Structural Engineering Report No. 148

DESIGN OF WEB-FLANGE BEAM OR GIRDER SPLICES

by

D. L. GREEN

and

G. L. KULAK

May, 1987

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Deborah L. Green

and

Geoffrey L. Kulak

Department of Civil Engineering

University of Alberta

Edmonton, Alberta

May, 1987

Abstract

Splices in beams and girders are often required when the lengths of members are limited by fabrication, transportation, or handling facilities available, or by the construction process. This study investigates the behaviour and ultimate strength of a bolted web-flange beam or girder splice in which both the web and the flanges are spliced at the same location. Current design methods vary and there has been little experimental work done to verify these methods.

A recently proposed web-flange splice design procedure has been further developed herein. It is a development that is similar to the method currently used to analyze eccentrically loaded bolted connections, that is, it satisfies the equations of static equilibrium and uses the actual shear load versus shear deformation response of the bolts. It has been determined that for a web splice located at a point of contraflexure, the equilibrium equations developed in this method yield results identical to an analysis that treats the bolts on one side of the splice as loaded by a shear force acting at the centerline of the splice. For a web-flange splice located at a point where both shear and moment are present, this method yields results identical to an analysis that treats the bolts on one side of the splice as acting under the moment at the centerline of the splice in addition to the transverse shear acting at the centerline of the splice.

The experimental program was limited to the case of a girder in which only the web was spliced. It involved the testing of five large scale bolted web splices located at a point of contraflexure and one bolted web splice located at a point where both shear and moment were present. Two series of tests were carried out on single bolt specimens loaded in double shear, using both compression and tension jigs, to obtain the shear load versus shear deformation response of a single bolt.

For a bolted web splice, the best agreement between theory and experiment is achieved by using the ultimate strength method of analysis and the actual response to shear load of a single bolt tested in a tension jig, based on the assumption that the shear force acts at the centerline of the splice. Test results reported herein substantiate the analytical method developed for predicting the capacity of a web-flange beam or girder splice.

Acknowledgements

This report is based on the thesis of the same title submitted by Deborah L. Green to the Faculty of Graduate Studies and Research at the University of Alberta in partial fulfillment of the requirements for the degree of Master of Science. This project was supervised by Dr. Geoffrey L. Kulak.

The authors thank R.W. Bailey for the preliminary experimental work he carried out for this project.

The assistance of L. Burden and R. Helfrich during the experimental phase of the study is greatly appreciated.

Financial support for this study was provided by the Natural Sciences and Engineering Research Council of Canada and the assistance of this organization is gratefully acknowledged.

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a = coefficient used to describe the location of the applied load in the analytical splice model

b = coefficient used to describe the location of the applied load in the analytical splice model; bolt pitch

c.g. = center of gravity

d = distance between the centroids of the top and bottom flanges

e = base of natural logarithm = 2.718; eccentric distance

F_f = force in the top or bottom flange bolts on one side of the splice

F_x = horizontal force

F_y = vertical force

i.c. = instantaneous center of rotation

L_1 = distance between the splice centerline and the supports

M = bending moment

M_{ic} = bending moment at the instantaneous center of rotation

n = number of bolts on one side of the web splice

P = concentrated load

r_i = distance from the i^{th} bolt to the instantaneous center of rotation

r_o = radius of rotation (distance from the centroid of one bolt group to its instantaneous center of rotation)

R = bolt load at a given deformation

R_i = force in the i^{th} bolt

R_{iv} = vertical component of the bolt force R_i

R_u = ultimate shear strength of a single bolt

s = bolt gauge distance

t_o = thickness of the outside plates in the single bolt shear specimens

V = transverse shear

x = coefficient used to describe the location of the splice in the analytical model

x_o = distance from the centerline of the splice to the centroid of the bolt group on one side of the splice

Δ = total deformation including shearing, bending, and bearing deformation of a bolt and local deformation of the connecting material

Δ_u = ultimate deformation of a single bolt shear specimen

γ = regression coefficient

μ = regression coefficient

1. INTRODUCTION

1.1 General

Splices in beams and plate girders should be avoided whenever possible because they increase the cost of the members. However, there are certain cases when a splice is necessary. In the fabrication shop, splices are sometimes used to permit the size of the beam cross-section to be changed with length in order to meet the strength requirements more closely. More often, however, shop splices are used when the required beam lengths are not available or when the length of steel members is limited by the fabrication or handling facilities available.

Field splices are required when the length of steel members is limited by the transportation equipment available. The construction process may also dictate that beams or girders be spliced. The size and weight of steel members are limited by the equipment available to handle and erect them. In the erection process, stability considerations such as lateral-torsional buckling may lead to the selection of shorter than otherwise maximum fabricated lengths.

Thus, beams and girders may be spliced either in the shop or in the field. Although shop splices may be bolted, a complete penetration groove weld is usually used to join sections of a beam together. Field splices may be either welded or bolted. Welded splices usually look neater and cleaner than bolted splices, but they require more careful alignment and are more difficult to inspect. The welding process is affected by temperature, as well as by other environmental conditions. Field splices in large bridge or building girders are often bolted due to the difficulty of welding large members in the field.

A commonly used bolted splice is shown in Figure 1.1. Splice plates are lapped across the joint and bolted to the webs and the flanges of the beams or girders to transfer the load. The flange plates may be placed on one side of the flanges only, as shown in Figure 1.1, or they may be present on both sides of the flanges. Sometimes, either the web or the flanges are spliced alone, but usually both the web and the flanges are spliced at the same location. This type of splice is usually referred to as a web-flange splice.

Another type of bolted splice is the end-plate splice shown in Figure 1.2. Plates are shop welded to the ends of the beams or girders, and these are field bolted together at the joint.

This report deals with the design of a bolted web-flange beam or girder splice in which both the web and flange material are spliced at the same location. Current design methods vary. The validity of some of the methods identified in the technical literature has not been substantiated analytically. Furthermore, there has been very little experimental work done to verify any of the analytical approaches. The analytical procedure presented in this report is, in principle, a general procedure that is applicable to both bolted and welded splices. However, its development herein is based on a bolted splice. Tests to substantiate the analytical method were also limited to bolted splices.

1.2 Scope

An analytical method, presented in Chapter 3, has been developed that identifies the forces required for the design of a web-flange beam or girder splice. In order to substantiate the validity of the analytical procedure, a series of six tests was established to determine the ultimate capacity of bolted web splices. In five of these tests, the center of the splice was located at a point in the beam where the moment theoretically was zero. Thus, these connections were loaded primarily in shear. In the sixth test, the splice was located in a region where both shear and moment were present. Because of the complexity of such tests and because no other tests relating to the new analytical method are available, it was decided not to include splices involving both web and flange material in this test program.

Two series of tests were carried out on single bolt specimens loaded in double shear, using both compression and tension jigs, in order to determine the load versus deformation response of the bolts. The actual shear load versus shear deformation response of the single bolts was used to predict the ultimate shear capacity of the bolted web splice specimens tested.

1.3 Objectives

The objectives of this study were:

1. To develop an analytical method for predicting the capacity of bolted web-flange beam or girder splices.
2. To compare the experimental load versus deformation response of single bolts loaded in double shear with previous test results.
3. To test bolted web splices located both in regions where only shear is present and in regions where both shear and moment are present.
4. To compare the web splice test results with theoretically based predictions using the analytical method developed herein.
5. To make recommendations for design rules for bolted web-flange beam or girder splices.

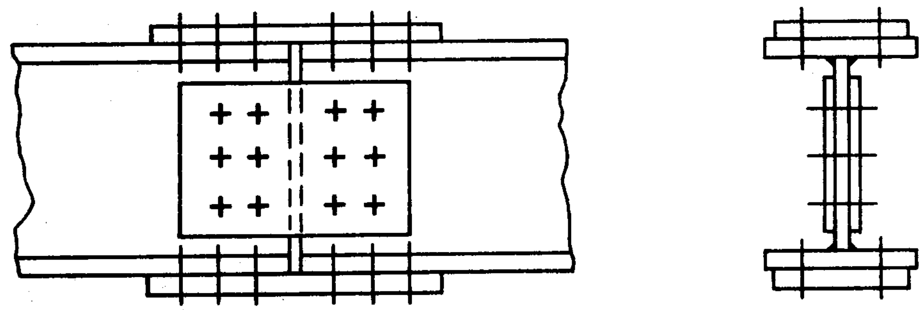

Figure 1.1 Bolted Web-Flange Girder Splice

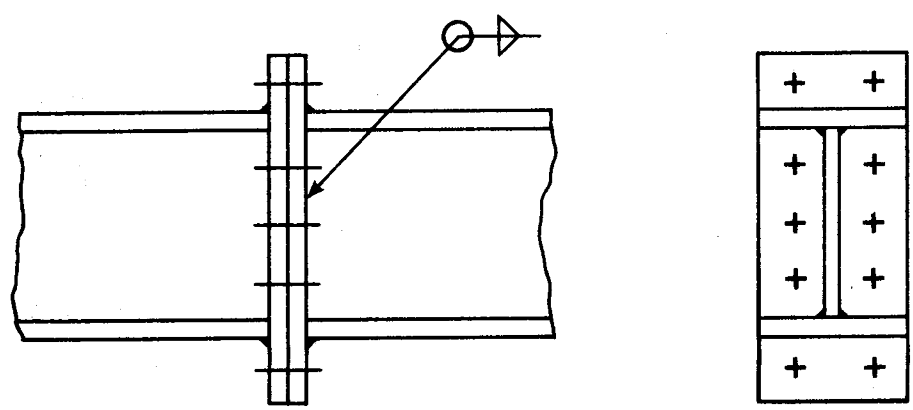

Figure 1.2 Bolted End Plate Girder Splice

2. LITERATURE REVIEW

2.1 Introduction

Plate girders were fabricated traditionally by riveting pairs of steel angles to the top and bottom of a web plate, as shown in Figure 2.1(a). They were designed using the assumption that the flange angles carried either most or all of the moment. In regions where plate girders were required to resist large bending moments, cover plates were riveted to the flange angles, as shown in Figure 2.1(b). Because of the high labour content of such girders, in almost all cases plate girders are currently fabricated simply by welding the flange plates to the web plate, as shown in Figure 2.2. The change in the fabrication method of plate girders has resulted in a corresponding change in the design and fabrication of plate girder splices. Before the use of welding and high-strength bolting developed, riveting was the most widely used method to connect steel members. However, rivets are now obsolete and have been replaced by high-strength bolts. This chapter first examines the historical approach to designing beam or girder splices, and then reviews the current methods used.

2.2 History of Beam or Girder Splices

In the early 1900's splices were designed to be as equivalent as possible to the net section of the beam or girder at the joint. Hence, some design specifications required splices to develop the full shear and moment capacities of the sections being joined (1,2,3). Many designers felt this requirement was too severe, and they often preferred to design splices to resist the maximum shear present combined with the moment capacity of the section at the splice. In 1947, changes in the American Institute of Steel Construction (AISC) specification requirements permitted designers to design splices for the actual stresses if both the web and the flanges were spliced (4). However, the splices were required to resist a minimum of 50% of the member strengths.

Flange angles and cover plates were customarily spliced for their full capacity, although flange splices were rarely required for plate girders that were fabricated using flange angles. Angles and cover plates could be obtained in adequate lengths except for very long girders. Where necessary, flange angles were spliced with:

1. a single splice angle to cover the cut (Figure 2.3(a)),
2. one splice angle on each side of the web plate (Figure 2.3(b)), or
3. a splice angle on one side of the web plate and a splice plate on the other side (Figure 2.3(c)).

The splice angles were often referred to as "bosom angles".

The same rivets used to connect the flange angles to the web plate could also be used to connect the splice angles to the flange angles. Sometimes, however, the rivet spacing was reduced near the splice in order to minimize the length of the splice angles required. When cover plates were a part of the cross-section, they were spliced using a "patch plate", as shown in Figure 2.4. The American Railway Engineering Association (AREA) specification (5) did not permit the splicing of two components at the same location, and the American Association of State Highway Officials (AASHO) specification (6) recommended that designers avoid doing this. These requirements were questioned by some designers who believed that there was no valid reason not to splice all of the flange angles at the same location (7).

An early type of web splice was the single-plate or shear splice (8,9,10,11,12). It consisted of two steel plates, one on either side of the web, as shown in Figure 2.5. The flange angles were not spliced at this location. When the bending resistance of the web was neglected in the design of a plate girder, this type of splice was designed to carry only shear. Shear is almost uniformly distributed over the entire web (8). Therefore, the rivets were uniformly spaced and it was considered to be good design practice to use at least two rows of rivets on each side of the joint. The minimum number of fasteners required on one side of the splice was established by dividing the full shear capacity of the web by the allowable working strength of one rivet. Thus, the fastener group on either side of the splice was assumed to be loaded through its center of gravity. When the single-plate splice was designed to carry both the vertical shear and the resisting moment of the web, it was sometimes referred to as a "rational splice" (9). The transverse shear was assumed to be equally distributed among the rivets as a vertical force on each. The horizontal force on each rivet, produced by the bending moment, was assumed to increase proportionally with the distance of the rivet from the neutral axis. The critical rivet therefore was the one that was furthest from the neutral axis. The resultant of the shear and bending force components on each rivet was calculated and compared with the allowable rivet load.

Another type of web splice was the triple-plate, or moment, splice (9,10,11,12). This consisted of three sets of splice plates, as shown in Figure 2.6. Again, the flange angles are continuous at the location of the web splice. The triple-plate splice was designed using the assumption that the middle set of splice plates (shear plates) carried all of the shear and the top and bottom sets of splice plates (moment plates) carried all of the web moment. The shear and moment are actually carried by all three sets of plates, but the error resulting from the use of this assumption is relatively small for deep beams and girders. Shedd (9) recommended that this type of splice be used only for girders with a minimum depth of six feet. The moment plates were designed to carry the entire moment that the web was designed to resist. Prior to about 1950, it was assumed that one-eighth of the web area in a plate girder was effective in resisting moment (8). The stress distribution obtained using conventional beam theory was altered in this way because a thin web is relatively unstable and does not have the capacity to carry its share of the bending stresses at high loads and, as a result, most of the bending stresses are distributed to the flange. Based on the results of more recent experiments, plate girders are currently designed using the assumption that one-sixth of the web area is effective in resisting moment (13).

As far as can be established, the work of Garrelts and Madsen (14) in 1941 is the only experimental study that has been carried out to investigate the behaviour of riveted or bolted web-flange plate girder splices. Four types of web splices were subjected to elastic range and ultimate load tests. Each specimen failed when the compression flange angles buckled near the splice. Although failure did not occur in the splices themselves, the relative behaviour of the four types of splices was observed. Longitudinal stresses in the flange angles and normal and shear stresses in the web and splice plates were calculated using measured strains. From an examination of the stresses calculated from the measured strains, it was concluded that each set of splice plates only carried the stress in the part of the web which they covered. The stress not carried by the splice plates produced an increased stress in the flange angles. The study also showed that the triple-plate splice was more effective than the single-plate splice.

2.3 Current Procedures for Design of Web-Flange Beam Splices

Current North American practice for the design of beam splices has been summarized by Fisher and Struik in "Guide to Design Criteria for Bolted and Riveted Joints" (15). Kulak, Fisher and Struik (16) have written a second edition of this book that includes a new approach to the design of beam splices. This method is reviewed in Chapter 3 of this report.

Fisher and Struik (15) present design methods for two types of beam or girder splices. The web-flange splice, shown in Figure 1.1, is the most common type of beam or girder splice currently used. Usually, splice plates are bolted to both sides of the web and to the outside of the flanges. However, splice plates may be required on both the outside and the inside of the flanges when the flange splices must transfer large forces. The splice plates must be large enough to accomodate the required number of fasteners. To ensure that the splice plates do not fail, their cross-sectional area must be at least equal to the cross-sectional area of the material being spliced. The fasteners in a web-flange splice are loaded either in single shear or double shear, depending on whether splice plates are used on one side or both sides of the material being spliced. A web-flange splice transfers load in such a manner that the fasteners are not subjected to axial forces.

For the design of a bolted web-flange beam or girder splice, Fisher and Struik (15) recommend that the web splice be assumed to transfer all of the shear and that the flange splices be assumed to transfer all of the moment at the section. Because the portion of the moment carried by the web is relatively small, it is ignored in the design of the web splice. The bolt group on one side of the splice is designed on the assumption that the shear force acts at the centroid of the bolt group on the opposite side of the splice. Fisher and Struik recognize that this assumption is more conservative than that used in some other design methods. However, at the time they prepared their recommendations not enough experimental data were available to support a reduced eccentricity. The experimental data obtained by Garrelts and Madsen (14) do not verify the exact distribution of the force in a web splice. In a commonly used British design manual (17), the bolt group on one side of a web splice located at a point of contraflexure also is designed using the assumption that the shear force acts at the centroid of the opposite bolt group. In the design of a flange splice, Fisher and Struik (15) assume that the fasteners in each flange must be able to resist a force equal to the moment at the cross-section divided by the depth of the section.

Ballio and Mazzolani (18) present two alternative approaches for designing web-flange beam or girder splices. In both approaches, the moment at the location of the splice is proportioned between the web splice and the flange splices. Similar to the recommendation of Fisher and Struik, the first approach considers the shear force to act at the centroid of the opposite bolt group. The second approach assumes that the shear force acts at the centerline of the splice. Bresler and Lin (19), Salmon and Johnson (20), and Nethercot (21) also use this second approach, and they further recommend that the web splice be designed to transmit both the eccentric shear force and the portion of the moment that the web was designed to carry. Only Salmon and Johnson (20) provide an explanation for the assumption made in the second approach. Based on the principle commonly used to design connections, they design a bolt group on one side of the splice to resist the internal shear and moment acting at its center of gravity. This yields results that are identical to assuming that the shear force acts at the centerline of the splice.

If a designer chooses to neglect the eccentric effect of the shear force, the web splice is designed simply for the actual shear and moment present at the centerline of the splice. Salmon and Johnson (20) recommend neglecting the effect of the eccentricity except in cases where both the moment and shear are high. Bresler and Lin (19) recommend neglecting the effect of the eccentricity when the eccentricity is much less than the height of the web.

North American and European specifications require that beam or girder splices be capable of developing specified strengths. However, they do not provide insight into how the eccentric effect of the shear force should be accounted for in the design of a web splice or how the moment at the section should be proportioned between the web splice and the flange splices.

The current AISC specifications (22,23) require groove welded beam or girder splices to develop the full strength of the smaller section being spliced, while bolted splices are required to resist the most unfavorable combination of shear and moment at the location of the splice. CSA Standard CAN3-S16.1-M84 Steel Structures for Buildings - Limit States Design (24) requires that connections be designed to resist the maximum factored loads expected to be applied to them. As is the case with most specifications, S16.1 requires that the fasteners in a connection not fail before the members being joined have reached their ultimate capacity. The current AASHTO (formerly AASHO) specification (25) and CSA Standard CAN3 S6-M Design of Highway Bridges (26) require that beam splices be designed to resist a shear and moment equal to the average of the calculated shear and moment at the service loads and the section capacity at the location of the splice. The beam splice must be able to resist at least 75% of the member strengths. The current AREA specification (27) requires that splices be designed to resist the maximum moment and simultaneous shear, or the maximum shear and simultaneous moment.

The current British specification (28) permits beam or girder splices to be designed to resist the actual stresses in the connected members at the location of the splice.

Other European specifications (29,30) require that beam or girder splices be designed to resist the capacity of the connected members. However, these specifications permit exceptions for splices in beams or girders where a failure of the splice would have no detrimental effect (such as progressive collapse) and for splices located in regions of minor load. In such cases, the splices are designed to resist 1.5 times the factored loads if these loads are less than 2/3 of the load carrying capacity of the spliced member (29). The Swiss specification (30) requires that splices be designed to resist a minimum of 50% of the member capacity.

2.4 Analysis of Eccentrically Loaded Bolted Connections

As has already been identified, designers frequently design the bolt group on either side of a web splice for an eccentric load equal to the transverse shear force. Since 1963 considerable research has been carried out to study the behaviour of eccentrically loaded bolted connections in which the load is applied in the same plane as the bolts. The methods used to analyze this type of connection have varied. The two most significant methods that have been used by designers will be presented herein.

Bolt groups subjected to eccentrically applied loads have traditionally been analyzed using a theory which is based on the assumptions that the behaviour of the bolts is elastic and that the bolt group rotates about its center of gravity (22,31). It is assumed that each bolt resists an equal share of the vertical load. The moment caused by the eccentric load produces both a horizontal and a vertical force on each bolt. The magnitude of these forces is directly proportional to the distance of the bolt from the centroid of the bolt group. Therefore, the critical bolt is the one that is furthest from the centroid of the bolt group. This method of analysis results in factors of safety which are both high and inconsistent when compared with test results (32).

Tests have shown that bolts do not have a significant elastic region of shear load versus shear deformation behaviour, and do not have a well-defined shear yield stress (33). Furthermore, except for the case of pure moment, there is no basis for assuming that a bolt group acting under an eccentric load rotates about its center of gravity. Thus, current methods of design for eccentrically loaded bolted connections (13,31,32,34) use an ultimate strength analysis that employs the actual shear load versus shear deformation response of a single bolt and the static equations of equilibrium to predict the ultimate strength of a bolt group. The bolts are assumed to rotate about an instantaneous center. The direction of the force on each bolt is perpendicular to a radius from the instantaneous center of rotation and the deformation of each bolt varies linearly with its distance from the instantaneous center.

The force on each bolt can be expressed as (31,32,34):

$$R = R_u(1-e^{-\mu\Delta})^{\lambda} \tag{2.1}$$

where

- e = base of natural logarithm = 2.718
- R = bolt load at a given deformation
- R_u = ultimate shear strength of a single bolt
- Δ = total deformation including shearing, bending, and bearing deformation of the bolt and local deformation of the connecting material
- λ = regression coefficient
- μ = regression coefficient

It is assumed that when the ultimate load of the connection is reached, the bolt furthest from the instantaneous center has just reached its ultimate deformation. The load corresponding to this deformation is the ultimate shear load that the bolt can sustain (R_u). The deformations of each of the other bolts will be proportional to their radii from the instantaneous center of rotation. Knowing these deformations, the force on each of the other bolts can be obtained from the load versus deformation response (Equation 2.1). The location of the instantaneous center of rotation must be chosen by trial and an iterative procedure used until the three equations of statics are satisfied.

The Canadian Institute of Steel Construction (CISC) Handbook (31) provides tables of ultimate loads for eccentrically loaded bolt groups. These tables were developed using the ultimate strength method described above and the bolt response described by Equation 2.1 using R_u=74 kips, μ=10.0, λ=0.55, and Δ_u=0.34 inches.* In order to present the tables in a non-dimensional format, the connections strengths were then divided by R_u. This load versus deformation response was obtained experimentally by Crawford and Kulak (32) for 3/4 inch diameter ASTM A325 bolts manufactured to minimum strength. The bolts were tested in double shear using a 3/4 inch thick middle plate flanked by a 1/2 inch plate on each side. All plate material was steel with a nominal yield strength of 36 ksi. Experimental and analytical studies have shown also that the coefficients developed in the CISC tables can be conservatively applied to slip-resistant connections (35).

In a bolted connection subjected to an eccentrically applied load that is increasing, the direction of the resulting force on each bolt changes as the instantaneous center of rotation moves. This does not happen in the single bolt shear tests where the direction of both the applied force and the resulting deformation remains constant.

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*Throughout this report, the S.I. dimensional system will generally be used. However, the experimental work to obtain this bolt response was carried out using Imperial units and will be referred to as such.

Crawford and Kulak (32) recognized that the bolts in a multi-bolt connection are not likely to reach the maximum deformation of a single bolt in shear because of this effective rotation. Consequently, the forces that the connection bolts are able to develop are also decreased.

