealized Connection with Global Coordinates

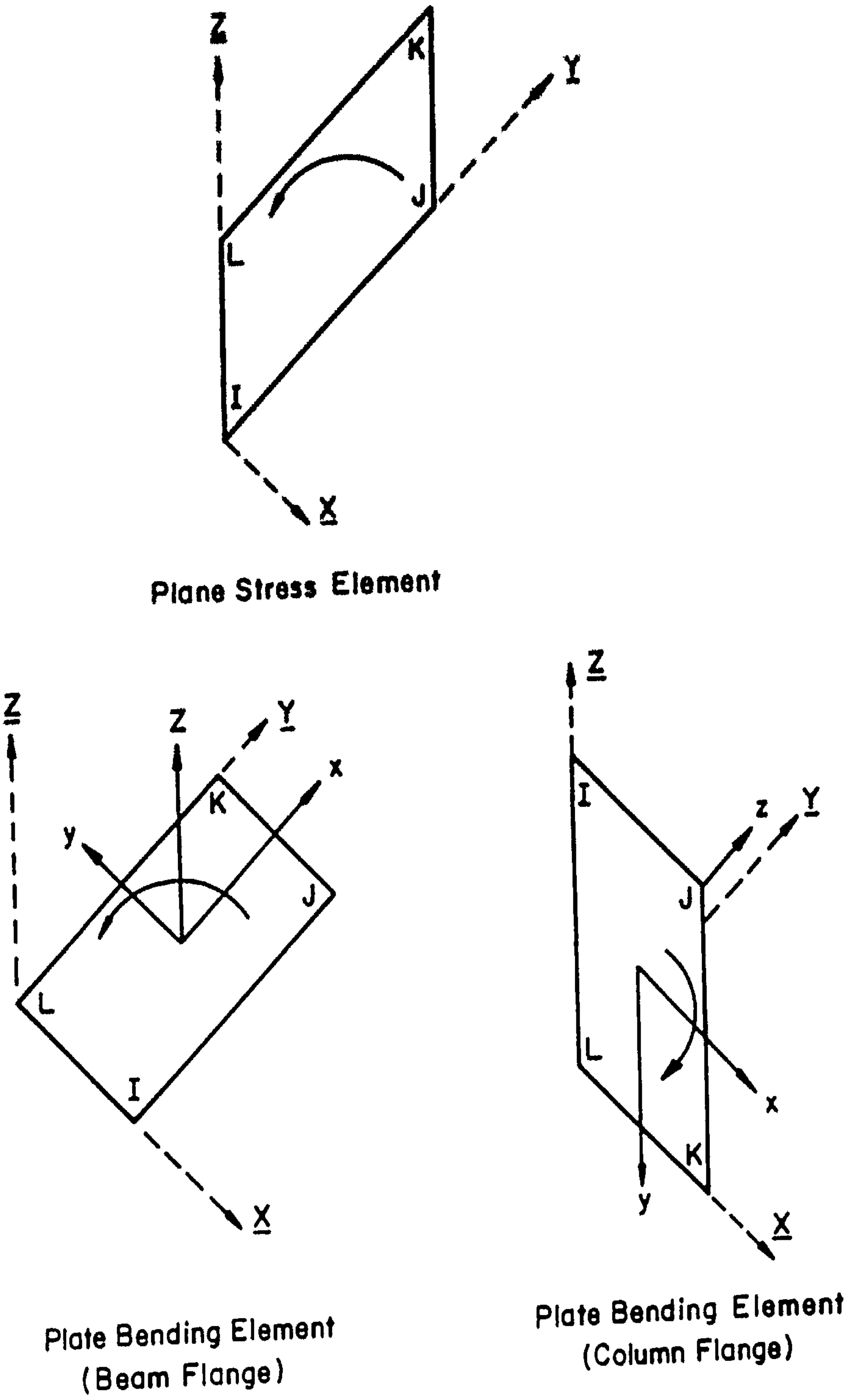


Figure 19: Element Coordinate Axes

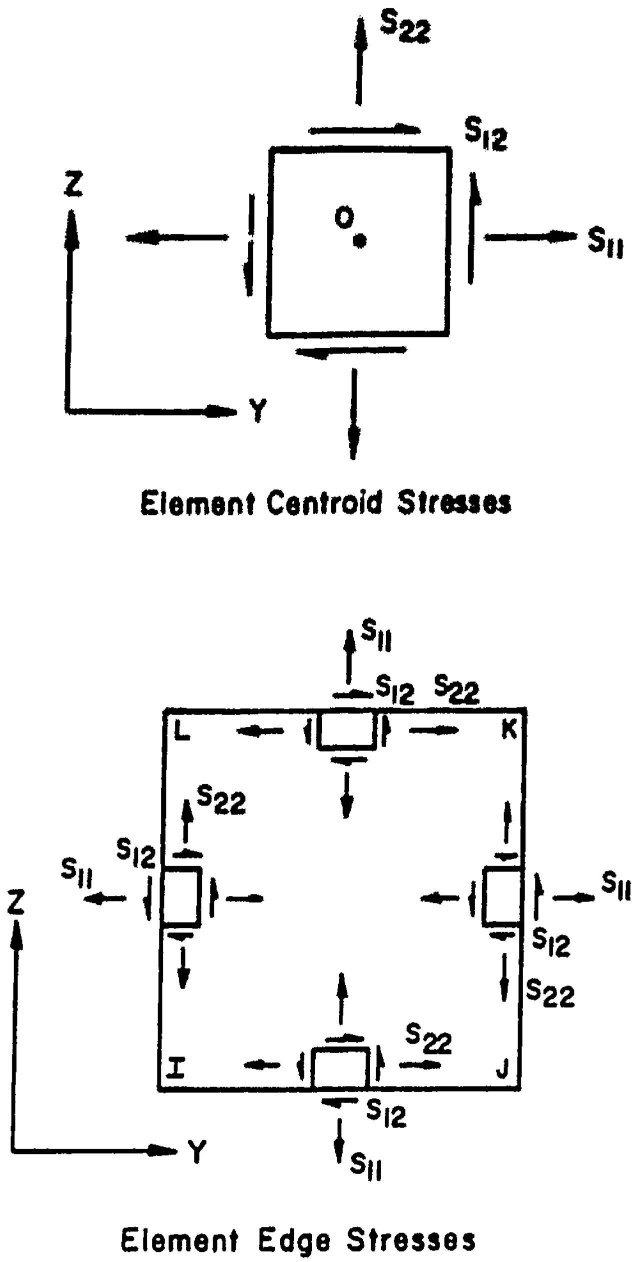


