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ELASTIC-PLASTIC FINITE ELEMENT ANALYSIS OF

TENSION FLANGE CONNECTION PLATES IN

BEAM-TO-COLUMN WEB MOMENT CONNECTIONS

by

Randall M. Haist

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Dr. George C. Driscoll   .

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Dr. David A. VanHorn

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ELASTIC-PLASTIC FINITE ELEMENT ANALYSIS OF TENSION FLANGE CONNECTION PLATES IN BEAM-TO-COLUMN WEB MOMENT CONNECTIONS

Randall M. Haist

ABSTRACT

The connection plate thickness and the existence of a weld between the connection plate and the column web were theoretically investigated to find their effect on the behavior of the connection. With little resistance from the column web, most of the out-of-plane force of the beam tension flange is transferred to the column flanges resulting in high stress concentrations at the re-entrant corners of the connection plate and beam flange. Initial tensile stress concentrations between 2.5 and 3.0 are present in the four connection geometries studied.

These high stress concentrations result in early initial yielding, at loads only slightly higher than half the working load of 0.6 of the nominal yield stress. Due to this early yielding, large plastic strains develop at the re-entrant corners. At full plastification, plastic strains ranging from 1.838 to 2.228 percent are present. Large plastic strains, such as these, can exhaust the ductility of the material and result in fracture of the connection. For connections without a weld between the connection plate and column web, the plastic strain is

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more severe than for similar connections that have the weld. Therefore, the use of connections without a weld to the column web is not recommended.

It was found that a thicker connection plate forces yielding of the connection into the beam flange, and lowers the initial stress concentration. However, increasing the thickness, increases the stiffness of the connection. This stiffness becomes critical as the connection approaches full plastification. Therefore, a formula to account for the shear lag effect in the connection should be used to determine an appropriate thickness.

Based on the results discussed in this report and results of other investigations, recommendations regarding the design of beam-to-column web moment connections need to be made, so that fracture due to stress concentrations and restraints can be avoided.

1. INTRODUCTION

1.1 Background

An important element in the design and behavior of steel-framed multistory buildings is the connection of beams to columns. In beam-to-column connections the beam is either connected to the column flange or to the column web. While column flange connections have been thoroughly studied, column web connections have received limited research in the past. Some of the previous research, carried out at Lehigh University, focused on a study of unsymmetrical web connections where a beam was attached to only one side of the column web of an axially loaded column (Rentschler, et al, 1980). This is a more critical loading condition than occurs in a symmetrical web connection.

Part of the web connection study done at Lehigh University involved the static testing of four full-scale assemblages. The connections simulated four different geometries of actual building connections, with each test specimen consisting of a 5.5 m (18 ft) long column and an approximately 1.5 m (5 ft) long beam attached at the midheight of the column. The column and beam sizes were sections typical of multistory buildings. Each connection consisted of a W14x246 column and a W27x94

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beam, and was loaded unsymmetrically by an increasing monotonic load to simulate static conditions. The connections were designed so that a plastic moment would form in the beam at the tip of the column flanges.

During the testing of the four beam-to-column assemblages, two of the connections failed due to fracture of the tension flange connection plate. The one test specimen was a flange-welded, web-bolted connection. The beam flanges were groove welded to the flange connection plates which were attached to the column web and flanges with fillet welds. The web of the beam was bolted to a web plate which was connected to the column web and flange connection plates by fillet welds. The second failure occurred in a flange-bolted, web-bolted connection. The beam flanges and web were bolted to the moment connection plates and web connection plate. These plates were welded together and to the column flanges and web by means of fillet welds (For details of the connections see Rentschler, et al, 1980).

The first specimen failed due to fracture across the entire width of the tension flange connection plate in the region of the transverse groove weld. The failure occurred suddenly with no evidence of tearing prior to the last load increment. The second specimen also failed

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due to tearing of the tension flange connection plate. Although the fracture did not propagate across the entire width of the connection plate, as in the previous specimen, no further testing was attempted. Both fractures of the tension flange connection plates were unexpected.

Metallurgical studies were made to determine if the fractures were due to material or fabrication flaws (Driscoll, 1979). The fracture of the first connection began at the edge of the weld joining the beam tension flange to the connection plate. Fracture of the second connection began at a cosmetic welding pass between the tip of the column flange and the side of the connection plate. Both fractures were predominantly brittle in nature with no evidence of defects on the fracture surfaces which could have contributed to the failures. It was concluded that the failures occurred due to large strain concentrations in the connections.

The overall decision was that both connections were made of steel and welds of normal soundness and quality and that the fractures occurred because of large stress concentrations due to the geometry of the design details. Although the fourth connection of the test series performed as desired and failed due to large

deformations, the unexpected fractures of two of the connections and the findings of the metallurgical study, indicated a need of further investigation.

1.2 Purpose

The purpose of this research is to conduct a theoretical and experimental investigation of the behavior and design of beam tension flange connections in beam-to-column web moment connections. The research is needed in order to determine the best geometrically detailed connection plates that will provide both safe and economical connections. The overall objective of the research is establishing recommendations for the design of connections in practice that will avoid premature fracture due to stress concentrations and restraints.

1.3 Scope

The scope of the study will include the theoretical and experimental investigation of several different connection details. A total of ten specimens will be tested in the initial phase of the program. Figure 1 shows the ten specimens to be tested. Each specimen will use a W14x257 section for its column and a 25.4 x 2.54 cm (10 x 1 inch) plate for the beam flange. The connection plates will be the only varying factor in this phase of the research. Specimens (1), (2), and (3) will all be

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tested using a 2.54 cm (1 inch) connection plate. Each

of these three specimens will be tested with and without

a stiffener. The varying factor will be the shape of the

connection plate. Specimen (1) will have its connection

plate terminated at about the tip of the column flanges,

while specimens (2) and (3) will extend the connection

plate 7.5 cm (3 inches) beyond the column flange tips.

In addition to the extension, connection plate (3) will

also be tapered. Specimens (4) and (5) will be tested

using a 4.13 cm (1-5/8 inch) connection plate to account

for the effect of shear lag. Specimens (4) and (5) are

similar to specimens (1) and (3) respectively. These two

specimens will be tested with and without a weld

attaching the connection plate to the column web.

The proposed test setup is shown in Fig. 2. Each

specimen will consist of a 25.4 x 2.54 cm (10 x 1 inch)

plate 1.5 m (5 ft) in length attached to a connection

plate which will be welded to a column stub 1.2 m (4 ft)

in length. The specimen will then be bolted to a test

fixture of greater strength by joining plates. This

setup will allow the specimen to be pulled in direct

tension thereby simulating the forces due to a moment

reaction in the beam tension flange of a beam-to-column

moment connection. By using this setup, the behavior of

the beam tension flange and connection plate will be isolated and studied separately without the complexities of full-scale connection testing, while still simulating the important stress and restraint characteristics of full-scale connections. The behavior of the beam tension flange and connection plate is believed to be the most important influence on fracture (Driscoll, Shen, et al, 1981). Although this is a simplified setup, it will be an efficient method for the study of important parameters of connection details. The connection details which perform best under this simplified setup will then be fabricated as full-scale connections and tested to verify their behavior.

The emphasis of this report will be a discussion of some of the theoretical results. First, a description of the theoretical model will be made along with a discussion of the theory behind the procedure followed. Next, a comparison between the predicted results of specimens (A), (D), and (E) will be made in order to determine the influence of plate thickness and column web welding. In addition to this, a theoretical study of a 3.49 cm (1-3/8 inch) plate, similar to specimens (A) and (D), was conducted, and the results of this study will be compared to the results of specimens (A) and (D).

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Finally, conclusions based on the above studies will be discussed and recommendations concerning the design of beam tension flange moment connection plates given.

2. THEORETICAL MODEL

2.1 Finite Element Model

The theoretical study conducted on the specimens involved an elastic-plastic finite element analysis. The finite element model is shown in Fig. 3. Due to the symmetry of the actual test specimen, only a quarter of the connection needed to be modeled. The actual size of the test specimen was used for the finite element model with the exception of the beam flange. In the finite element program, the beam flange extends beyond the column flange tips only 22.9 cm (9 inches). Since the critical area of interest is where the beam flange attaches to the connection plate, and since the length of the beam flanges in the actual specimen is for gripping purposes during testing, it was not necessary to model the entire length.

Since the bending of the column is an important factor influencing the behavior of a connection, plate bending elements were used to model the column web and flanges. These elements accounted for both bending and displacement that occurred within the column due to the loading. The column was allowed to move in all three directions except at the top and bottom where it was fixed in the x and y-directions (see Fig. 3). By

prohibiting movement at the column ends in the x and y-directions, the model simulated the restraint in the actual test specimen caused by the end plates and test fixture. However, by permitting movement in the z-direction, vertical movement of the column ends due to bending of the column was allowed. Loading of the model was done at the end of the beam flange. Loads were applied at the nodes in appropriate proportion such that a uniform stress of 345 MPa (50 ksi), which is the yield strength of the actual beam flange, was simulated as closely as possible. Due to the loading geometry, both the connection plate and beam flange are only subjected to loads in their plane. For this reason, plane stress membrane elements which only account for in-plane displacements were used.

The same model was used for all the different connection geometries. By changing the elastic modulus of certain elements to a very small stiffness, the connection could be made to act as if these elements did not exist. Therefore, it was possible to use the same model for a connection that had either a tapered connection plate or one that was flush with the column flange tips, and whether or not a weld existed between the column web and the connection plate.

A572 steel with a yield strength of 345 MPa (50 ksi) was used. The stress-strain relationship that was assumed for each element is shown in Fig. 4. An elastic modulus of 200 GPa (29000 ksi) was used for stresses ranging from zero to yield. Upon yielding, the modulus was then assumed to be 4.14 GPa (600 ksi). This value was used to account for additional strength due to strain hardening of the elements.

An interesting modeling approach used in the finite element model was the use of rigid beam elements to account for the thickness of the column web and flanges (Shen and Driscoll, 1981). For a W14x257, the web thickness and flange thickness are 2.985 cm (1.175 inches) and 4.801 cm (1.890 inches), respectively. For stocky members, such as this, ignoring the thickness can lead to substantial error. However, by using beam elements between the nodes of the column and the nodes of the connection plate the thickness of the column can be represented. The stiffness or modulus of elasticity of these beam elements must be very large relative to the stiffness of the other elements of the connection. By using a very large modulus of elasticity, the beam element will act as a rigid link between the column and the connection plate, thereby simulating the actual

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column thickness. This approach is based upon Kirchhoff's approximation that straight lines normal to the undeformed middle plane of the plate remain approximately straight and inextensional under the deformation of the plate. Also, it is assumed that the lines normal to the undeformed plate remain normal to the plate after deformation (Boresi, et al, 1952).

Part of the previous research carried out at Lehigh University on beam-to-column web connections involved a theoretical study of full-scale connections using an elastic finite element analysis (Rentschler, 1979). The study was made in order to determine elastic stress distributions and deformations in connections similar to the four connections that had been experimentally tested. In that model, however, the effect of the column thickness was not taken into account, other than in determining the stiffness of the column. Because of this, the results of the present study should not be compared to the previous finite element work done on beam-to-column web connections.

