FEARS STRUCTURAL ENGINEERING LABORATORY

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Final Report

COLUMN WEB AND FLANGE STRENGTH at END-PLATE CONNECTIONS

by

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ABSTRACT

Design procedures are developed for use in determining if column stiffeners are required for beam-to-column end-plate moment connections. The purpose of this study is to determine the effects of an end-plate on column behavior. The fabrication and installation of stiffeners is expensive and it was felt that the use of present design criteria for end-plate connections may result in an overly conservative design.

The study is broken into two sections. The first deals with the compression region of these connections. A literature survey was undertaken to study past work on this topic, a finite element analysis program was used to model the connection, several tests were performed and a recommended design procedure is presented.

The second section is a study of the tension region of end-plate connections. Again a literature study was undertaken, tension region tests were conducted, results summarized and final recommendations are presented.

COLUMN WEB AND FLANGE STRENGTH AT END-PLATE CONNECTIONS

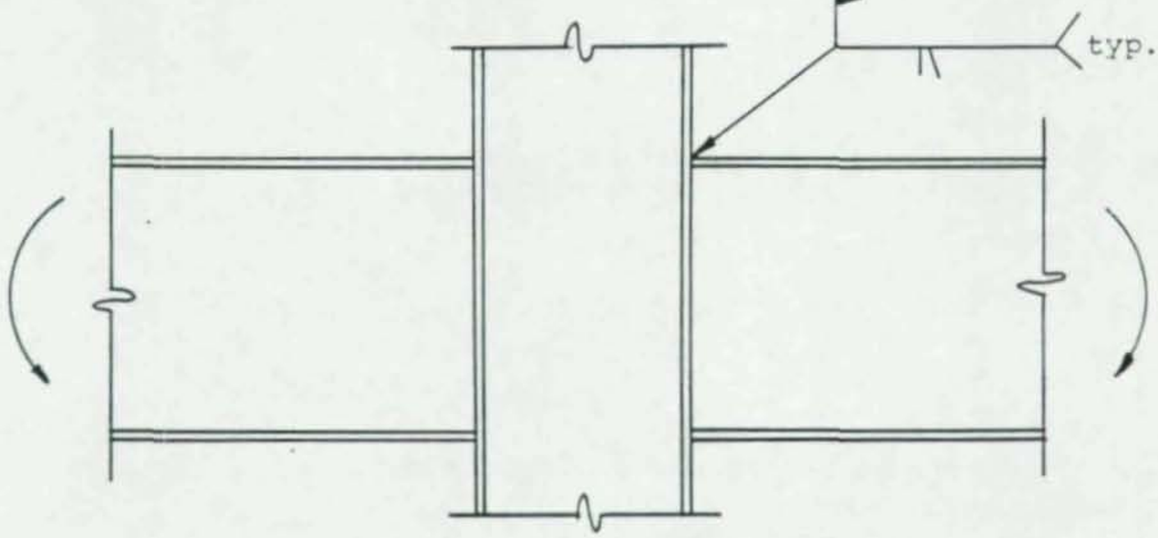
CHAPTER I

INTRODUCTION

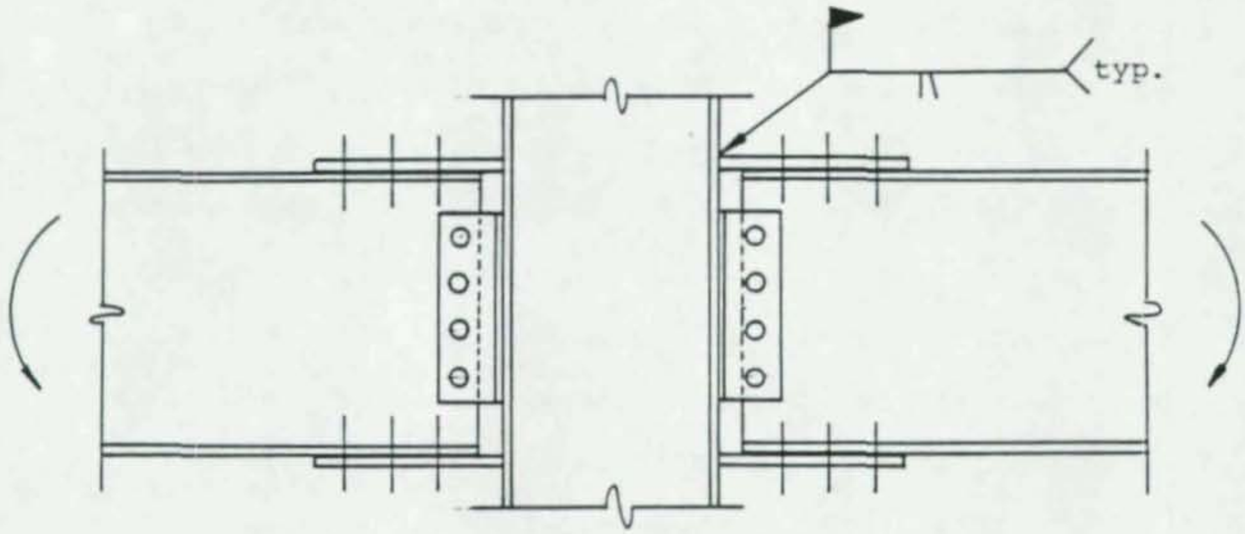
1.1 General

Current North American specifications provide design criteria to prevent local failure of H-shaped columns when flanges or moment connection plates are welded to the column flange as shown in Figure 1. This design criteria was developed strictly for these types of connections. Application of the criteria when end-plate connections are used, Figure 2, may result in the unnecessary use of column stiffeners opposite the beam flanges. The installation of column stiffeners is expensive. The stiffeners can interfere with weak axis framing into the column as shown in Figure 3, and erection is more difficult when stiffeners are installed. If stiffeners between the flanges of H-shaped columns can be eliminated the fabrication process is greatly simplified.

This study is concerned with column behavior at the type of connection shown in Figures 2 and 3 in which an end-plate is shop welded to the beam and then field bolted to the column flange.



(a) All Welded Connection



(b) Welded and Bolted Connection

Figure 1. Continuous Beam-to-Column Connections

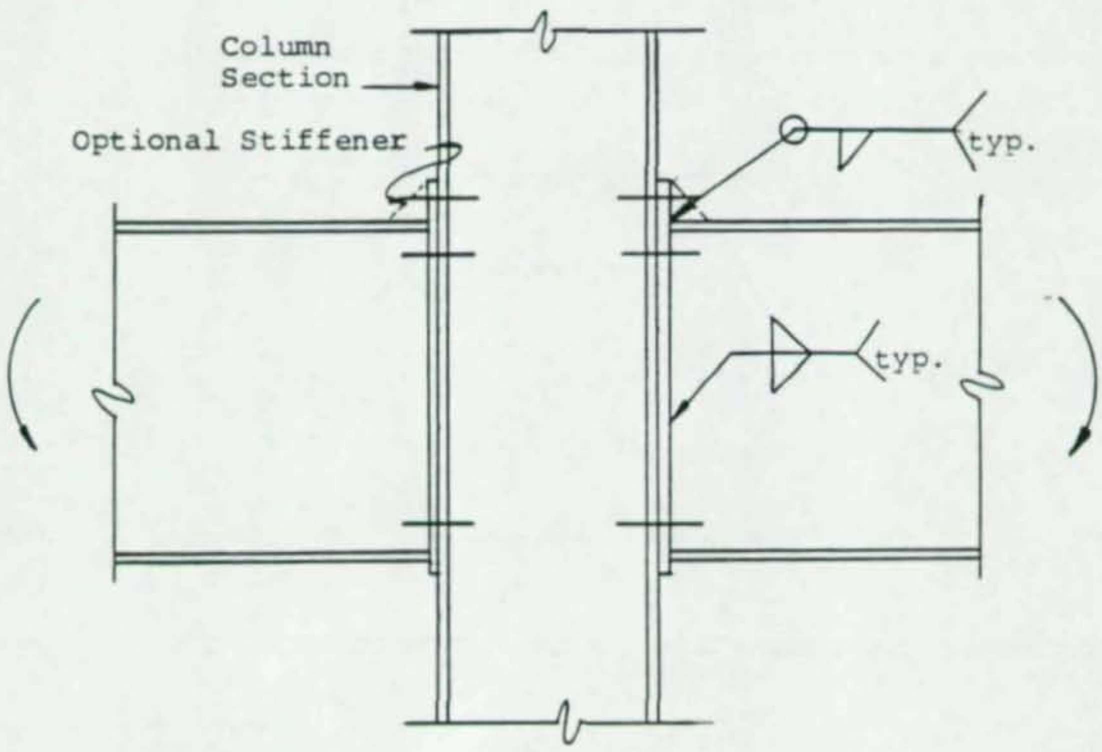
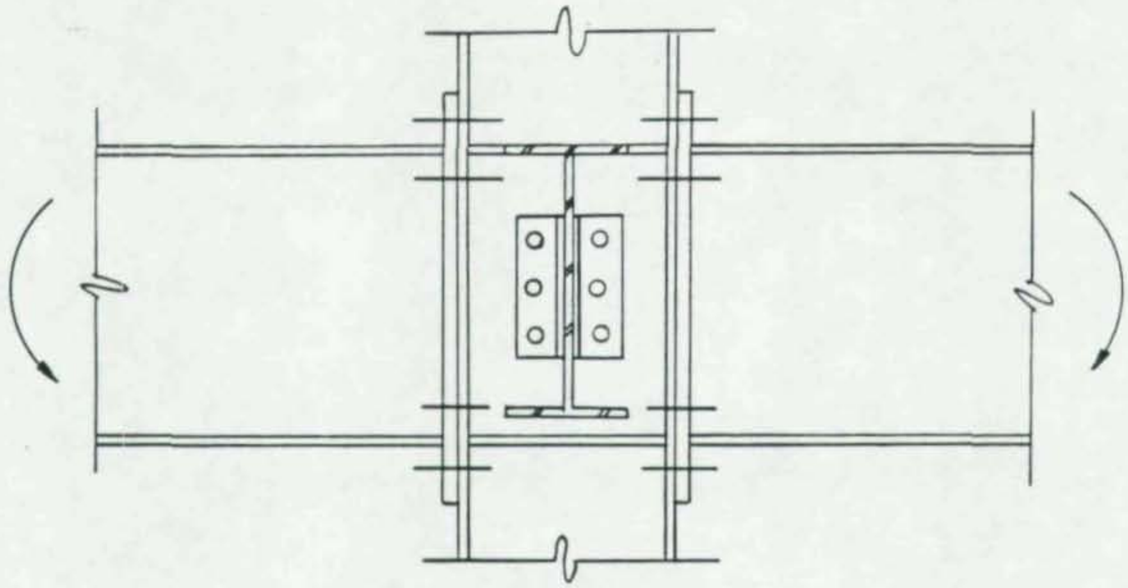
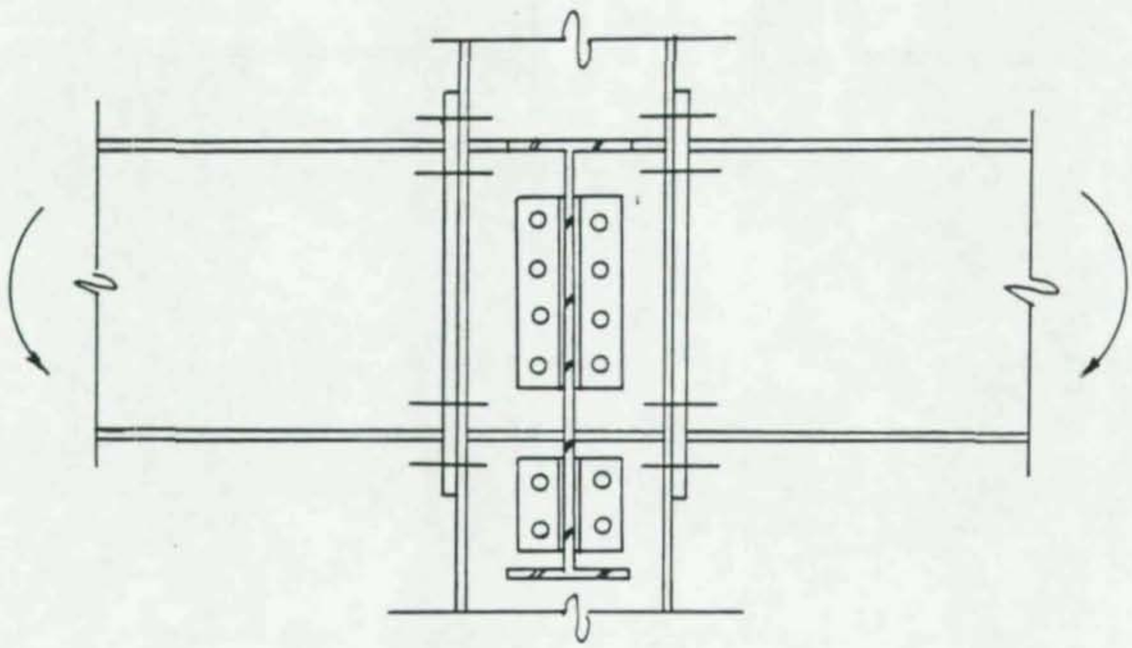


Figure 2. Typical End-Plate Moment Connection



(a) Shallow Beam



(b) Deep Beam

Figure 3. Weak Axis Framing Details

1.2 Objectives and Scope

The principal objective of this study is to develop design procedures for determining column flange and web strength at end-plate beam-to-column moment connections. The study is divided into two parts. The first part concerns column web compression strength opposite the beam compression flange. The investigation includes a literature review of previous work, experimental studies of compression region end-plate connections, two dimensional finite element analyses for comparison with experimental results, and finally a proposed design procedure.

The second part is concerned with the column tension region of end-plate beam-to-column connections and, more specifically, the effect of the connection on column flange strength. Here, a literature review was conducted, experimental studies were performed for comparison with literature findings, and a design procedure proposed.

CHAPTER II

COLUMN WEB COMPRESSION STRENGTH

2.1 Introduction

The critical section in the column compression region of beam-to-column moment connections is at the toe of the column web fillet. For design of welded connections, the present (1978) AISC Specification[1] criterion is based on a load path which is assumed to vary linearly on a 2½:1 slope from the beam flange through the column flange and fillet as shown in Figure 4. If the stress at this critical section exceeds the yield stress of the column material, a column web stiffener is required opposite the beam compression flange.

For the case of end-plate moment connections, the width of the stress pattern at the critical section may be considerably wider due to the insertion of the end-plate into the load path. The fillet weld connecting the beam and end-plate may also influence the width, as well as, end-plate stiffeners of the type shown in Figure 2. To verify or discredit these assertions, an extensive literature survey was conducted, followed by experimental and analytical studies. The result is a proposed design criterion for column web strength at end-plate connections. Details of the study are

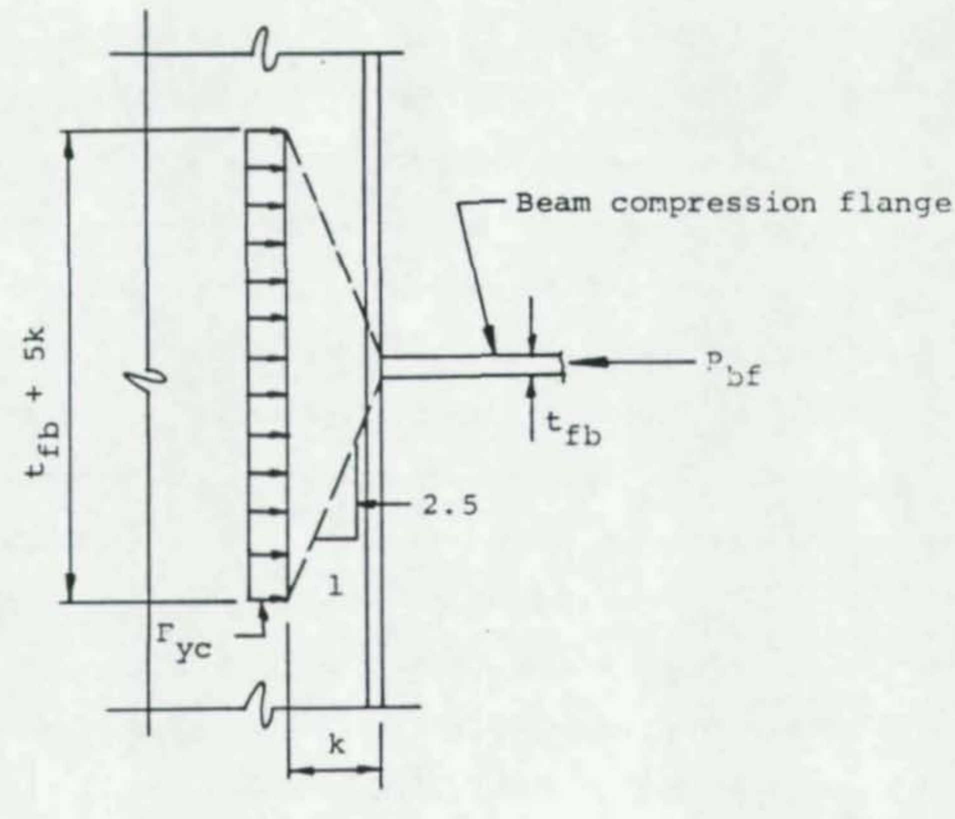


Figure 4. Distribution of Stresses at Beam Compression Flange

found in the following section.

2.2 Literature Review

Graham, Sherbourne, and Khabbaz[2] studied beam-to-column connections with the beam welded directly to the column flange (Figure 1a). Several tests were conducted with full two and four way connection set-ups. An examination of the results led to the adoption of a much simpler set-up in which the beam flange was simulated by a bar welded to the column flange and having the same dimensions as the beam flange, Figure 5. The specimens were tested in a 300 kip capacity universal testing machine with the column in a horizontal position. A W16x36 was simulated for the beam and several 8, 10, 12 and 14 inch wide flange sections were used as column sections.

This simulated beam flange connection neglected possible effects of column axial load and the effect of the compression from the beam web on the column web strength. It was also stated by the authors that the effect of the tension region of the web on the compression region is negligible if the tension region does not fail. It is not clear to the writers exactly what the authors meant by this statement.

Based on test results using the simulated beam flange, the authors conservatively suggest that the stress distributes through the column web on a 3½:1 slope. Use of this relationship results in the following equation for the maximum force

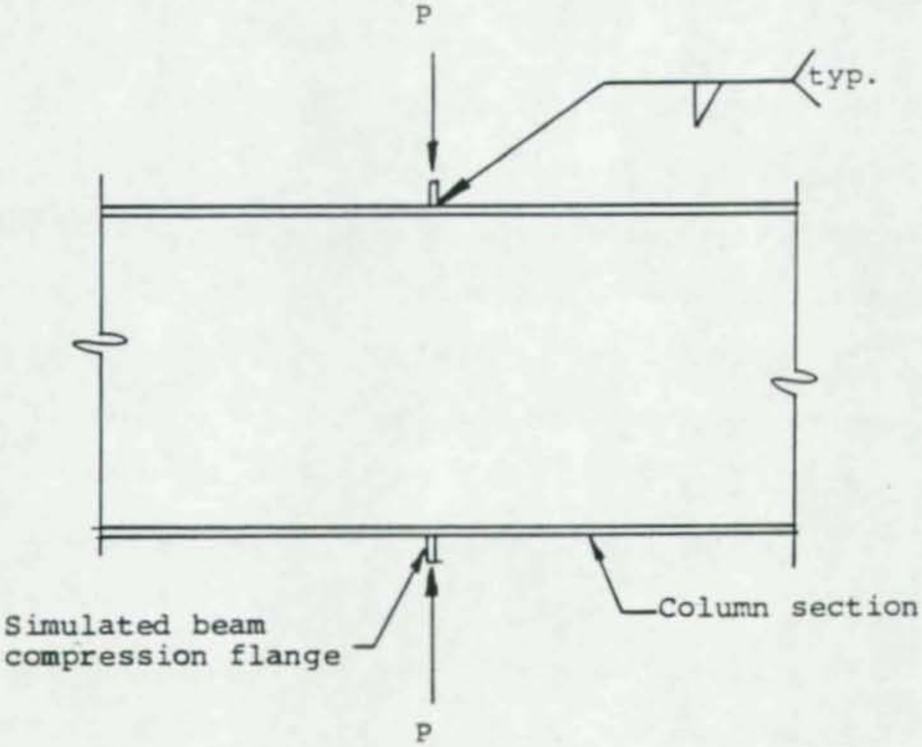


Figure 5. Test Set-up Used in Reference 2

which can be resisted by the column web,

$$P_{max} = F_{yc} t_{wc} (t_{fb} + 7k) \tag{1}$$

where P_{max} = maximum force the column web is capable of resisting (kips), F_{yc} = yield stress of column material (ksi), t_{wc} = column web thickness (in.), t_{fb} = beam flange thickness (in.), k = column "k" distance (in.). The authors state that Equation 1 is conservative when compared with test results obtained from the simulated beam flange compression region tests.

Because of additional compression supplied by the beam web, the full connection test set-up gave lower results than those obtained in the simulated beam flange test. According to the authors, if the stress is distributed on 2½:1 slope through the column, a conservative estimate for the full connection test is obtained. Hence,

$$P_{max} = F_{yc} t_{wc} (t_{fb} + 5k) \tag{2}$$

It follows that column stiffeners are not required adjacent to the beam compression flange if

$$t_{wc} \geq \frac{b_f t_{fb}}{t_{fb} + 5k} \tag{3}$$

where b_f = beam flange width (in.).

Newlin and Chen[3] attempted to develop a method of determining ultimate loads for the compression region of column sections having slender webs. In this study a slender web is defined as (This is the same definition as in Ref. 1.):

$$\frac{d_c}{t_{wc}} \geq \frac{180}{\sqrt{F_{yc}}} \tag{4}$$

where d_c = column web depth clear of fillets (in.). Further, they attempted to develop a single formula for predicting the maximum web capacity of a column section regardless of the d_c/t_{wc} ratio rather than separate equations for strength and stability.

Fifteen tests in several series were performed to investigate the effect of varying flange and loading conditions. In addition, results from tests conducted by Chen and Oppenheim[4] were also included in the study. One series of tests was used to investigate the effect of opposing beams of unequal depth at an interior beam-to-column moment connection. This geometry results in a situation where the loads applied to the compression region are eccentric. A second series was used to investigate the contribution of the column flange to the load carrying capacity of the column web. In this series, cover plates 1 in. thick, 20 in. long, and slightly wider than the specimen flanges to permit fillet welding all around were used. All tests were performed by simulating the beam flange by welding a bar to either the cover plate or directly to the column. Tests were conducted in the same manner as in Reference 2 with the column section placed horizontally in an 800 kip capacity mechanical testing machine.

Test results show that the ultimate load of a column web is "essentially unaffected by the eccentric load condition". It appears that eccentric loading has the effect of adding a small amount of stiffening to the stiffness of the web. It was concluded that Equation 2 is conservative for eccentric loading conditions.

To investigate the contribution of the column flange on column web buckling, W10x29 and W12x27 A36 column sections were first tested as control specimens. Plates, 1 in. by 20 in., were then welded to both flanges of each section and the tests repeated. The resulting load versus deflection curves for the W10x29 tests are shown in Figure 6. The increase in ultimate load with the cover plates added was approximately 31% for the W10x29 section and 33% for the W12x27 section. However, a reserve strength of only 4.8% existed for both sections at ultimate load with very limited ductility. As a result, the authors state that the "presence of a cover plate on a column flange should not be considered as part of the k dimension", and "these results further support the relative insignificance of the column flange thickness as compared with web dimensions."

It is noted that the P_{max} values shown in Figure 6 are based on measured dimensions and measured yield stress. The W10x29 control test k value was reported as 0.73 in. compared to a value of 1 1/16 in. from the 7th edition AISC Manual of Steel Construction[5]. No explanation was found for this rather large difference. The k value used to compute P_{max} for the cover plate test was taken as 1.73 in., the sum of the beam k and the 1 in. cover plate thickness.

Newlin and Chen[3] recommend that Equation 2 not be

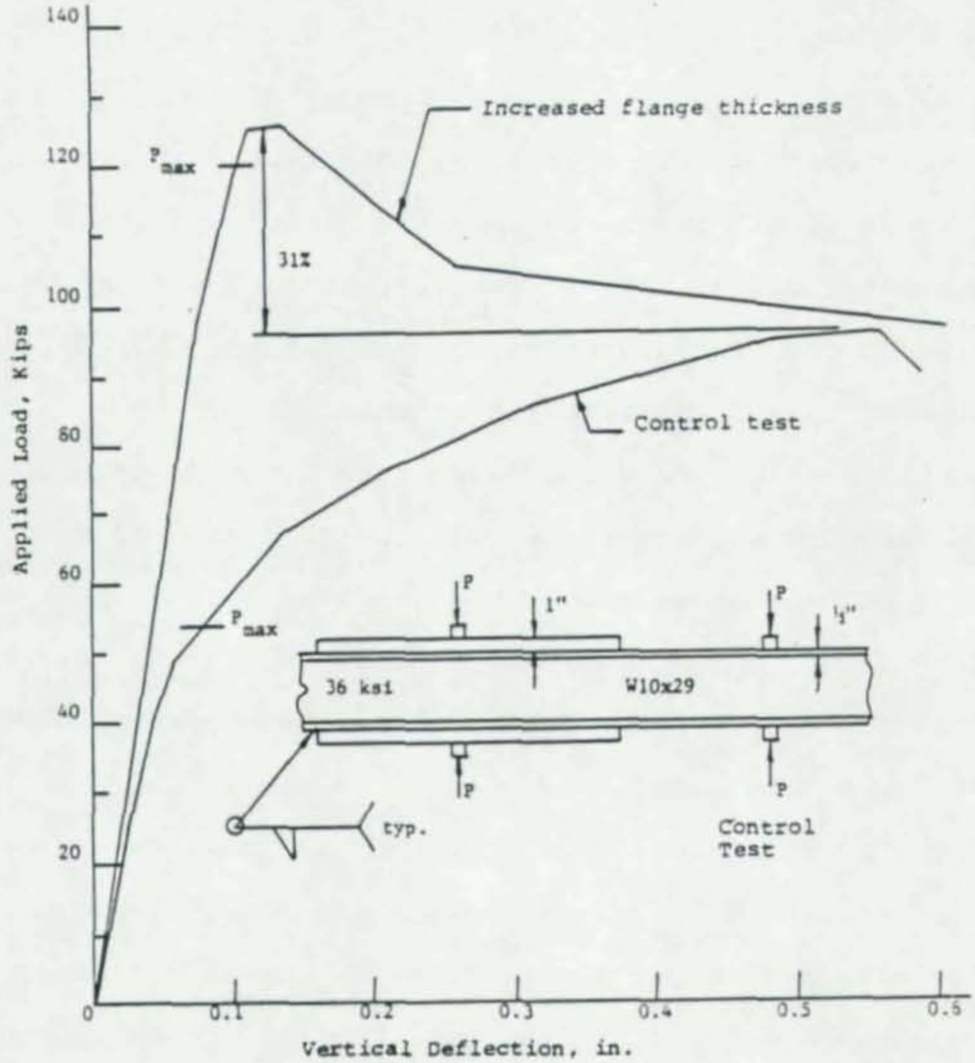


Figure 6. Effect of Cover Plate on Web Capacity
(From Reference 3)

used for design and that an interaction equation of the form

$$P_{max} = \frac{F_{yc}^{3/2} d_c}{180} \left(\frac{125t_{wc}}{\sqrt[4]{F_{yc}}} - d_c \right) \quad (5)$$

be used to check both web strength and stability. Or, in lieu of Equation 5, a strength check be made using Equation 2 and a stability check using

$$P_{max} = \frac{4100t_{wc}^3 \sqrt{F_{yc}}}{d_c} \quad (6)$$

Mann and Morris[6] reviewed the results of several research programs pertaining to column webs at end-plate connections and proposed design criteria. It was stated in their report that the 1977 edition of the "European Convention of Constructional Steelwork Recommendations for Steel Construction" provides the following expression for the maximum load carrying capacity of the column web in the presence of an end-plate.

$$P_{max} = F_{yc}\, t_{wc}\, (t_{fb} + 5k + t_e + d) \quad (7)$$

where t_e = end-plate thickness (in.), and d = projection of the end-plate beyond the compression flange of the beam but not greater than t_e (in.) No test data is provided and it is not clear how this equation was developed. The expression is based on the assumption that the stress is distributed on a 1:1 slope through the end-plate and on a 2½:1 slope through the column.

Witteveen, Start, Bijlaard and Zoetemeijer[7] conducted tests in the Netherlands in an attempt to develop design rules to compute the moment capacity of unstiffened welded (no end-plate) and bolted (end-plate) connections. Both full connection tests and simulated compression flange tests, similar to that reported above, were conducted. Specific beam and column sizes used in the testing program are not given in the paper.

For beams welded directly to the column flange, it is recommended by Witteveen et al, that the column web strength be calculated from

$$P_{max} = F_{yc}\, t_{wc}\, \{t_{fb} + 5(t_{fc} + r_c)\} \tag{8}$$

where t_{fc} = column flange thickness (in.), and r_c = fillet between the flange and the web of the column (in.). For bolted end-plate connections, it is recommended that the end-plate and weld be considered in determining the ultimate load carrying capacity of the web

$$P_{max} = F_{yc}\, t_{wc}\, \{t_{fb} + 2\sqrt{2}\,a + 2t_e + 5(t_{fc} + r_c)\} \tag{9}$$

where a = weld dimension (in.). In this expression the stress distribution is assumed to be 1:1 through both the weld and end-plate. Test results are not given, but it is stated that Equations 8 and 9 are lower bound solutions for the "failure load (buckling, crippling or yielding of the web in compression) obtained from tests on European rolled sections".

Aribert, Lachal and Vawawy[8] tested European column sections HEB 140, 200 and 300, Figure 7, with end-plate connections[1]. The compression beam flange was simulated by welding a bar to the end-plate. The test set-up was similar to that used in Reference 2. From tests conducted with the end-plate thickness fixed at 15 mm (0.591 in.) and the length of plate varied, little change was found in results for practical variations in length. The end-plate length was then fixed at 150 mm (5.91 in.) and the thickness varied from 10 to 30 mm (0.394 to 1.181 in.). Plate thickness was found to play a predominant role in web strength induced by the end-plate.