Figure 20: Sap4 Output Sign Convention

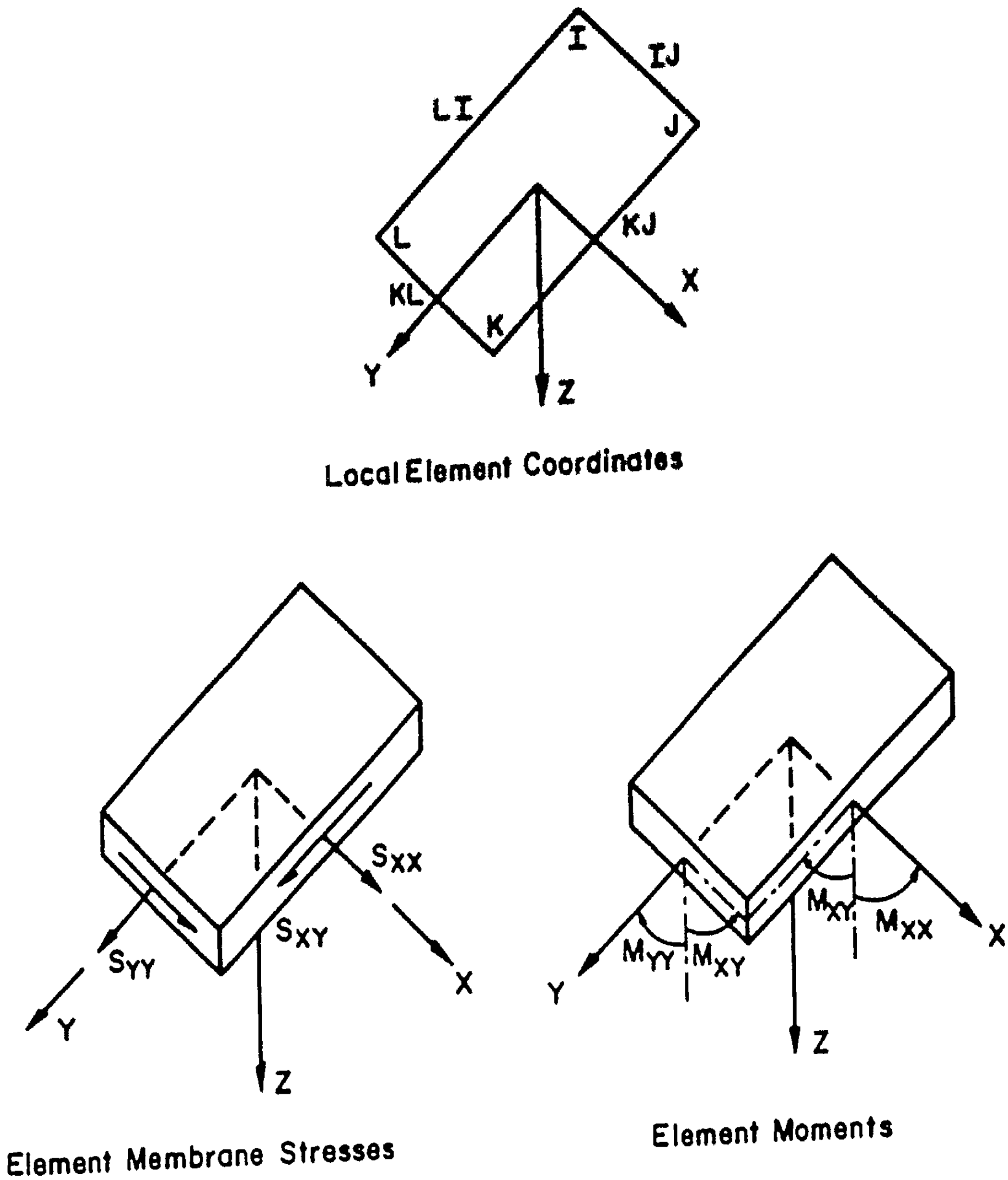
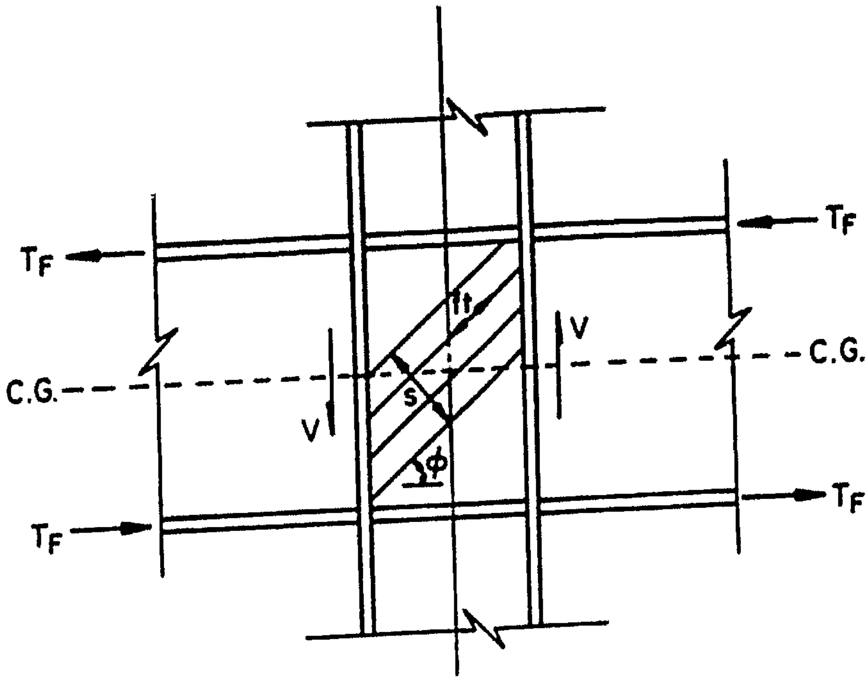