2.2 Nonlinear Procedure

The theory behind the elastic-plastic procedure used in the theoretical model is based on Von Mises' yield criterion. For three-dimensional stresses, the Von Mises

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formula is:

$$\sigma_{ef} = \sqrt{\sigma_1^2 + \sigma_2^2 + \sigma_3^2 - \sigma_1\sigma_2 - \sigma_2\sigma_3 - \sigma_1\sigma_3} = \sigma_Y \quad (1)$$

where σ_{ef} is the effective stress, σ_1, σ_2, σ_3 are the principal stresses, and σ_Y is the yield stress. This formula uses the theory that yielding and plastic deformations are independent of the mean normal stress $\bar{\sigma}$, where $\bar{\sigma} = (\sigma_1 + \sigma_2 + \sigma_3)/3$. Therefore, the actual stress that causes yielding is σ'_i, where $\sigma'_i = \sigma_i - \bar{\sigma}$, and i equals 1, 2, or 3.

It was assumed that yielding occurred only in the connection plate and beam flange elements, and not in the column. This is a reasonable assumption based upon an elastic finite element analysis (Shen and Driscoll, 1981). Since the elements in the connection plate and beam flange are only two-dimensional elements, the stress normal to the plane of the element is undefined. Therefore, in equation (1), σ_3 is set equal to zero and the Von Mises formula can be reduced to two-dimensional stresses:

$$\sigma_{ef} = \sqrt{\sigma_1^2 + \sigma_2^2 - \sigma_1\sigma_2} = \sigma_Y \quad (2)$$

or in general form:

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$$\sigma_{ef} = \sqrt{\sigma_x^2 + \sigma_y^2 - \sigma_x\sigma_y + 3\tau_{xy}^2} = \sigma_Y \qquad (3)$$

It was desired to keep all calculations in a non-dimensional form. Therefore, dividing equation (3) by σ_Y results in:

$$S_{ef} = \sqrt{S_x^2 + S_y^2 - S_xS_y + 3S_{xy}^2} = 1.0 \qquad (4)$$

where

$$S_{ef} = \frac{\sigma_{ef}}{\sigma_Y} , \quad S_x = \frac{\sigma_x}{\sigma_Y} , \quad S_y = \frac{\sigma_y}{\sigma_Y} , \quad \text{and } S_{xy} = \frac{\tau_{xy}}{\sigma_Y}$$

In addition to this, all of the moduli and the applied uniform stress of 345 MPa (50 ksi) were divided by σ_Y.

A step-by-step procedure (Shen, et al, 1981) was used to determine the yielding sequence of the elements in the model.

1. Each element was given initial stresses and plastic strain equal to zero, and saved in a file.

2. The finite element program (Bathe, et al, 1974) was run using a non-dimensional, uniform stress of 1.0, and the resulting stresses put on file.

3. The yielding criterion for each element was checked and the load factor, F, which causes the element to just begin to yield was calculated based on:

$$((S_x^{i-1} + FT_x^i)^2 + (S_y^{i-1} + FT_y^i)^2 -$$

$$(S_x^{i-1} + FT_x^i)(S_y^{i-1} + FT_y^i) +$$

$$3(S_{xy}^{i-1} + FT_{xy}^i)^2)^{\frac{1}{2}} = 1.0 \quad (5)$$

where S^{i-1} are the stresses stored from the previous cycle, and T^i are the stresses from the current cycle of the finite element analysis.

4. The smallest load factor of all of the elements, F_{min}, was determined. This value is the load factor required to cause yielding to just occur in the highest stressed element.

5. The current stresses were calculated, and saved in a file, for each element using:

$$S^i = S^{i-1} + F_{min}^i T^i \quad (6)$$

6. The current plastic strains were calculated

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for each element that had previously yielded
using:

$$\epsilon_p^i = \epsilon_p^{i-1} + 0.0805F_{min}^i((T_1^i)^2 +$$
$$(T_2^i)^2 - (T_1^i)(T_2^i))^{\frac{1}{2}} \quad (7)$$

Equation (7) was developed by Shen (Shen, et al, 1981).

7. All newly yielded elements were determined, and the modulus of elasticity of these elements changed from 200 GPa (29000 ksi) to 4.14 GPa (600 ksi). It was assumed that if S_{ef} of the element was greater than or equal to 0.99, the element would be considered yielded.

8. Steps (2) through (7) were repeated until an accumulated load factor of 1.0 was reached.

By following this procedure, it was possible to include the effect of yielding of the specimen due to high stress concentrations. This would not be possible in a purely elastic finite element analysis. It also allowed for the observance of the stresses in each element as redistribution of stress due to yielding occurred and to observe the yielding pattern as it developed.

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3. RESULTS OF THEORETICAL ANALYSIS

3.1 Shear Lag Effect

In beam-to-column web connections, the web of the column offers little resistance, relative to the thicker column flanges, to the out-of-plane force of the beam flange. Since forces in connections are attracted to areas of greatest stiffness, most of the beam flange force is therefore transmitted to the column flanges. A non-uniform stress state develops as the stress is transferred to the column flanges, creating a shear lag effect, and resulting in high stress concentrations at the edges of the beam flange and connection plate (Driscoll, Lu, et al, 1981). If there is no weld between the column web and the connection plate, the column flanges must carry the entire force from the beam flange. Although this creates a more severe shear lag effect, such a connection is of interest to fabricators because of its reduced cost of fabrication.

By increasing the thickness of the connection plate, yielding of the connection plate can be avoided, and the shear lag effect accounted for. Mr. William A. Milek, of the American Institute of Steel Construction, has suggested an approximate formula to account for the effect of shear lag. The area of the connection plate

that should be used is equal to the area of the beam
flange divided by a factor C_t, where:

$$C_t = (1 - \frac{\bar{x}}{L}) \qquad (8)$$

and:

$$\frac{\bar{x}}{L} \approx \frac{d}{2b} \qquad (9)$$

for connections with the connection plate welded only to
the column flanges, and:

$$\frac{\bar{x}}{L} \approx \frac{d}{4b} \qquad (10)$$

for connections with the connection plate welded to both
the column flanges and web, and with a stiffener welded
to the column web. In equations (9) and (10), "d" is the
depth of the column and "b" is the width of the column
flange. Using equations (8) and (9), a plate thickness
of 4.13 cm (1-5/8 inch) was calculated for use with a
beam flange of 25.4 x 2.54 cm (10 x 1 inch) for a
connection plate with no weld to the column web, as in
specimen (E). This thickness was also used for specimen
(D) since it was welded to the column web but did not
contain a backup stiffener.

3.2 Introductory Discussion

The results of specimens (A),(D), and (E) are compared in order to determine the influence of plate thickness and column web welding. Specimen (A) is considered the control specimen. It is similar to the full-scale connections studied previously at Lehigh University to the extent that the connection plate is the same thickness as the beam flange, the connection plate is attached to both the column flanges and column web, and no stiffener is welded to the column web opposite the beam flange connection plate.

In the comparison of specimens (A), (D), and (E), specimen (D) will first be compared to specimen (A). This will allow the effect of connection plate thickness to be observed with no other varying factors. Next, specimens (D) and (E) will be compared to see the effect of welding to the column web. Again, this will only consider one variable. Specimens (A) and (E) will not be compared because this would take into account two varying factors.

3.3 Load-Displacement Curves

The load-displacement curves for specimens (A), (D), and (E) are shown in Fig. 5. Specimen (D) has a greater stiffness than that of specimen (A). This is due to the

thicker connection plate of specimen (D). At a load factor of 0.6 of the nominal yield stress, specimen (D) deflects 20 percent less than specimen (A), and at a load factor of 1.0, specimen (D) deflects approximately 8 percent less than specimen (A). Also, specimen (D) begins yielding at a higher load factor, 0.386 compared to 0.333 (16 percent), than that of specimen (A). Yielding first occurs at the re-entrant corner in both specimens. Therefore, in order for specimen (A) to yield first, it must have a larger stress concentration than specimen (D) at that point in the connection. With a thinner connection plate, less restraint is provided in specimen (A), and therefore the stress distribution has greater non-uniformity than in the thicker connection plate. Thus, earlier yielding occurs.

Figure 5 shows by the larger deflections of specimen (E), that the stiffness of specimen (E) is less than that of specimen (D). At a load factor of 0.6 of the nominal yield stress, specimen (E) deflects approximately 14 percent more than specimen (D), and at a load factor of 1.0, approximately 19 percent more. Specimen (E) also begins yielding at a lower load factor than that of specimen (D). This load is 0.339 compared to 0.386 or approximately 12 percent lower. The difference in first

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yield is due to the connection plate of specimen (E) not being welded to the column web. Because the web offers no resistance, the entire stress must be transferred to the flanges, resulting in a larger stress concentration at the re-entrant corner.

Because of their geometric properties, specimens (A), (D), and (E), have large stress concentrations at the re-entrant corners of the connection plates and beam flanges. The initial tensile stress concentration at this point in all three specimens is between 2.5 and 3.0. These stress concentrations are large enough such that yielding begins at a load factor only slightly higher than half the working load of 0.6 of the nominal yield stress.

3.4 Stress Distribution

Figure 6 is a plot of the longitudinal stress, Sx, along the connection plate and beam flange interface for specimens (A), (D), and (E) at applied load factors of 0.6 and 1.0 of the nominal yield stress. The curves for all three specimens are very similar in shape. It is seen that even at the working load of 0.6 of the nominal yield stress, very high stresses are already present at the edge of the connection plate and beam flange. The stress towards the middle of the beam flange, although

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lower than 0.6, is relatively uniform compared to the overall shape of the curve. A very sharp increase in stress occurs along the interface where the elements are approaching the yield criterion. The curve then levels off again where yielding has already taken place and plastic hardening is occurring. The stress concentration at the re-entrant corner is easily seen by the large non-uniformity in the state of stress at this cross-section. Even specimen (D), which has the most uniform state of stress at this interface because of the larger restraint characteristics of its connection plate, shows large non-uniformity of stress with an increase of Sx across the flange width of 120 percent. Although the stresses are more uniform at a load factor of 1.0 because of the redistribution of stresses, there is still a very noticeable stress concentration at the edge of the beam flange.

The plot of the transverse stress, Sy, along the interface of the connection plate and beam flange is shown in Fig. 7. As is the case for Sx along this line, the curves of Sy for all three specimens are similar, with the exception of specimen (E) at the load factor of 1.0. This exception is due to the large transfer of stress from the middle of the beam flange to the edge, so

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that the column flanges can carry the entire load. As with Sx, Sy at 0.6 of the nominal yield stress is uniform in the middle of the beam flange where yielding has not yet taken place. The stress increases near the area of yielding, due to the Poisson effect of yielding, before beginning to drop to zero at the flange edge.

Figure 8 shows Sy in the connection plate along the column flange for specimens (A), (D), and (E). The most significant aspect of this figure is the relatively high stress that develops at the edge of the connection plate near the column web of specimen (E). Specimens (A) and (D) do not have this large stress because of their column web weld. As is shown in Fig. 13, this point in the connection plate yields, and is therefore a point in the connection which is susceptible to fracture. However, the stress perpendicular to the weld, Sy, and the plastic strain, 0.043 percent at a load factor of 1.0, are not large at this point compared to the stress and plastic strain at the re-entrant corner, and therefore should not play a critical role in the failure of the connection. It is also noted that, because the connection plates in specimens (D) and (E) are extended slightly beyond the column flange tips, Sy approaches zero rather than a large compressive stress, as in specimen (A) where the

connection plate is flush with the column flange tips.

The shear stress, Sxy, along the column flange is shown in Fig. 9. The shear stress in specimen (E) is greater than that in specimen (D) throughout the entire length of the column flange. This, again, is due to the fact that the entire load is carried by the column flanges in specimen (E). Also, instead of a gradual reduction in Sxy towards the column web, as in specimen (D), Sxy in specimen (E) increases since there is no column web weld to add restraint to the connection plate.