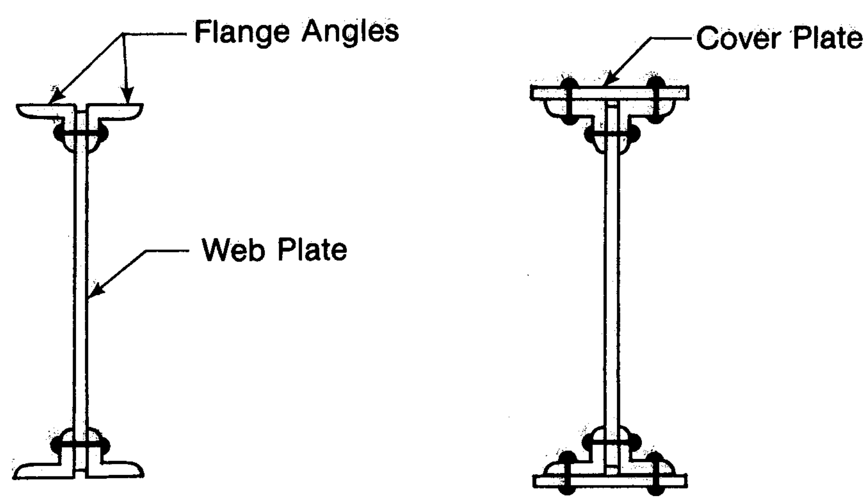


Figure 2.1 Traditional Plate Girder Cross-Section

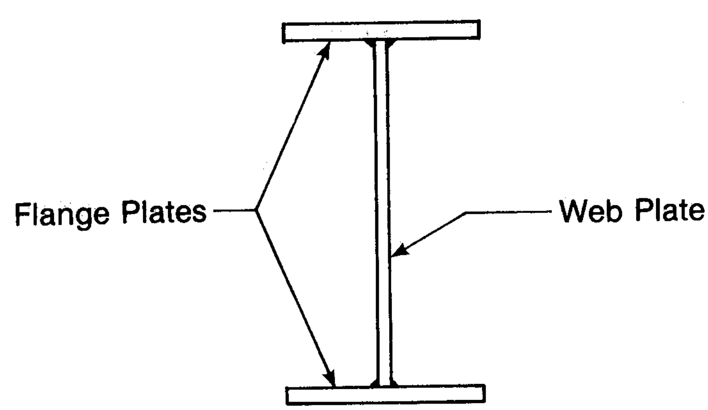


Figure 2.2 Modern Plate Girder Cross-Section

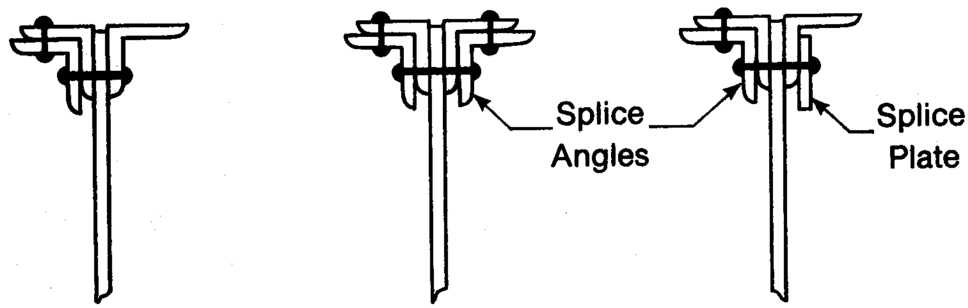


Figure 2.3 Traditional Plate Girder Flange Splices

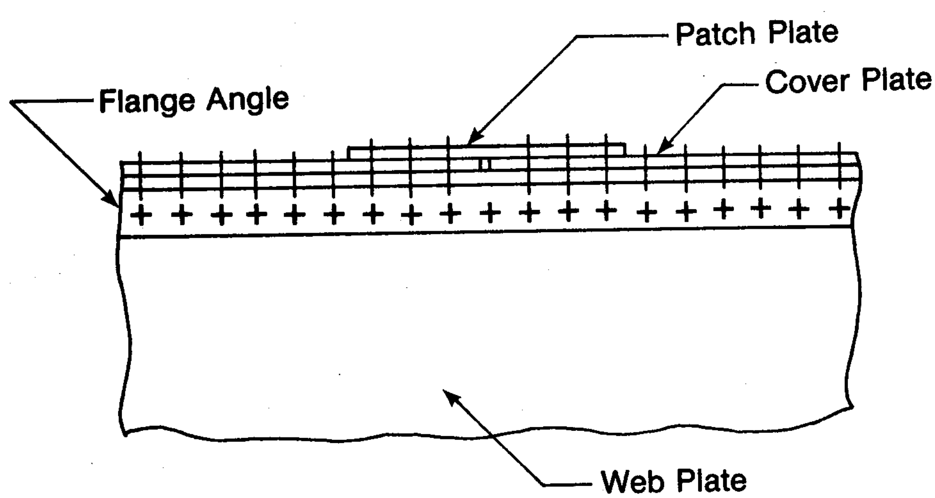


Figure 2.4 Cover Plate Splice

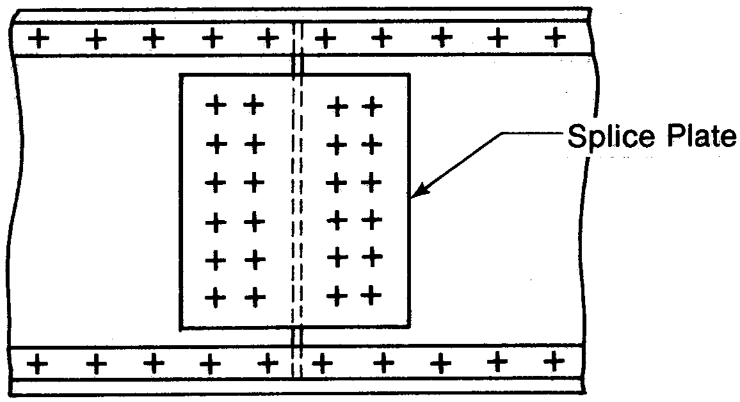


Figure 2.5 Single-Plate Splice

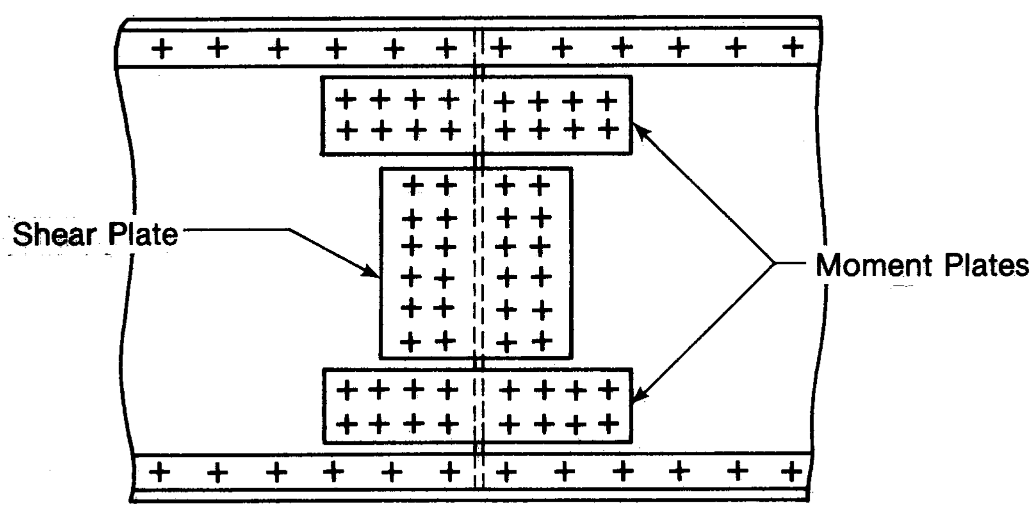


Figure 2.6 Triple-Plate Splice

3. ANALYTICAL STUDY

3.1 Introduction

A theoretical approach to predict the ultimate capacity of a bolted web-flange beam or girder splice has recently been proposed (16). It is a development that is similar to the method currently used to determine the maximum strength of eccentrically loaded bolted connections, that is, it is a rational approach that satisfies the equations of static equilibrium and uses the true shear load versus shear deformation response of the fasteners.

In presenting the proposed analytical method, a general development will be introduced first. This includes the case of a beam or girder that has both the web and the flanges spliced at the same location. The special case of a beam or girder in which only the web is spliced will then be identified. This special case is of interest for three reasons. Firstly, designers frequently splice continuous beams or girders at points of contraflexure (20,25,27). Thus, such splices are designed to transmit only shear. Secondly, for the design of a beam or girder splice located in a region where both shear and moment are present, it is customary to assume that the web splice transfers all of the transverse shear and that the flange splices transfer either all or a specified portion of the moment (13,15,18,19,20). The validity of these assumptions must be examined. Thirdly, the experimental program for this project included the testing of web splices only. The majority of the web splices tested were located at points of contraflexure, although in one case a web splice was tested that was located in a region where both shear and moment were present.

3.2 Proposed Analysis of a Beam or Girder Splice

The validity of the assumptions currently used to design bolted web-flange beam splices will now be examined. Current methods assume that the bolt group on one side of the web splice must be designed to resist all of the transverse shear at the spliced section. This is a reasonable assumption because the transverse stiffness of the flange plates (the only other component that could transfer the shear) is relatively small. In current methods, the bolt group on one side of the web splice is designed to resist a vertical force (equal to the transverse shear at the section) that is considered to act either at the centerline of the splice or at the centroid of the opposite bolt group (13,15,18,19,20). A comparison between the two assumptions for a wide range of web splice bolt arrangements is given in Section 3.3. Neither of these assumptions for the location of the vertical force has been experimentally or analytically verified. One of the methods currently used assumes that the flange splices must carry all of the moment at the location of the splice (13,15). This is a conservative assumption because the web splice also has capacity to transfer some of the moment. However, the converse of this is that the shear capacity of a web splice is reduced when it must carry moment in addition to shear.

Kulak, Fisher and Struik (16) have proposed a new approach to the design of bolted web-flange splices that avoids irrational assumptions. Figure 3.1(a) shows a simple beam that contains a bolted web-flange splice located in a region where both shear and moment are present. A free-body diagram taken by cutting the beam through one set of fasteners is shown in Figure 3.1(b). The forces in these fasteners are assumed to rotate about an instantaneous center, as shown in this figure. The direction of the force on each bolt is perpendicular to a radius from the instantaneous center of rotation, assuming rigid body rotation of the connected parts. By using the assumptions that were presented in Section 2.3 for the ultimate strength method currently used to analyze eccentrically loaded connections, the location of the instantaneous center of rotation is found when the three equations of equilibrium are satisfied, namely:

$$\Sigma F_x = 0 \tag{3.1}$$

$$\Sigma F_y = 0 \tag{3.2}$$

$$\Sigma M_{ic} = 0 \tag{3.3}$$

Equation 3.1 is automatically satisfied because there are no external horizontal forces present. Equation 3.2 is satisfied when the sum of the vertical components of the bolt forces is equal to the shear acting at the section. For the member shown in Figure 3.1, the result of taking the sum of the vertical forces to equal zero is:

$$\frac{Pb}{L} - \sum_{i=1}^{n} R_{iv} = 0 \tag{3.4}$$

The result of taking the sum of the moments about the instantaneous center to equal zero is:

$$\frac{Pb}{L}(x+x_o+r_o) - F_f d - \sum_{i=1}^{n} R_i r_i = 0 \tag{3.5}$$

Equation 3.5 can be rewritten as:

$$\frac{Pbx}{L} + \frac{Pb}{L}(x_o+ r_o) - F_f d - \sum_{i=1}^{n} R_i r_i = 0 \tag{3.6}$$

where:

- d = distance between the centroids of the top and bottom flanges
- F_f = force in the top or bottom flange bolts on one side of the splice
- n = number of bolts on one side of the web splice
- r_i = distance from the i^{th} bolt to the instantaneous center of rotation

r_o = distance from the centroid of one bolt group to its instantaneous center of rotation

R_i = force in the i^{th} bolt

R_{iv} = vertical component of the bolt force R_i

x_o = distance from the centerline of the splice to the centroid of the bolt group on one side of the splice

Although this development started with a single concentrated load acting on a simply-supported beam, it can be shown that the foregoing statements are generally true for any loading case. Thus, in order to present these equations in a more general form, Pb/L will be replaced by V, the shear at the section, and Pbx/L will be replaced by M, the moment at the centerline of the splice. Using this notation, the equilibrium equations become:

$$\sum_{i=1}^{n} R_{iv} - V = 0 \qquad (3.7)$$

$$\sum_{i=1}^{n} R_i r_i + F_f d - [M + V(x_o + r_o)] = 0 \qquad (3.8)$$

Equation 3.7 identifies that it is the vertical components of the bolt forces that resist the transverse shear at the section and Equation 3.8 identifies how the moment transferred across the splice is shared between the bolts in the web splice and the bolts in the flange splices.

Although Kulak, *et al.* (16) recognized that it would be advantageous from a designer's point of view if a location of the eccentric shear force could be established that would yield results identical to those given by the equilibrium equations, they found no particular relationship between the eccentricity of the the shear force and the center of gravity of either bolt group. However, these equations have been further examined herein. The sum of the first two terms in Equation 3.8, $\sum_{i=1}^{n} R_i r_i$ and $F_f d$, represents the moment at the instantaneous center of rotation created by the resistance of the bolts. The other term in Equation 3.8, $M + V(x_o+r_o)$, represents the moment at the instantaneous center of rotation produced by the external force applied to the beam as shown in the free-body diagram (Figure 3.1(b)). Kulak, *et al.* (1) express this term more simply as M, the moment at the instantaneous center of rotation. However, it is more convenient from a designer's point of view to use the expanded form in Equation 3.8, with M and V defined as the moment and shear at the centerline of the splice.

If the moment at the location of the splice is equal to zero, then the moment at the instantaneous center can only be a result of the shear at the section. From this it can be concluded that the term $V(x_o+r_o)$ is the moment at the instantaneous center produced by the eccentric shear force. This solution to the problem is identical to that for a bolt group loaded eccentrically by a force "V" located at a distance "x_o" from its center of gravity. Thus, designers who are familiar with the treatment of eccentrically loaded connections will find it convenient to deal with the problem in those terms. In summary, for this important case of $M = 0$, $V \neq 0$, the design of the bolt group in a web splice can proceed on the same basis as that for a bolt group acting under a load equal to the shear at the splice and located at the centerline of the splice. Figure 3.2 shows this pictorially for a general group of bolts. The distance between the shear force acting at the centerline of the splice and the centroid of the bolt group, called x_o in the development so far (see Fig. 3.1), is usually called "e" by designers (20). The latter notation is used in Figure 3.2. Equation 3.8 is then rewritten as:

$$\sum_{i=1}^{n} R_i r_i + F_f d - [M + V(e+r_o)] = 0 \qquad (3.9)$$

For the beam loaded as shown in Figure 3.1, the bending moment is not symmetric about the centerline of the splice. Consequently, the forces acting on the bolts on one side of the splice are not identical to those acting on the bolts on the opposite side. In this example, the right-hand bolt group is critical because it is on the side of the splice with the greater moment present. As previously stated, $M + V(e+r_o)$ represents the moment at the location of the instantaneous center and it is this moment that must be resisted by the bolts on the right-hand side of the splice. For the right-hand bolt group, M and $V(e+r_o)$ are both acting in the same direction. However, for the left-hand bolt group M and $V(e+r_o)$ are acting in opposite directions to one another. It is more economical to design the splice to be symmetric about the joint. Hence, both bolt groups are designed to resist the forces to which the critical bolt group (the right-hand bolt group in this example) is subjected.

Equations 3.7 and 3.9 are general and can be applied to bolted web-flange splices in both simple and continuous beams. For the special case of a beam in which only the web is spliced but wherein both shear and moment are present, there are no forces transferred across the flanges; hence, $F_f=0$. Substituting $F_f=0$ into Equations 3.7 and 3.9 yields results identical to designing the bolt group on one side of the web splice to resist the shear and moment at the centerline of the splice applied to the bolt group as shown in Figure 3.3. The ultimate capacity of the bolt group can then be determined using the ultimate strength method for analyzing eccentrically loaded connections that was presented in Section 2.3.

For the case of a beam or girder in which both the web and the flanges are spliced, the designer will have to make an assumption regarding the portion of the moment at the location of the splice that the flange splices will be designed to resist. This assumption is then used in Equation 3.9 to identify how the moment is shared between the web splice and the flange splices. There are two approaches that have traditionally been used by designers to proportion the moment at the section. Either one of these may be used as long as the equilibrium statements (Eqs. 3.7 and 3.9) are satisfied.

One approach is to design the flange splices to resist 100% of the moment at the centerline of the splice (15). This leads to $F_f d = M$ in Equation 3.9, and therefore the web splice is designed to resist only the eccentric shear force.

The alternative approach is to design the web splice to resist a portion of the total moment at the section in addition to the transverse shear force (18,19,20). In this case, the flange splices are designed to resist less than 100% of the moment, that is, $F_f d < M$. The moment not resisted by the web splice must be carried by the flange splices. The moment that the web splice must resist is equal to the portion of the moment that the web in the beam or girder was designed to resist. For a beam, this moment can be calculated using conventional beam theory. For the design of a plate girder, it is customary to assume that one-sixth of the web area (adjacent to the flange) is effective in resisting moment because the thin web is relatively unstable (13). The moment carried by the plate girder web can be calculated using this theory.

The method of analysis presented in this chapter was used to predict the ultimate strength of six large scale web splice test specimens. Each specimen contained a bolted web splice that was located in a continuous beam. For those splices that were located at a point of contraflexure, the theoretical ultimate capacities of the bolt groups on either side of the joint were identical. For the splice that was located in a region where both shear and moment were present, the theoretical ultimate capacity of the bolt group on one side of the joint was higher than that on the other side of the joint. The test results are presented and compared with predicted shear capacities in Chapter 5.

3.3 Comparison of Web Splice Design Assumptions

As has already been identified, the bolt group on one side of a web splice currently is designed to resist a vertical force (equal to the transverse shear at the location of the splice) that is considered to act either at the centerline of the splice or at the centroid of the opposite bolt group. In Tables 3.1 to 3.7, non-dimensional coefficients are presented for a wide range of web splice bolt arrangements. The number of vertical lines of bolts on either side of the splice varies from one to four, the number of bolts in each line varies from two to twelve, and the bolt pitch varies from 80 mm to 160 mm. Except for the case of only one line of bolts on either side of the splice, the distance of the bolts from the centerline of the splice also varies.

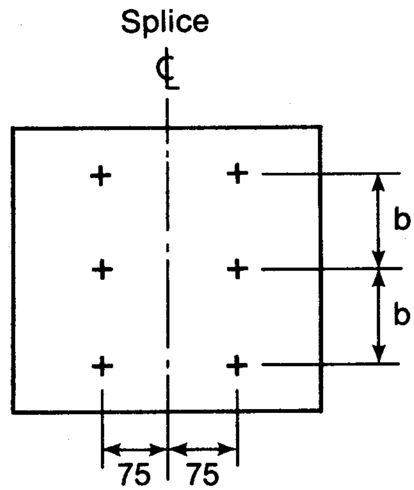
Using the magnitude of the eccentricity of the shear force corresponding to each of the assumptions for web splice design presented above, the non-dimensional coefficients have been selected from tables provided in the CISC Handbook (31) for the analysis of eccentrically loaded bolted connections. The shear strength of a web splice is calculated by multiplying the corresponding coefficient by the shear strength of one bolt. Hence, the coefficients are directly proportional to the connection strengths. The web splice bolt arrangements can therefore be compared on the basis of the values of the non-dimensional coefficients.

In Tables 3.1 to 3.7, the ratio of the predicted shear strength of a web splice based on the assumption that the shear force acts at the centerline of the splice to the predicted shear strength of the connection based on the assumption that the shear force acts at the centroid of the opposite bolt group is given for each bolt arrangement. Because the latter assumption is conservative compared with the former assumption, these ratios are all greater than 1.0. Although the comparisons contained within these tables constitute a large sample of possible splice arrangements, they do not consider all possibilities. Thus, only the trends can be commented upon.

As could be anticipated for connections that contain a relatively large number of bolts in a vertical line and have a relatively small distance between the centroids of the bolt groups on either side of the web splice, the ratio Cp/Cc is close to 1.0. The cases in Tables 3.1 and 3.2 with 12 bolts in a line provide examples of this. For these cases both methods of analysis yield similar results because the predicted shear strengths are close to the shear strengths that would result if the eccentric effect of the shear force were ignored. However, for other cases tabulated in Tables 3.1 and 3.2 (that is, with fewer than 12 bolts in a vertical line), the ratio can be significantly greater than unity. Similar inferences can be drawn from comparisons made using Tables 3.3 to 3.7.

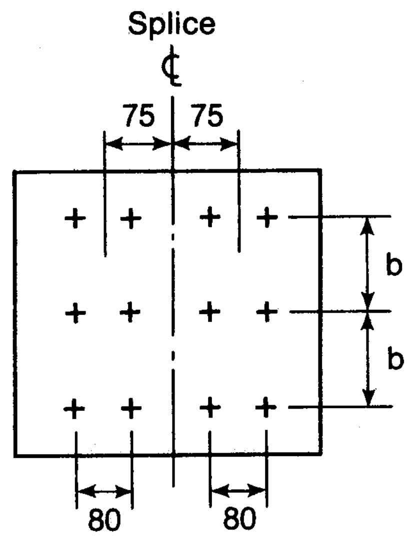
By examining the entire range of data contained within Tables 3.1 to 3.7, it is seen that the ratio Cp/Cc varies from a low of 1.03 to a high of 1.84. The mean of all the tabulated values is 1.39. To the extent that the data contained in these tables are representative, it indicates that a considerable saving could be made in the design of a web splice using the assumption that the shear force acts at the centerline of the splice rather than at the centroid of the opposite bolt group. In Chapter 5, the web splice design assumptions presented herein are further compared on the basis of experimental results.