In addition to the testing, a numerical model was developed in which the column flange was modeled as a beam resting on many equally spaced springs of equivalent stiffness. Maximum elastic, plastic and ultimate load expressions were obtained from the model. Each equation was parabolic in form. Comparison with test results showed there was little to gain from the parabolic form and a set of linear expressions was proposed based on test results:

Maximum elastic load

$$P_{e_{max}} = F_{yc}\, t_{wc}\, (t_e + 2.3k) \qquad (10)$$

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[1]This paper is in French. A translation of a pertinent section is found in Reference 9.

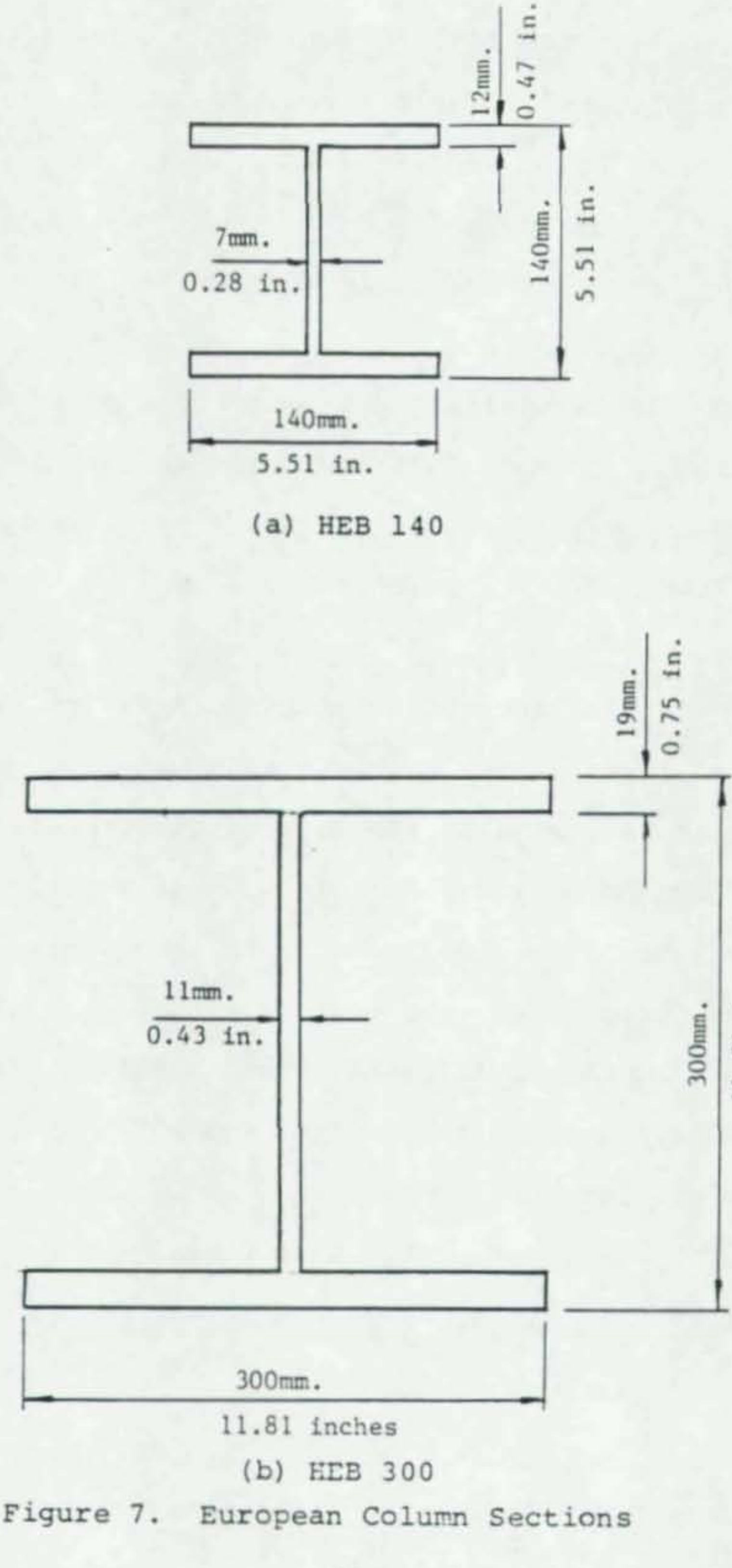


Figure 7. European Column Sections

Maximum plastic load

$$P_{p_{max}} = F_{yc}\, t_{wc}\, (2t_e + 5k) \tag{11}$$

Ultimate load

$$P_{u_{max}} = F_{yc}\, t_{wc}\, (6t_e + 7k) \tag{12}$$

It is stated that these equations are valid only if the flange thickness is less than approximately twice the end-plate thickness. In these equations, the effect of the weld is neglected.

Section 1.15.5.2 of the 1978 AISC Specification [1] specifies the required stiffener area to prevent column web crippling when flanges or moment connection plates for end connections of beams and girders are welded to the flange of H-shaped columns as

$$A_{st} = \frac{P_{bf} - F_{yc}\, t_{wc}(t_{fb} + 5k)}{F_{yst}} \tag{13}$$

where A_{st} = stiffener area (in.^{2}), P_{bf} = the computed force delivered by the flange or moment connection plate multiplied by 5/3, when the computed force is due to live and dead load only, or by 4/3, when the computed force is due to live and dead load in conjunction with wind or earthquake forces (kips), and F_{yst} = stiffener yield stress (ksi). Stiffeners are not required if A_{st} is negative. In the commentary on Section 1.15.5, it is stated that the actual force times the load factor, i.e. P_{bf}, need not exceed the area of the flange or connection plate delivering the force times the yield strength of the material.

In addition to Equation 13, a column web stability check is required. Compression flange stiffeners are required if the column web depth clear of fillets, d_c, is greater than

$$\frac{4100 t_{wc}^{3} \sqrt{F_{yc}}}{P_{bf}} \tag{14}$$

which is the same expression as recommended by Newlin and Chen[3] (Equation 6 of this report).

No mention of end-plate connections is made in the 1978 AISC specification. In an end-plate design example in the 8th edition AISC Manual of Steel Construction[10], page 4-115, it is suggested that end-plate effects can be "conservatively" accounted for by assuming a stress distribution on a 1:1 slope through the end-plate. This would result in the following equation for stiffener area

$$A_{st} = \frac{P_{bf} - F_{yc} t_{wc} (t_{fb} + 5k + 2t_e)}{F_{yst}} \tag{15}$$

No mention is made of possible weld effects and it is not known if the assumed distribution is based on test results.

In the literature survey, only two tests were found where end-plates effects were considered with American sections[3]. From these tests it was concluded that cover plates (end-plates) were not effective because of minimal reserve strength above first yield (<5%) and lack of ductility in the connection. However, in these tests the yield level was increased to a force level equivalent to a stress distribution based on a 2½:1 slope through the end-plate and column web.

Literature concerning European testing and design practice are consistent in recommending an assumed stress distribution through the end-plate of 1:1. In addition, one paper recommends the use of the beam flange to end-plate weld dimension when calculating the length of the critical section for determining the column web compressive strength.

To study possible effects of the end-plate contribution to column web compressive strength on the need for column stiffeners, W14 column sections in combination with beam sections were analyzed using different stress distributions through the end-plate. Only commonly used W18, 21, 24, 27, 30, and 33 beam sections were considered. Calculations were made for no end-plate, and distributions through the end-plate of 2½:1 and 1:1, In all cases the critical section was taken as at the "k" distance as determined from Reference 10. End-plate thickness was determined using the design procedure in Reference 10 for connections with four bolts at the tension flange. All calculations were for a yield stress of 36 ksi.

For each beam/column combination, a critical stress was calculated using

$$f = \frac{P_{bf}}{(t_{fb} + 5k)t_{wc}} \tag{16}$$

for the no end-plate condition,

$$f = \frac{P_{bf}}{(t_{fb} + 5k + 2t_e)t_{wc}} \tag{17}$$

for the 1:1 distribution, and

$$f = \frac{P_{bf}}{\{t_{fb} + 5(k+t_e)\}t_{wc}} \tag{18}$$

for the 2½:1 distribution. The beam flange force, P_{bf}, was determined in all cases from

$$P_{bf} = \frac{5M}{3(d-t_{fb})} \tag{19}$$

with

$$M = F_b S_x \tag{20}$$

and

$$F_b = 0.66F_y \tag{21}$$

where F_b = allowable bending stress (ksi), S_x = strong axis beam section modulus (in.^{3}), M = allowable moment capacity (in. kips), d = beam depth (inches), and other terms are defined as previous. Results are shown in Tables B.1, B.2 and B.3 of Appendix B. In these tables, a column stiffener is required if the critical stress shown is greater than 36 ksi.

The effect of each stress distribution condition on stiffener requirements is also shown in Table 1. Here, the

Table 1

Heaviest W-Sections Not Requiring Stiffeners with W14 Columns

W14 Column Section, A36 Steel

	End Condition	W14x 90	W14x 99	W14x 109	W14x 120	W14x 132	W14x 145	W14x 159	W14x 176	W14x 193	W14x 211	W14x 233	W14x 257	W14x 238
W18	No end-plate		18x35	18x40	18x46	18x50	18x55	18x65	18x71	18x76	18x97	18x106	18x119	18x119
	End-plate (1:1)	18x35	18x40	18x46	18x55	18x60	18x71	18x71	18x86	18x106	18x119	18x119	18x119	18x119
	End-plate (2½:1)	18x46	18x50	18x65	18x71	18x76	18x86	18x106	18x119	18x119	18x119	18x119	18x119	18x119
W21	No end-plate				21x44	21x50	21x50	21x65	21x68	21x68	21x68	21x68	21x68	21x68
	(1:1)			21x50	21x50	21x62	21x68	21x68	21x68	21x68	21x68	21x68	21x68	21x68
	(2½:1)	21x50	21x50	21x62	21x68	21x68	21x68	21x68	21x68	21x68	21x68	21x68	21x68	21x68
W24	No end-plate					24x55	24x62	24x68	24x76	24x84	24x94	24x94	24x94	24x94
	(1:1)				24x55	24x62	24x68	24x76	24x94	24x94	24x94	24x94	24x94	24x94
	(2½:1)		24x55	24x62	24x76	24x84	24x94	24x94	24x94	24x94	24x94	24x94	24x94	24x94
W27	No end-plate									27x84	27x94	27x94	27x94	27x94
	(1:1)							27x84	27x94	27x94	27x94	27x94	27x94	27x94
	(2½:1)					27x84	27x94	27x94	27x94	27x94	27x94	27x94	27x94	27x94

Table 1 (continued)

Heaviest W-Sections Not Requiring Stiffeners with W14 Columns

W14 Column Section, A36 Steel

	End Condition	W14x 90	W14x 99	W14x 109	W14x 120	W14x 132	W14x 145	W14x 159	W14x 176	W14x 193	W14x 211	W14x 233	W14x 257	W14x 238
W30	No end-plate										30x108	30x116	30x124	30x124
	(1:1)								30x99	30x108	30x124	30x124	30x124	30x124
	(2½:1)							30x108	30x116	30x124	30x124	30x124	30x124	30x124
W33	No end-plate											33x118	33x130	33x130
	(1:1)										33x130	33x130	33x130	33x130
	(2½:1)								33x130	33x130	33x130	33x130	33x130	33x130

heaviest beam not requiring stiffeners for a specific W14 column section is shown. In general, the use of a 1:1 distribution through the end-plate changes the results by one section and the 2½:1 distribution by a minimum of two sections.

From these results it appears that sufficient column web stiffeners can be eliminated through the use of the end-plate contribution as to warrant further study.

With these results and the European recommendations, a study to accurately determine the contribution of end-plate thickness to column web strength appeared to be warranted. Consequently, both analytical and experimental studies were condcuted using American sections. Details are presented in the following sections.

2.3 Finite Element Analyses

To analytically determine stress distributions and yield patterns in the compression region of the column web at end-plate connection, an inelastic, two-dimensional, finite element program developed by Iranmanesh[11] was used. To reduce computational costs and to more closely model the test set-up used in the experimental phase of the study, only a portion of the beam consisting of the flange and web was used. Load was applied directly to the beam flange. Figure 8 shows a typical mesh, support conditions and loading. Smaller elements were used in the region on the web at approximately the

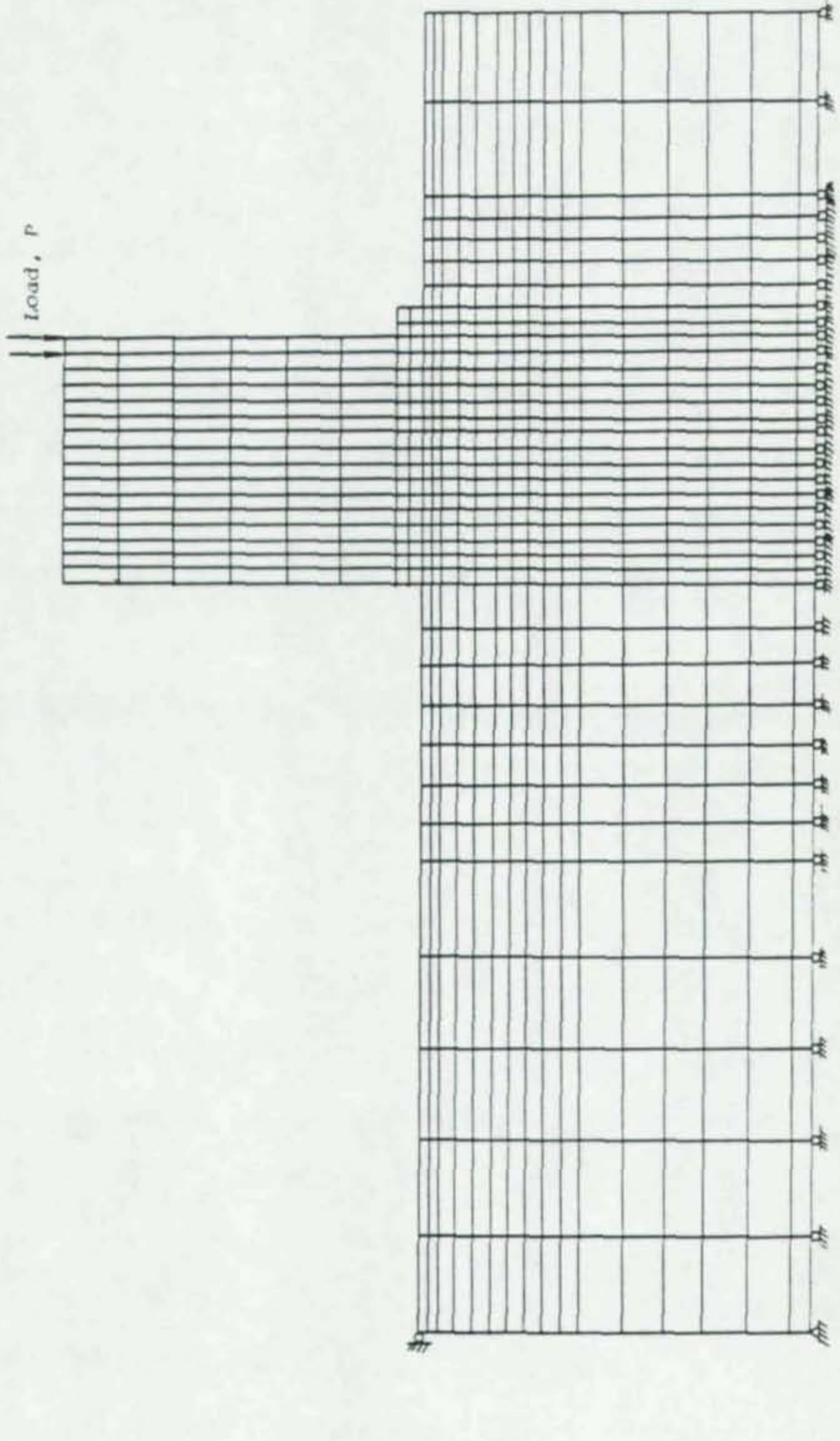


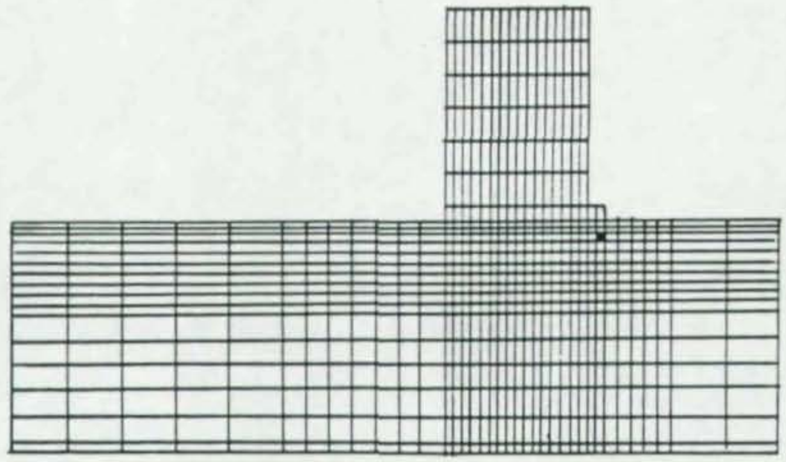
Figure 8. Typical Finite Element Model

"k" distance from the edge of the column flange.

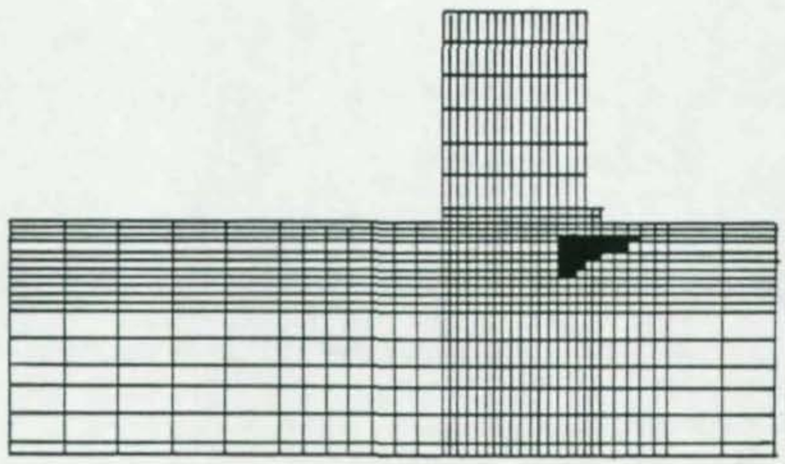
Since the computer program used was limited to two-dimensional elements, the variation in thickness through the depth of the model, e.g. web, flange, weld and fillet thicknesses, was modeled by increasing the element stiffnesses based on the ratio of the element thickness to the thickness of the column web elements. A modulus of elasticity of 29000 ksi in the elastic range and an assumed yield stress of 36 ksi were used.

For purposes of defining load levels, first yield was defined as the load at which the first element reached the yield strain, ε_y. Second yield was defined as when any element reached $3\varepsilon_y$, third yield at $5\varepsilon_y$, fourth yield at $7\varepsilon_y$, fifth yield at $9\varepsilon_y$ with an upper limit of $12\varepsilon_y$ when the analysis was terminated.

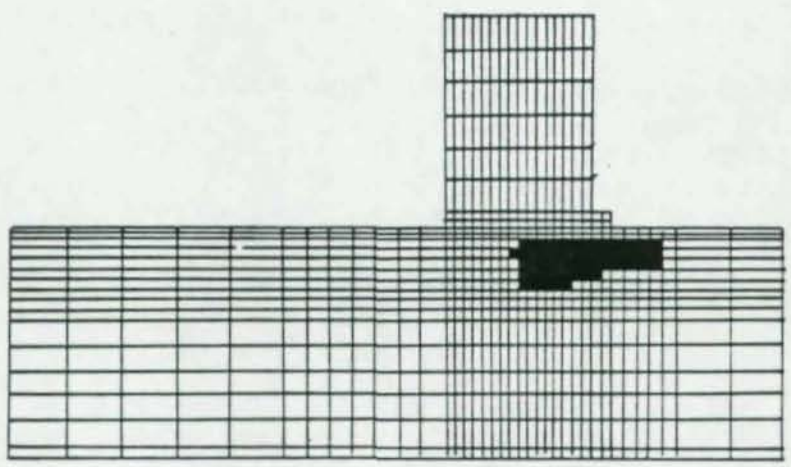
Plots of load versus deflection data were made for each test in the experimental program and are presented with the experimental results later in this chapter. Typical progression of yielding through the web is demonstrated in Figure 9. Further discussion of results and comparison with experimental data is found in the following sections.



(a) 1st Yield, 145.1 kips

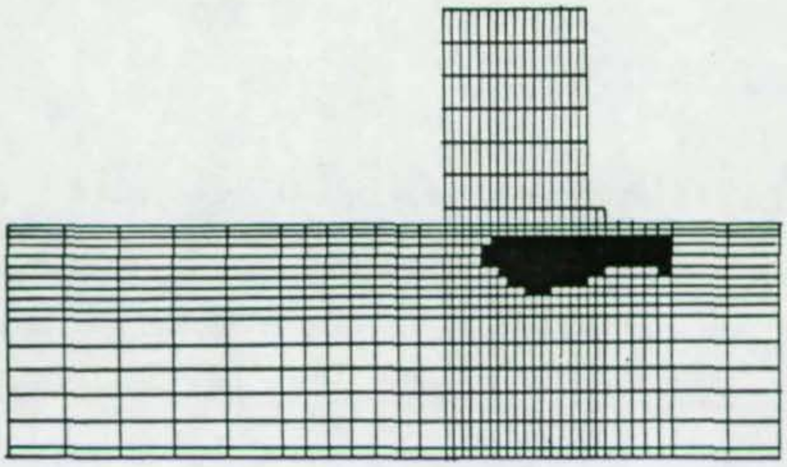


(b) 2nd Yield, 227.8 kips

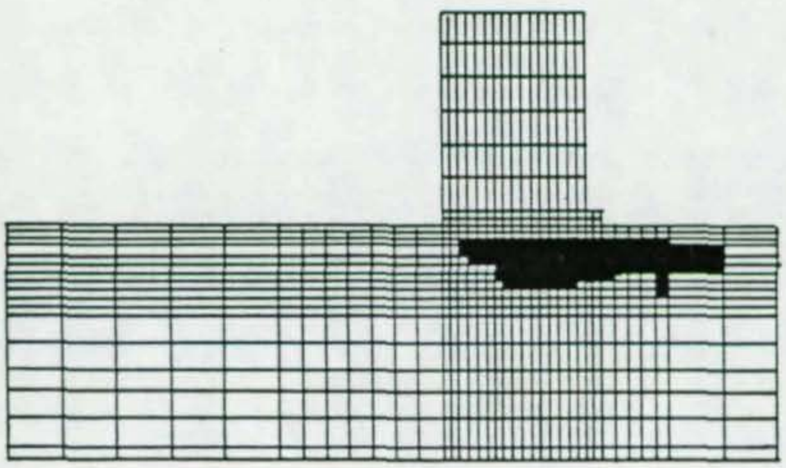


(c) 3rd Yield, 246.8 kips

Figure 9. Yield Patterns for WT16.5x70.5 with W14x90



(d) 4th Yield, 283.1 kips



(e) 5th Yield, 298.7 kips

Figure 9. Yield Patterns for WT16.5x70.5 with W14x90 (cont.)

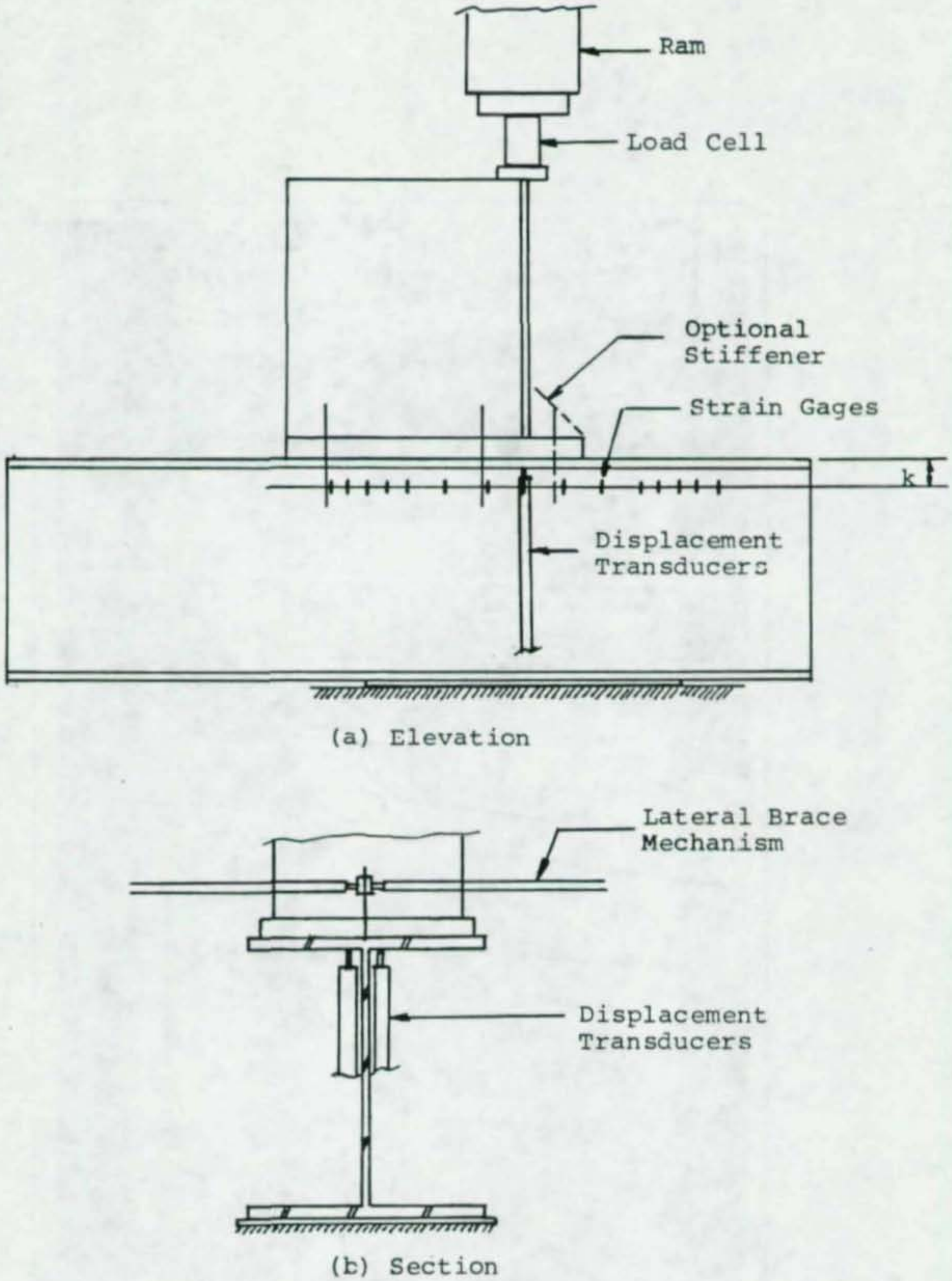
2.4 Testing Program

2.4.1 Scope

In the literature review it was found that a number of research projects have been conducted to determine the column web strength at beam-to-column moment connections. However, except for one test, all of the research conducted in the United States has been limited to welded moment connections and the results of that test are considered inconclusive. The European studies reviewed involved only European section. Thus, a limited number of tests were conducted to substantiate that load in the compression region of beam-to-column moment end-plate connections is distributed over a greater length of the column web than for welded connections.

Six tests were conducted with combinations of beam and column sections as shown in Table 2. The test set-up is shown in Figure 10. Beam and column sizes were chosen to represent reasonable combinations and such that the beam flange was capable of developing a force greater than the "5k" capacity of the column web. The end-plate and bolts were sized using the procedure presented in the 8th edition AISC Manual of Steel Construction(10).