3.5 Effective Plastic Strain

The effective plastic strain, ϵ_p, for specimens (D) and (E) is shown in Fig. 10. The large strain concentration at the re-entrant corner of both specimens is shown by the quick drop in strain towards the center of the beam flange. Because of the transfer of stresses due to no column web weld, specimen (E) has a higher plastic strain than specimen (D) along the connection plate and beam flange interface. At a load factor of 0.6, the effective plastic strains at the critical corner of the connection in specimens (D) and (E) are 0.314 percent and 0.513 percent, respectively. Even at the working load level, significant plastic strain has already developed. At a load factor of 1.0, the

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effective plastic strains are 1.838 percent and 2.228 percent for specimens (D) and (E), respectively. These plastic strains are approximately 10.7 and 12.9 times the elastic strain, of 0.172 percent, at yield. Large plastic strains, such as these, can exhaust the ductility of the material and cause initial minor flaws in the material or welds to result in fracture of the connection. The higher the plastic strain, the more susceptible the connection is to fracture.

3.6 Yielding Sequence

Figures 11, 12, and 13 show the yielding sequence for specimens (A), (D), and (E), respectively. The progression of yielding is shown for five stages of loading for each specimen. At the working load level of 0.6 of the nominal yield stress, specimen (A) shows slight yielding around the re-entrant corner. The next stage, 0.85, shows greater yielding in the same region, with most of the plastification occurring in the connection plate. The next two stages of 0.95 and 0.98 show greater yielding throughout the connection plate and beam flange, with the final stage of 1.0 showing major plastification of the entire connection. This final region of yielding shows clearly how most of the load is transferred to the column flanges from a uniform state of

stress at the end of the beam flange.

The yielding sequence of specimen (D) is shown in Fig. 12. The most significant difference between the yielding pattern of specimen (D) and that of specimen (A) is that all of the yielding occurs in the beam flange. The thicker connection plate forces the yielding out into the beam flange. This is a more favorable condition, because the beam flange is less restrained than the connection plate and therefore less susceptible to fracture. As in specimen (A), yielding first occurs at the re-entrant corner in specimen (D), but then yielding progresses into the beam flange rather than the connection plate.

Like specimen (D), yielding of specimen (E) occurs in the beam flange as is shown in Fig. 13. An exception to this is at the edge of the connection plate near the column web. However, as previously discussed, the plastic strain at this point is not significant and therefore, yielding of this point is not of major concern. At the working load of 0.6 of the nominal yield stress, yielding is already taking place in specimens (D) and (E). As was shown in the load-displacement curve of Fig. 5, yielding of specimen (E) began at a lower load than specimen (D). This, together with the slightly greater yielding of specimen (E) at stages 0.6 and 0.65,

shows that the stress concentration at the re-entrant corner is higher than that of specimen (D), due to no welding of the connection plate to the column web. At stage 0.95, the yielded region of specimen (E) at the re-entrant corner is the same as the yielded region of specimen (D) which extends halfway across the beam flange at the connection plate interface. In addition to this, specimen (E) has slight yielding of the connection plate near the column web, and also is just starting to yield at the end of the beam flange. At 0.98, specimen (E) shows slightly greater yielding than specimen (D) along the connection plate and beam flange interface, and at 1.0, major overall yielding, as in specimen (D), is taking place.

3.7 Plate Thickness Variation

The yielding sequence of specimen (D) shows that a thicker connection plate does indeed provide resistance for the shear lag that develops, and forces the yielding of the connection into the less restrained beam flange. For this reason, an additional study was performed using a connection plate thickness of 3.49 cm (1-3/8 inch), designated here as specimen (H), to determine if the 4.13 cm (1-5/8 inch) connection plate of specimen (D) is necessary. Specimen (H) is in addition to the ten

specimens in the proposed experimental test program, and is the same as specimens (A) and (D) except for the connection plate thickness.

Table 1 is a comparison of first yield, effective stress, and effective plastic strain at the critical re-entrant corner of the beam flange of specimens (A), (D), and (H). As the thickness of the plate increases, more restraint is provided, thereby causing greater uniformity in the stress at the cross-section and lowering the initial stress concentration at the corner, as shown by the values of first yield. At the working load of 0.6 of the nominal yield stress, the connection plates of specimens (D) and (H) reduce the plastic strain at the corner, relative to that of specimen (A). However, at a load factor of 1.0, while specimen (D) has a lower plastic strain than specimen (A), specimen (H) has a plastic strain similar in magnitude to that of specimen (A). This result is due to the fact that the restraint from a thicker connection plate, while it reduces the stress concentration, also increases the stiffness of the connection. For a 2.54 cm (1 inch) thick beam flange at a load factor of 1.0, Table 1 shows that the connection plate thickness at which the reduction in stress concentration no longer outweighs the

increase in stiffness at the re-entrant corner is approximately 3.49 cm (1-3/8 inch). Therefore, while improving strain conditions at the working load of 0.6 of the nominal yield stress, the connection plate thickness of specimen (H) makes no difference in the strain at the full plastification level.

Figure 14 shows the yielding sequence of specimen (H). This yielding sequence is nearly identical to that of specimen (D). As in specimen (D), no yielding occurs in the connection plate. Table 2 shows the percent of applied force carried by the column flanges of specimens (A), (D), and (H) for load factors of 0.6 and 1.0. Even though more load is transferred to the flanges by the thicker connection plates, these plates reduce the stress and therefore avoid yielding around the column flanges. At an applied load factor of 1.0, the column flanges of specimen (A) carry only approximately half of the total applied load. This is due to the yielding of the connection plate in the region of the column flanges.

Although a thicker connection plate forces yielding into the beam flange and reduces the initial stress concentration at the critical point, the added stiffness caused by the thickness is harmful to the plastic strain as the load approaches full plastification, and depending

on the thickness can cause the connection to be more

susceptible to fracture than can the thinner connection

plate.

4. CONCLUSIONS AND RECOMMENDATIONS

4.1 Conclusions

Conclusions based on the theoretical analysis previously discussed are provided. It should be noted that the conclusions are based only on theoretical analysis and have not been verified by experimental testing at this time.

1. Because of the geometric properties of the specimens studied, large stress concentrations form at the re-entrant corners of the connection plates and beam flanges. These stress concentrations result in early initial yielding of the connections at loads only slightly higher than half the working load of 0.6 of the nominal yield stress.

2. All four connections studied have an initial tensile stress concentration, at the re-entrant corner, between 2.5 and 3.0.

3. Increasing the thickness of the connection plate reduces the initial stress concentration, and therefore increases the load required to cause first yield.

4. When there is no weld between the connection

plate and the column web, the stress
concentration is higher and thus causes first
yield of the connection at a lower load.

5. By using a thicker connection plate, yielding
   of the connection is forced into the beam
   flange and away from the restraints of the
   column.

6. Due to the early yielding at the re-entrant
   corner, large plastic strains develop in the
   connections at this point. At the load factor
   of 1.0, the specimens studied have effective
   plastic strains between 1.838 and 2.228
   percent. These plastic strains are over 10
   times the elastic strain at yield. Large
   plastic strains, such as these, can exhaust
   the ductility of the material and result in
   fracture of the connection.

7. Increasing the thickness of the connection
   plate reduces the initial stress
   concentration, but increases the stiffness.
   Because of the greater stiffness, the plastic
   strain increases at a faster rate as the load
   approaches full plastification. Therefore, a

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thicker connection plate is beneficial to the
connection, provided the initial stress
concentration is low enough to offset the
disadvantages of an increase in stiffness.

4.2 Recommendations

Tentative recommendations for the four connection
plate geometries studied can be made for design purposes,
based on the conclusions presented above. These
recommendations and recommendations from other studies on
different connection geometries should be combined to
make final guidelines for design.

1. Use connection plates with a thickness based on
   equation (9) and welded to the column web.
   This type of connection forces yielding into
   the beam flange, reduces the initial stress
   concentration, and lowers the plastic strain
   of the critical re-entrant corner.

2. Avoid the use of connections without a weld
   between the column web and connection plate.
   This type of connection has the largest
   plastic strain at the re-entrant corner, and
   therefore increases the susceptibility of the
   connection to fracture.

3. Connections with plates the same thickness as the beam flange or with thicknesses between that of the beam flange and the thickness based on equation (9), do not perform as well as connections with plate thicknesses based on equation (9). However, if the change in beam flange thickness to the connection plate thickness based on equation (9) is thought to be too large, then an intermediate thickness is recommended over the use of a connection plate with the same thickness as the beam flange.

Table 1 First Yield, Effective Stress, and
         Effective Plastic Strain at the
         Re-entrant Corner

Specimen	First Yield *	$\sigma = 0.6\ \sigma_Y$		$\sigma = 1.0\ \sigma_Y$	
		S_{eff}	ϵ_p	S_{eff}	ϵ_p
A (2.54 cm)	0.333	1.065	0.565	1.237	1.972
H (3.49 cm)	0.372	1.043	0.369	1.231	1.974
D (4.13 cm)	0.386	1.036	0.314	1.215	1.838

* First yield equals $\frac{\sigma}{\sigma_Y}$, where σ is the
stress that causes the first element to yield

Table 2 Force Carried by the Column Flanges

Specimen	Percent Of The Applied Force Carried By The Column Flanges	
	0.6 σ_Y	1.0 σ_Y
A	74.1	54.8
H	78.6	77.2
D	79.2	78.1
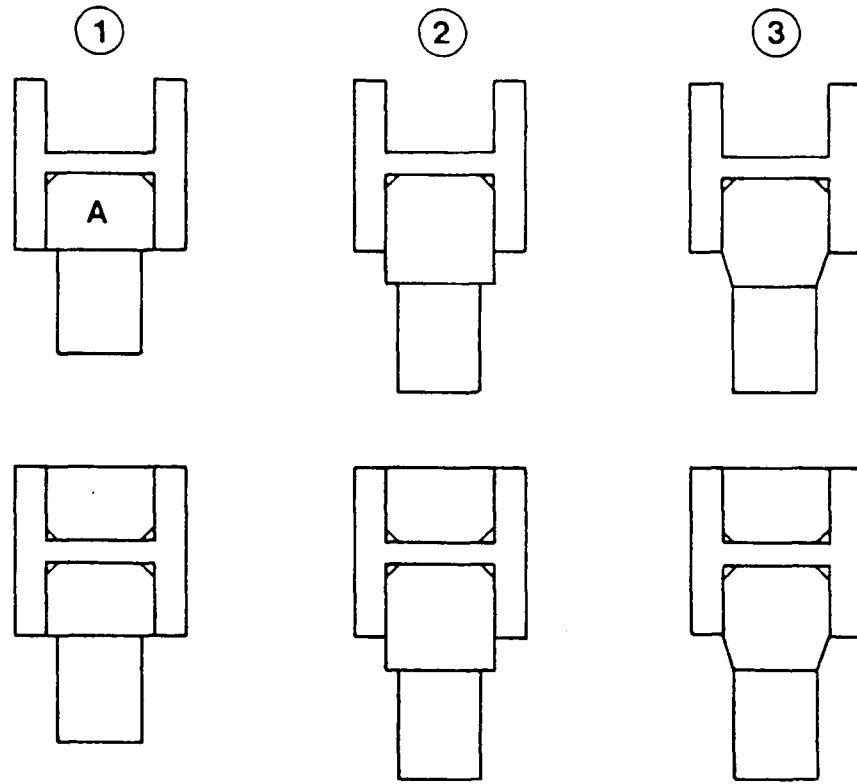
2.54 cm Plates