Table 3.1 Web Splice Shear Capacity Coefficients



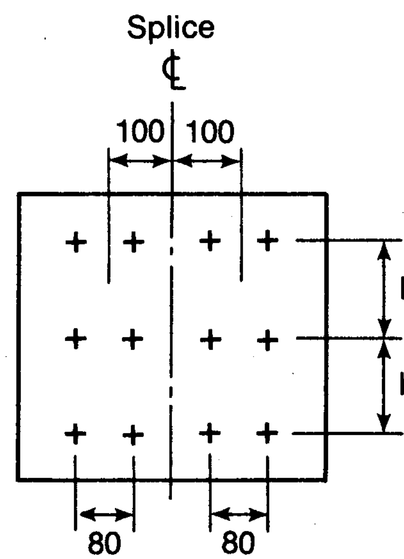
Pitch b mm	Number of Bolts in Each Row	Coefficient Using Proposed Assumption (Cp)	Coefficient Using Conventional Assumption (Cc)	$\frac{Cp}{Cc}$
80	2	0.94	0.51	1.84
	3	1.86	1.24	1.50
	4	2.95	2.15	1.37
	6	5.17	4.21	1.23
	9	8.40	7.51	1.12
	12	11.5	10.8	1.06
120	2	1.25	0.74	1.69
	3	2.32	1.50	1.55
	4	3.44	2.51	1.37
	6	5.60	4.68	1.20
	9	8.72	7.98	1.09
	12	11.8	11.2	1.05
160	2	1.45	0.94	1.54
	3	2.57	1.86	1.38
	4	3.66	2.95	1.24
	6	5.77	5.17	1.12
	9	8.84	8.40	1.05
	12	11.9	11.5	1.03

Table 3.2 Web Splice Shear Capacity Coefficients



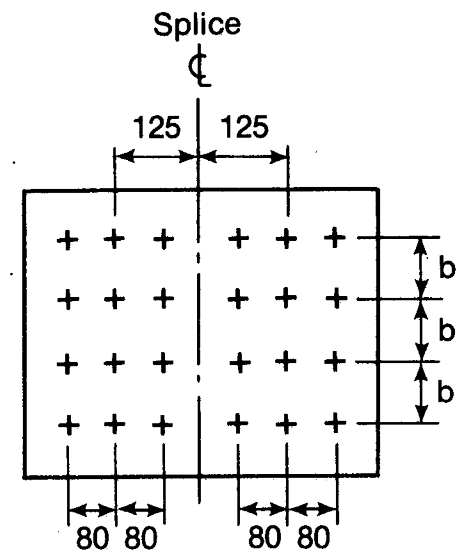
Pitch b mm	Number of Bolts in Each Row	Coefficient Using Proposed Assumption (Cp)	Coefficient Using Conventional Assumption (Cc)	Cp/Cc
80	2	2.15	1.28	1.68
	4	5.89	3.90	1.51
	6	10.2	7.57	1.35
	8	14.6	11.8	1.24
	10	18.8	16.2	1.16
	12	23.0	20.6	1.12
120	2	2.57	1.63	1.58
	4	6.84	5.04	1.36
	6	11.1	9.35	1.19
	8	15.3	13.7	1.12
	10	19.5	18.1	1.09
	12	23.5	22.3	1.05
160	2	2.89	1.95	1.48
	4	7.29	5.89	1.24
	6	11.5	10.3	1.12
	8	15.6	14.6	1.07
	10	19.7	18.9	1.04
	12	23.7	23.0	1.03

Table 3.3 Web Splice Shear Capacity Coefficients



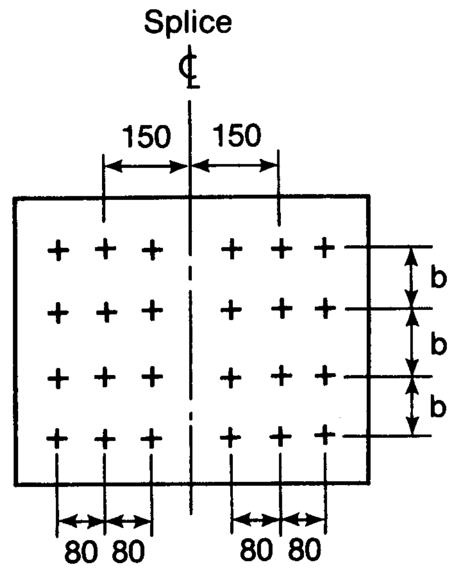
Pitch b mm	Number of Bolts in Each Row	Coefficient Using Proposed Assumption (Cp)	Coefficient Using Conventional Assumption (Cc)	Cp / Cc
80	2	1.76	1.03	1.71
	4	5.09	3.14	1.62
	6	9.34	6.22	1.50
	8	13.7	10.1	1.36
	10	18.1	14.3	1.27
	12	22.3	18.6	1.20
120	2	2.15	1.28	1.68
	4	6.21	4.15	1.50
	6	10.6	8.11	1.31
	8	14.9	12.4	1.20
	10	19.1	16.9	1.13
	12	23.2	21.2	1.09
160	2	2.51	1.58	1.59
	4	6.85	5.03	1.36
	6	11.2	9.35	1.20
	8	15.3	13.7	1.12
	10	19.5	18.1	1.08
	12	23.5	22.3	1.05

Table 3.4 Web Splice Shear Capacity Coefficients



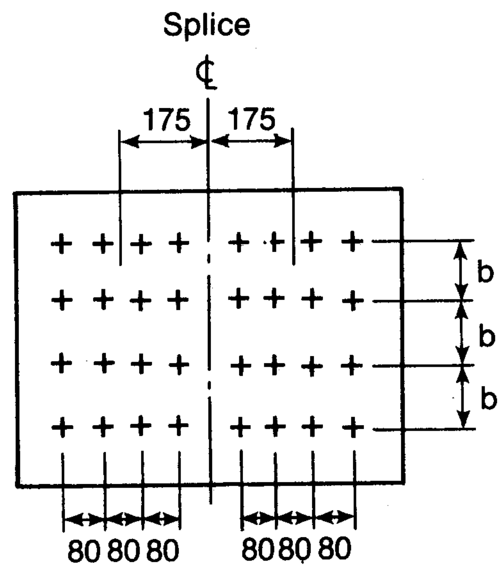
Pitch b mm	Number of Bolts in Each Row	Coefficient Using Proposed Assumption (Cp)	Coefficient Using Conventional Assumption (Cc)	Cp/Cc
80	2	2.63	1.54	1.71
	4	7.00	4.26	1.64
	6	12.6	8.19	1.54
	8	19.0	13.2	1.44
	10	25.6	18.9	1.35
	12	32.1	25.1	1.28
120	2	3.06	1.83	1.67
	4	8.48	5.46	1.55
	6	14.9	10.7	1.39
	8	21.4	16.8	1.27
	10	27.8	23.3	1.19
	12	34.1	29.9	1.14
160	2	3.45	2.11	1.64
	4	9.54	6.61	1.44
	6	16.1	12.6	1.28
	8	22.5	19.1	1.18
	10	28.7	25.7	1.12
	12	34.9	32.2	1.08

Table 3.5 Web Splice Shear Capacity Coefficients



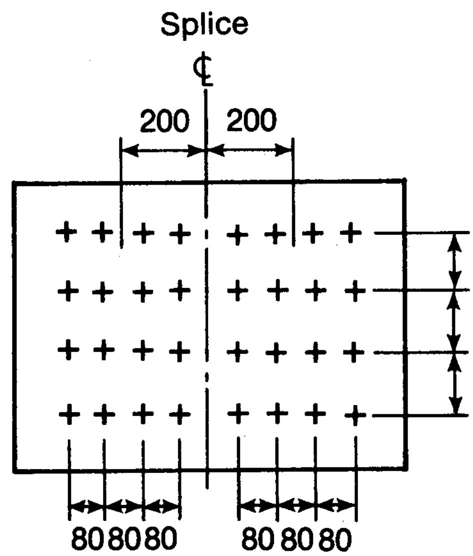
Pitch b mm	Number of Bolts in Each Row	Coefficient Using Proposed Assumption (Cp)	Coefficient Using Conventional Assumption (Cc)	$\frac{Cp}{Cc}$
80	2	2.30	1.30	1.77
	4	6.22	3.70	1.68
	6	11.5	7.13	1.61
	8	17.7	11.5	1.54
	10	24.2	16.8	1.44
	12	30.8	22.6	1.36
120	2	2.70	1.56	1.73
	4	6.22	4.76	1.61
	6	11.5	9.41	1.49
	8	17.7	15.1	1.36
	10	24.2	21.4	1.26
	12	30.8	27.9	1.20
160	2	3.07	1.83	1.68
	4	8.82	5.76	1.53
	6	15.4	11.3	1.36
	8	21.9	17.7	1.24
	10	28.2	24.3	1.16
	12	34.5	30.9	1.12

Table 3.6 Web Splice Shear Capacity Coefficients

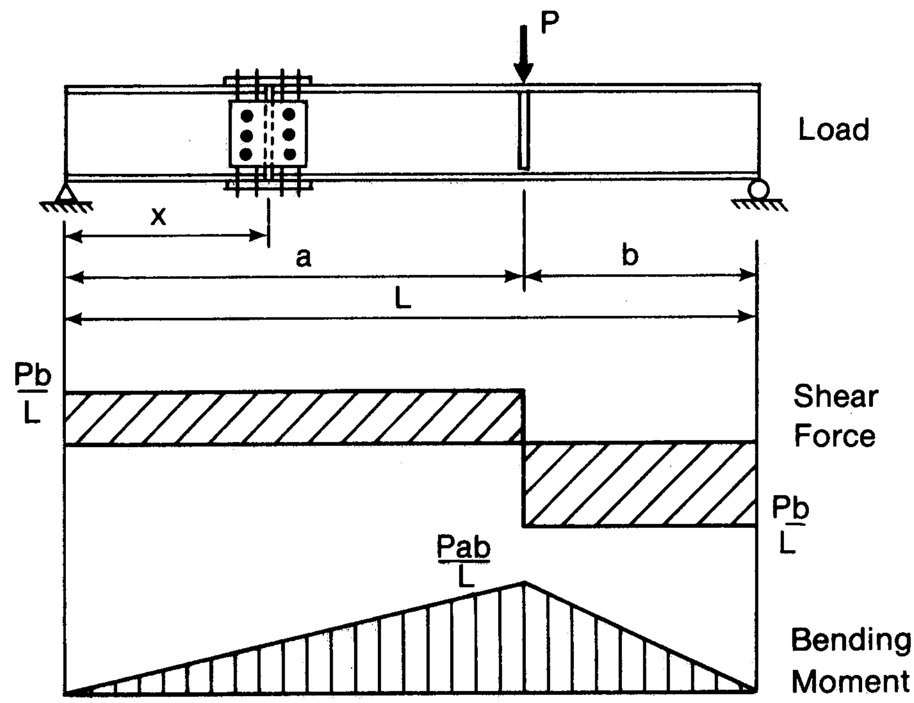


Pitch b mm	Number of Bolts in Each Row	Coefficient Using Proposed Assumption (Cp)	Coefficient Using Conventional Assumption (Cc)	Cp/Cc
80	2	3.20	1.86	1.72
	4	8.05	4.83	1.67
	6	14.4	8.97	1.61
	8	22.1	14.3	1.55
	10	30.4	20.5	1.48
	12	39.0	27.6	1.41
120	2	3.53	2.11	1.67
	4	9.57	5.98	1.60
	6	17.4	11.6	1.50
	8	26.0	18.5	1.41
	10	34.7	26.3	1.32
	12	43.4	34.8	1.25
160	2	3.96	2.38	1.66
	4	11.0	7.14	1.54
	6	19.5	13.9	1.40
	8	28.3	21.9	1.29
	10	36.8	30.4	1.21
	12	45.3	39.2	1.16

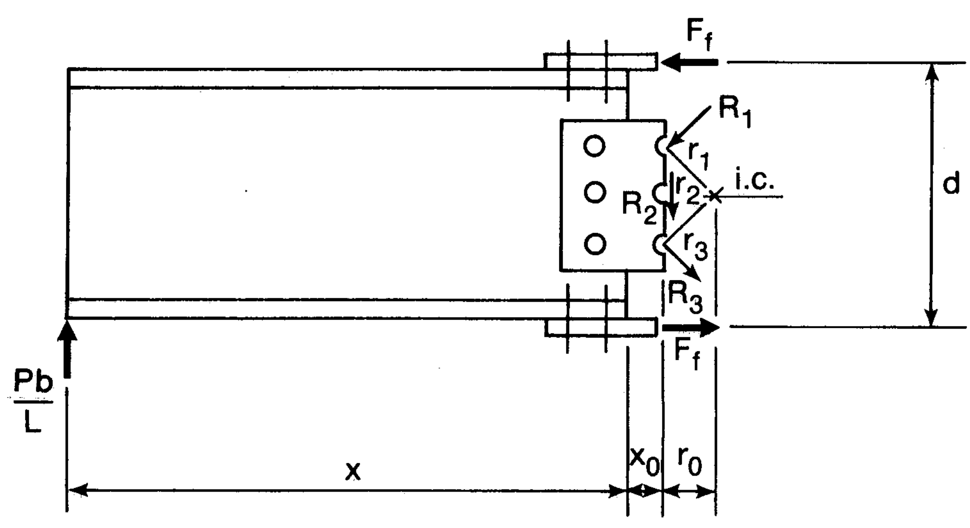
Table 3.7 Web Splice Shear Capacity Coefficients



Pitch b mm	Number of Bolts in Each Row	Coefficient Using Proposed Assumption (Cp)	Coefficient Using Conventional Assumption (Cc)	Cp/Cc
80	2	2.89	1.62	1.78
	4	7.33	4.26	1.72
	6	13.3	7.94	1.68
	8	20.6	12.7	1.62
	10	28.6	18.3	1.56
	12	37.1	25.0	1.48
120	2	3.25	1.84	1.77
	4	8.88	5.29	1.68
	6	16.3	10.3	1.58
	8	24.8	16.7	1.49
	10	33.5	24.1	1.39
	12	42.2	32.3	1.31
160	2	3.60	2.10	1.71
	4	10.3	6.35	1.62
	6	18.6	12.5	1.49
	8	27.3	20.1	1.36
	10	36.0	28.5	1.26
	12	44.5	37.3	1.19



(a)



(b)

Figure 3.1 Analytical Model for a Web-Flange Splice

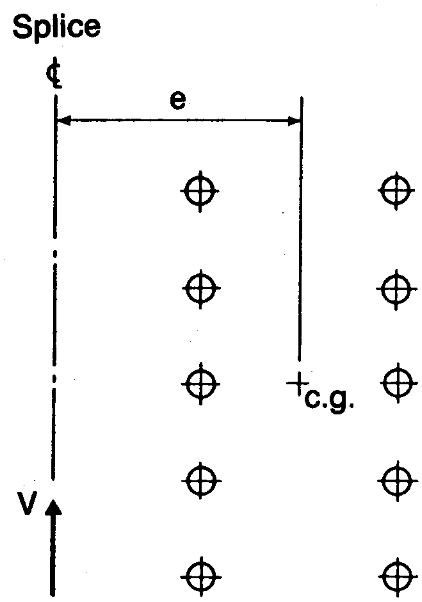


Figure 3.2 Eccentrically Loaded Web Splice Bolt Group

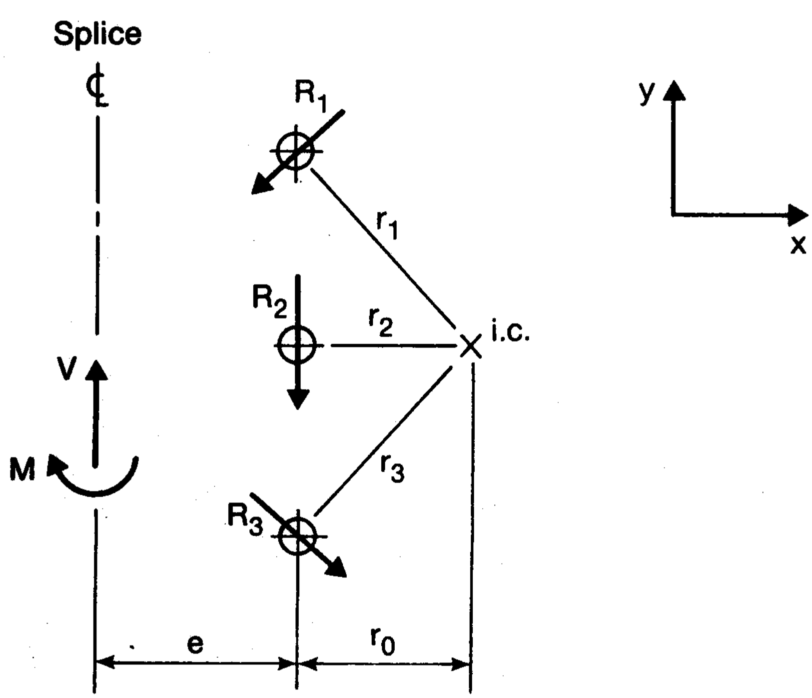


Figure 3.3 Web Splice Bolt Group Design Forces

4. EXPERIMENTAL PROGRAM

4.1 Introduction

The purpose of the experimental program was to substantiate the analytical procedure presented in Chapter 3. Large scale tests were carried out in order to determine the ultimate capacity of six bolted web splices in which the number and arrangement of bolts varied. Two series of ancillary tests were carried out in order to establish the shear load versus shear deformation response of the bolts. This response was used in the analytical procedure to predict the ultimate capacities of the web splice test specimens.

4.2 Single Bolt Shear Tests

4.2.1 Specimen Description

In this project, two series of single bolt shear specimens were tested, using both compression and tension jigs, to establish the load versus deformation response of single bolts loaded in double shear. A total of fourteen specimens were tested, eight belonging to series A and six belonging to series B. The specimens in each series were detailed to conform as closely as possible to the details in each of the full scale test configurations. Single bolt test series A corresponded to conditions in full scale test specimens C1 through C4 and single bolt test series B corresponded to conditions in full scale test specimens C5 and C6.

Each single bolt shear compression jig specimen was prepared by match-drilling a hole, using a 3/4 inch* drill bit, through three 100 mm by 100 mm steel plates. In all cases, the steel plates were cut from the same material that was used to fabricate the bolted web splice test specimens described subsequently in Section 4.3.1. Each plate had one milled edge in order to provide a smooth loading surface. The test bolts were 3/4 inch diameter ASTM A325 bolts, 90 mm in length, of which 35 mm was threaded. Any possible variation in the bolt properties was minimized by using bolts that were all from the same production lot.

Each single bolt shear tension jig specimen was prepared in a similar manner to the compression jig specimens. Although the length of the plates used in these specimens varied from that used in the compression jig specimens, the width and the thickness of the plates were identical. The test bolts were from the same lot as those used in the compression jig specimens. The outside plates lapped the middle plates, and these plates were bolted together so that only the test bolt was critical.

The compression jig specimens were assembled as shown in Figure 4.1 and the tension jig specimens were assembled as shown in Figure 4.2. The thickness of the middle plate was 19 mm in all cases, corresponding to the thickness of

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*Bolts and drill bits are still manufactured according to the Imperial system and will be referred to as such.

the beam web in the full scale test specimens. Single bolt shear specimens A1-C through A5-C and A1-T through A3-T were identical within the group. The thickness of the outside plates was 19 mm, corresponding to the thickness of the splice plates in web splice specimens C1 through C4. The bolts in series A compression jig specimens were tightened by turning the nut one-half turn past the snug position. However, the bolts in series A tension jig specimens were only tightened to the snug position. Single bolt shear specimens B1-C through B3-C and B1-T through B3-T were likewise identical within the group. (The identifier "C" or "T" refers to the type of jig used to test the bolt.) The thickness of the outside plates was 13 mm, corresponding to the thickness of the splice plates in web splice specimens C5 and C6. The bolts in these specimens were tightened to the snug position.

4.2.2 Test Set-Up and Procedure

The single bolt shear compression jig specimens were tested using an Amsler 400 kN capacity compression testing machine and the single bolt shear tension jig specimens were tested using a Materials Testing System (MTS) 1000 kN capacity testing machine. Each specimen was subjected to an initial load to ensure that the bolt was bearing against the steel. That load was then removed and the test begun. The test load was initially applied in increments of approximately 25 kN. The load increments were decreased as the maximum load was approached.

In order to measure the bolt deformations, dial gauges were placed on either side of the specimen. The bolt deformation was taken as the average of these two readings. The mounts for the gauges and the reaction plates for the dial gauge stems were all attached in line with the longitudinal axis of the test bolt. In this way only the shearing, bending, and bearing deformations of the test bolt and local bearing deformations of the plate adjacent to the test bolt were measured. As a check, a third dial gauge was used to measure the movement of the Amsler loading head. The movement of the MTS loading head was measured by the machine itself. Dial gauge readings were taken at each load step.

4.3 Web Splice Tests

4.3.1 Specimen Description

The web splice test specimens were constructed by joining two steel beams together using two steel plates, one on either side of the web. These plates were lapped across the joint and bolted to the beam webs. The same beams were used in all six tests. As will be described subsequently, new holes were drilled in the beam web for each different test. These holes were either in new material or sufficiently far removed from the holes of a previous test. Each beam was initially 2.5 m in length and had been fabricated by welding three plates together (two flange plates and a web plate). The dimensions of the beams were chosen so that the beams would not yield before the bolted web splice reached its ultimate capacity. The steel in the beams was required to meet CSA Specification G40.21-M 300W (36).

The cross-section of the beams is shown in Figure 4.3. Web stiffeners were welded to the beams at the load and reaction points in order to prevent the web from crippling.

Details of the geometry of the bolted web splices are provided in Table 1. Figure 4.4 shows a bolted web splice located in one of the full scale test specimens (C2) before it was tested. In all of the full scale tests, the connections used 3/4 inch diameter ASTM A325 bolts from the same lot as those used in the single bolt shear specimens. The bolt holes were drilled with the beams lying flat on the floor, that is, with the web in a horizontal position. The holes were match-drilled through both splice plates and the web. A 3/4 inch drill bit was used to ensure that all of the bolts would be bearing against the web and the splice plates as soon as a load was applied.

The initial slippage was minimized by the small clearance of the bolt holes. This ensured that each bolt was carrying its portion of the load immediately. Although this represents an idealized condition, it prevents another variable (slippage) from being introduced into the experiment. It is felt that as most standard connections approach their ultimate capacities, all of the bolts are bearing against the steel (32). Thus, the idealized condition approximates the behaviour of the bolts in a real connection as the connection reaches its ultimate capacity.

The bolts were inserted into the holes and the nuts tightened to the snug position. A washer was present under the nut of each bolt. It was necessary to place an additional washer under the head of the bolts in specimens C5 and C6 because of the thinner splice plates used.

As shown in Figure 4.3, the thickness of the beam webs was 19 mm in all cases. The thickness of the splice plates used in specimens C1 through C4 was also 19 mm. These splice plates were cut from the same plate that was used to make the beam webs. The thickness of the splice plates used in specimens C5 and C6 was 13 mm. The splice plate dimensions were 300 mm by 300 mm for specimens C1 through C4. However, the splice plate size was increased to 350 mm by 300 mm for specimen C5 and to 390 mm by 320 mm for specimen C6. The dimensions chosen were simply a reflection of the bolt patterns used. Prior to testing, the splice plates were white-washed in order that any yielding of the steel could be observed.

After specimen C1 was tested, the beam ends were reversed so that the unused ends were at the splice location. These ends were used to construct both specimens C2 and C3. The bolt holes for specimen C3 were located so that the holes from specimen C2 as already tested would not interfere. The evaluation of where the bolt holes could be located in cases like this included the predicted direction of the forces on each bolt in order that the web material would not fail where the bolt pushed against it. After specimen C3 was tested, 70 mm was cut off each end of both steel beams. This left the beams without any of the bolt holes in the webs from the first three tests. The length of the spans between the load and reaction points remained at 1000 mm. However, the length of the spans between the reaction points and the centerline of the splice was reduced from 1000 mm to 930 mm. After specimen C4 was tested, the beams were again reversed. Specimens C5 and C6 were constructed using the same beam ends. The bolt holes for specimen C6 were positioned so that the holes which remained from specimen C5 would not influence the test of specimen C6.

4.3.2 Test Set-Up and Instrumentation

The set-up used to test the bolted web splice specimens is shown in Figures 4.5 and 4.6. The load frame contained a distributing beam that was approximately 3 m above the floor. Two independently controlled 1800 kN capacity hydraulic jacks were used to apply loads to the specimen, as shown in Figure 4.7. Each beam was supported at one location. The east jack and the west support reacted against the distributing beam above the specimen, while the west jack and the east support reacted against the laboratory floor.

The load and reaction points were simply supported on steel knife edges to allow the beams to rotate about their transverse axes. The loads applied to the specimen were measured using 1300 kN and 2200 kN capacity electronic load cells placed at the load and reaction points. The accuracy of the load measurement is considered to be within 1%. The west reaction was fixed against translation, but steel rollers were placed at both load points and at the east reaction point to allow horizontal translation in order that axial forces not be induced into the beams.

It was very undesirable for the ends of the beams to touch each other at any time during the tests because the tests were designed so that only the bolted web splice transferred the forces across the joint. A 4 mm gap was left between the beams in specimens C1 through C5. When the first specimen (C1) was tested, the bottom flanges began to touch each other at the splice before the ultimate load was reached. Therefore, in testing specimen C1 a narrow strip of the bottom flange along with a portion of the adjacent web was cut off with a torch to widen the gap. In subsequent tests, this was done before the specimens were tested. A larger gap, 15 mm, was left between the beams in specimen C6 because the expected moment at the location of the splice was significantly greater than zero.

Electrical resistance strain gauges were placed in pairs along the top and bottom flanges of the beams, above and below the web, to monitor the bending moment behaviour of the specimens. They were also placed along the theoretical neutral axis of the web near the splice, except in specimen C1 which had no gauges on the web. All of the strain gauges were orientated so as to measure strain in the longitudinal direction. Displacement transducers (LVDT's) were positioned at the two load points and above the centroid of each bolt group in order to measure the vertical deflections of the beams. Eight LVDT's were used to monitor the horizontal and vertical movements of the upper west and lower east corners of both splice plates relative to the beam webs. The LVDT's were positioned approximately 20 mm from the edge of each splice plate (see Figure 4.4).

4.3.3 Test Procedure

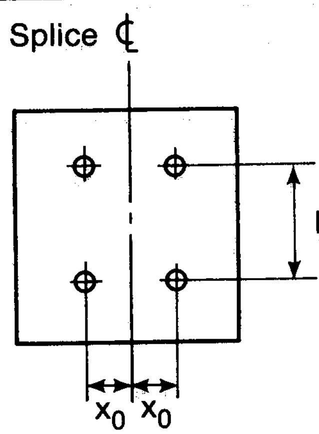
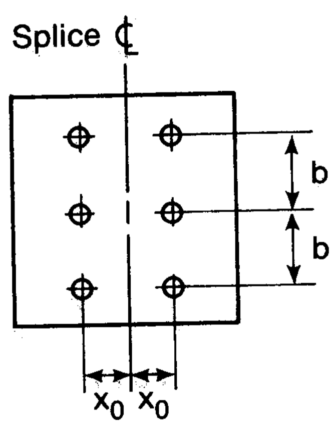
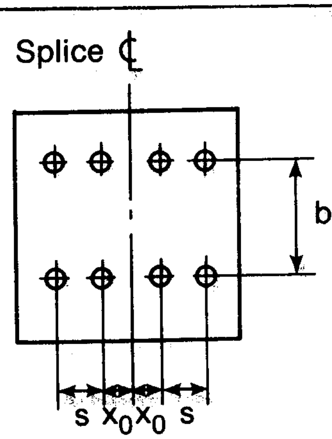
For specimens C1 through C5, the east and west jacks were used to apply equal loads so that the centerline of each splice was at the theoretical location of zero moment. The resulting load, shear force and bending moment diagrams for specimens C1, C2 and C3 are shown in Figure 4.8. Theoretically, the shear at the splice was equal to the applied load, P. (In analyzing the test results, measured loads rather than theoretical loads will be used.) The load diagrams for specimens C4 and C5 are identical to that shown in Figure 4.8 except that the center span lengths have been reduced from 1000 mm to 930 mm, thus increasing the reactions to 2.07P. Theoretically, the shear at the location of the splice was 7% higher than the applied loads.