Figure 21: Sap4 Output Sign Convention



Where:

ft - Tensile Field Stress
s - Width of Strip
T_F - Flange Force
V - Shear Force
ϕ - Angle

Figure 22: Tension Field Action Diagram

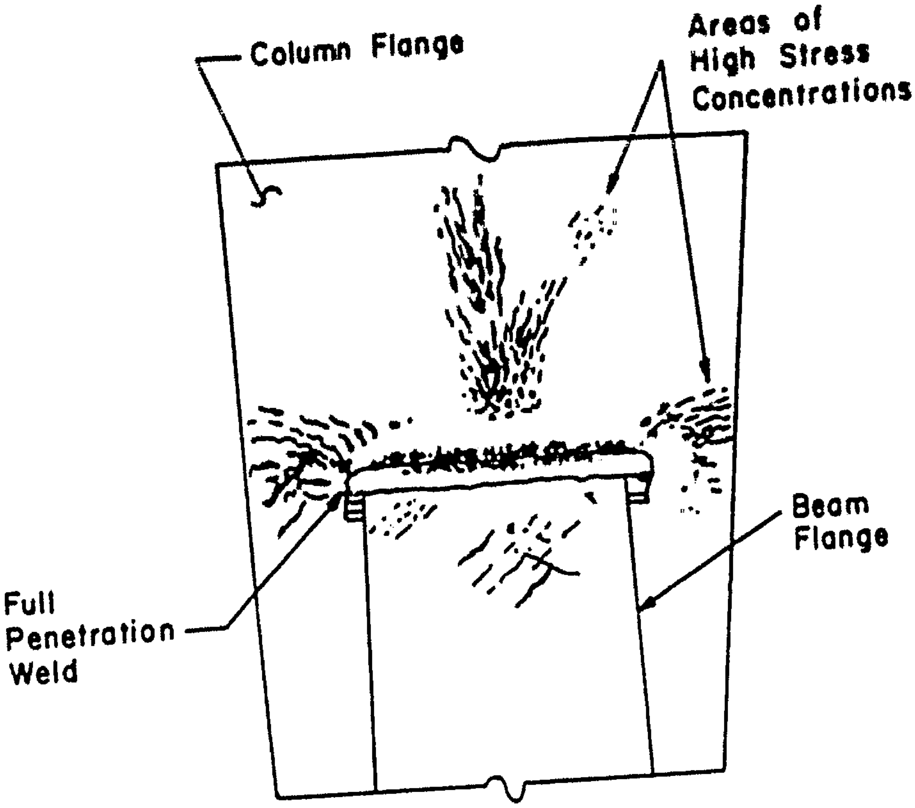
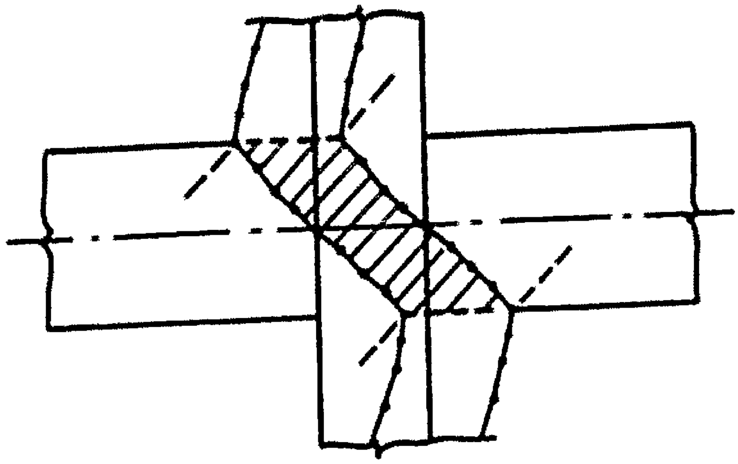
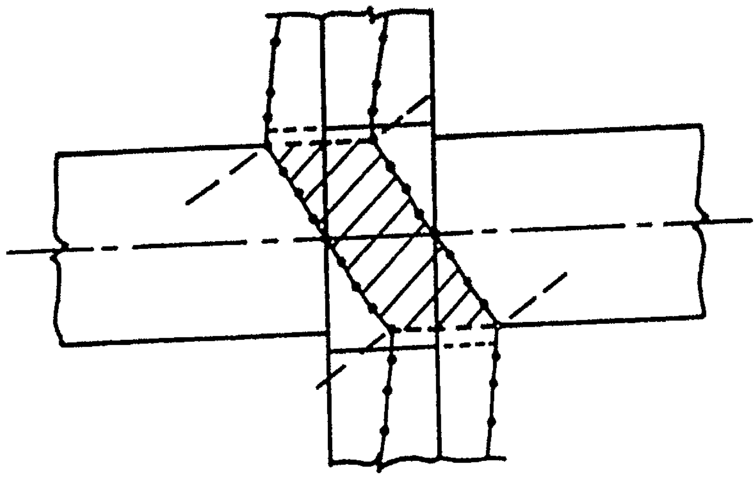