Tests 1, 3 and 5 consisted of a WT beam section welded to the end-plate which was then bolted to the column with four bolts between the edge of the WT stem and the flange as shown in Figure 10. Tests 2, 4 and 6 were conducted with the same sections as Tests 1, 3 and 5, respectively, but



(a) Elevation

(b) Section

Figure 10. Web Strength Test Set-up

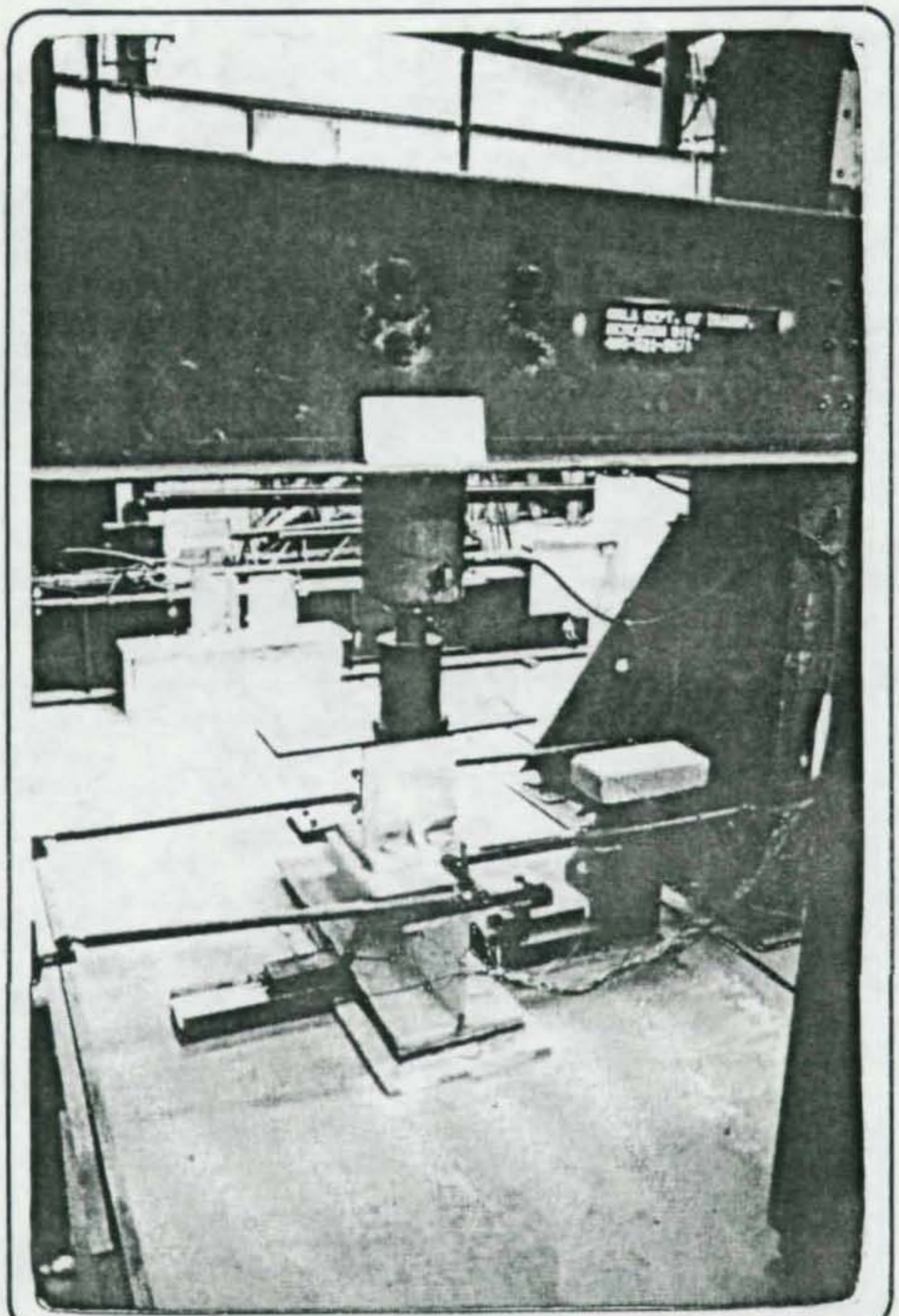


Figure 11. Photograph of Web Test Set-up

Table 2
Column Web Strength Tests

Test	Beam	Column	Stiffener	End-Plate Thickness (in)	Col. Yield Stress (ksi)	5k* (kips)	6k (kips)	7k (kips)
1	WT9x25	W14x90	No	7/8	38.6	174	197	221
2	WT9x25	W14x90	Yes	7/8	38.6			
3	WT9x48.5	W14x111	No	1 1/2	33.3	221	251	280
4	WT9x48.5	W14x111	Yes	1 1/2	33.3			
5	WT16.5x70.5	W14x99	No	1 1/4	38.8	235	263	291
6	WT16.5x70.5	W14x99	Yes	1 1/4	38.8			

*$F_{yc} t_{wc} (t_{fb} + 5k + 2t_e + 2t_w)$

with an extended end-plate and two additional bolts on the outside of the beam flange as shown in Figure 10. A triangular stiffener plate was welded between the WT flange and the extended portion of the end-plate.

Standard tensile column tests were made from samples cut from the column webs. Results are given in Table 3. The measured yield stresses varied from 33.3 ksi to 38.8 ksi.

2.4.2 Test Set-up and Procedure

General details of the test set-up are shown in Figure 10. Figure 11 is a photograph of the set-up. The column was placed in a horizontal position with the load applied through the flange of the WT section using a 750 kip capacity hydraulic ram with manual pumps. The load was monitored with a 350 kip capacity load cell located between the ram and specimen.

A thin plate was placed between the column flange and the reaction floor. This plate was sized to be approximately the same length as the distribution length of the load through the column web and, in this way, represented a beam framing into the opposite flange from the test flange. Lateral movement of the upper column flange was restricted by a lateral brace mechanism attached to the test frame.

Instrumentation consisted of strain gages and displacement transducers. For all six tests, strain was measured on each side of the column web at the toe of the fillet. Strain gages were located along the web to cover the expected

Table 3
Column Web Coupon Test Results

Coupon No.	Test Specimen	Location	Yield Strength	Tensile Strength	% of Elongation
B	W14x78	Web	39.7	65.7	26
C	W14x84	Web	38.3	64.55	27
D	W14x90	Web	38.6	63.30	30.3
E	W14x99	Web	38.8	68.5	25.3
G	W14x103	Web	34.6	63.6	27.5
H	W14x111	Web	33.3	62.5	28.8

load distribution length. In addition to the gages on the column web, strain gages were placed on the flange and stem of the WT beam section. Figure 10 shows typical locations.

Two displacement transducers were used to measure vertical displacement of the column flanges. These were placed on opposite sides of the web directly below the beam flange. The transducers were placed as close as possible to the fillet. An additional displacement transducer was placed horizontally on one side of the web to measure lateral displacement of the column. A Hewlett-Packard 3497 Data Acquisition/Control Unit was used with an HP 85 desktop computer to collect and record data.

At the beginning of each test, the specimen was loaded to approximately 20% of the expected maximum load to check the instrumentation. The specimen was then unloaded and initial strain and displacement readings were taken for zero load. The specimen was then loaded in approximately 10 kip increments with readings of all instrumentation recorded at each increment. A load-deflection curve was plotted to monitor any nonlinearity. The loading was continued until failure of the specimen occurred.

2.4.3 Test Results

Test results consist of load versus deflection data and stress distribution data. The load versus deflection data includes a theoretical plot obtained from the finite element analysis as well as the experimental displacement of the column flange on each side of the web. The stress distribution was plotted at various load levels showing the yielding in the column web. This distribution is compared with the expected distribution length.

The "5k", "6k" and "7k" load levels shown on the various plots, are based on the equation

$$P = F_{yc}\, t_{wc}\, (t_{fb} + 5k + 2t_e + 2t_w) \qquad (22)$$

with 5k replaced with 6k and 7k for the higher levels. All terms are defined as previous and measured yield stress and cross-section dimensions were used for the calculations. The corresponding yielded length along the critical column web section will be referred to as the "5k", "6k" or "7k" length in the following discussion of the test results.

Test 1. Test 1 consisted of a WT9x25 beam with a W14x90 column. The beam was attached to the column with 1 1/8 in. diameter A325 bolts through a 7/8 in. thick end-plate. The material yield stress obtained from a coupon test was 38.6 ksi.

The theoretical, from the finite element analysis, and experimental load-deflection curves are shown in Figure 12. Because of instrumentation errors in Test 1, the results are not considered reliable and should be neglected. The stress distribution curves at increasing load levels are presented in Figure 13 but are also in error and should not be used. The general pattern of distribution can be seen however. It is not believed the maximum capacity of the column web was reached in the test.

Test 2. Test 2 was conducted with the same beam and column section as well as bolt size and end-plate thickness as used in Test 1, but with an extended end-plate and triangular stiffener as described previously.

From the load vs. deflection plot shown in Figure 14, it is evident that the column web moved laterally slightly above the "6k" load level. From the measured stress distributions shown in Figure 15, it is seen that the column web at the critical section was yielded along a length of approximately "6k" at the maximum load level. It is noted that the extended end-plate and triangular stiffener had little effect on the yield length.

Test 3. A WT9x48.5 beam was welded to a 1 1/2 in. thick end-plate for test specimen 3. Six 1 3/8 in. diameter A325 bolts were used to attach the end-plate to a W14x111 column. The material yield stress was found to be 33.3 ksi.

The maximum load applied was between the "6k" and "7k" load levels of 251 and 280 kips, respectively, as can be seen in Figure 16. The yield length along the column web at this load level was approximately "6k".

Test 4. The test configuration for Test 4 was identical to Test 3, except for the extended end-plate and triangular stiffener. The measured material yield stress was 33.3 ksi.

The maximum applied load exceeded the "7k" level as is seen in Figure 18 and the yielded portion of the web exceeded the "7k" distance, Figure 19.

Test 5. A WT16.5x70.5, the largest WT used in the testing program, was bolted to a W14x99 column section for this test. Six 1 1/2 in. diameter A325 bolts were used to connect the 1 1/2 in. thick end-plate to the column flange. The yield stress of the column web was found to be 38.8 ksi.

The maximum applied load was slightly less than the "7k" level, Figure 20, and the yield length along the column web was approximately the "7k" length. The distribution was heavily centered toward the WT web as can be seen in Figure 21.

Test 6. Test 6 was conducted with identical sections to those used in Test 5. An extended 1 1/2 in. thick end-plate with triangular stiffener was used together with 1 1/2 in. diameter A325 bolts.

Comparison of Figures 20 and 22 shows little increase in load carrying capacity with introduction of the stiffener. The "7k" load was exceeded in Test 6, but only slightly. However, the width of the yielded portion of the web is greater in Test 6, Figure 23, than Test 5, Figure 21.

2.4.4 Summary

With the exception of Test 1, the "6k" load level was exceeded in all tests. Further, the length of the yielded portion of the web generally agreed with the maximum load reached. Finally, the measured yield patterns were in general agreement with those obtained in the finite