The east and west jacks were used to apply unequal loads, 2P and P, respectively, to specimen C6. The load applied to the east end of this specimen was twice as great as that applied to the west end, resulting in a moment at the location of the splice that was much greater than zero. The load, shear force and bending moment diagrams for specimen C6 are shown in Figure 4.9.

Initially, the specimens were loaded in increments of approximately 25 kN. The load increments were decreased in magnitude as the ultimate load of each connection was approached. Load cell, strain gauge and transducer readings were electronically obtained and recorded at each load step. Testing was stopped when the ultimate load was reached.

When testing had been completed, the bolts and splice plates were removed from the specimens. The angles of deformation of the bolt holes in the web were measured with respect to the horizontal axis. The length of each bolt hole, in the direction in which it had been deformed, was also measured.

Table 4.1 Details of Test Specimens

Specimen	Bolt Group	x_o mm	b mm	s mm
C1		32	100	-
C2		32	80	-
C3		50	220	-
C4		50	90	-
C5		50	120	60
C6		70	220	70

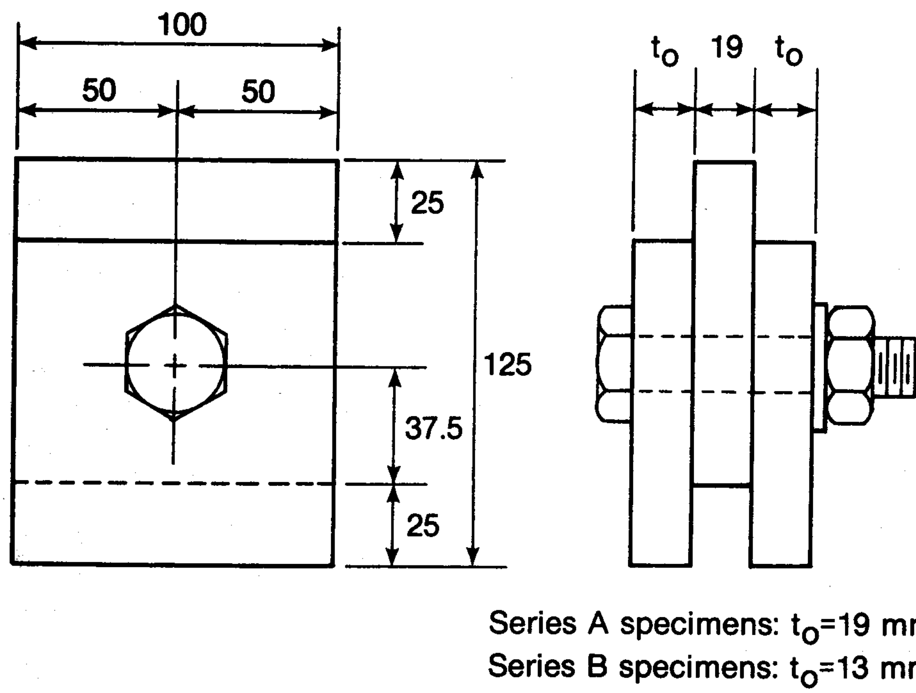


Figure 4.1 Single Bolt Shear Specimen - Compression Jig

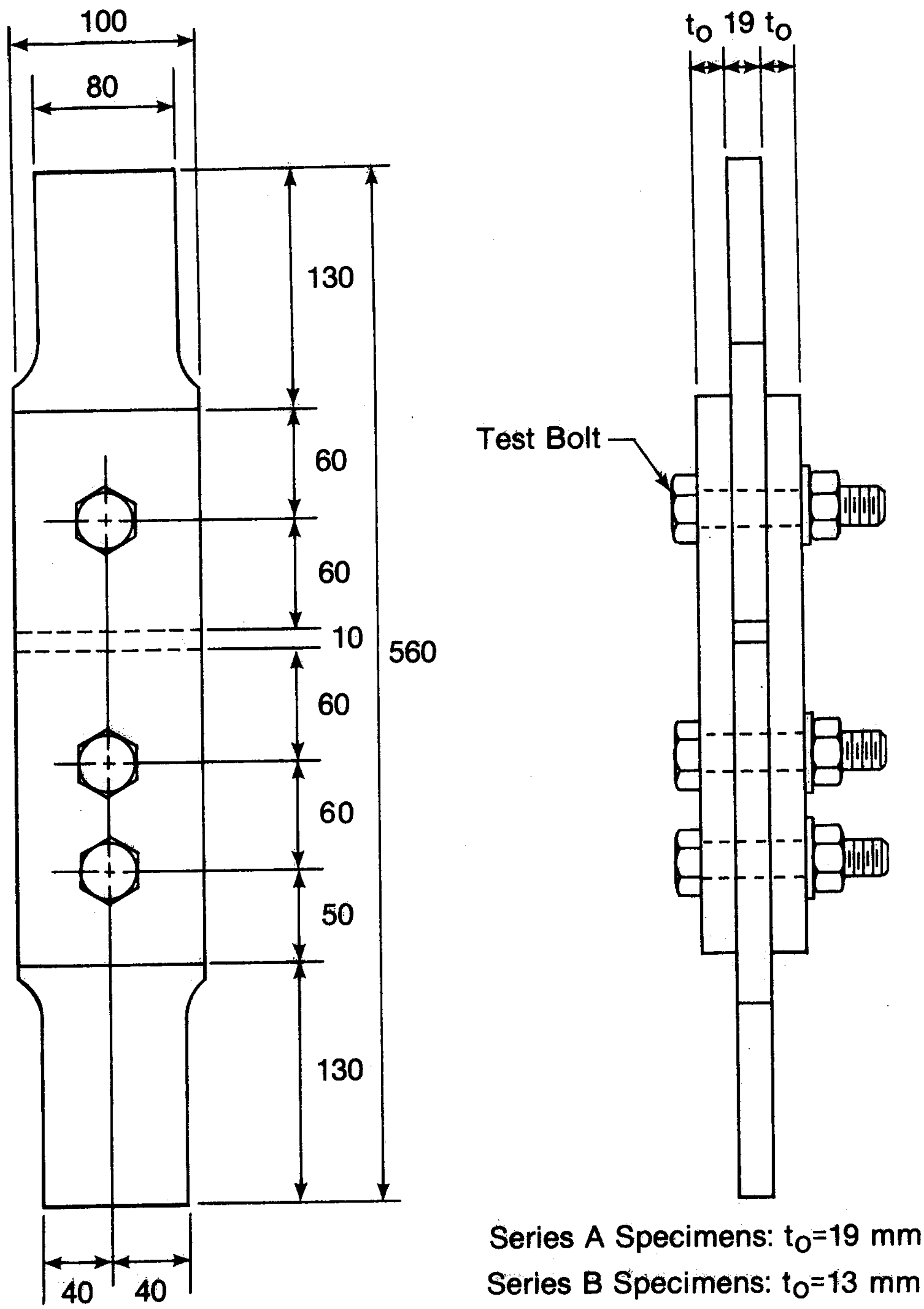


Figure 4.2 Single Bolt Shear Specimen - Tension Jig

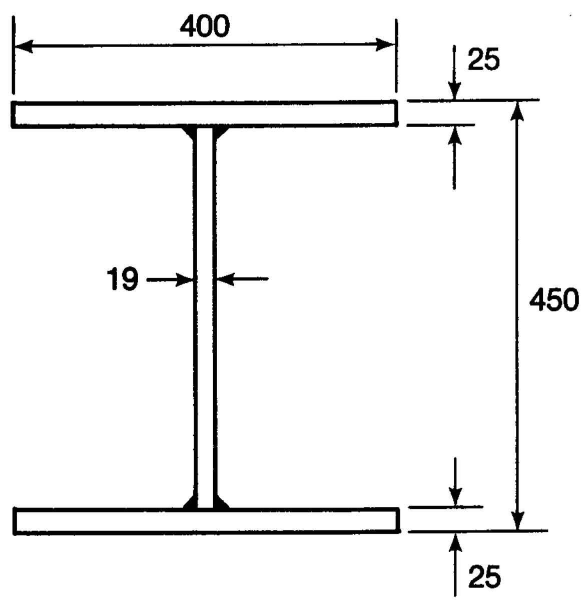


Figure 4.3 Large Scale Test Beam Cross-Section

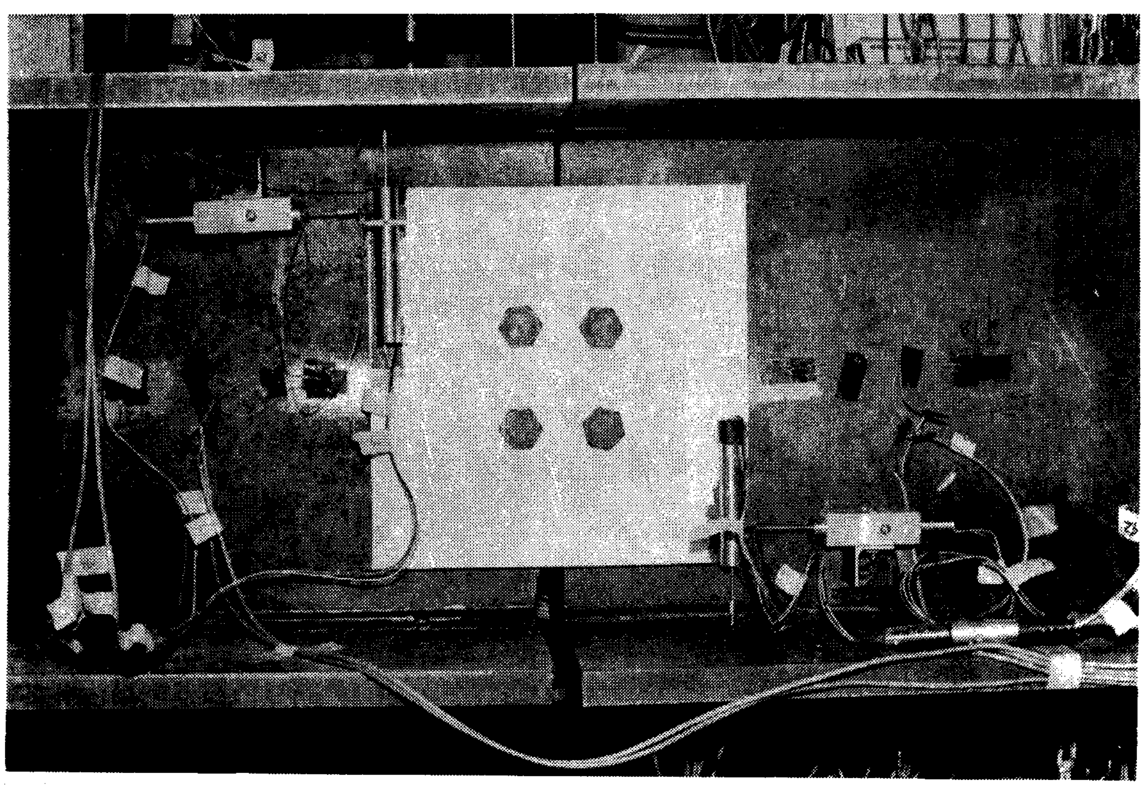


Figure 4.4 Large Scale Bolted Web Splice Specimen

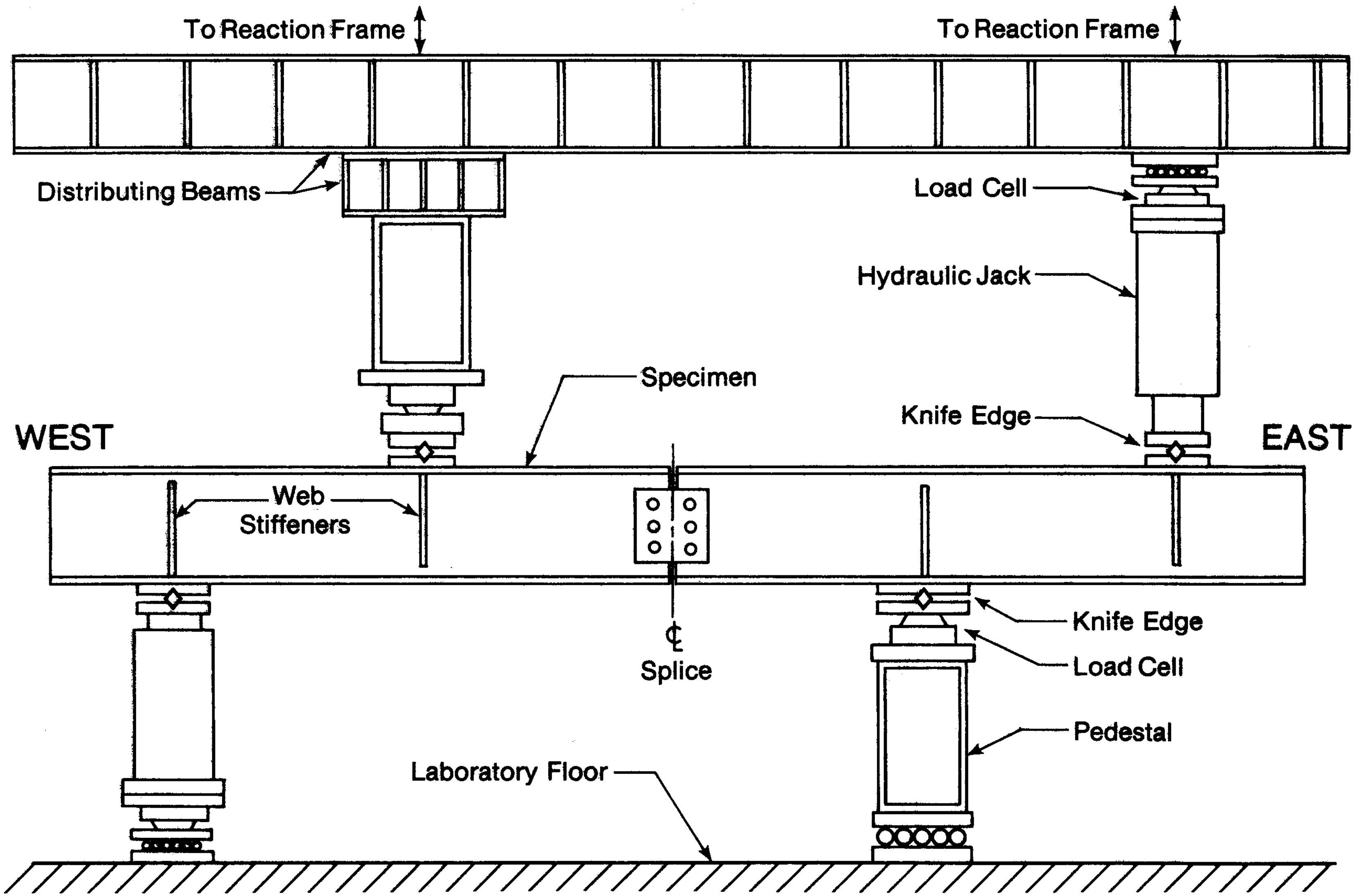


Figure 4.5 Schematic Diagram of Test Set-Up

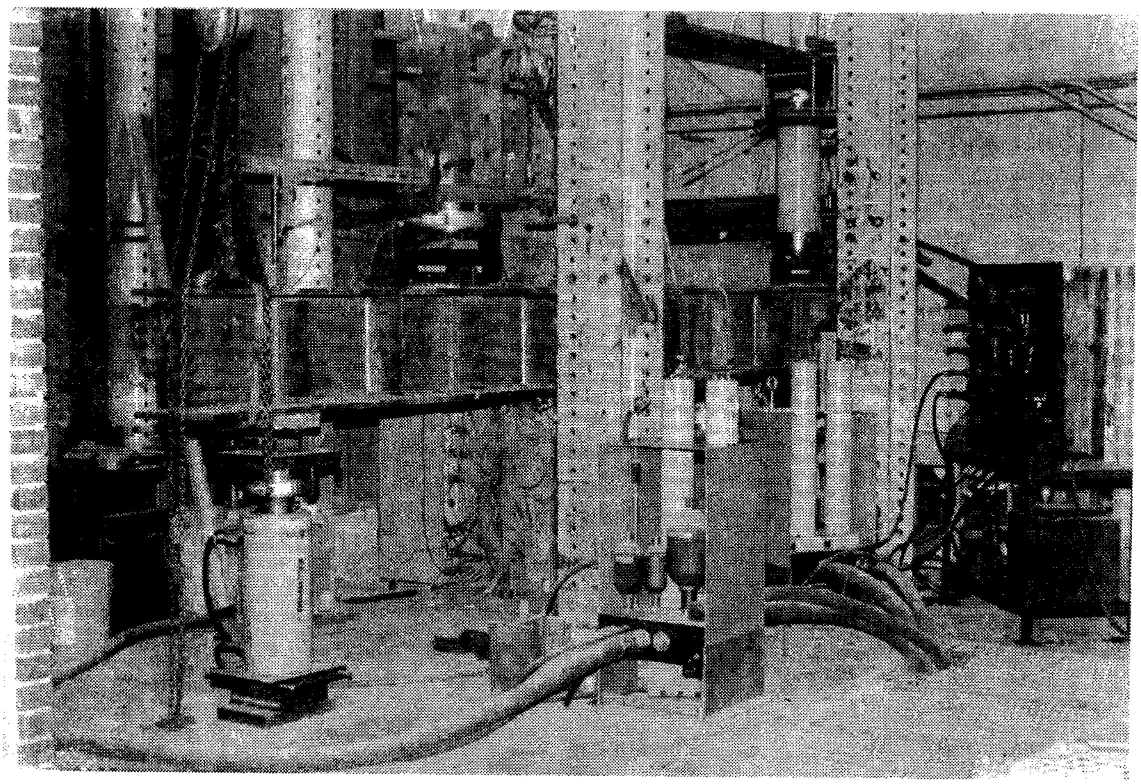
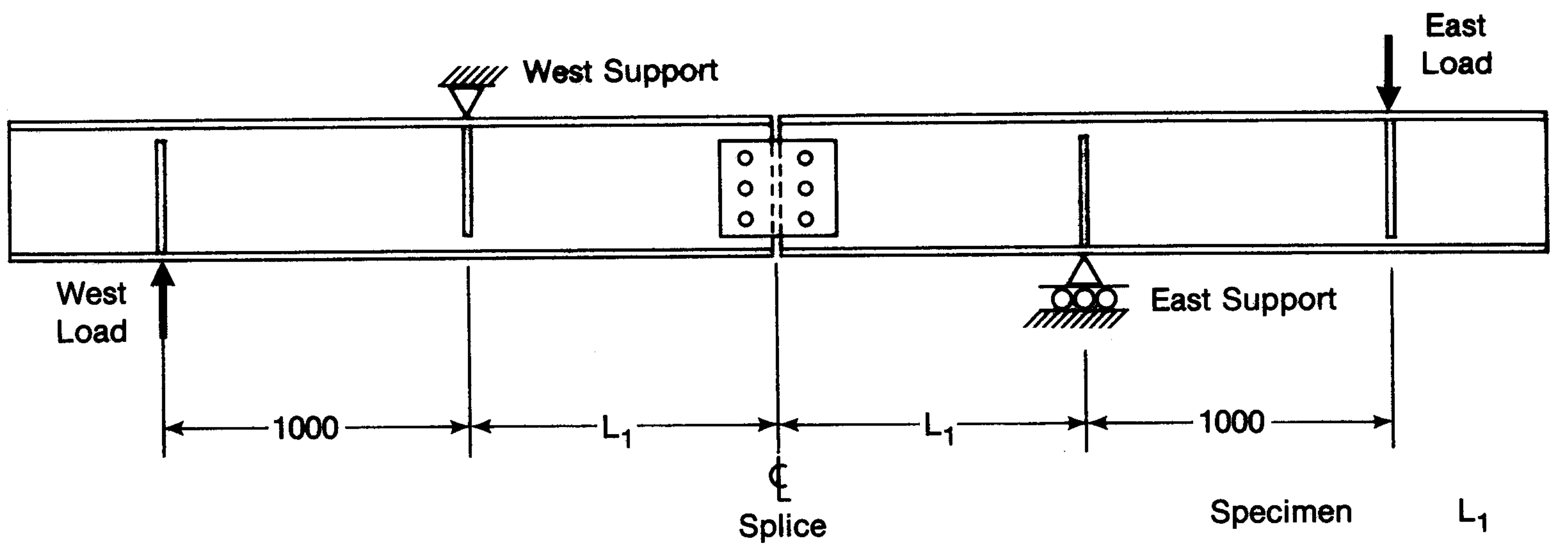


Figure 4.6 Test Set-Up



Specimen	L_1
C1	1000
C2	1000
C3	1000
C4	930
C5	930
C6	930

Figure 4.7 Beam Loading Arrangement

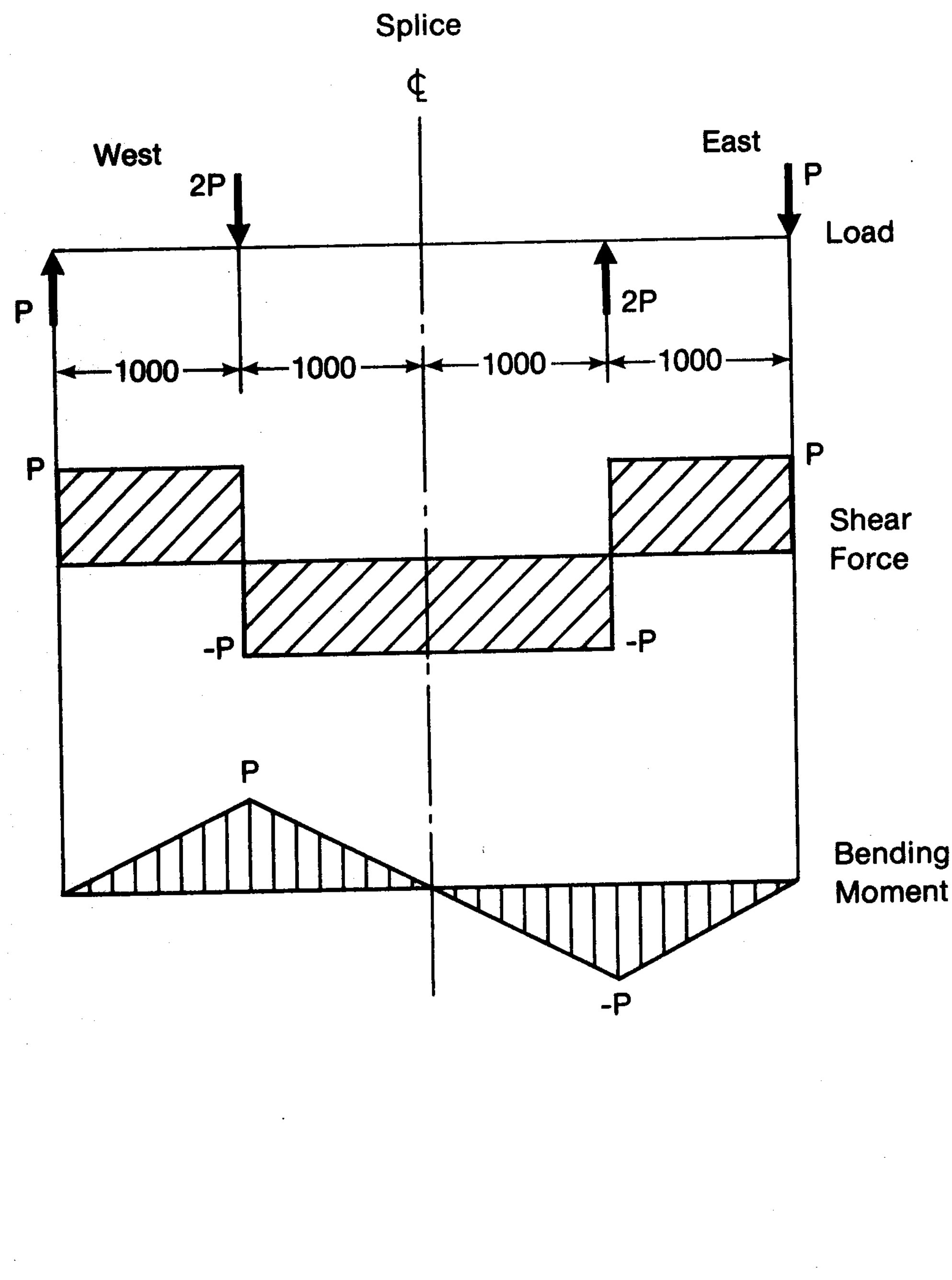


Figure 4.8 Load, Shear Force, and Bending Moment Diagrams for Specimens C1 through C5