Figure 23: High Stress Concentrations

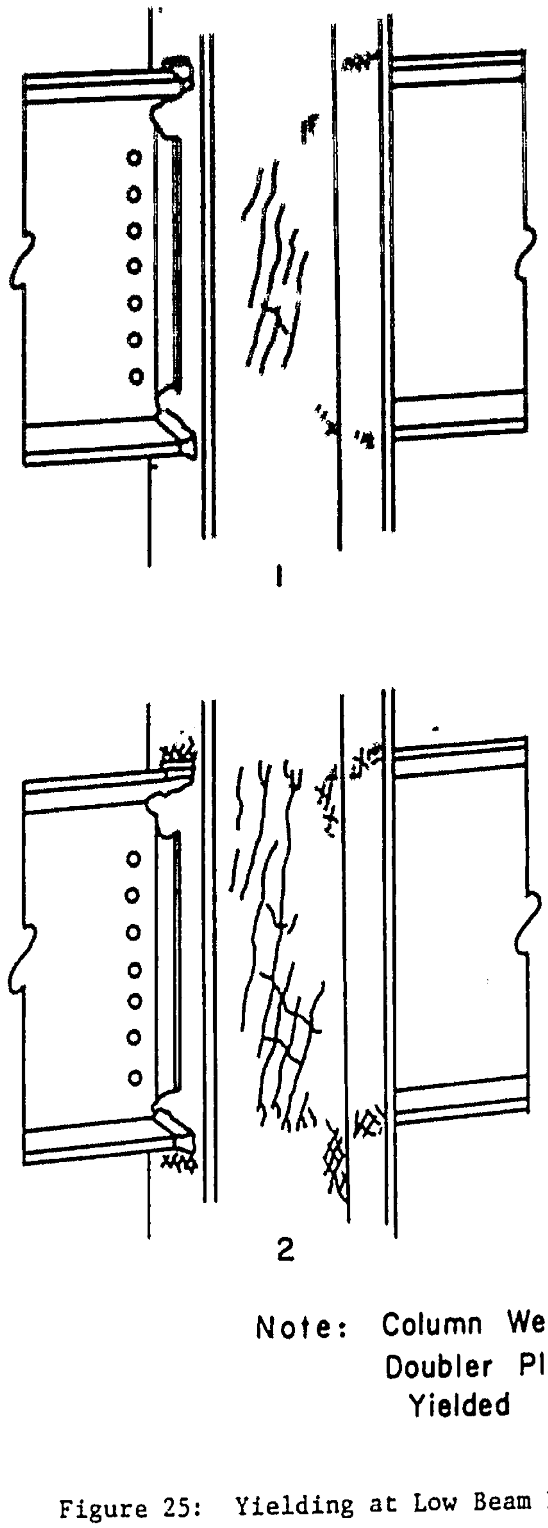


Panel Zone Deformation of Specimen 1A (elastic)



Panel Zone Deformation of Specimen 3A (elastic)

Figure 24: Elastic Panel Zone Deformation of Specimens 1A & 3A