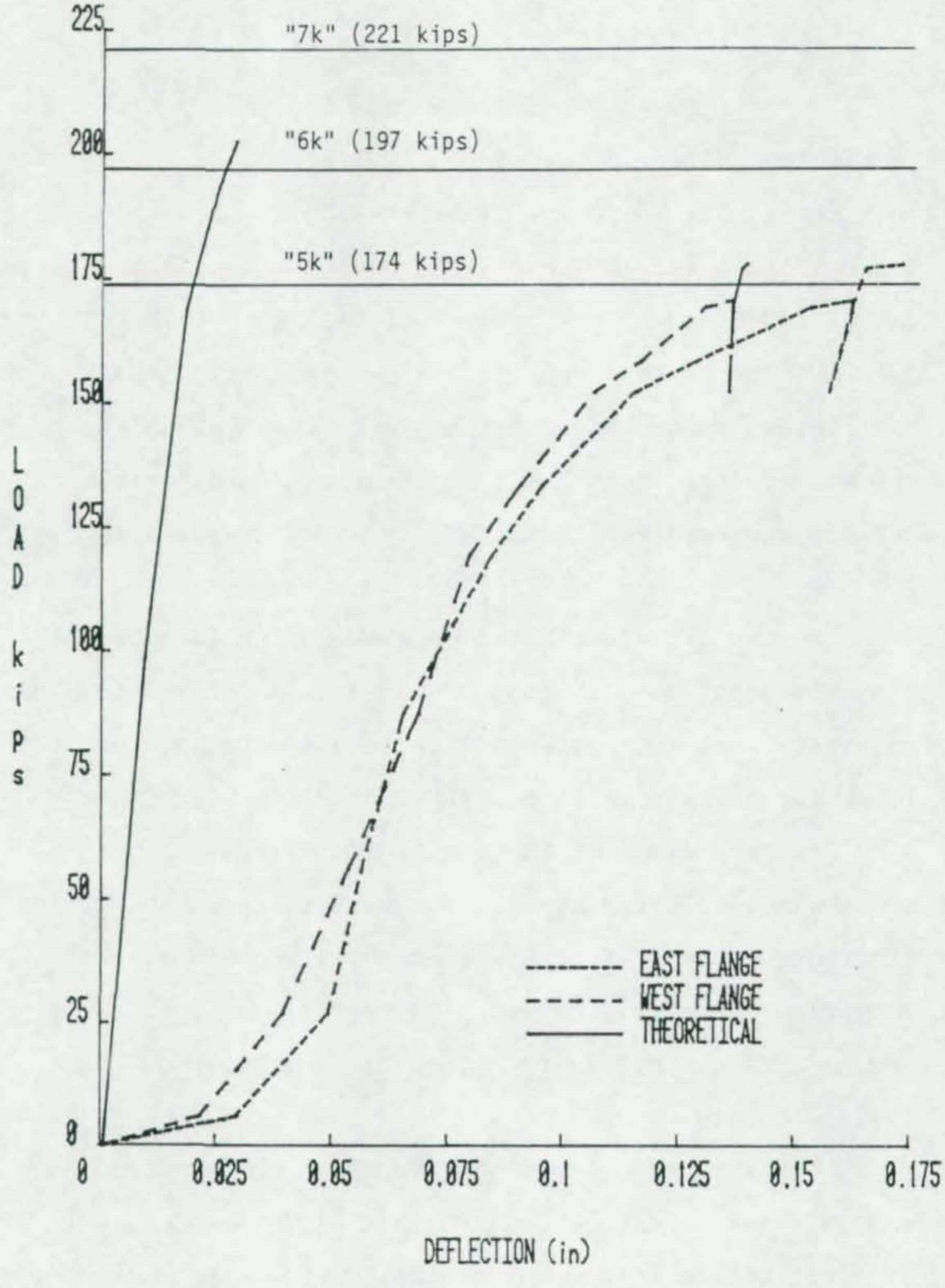


Figure 12. Load vs. Deflection - Test 1

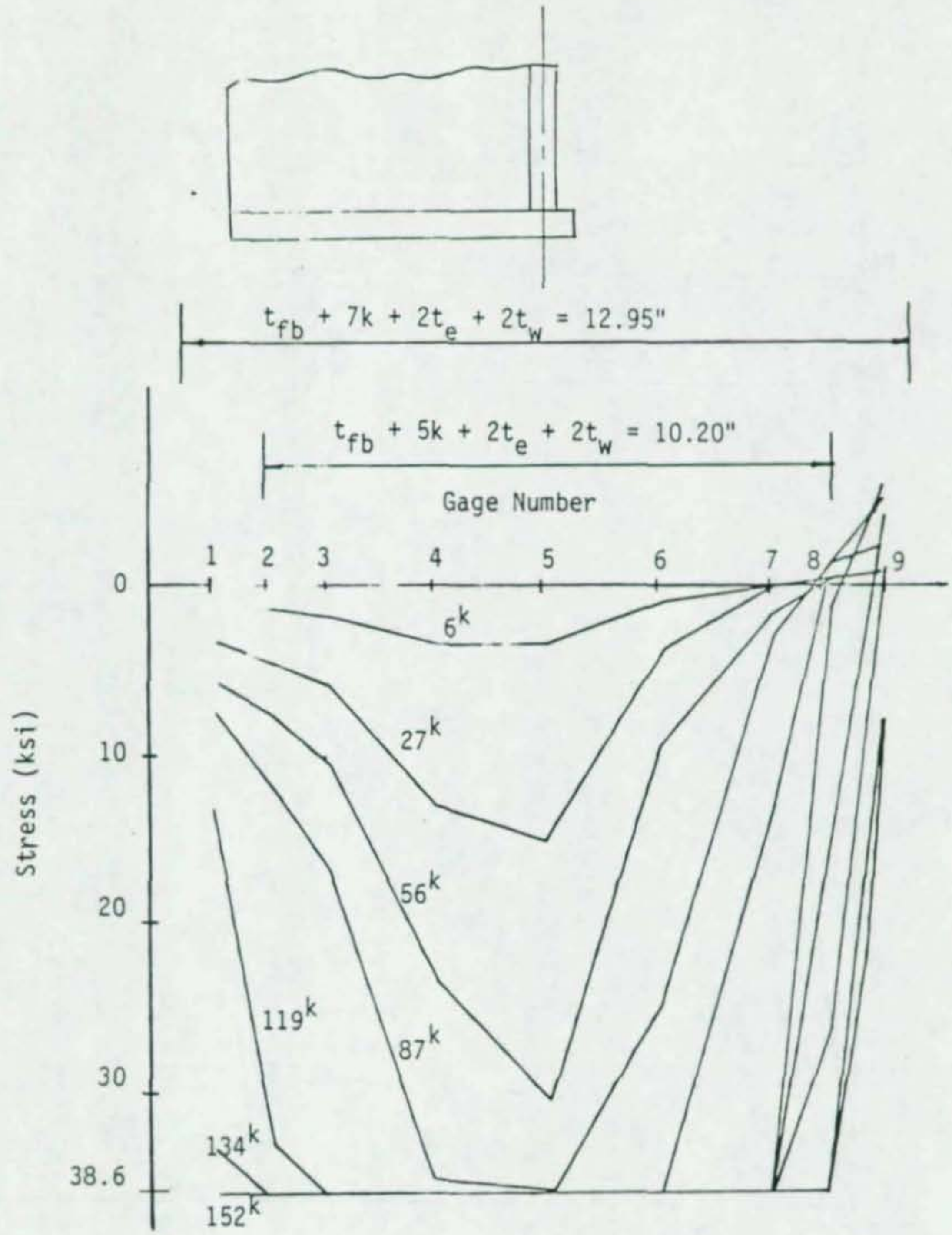


Figure 13. Stress Distribution - Test 1

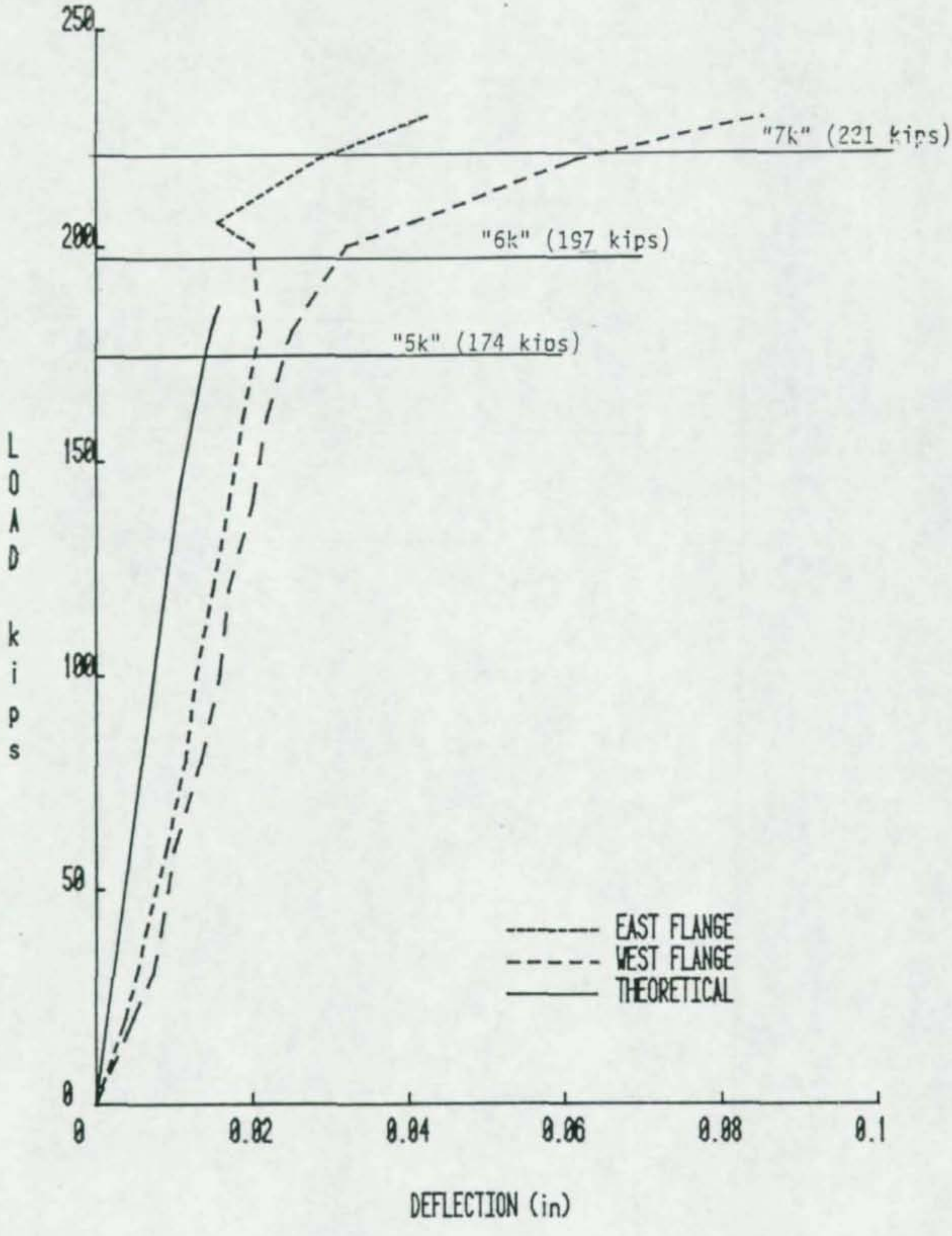


Figure 14. Load vs. Deflection - Test 2

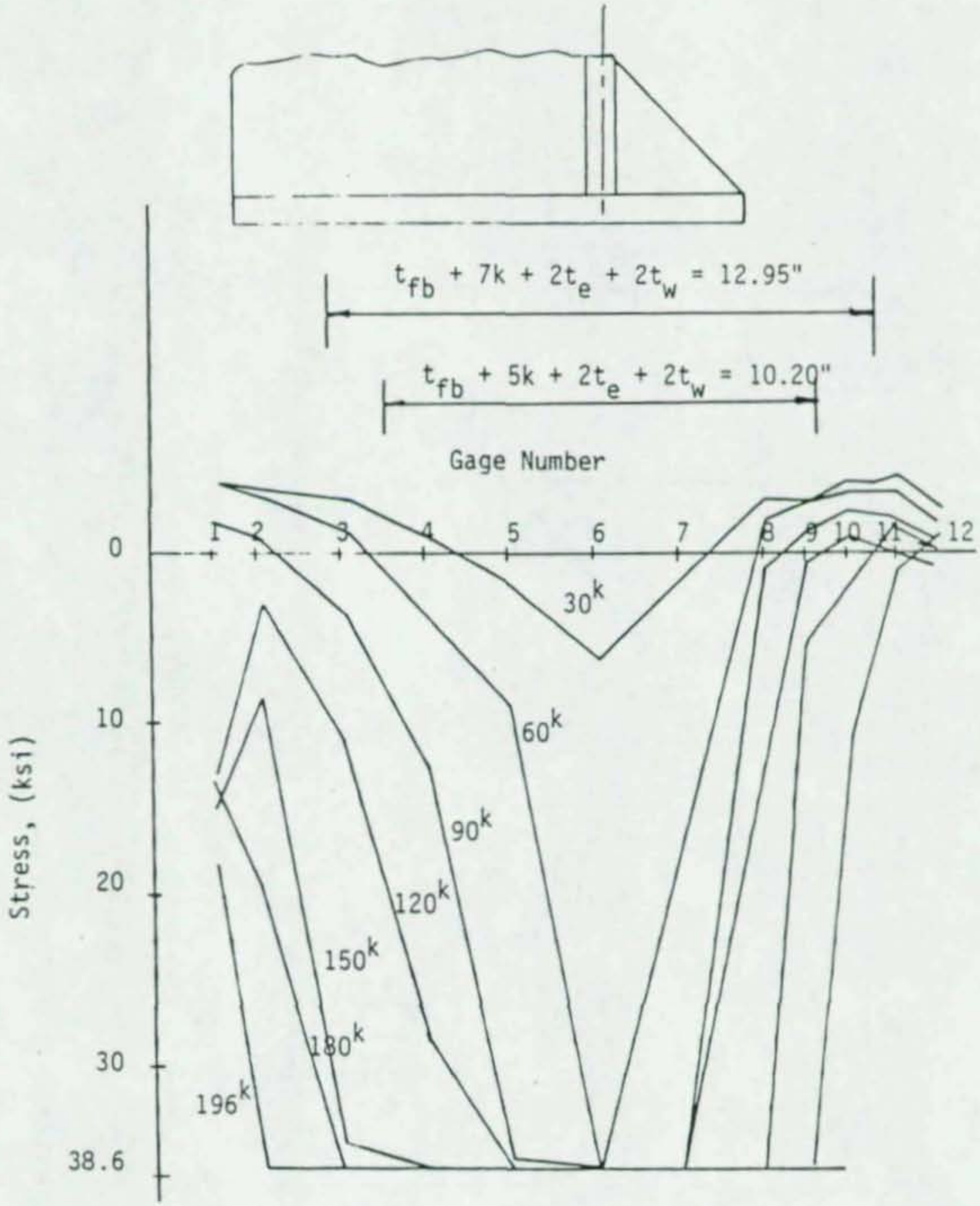


Figure 15. Stress Distribution - Test 2

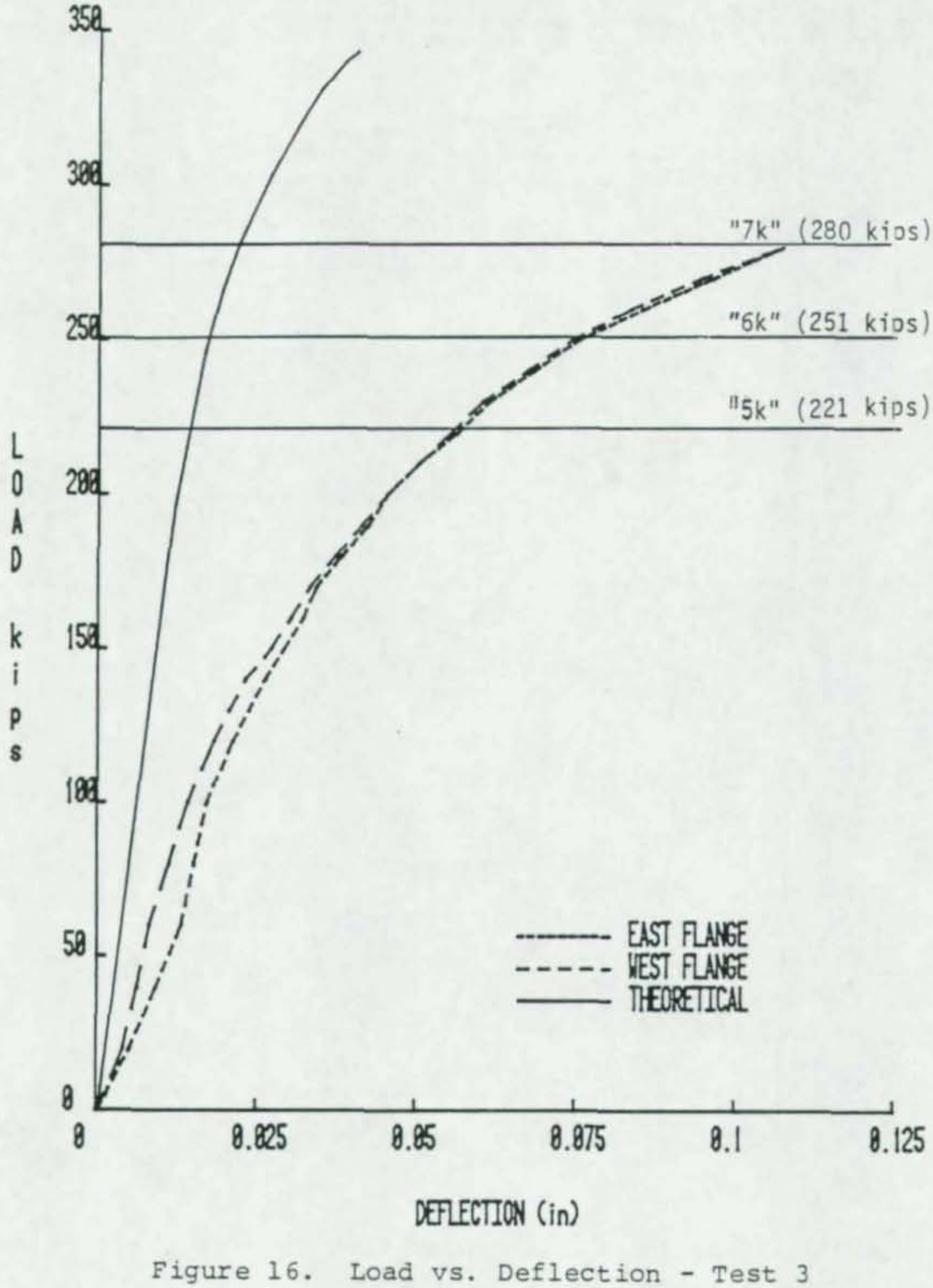


Figure 16. Load vs. Deflection - Test 3

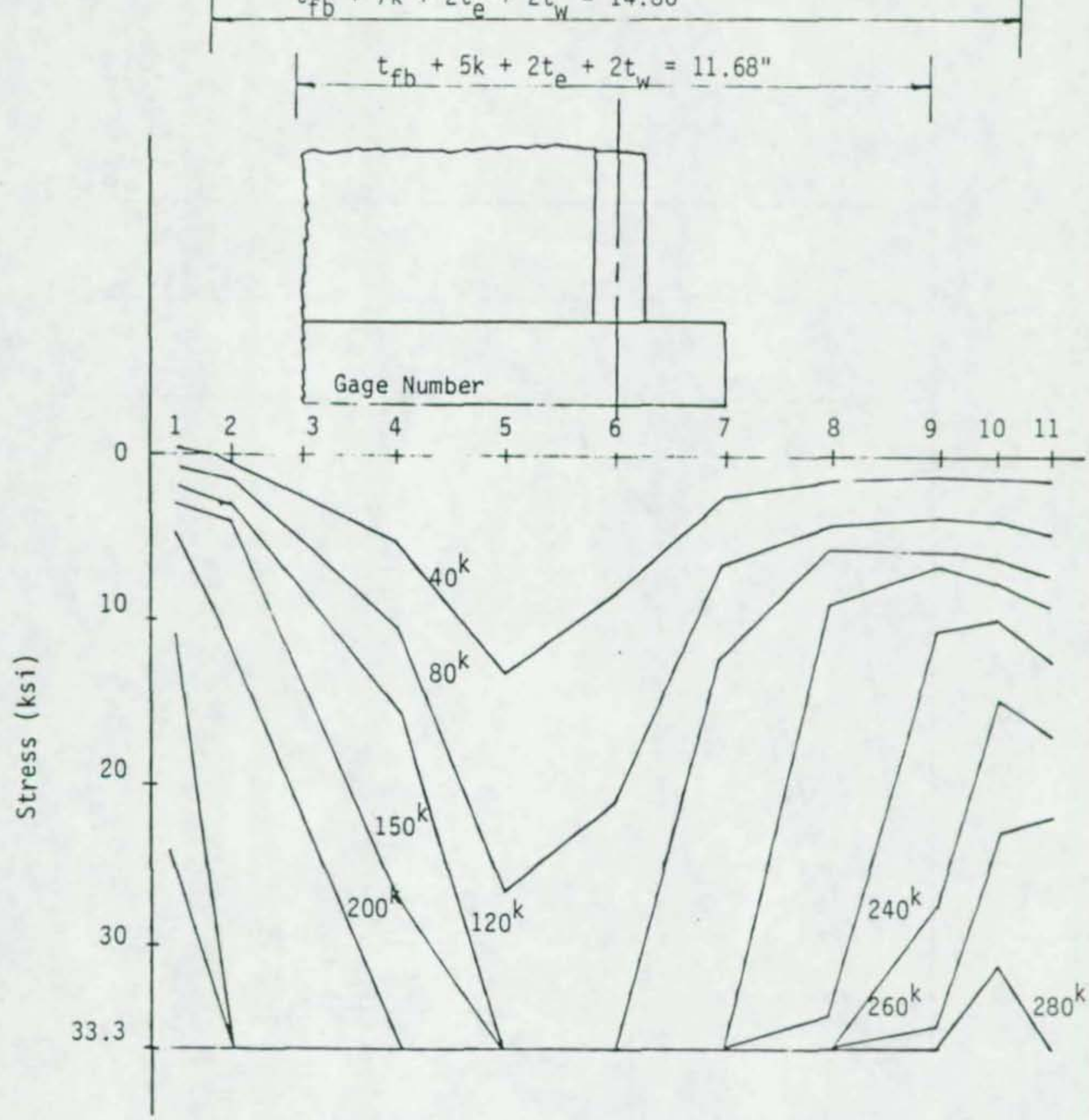


Figure 17. Stress Distribution - Test 3

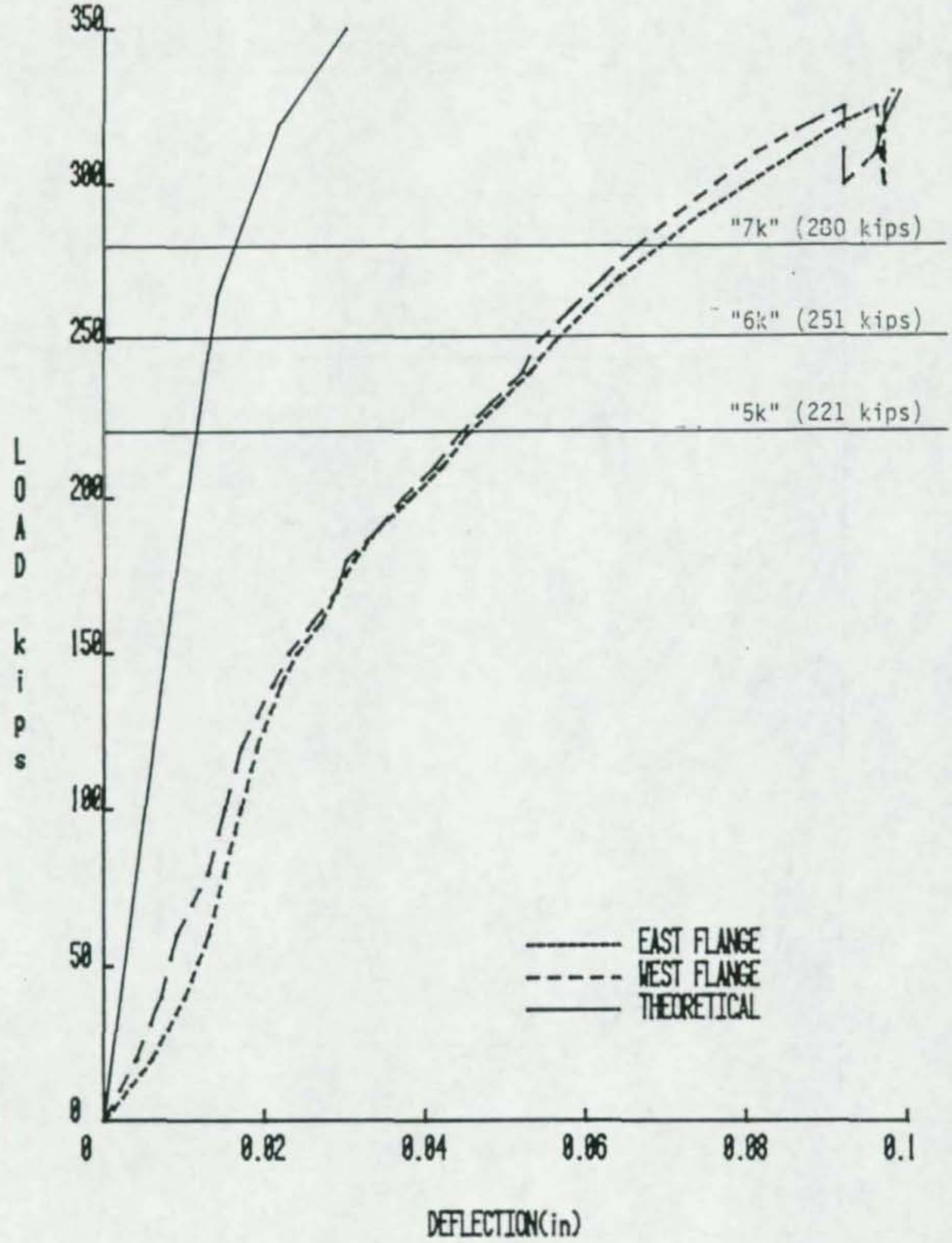


Figure 18. Load vs. Deflection - Test 4

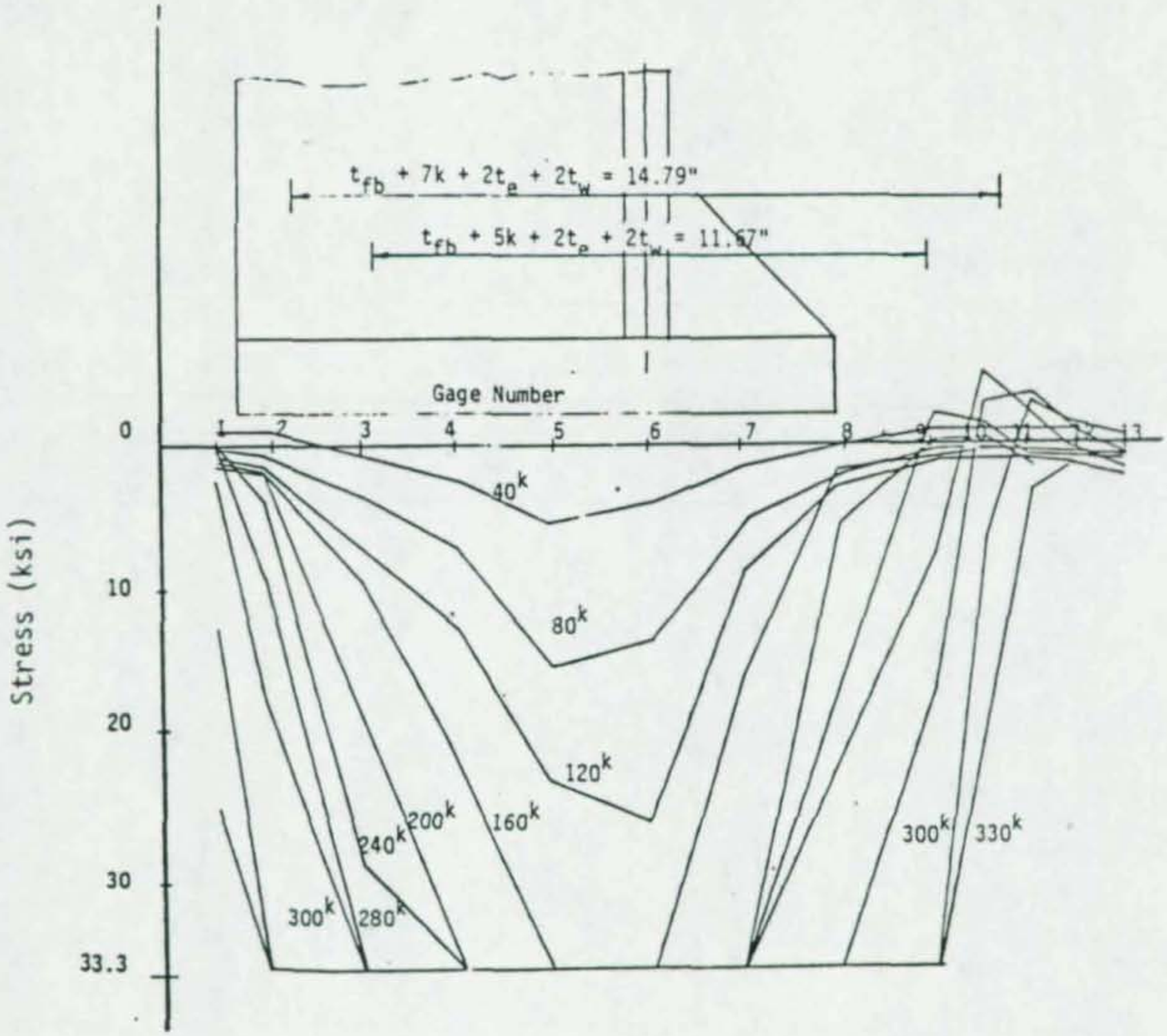


Figure 19. Stress Distribution - Test 4

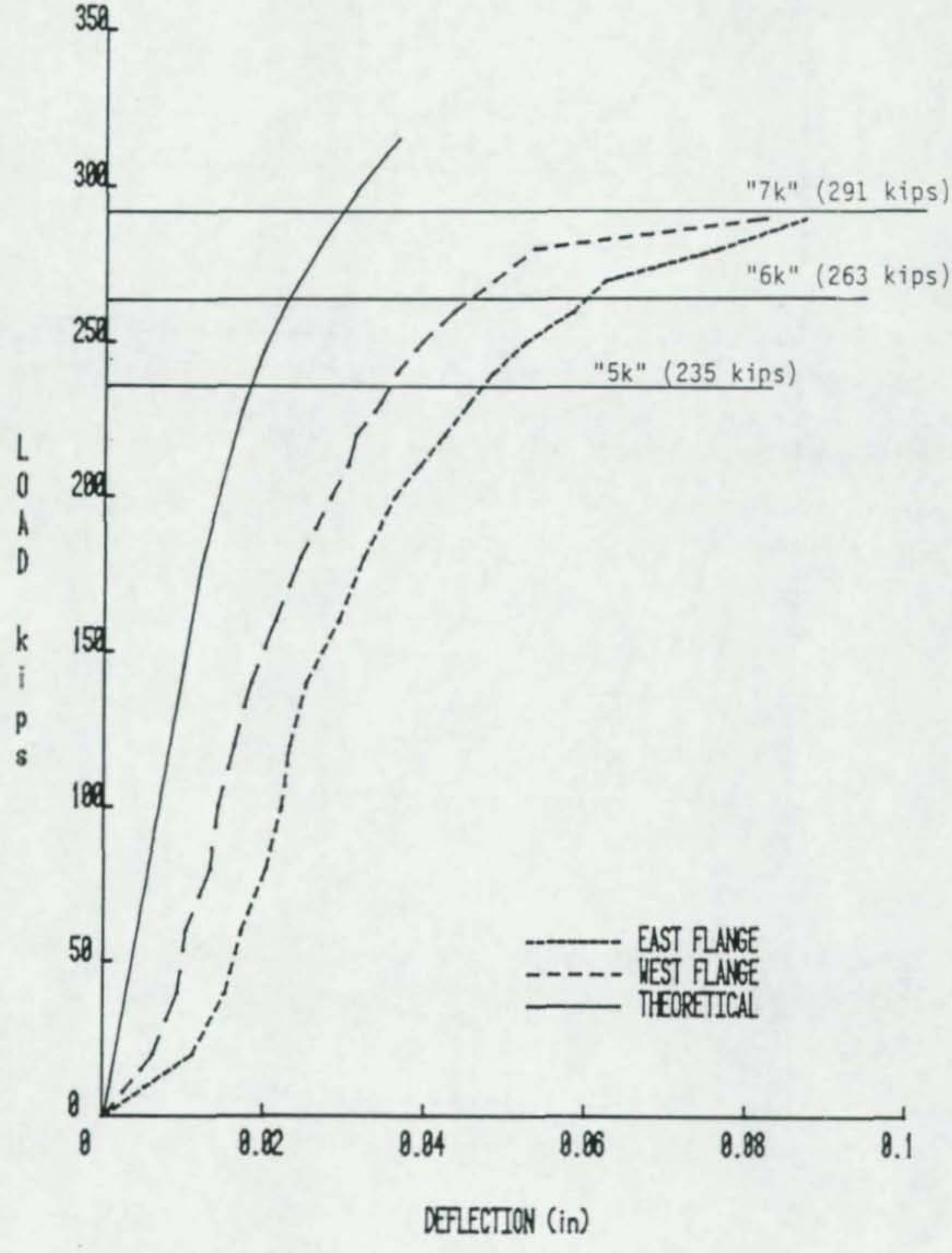


Figure 20. Load vs. Deflection - Test 5

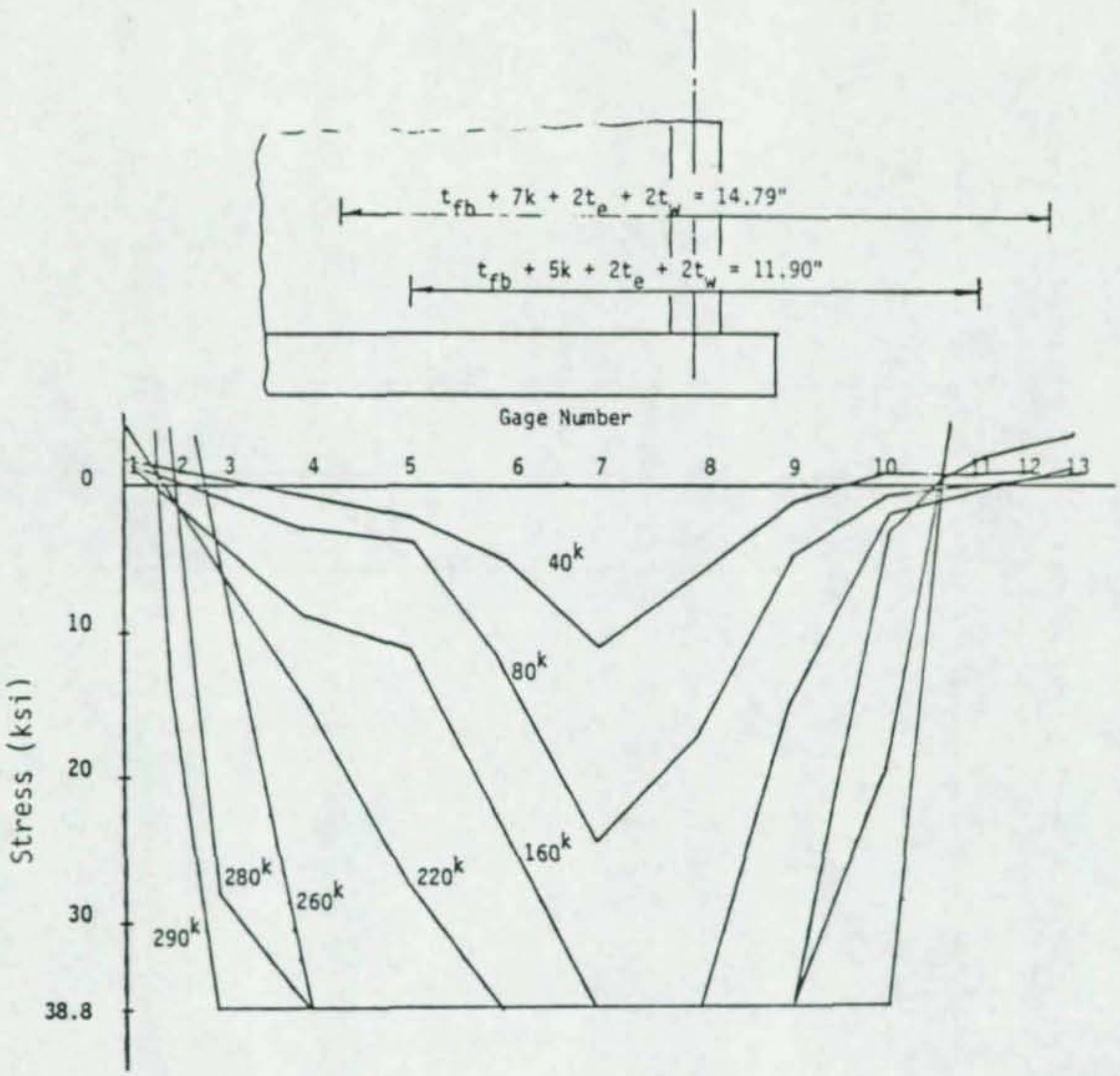


Figure 21. Stress Distribution - Test 5

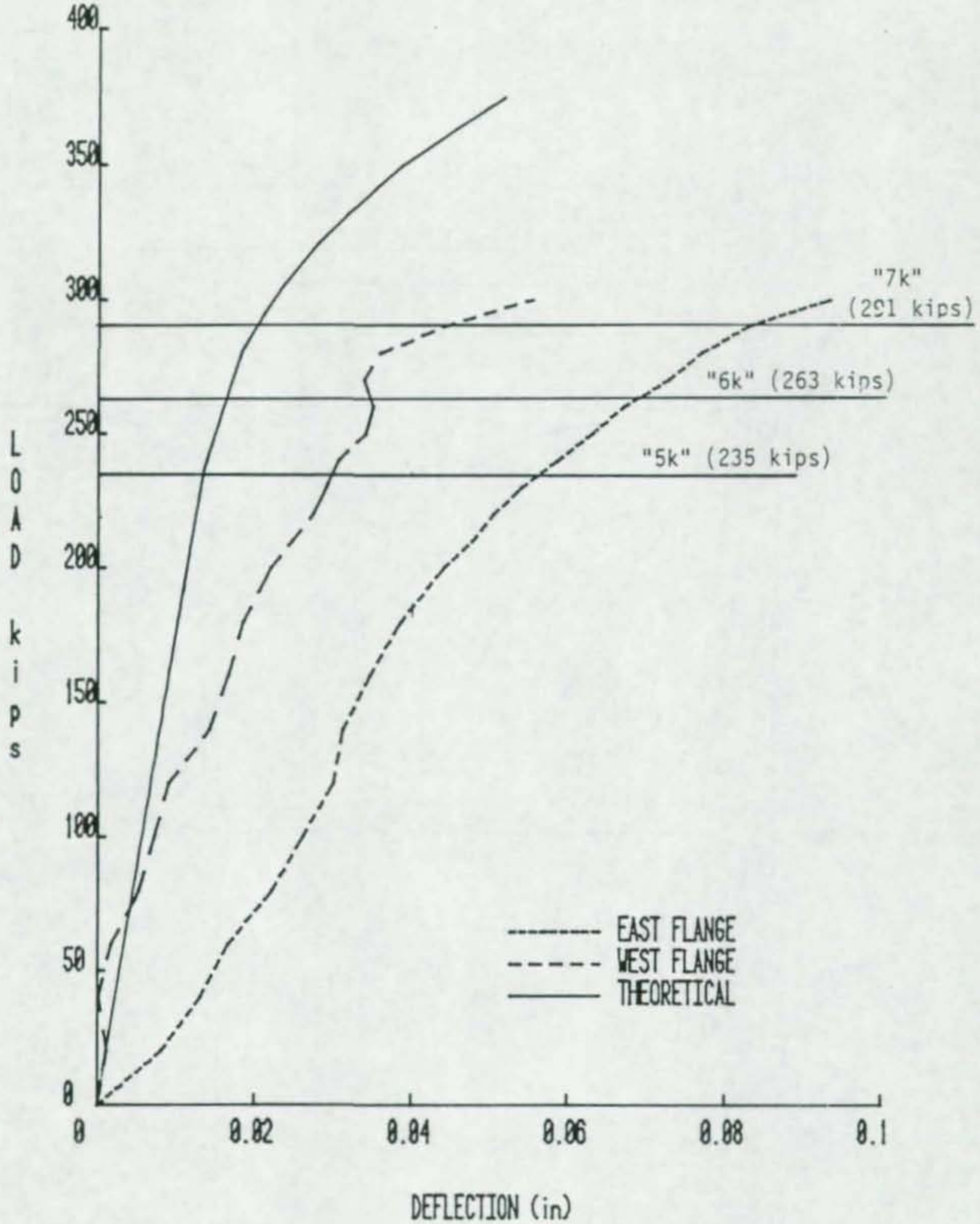


Figure 22. Load vs. Deflection - Test 6

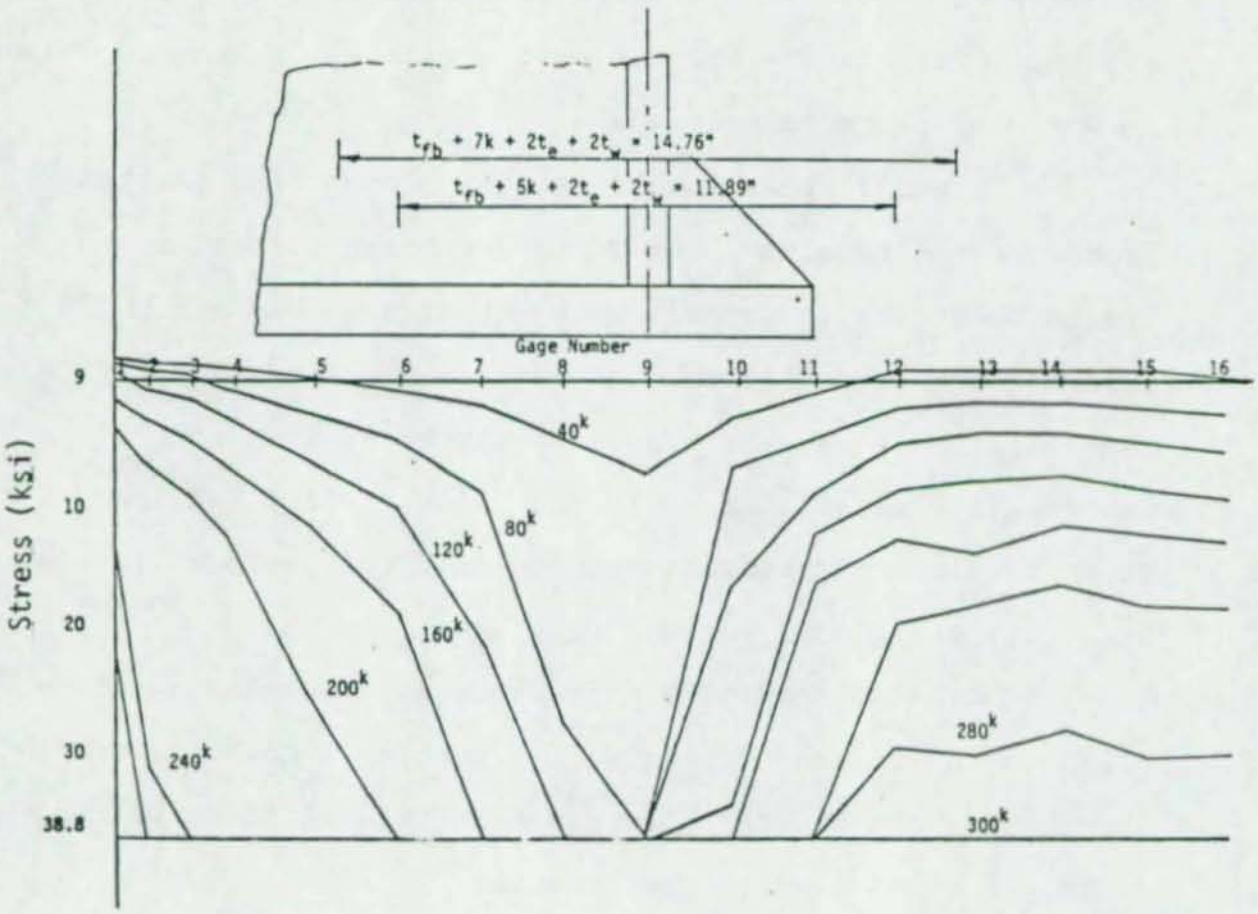


Figure 23. Stress Distribution - Test 6

element analyses. Thus the results of this investigation indicate that the "6k" load level is an acceptable and slightly conservative web strength estimate for beam-to-column, moment, end-plate connections.

Several items should be noted concerning the test procedure and results. For Tests 2 through 6, failure occurred by excessive lateral movement of the column top flanges, in some instances breaking the lateral brace mechanism. This instability was caused by lack of restraint from the column web once the material had yielded below the applied load. Thus, particular attention must be paid to the local lateral stability of columns without weak axis framing.

It also should be noted that no axial load was applied to the column section during testing. However, Graham, Sherbourne, and Khabbaz(2) state that axial load has a negligible effect on similar test results.

2.5 Design Recommendation

Based on the results presented above, it is recommended that the column web strength at the compression region of beam-to-column, moment, end-plate connections be estimated from

$$P_{max} = F_{yc}\, t_{wc}(t_{fb}+6k+2t_e+2t_w) \tag{23}$$

where all terms have been defined previously, provided sufficient lateral bracing is provided to prevent out-of-plane buckling of the column flanges.

This recommendation is a significant liberalization of the current recommendations for welded, beam-to-column, moment connections, but appears to be justified from previous European studies and the results of this current study.

CHAPTER III

COLUMN TENSION REGION STRENGTH

3.1 Introduction

Design criteria to prevent failure of the column web and the beam flange-to-column flange weld at the tension region of beam-to-column moment connections is presented in Section 1.15.5 of the 1978 AISC specification[1]. The criteria is intended for use in connections in which the beam flange or flange extension is welded directly to the column flange. No provisions are provided in this specification to prevent column web or flange failure near the tension bolts of end-plate connections. The purpose of this study is to develop design criteria for the column tension region at beam-to-column moment end-plate connections.

In this chapter, an extensive literature review, including studies of welded beam flange-to-column connections, tee-hanger studies, and studies directly involving beam-to-column moment end-plate connections, is first presented. Using combinations of typical beam and column sections, results from selected criteria found in the literature are evaluated. Test results to confirm findings in the literature are then presented. Finally design procedures for the prevention of column flange failure in the tension region of beam-to-column moment end-plate connections are suggested.

3.2 Literature Review

3.2.1 Welded Beam-to-Column Connections

Graham, Sherbourne, and Khabbaz[2] conducted eleven tests in which the beam tension flange was simulated by plates approximately the same size as the flange welded directly to the column flange. The column was placed horizontally in an 800 kip universal testing machine and tension was applied to the plates on each side of the column as shown in Figure 24.

Several different column sections were tested with the connection plate width and thickness varied to simulate different beam sections. Seven of the eleven tests failed when the butt weld cracked at the center of the connection plate, opposite the column web. This crack occurred after considerable bending in the column flanges. Two of the tests failed when a crack started in the fillet of the column and the final two failed when the weld began tearing away from the column flange from the outside proceeding to the center. In all of the tension region tests, the axial column load was neglected as was the effect of the compression region on the tension region. These were shown in earlier testing to have little effect on test results.

The failure mechanism in the tension region consists

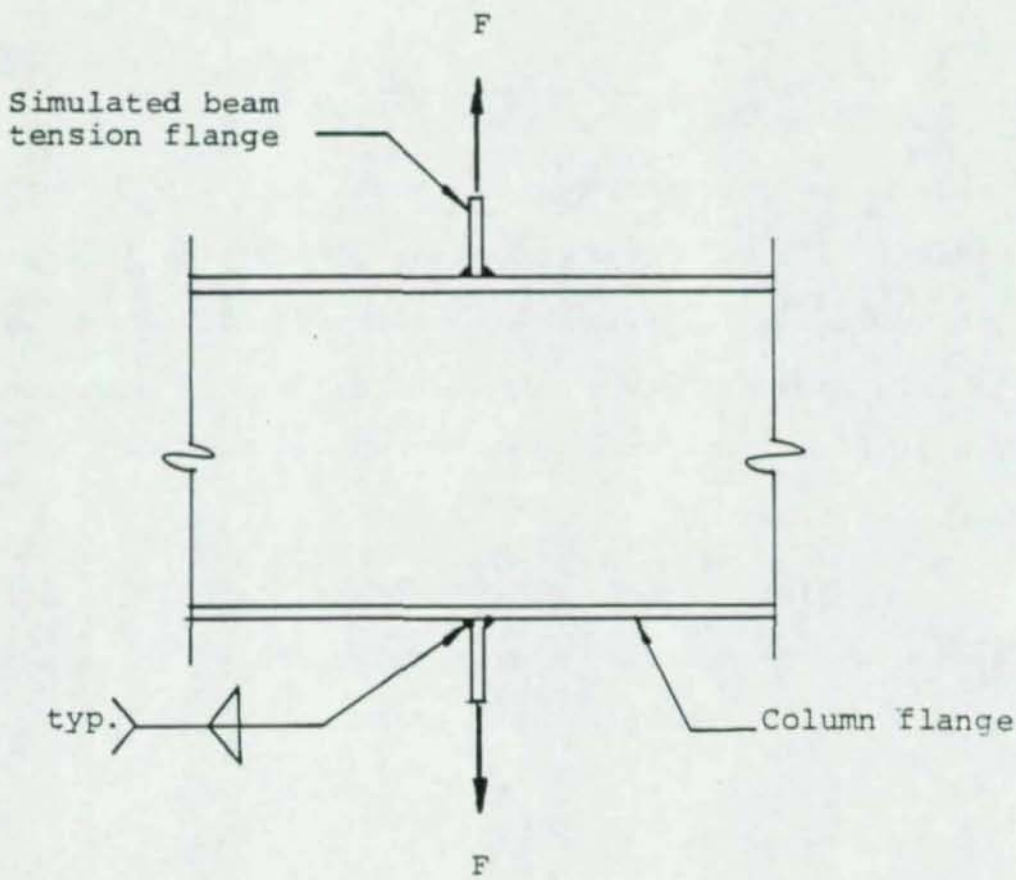


Figure 24. Test Set-up Used in Reference 2

of the column flange acting as two plates fixed along three edges with one edge free. When the "plates" deflect at their free edges it causes considerable overstressing in the butt weld and column fillet. From analytical and experimental results, it was found that stiffeners are not required in the tension region if

$$t_{fc} \geq 0.4\sqrt{b_f t_{fb}} \tag{24}$$

and

$$t_{fc} \geq 0.4\sqrt{t_{wc}(t_{fb} + 5k)} \tag{25}$$

where t_{fc} = column flange thickness (in.), b_f = beam flange width (in.), t_{fb} = beam flange thickness (in.), t_{wc} = column web thickness (in.), and k = column "k" distance (in.). Equation 24 prevents premature weld failure and Equation 25, column web yielding. The same yield stress is assumed for both the beam and column.