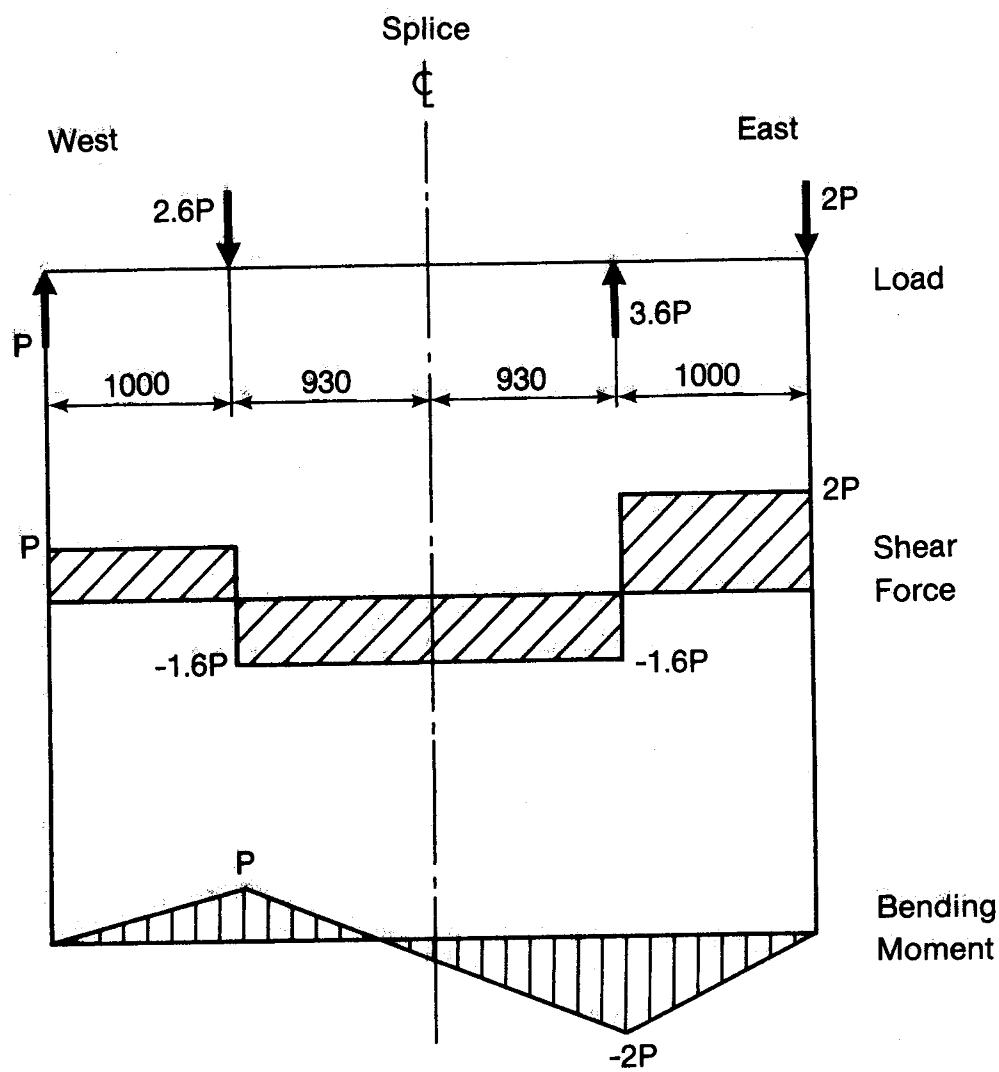


Figure 4.9 Load, Shear Force and Bending Moment Diagrams for Specimen C6

5. EXPERIMENTAL RESULTS

5.1 Introduction

The results of the experimental program outlined in Chapter 4 are presented in this chapter and these results are discussed within the context of the objectives presented in Chapter 1. The response to load of the single bolt shear specimens is presented and compared with previous experimental results obtained by others. The full-size web splice test results are discussed and compared with theoretically based predictions using both the analytical method presented in Chapter 3 and the more conventional method of analysis.

5.2 Single Bolt Shear Tests - Compression Jig

5.2.1 Load Versus Deformation Behaviour

Table 5.1 lists the values of the ultimate loads and corresponding deformations of the single bolts tested in double shear using compression jigs. For series A compression jig specimens, the mean ultimate load is 368 kN and the average deformation at the ultimate load is 4.91 mm. The coefficients of variation for these data are 1.4% and 4.3%, respectively. Equipment problems were experienced during the testing of specimen A3, and therefore the results from this test have not been included in these averages. For series B compression jig specimens, the mean ultimate load is 369 kN and the average deformation at the ultimate load is 6.19 mm. The coefficients of variation for these data are 1.2% and 8.2%, respectively.

The load versus deformation responses of the single bolt shear compression jig specimens in series A are shown in Figure 5.1. The equation that best describes the average response of these fasteners is:

$$R = 371.2(1 - e^{-0.67\Delta})^{0.29} \tag{5.1}$$

This equation is represented by the solid line curve in Figure 5.1.

The load versus deformation responses of the single bolt shear compression jig specimens in series B are shown in Figure 5.2. The equation that best describes the average response of these fasteners is:

$$R = 377.5(1 - e^{-0.54\Delta})^{0.64} \tag{5.2}$$

This equation is represented by the solid line curve in Figure 5.2.

In order to obtain the best fit of the experimental data, values greater than the ultimate bolt strength (R_u) actually obtained in the tests are used as the first coefficient in both Equations 5.1 and 5.2. However, these equations will give a bolt force (R) equal to the average experimental ultimate bolt force (R_u) when the deformation (Δ) is set equal to the average experimental deformation at ultimate.

The mean ultimate capacity of the single bolt shear specimens in series A is equal to the mean ultimate capacity of the specimens in series B. It has been shown that the type of connecting material has no effect on the shear strength of bolts (33). Because all of the bolts were from the same lot, little variation in their ultimate shear capacity was expected.

As can be seen in the load versus deformation response curves (Figures 5.1 and 5.2), there is no well defined yield point for the high-strength bolts loaded in shear using compression jigs. The relationship between load and deformation can be approximated as being linear for small deformations. However, as the applied load increases the resulting deformation also increases, but at an increasing rate. Thus, the relationship between load and deformation deviates from linearity as the load increases.

The average deformation at the ultimate load is 26% higher for the specimens in series B than for those specimens in series A. The total deformation measured for each single bolt shear specimen included the shearing, bending, and bearing deformation of the bolt and the local deformation of the connecting material. Each of these components will be examined to determine their effect on the difference between the ultimate deformations that occurred in the two series of specimens.

The middle plate in both series A specimens and series B specimens was 19 mm thick. However, the outside plates were 19 mm thick in series A specimens but only 13 mm thick in series B specimens. Thus, the amount of bolt bending should be greater in series A than in series B specimens. The difference in bolt spans in these two cases is very small, however, and it is likely that the deformation caused by bolt bending was about the same in the two cases. Little variation in the bolt properties was expected because all of the bolts were from the same lot. Therefore, it can be assumed that the shearing, bending, and bearing deformations of the bolts themselves were, within reason, the same in both series.

One of the factors that affects the deformation of the material around a bolt is the area of the material against which the bolt is bearing. As previously described, the thickness of the outside plates was greater in series A specimens than in series B specimens. Thus, the outside plates in series B specimens provided less bearing area than those in series A specimens, and as a consequence more deformation occurred in the material around the bolt holes in the outside plates of series B specimens than in the outside plates of series A specimens.

Another factor that affects the deformation of the material around a bolt hole is the strength of the material. The outside plates in series A specimens were not cut from the same steel plate as the outside plates in series B specimens. However, all of the steel had a nominal yield strength of 300 MPa and therefore not much variation in the properties of the steel plates was expected.

The only other physical difference between the two series of single bolt shear specimens was the additional washer underneath the heads of the bolts in the series B specimens. This feature is not considered to have had any significant effect on the load response of the specimens.

In summary, it is reasonable to assume that the difference in the ultimate load deformation of the two series of compression jig specimens can be attributed mainly to the differences in the thicknesses of the steel plates used in the two series and not to variation in bolt or steel properties.

A sectional view of a typical single bolt shear compression jig specimen after it had been tested and then sawn in half is shown in Figure 5.3. As seen in this figure, the permanent deformation of the bolt indicates that although the bolt was loaded primarily in shear, it was also subjected to a small amount of bending. Permanent deformation of the steel around the bolt holes was observed where the bolts had been bearing against the steel plates. In all cases, the bolt hole in the middle plate of each specimen was more deformed than the bolt holes in the outside plates. This was a consequence of the greater bearing area provided by the combined outside plates as compared with that provided by the middle plate.

The deformations at low loads are higher for series B specimens than for series A specimens. Before testing began, all of the single bolt shear specimens were subjected to an initial load to ensure that the bolts were bearing against the connecting steel plates. A preload of 22 kN was applied to the specimens in series B and then the bolts were tightened to snug. However, the bolts in series A specimens were tightened to one half turn past the snug position before the preload was applied. Slip between the connecting plates in these specimens occurred when the preload reached approximately 130 kN. Some plastic deformation of the bolt and surrounding material may have occurred during the preload, resulting in small deformations when the preload was removed and the test load then applied.

5.2.2 Comparison with Previous Results

As shown in Figure 5.4, the shape of the shear load versus shear deformation response curves for the single bolts tested using compression jigs in this program is similar to the shape of the response curve that was used to develop the non-dimensional coefficients in the CISC Handbook tables for analysis of eccentrically loaded bolt groups (31). The mean ultimate load for the single bolt shear compression jig specimens tested herein is 12% higher than the ultimate bolt force used to develop the CISC Handbook tables. The load versus deformation response curve in the CISC Handbook is based on experimental work carried out by Crawford and Kulak (32) in which the single bolts were tested in compression jigs. The bolts used in their tests were specially manufactured to just meet the minimum strength requirements of ASTM A325. Compared with the results of the tests carried out herein, the bolt strength used to develop the tables is conservative, as it was intended to be.

The dimensions of the plates used in the single bolt shear specimens tested by Crawford and Kulak (32) were identical to those used in the single bolt shear specimens in series B. However, the plates used by Crawford and Kulak had a nominal yield strength of 250 MPa as compared with 300 MPa for the steel plate in the specimens tested in this project. Most of the difference between the deformation of the single bolt shear specimens in series B and those tested by Crawford and Kulak can be attributed to the difference in the yield strength of the steel plates. A smaller portion of the difference in the deformations between the two tests is a result of the difference in the bolt properties. All of the bolts were ASTM A325 bolts, and therefore only small differences in the load responses of the bolts themselves are expected.

5.3 Single Bolt Shear Tests - Tension Jig

5.3.1 Load Versus Deformation Behaviour

Table 5.2 lists the values of the ultimate loads and corresponding deformations of the single bolts tested in double shear using tension jigs. For series A tension jig specimens, the mean ultimate load is 333 kN and the average deformation at the ultimate load is 5.24 mm. The coefficients of variation for these data are 1.0% and 3.8%, respectively. For series B tension jig specimens, the mean ultimate load is 344 kN and the average deformation at the ultimate load is 6.29 mm. The coefficients of variation for these data are 4.0% and 5.3%, respectively.

The load versus deformation responses of the single bolt shear tension jig specimens in series A and series B are shown in Figure 5.5 and Figure 5.6, respectively. For each series of specimens, the average response to shear load is shown as a solid line curve in these figures. Similar to the shear load response curves of the single bolt compression jig specimens, there is no well-defined yield point.

As is the case for the compression jig specimens, the difference in the ultimate load deformation of the two series of tension jig specimens can be attributed mainly to the differences in the thicknesses of the steel plates used in the two series. Although the average ultimate deformations are higher for the bolts tested in tension jigs than for the bolts tested in compression jigs, these differences are relatively small.

During the testing of specimens A2-T, A3-T, and B2-T, the plates began to separate near the test bolt as the ultimate load was approached. Above the test bolt, the maximum size of the gaps between the middle plate and the outside plates was approximately 1 mm. No noticeable separation of the plates occurred as the other specimens were tested.

5.3.2 Comparison with Compression Jig Test Results

The mean ultimate load for series A tension jig specimens is 90% of the mean ultimate load for series A compression jig specimens. The mean ultimate load for series B tension jig specimens is 93% of the mean ultimate load for series B compression jig specimens.

An experimental study carried out by Wallaert and Fisher (33) showed that the shear strength of high strength bolts in plates loaded in compression was approximately 10% higher than the shear strength of high strength bolts in plates loaded in tension. The lower shear strengths of the bolts tested in tension jigs were attributed to prying action of the outside plates. Prying action produces a tensile force in the bolt and, by extending Von Mise's yield criterion to ultimate conditions, it can be shown that the shear stress component decreases as the tensile stress component increases. Further tests showed that the shear strength of bolts tested in tension jigs approached the shear strength of bolts tested in compression jigs when the lap plate prying action in the tension jig was minimized.

As was noted previously, prying action of the outside plates in the tension jig specimens was only observed in half of the specimens tested in this study. Although prying action contributed to the reduced shear strengths, it likely is not the only contributing factor. A possible explanation is that the bolts in the tension jig specimens were loaded in such a manner as to induce a larger tension force in these bolts than in the compression jig test bolts and as a consequence, lower bolt shear strengths resulted.

5.4 Web Splice Tests

5.4.1 General Behaviour

Considerable permanent deformation of the web steel occurred around the bolt holes, as shown in Figure 5.7 for specimen C1. By the time the connection in each large scale specimen failed, the bolt holes were oblong because of yielding of the steel under the bearing action of the bolts. Typically, a bolt hole was deformed by approximately 2 mm in the direction of the bolt force.

In specimens C1 and C2, the web bulged noticeably at the ends of the beams where the top east and bottom west bolts pushed against it, as shown in Figure 5.7 for specimen C1. This happened because the distance provided from these bolts to the end of the beams (at the splice centerline) was not sufficient. This could have been anticipated using the analytical method presented in Chapter 3 to predict the direction of the bolt forces. The holes for these bolts were deformed by approximately 3 mm and 5 mm for specimens C1 and C2, respectively.

One of the ways in which the validity of an analytical method for the beam web splice can be evaluated is to compare the observed direction of the bolt force with a predicted direction. The experimental and theoretical angles of deformation of the web bolt holes, measured with respect to the horizontal axis, are compared in Table 5.3. The experimental angles of deformation were obtained by measuring the deformation of the bolt holes in the beam webs after failure had occurred. This measurement involved some judgement and the experimental angles must therefore be considered to be approximations. The predicted direction of the bolt force is available directly from the analysis of the connection strength. In Table 5.3, the theoretical angles of deformation were calculated using both the conventional method of analysis and the method of analysis developed in Chapter 3. Clearly, the best agreement between the experimental and theoretical angles of deformation results using the method of analysis developed in Chapter 3. Thus, the measured angles of deformation support the proposed method of analysis. (The west bolt group in specimen C6 has been excluded from Table 5.3 because it was not subjected to its ultimate load.)

It can be seen in Figure 5.8 that some yielding of the steel occurred in the splice plates around the bolt holes on the east side of the splice in specimen C6. (The chalk line near the top corner of the splice plate shown in this figure indicates the position of the splice plate before testing had begun.) Yielding of the splice plates was also observed around all of the bolt holes in specimen C5. The white-washing that had been applied to the splice plates in specimens C2 through C5 gave no visual indication of yielding. It should be noted that the splice plates in specimen C1 were not white-washed.

Yielding of the steel plates around the bolt holes occurred only in specimens C5 and C6 because thinner splice plates were used in these specimens than in specimens C1 through C4. Although the splice plates in specimen C1 were not white-washed, it can be assumed that there was no visible yielding in these plates because all of the splice plates used in specimens C1, C2, C3 and C4 were identical. The deformed shape of the bolt holes was an indication that local yielding of the splice plates actually did occur in these specimens. However, the affected areas were underneath the heads and washers of the bolts where the plates had not been white-washed. The affected areas for specimens C5 and C6 were much larger. Yielding had spread into the area of the plates that had been white-washed, and therefore was visible. In all cases, yielding of the steel around each hole occurred in the general direction of the predicted bolt force.

Rotation of the beams was mainly a consequence of the deformation of the splice components. Although it was not measured, very little rotation of the beams was observed at low loads except in specimens C1, C2 and C6. In all cases, the rotation of the beams was very noticeable as the web splice connection in each specimen reached its ultimate load. For specimens C1 and C2, the larger degree of rotation can be attributed to the large bearing deformation in the web near the top east and bottom west bolts. For specimen C6, the larger degree of rotation can be attributed to an actual moment at the centerline of the splice that was significantly greater than zero. As a consequence of this moment, the forces and deformations of the bolts on the east side of the splice were not identical to those of the bolts on the west side of the splice.

Figure 5.9 shows the relationship between the load applied to one end of the beam and the horizontal deflection, measured relative to the beam web, of the lower east corner of the south splice plate of specimen C1. This figure represents the typical load versus deflection curve for all of the measured horizontal and vertical movements of the splice plates. Although the magnitude of the loads and deflections varied, the shape of the load versus splice plate deflection curves does not change significantly. The shape of these curves was similar to the shape of the load response curves for the single bolt shear specimens in that there was no well defined yield point.

The movements of the splice plates relative to the beam webs were used to locate the experimental instantaneous centers of the bolt groups in the web splice specimens. The radius of rotation (r_o) of a bolt group on one side of the splice is equal to the distance from the centroid of the bolt group to its instantaneous center of rotation. In Table 5.4, the experimental radii of rotation are compared with the theoretical radii of rotation for the bolt groups on either side of the web splices in the large scale test specimens. The theoretically based predicted values and the experimental values are not in good agreement. Except for specimens C1 and C2, failure occurred in the bolt group on the side of the splice that had the smaller measured radius of rotation.

The experimental radii of rotation were calculated by dividing the vertical displacement of each bolt group by the tangent of the angle of rotation of the corresponding splice plate. The average of the measured north and south splice plate deflections was used in these calculations. The angle of rotation of each splice plate was calculated from the measured horizontal deflections and it was based on the assumption that the deflection at the top of the plate was equal to the deflection at the bottom of the plate. This is a reasonable assumption because there were no external horizontal forces applied to the beam, and therefore the horizontal components of the bolt forces and deformations on either side of the neutral axis are theoretically equal and opposite to one another. The theoretical radii of rotation were calculated using the analytical method presented in Chapter 3.

As described above, the experimental radii of rotation were calculated using the movements of the splice plates measured relative to the beam webs. These movements are a function of the total bolt deformation which includes both the bolt deformation and the local deformation of the connecting material. In specimens C1 and C2, the local deformation of the web material around the top east and bottom west bolt holes was much greater than the local deformation of the middle plate in the single bolt shear specimens. Thus, the experimental radii of rotation for specimens C1 and C2 should not be expected to agree closely with the theoretical values.

In the tests carried out by Crawford and Kulak (32) on eccentrically loaded bolted connections, the experimental radii of rotation also did not agree closely with the predicted values. It was recognized in their study that a small error in measurement can significantly affect the calculation of the experimental radius of rotation.

The measured and calculated strains at two selected load steps (one of which is the ultimate) for specimens C1 through C6 are shown in Figures 5.10 to 5.15. As described in Chapter 4, the measured strains were taken on the flanges at particular locations along the beams. The calculated strains were determined using the measured applied loads and the nominal physical properties of the test beams. Some measured strains that were clearly incorrect are not shown in these figures. However, in some cases, it was difficult to differentiate between correct and incorrect strain readings.

In general, using the strain gauges that were not in the immediate vicinity of the splice itself, the measured strains agree well with the calculated strains, particularly at the lower load steps. The flange strains measured in regions close to the splices do not agree with calculated strains as well as those flange strains measured in regions further from the splices. Although it is not possible to determine the exact location of zero moment from these data, the measured strains indicate that the actual location of zero moment is close to the theoretical location for all of the specimens.

The strains that were measured locally near the locations of the splices indicate that the stress at the locations of zero moment was not equal to zero. One of the possible explanations for this is that an axial force was present in the beams. Since the test set-up did not permit axial forces to develop, a more reasonable explanation is that the web splice disturbed the normal stress distribution of the beam in the region of the splice.

5.4.2 Ultimate Strength of Connections

Failure of the web splice specimens occurred shortly after the maximum loads were reached. All of the failures were sudden, except for specimen C6. Specimen C1 failed when both of the bolts on the east side of the splice sheared off. Figure 5.16(a) shows the top east bolt from this specimen as it would have looked just before failure occurred. The sheared surfaces of this bolt can be clearly seen in Figure 5.16(b). Although the bolts on the west side of the splice did not rupture, they were badly deformed.

Specimens C2 and C3 also failed when both of the east bolts sheared off. The west bolts in specimen C3 were not deformed as much as the west bolts in specimens C1 and C2. One of the west bolts in specimen C2 sheared off in a single plane only. Specimen C4 failed when all three of the bolts on the east side of the splice sheared off. The upper and lower bolts on the west side of the splice were deformed more than the middle bolt, as expected. Specimen C5 failed when all four of the east bolts sheared off, although the lower east bolt sheared off in only a single plane. Three of the west bolts sheared off in a single plane while the upper west bolt was very deformed. Specimen C6 failed when the lower west bolt on the east side of the splice sheared off in one plane. The remaining bolts on the east side were badly deformed. The bolts on the west side were not deformed, but they were slightly polished as a result of bearing against the steel splice plates and web.

The experimental and predicted ultimate shear capacities of the bolted web splice specimens are listed in Table 5.5. The shear used to identify the load at which failure occurred was obtained by averaging the difference between the maximum east jack and east reaction loads with the difference between the maximum west jack and west reaction loads. As can be seen in Figures 4.7 and 4.8, the magnitude of the shear force at the centerline of the splice is equal to the difference between the magnitude of the applied load and the reaction forces on either side of the splice. Theoretically, the difference between the east applied load and reaction is identical to the difference between the west applied load and reaction. However, the experimental differences were not identical and were therefore averaged. The predicted shear capacities of the web splices in Table 5.5 were calculated using the method of analysis presented in Chapter 3, that is, with the shear force considered to be acting at the centerline of the splice. The response to shear load obtained from the single bolt compression jig tests was used in the analysis to predict the shear capacities of the web splice connections.

The ultimate strength method currently used to analyze eccentrically loaded bolted connections uses the shear load versus shear deformation response of a single bolt in a compression jig (Section 2.4) (13,31). For this reason, the experimental response to shear load of the single bolts tested in compression jigs will be used initially to predict the ultimate shear capacities of the bolted web splice specimens. However, the shear capacities of the web splice specimens will subsequently be predicted using the tension jig shear test results.

5.4.3 Comparison of Test Results with Analytical Predictions

Using the method of analysis presented in Chapter 3 and the response to shear load of a single bolt in a compression jig, the ultimate shear capacity test to predicted ratio varies from 0.85 to 0.90 for the bolted web splices (Table 5.5). The average test to predicted ratio is 0.87 with a coefficient of variation of 2.3%. These test to predicted ratios indicate that this method of analysis yields results that are consistent, but unconservative. The factors that may cause the results to be unconservative will be examined subsequently. Table 5.5 also contains an alternative way of looking at the results. Assuming that the test result is the "true" value, then the percent error of the predictor is shown to vary from +10.6 to +16.1.

Crawford and Kulak (32) obtained similar experimental results in their study of eccentrically loaded bolted connections. For their tests, the ultimate strength test to predicted ratio varied from 0.86 to 0.94. These predicted loads were calculated also using the ultimate strength method of analysis and the response to shear load of single bolts in compression jigs.

The ultimate shear capacities of web splice specimens C1 through C5 were predicted based on the assumption that the moment at the centerline of the splice was equal to zero. Under idealized conditions, the bolt groups on either side of the web splices in these specimens would fail simultaneously. The location of zero moment is dependent on both the magnitudes and the positions of the applied loads and the reactions. It is not possible practically for the actual test set-up to be identical to the idealized test set-up. Even a small difference between the idealized and experimental test set-ups will result in one bolt group failing before the other one. In the tests carried out in this study, the east bolt group failed first in all of the specimens. However, visual inspection of the west bolts in specimens C1 through C5 after the tests indicated that the west bolt groups had almost reached their ultimate loads.