Note: Column Web Shown
Doubler Plate Has Not
Yielded

Figure 25: Yielding at Low Beam Loads

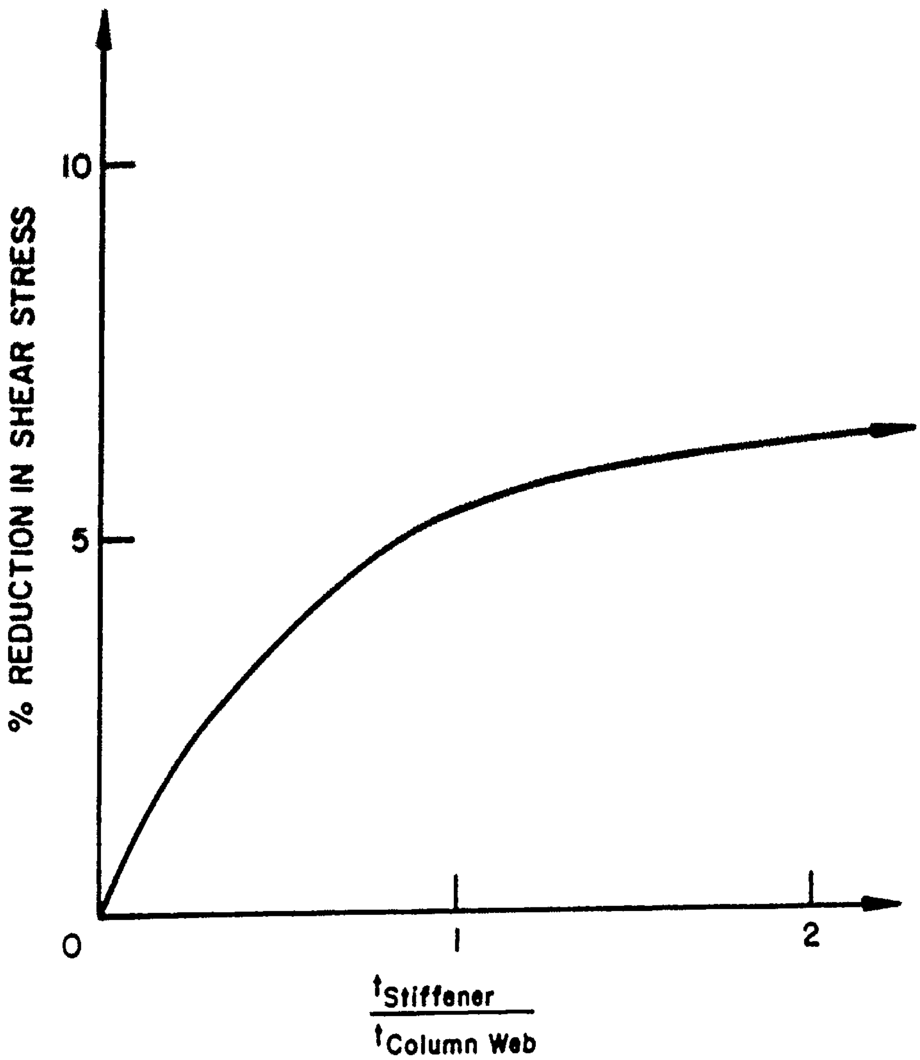
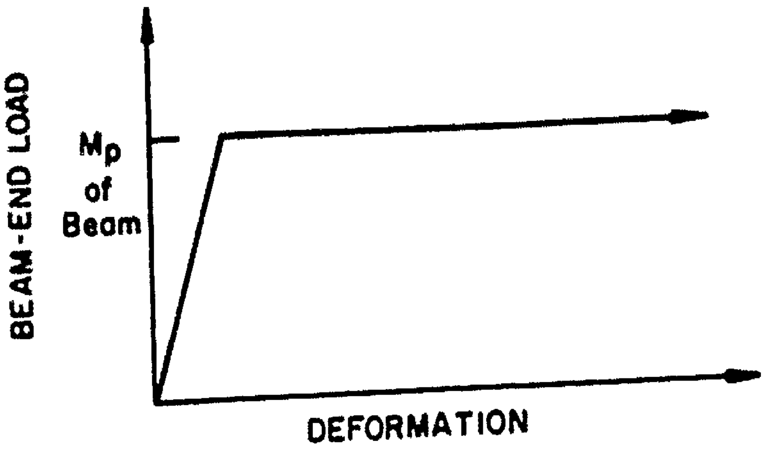
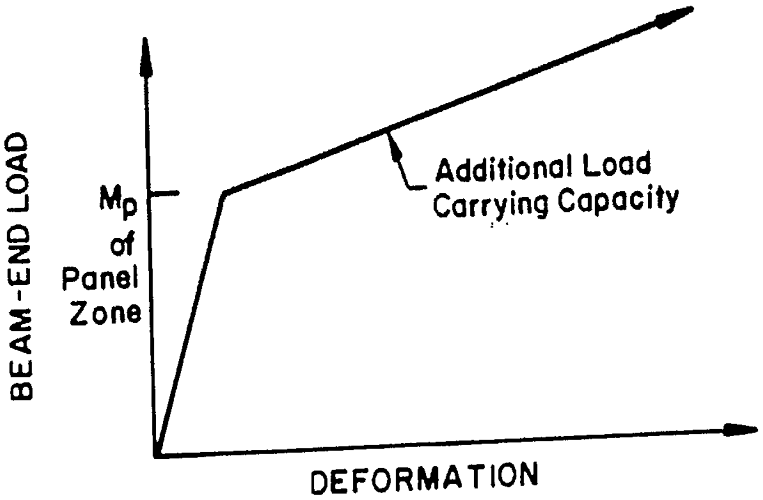