Fisher and Struik(12) state that flange deformations caused by concentrated forces delivered by the tension flange of the beam can be prevented if

$$t_{fc} \geq 0.4\left[A_f \cdot \frac{F_{yb}}{F_{yc}}\right]^{1/2} \tag{26}$$

where A_f = area of the beam flange (sq. in.) and F_{yb} = yield stress of the beam (ksi). If the beam and column yield stresses are equal, Equation 26 is identical to Equation 24. For end-plate connections, Fisher and Struik state that the use of Equation 26 yields over-conservative results since the tension force is distributed into the column flange only by the bolts, inducing larger moments in the column flange.

Witteveen, Stark, Bijlaard, and Zoetemeijer[7] conducted tests in the Netherlands on European sections for beam-to-column connections with and without end-plates. For a beam welded directly to the column flange, the total force which can be transmitted to the column flange was found to be

$$F_t = F_{yc}\{t_{fb}(t_{wc} + 2r_c) + 7t_{fc}{}^2\} \tag{27}$$

where F_{yc} = column yield stress and r_c = column fillet (in.). This equation is based on the assumption that forces in the center portion of the beam flange are transferred directly to the column web over a distance $t_{wc} + 2r_c$. The remaining force is transmitted through bending of the column flanges. The force associated with the development of a yield line on each side of the column web can be expressed as

$$\bar{F}_t = C_1 F_{yc} t_{fc}^2 \tag{28}$$

where C_1 is a constant. Witteveen <u>et al</u> by experiment have found C_1 to vary between 3.5 and 5 for European sections. Conservatively using 3.5 for C_1, the flange bending contribution in Equation 27 is then $7F_{yc}t_{fc}^2$.

Mann and Morris[6] have reviewed numerous studies concerning beam-to-column moment connections. They state that Reference 13 gives the following limitations for the thickness of unstiffened column flanges

$$t_{fc} \geq 0.4 \left[\frac{F_t}{F_{yc}}\right]^{1/2} \tag{29a}$$

and for stiffened column flanges

$$t_{fc} \geq 0.3 \left[\frac{F_t}{F_{yc}}\right]^{1/2} \tag{29b}$$

If the contribution of the beam web to F_t is neglected and the yield stresses of both the column flange and the beam flange are equal, Equation 29a reduces to Equation 26. Mann and Morris also state that Equations 29a and 29b have been incorporated into the European Recommendations for Steel Construction(14) with the comment that until further experimental work has been done to provide suitable criteria then "the provision of stiffeners and their design should be carried out conservatively." It is not clear in the Mann and Morris paper if this comment is directed toward welded connections or bolted end-plate connections or both types.

Stiffener provisions for welded beam-to-column connections in the 1978 AISC Specification (1) are based on the work of Graham, Sherbourne and Khabbaz(2). In Section 1.15.5, the required stiffener area is expressed as

$$A_{st} = \frac{P_{bf} - F_{yc}t_{wc}(t_{fb} + 5k)}{F_{yst}} \tag{30}$$

where A_{st} = stiffener area required (sq. in.), P_{bf} = the computed force delivered by the flange multiplied by 5/3, when the computed force is due to live and dead load only, or 4/3 when the computed force is due to live and dead load in conjunction with wind or earthquake forces (kips), and F_{yst} = stiffener yield stress (ksi). No stiffeners are required if the value of A_{st} is negative. In addition, it is necessary to provide a pair of stiffeners opposite the tension flange if

$$t_{fc} < 0.4\sqrt{P_{bf}/F_{yc}} \tag{31}$$

3.2.2 Tee-Hanger Connections

Douty and McGuire[15] developed a design procedure for tee-hanger connections based on test results from tee stubs bolted to a rigid support as shown in Figure 4. When the load 2F is applied, it is stated that the tee stem flange will bend causing prying action at the edges of the flange, represented as the force Q in Figure 25. From statics the total force in each bolt is given by

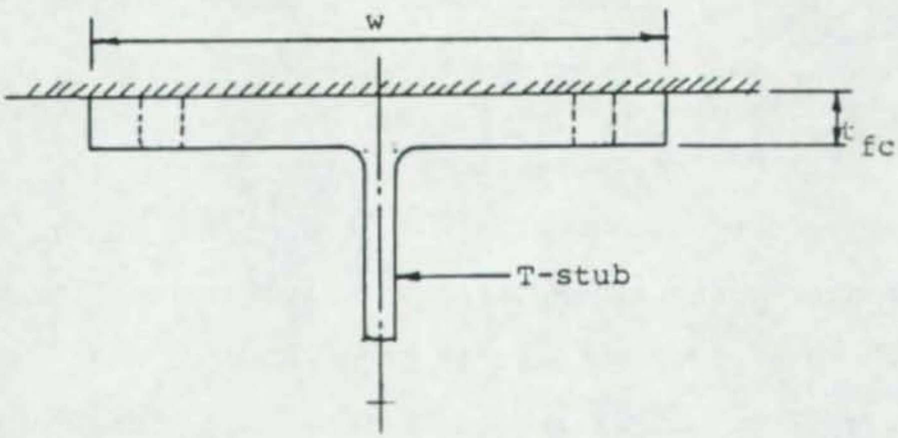
$$T = F + Q + C \leq T_u \tag{32}$$

where C = residual contact force as shown in Figure 25b and T_u = maximum bolt force. A semi-empirical expression for Q in terms of F was developed using elastic analysis with modifications to reflect test results. Because of its complexity, the relationship was modified in Reference 16to

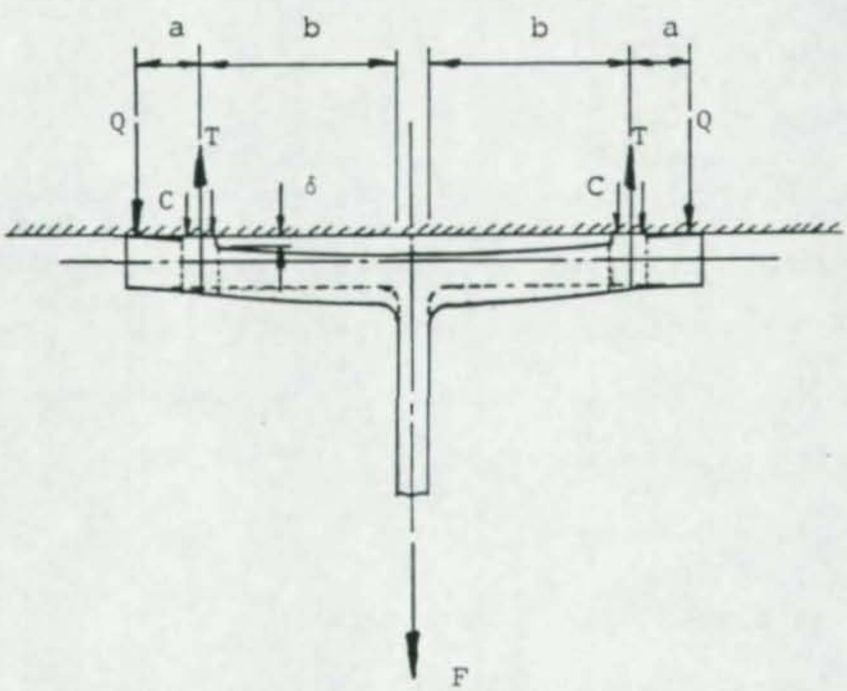
$$\frac{Q}{F} = \frac{3b}{8a} - \frac{t_{fc}^{3}}{20} \tag{33}$$

where a and b are defined in Figure 25.

From tests conducted by Nair, Birkemoe and Munse[17], the prying ratio at ultimate load for connections with A325 bolts and A36 steel was found to be



a) Before loading is applied



b) After loading is applied

Figure 25. Model used by Douty and McGuire and Others

$$\frac{Q}{F} = \frac{100bd_b^{\ 2} - 18wt_{fc}^{\ 2}}{70ad_b^{\ 2} + 21wt_{fc}^{\ 2}} \tag{34}$$

where d_b = bolt diameter. A similar expression was found for A490 bolts. These relationships were adopted for tee-hanger design in the 7th ed. AISC Manual of Steel Construction[18].

In References 15 to 18 it is suggested that the tee stem flange thickness be determined from

$$M_{max} = F_{yc} \frac{wt_{fc}^{\ 2}}{4} \tag{35}$$

where w = the tee stem length per bolt row and M_{max} is the maximum of

$$M_1 = Qa \tag{36a}$$

$$M_2 = Fb + Q(a + 2b) \tag{36b}$$

Fisher and Struik[12] show that Equation 33 overestimates the prying force and a conservative design results. They state that Equation 34 gives slightly better agreement with test results but the relationship is applicable only to the specific combination of bolt and plate material. Instead a procedure based on the work of Struik and DeBack[19] is recommended.

Referring to Figure 25, the required tee-stem flange thickness is found from

$$t = \left[\frac{4Ta'b'}{w\ F_{yc}\{a' + \alpha\delta(a' + b')\}}\right]^{1/2} \tag{37}$$

where $a' = a + d_b/2$, $b' = b - d_b/2$, α = the ratio between the moment per unit width at the centerline of the bolt line and the flange moment at the web face, and δ = the ratio of the net area at the bolt line and the gross area at the web face of the flange. When $\alpha = 0$, it corresponds to the case of single curvature bending, and $\alpha = 1$ corresponds to double curvature bending. The fastener load T is taken as

$$T = T_u \tag{38}$$

if $\alpha = 1.0$ and if $\alpha \geq 1.0$ it is taken as 1.0 and

$$T = F\left[1 + \frac{\delta}{(1+\delta)} \frac{b'}{a'}\right] \tag{39}$$

Finally, the following must be satisfied

$$F\left\{1 + \frac{\delta\alpha}{(1+\delta\alpha)} \frac{b'}{a'}\right\} \leq T_u \tag{40}$$

and

$$a \leq 1.25\ b \tag{41}$$

Use of this procedure requires iteration to find α and Q.

Granstrom[20] has extensively studied end-plate connections. Much of his experimental work involved tests of tee-hangers. The effects of curved end-plates (caused by weld distortion), severe tightening, bolt thread length and weld strength were all investigated with emphasis on strength predictions rather than stress predictions. Granstrom combined results from his tests using mild steel with those reported by five other researchers to develop the "design line" shown in Figure 26. Granstrom's design method is as follows:

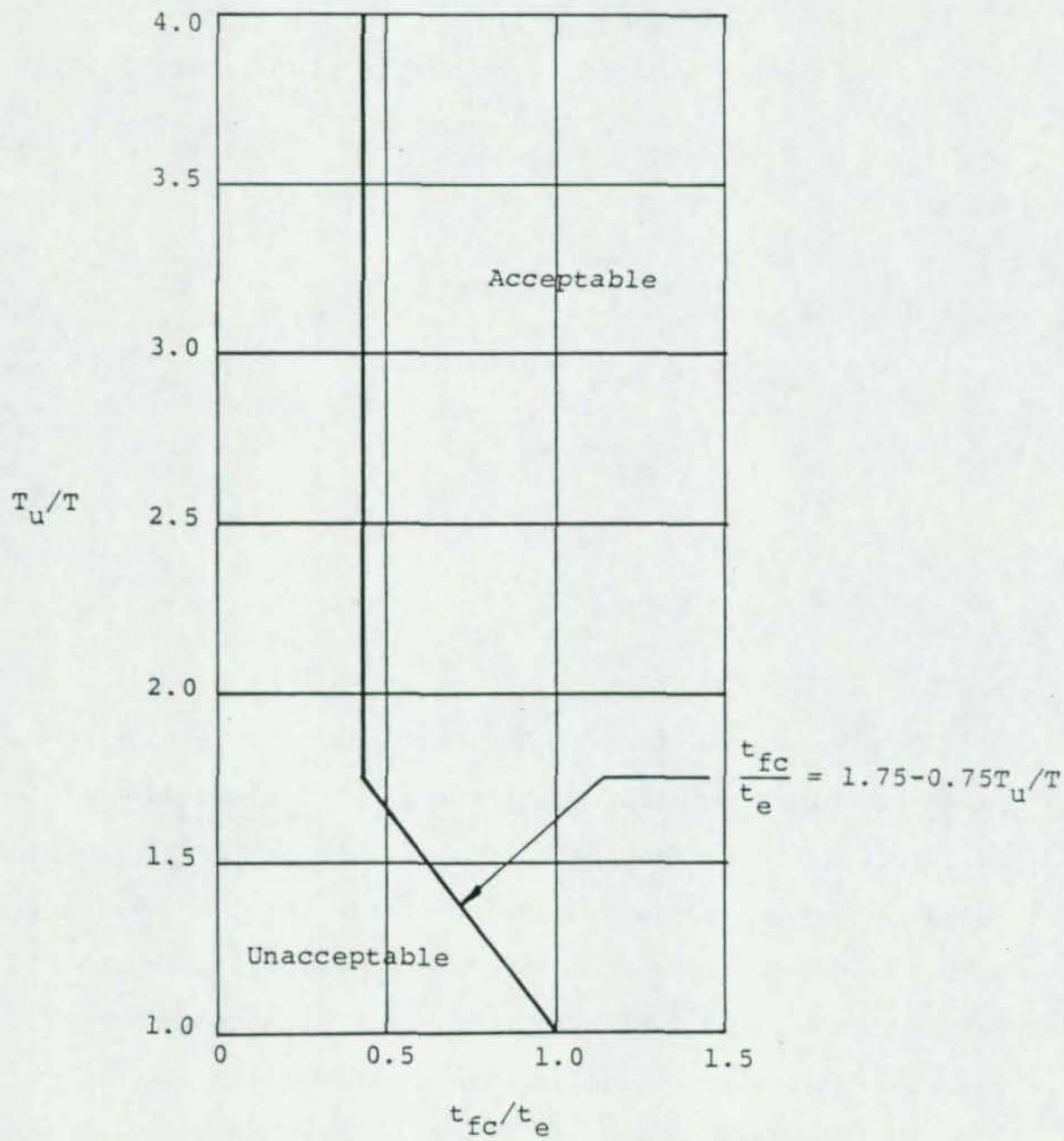


Figure 26. Granstrom's "Design Line"

1. Find a suitable number and size of bolts such that the load T tributary to each bolt does not exceed the bolt capacity.

2. Find the ratio T_u/T.

3. Find the required t_{fc}/t_e ratio from Figure 26, i.e.,

$$\frac{t_{fc}}{t_e} = 1.75 - 0.75 \frac{T_u}{T} \tag{42}$$

if $t_{fc}/t_e \geq 0.4$, otherwise

$$t_{fc}/t_e = 0.4 \tag{43}$$

4. Compute t_c from

$$t_c = \sqrt{\frac{4\ T_u\ e}{w\ F_{yc}}} \tag{44}$$

where $e = b - 2/3r_c - d_b/4$

3.2.3 End-Plate Connections

Fisher and Struik[12] state that the problem of end-plate to column connection is extremely complex and that "no satisfactory design approach is available at the present time (1973)." However, they suggest criteria based on welded beam-to-column connections, pending further research, and which result "in a conservative design" since "the concentrated forces are more localized in welded connections".

To prevent web yielding in the tension region, they suggest

$$F_t \leq F_{yc}\ t_{wc}\ (Q' + 5k) \tag{45}$$

where Q'is the sum of the beam flange thickness and twice the end-plate thickness. If Equation 45 is not satisfied, stiffeners are to be provided.

Fisher and Struik recommend the use of European criteria found in Reference 21 for stiffener requirements regarding the column flange in the tension region. The column flange is considered adequate if the moment induced by the end-plate connection on the flange over an effective length b_{eff} is within certain limits. From equilibrium on the assumed critical column flange section

$$\frac{F_t}{2} \cdot \frac{g}{2} \leq \overline{M} \tag{46}$$

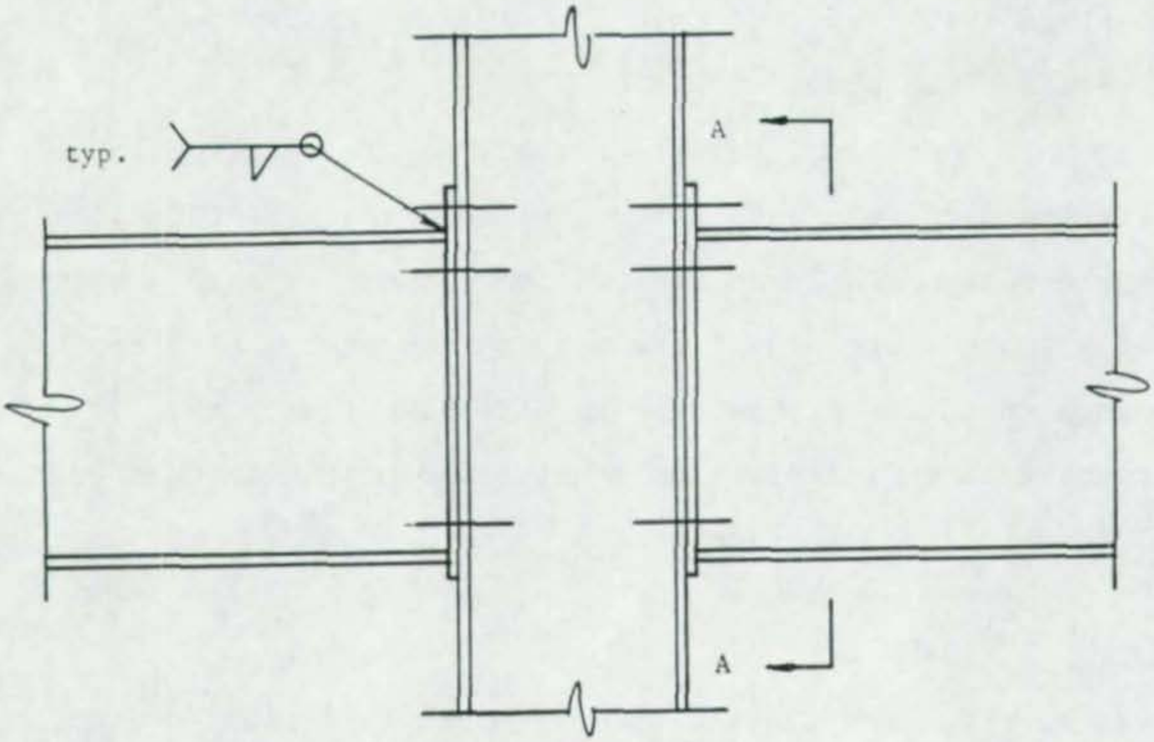
where F_t = applied tension force from the end-plate connection (kips), g = the fastener gage (in.) and $\overline{M}$ = permissible moment on the effective column flange length, b_{eff}. The effective flange length is given by

$$b_{eff} = c + \frac{3g}{2} \tag{47}$$

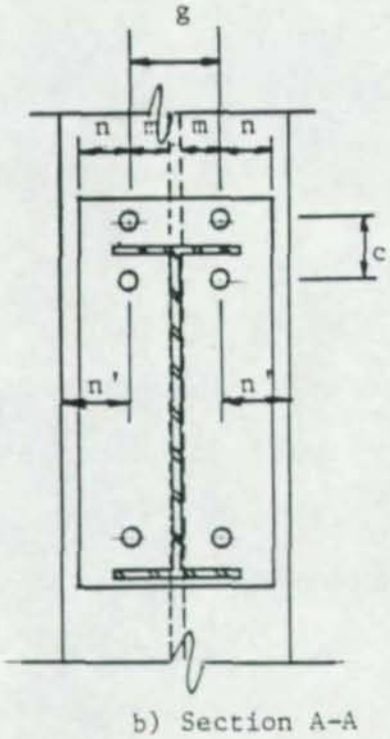
where c is defined in Figure 27. The failure moment is then

$$\overline{M} = \frac{b_{eff}\ t_{fc}^{2}}{4} \cdot F_{yc} \tag{48}$$

The authors suggest column stiffeners should be proportioned only to carry the excess concentrated force that the column web and flange are unable to resist. It is further recommended that for beam-to-column connections where only one beam frames into the column or where the moment from one beam is much greater than the other beam, the column web should be checked for shearing stresses.



a) Elevation



b) Section A-A

Figure 27. End-Plate Geometry

Witteveen, et al.[7] have also studied end-plate connections. Three modes of failure were found for the column flange. The first mode prevails when the column flanges are heavy when compared with the bolts. The failure load is

$$F_t = T_u \tag{49}$$

The second failure mode is when the stiffnesses of the bolts and flanges are such that prying forces can develop. Yield lines form in the flange near the fillet between the flange and the web and the bolts fail. The failure load is given by

$$F_t = \frac{2b_m M_p + 2T_u n}{m - 4/5r_c + n} \tag{50}$$

where b_m = effective length of the yield lines

$$b_m = 8(m-4/5r_c) + 2.5n' \text{ if } c > 4(m-4/5r_c) + 1.25n' \tag{51}$$

or

$$b_m = c + 4(m-4/5r_c) + 1.25n' \text{ if } c \leq 4(m-4/5r_c) + 1.25n' \tag{52}$$

and M_p = plastic moment capacity per unit length of the flange of the column = $t_{fc}^2 F_{yc}/4$. See Figure 27 for m, n and n'.

The third failure mode occurs when yield lines form in the flanges near the bolts and the fillet, between the flanges and the web. The failure load is

$$F_t = \frac{4b_m M_p}{(m-4/5r_c)} \tag{53}$$

Yielding of the column web is the same as that found for the compression region for situations where the beam is welded to the column flange.

$$F_t = F_{yc} t_{wc} (t_{fb} + 5k) \tag{54}$$

In end-plate connections the width of distribution of stresses is greater than that for welded connections. The authors assume this distribution depends upon the bolt location in the same way as the effective length of the yield lines, b_m, in the flange. This results in

$$F_t = F_{yc} t_{wc} b_m \tag{55}$$

Witteveen, et al.[7] recommend that the end-plate be designed to yield fully and thus behave like a tee-stub. The recommended end-plate thickness is given by

$$t_e \geq \sqrt{\frac{F_t \; m_e}{b_m \; F_{yp}}} \tag{56}$$

where F_t = the lowest value from Equation 49, 50 or 53, m_e is defined in Figure 28 and b_m is determined from

$$b_m = g + 4m_e + 1.25n \text{ if } g \leq 4m_e + 1.25n \tag{57}$$

$$b_m = 8m_e + 2.5n \text{ if } g > 4m_e + 1.25n \tag{58}$$

$$b_m \leq b_e \tag{59}$$

where g, m_e, n, and b_e are defined in Figure 28. The bolts should be designed such that

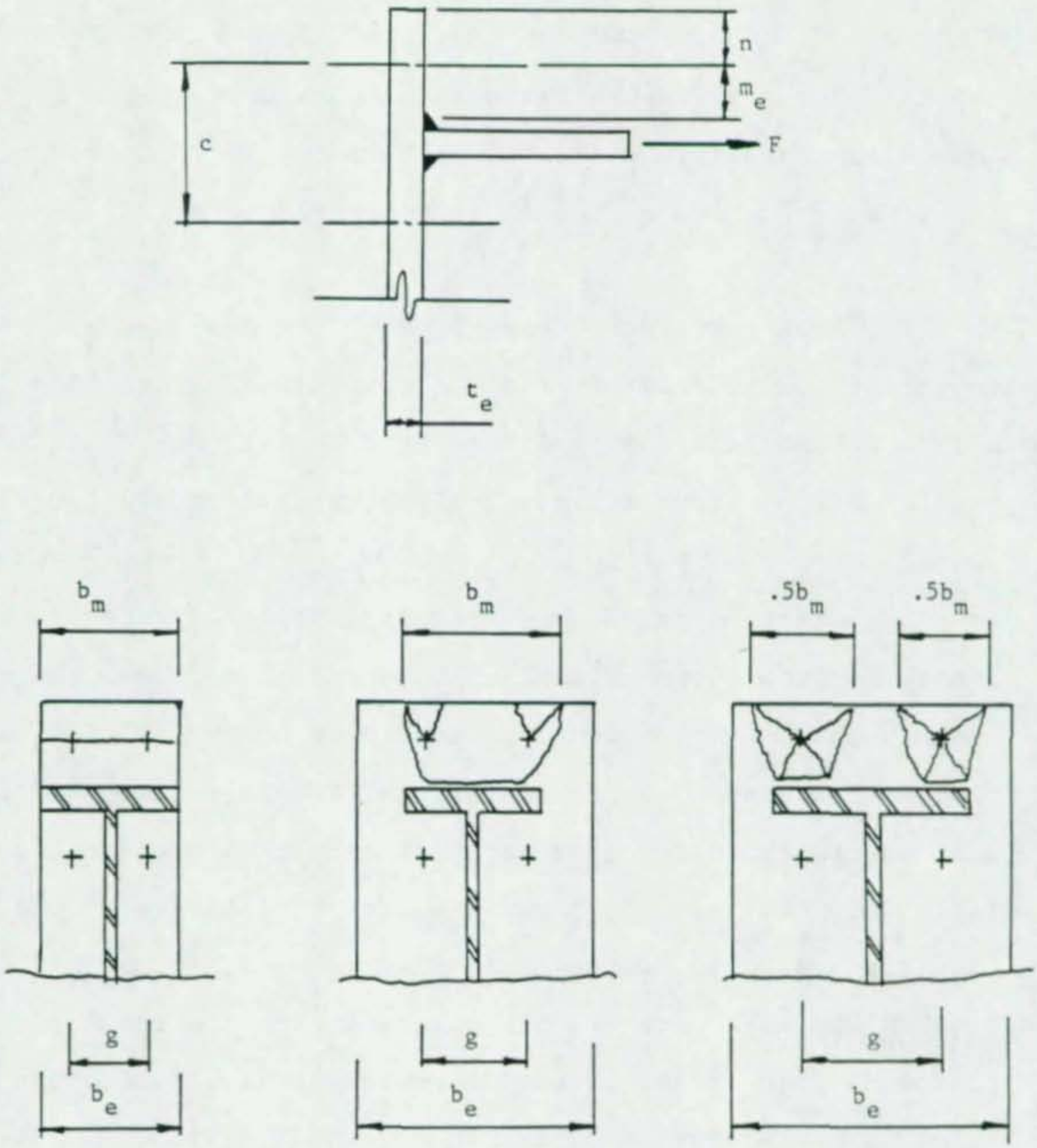


Figure 28. Effective Width of End-Plate

$$F_t \leq \frac{2b_m M_p + \Sigma T_u n}{(m + n)} \tag{60}$$

where b_m, M_p, T_u, n, and m are defined as previous.

The results are based upon tests done at the Technological University, Delft, The Netherlands by P. Zoetemeijer.[22] An extensive series of tests was conducted to verify the theory developed.

Four tests were done to check the design theory for tee-stubs. Bolt fracture was the governing failure mode for all four tests and was predicted by calculation. The results gave a safety factor against failure greater than 2 for all tests.

Nineteen tests were conducted to verify the design theory for the column flange in the tension region. The tee-stub to column connection consisted of a constant tee-stub flange thickness of approximately 1¼" and bolt pitch of 3.15". Other pertinent dimensions and test results are presented in Table 4. The first eight tests (Tests 5 to 12) were conducted to insure validity of the basic theory. Tests 13 and 14 considered the influence of bolt tightening on the results. The following five (Tests 15 to 19) checked flange stiffeners parallel to the flanges as shown in Figure 30. The final four tests investigated the bending of the bolts caused by large column flange deformations.

It was concluded from these tests that the connection

Table 4
Tee-Stub to Column Flange Test Results
(After Reference 22)

Test No.	Standard Section HE	m in.	n' in.	n in.	t_{fc} in.	F_{yc} ksi	Highest Test Load kips	F_t Eqn. 27 kips	F_t Eqn. 30 kips	Safety Factor	Failure Mode
5	140A	1.299	.965	.965	.315	37.7	49.5	54.9	27.4	-	Second
6	140A	.906	1.358	1.132	.315	37.7	67.4	69.0	35.1	-	Second
7	160A	1.299	1.260	1.260	.335	38.8	67.4	62.9	33.0	-	Second
8	160B	1.299	1.220	1.220	.492	41.8	134.9	75.5	76.9	2.67	First
9	160B	.906	1.614	1.132	.492	41.8	134.9	85.9	98.2	2.35	First
10	160M	1.142	1.378	1.260	.906	39.2	148.4	108.8	294.5	2.04	First
11	200B	1.299	1.909	1.122	.591	43.5	125.9	87.0	125.4	2.17	First
12	240B	1.024	2.835	1.260	.669	43.5	152.6	105.0	265.5	2.18	First
13	140A	1.299	.965	.965	.315	37.7	49.5	54.9	27.4	-	Second
14	140A	1.299	.965	.965	.315	37.7	40.5	54.9	27.4	-	Second
15*	160A	1.299	1.260	1.260	.335	38.8	78.7	62.7	54.6	2.16	Second
16*	160A	1.299	1.260	1.260	.335	38.8	92.2	62.7	81.4	2.21	First
17*	160A	1.299	1.260	1.260	.374	44.6	101.6	67.2	95.8	2.26	First
18*	160A	1.299	1.260	1.260	.374	44.6	103.0	67.2	95.8	2.29	First
19*	160A	1.299	1.260	1.260	.374	44.6	103.0	67.2	95.8	2.29	First
20	200B	1.299	1.909	1.122	.571	30.5	120.3	74.9	82.1	2.41	First
21	200B	1.299	1.909	1.122	.571	30.5	103.0	74.9	82.1	2.06	First
22	200B	1.299	1.909	1.122	.571	30.5	111.3	70.4	82.1	2.38	First
23	200B	1.299	1.909	1.122	.571	30.5	128.1	70.4	82.1	2.73	First

*Column stiffeners.

stiffness increases with bolt tightening, and the use of parallel stiffeners increases strength and stiffness considerably, although the stiffener plate length has little effect on the collapse load.