During testing it was difficult to apply equal loads to the east and west ends of the beams as the ultimate load was approached. For specimens C1 through C5, the maximum west jack load (see Fig. 4.7) was slightly higher than the maximum east jack load. Theoretically, this results in a small positive moment at the centerline of the splice and this would contribute to decreasing the shear capacity of the web splice as predicted on the basis of zero moment. However, a positive moment at the splice should cause the bolt group on the west side of the splice to fail first. This contradicts the results of the experiments in which the east bolt group was always the first to fail. However, because the difference between the measured east and west jack loads was less than 2%, it was ignored in the theoretical shear capacity analysis.

As the ultimate load for specimen C6 was approached, the ratio between the east and west jack loads (Figure 4.8) was equal to 1.82. This is significantly less than the ratio of 2.0 that existed during most of the test. This resulted in a shift in the theoretical location of the inflection point (Figure 5.15). However, the theoretical ultimate shear capacity (Table 5.5) was calculated using the experimental ratio of the ultimate applied loads just noted, that is, 1.82.

The east jack and reaction and the west jack were free to translate horizontally. Horizontal movement occurred as a result of both bending of the beams and deformation of the splice components, but this movement was not symmetrical about the centerline of the splice because the west reaction was fixed. Although these horizontal movements were relatively small, they were large enough to cause the location of zero moment not to coincide with the centerline of the splice. Because the east bolt group was always the first to fail, it is concluded that the horizontal translations resulted in the location of zero moment being slightly west of the centerline of the splice in specimens C1 through C5. As noted previously, a moment at the centerline of the splice would contribute to an unconservative prediction of the shear capacity of a web splice if the analysis was carried out based on the assumption that the moment at the centerline of the splice was equal to zero.

The magnitude of the moment that would have to exist at the centerline of each web splice in order for the predicted shear capacity to be equal to the experimental shear capacity was calculated and is shown in Table 5.6. The distance between the location of this moment and the assumed location of the centerline of the splice is also shown in Table 5.6. In all cases, this distance is relatively small because the bolted web splice connections tested were located in regions of the specimens with relatively steep moment gradients (Table 5.6). It is possible that the actual difference between the theoretical and experimental location of the centerline of each splice did approach this magnitude. The largest distance, 40 mm, was for specimen C3. As was noted previously, the west bolts in this specimen were not as deformed as the west bolts in the other specimens. This indicates that the location of zero moment in this test was further east of the centerline of the splice than it was in the other tests.

Crawford and Kulak (32) suggested that one reason connections which are subjected to eccentrically applied loads are not able to reach their theoretical ultimate capacities is because in a full-scale test the direction of the force on each bolt changes as the instantaneous center of rotation moves, whereas the direction of the force and corresponding deformation never changes in the single bolt calibration test. In the test program carried out for this report, specimens C1, C2 and C3 contained two bolts in a single line on either side of the splice. For this bolt arrangement, the theoretical location of the instantaneous center of rotation does not change as the magnitude of the eccentrically applied load increases. Consequently, there is no change in the direction of the force on each bolt. For the case of only two bolts in a vertical line, the location of the instantaneous center is uniquely defined and is the same for any value of an applied eccentric force. Examination of Equations 3.7 and 3.9 for this case will show that,

$$r_o = \frac{b^2}{4e} \tag{5.3}$$

For any other number of bolts greater than two, the location of the instantaneous center of rotation is a function of the load level in addition to the geometry used.

For the bolt arrangements used in specimens C4 and C5 the theoretical location of the instantaneous center moved a relatively small distance when the applied load was within the range where most of the deformation occurred. This resulted in only minor changes in the direction of the forces acting on the bolts. However, a shift in the location of the inflection point occurred as the ultimate load was approached during the testing of specimen C6. Consequently, the location of the instantaneous center, and hence the direction of the forces on the bolts, changed. Therefore, except for specimen C6, movement of the instantaneous center during the loading history probably was only a small contributor to the difference between theoretical predictions and test results in this program.

If a bolt group is required to carry a transverse shear force that acts at the center of gravity of that bolt group, then each bolt must resist only a vertical force. In this case, the instantaneous center of rotation of the bolt group is located at an infinite distance from the bolt group (along its neutral axis). However, if the bolt group carries moment in addition to shear, then each bolt must resist both horizontal and vertical components of force. (This moment may be caused by a shear force that is considered to be eccentrically applied to the bolt group.) For this bolt group, the instantaneous center of rotation is located at a finite distance from the bolt group. As the moment increases, the horizontal components of the bolt forces increase and the radius of rotation decreases. Thus, the vertical components of the bolt forces, and thereby the shear capacity of the bolt group, also decrease.

It should therefore be evident that an experimental radius of rotation that is smaller than the predicted value indicates that the experimental shear capacity of a connection is less than the predicted shear capacity. Except for specimens C2 and C6, the experimental radii of rotation were less than the predicted values. As previously explained, the experimental radii of rotation for specimen C2 are not considered to be accurate. The experimental radius of rotation for the critical bolt group in specimen C6 is only 10% higher than the predicted value. Thus, for the majority of specimens tested, the measured location of the center of rotation supports the observation that the theoretical shear capacity predictions will be greater than the test values.

Tests carried out in this study show that the shear strength of high strength bolts in plates loaded in compression is higher than the shear strength of high strength bolts in plates loaded in tension (Section 5.3.2). This effect should result in the upper east and lower west bolts in the web splices failing at a lower load than was assumed in the connection ultimate strength analysis because these bolts are located in tension regions of the beam webs. The failure of one of these bolts will lead to immediate failure of the entire connection. This reduced shear capacity of the bolts in the tension regions can contribute significantly to decreasing the web splice shear capacities as predicted on the basis of the response to shear load of the single bolt compression jig specimens.

The theoretical ultimate shear capacities of the web splices have also been calculated using the response to shear load of the single bolt tension jig specimens and these predictions are compared with the test results in Table 5.7. The test to predicted ratio ranges from 0.92 to 1.00 and the corresponding percent error of the predictor varies from 0.4 to +9.0. These ratios indicate that this method of analysis yields results that are only slightly conservative. The actual connection strengths are significantly closer to the connection strengths predicted using the tension jig test results than when the connection strengths were predicted using the compression jig test results.

The largest difference between the test and predicted capacities of the splices is for specimen C6. As previously discussed, it is felt that this test result was influenced by a shift in the location of the inflection point as the ultimate load was approached. The largest difference between the test and predicted loads of the splices located at points of contraflexure is for specimen C3. It was previously noted that the west bolts in this specimen were not as deformed as were the west bolts in the other specimens, indicating that the point of inflection was further west of the splice centerline in this test as compared with the other tests.

5.4.4 Comparison of Test Results with Conventional Analysis

In Table 5.8, the predicted shear capacities of the web splices were calculated using the assumption that the shear force acts at the centroid of the opposite bolt group. The response to shear load of the compression jig specimens was used in the analysis. (The assumption used in the conventional method of analysis is usually used together with the assumption that the web splice is designed to transfer transverse shear only. Because specimen C6 was not located at a point of contraflexure, it has been excluded from this table.) The ratio of the ultimate test shear capacity to the predicted value ranges from 1.04 to 1.43 using this conventional method of analysis. The average test to predicted ratio is 1.25 with a coefficient of variation of 11.5%. These test to predicted ratios (Table 5.8) indicate that the conventional method of analysis yields results that are both conservative and inconsistent.

The best agreement between the experimental and predicted ultimate shear capacity is for specimen C3. The bolts in this specimen were arranged in such a manner that the pitch was relatively large and the distance of the bolts from the centerline of the splice was relatively small.

The poorest agreement between the experimental and predicted ultimate shear capacity is for specimen C5. The pitch of the bolts in this specimen was relatively small. There were two lines of bolts on either side of the splice, resulting in a greater distance between the centroids of the two bolt groups than for the bolt arrangements in the other specimens.

The theoretical ultimate shear capacities of the web splices have also been calculated using the response to shear load of the single bolt tension jig specimens and the assumption that the shear force acts at the centerline of the opposite bolt group. As shown in Table 5.9, the ultimate shear capacity test to predicted ratio using this approach ranges from 1.16 to 1.47. The corresponding percent error of the predictor ranges from -13.5% to -31.9%. The predicted shear capacities based on the tension jig test are more conservative than the predictions based on the compression jig test results (using the assumption that the shear force acts at the centroid of the opposite bolt group). The test to predicted ratios are also very inconsistent.

Table 5.1 Single Bolt Shear Test Results - Compression Jig

Specimen	Maximum Load (Double Shear) kN	Maximum Deformation mm
A1-C	365	4.95
A2-C	366	4.60
A4-C	376	5.05
A5-C	367	5.05
B1-C	364	5.61
B2-C	370	6.50
B3-C	373	6.47

Table 5.2 Single Bolt Shear Test Results - Tension Jig

Specimen	Maximum Load (Double Shear) kN	Maximum Deformation mm
A1-T	335	5.13
A2-T	329	5.47
A3-T	335	5.12
B1-T	360	6.56
B2-T	338	5.92
B3-T	335	6.40

Table 5.3 Bolt Force Directions

Specimen	Bolt	Predicted Angle degrees		Test Angle degrees
		Proposed Method	Conventional Method	
C1	1	57	38	60
	2	57	38	60
	3	57	38	60
	4	57	38	55
C2	1	51	27	59
	2	51	27	47
	3	51	27	37
	4	51	27	59
C3	1	65	48	72
	2	65	48	72
	3	65	48	67
	4	65	48	65
C4	1	43	22	43
	2	90	90	87
	3	43	22	40
	4	43	22	43
	5	90	90	87
	6	43	22	45
C5	1	34	2	25
	2	34	2	25
	3	65	46	53
	4	65	46	55
	5	65	46	55
	6	65	46	53
	7	34	2	28
	8	34	2	35
C6	1	-	-	-
	2	-	-	-
	3	-	-	-
	4	-	-	-
	5	34	-	35
	6	34	-	28
	7	2	-	0
	8	2	-	0

Table 5.4 Radii of Rotation

Specimen	Experimental r_o mm		Theoretical r_o mm
	East	West	
C1	36	34	78
C2	91	72	50
C3	139	174	242
C4	16	30	98
C5	29	29	58
C6	44	-	39

Table 5.5 Comparison of Web Splice Test Results with Predictions Based on the Proposed Method of Analysis and Compression Jig Test Results

Specimen	Predicted Shear Capacity kN	Test Shear Capacity kN	Test / Predicted	% Error *
C1	618	551	0.89	+12.2
C2	573	518	0.90	+10.6
C3	671	570	0.85	+17.7
C4	907	783	0.86	+15.8
C5	902	798	0.88	+13.0
C6	426	367	0.86	+16.1

*The test value is taken as the true value.

Table 5.6 Bending Moment at the Splice Centerline

Specimen	Moment at Splice Centerline used to Predict Shear Capacity kN.m	Moment Required for Predicted Shear Capacity to Equal Test Shear Capacity kN.m	Experimental Moment Gradient kN.m/m	Distance Between Required & Assumed Moment mm
C1	0	6.8	551	12.3
C2	0	4.4	518	8.5
C3	0	22.9	570	40.2
C4	0	14.6	783	18.6
C5	0	24.8	798	31.1
C6	116	124.7	367	23.7

Table 5.7 Comparison of Web Splice Test Results with Predictions Based on the Proposed Method of Analysis and Tension Jig Test Results

Specimen	Predicted Shear Capacity kN	Test Shear Capacity kN	Test / Predicted	% Error *
C1	561	551	0.98	+1.8
C2	520	518	1.00	0.4
C3	606	570	0.94	+6.3
C4	810	783	0.97	+3.4
C5	824	798	0.97	+3.3
C6	400	367	0.92	+9.0

*The test value is taken as the true value.

Table 5.8 Comparison of Web Splice Test Results with Predictions Based on the Conventional Method of Analysis and Compression Jig Test Results

Specimen	Predicted Shear Capacity kN	Test Shear Capacity kN	Test / Predicted	% Error *
C1	457	551	1.21	-17.1
C2	393	518	1.32	-24.1
C3	549	570	1.04	-3.7
C4	618	783	1.27	-21.1
C5	560	798	1.43	-29.8

*The test value is taken as the "true" value.

Table 5.9 Comparison of Web Splice Test Results with Predictions Based on the Conventional Method of Analysis and Tension Jig Test Results

Specimen	Predicted Shear Capacity kN	Test Shear Capacity kN	Test / Predicted	% Error *
C1	410	551	1.34	-25.6
C2	353	518	1.47	-31.9
C3	493	570	1.16	-13.5
C4	546	783	1.43	-30.3
C5	519	798	1.54	-35.0

*The test value is taken as the true value.

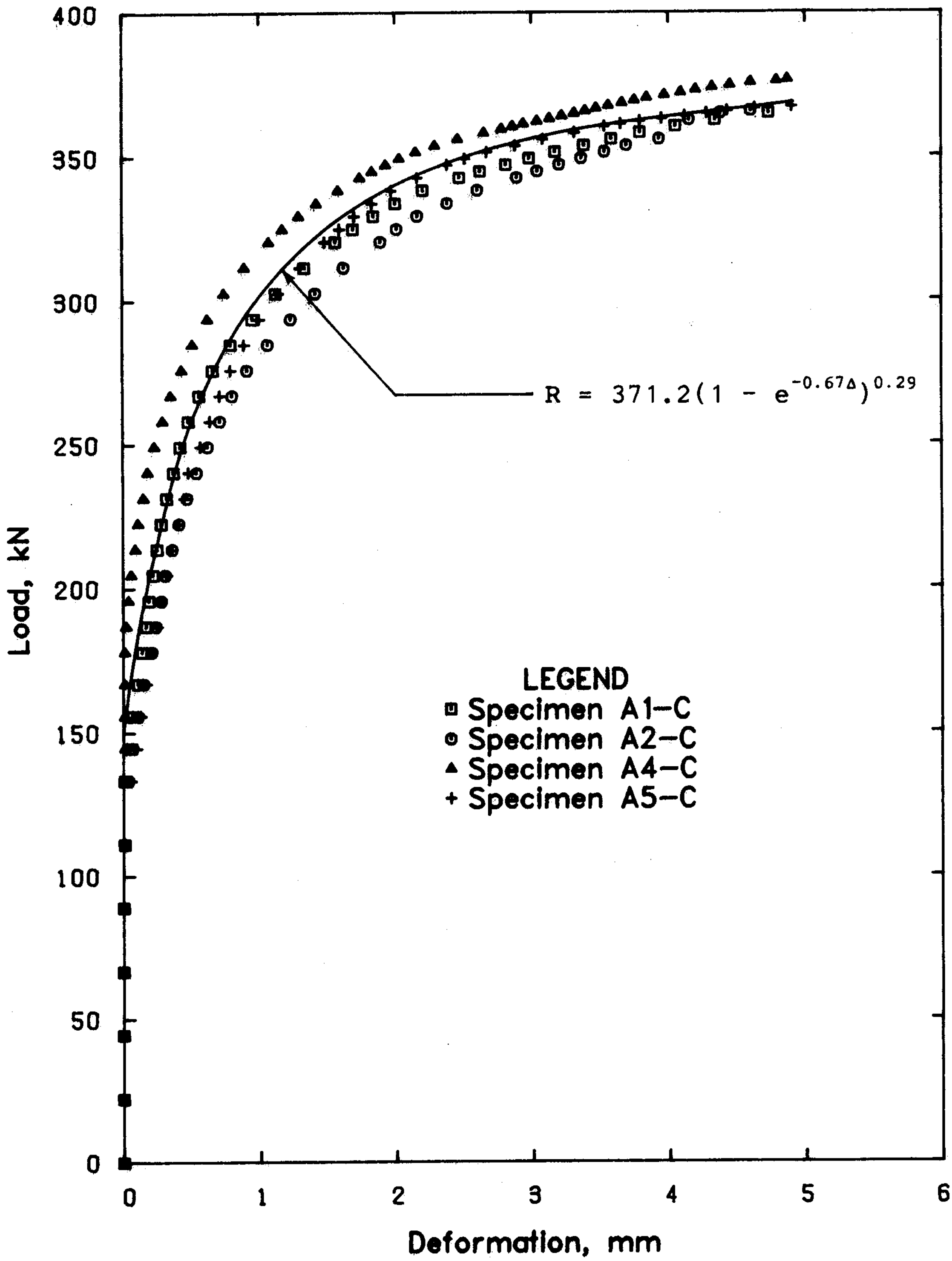


Figure 5.1 Shear Load Versus Shear Deformation Response of Series A Single Bolt Shear Specimens - Compression Jig

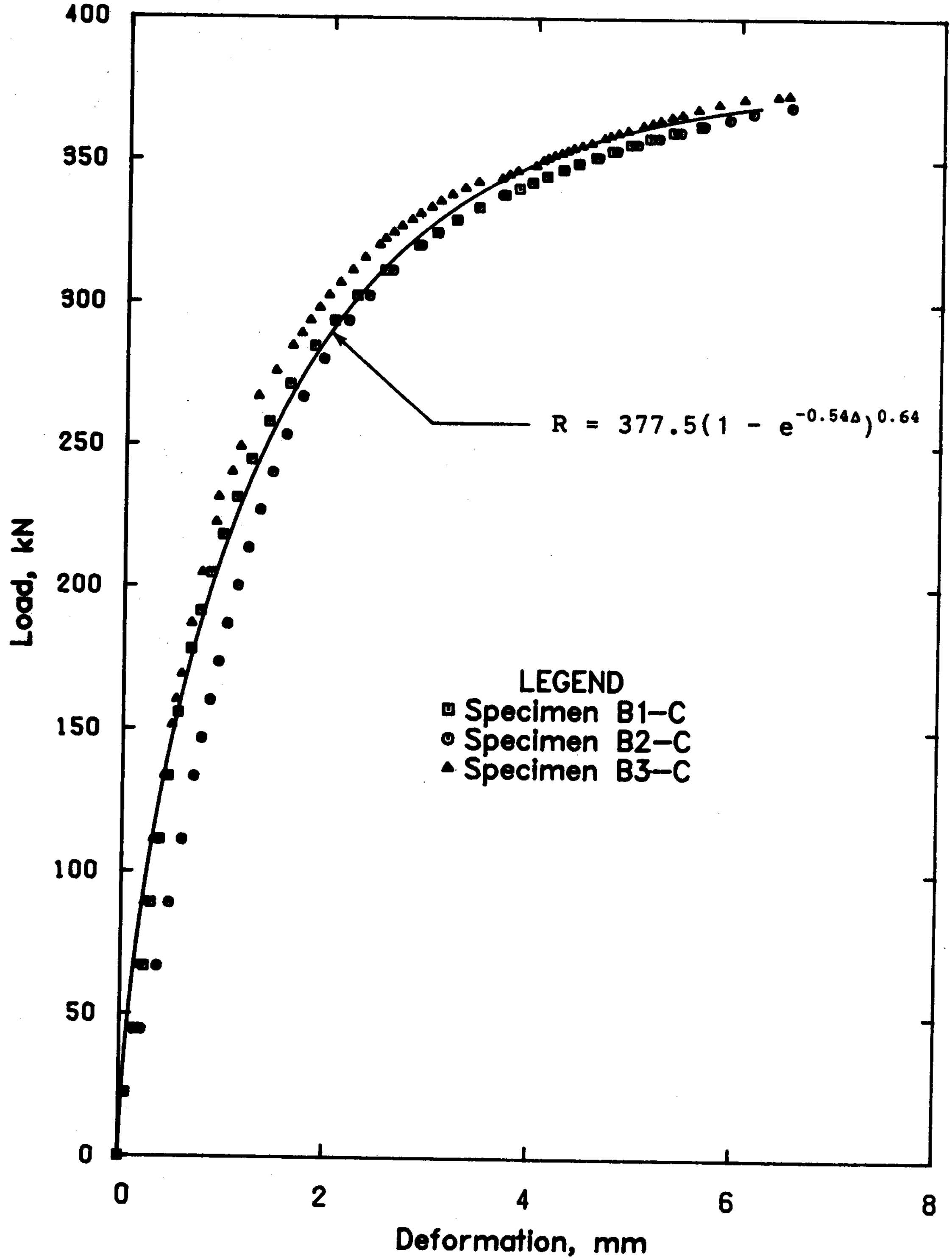


Figure 5.2 Shear Load Versus Shear Deformation Response of Series B Single Bolt Shear Specimens - Compression Jig

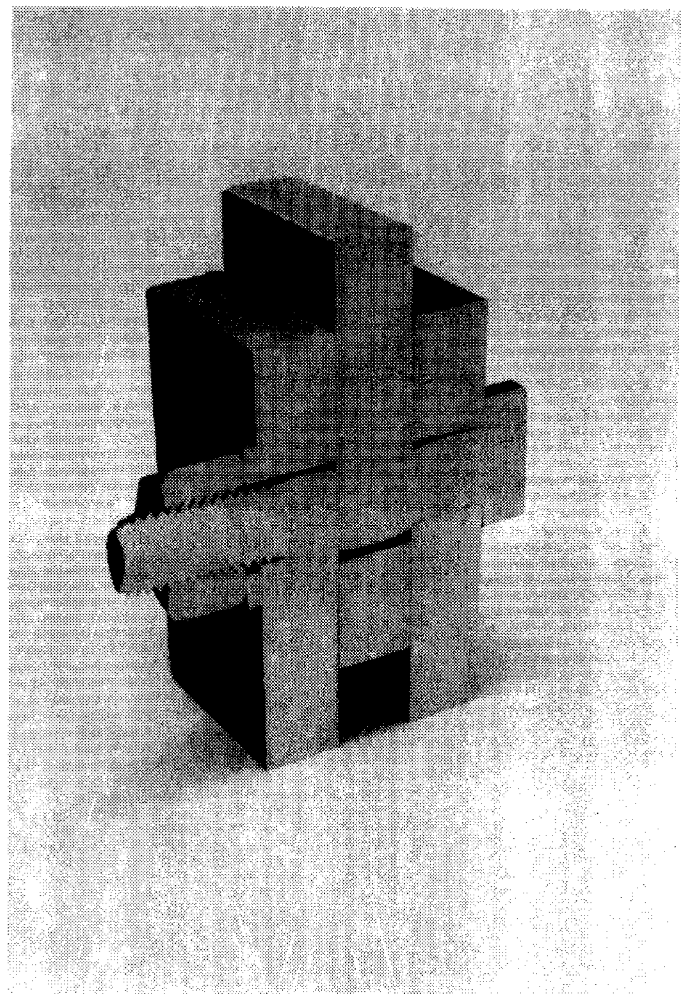


Figure 5.3 Sectional View of a Failed Single Bolt Shear Compression Jig Specimen

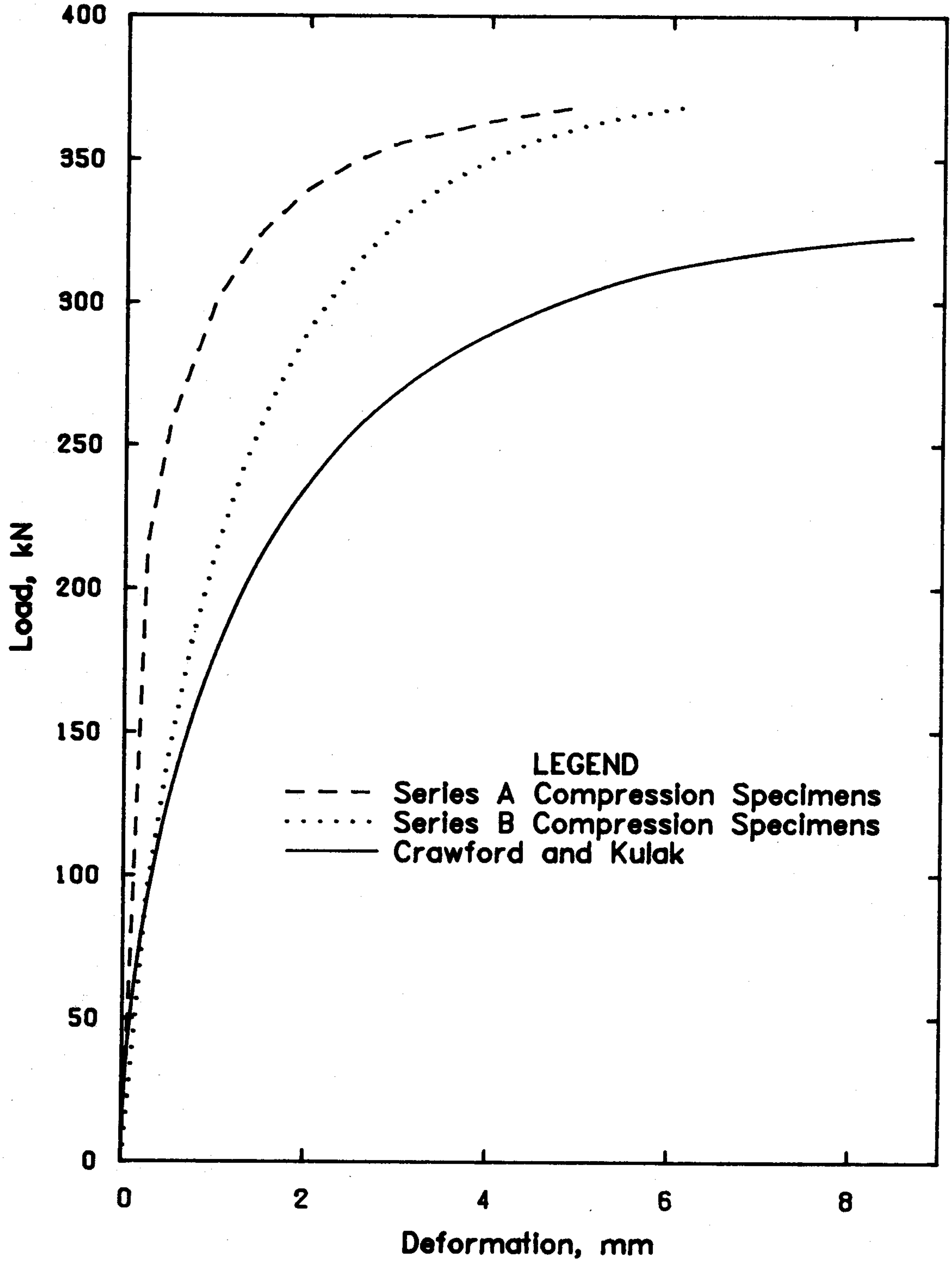


Figure 5.4 Comparison of Load Responses of Single Bolt Shear Specimens - Compression Jig