Fig. 1 Test Series

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Simulated Beam Flange

Connection Plate

Column

Specimen

Welded End Plate

Reusable Bolted Joining Plate

Reusable Test Fixture
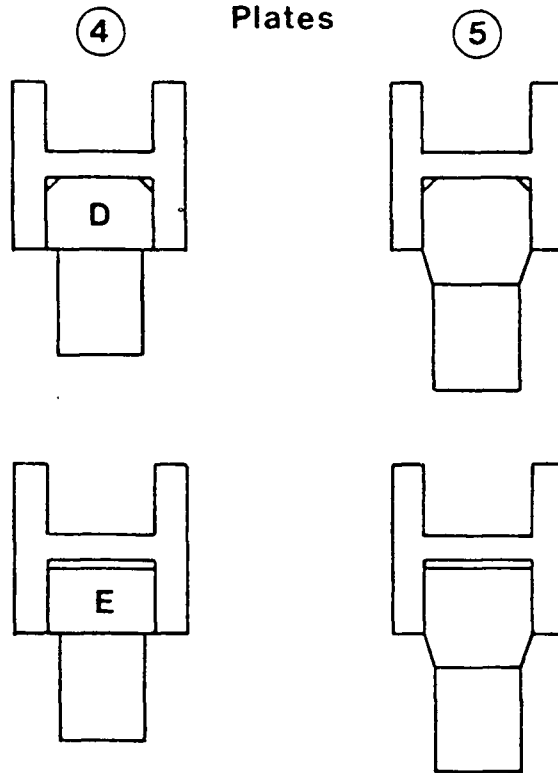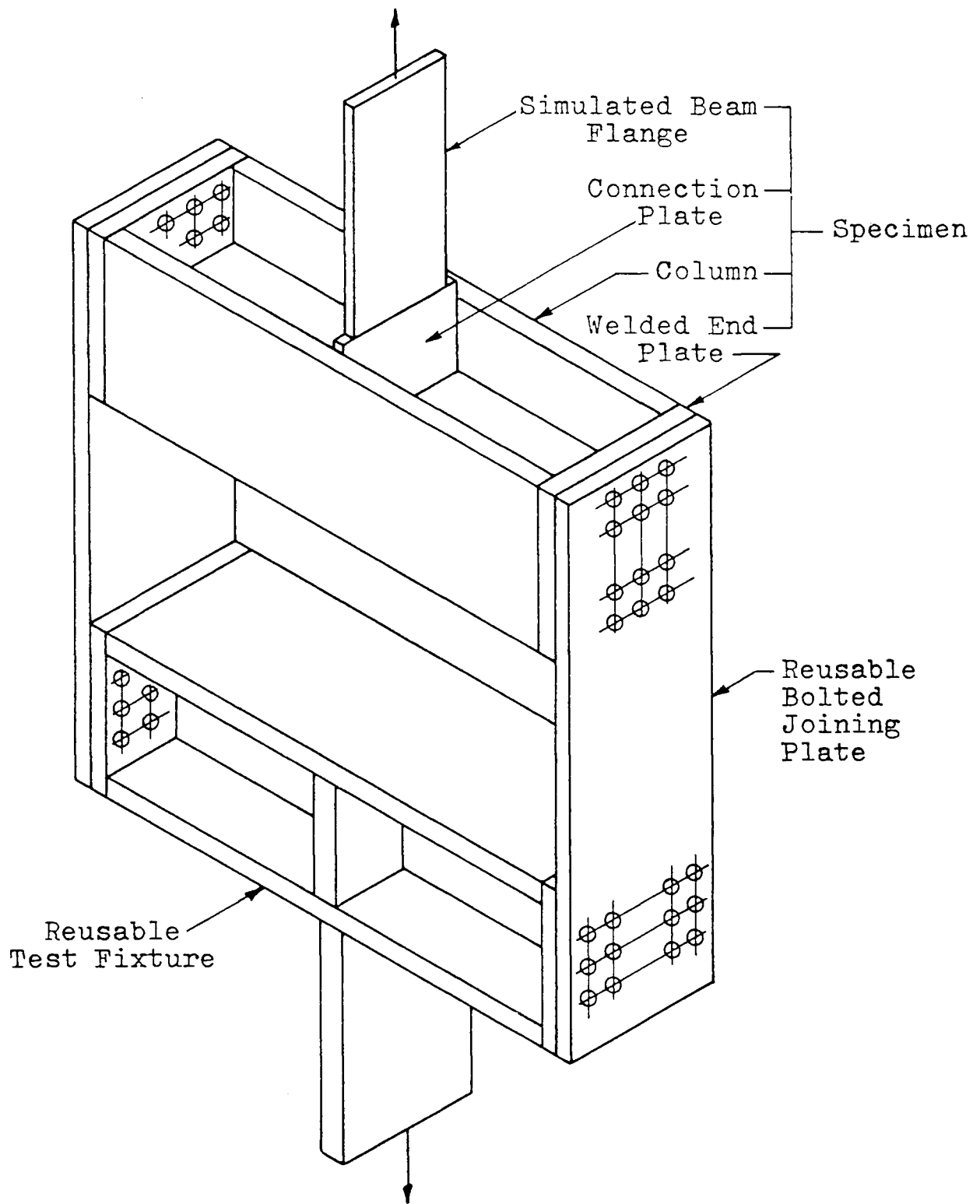
Fig. 2 Test Setup
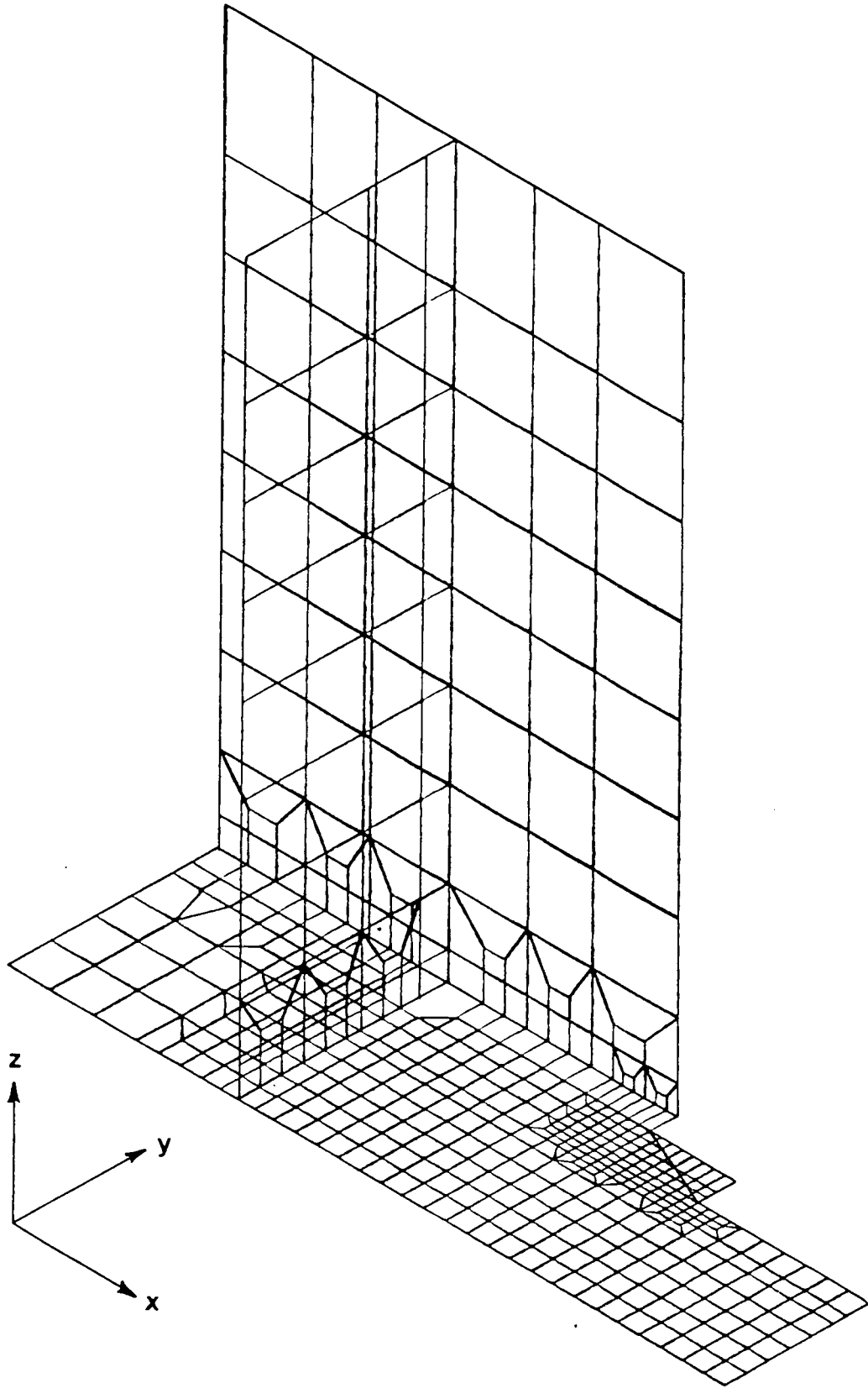
Fig. 3 Finite Element Model

$\sigma_Y = 345$ MPa (50 ksi) (A572)

$E = 200$ GPa (29000 ksi)