Table 4 comes from Reference 22 with dimensions converted to in. and kip units. Factor of safety was reported for only the first mechanism, i.e. bolt failure. The range was from 2.04 to 2.73.

Additionally, five tests were done to check if the tee-stub design method can also be used in connections where either the tee-stubs or the column flange can collapse. It was concluded that the previous design philosophy is applicable.

Twenty-three bolted moment connections, designed so that the column flange was the determining factor, were tested to verify that the moment rotation behavior was consistent with the design equations. Several different types of end connections were studied including extended end-plates, flush end-plates, tee-stubs with angle web connectors, tee-stubs without angle web connectors, and stiffened column flanges as in Figure 30. The design equations were found to be correct in all cases where the beam span in a braced frame does not exceed 30 times the beam depth. Tests with spans larger than 30 times the beam depth did not give consistently conservative results. No explanation was given by the author.

Finally, several full scale frames, designed according to the previous theories, were loaded to failure with an increasing uniformly distributed load. These tests again verified the design equations formulated for moment connections.

In summary, it was concluded that Equations 49, 50 and 53 can be used to design statically loaded, bolted beam-to-column connections. Equations 49 and 50 govern if the failure mode is bolt fracture while Equation 53 governs if collapse of the tee-stub or column flange is the determining factor. The tee-stub and column flange can be designed independent of each other. The author states that the effective length for tee-stub design cannot be greater than the tee-stub flange width, but it is not clear what is meant by this. Test results showed that stiffener plates bolted parallel to the flanges are only effective if the failure mode is in the column flange. Finally, the connections designed with Equations 49, 50 and 53 should be limited to those in which the span length of the beam does not exceed 30 times the beam depth.

Mann and Morris[6] present a design procedure for end-plate connections which is based on the work of Packer and Morris[23]. In the latter work, only the case where the column flange was much less stiff than the end-plate was studied. For this situation, three possible failure modes exist. If the flange is very stiff there are no prying forces and the failure occurs when the bolts rupture. To avoid this type of failure in a four bolt tension flange connection

$$F_t \leq 4T_u \tag{61}$$

where F_t and T_u were defined previously.

The second failure mode occurs when the column flange is less stiff which results in a combination of bolt fracture and flange yielding near the column web. This can be avoided if

$$F_t \leq F_{mb} \tag{62}$$

where

$$F_{mb} = t_{fc}^2 F_{yc} \left(3.14 + \frac{0.5c}{m+n} \right) + \frac{4T_u n}{m+n} \tag{63}$$

The quantities m, n and c are defined in Figure 27. It is suggested that to prevent bolt failure only 80% of T_u be used.

The third failure mode is when a mechanism occurs with yield lines forming so as to cause double curvature in the flange plate. Several possible yield line patterns were examined with the one giving the best fit to the experimental evidence selected. To prevent this failure mode

$$F_t \leq F_{mc} \tag{64}$$

where

$$F_{mc} = t_{fc}^2 F_{yc} \{3.14 + (2n' + c - d_h)/m\} \tag{65}$$

in which n' = distance from the edge of the column flange to the center of holes (in.), Figure 27, and d_h = bolt hole diameter. Provisions for estimating the actual bolt force are not provided if this failure mode governs.

The lower value obtained from Equations 63 and 65 determines the failure mode by which the column flange will fail. The authors state that for end-plate connections web yielding is not critical due to the wide distribution of forces into the web.

The resistance of the column flange, stiffened as shown in Figure 29, is given as

$$F_{ms} = t_{fc}^{2} F_{yc} \{(\frac{1}{v} + \frac{1}{x}) (2m + 2n' - d_h) + \frac{2v+2x-d_h}{m}\} \tag{66}$$

where

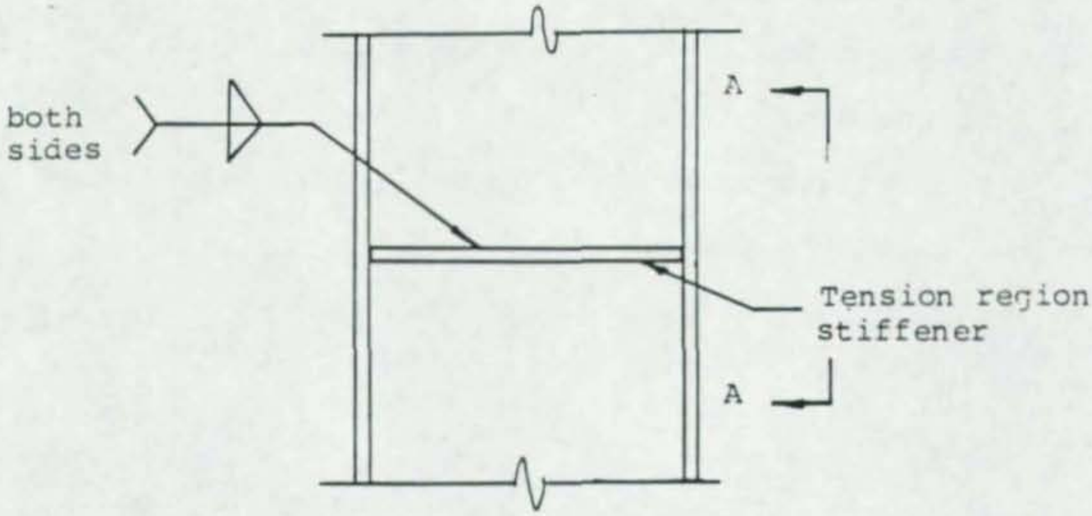
$x = \{m(m + n' - 0.5d_h)\}^{\frac{1}{2}}$, and v is as defined in Figure 29.

Mann and Morris state that Zoetemeijer [22] has recommended another form of flange reinforcement as shown in Figure 30 and results in a resistance given as

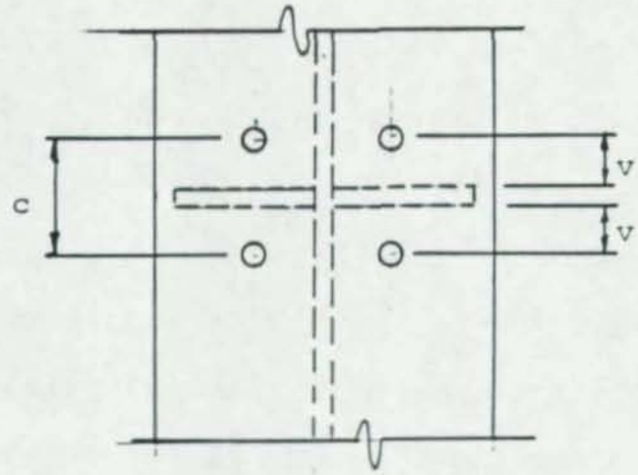
$$F_{mz} = (t_{fc}^{2} F_{yc} + \frac{t_s^{2} F_{yst}}{2}) (\frac{c + 4m + 1.25n'}{m}) \tag{67}$$

where t_s = stiffener thickness (in.) and, F_{ys} = stiffener yield stress (ksi). Stiffening the column flange in this manner induces the second mode of failure.

Packer and Morris [23] state that prying action caused by the flexibility of the column flanges could be induced thereby affecting the behavior of the tension bolts in the tension region of beam-to-column connections. The interaction between end-plates, bolts, and column flanges

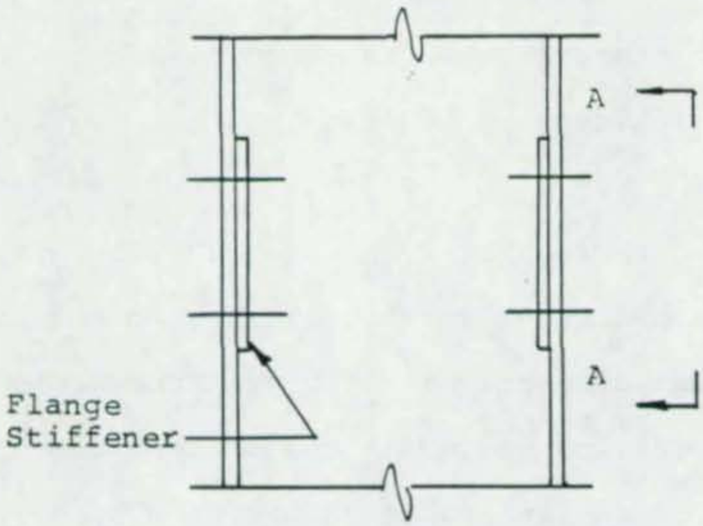


a) Elevation

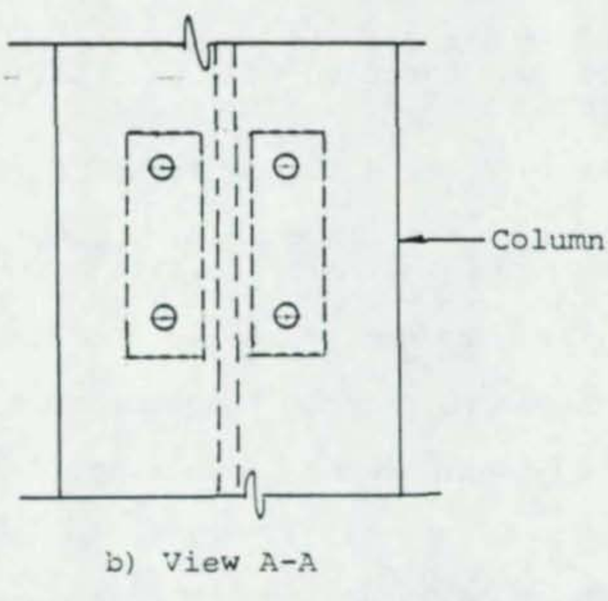


b) View A-A

Figure 29. Stiffened Column Web and Flange used in Reference 6.



a) Elevation



b) View A-A

Figure 30. Stiffened Column Flange used in Reference 22

must be considered. Zoetemeijer looked at the interaction between these components for tee-stubs connected to a flexible base, a stub-column section. He produced straight line yield patterns to represent the tee-stub flange and column flanges flexural failure.

Packer and Morris[23] allowed for curved yield line boundaries which more accurately predict the flexural yield loads in the column flanges for both stiffened and unstiffened beam-to-column connections. Applying the yield line theory to end-plates and measuring the bolt forces during connection tests, a design procedure was developed.

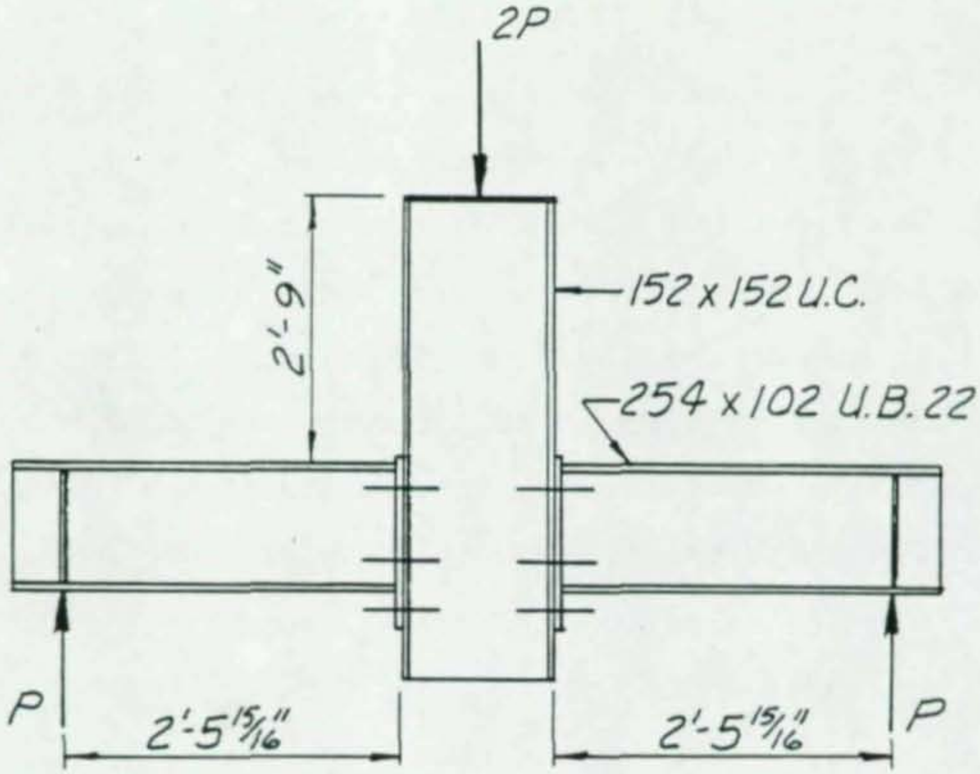
A series of eight tests on tee-stubs connected to columns representing the tension zone of an extended end-plate connection were conducted. The tee-stubs were loaded incrementally through the tee-stub web, and the joint yield load was determined by one of the following criteria:

(i) a sudden increase in the prying action force caused by column flange or tee-stub flange deformation and detected by a sudden increase in the bolt load,

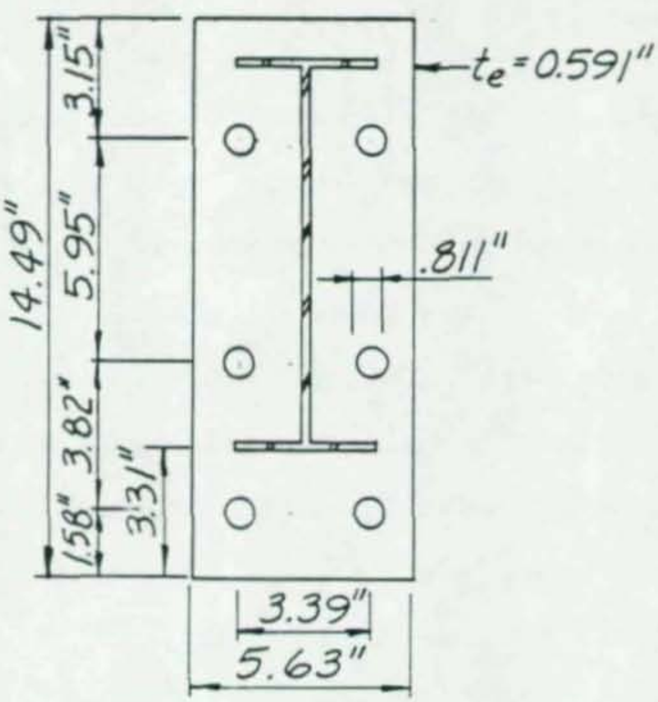
(ii) extensive column flange deflections, or

(iii) formation of a hinge pattern given by extensive cracking of the brittle resin coating the specimen.

Five beam-column joints were tested to determine if the formula developed for the tee-stub models could be applied to an end-plate moment connection. The test set-up used is shown in Figure 31. The same beam and end-plate sizes were used and the column flange thickness was varied.



(b) Full connection set-up



(a) Typical End-Plate Dimensions

Figure 31. Test Set-up Used in Reference 23

It was concluded that a tee-stub to column model of the joint in end-plate moment connections accurately represents the flexural behavior of the column flanges, with the beam flange transmitting the tension force to the column. One test was done with an axial load applied to the column and it was determined that this compressive force reduced the column flange yield moment. The results of the beam-to-column end-plate tests are presented here in Table 5.

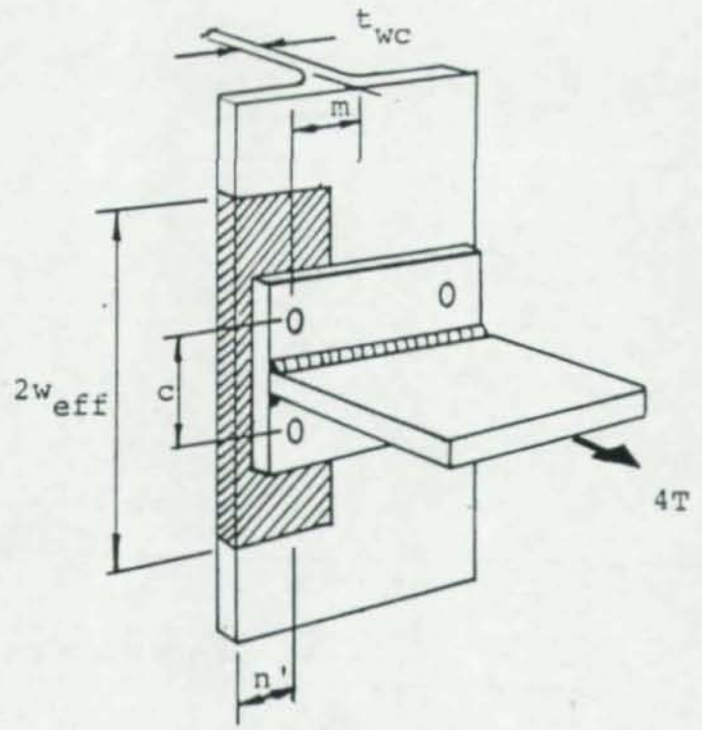
In summary, Mann and Morris recommend the following design procedure. If $F_t < F_{mb}$, and $F_t < F_{mc}$ then the column flange is adequate. On the other hand, if $F_{mc} < F_{mb}$ and $F_t < F_{mc}$ then the column flange should be stiffened. Full depth stiffeners between the column flanges as in Figure 29 give a resistance of F_{ms}. If $F_t > F_{ms}$ then a column section size should be increased to one with thicker flanges. If $F_{mb} < F_{mc}$ then the bolt size can be increased to increase P_u.

Finally, Mann and Morris state that the column web in the tension zone for end-plate connections normally is not critical since the force is usually distributed to the bolts through the end-plate before reaching the web.

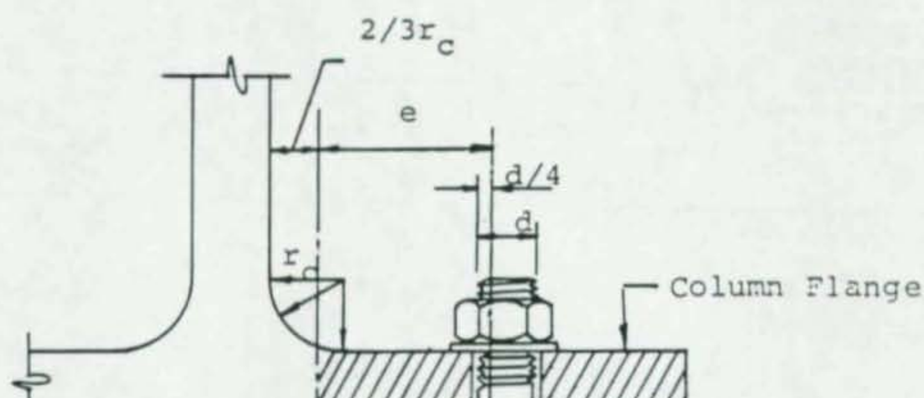
Granstrom[20] has extended his tee-hanger results to include column flanges. The procedure to determine the required column flange thickness is the same as that described in Section 3.2.2 for tee-hanger flange thickness except that w_{eff} is substituted for w. The quantity w_{eff} is defined as

Table 5
Results of Beam-Column Connection
Tests From Reference 23

	J1	J2	J3	J4	J5
Col. Flg. Thickness	.469"	.374"	.256"	.256"	.256"
Col. Type	152x152 UC37	152x152 UC30	152x152 UC23	152x152 UC23	152x152 UC23
Col. Flange Dimensions for Yield Patterns	d_h = .811"	c = 3.819"	g = 3.386"	n = 1.122"	
	m = 1.390"	m = 1.413"	M = 1.429"	m = 1.429"	m = 1.429"
	n'= 1.319"	n'= 1.319"	n'= 1.319"	v = 1.575"	n'= 1.319"
Actual Yield Moment and Mode	31.7 Mech. C in end-P_L	25.8 Mech. C in end P_L	18.4 Mech. C in col. flgs.	22.1 Mech. S in col. flgs.	15.5 Mech. C in col. flgs.
End-P_L Dimensions	n = 1.575" m = 1.339"	t_e = 0.591"			
End-P_L Failure	Predicted end-P_L collapse moment is 35.5 ft.K				
Pred./Actual	(1.12)	(1.38)	-	-	-
Predicted yield moment of column flange by:					
Pattern C1	50.6	40.2	17.6	-	17.6
Pred./Actual	-	-	(0.96)	-	(1.14)
Pattern C4	51.3	40.8	18.0	-	18.0
Pred./Actual	-	-	(0.97)	-	(1.16)
Pattern S2	-	-	-	24.0	-
Pred./Actual	-	-	-	(1.08)	-
Ultimate Moment and Failure Mode	71.8 Local beam flg. buckling.	64.7 Local beam flg. buckling	58.3 Local beam flg. buckling.	71.2 Local beam flg. buckling.	60.9 Bolt failure.



a) Tee-hanger to Column Connection



b) Reference Dimensions

Figure 32. Column Flange Model used by Granstrom

$$w_{eff} = c + 4(m-2/3r_c) + 1.25n' \tag{68}$$

where c, m, r_c and n' are defined in Figure 32. The procedure is reportedly based on the results of 21 end-plate tests.

Granstrom states that if the required flange thickness is greater than the actual thickness it may be possible to avoid the use of stiffeners by increasing the bolt diameter or pitch, or by decreasing the dimension m, or any combination. It is also suggested that the column web be checked to see that the tensile load applied by the beam flange is less than the web capacity, that is

$$F_t \leq 2w_{eff}\ t_{wc}\ F_{yc} \tag{69}$$

Stiffeners must be provided if this relationship is not satisfied. Bolts are to be sized on the basis of tributary force to each bolt without regard to possible prying action.

Tarpy and Cardinal (24) conducted tests on unstiffened beam-to-column flange end-plate connections. The end-plate thickness ranged from approximately 1.5 to 4 times the column flange thickness and sections which would require stiffeners using current design specifications were tested. The test set-up was as shown in Figure 33.

An initial column axial load was applied which was about one-third of the yield load and was maintained throughout the test. The beams were then loaded in increments until failure occurred either by excessive rotation, yielding or web buckling. The initial sign of failure was the separation

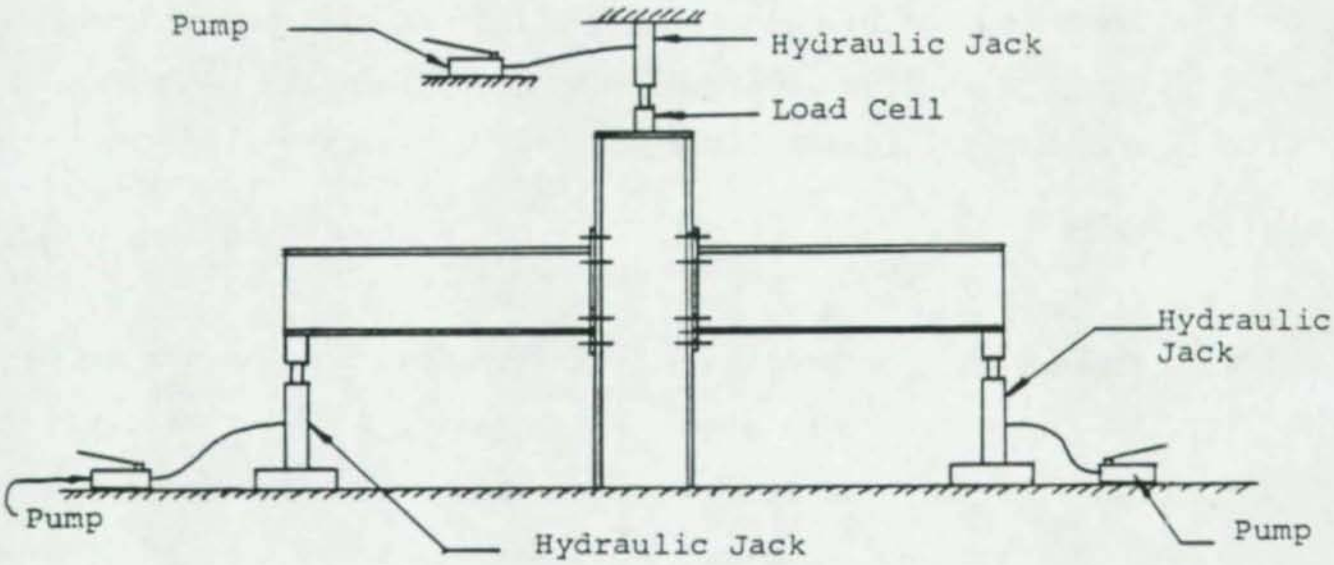


Figure 33. Test Set-up used in Reference [24]

between the vertical centerline of the column flange and the horizontal edge of the end-plate in the tension region. As the loading was increased the end-plate and column flange separated between the tension and compression regions.

An elastic analytical finite element model for the full depth of the connection was developed coupling the end-plate and column flange. The displacement of the column flange and the stress at the toe of the fillet were the dependent variables while the connection geometry parameters were the independent variables. Standard multiple linear regression analysis techniques were used to derive the prediction equations for the two independent variables.

The maximum transverse displacement of the column flange (in.) as a function of the connection geometry was predicted as:

$$\Delta = \frac{1.54 \times 10^{-6}\, b_{fc}^{\,0.76}\, g^{2.09}\, M^{1.31}}{c^{0.84}\, t_{fc}^{\,2.38}\, t_e^{\,1.84}\, d^{0.74}} \tag{70}$$

where b_{fc} = column flange width (in.), g = gage (in.), M = applied beam end moment (in.-kips), and d = beam depth (in.). In the tension region of the column flange the average stress at the toe of the fillet (ksi) was predicted as:

$$\sigma = \frac{0.344 c^{0.35}\, M^{1.55}}{t_{fc}^{\,1.35}\, t_e^{\,1.42}\, g^{0.90}\, b_{fc}^{\,0.22}\, d^{1.13}} \tag{71}$$

A moment-rotation/strength relationship was developed using the previous regression analysis. If displacements are assumed to be small, the rotation of the connection can be expressed as

$$\theta = \Delta/d \tag{72}$$

Solving for the applied moment gives the moment rotation for the unstiffened beam-to-column flange end-plate connections as

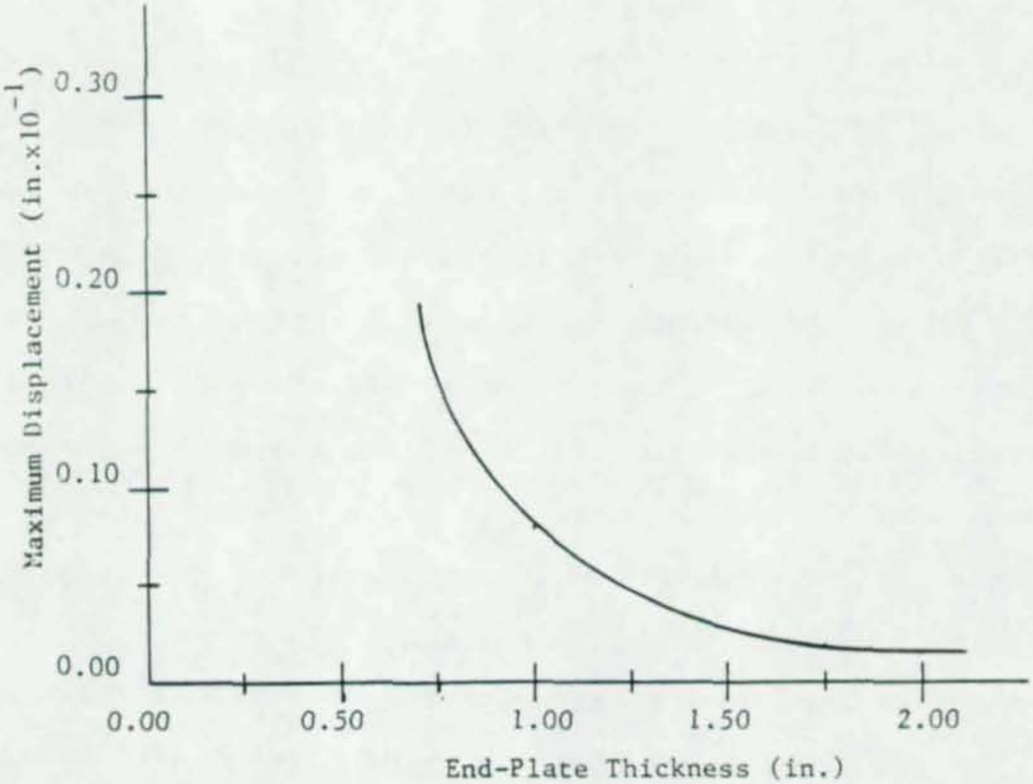
$$M = \frac{2.65 \times 10^4 \, c^{0.65} \, t_{fc}^{\,1.81} \, t_e^{\,1.40} \, d^{1.32} \, \theta^{0.76}}{b_{fc}^{\,0.58} \, g^{1.59}} \tag{73}$$

Assuming an allowable stress of $0.75F_y$ for plate bending, the allowable strength equation is expressed as

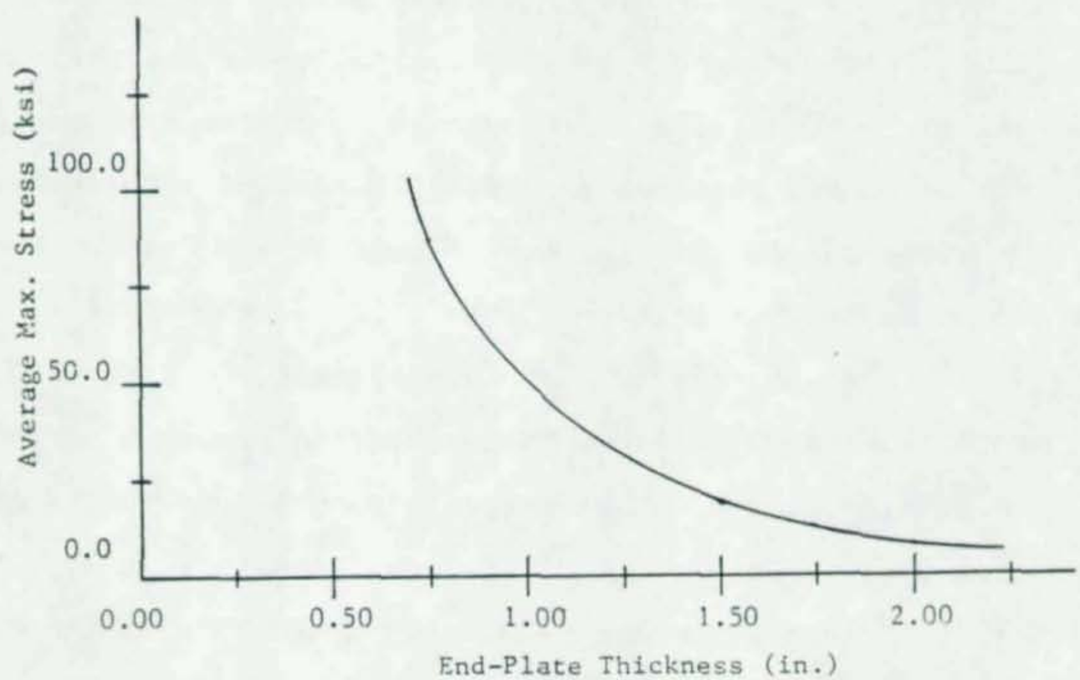
$$M = 1.65F_y^{\,0.64} \, t_e^{\,0.92} \, t_{fc}^{\,0.82} \, g^{0.14} \, d^{0.72} \, c^{-0.22} \tag{74}$$

Several different connections were investigated using the finite element model with varying end-plate and column flange properties. For the case of a thin end-plate the column flange is the primary bending element as the end-plate deforms with the flange. On the other hand, for the thick plate, the end-plate is the main bending element with the flange following the plate.