Figure 26: Panel Zone Shear Stress Reduction vs. Thickness Ratio



Load Response of Beam Alone



Load Response of Entire Connection

Figure 27: Plastic Hinge Development in Beam and Overall Connection

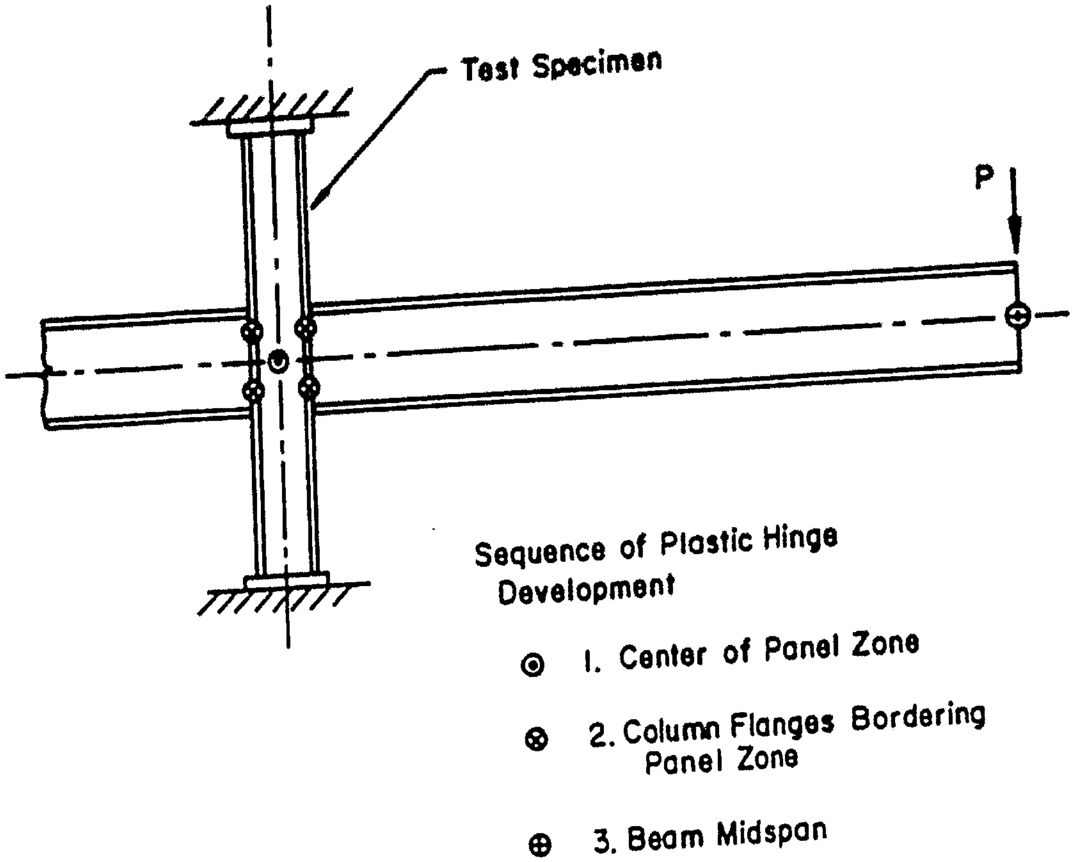


Figure 28: Sequence of Hinge Development

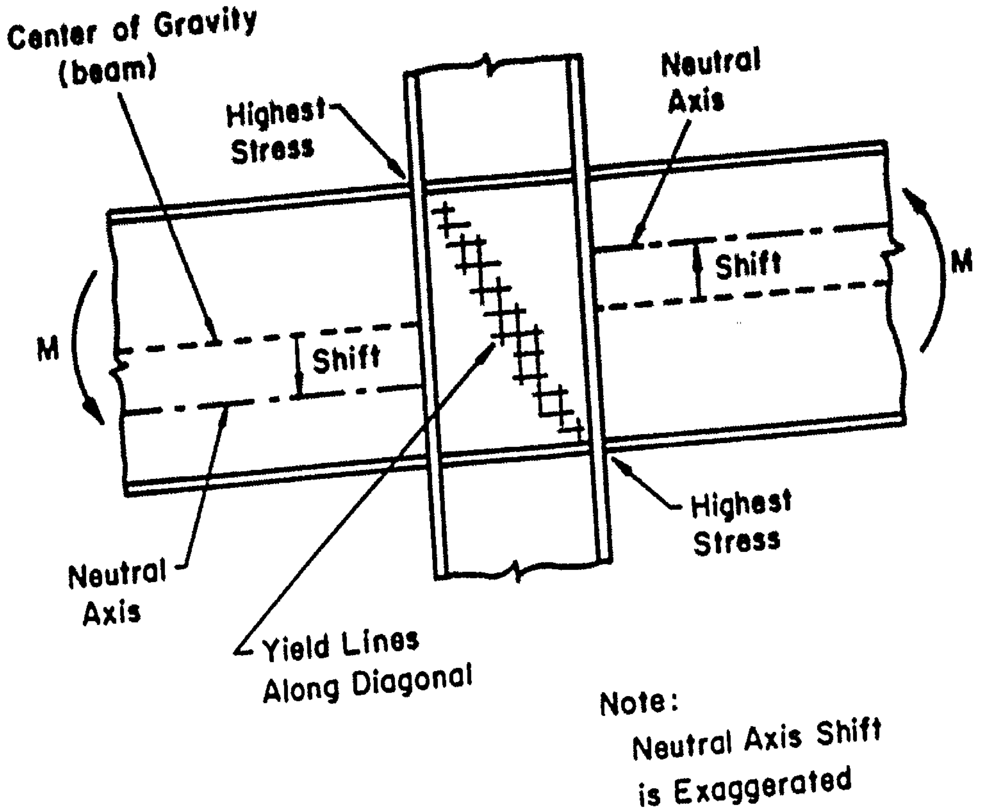


Figure 29: Shift of Neutral Axis Due to Residual Stresses

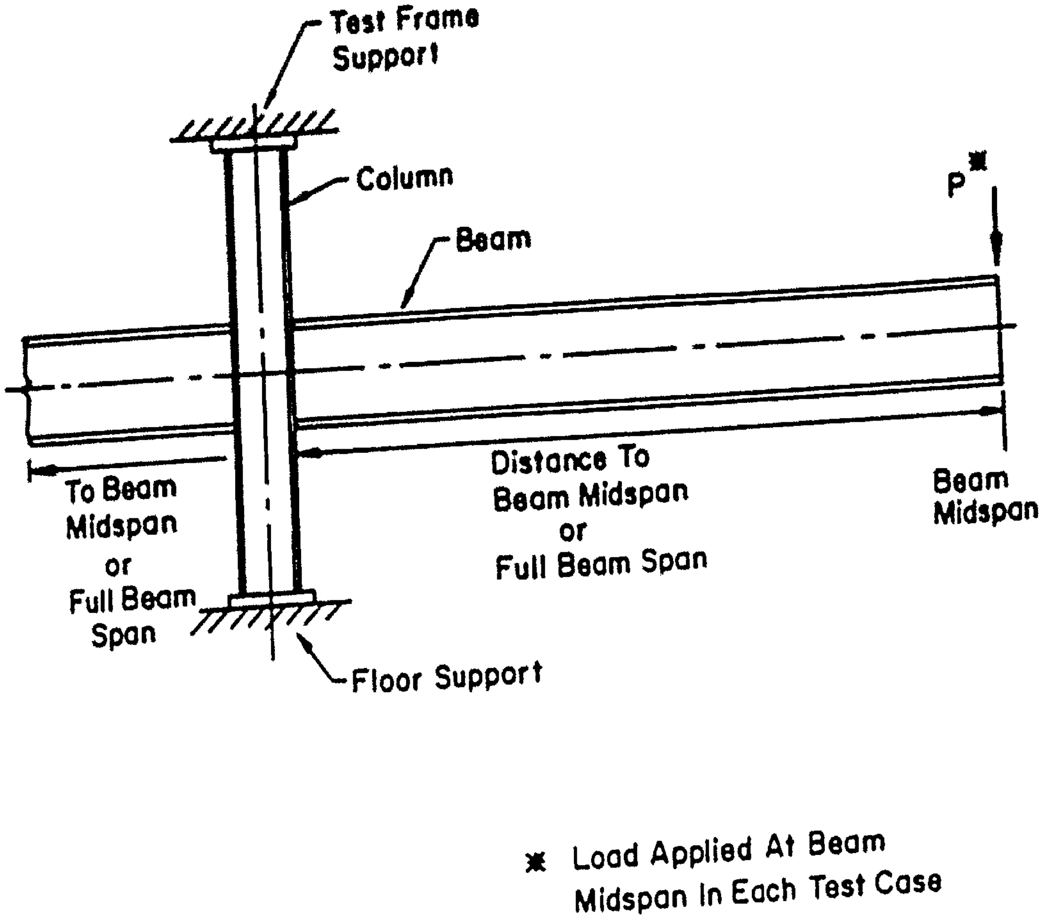