$E' = 4.14$ GPa (600 ksi)
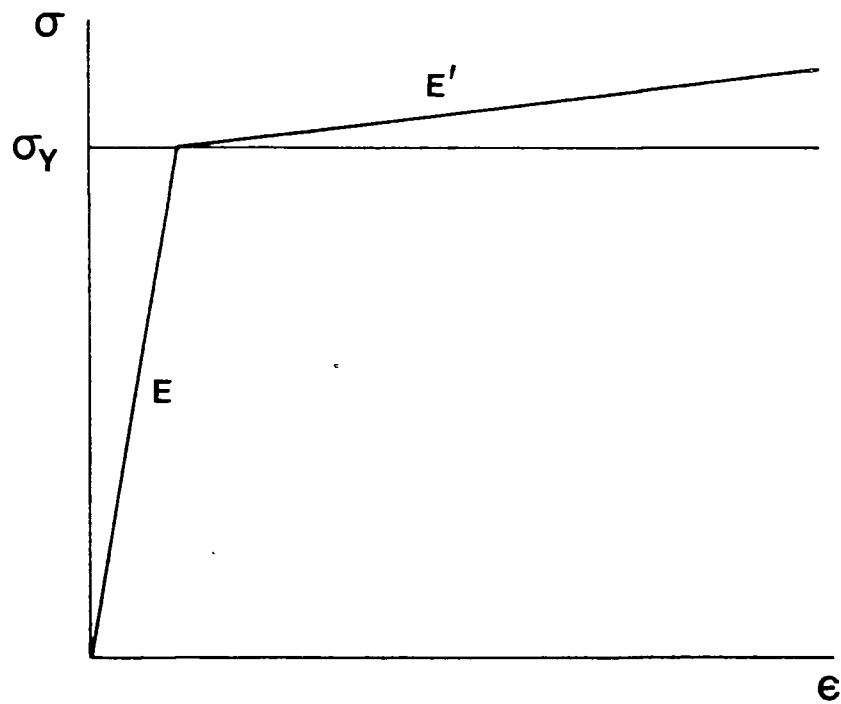
Fig. 4 Stress-Strain Relationship
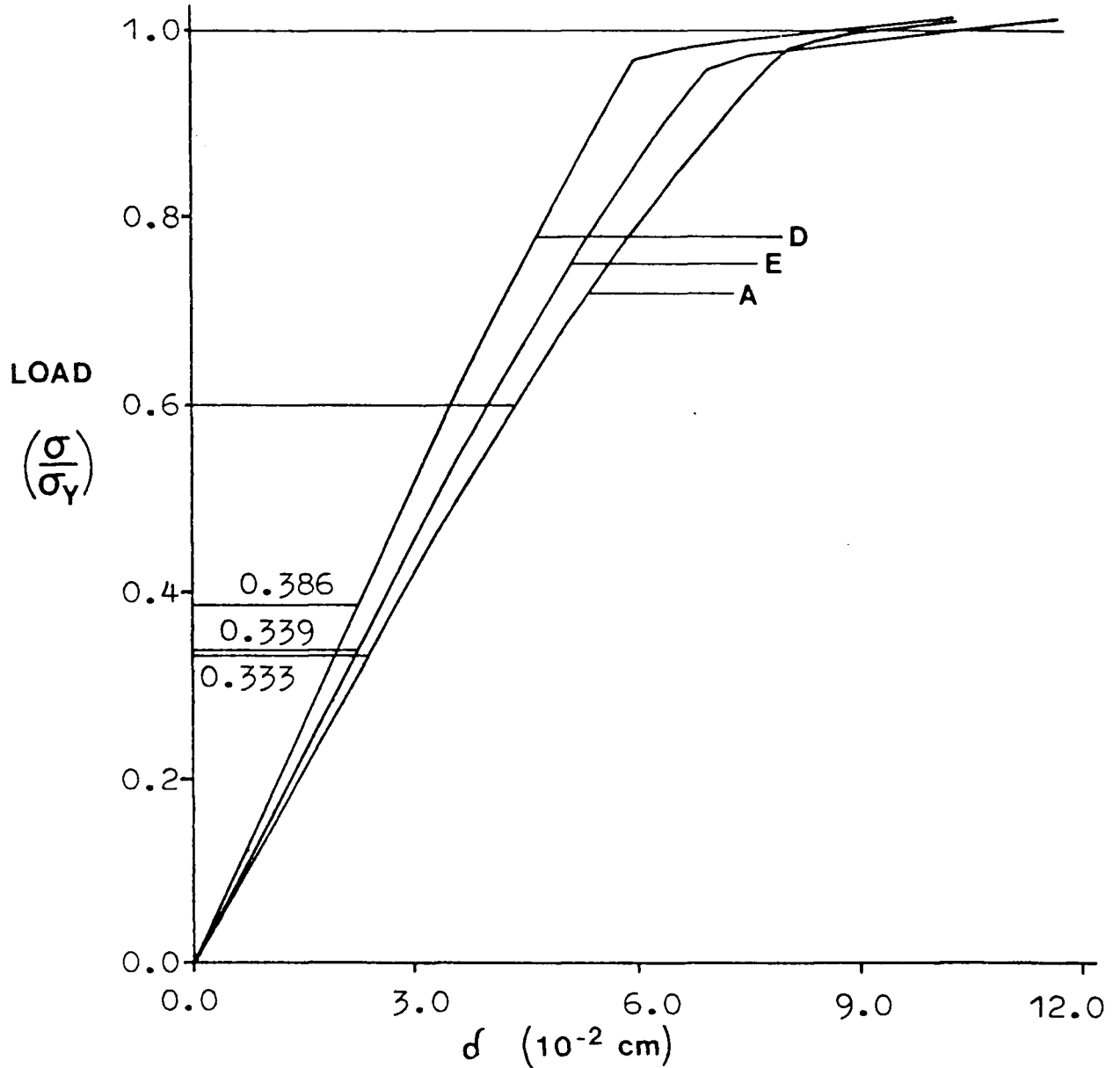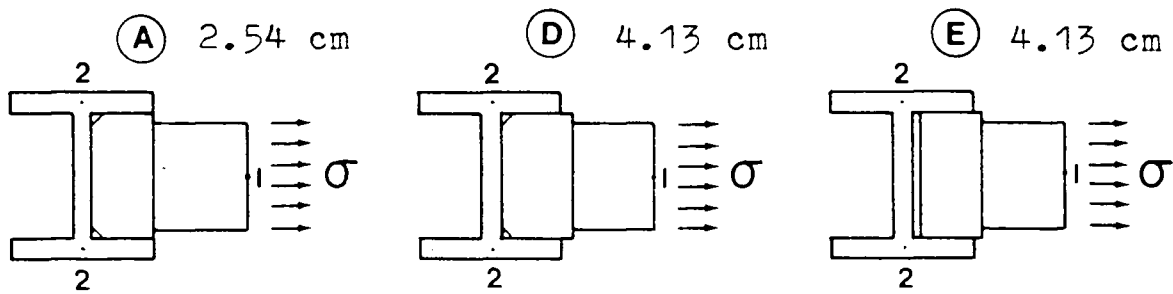
Longitudinal Displacement of
Point 1 Relative to Points 2

Fig. 5   Load-Displacement Curves

42
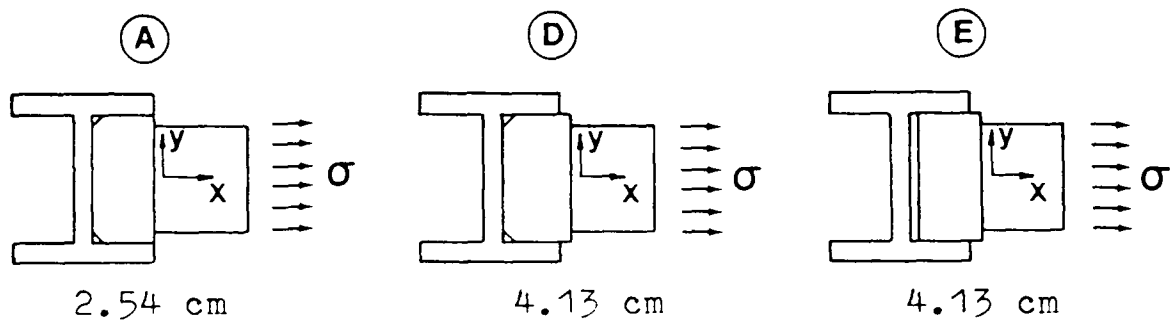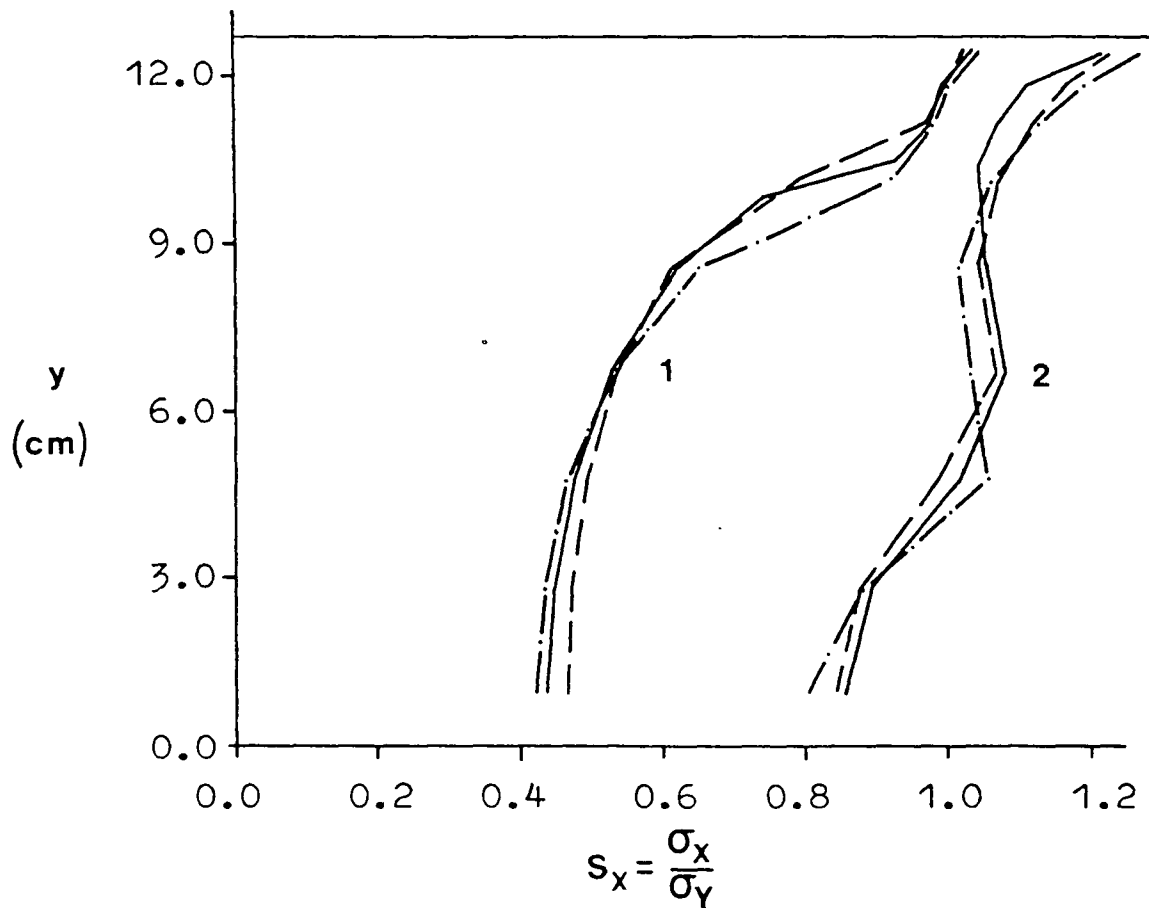
Fig. 6   Longitudinal Stress Distribution S_X Along the
Connection Plate and Beam Flange Interface