The end-plate thickness effect on the column flange can be seen in Figure 34 with the variation in maximum displacement and stress plotted versus end-plate thickness. As can be seen in Figure 34a the displacement decreases as the end-plate thickness is increased. The slope of the curve decreases as the end-plate thickness increases with about a 30% decrease in going from 0.75" to 1.0" but only about 1% from 1.15" to 1.75".



(a) Maximum Displacement Versus End-Plate Thickness



(b) Maximum Bending Stress Versus End-Plate Thickness

Figure 34. Influence of End-Plate Thickness on Displacements and Stresses from Reference 24

The stress variation is very similar to that of the displacements. "Because the point of maximum stress varies for different end-plate thicknesses, the stress shown in Figure 34b is the average of the stresses along the web center line in the tension region for an effective distance fo approximately the vertical pitch of the bolts plus the horizontal gage distance". The change in slope of stress is not as large as that of the displacements with the stress going down about 24% as the end-plate thickness goes from 0.75" to 1.5".

Based upon moment-rotation curves, the analytical model was found to be adequate by comparison with experimental results. The end-plate thickness is described as a "key parameter" on the response of the connection.

Krishnamurthy[25] and his associates have developed finite element methodology specifically for the analysis of end-plate connections. An exhaustive analytical study of unstiffened end-plates along with a series of experimental investigations lead to the development of the design procedure found in the 8th edition of the AISC Design Manual[10].

Krishnamurthy also investigated, to a limited extent, the behavior of stiffened tee-stubs and proposed a design methodology based on statistical analyses of results obtained from various cases considered in a parametric study. The study yielded favorable results insofar as reduction of the plate thickness was concerned.

Krishnamurthy contends that even though prying action is present, it is overly conservative to assume it to be acting at the edge of the plate as this normally results in thicker than necessary end-plates. His studies explain the prying force as a pressure bulb which is formed under the bolt head due to the pretensioning of the bolt and shifts towards the edge as the beam flange force increases. For any given loading the pressure bulb is located somewhere between the edge of the end-plate and the bolt head. In fact, for service load conditions, when the beam flange loads are small, it is more towards the bolt head than towards the edge and the plate moments are much smaller than those predicted by prying force formulas. Subsequently, Krishnamurthy abandoned this older approach and used results of the finite element analysis to develop a design method.

Krishnamurthy's design procedure for end-plates as presented in the eighth edition of the AISC Manual of Steel Construction[10] is as follows. The flange force applied by the tension flange of the beam is calculated as

$$F_f = \frac{M}{(d-t_{fb})} \qquad (75)$$

where all terms are defined as previous. The required nominal bolt area per bolt is computed as

$$A_b = \frac{F_f}{2n_b F_{ten}} \qquad (76)$$

where n_b = number of bolts on one transverse line, and F_{ten} = allowable tensile stress per bolt, ksi.

The effective span used to compute the bending moment in the end-plate may be taken as

$$p_e = p_f - (d_b/4) - w_t \tag{77}$$

where p_f = distance from centerline of bolt to nearer surface of the tension flange, in., $d_b + \frac{1}{2}$" is generally enough to provide wrench clearance, and w_t = fillet weld throat size or reinforcement of groove weld, in.

The end-plate is designed to resist the effective plate bending moment

$$M_e = \alpha_m F_f p_e/4 \tag{78}$$

where $\alpha_m = C_a C_b (A_f/A_w)^{1/3}(p_e/d_b)^{1/4}$, C_a = constant, (See Table A, P4-113, Steel Manual), $C_b = (b_f/b_e)^{1/2}$; A_f = area of tension flange, A_w = web area clear of flanges, b_f = beam flange width, in., and b_e = required plate width $\leq 1.15b_f$. The required plate thickness is

$$t_e = \left[\frac{6M_e}{b_e F_b}\right]^{\frac{1}{2}} \tag{79}$$

where $F_b = 0.75F_y$.

3.3 Evaluation of Procedures

3.3.1 Comparison of Methods

From the literature survey it was learned that several design criteria are available to prevent web yielding and flange failure in the tension region of beam-to-column end-plate moment connections. General agreement was found in the criteria used to prevent web yielding. Equations 45, 55 and 69 recommended by Fisher and Struik[12], Granstrom[20] and Witteveen et al.[7], respectively, are very similar. Equation 45 was apparently deduced from results for the compression region of beam-to-column end-plate connection tests and has not been substantiated by tests of the tension region. Equations 55 and 69 are essentially identical and seem to be based on adequate testing at least for European sections.

Criteria to prevent flange yielding vary widely in the five methods reviewed. The Fisher and Struik[12] recommended procedure is based on single curvature bending of the column over an effective length b_{eff} (Equations 46 and 47). Mann and Morris[6] consider three failure modes (Equations 61, 62 and 64) and include provisions for estimating the actual bolt force in two of the three modes. The criteria are somewhat complicated for routine design, but seem to be based on sound analytical and experimental evidence. The procedure recommended by Morris is an extension of Witeveen, et al.[7] in which Morris considered curved yield line boundaries. Granstrom's[20] recommendations are based on the analysis of test results. The procedure is relatively simple and suitable for routine design. Tarpy and Cardinal[24] results are based on finite element analyses with experimental verification. The final design equation (Equation 73) was developed from regression analysis of finite element results. The final equation is rather complicated for routine design use but is similar in form to the equation for end-plate design in the 8th ed. AISC design manual[10].

3.2.2 Numeric Evaluation

To better understand the implications of each of the five procedures, a computer program was written to determine required column flange thickness for end-plate connections for commonly used North American W-sections. End-plate thickness and connection geometry for specific beams was determined using the procedures in the 8th ed. AISC Manual of Steel Construction[10]. Reference 26 was used to obtain specific designs. The tension force delivered to the connection was calculated in accordance with the procedure in Reference 10:

$$F_t = \frac{5}{3} \frac{M_b}{(d-t_{fb})} \tag{90}$$

but not to exceed $A_f F_{yb}$ where M_b = actual beam moment. For all cases, the end-plate width was taken equal to the beam flange width.

The selected beams were analyzed at load levels corresponding to $0.5S_xF_{bx}$, $0.75S_xF_{bx}$ and $1.0S_xF_{bx}$ where S_x = strong axis section modulus and $F_{bx} = 0.66F_{yb}$. For this study F_{yb} was taken as 36 ksi. The required column flange thicknesses are shown in Tables C.1, C.2 and C.3 of Appendix C. Tarpy and Cardinal[24] gave thicknesses much larger than are practical and have been eliminated from the table.

Results from a modified version of Krishnamurthy's end-plate design method[25] are included in the table. The method is modified so that the column web simulates the beam flange in the original design procedure and there is no beam web. Since $A_w = 0$ there is no justification for taking any value of the ratio A_f/A_w other than infinity. A value of $A_f/A_w = 1.0$ was taken for comparison purposes only and cannot be theoretically justified. An effective flange length equal to 3.5 times the bolt pitch, c, was used in the calculations. Again, this value cannot be justified theoretically. For comparison purposes, the end-plate width was taken equal to the beam flange width, i.e., $b_f/b_e = 1.0$ in the Krishnamurthy formulation.

As can be seen in the tables the values for Witeveen et al.[7] and Fisher and Struik[12] are considerably larger than the remaining three methods for all cases. Granstrom[20] fluctuates from much less than Mann and Morris[6] to being very nearly equal. The values obtained from the modified Krishnamurthy procedure agree very closely with those of Mann and Morris[6]. Required column thicknesses using the modified Krishnamurthy procedure are slightly less (less than 0.1 in.) for the full allowable stress and slightly greater (less than 0.15 in.) for one-half of the allowable stress.

3.4 Testing Program

3.4.1 Scope

It is evident that a considerable amount of research and testing concerning the column tension region at bolted end-plate moment connections has been completed. However, as was shown, results from the various design procedures vary widely (Appendix C). Further, few tests have been conducted using North American sections. Thus, a small scale testing program was conducted to verify results from previously conducted studies for use with North American sections.

Four tests were conducted with combinations of beam and column sections as shown in Table 6. Extended end-plates with bolts on each side of the beam tension flange were used for all tests. The tee-beam sections and bolts were the same as used in the compression web tests (Section 2.4) except that the end-plate stiffeners were removed when necessary. As previously mentioned, the plate thickness and bolt size for each tee-beam were determined using the procedure found in the 8th edition AISC Manual of Steel Construction[10].

Standard tensile coupon tests were made using samples cut from each of the column webs. Results are given in Table 3. The measure yield stresses varied from 34.6 ksi to 39.7 ksi.

3.4.2 Test Set-up and Procedure

General details of the test set-up are shown in Figure 35; Figure 36 is a photograph of the set-up. The column was

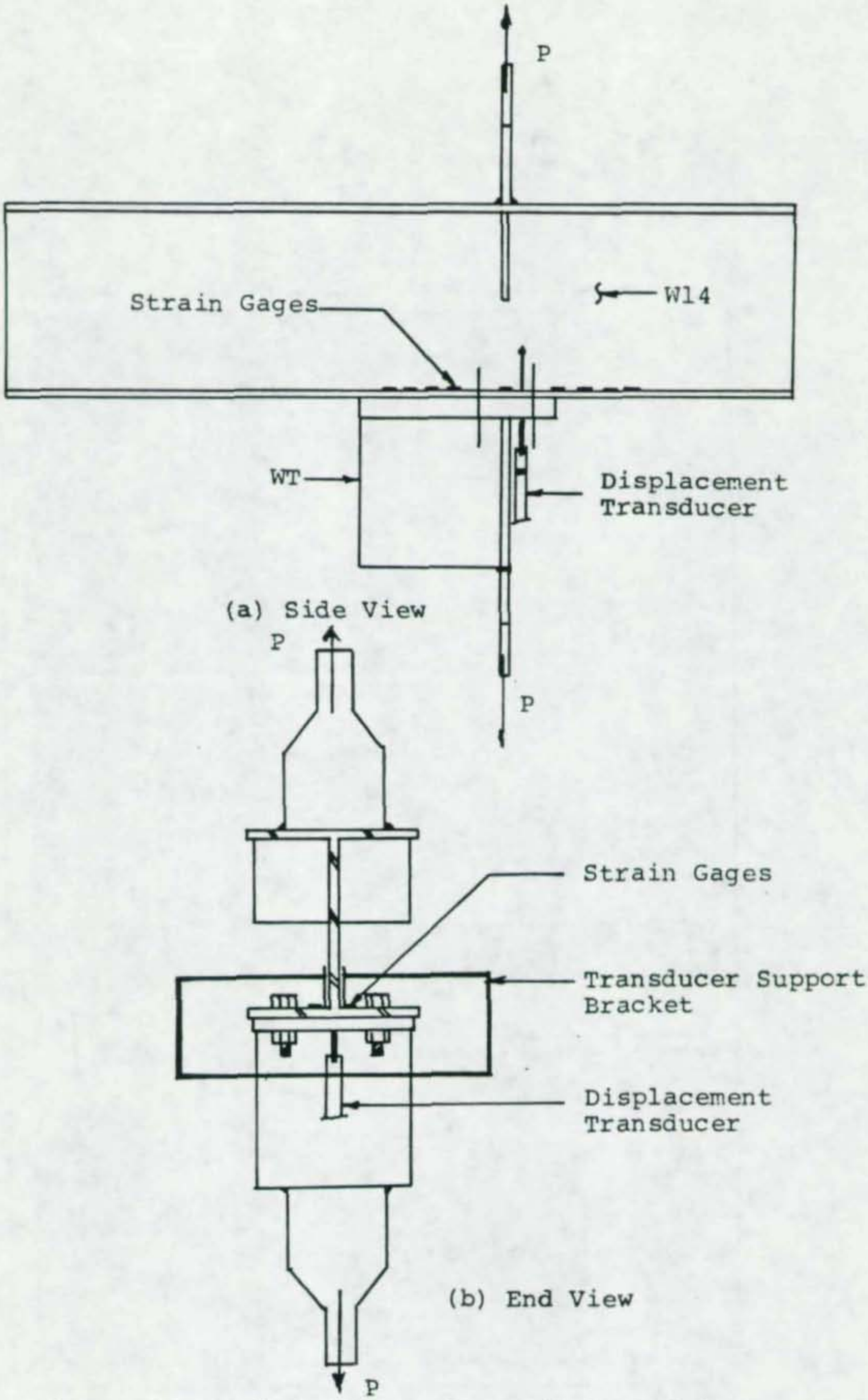


Figure 35. Column Flange Strength Test Set-up

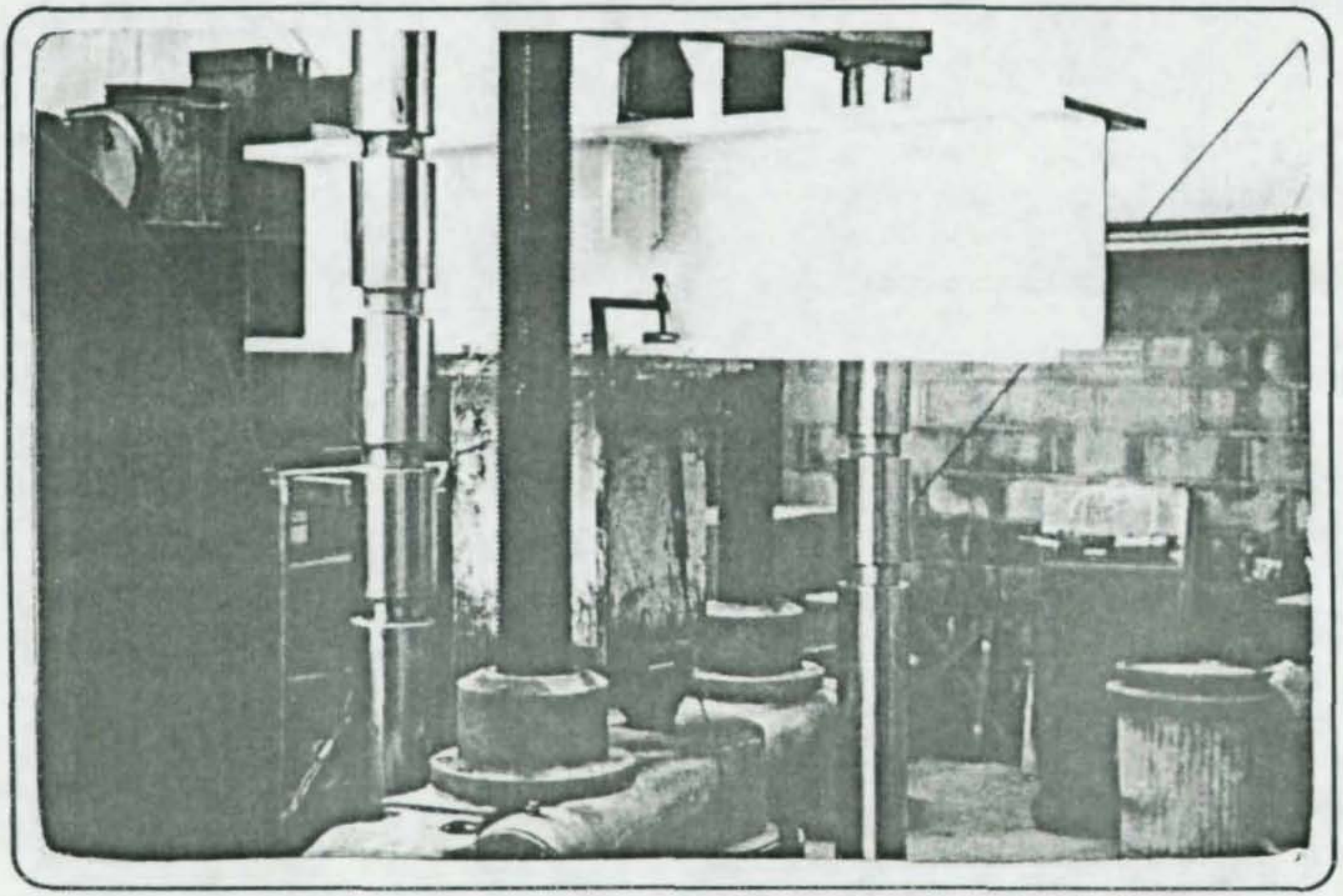


Figure 36. Photograph of Flange Strength Test Set-up

placed in a horizontal position with the load applied through a plate welded to the flange of the WT section and a second plate welded to the column flange opposite the connection. A stiffener plate was welded opposite the attached plate to prevent failure of the column flange on the opposite side from the end-plate. Load was applied using a 200 kip capacity, universal type testing machine.

Instrumentation consisted of strain gages and a displacement transducer. For all four tests, strain was measured on the inside of the column flange on each side of the web just outside the fillet radius and over a length of 18 in. centered at the WT flange, Figure 35. This length is greater than the effective flange width given by any of the analytical methods previously reviewed. A single displacement transducer was used to measure separation between the column flange and end-plate as shown in Figure 35. A Hewlett-Packard 3497 Data Acquisition/Control Unit was used with an HP 85 desk top computer to collect, record and plot data.

At the beginning of each test the specimen was loaded 40 kips to check all instrumentation. The specimen was then unloaded and initial strain and displacement readings were taken for zero load. The specimen was then loaded in 10 kip increments with readings of all instrumentation recorded at each increment. A load-deflection curve was plotted to monitor any nonlinearity. The loading was continued until the 200 kip capacity was reached.

3.4.3 Test Results

Test results consist of load versus plate separation and load versus stress at the strain gage locations. Stress was computed from measured strains assuming a modulus of elasticity of 29000 ksi, but with an upper limit of the measured yield stress. For each test, stress distributions were plotted at various load levels to show progression of yielding in the column flange. On each of these plots, the predicted failure loads using the Granstrom[20] and Mann and Morris[6] procedures are also shown. Summaries of data are found in Tables 6 and 7 and results are discussed in the following paragraphs.

Test 1. Test 1 consisted of a WT9x48.5 beam with a W14x84 column. The WT section was attached to the column flange with 1 3/8" diameter A325 bolts through a 1 1/2" thick end-plate. The material yield stress obtained from a coupon test was 38.3 ksi.

The measured load versus plate separation curve, Figure 37, remained linear to approximately 110 kips. A second break occurred at approximately 160 kips and a third at 180 kips. The failure load predicted by the Mann and Morris[6] procedure was 152 kips and by the Granstrom procedure, 179 kips.

Table 6
Column Flange Strength Tests

Test	Beam	Column	End-Plate Thickness (in)	Bolt Diameter (in)	Yield Stress (ksi)	0.02"* (kips)	Mann & Morris (kips)	Granstrom (kips)
1	WT9x48.5	W14x84	1½	1 3/8	38.3	157	152	179
2	WT9x48.5	W14x103	1½	1 3/8	34.6	138	191	183
3	WT9x25	W14x78	7/8	1 1/8	39.7	50	120	114
4	WT16.5x 70.5	W14x78	1¼	1 1/2	39.7	141	130	184

*Plate Separation of 0.02'

Table 7
Effective Column Flange Width

Test	Beam	Column	Mann & Morris		Granstrom	
			Required (in)	Measured* (in)	Required (in)	Measured* (in)
1	WT9x48.5	W14x84	16.9	9.5	17.2	11.5
2	WT9x48.5	W14x103	18.3	13.7	18.6	12.5
3	WT9x25	W14x78	15.7	9.2	16.1	9.5
4	WT16.5x 70.5	W14x78	16.8	9.0	17.2	12.7

*Yield length at predicted failure load

From Figure 38, it is evident the column flange was yielded over a considerable length at maximum load. Initial yielding occurred at 140 kips away from the WT flange location. This phenomenon did not occur in the remaining tests. The effective column flange length using the Mann and Morris procedure is 1.69 in. and the measured yield length, from Figure 38, at the Mann and Morris failure load is 9.5 in. The required and measured yield lengths for the Granstrom procedure are 17.2 in. and 11.5 in., respectively.

Test 2. Test 2 was made up of a WT9x48.5 beam attached to a W14x103 column section with 1 3/8" diameter A325 bolts. The end-plate was 1 1/2" thick. A material yield stress of 34.6 ksi was obtained from a coupon test.

The experimental load versus plate separation curve shown in Figure 39 was linear to approximately 120 kips. A second break occurred at approximately 160 kips. The Mann and Morris predicted failure load was 191 kips with Granstrom predicting essentially the same load at 183 kips.

From Figure 40, the column flange stress distribution can be seen to be spread over the length of the strain gage locations, the majority of which are yielded at maximum load. For Test 2, the initial yielding took place opposite the beam flange at 120 kips.

The calculated effective flanges using the Mann and Morris and Granstrom procedures were 18.3 in. and 18.6 in., respectively. The measured yield lengths at the corresponding predicted loads were 13.7 in. and 12.5 in.

Test 3. A W14x78 column section was used in Test 3 with a WT9x25 beam. A 7/8" thick end-plate was used and was attached with 1 1/8" diameter A325 bolts. From a coupon test the material yield stress was found to be 39.7 ksi.

Figure 41 shows the experimental load versus plate separation curve. The curve remains linear to approximately 60 kips where the first break occurs. The second break occurs at approximately 140 kips and the third near 180 kips. The Mann and Morris predicted failure load was 120 kips and the Granstrom prediction was 114 kips.

The flange stress distribution is plotted again at the various load levels in Figure 42. As in the previous two tests, the column flange is yielded over a considerable length at the maximum load of 200 kips. The initial yielding again occurred at the center gage at 120 kips.

Calculated and measured flange yield lengths for the Mann and Morris procedure were 15.7 in. and 9.2 in. and the Granstrom procedure 16.1 in. and 7.5 in.

Test 4. Test 4 consists of a WT16.5x70.5 beam and a W14x78 column. The 1 1/4" thick end-plate was bolted to the column with 1 1/2" diameter A325 bolts. The material yield stress from a coupon test was found to be 39.7 ksi.

The first break from linearity occurs at about 110 kips. The second break takes place at approximately 150 kips and the third near 170 kips. The Mann and Morris predicted failure load was at 130 kips with Granstrom considerably higher at 184 kips.

As seen in Figure 44 the column flange stress distribution again is spread over the length of the gages attached to the flange. As in Tests 2 and 3 the initial yielding occurred at the center gage at approximately 110 kips.

The calculated effective flange length from the Mann and Morris procedure was 16.8 in. and the measured yield length at the predicted failure load, Figure 44, was 9.0 in. The corresponding lengths from the Granstrom procedure are 17.2 in. and 12.7 in.

3.4.4 Column Flange Strength

The test results presented in Section 3.4.3 for the column tension region at end-plate connections show that the maximum load the column flange can sustain is greater than values predicted by either the Mann and Morris or Granstrom procedures. However, plate separation is considerable at higher loads.

For the tests reported here, the Mann and Morris procedure generally gives conservative results when compared with Granstrom results. The exception is Test 2 where the predicted failure loads from both procedures are essentially the same although Granstrom is slightly lower. For Test 3,