Figure 30: Possible Loading Procedure for Future Tests

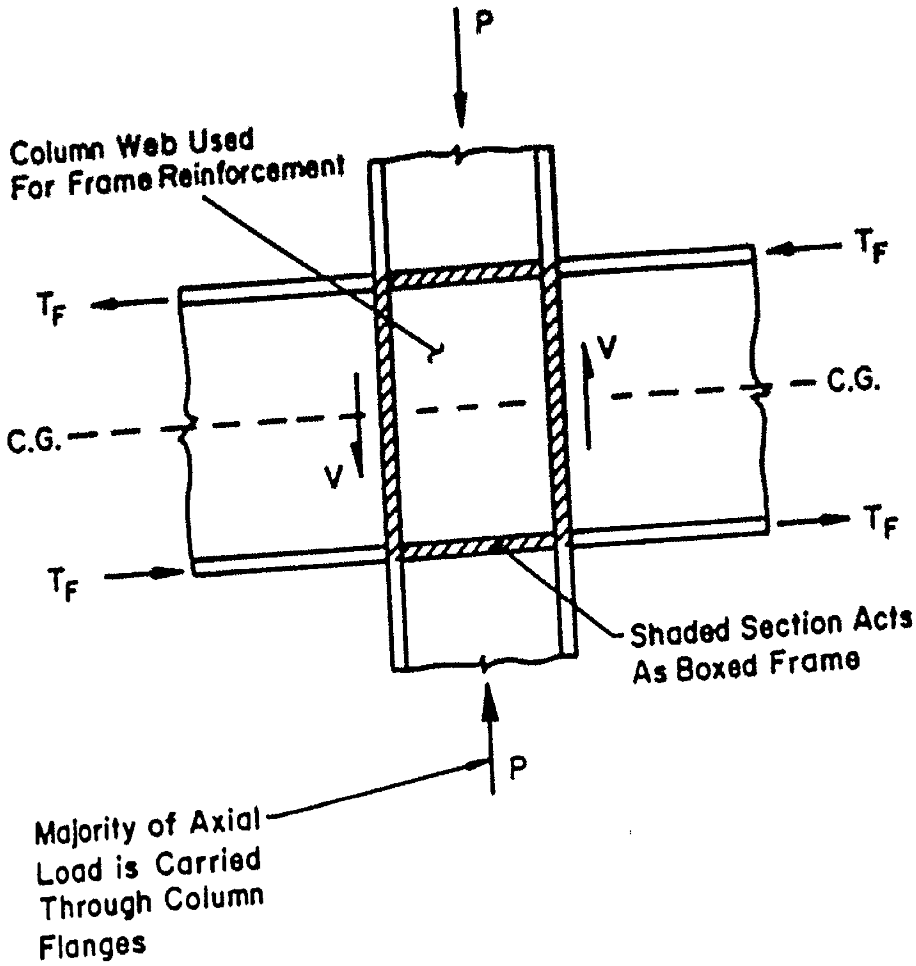


Figure 31: Frame Analogy Concept

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VITA

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He received his Bachelor of Science Degree in Civil Engineering from the University of Connecticut in June 1983. His major areas of study were structural design and analysis, soil mechanics and architectural design. Upon graduation, he furthered his studies at Lehigh University, pursuing his Master of Science Degree in Civil Engineering.

The author began work on his master's degree in August 1983 and finished his requirements in May 1985. During that period of time, the author worked as a research assistant at Lehigh University in the Operations Division of Fritz Engineering Laboratory under the instruction of Dr. Roger G. Slutter.