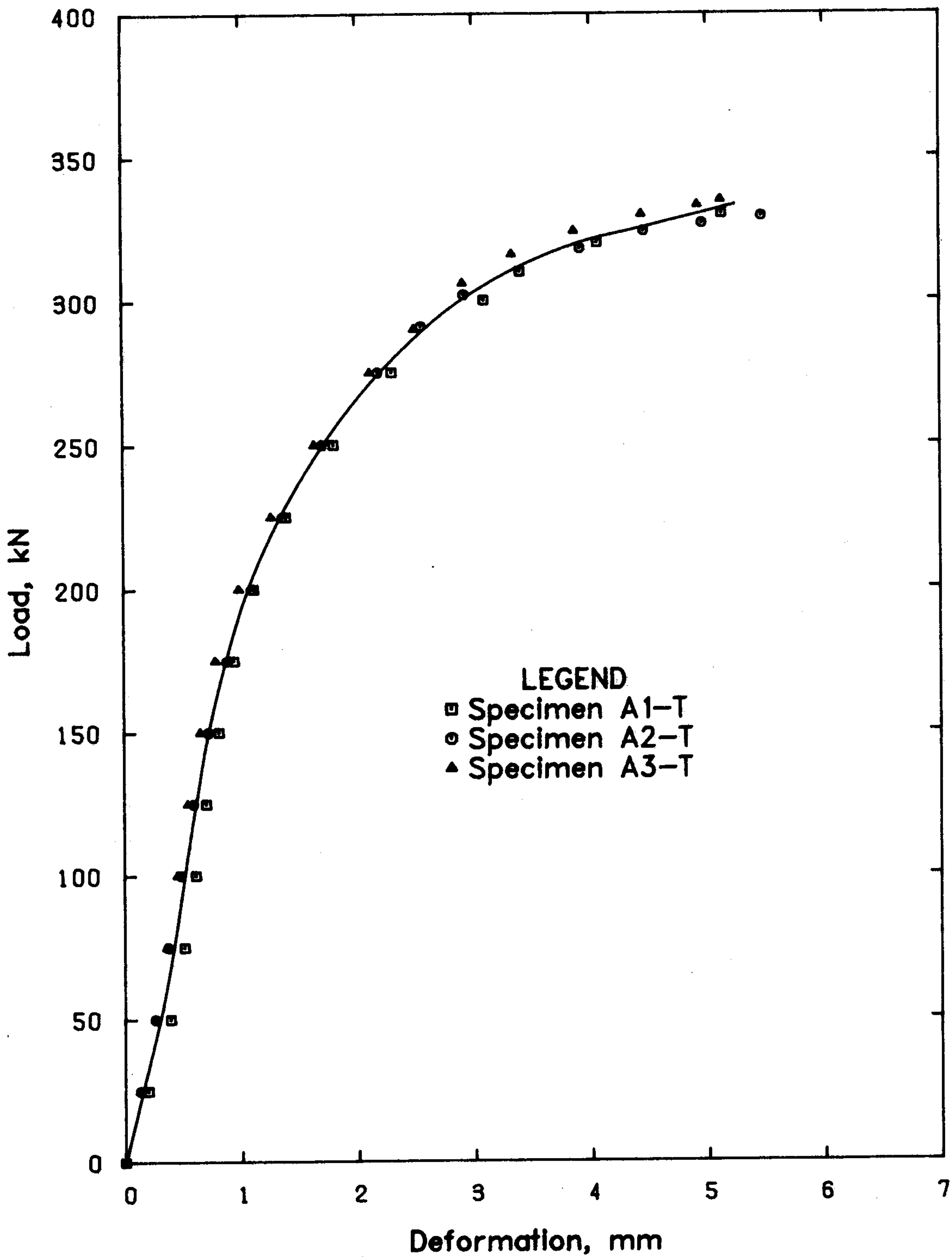


Figure 5.5 Shear Load Versus Shear Deformation Response of Series A Single Bolt Shear Specimens - Tension Jig

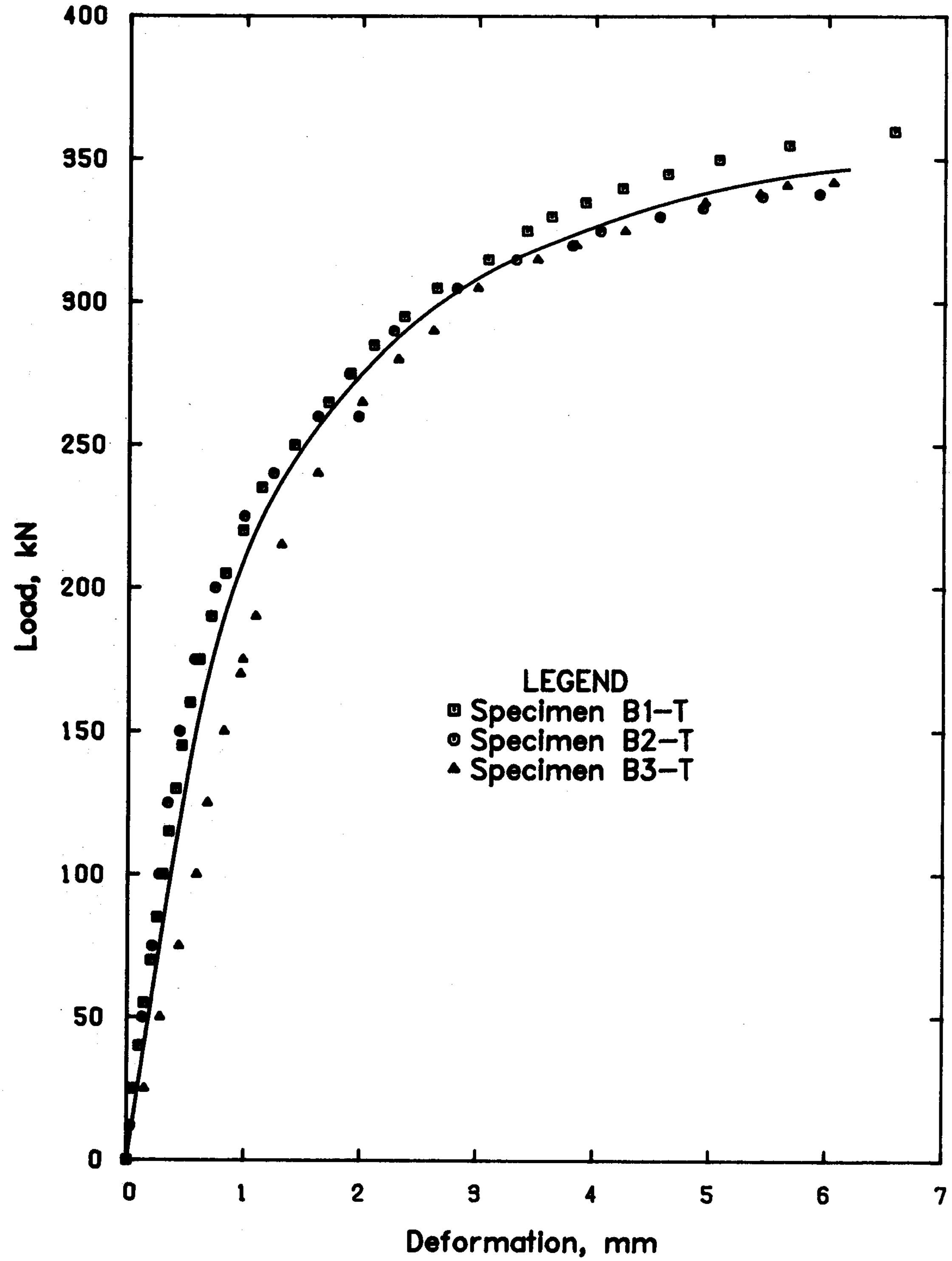


Figure 5.6 Shear Load Versus Shear Deformation Response of Series B Single Bolt Shear Specimens - Tension Jig