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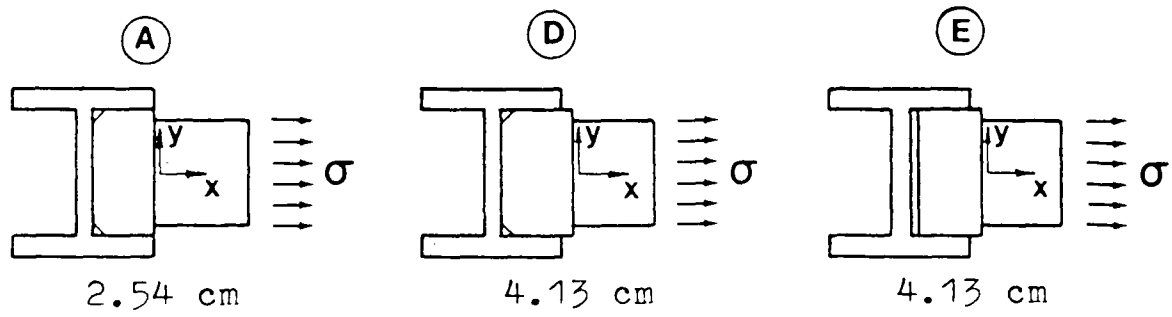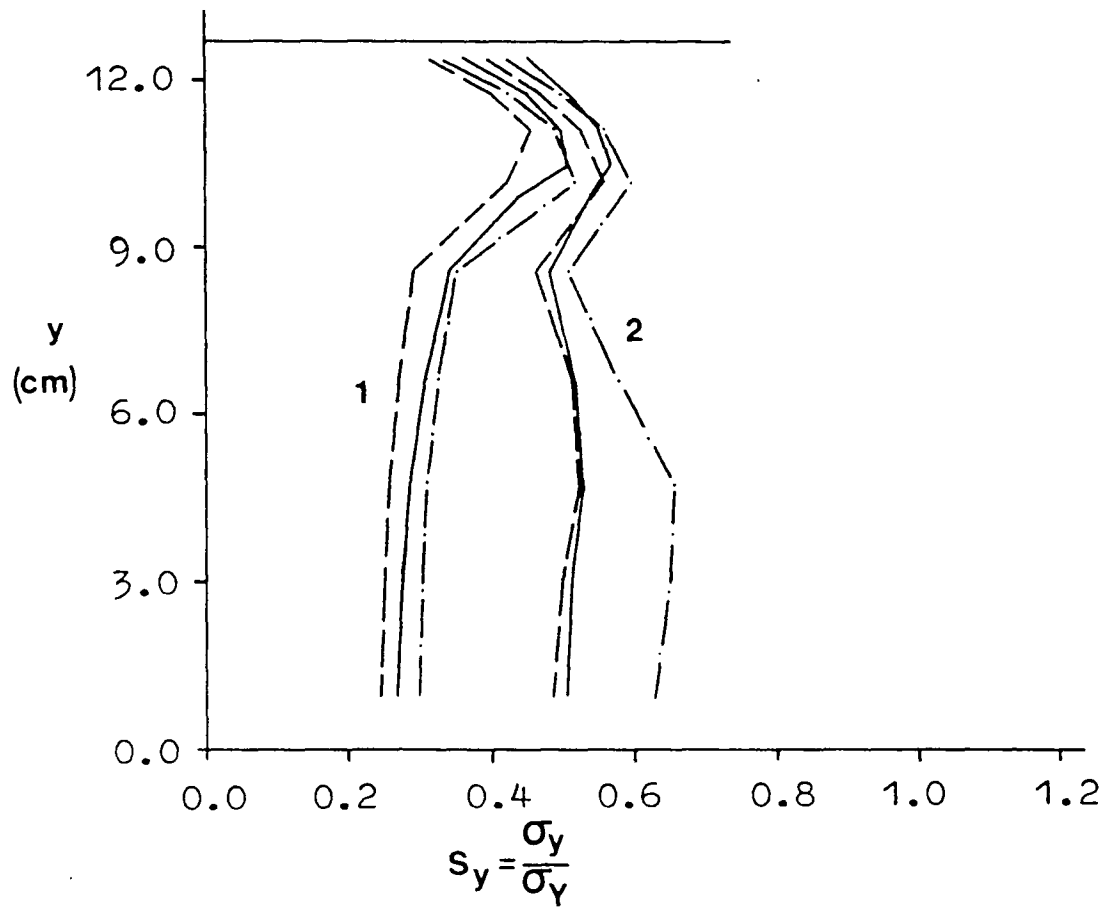
Fig. 7 Transverse Stress Distribution Sy Along the
Connection Plate and Beam Flange Interface

44
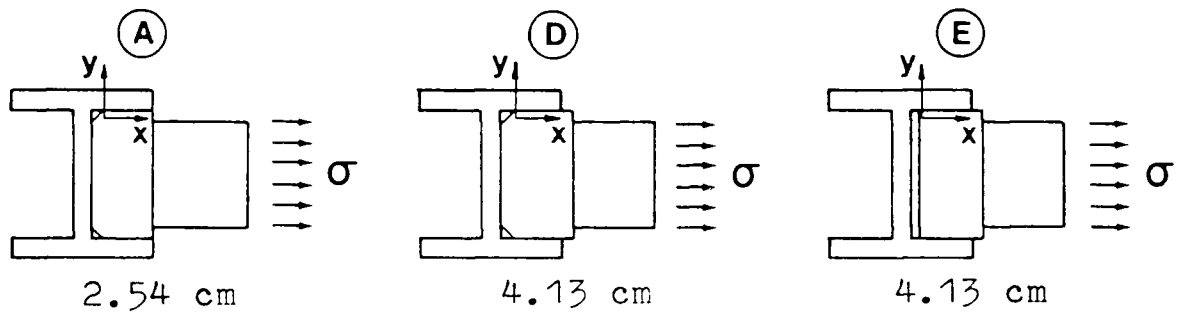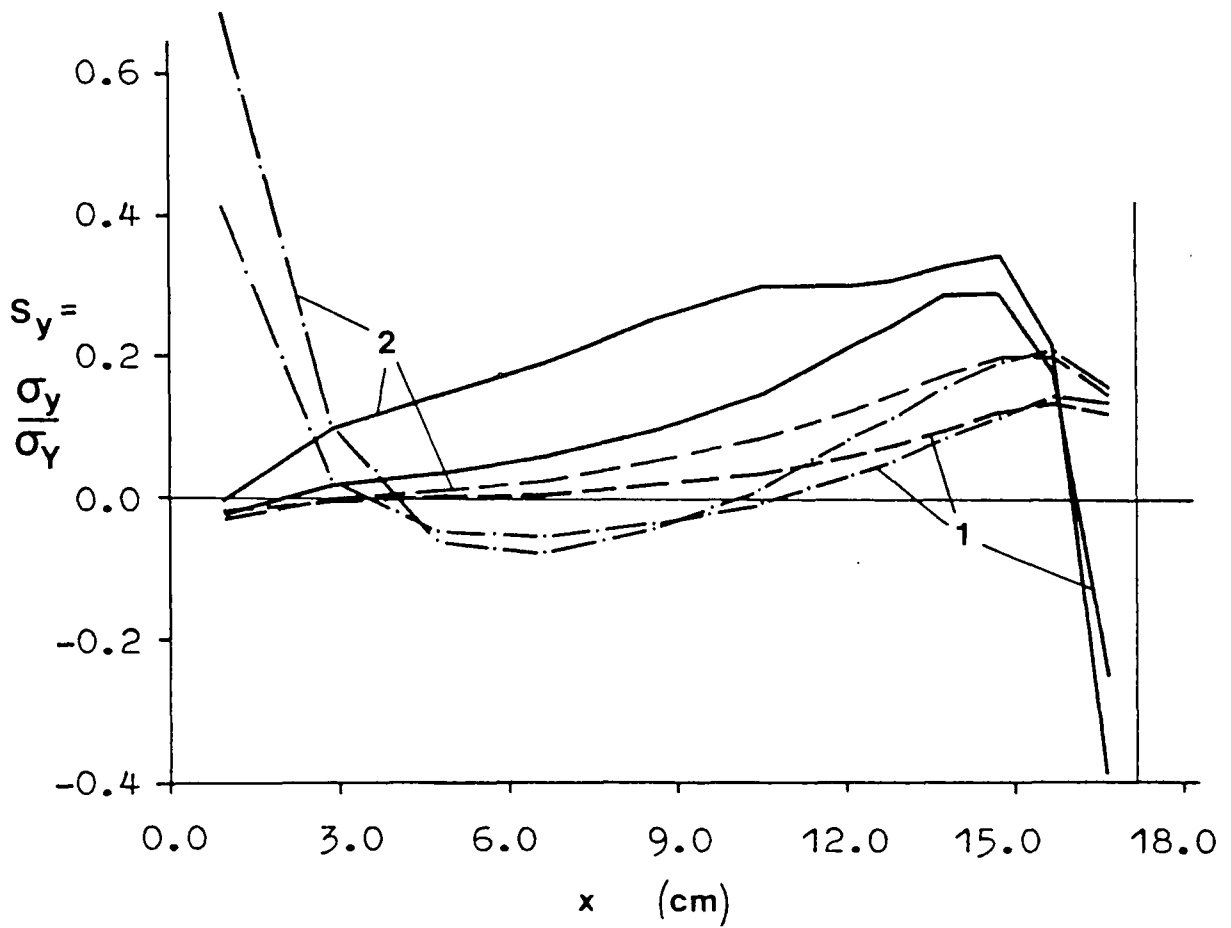
Fig. 8   Transverse Stress Distribution
Sy Along the Column Flange
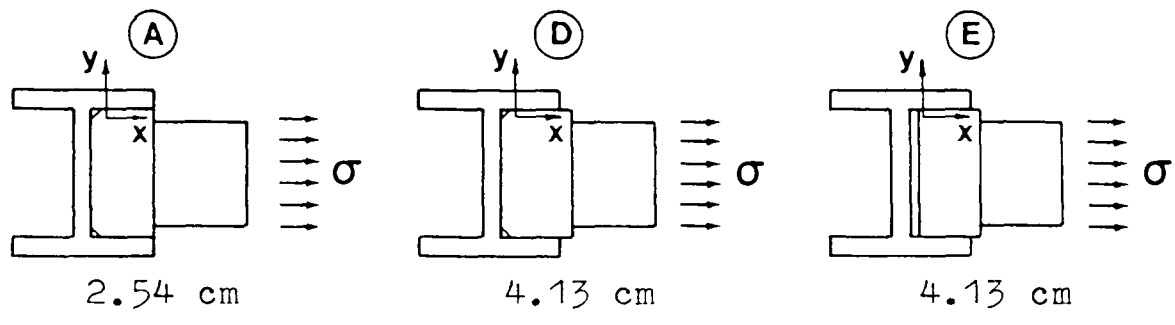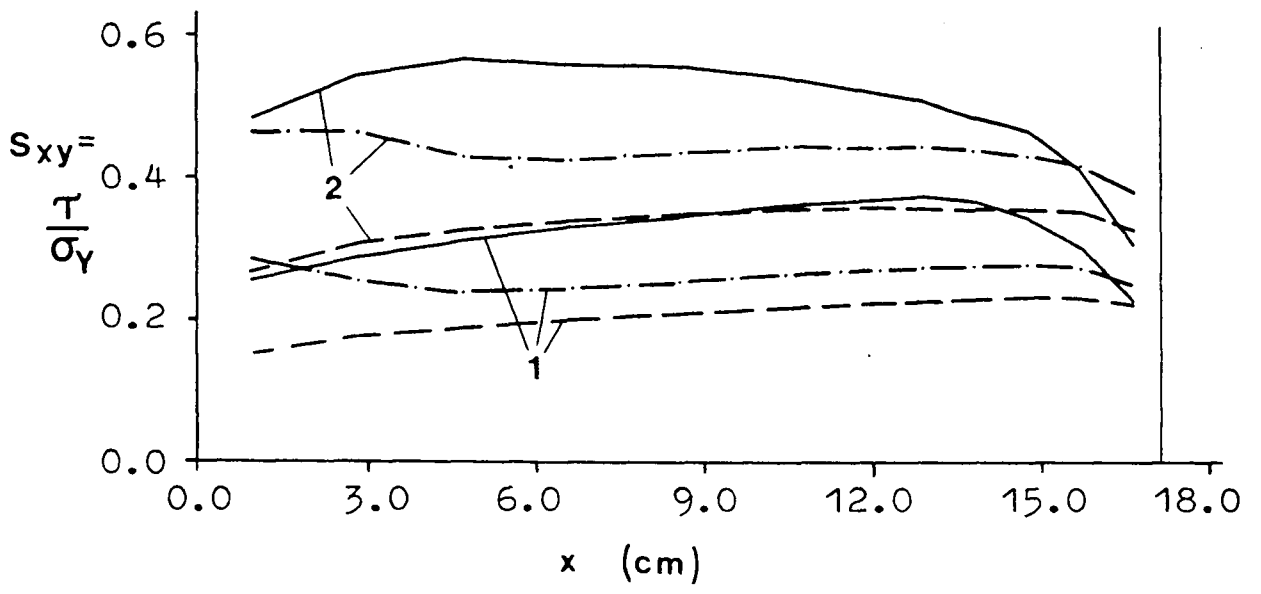
Fig. 9   Shear Stress Distribution
          S_{xy} Along the Column Flange
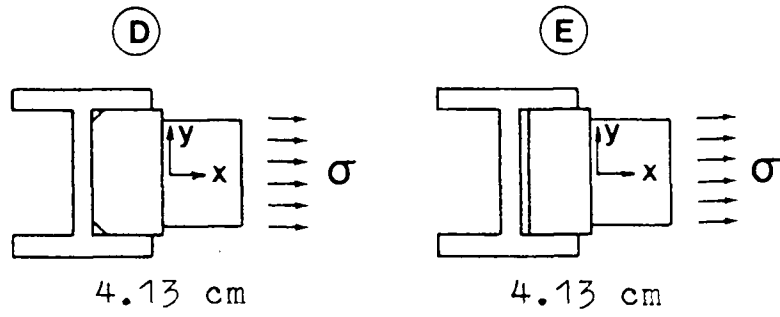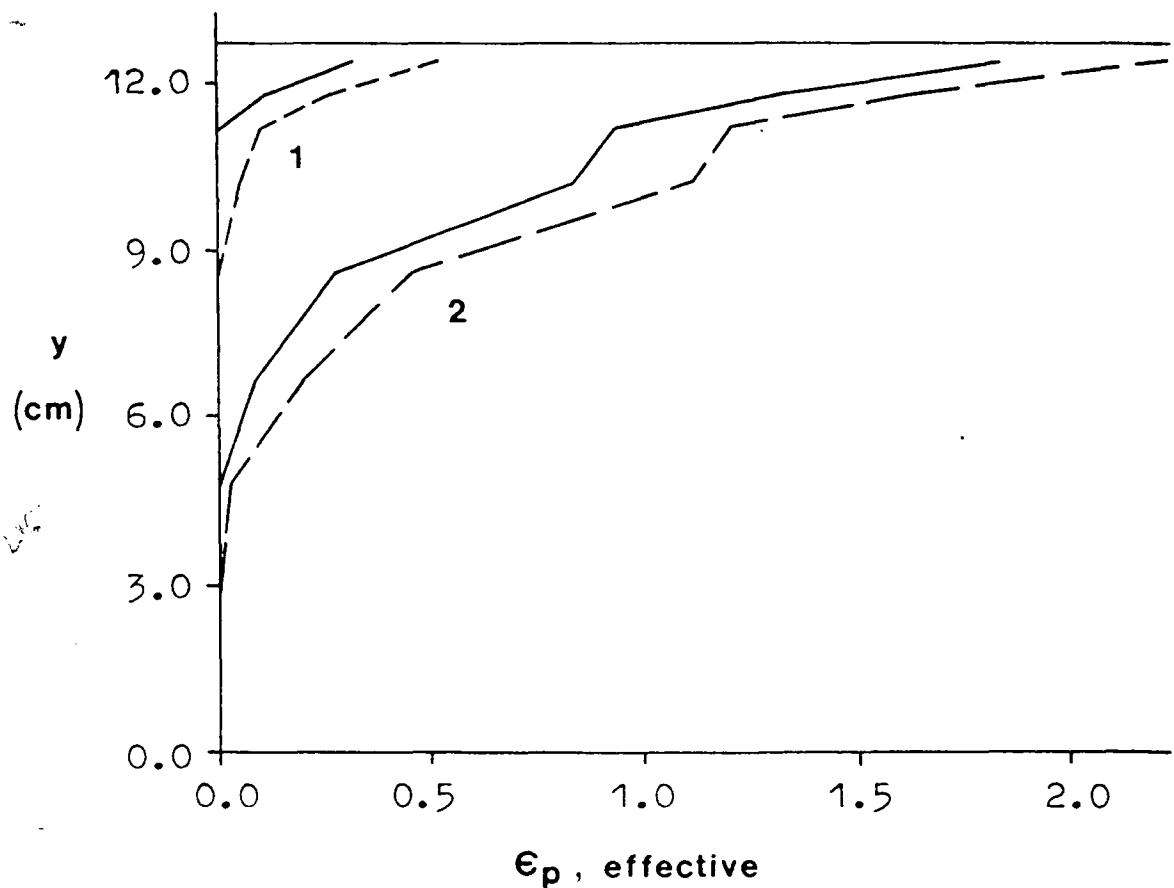
Fig. 10   Effective Plastic Strain ϵ_p Along the
Connection Plate and Beam Flange Interface

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$\sigma = 0.95\sigma_Y$

$\sigma = 0.98\sigma_Y$

$\sigma = 1.0\sigma_Y$

σ

Ⓐ

2.54 cm

$\sigma = 0.6\sigma_Y$
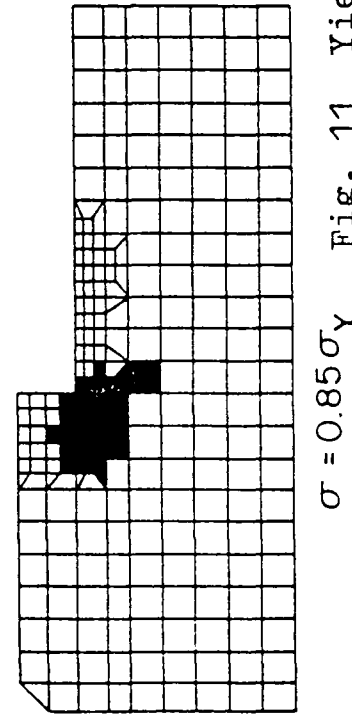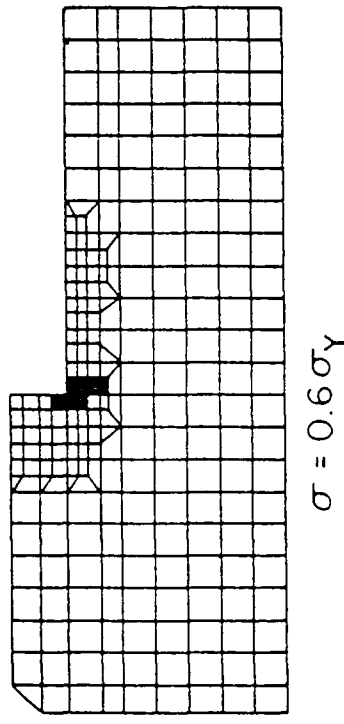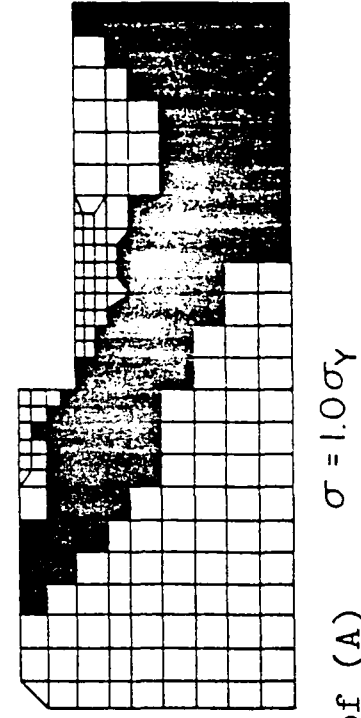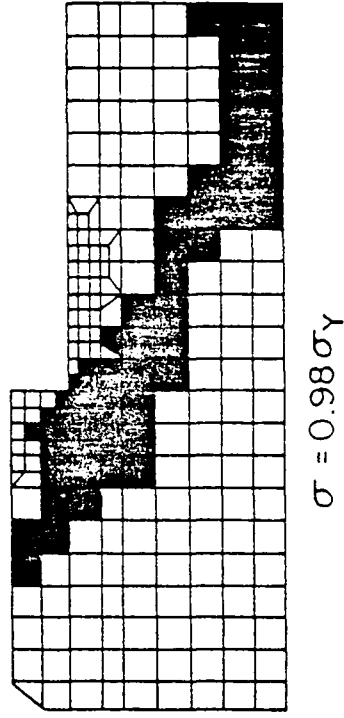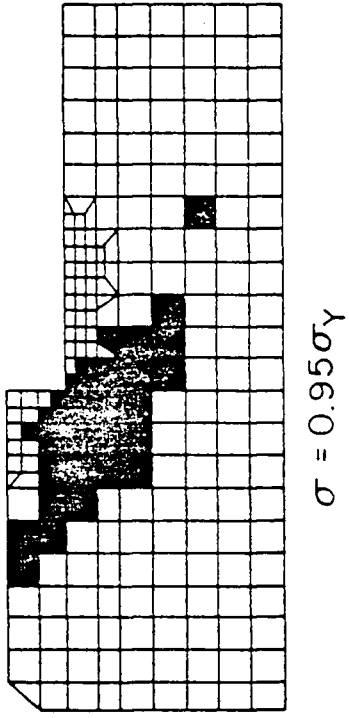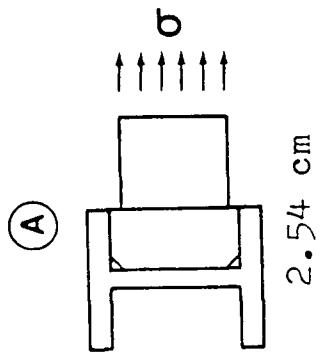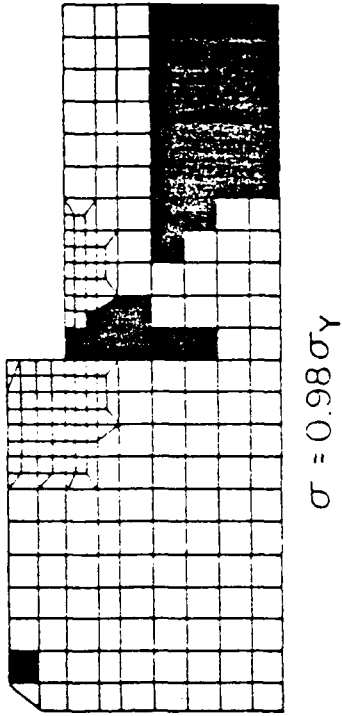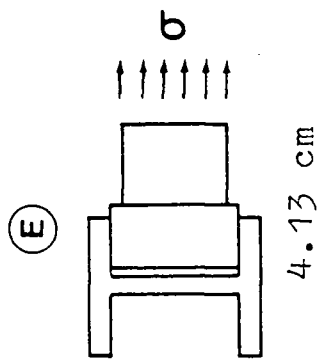
$\sigma = 0.85\sigma_Y$

Fig. 11 Yielding Sequence of (A)