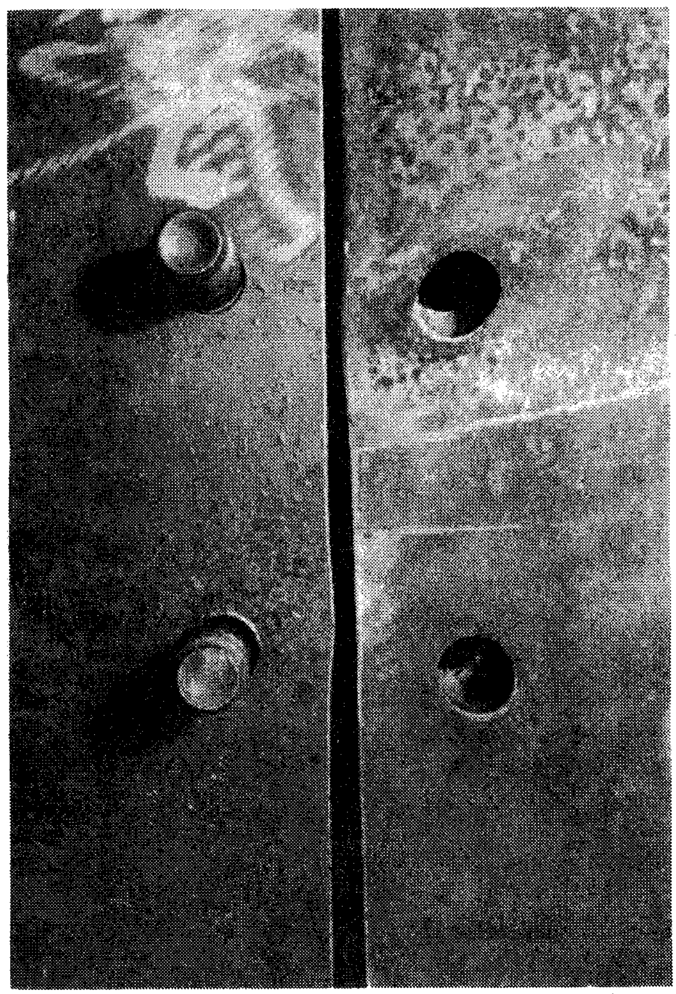


Figure 5.7 Permanent Web Deformation of a Failed Web Splice Specimen

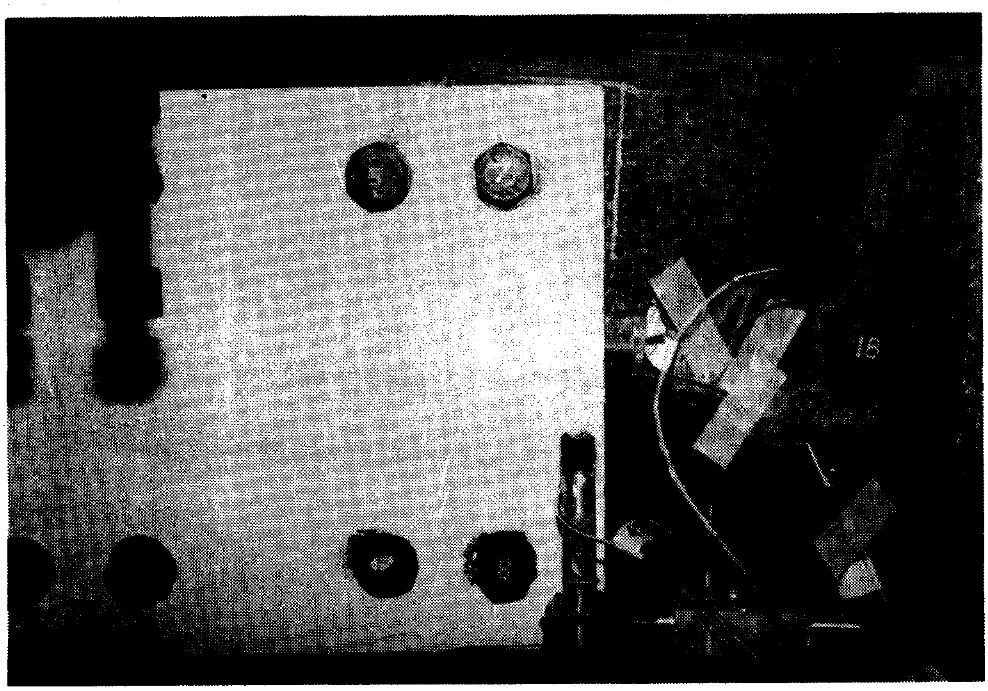


Figure 5.8 Local Splice Plate Yielding

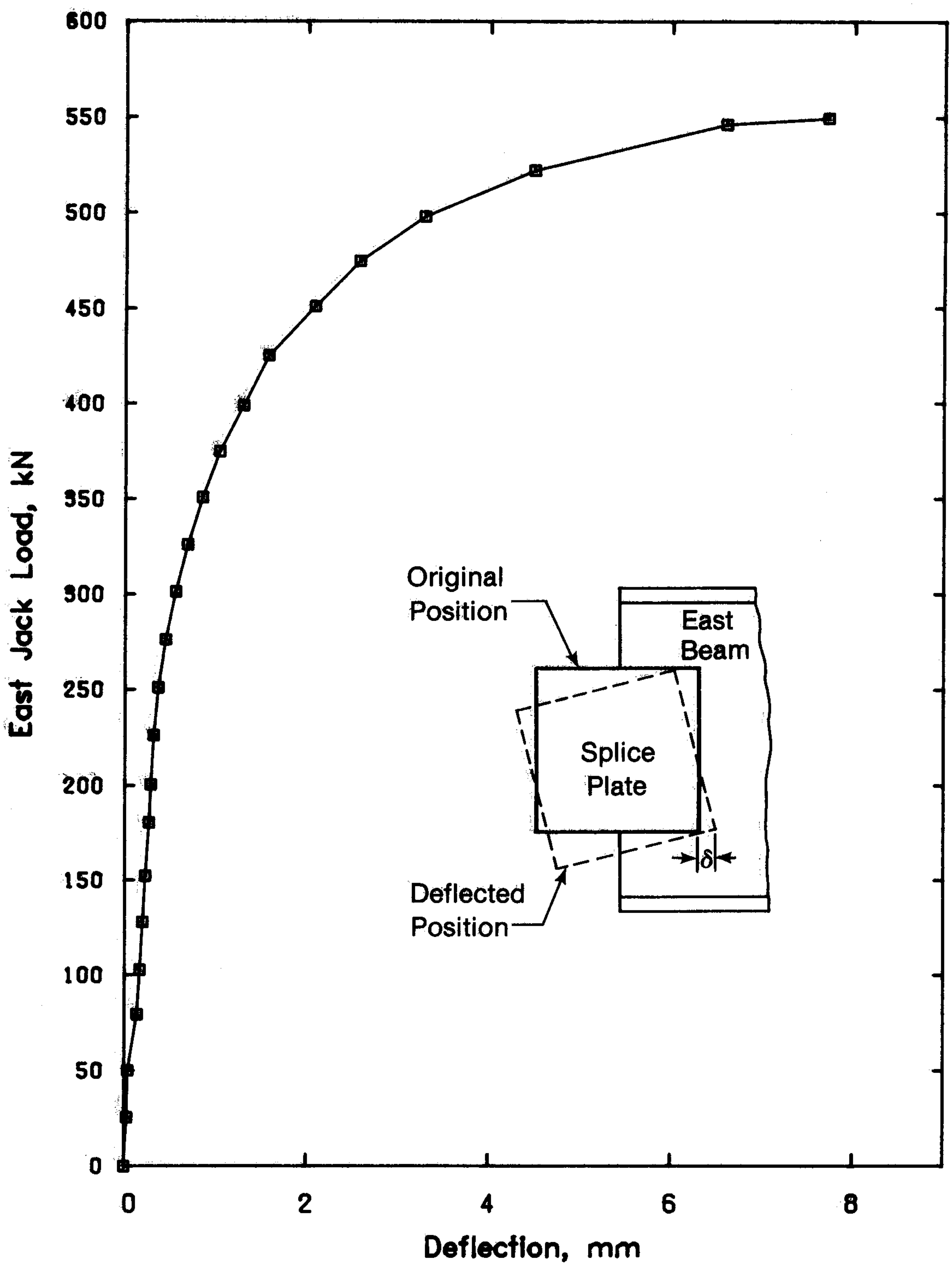


Figure 5.9 Typical Load Versus Splice Plate Deflection Response

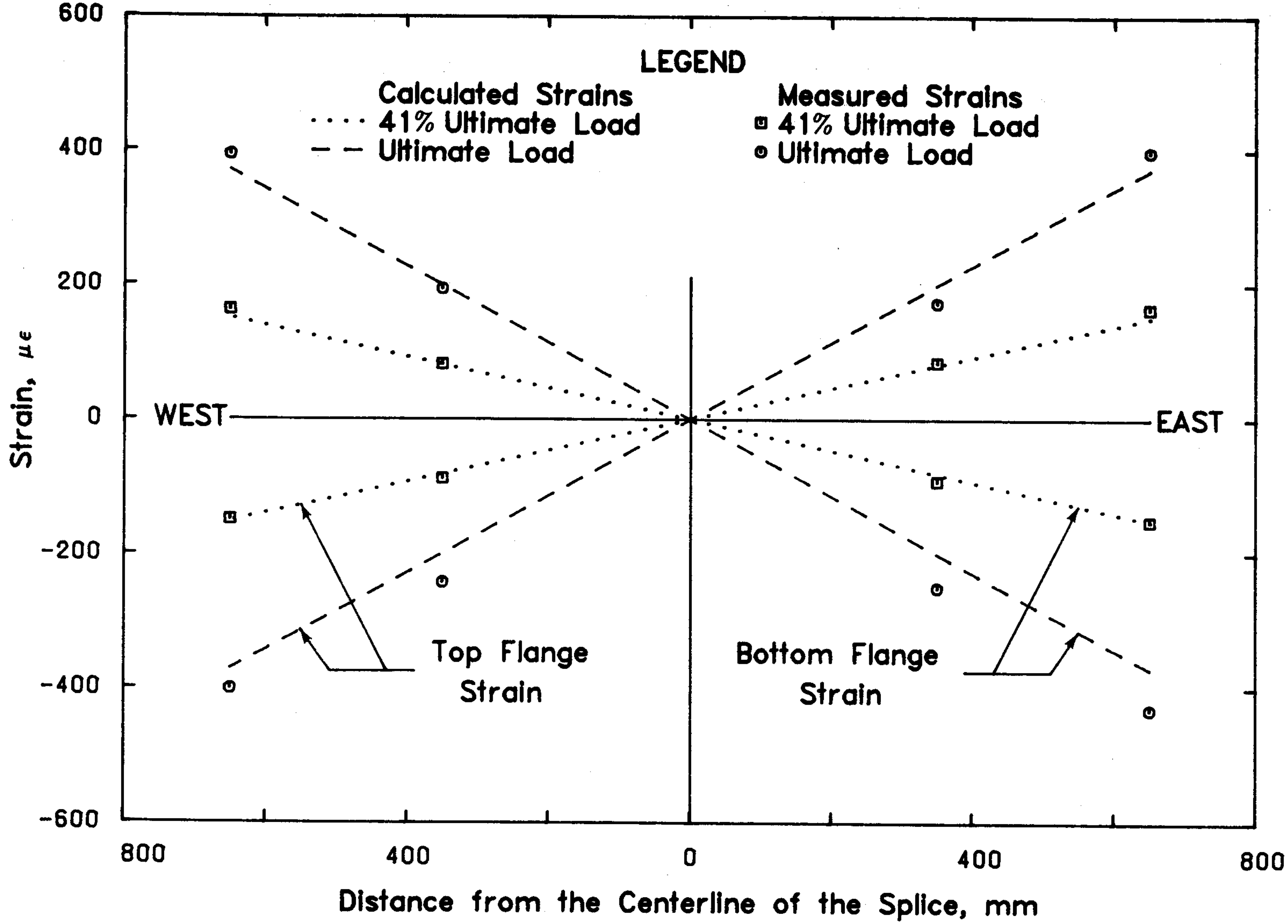


Figure 5.10 Flange Strains for Web Splice Specimen C1

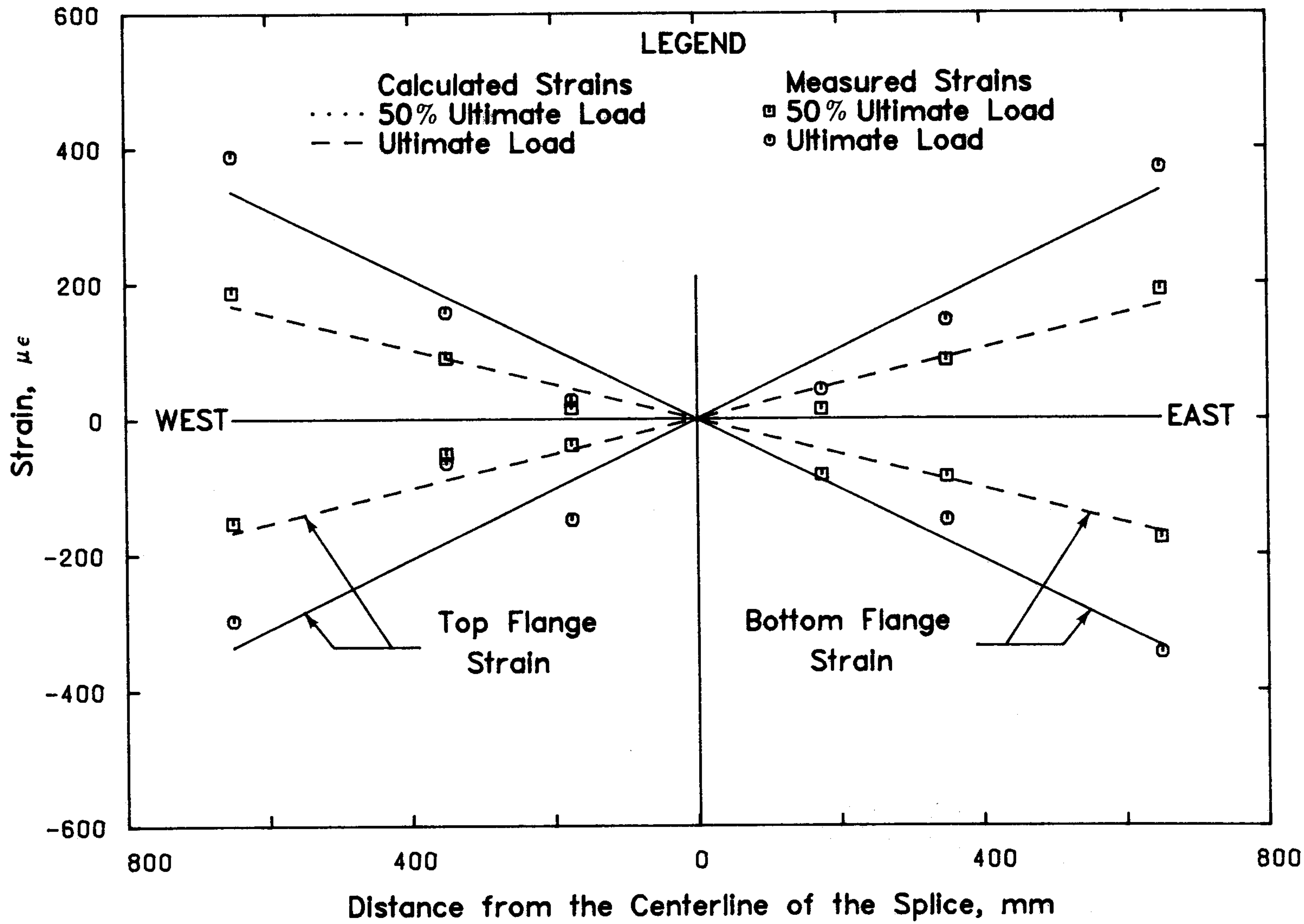


Figure 5.11 Flange Strains for Web Splice Specimen C2

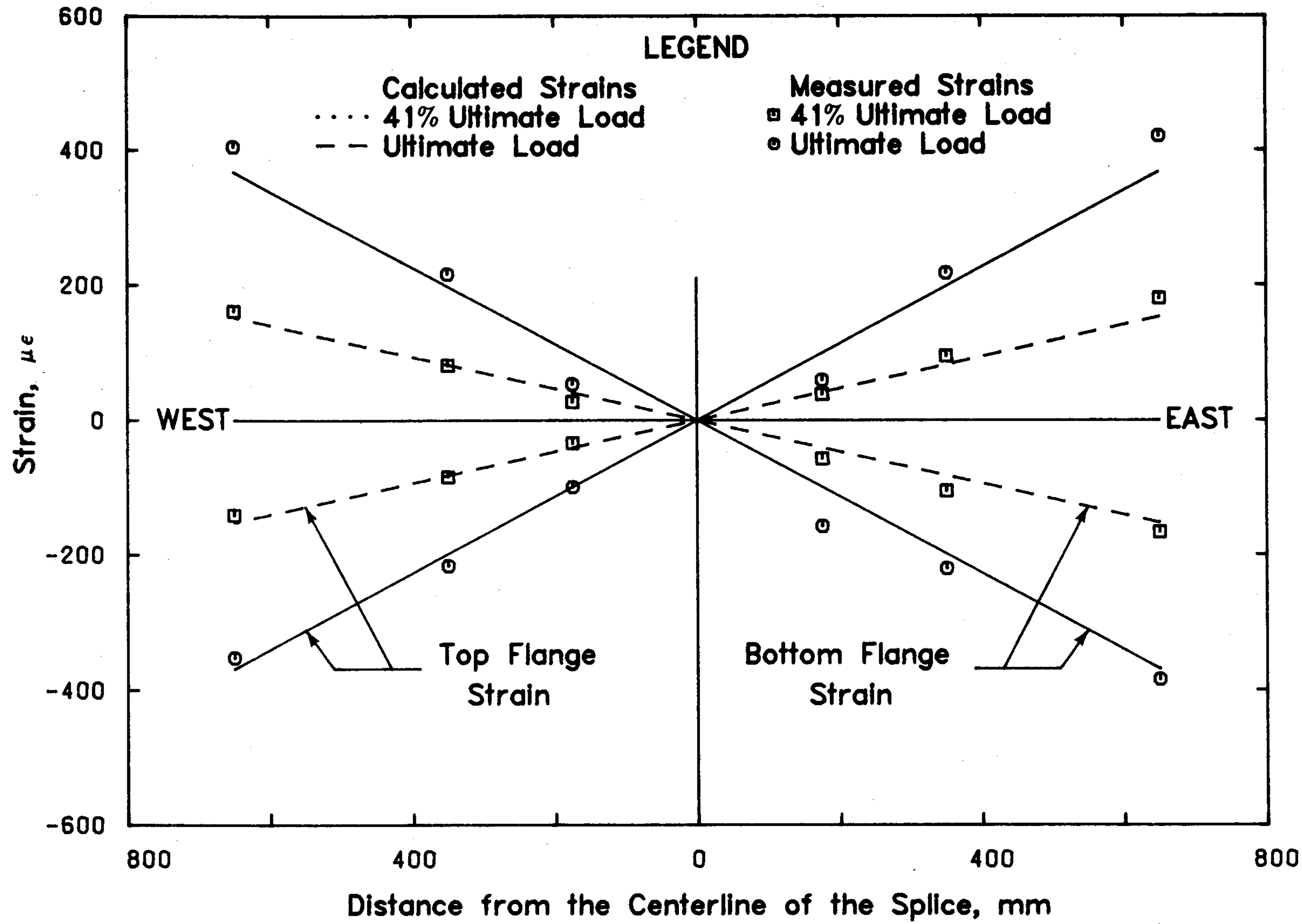


Figure 5.12 Flange Strains for Web Splice Specimen C3

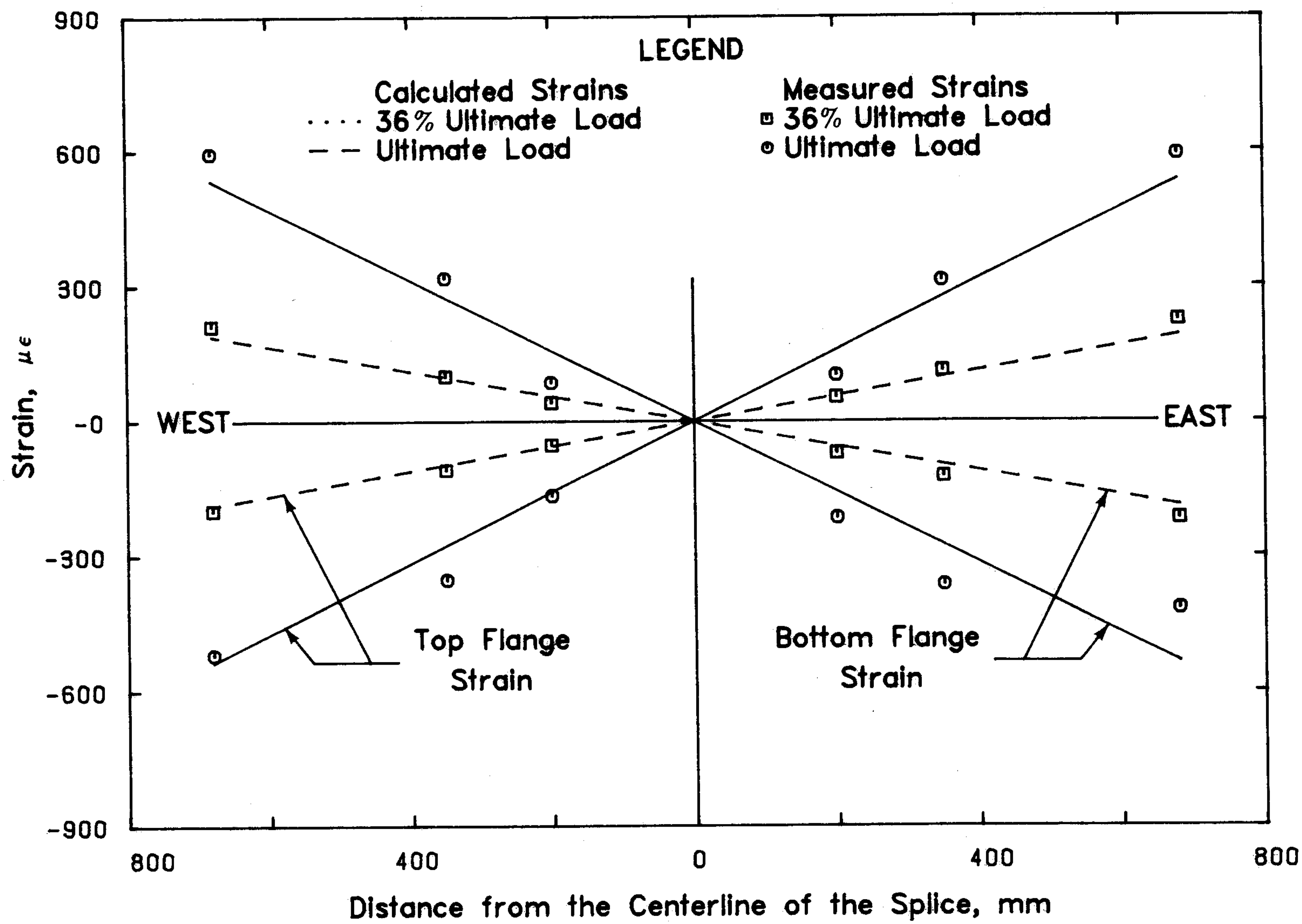


Figure 5.13 Flange Strains for Web Splice Specimen C4

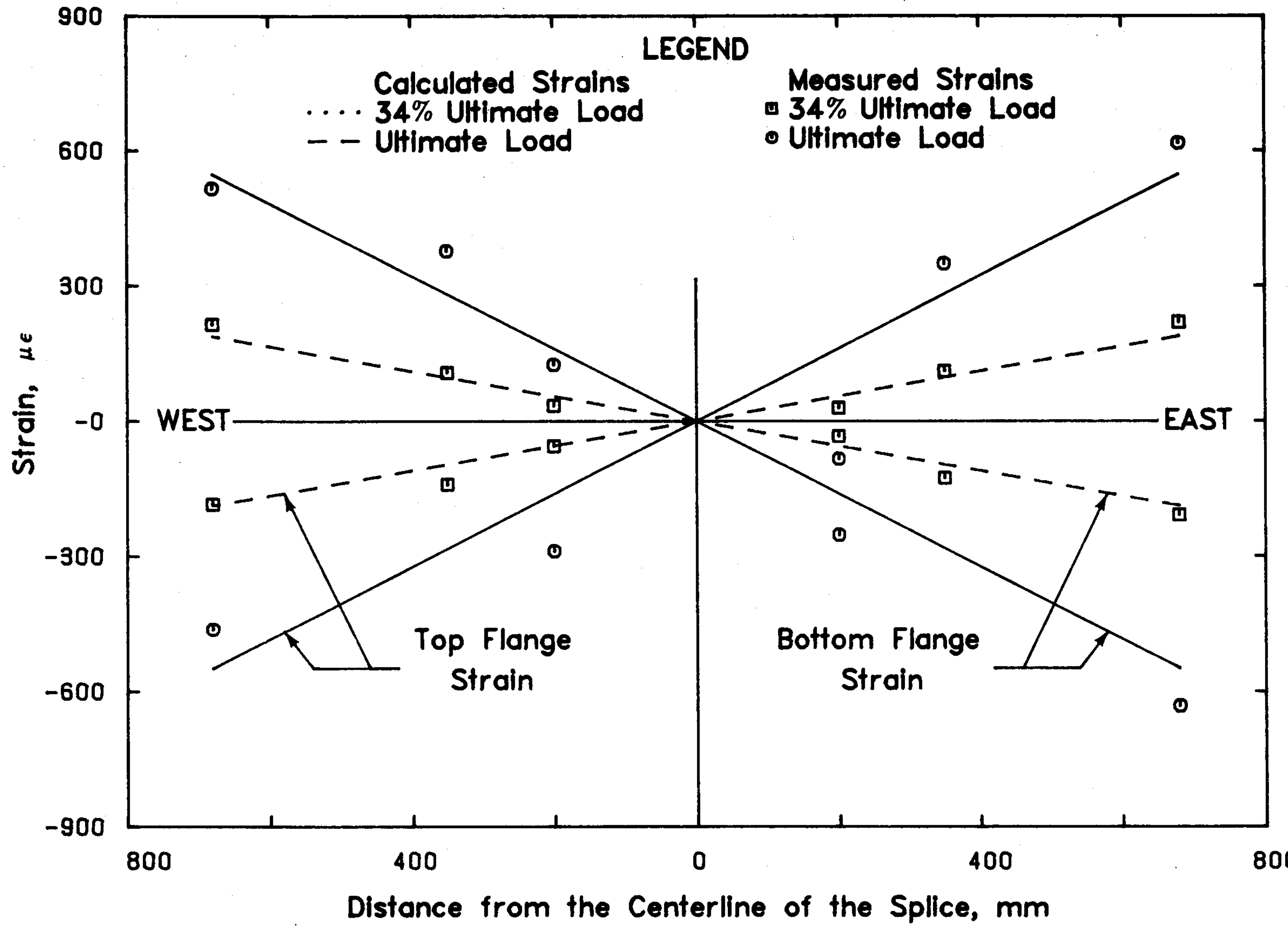


Figure 5.14 Flange Strains for Web Splice Specimen C5

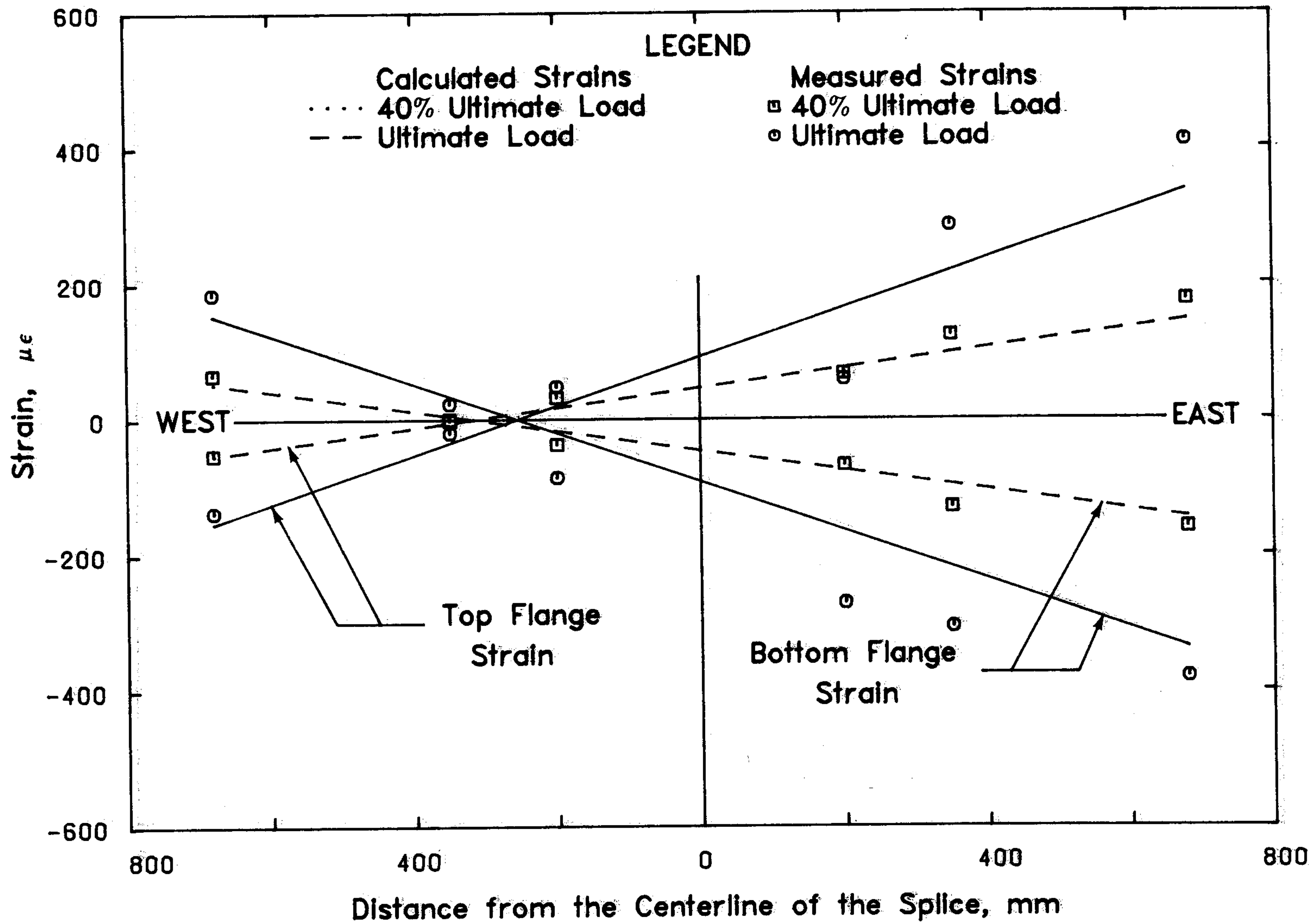
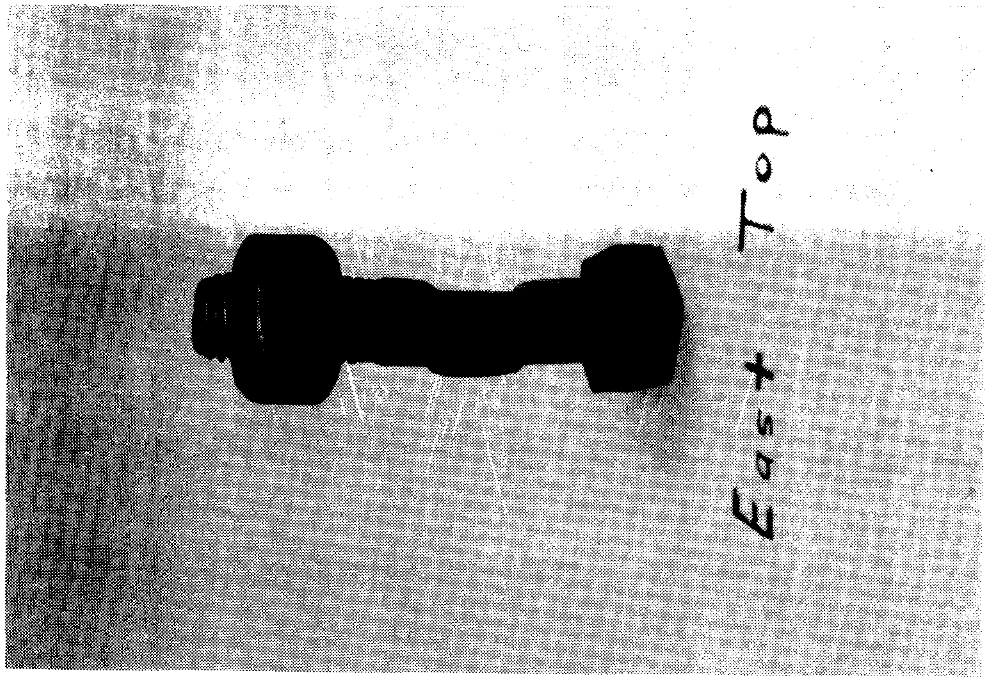
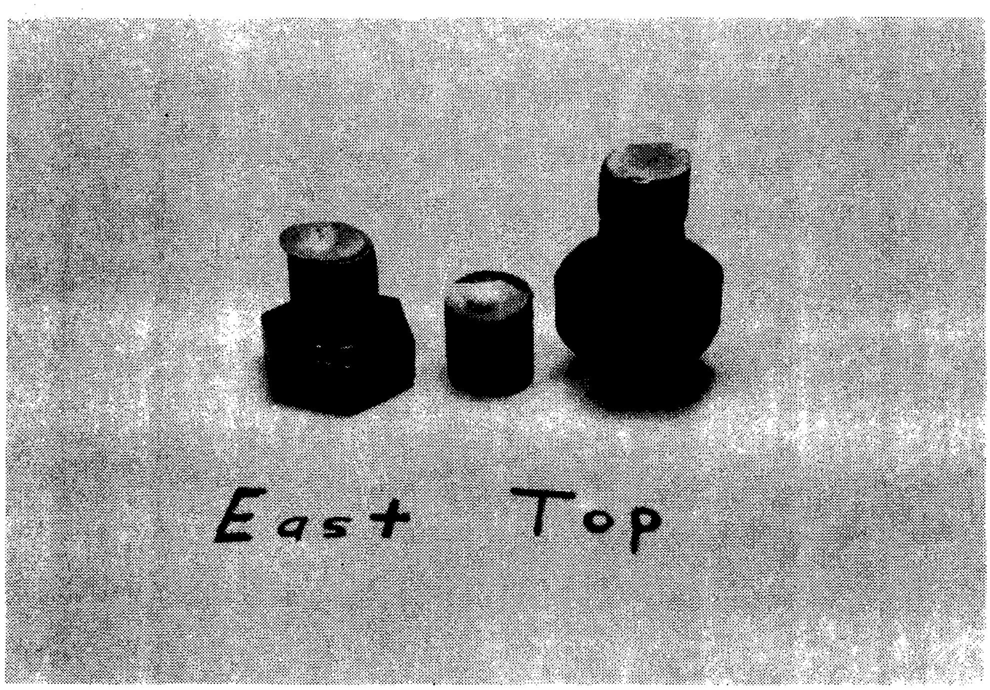


Figure 5.15 Flange Strains for Web Splice Specimen C6



(a)



(b)

Figure 5.16 Failed Web Splice Test Bolt

6. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 Summary and Conclusions

This investigation was undertaken to establish an analytical method for the design of a beam or girder web-flange splice. In order to substantiate the proposed analytical method, six large scale tests were conducted to determine the ultimate shear capacity of both bolted web splices located at points of contraflexure and a bolted web splice located in a region where both shear and moment were present.

In addition to the large scale web splice tests, two series of tests were carried out on single bolt specimens loaded in double shear in order to determine the shear load versus shear deformation response of a single bolt. The single bolt shear specimens were detailed so as to conform as closely as possible to the details in each of the full scale test configurations. The most significant difference between the two series of specimens was the thickness of the outside plates. For each series of single bolt specimens, the load response to shear was established using both compression and tension jigs.

A rational method of analysis that can identify the forces required for the design of a bolted web-flange splice was recently proposed by Kulak, *et al.* (16). This method was further developed in Chapter 3 of this report. For a web splice located at a point of contraflexure, the equilibrium equations derived in this method yield results identical to an analysis that treats the bolts on one side of the splice as loaded by a shear force acting at the centerline of the splice. For a web splice located at a point where both shear and moment are present, this method yields results identical to an analysis that treats the bolts on one side of the splice as acting under the moment at the centerline of the splice in addition to the transverse shear force acting at the centerline of the splice.

Analysis of the test results leads to the following conclusions:

1. The ultimate shear strength of high-strength bolts from the same production lot does not vary significantly when these bolts are tested using the same type of jig.
2. Variation in the ultimate deformation of single bolt shear specimens can be attributed mainly to variations in plate strength and thickness.
3. The ultimate shear strength of high strength bolts in plates that are loaded in tension is up to 10% less than the ultimate shear strength of high strength bolts in plates that are loaded in compression. The reduction in the ultimate shear strength is attributed to an increase in the axial bolt force. This effect may be a consequence of lap plate prying action.
4. The response curve for a single bolt loaded in shear that was used to develop the tables in the CISC Handbook (31) for analyzing eccentrically loaded bolted

connections is conservative compared with the test results obtained herein. The variation between the load response curve presented in the CISC Handbook and the load response curve for series B compression jig specimens is attributed to the differences in bolt and plate strengths. The shapes of the shear load response curves are similar and there is no well-defined yield point.

5. Using the actual response to shear load of a single bolt in a compression jig, analysis of the bolt group on one side of a web splice based on the assumption that the shear force acts at the centerline of the splice yields results that are consistent, but unconservative compared with test results.
6. The measured angles of deformation of the web bolt holes support the predictions of the bolt force directions obtained using the assumption that the shear force acts at the centerline of a web splice.
7. Using the actual response to shear load of a single bolt in a compression jig, analysis of the bolt group on one side of a web splice based on the assumption that the shear force acts at the centroid of the opposite bolt group yields results that are inconsistent and conservative when compared with test results. The analytical predictions are even more conservative when this assumption is used together with the actual response to shear load of a single bolt in a tension

jig.

8. For a bolted web splice, the best agreement between theory and experiment is achieved by using the ultimate strength method of analysis and the actual response to shear load of a single bolt in a tension jig, based on the assumption that the shear force acts at the centerline of the splice. For the specimens tested herein, the test to predicted ratio ranges from 0.92 to 1.00 using this design procedure.

2.2 Recommendations

1. The actual load response to shear of a single bolt tested in a tension jig provides a lower bound on the ultimate bolt strength. Therefore, it is recommended that eccentrically loaded bolted connections be designed using this load response rather than the currently used actual response to shear load of a single bolt tested in a compression jig.
2. Based on the experimental results obtained in this study, it is recommended that bolted web-flange beam or girder splices be designed using the equilibrium equations presented in Chapter 3.
3. Further tests of a beam splice in which both the web and the flanges are spliced at the same location is desirable.

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