48

σ = 0.95σ_Y

σ = 0.98σ_Y

σ = 1.0σ_Y

σ = 0.6σ_Y
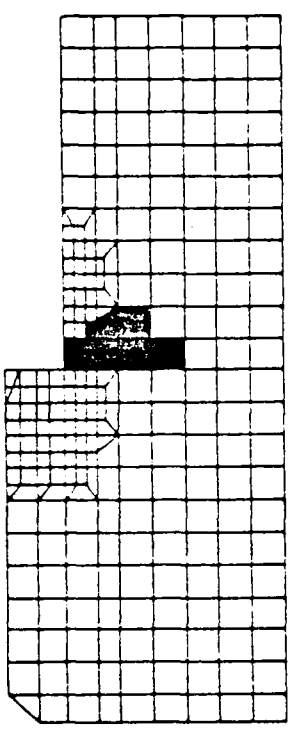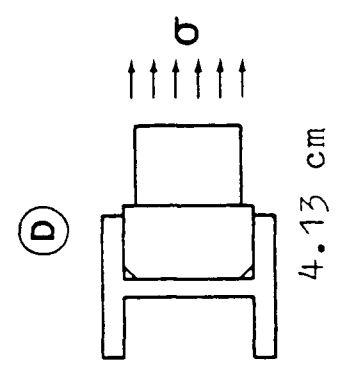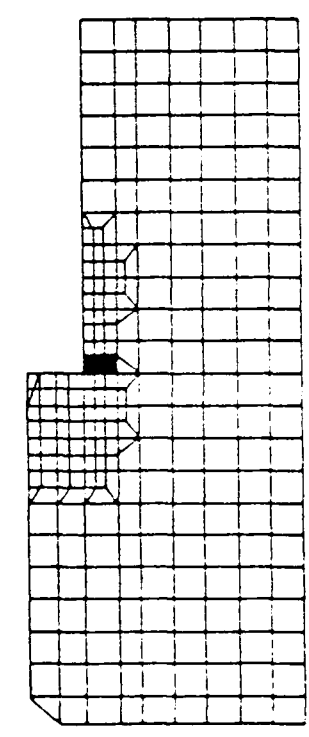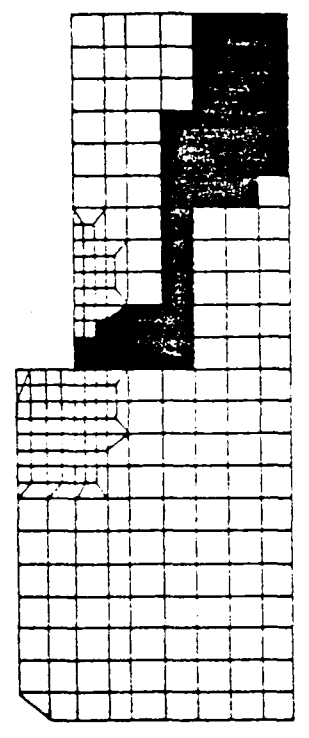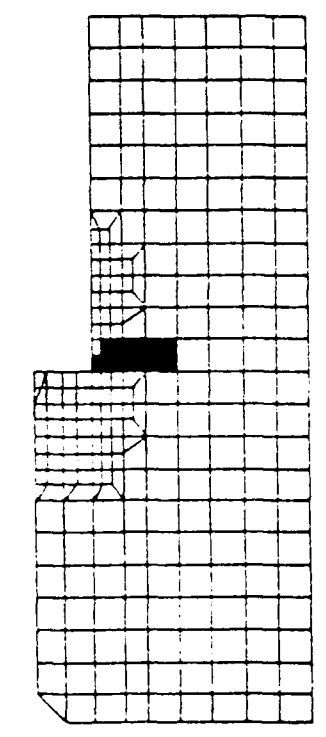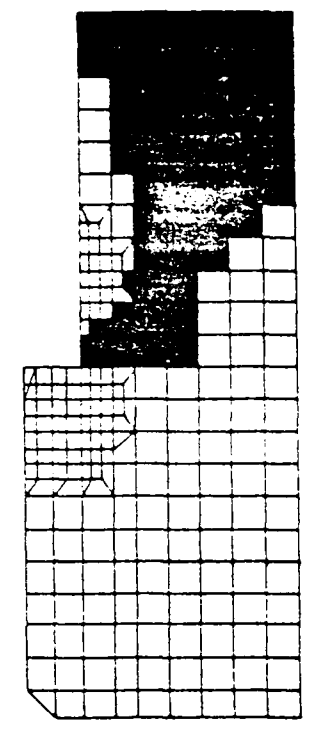
σ = 0.85σ_Y

Fig. 12 Yielding Sequence of (D)

4.13 cm

D

σ

49

$\sigma = 0.95\sigma_Y$

$\sigma = 0.98\sigma_Y$

$\sigma = 1.0\sigma_Y$

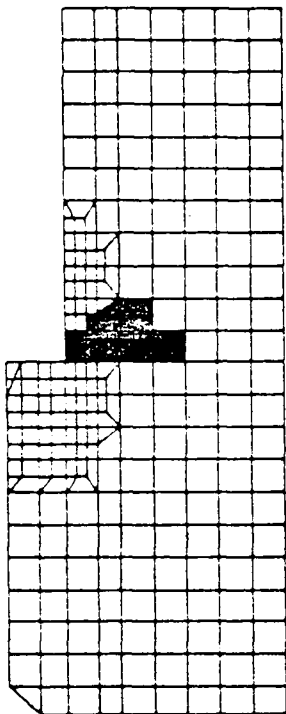$\sigma = 0.6\sigma_Y$

$\sigma = 0.85\sigma_Y$
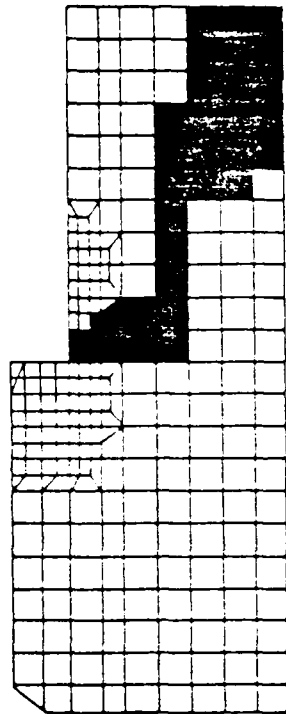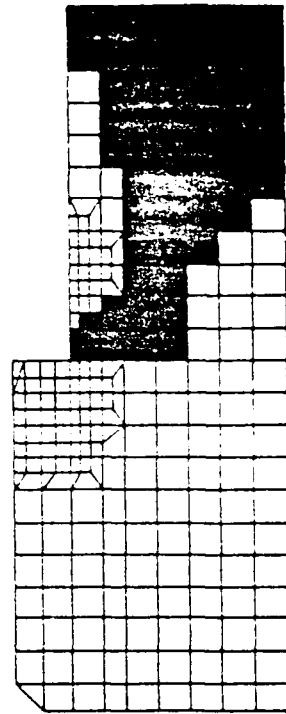
σ

4.13 cm

(E)
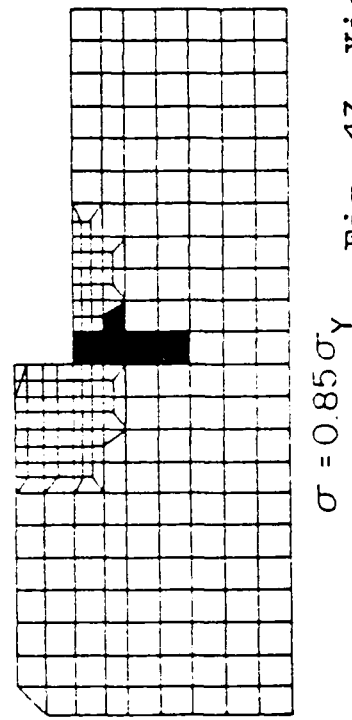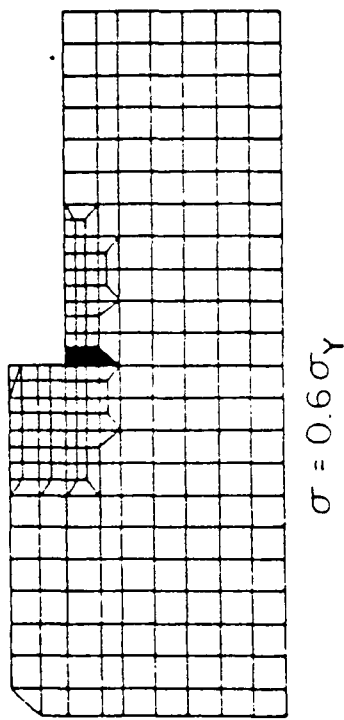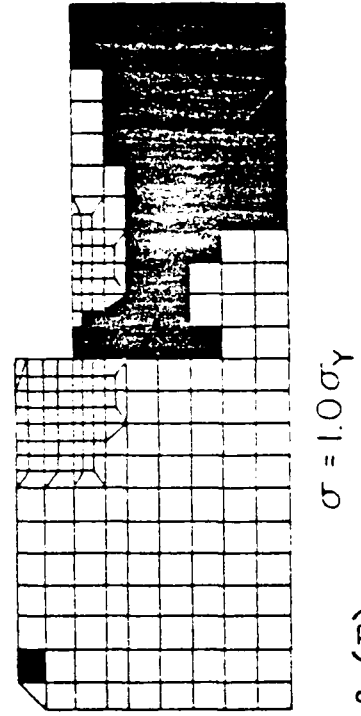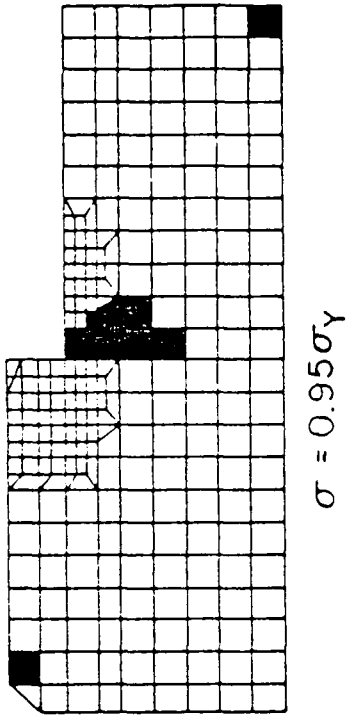
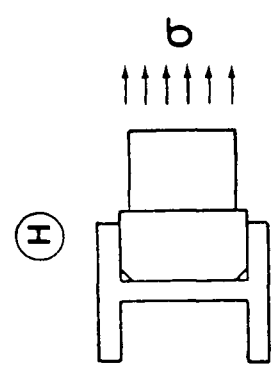Fig. 13 Yielding Sequence of (E)

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$\sigma = 0.95\sigma_Y$
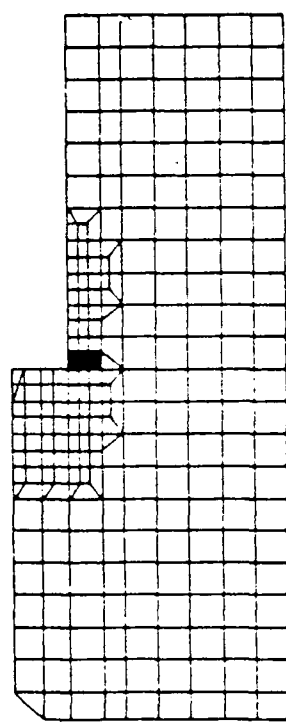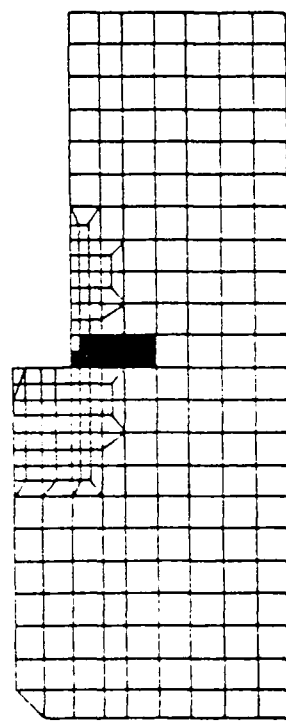
$\sigma = 0.98\sigma_Y$

$\sigma = 1.0\sigma_Y$

σ

3.49 cm

(H)

$\sigma = 0.6\sigma_Y$

$\sigma = 0.85\sigma_Y$

Fig. 14 Yielding Sequence of (H)

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VITA

Randall Matthew Haist was born on June 22, 1958 in Ivyland, Pennsylvania. He is the second of two children of Matthaus A. and Virginia F. Haist.

He attended Council Rock High School in Newtown, Pennsylvania and graduated in June, 1976. In September, 1976, he enrolled at Lehigh University, Bethlehem, Pennsylvania and studied Civil Engineering. In June, 1980, he graduated with high honors and received the degree of Bachelor of Science.

He received a fellowship from the American Institute of Steel Construction for graduate study at Lehigh University, and began his graduate program in September, 1981. Part of his studies involved the research of beam-to-column web moment connections.