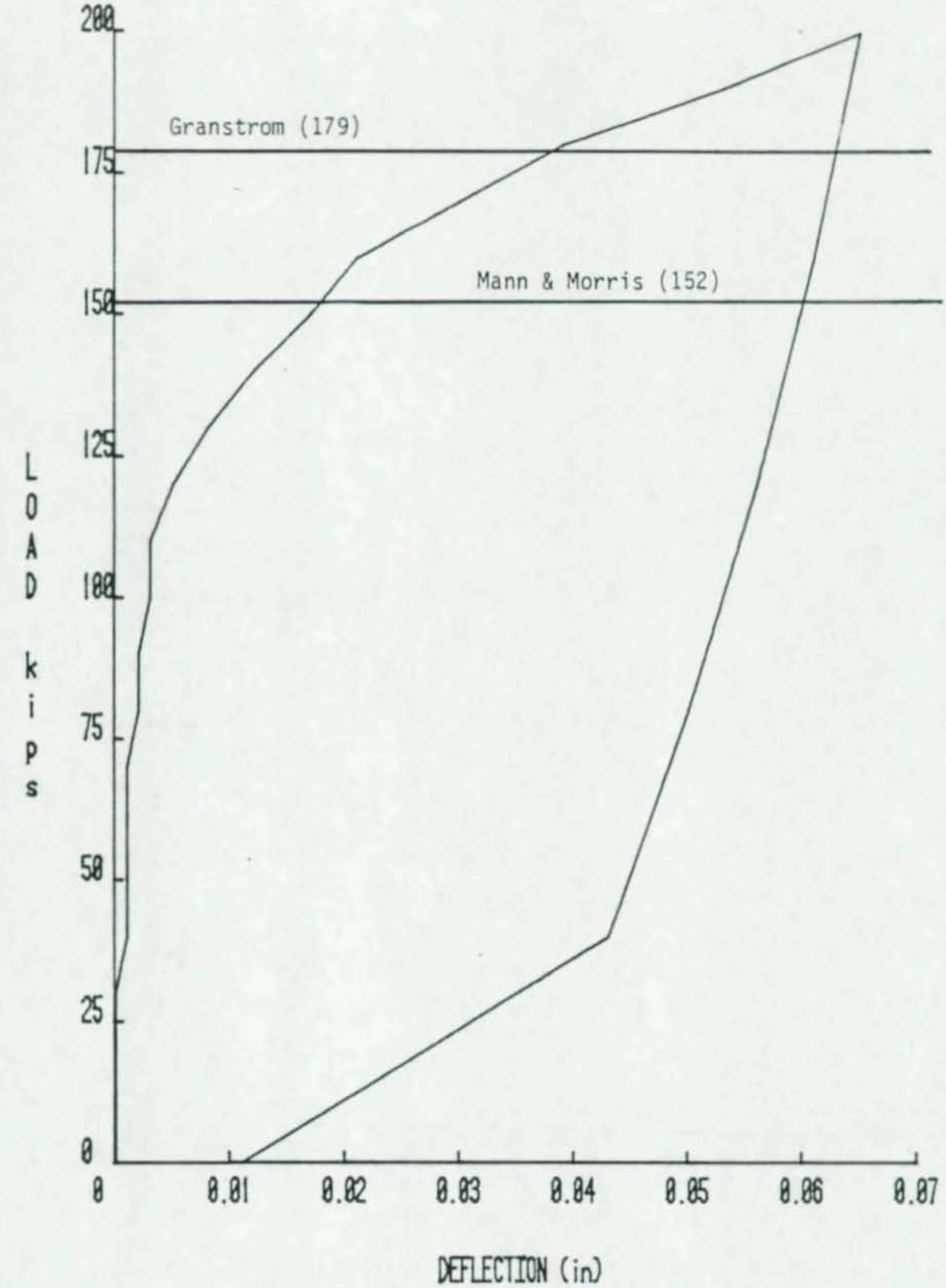


Figure 37. Load vs. Plate Separation - Test 1

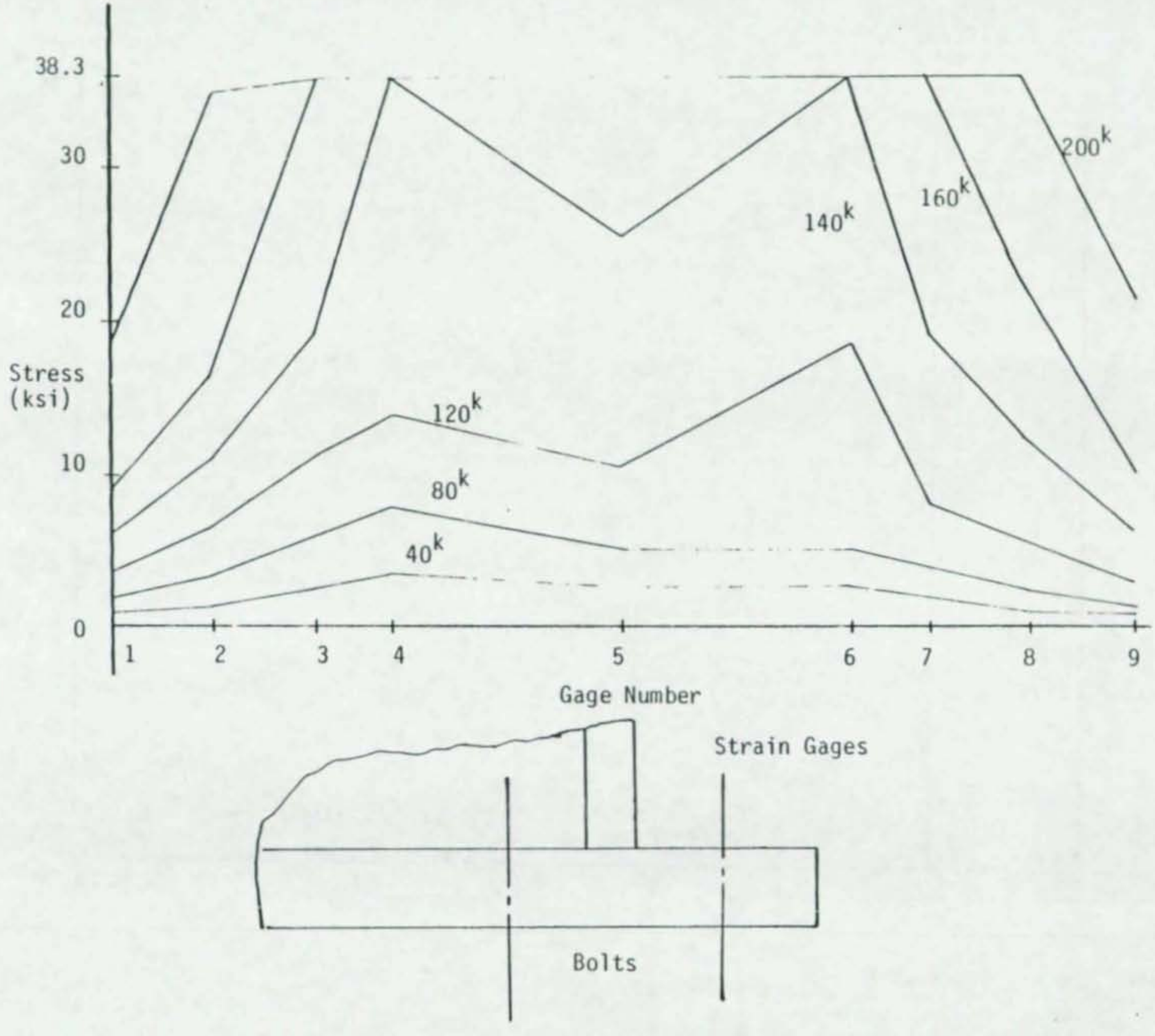


Figure 38. Flange Stress Distribution - Test 1

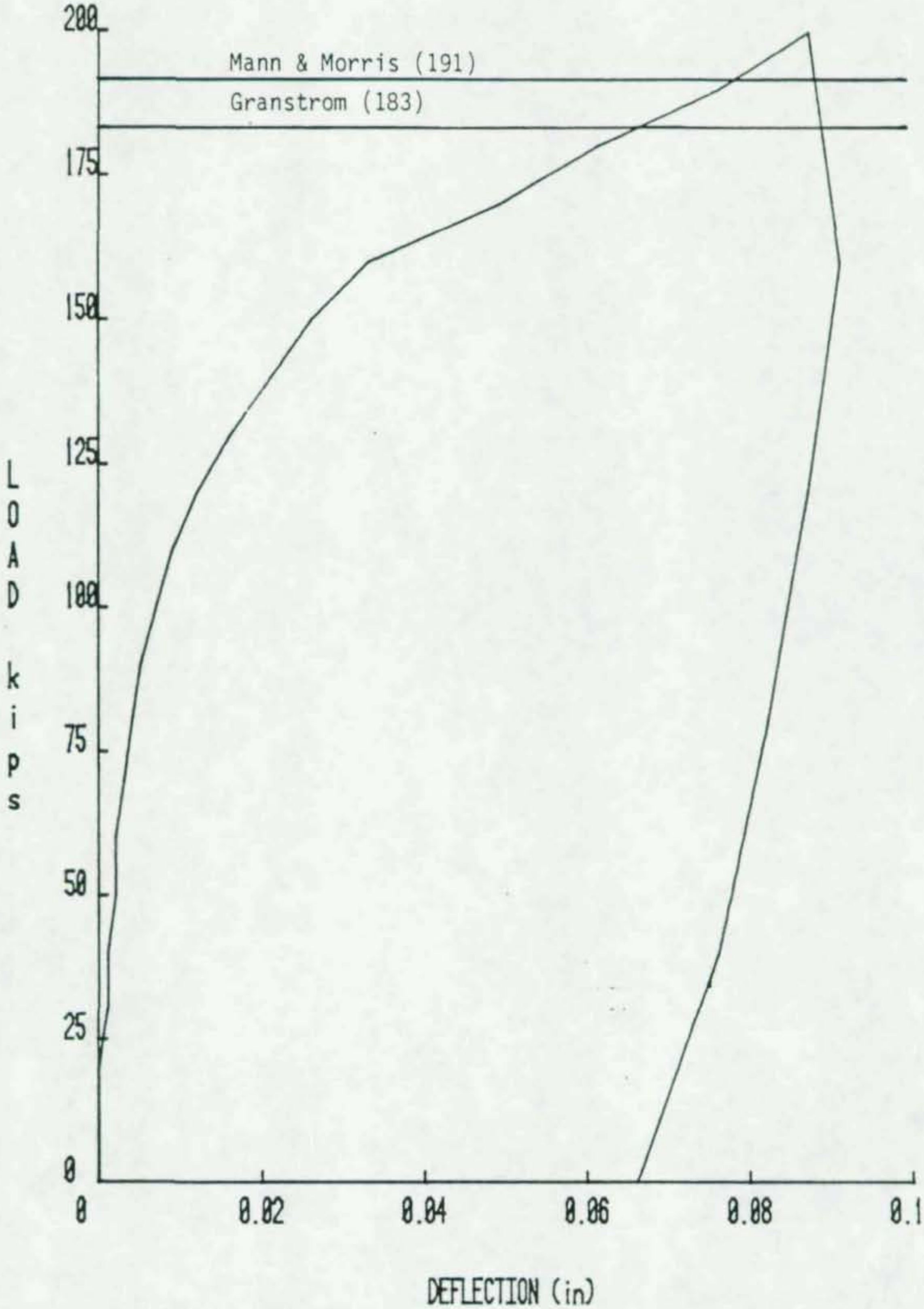


Figure 39. Load vs. Plate Separation - Test 2

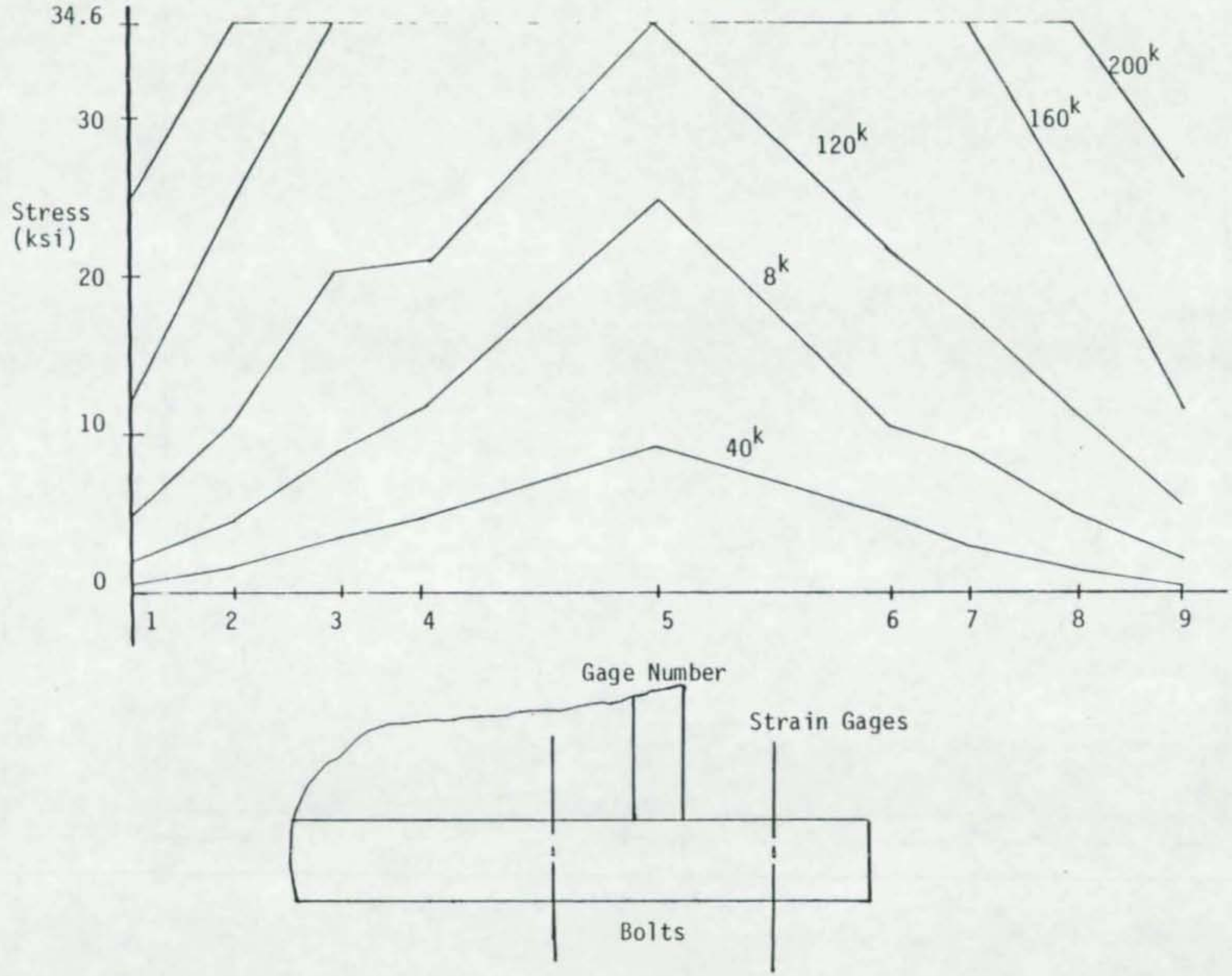


Figure 40. Flange Stress Distribution - Test 2

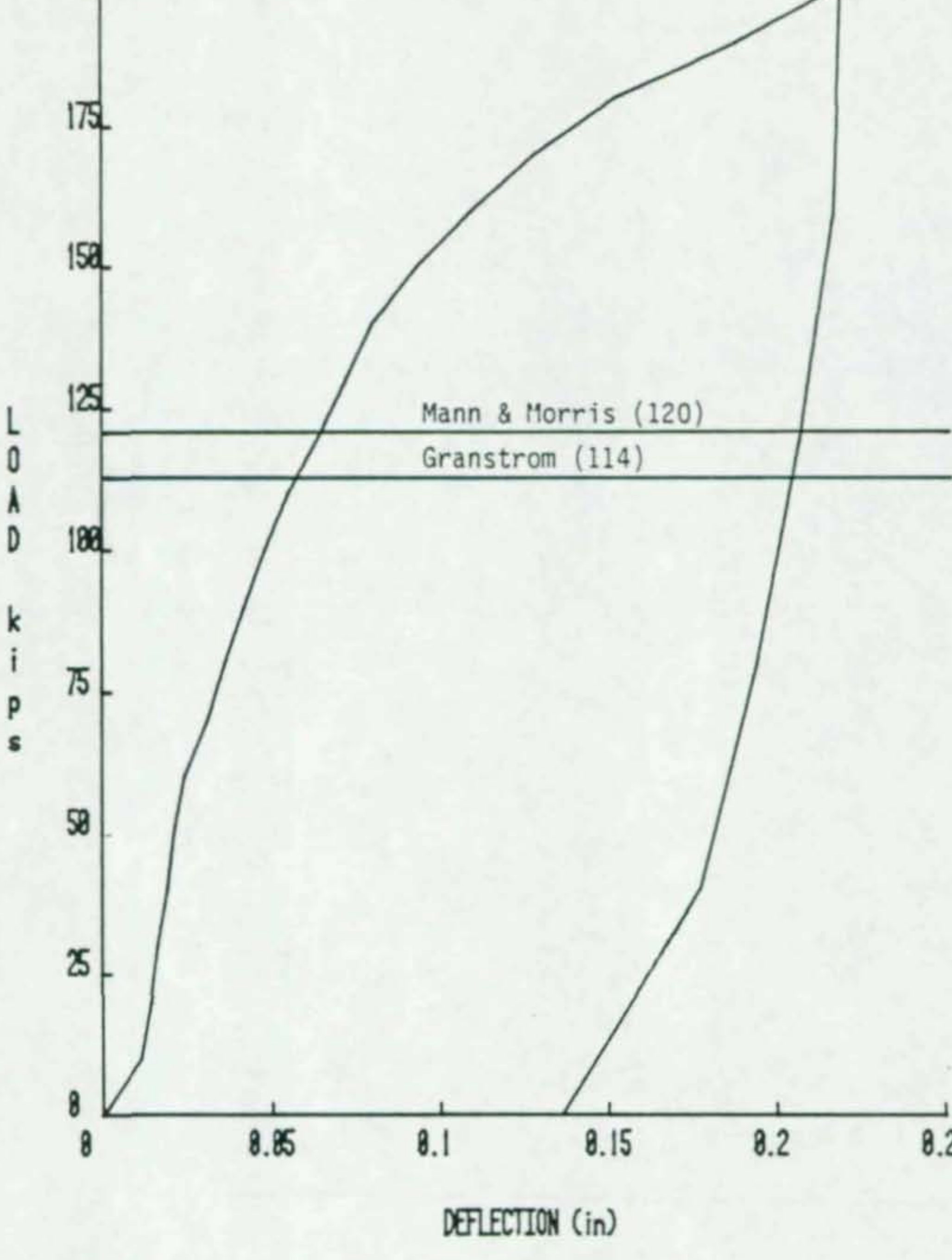


Figure 41. Load vs. Plate Separation - Test 3

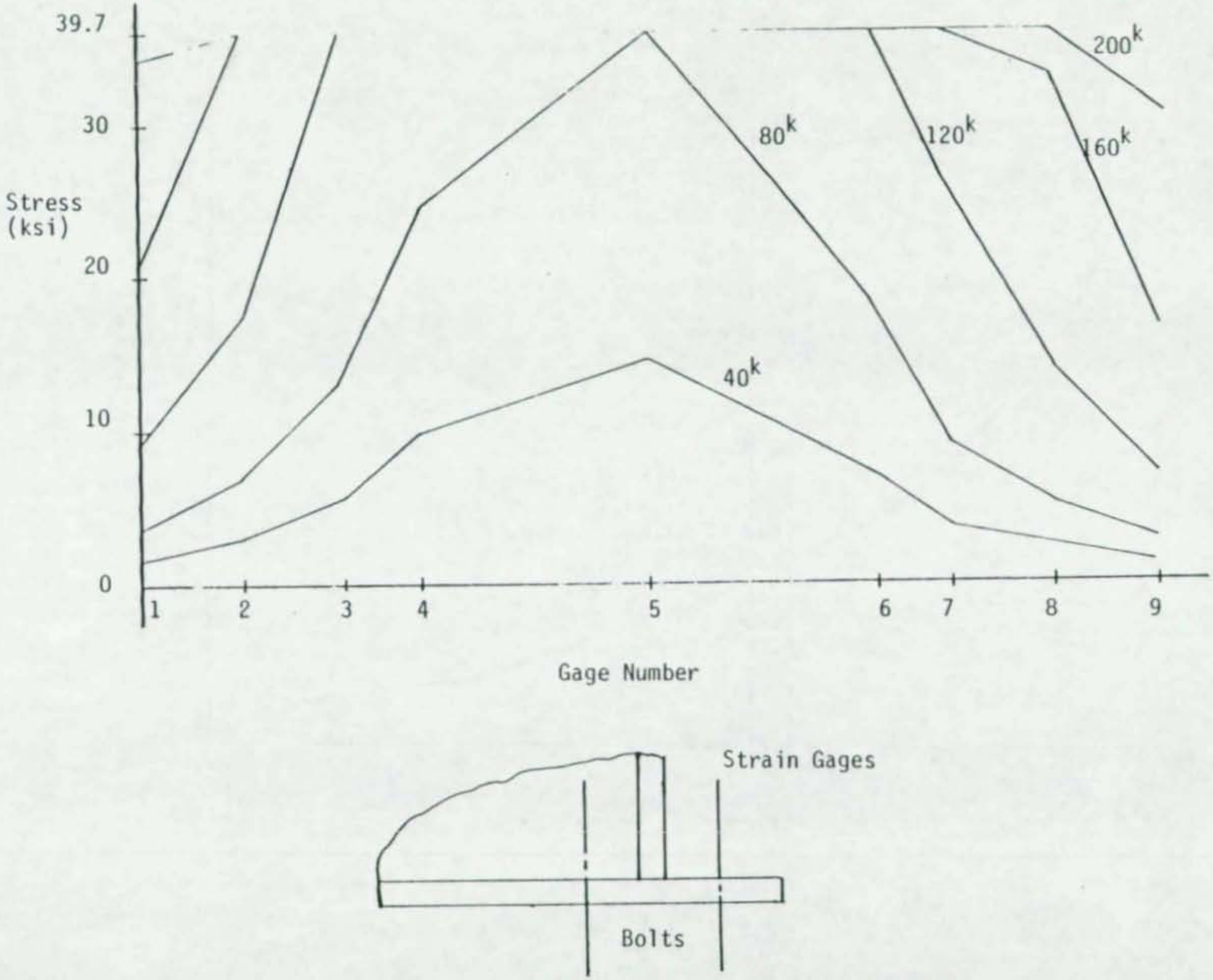


Figure 42. Flange Stress Distribution - Test 3

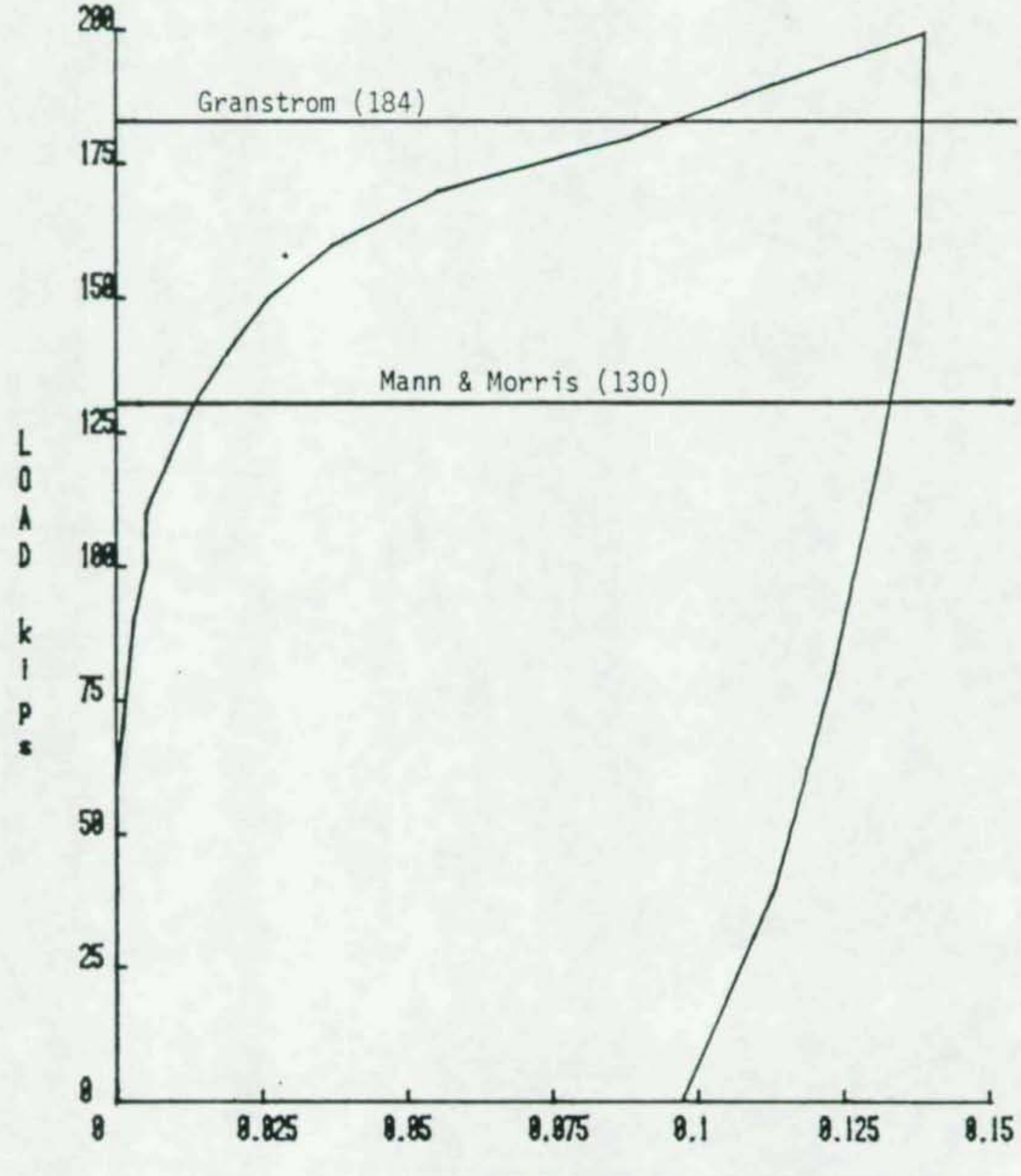


Figure 43. Load vs. Plate Separation - Test 4

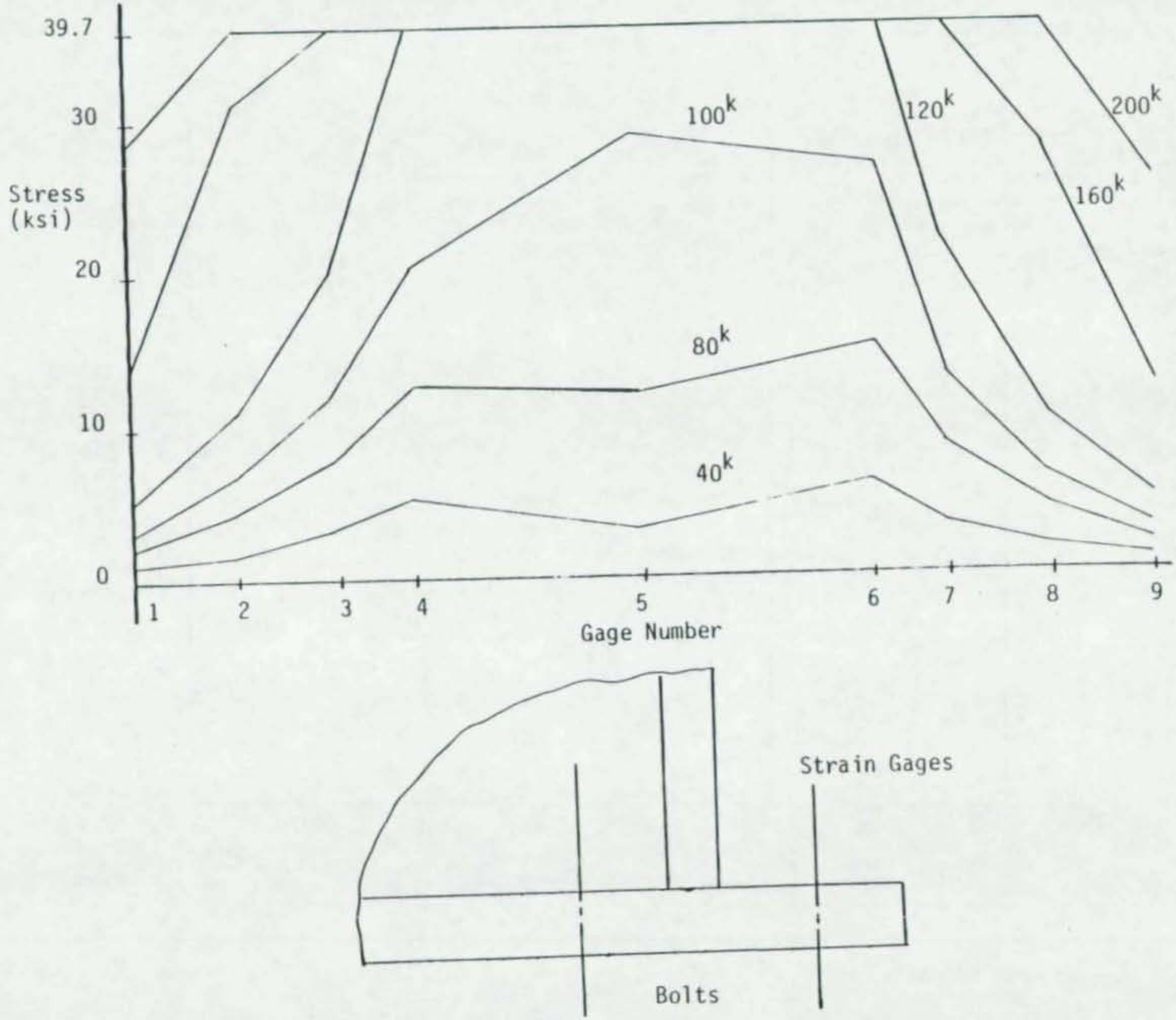


Figure 44. Flange Stress Distribution - Test 4

the results are similar with the Mann and Morris procedure predicting a slightly lower load. The largest difference in predicted failure loads was Test 4.

Comparison of effective flange length for the two procedures with measured yield length at the predicted failure loads, Table 7, shows the required effective lengths to be conservative for both methods. The yielded length of the column flange did not exceed the length of the strain gaged length in any test, Figures 38, 40, 42 and 44, although yielding would probably occur over the strain gage length at only slightly higher loads.

Upon removal of the maximum load (200 kips), permanent deformation was measured in all tests, Figures 37, 39, 41 and 43. Permanent set varied from 0.012 in. (Test 1) to 0.14 in. (Test 3).

3.4.5 Tension Region Column Web Strength

Although the testing program described above was developed specifically to check column flange strength at the tension region of end-plate connections, to some degree the adequacy of recommended procedures for estimating tension region column web strength can also be evaluated using the test data. Strain gages were not placed on the column webs, so web stress distributions are unknown. However, whitewash was applied and no evidence of web yielding was observed in any of the four tests.

The current AISC provision for the tension web at welded connections is the same as for the compression web. That is, the maximum permitted load is found from

$$P_{max} = F_{yc}\ t_{wc}\ (t_{fb} + 5k) \tag{81}$$

which is identical to Equation 2. Witteeven et al.[7] recommend that the web strength be determined from

$$F_t = F_{yc}\ t_{wc}\ b_m \tag{82}$$

which is the same as Equation 55. The term b_m is the effective length of the column flange based on a yield line analysis and is conservatively taken as the permitted length of yielding in the web. Granstrom[20] recommends a similar relationship (Equation 69)

$$F_t = 2w_{eff}\ t_{wc}\ F_{yc} \tag{83}$$

where $2w_{eff}$ is the effective length of the column flange and, again, is taken conservatively as the length of the yielded portion of the web. Mann and Morris[6] do not provide recommendations for determining column tension web strength and simply state that "usually the design of the column web in the tension zone is not critical." In the modified Krishnamurthy procedure developed in Section 3.2.2, an effective flange length equal to 3.5 times the bolt pitch, c, was used. The corresponding tension web strength would then be

$$F_t = 3.5\ t_{wc}\ F_{yc}\ C \tag{84}$$

Calculated values from each of the four equations for each of the column flange tests are shown in Table 8. Measured

Table 8
Tension Region Column Web Strength Provisions

Column Flange Test No.	Tee-Beam Section	Column Section	Predicted Tension Web Strength			
			Eqn. 81[1] (kips)	Eqn. 82[2] (kips)	Eqn. 83[3] (kips)	Eqn. 84[4] (kips)
1	WT9x48.5	W14x84	139	292	297	279
2	WT9x48.5	W14x103	143	314	319	277
3	WT9x25	W14x78	127	267	274	227
4	WT16.5x70.5	W14x78	133	286	293	295

[1] AISC welded connections

[2] Witteeven *et al*[7]

[3] Granstrom[20]

[4] Modified Krishnamurthy

Note: Maximum applied load
for all tests was 200 kips.

yield stresses were used in the calculations. The maximum applied load in each of the tests was 200 kips because of testing machine limitations.

It is evident that the approach used for welded beam-to-column connections greatly underestimates column tension strength at four-bolt end-plate connections. Adequacy of the other three methods cannot be assessed since the predicted failure loads from these methods exceeded the applied load in all cases. It is noted that the methods proposed by Witteveen et al.[7] (Equation 82) and Granstrom[20] (Equation 83) give essentially the same results. The modified Krishnamurthy method gives, in general, lower values than found from either Equation 82 or 83.

3.5 Design Recommendations

3.5.1 Column Flange Strength

Based upon the literature review it is evident that Morris[6,23] has undertaken the most comprehensive study of beam-to-column end-plate connections in the tension region. Further, the tests conducted as part of this study show that his recommendations give reasonable results. Hence, it is recommended that the Mann and Morris procedure outlined in Section 3.2.3 be used to estimate column flange strength.

Since the Krishnamurthy[25] end-plate design method, modified for column flanges, gave results (Tables C.1, C.2

and C.3) very close to those of Morris it may be that the reader will wish to use this method instead of the less familiar procedure proposed by Morris. A design example is included for this method also.

Mann and Morris Procedure

Beam W21x111

Column W14x176

A36 Steel

A325 Bolts

Assume end moment = full beam moment capacity, i.e.

$$M = 0.66F_yS_x = \frac{24(249)}{12} = 498'^{k}$$

From end-plate design procedure, AISC Manual of Steel Construction, 8th Edition[10]:

$t_e = 1\ 3/8"$

$b_e = 14\ 1/2"$

$g = 5\ 1/2"$

$c = 4\ 7/8"$

$d_b = 1\ 1/2"$ diameter A325

Section properties:

W21x111	W14x176
$d = 21.51"$	$t_{fc} = 1.310"$
$t_{fb} = 0.875"$	$b_{fc} = 15.65"$
	$t_{wc} = 0.830"$

$m = \frac{1}{2}(g - t_{wc}) = \frac{1}{2}(5\frac{1}{2} - 0.830) = 2.335"$

$n = \frac{1}{2}(b_e - g) = \frac{1}{2}(14\frac{1}{2} - 5\frac{1}{2}) = 4.50"$

$n' = \frac{1}{2}(b_{fc} - g) = \frac{1}{2}(15.65 - 5\frac{1}{2}) = 5.075"$

$d_h = d_b + 1/16" = 1\frac{1}{2} + 1/16 = 1\ 9/16"$

Bolt capacity $= T_u = A_b F_u = 1.7671(88) = 155.5^k$

To prevent bolt failure use 80% of T_u

Allowable flange force (Equations 61, 63 and 65)

$$F_{allow} = \min \left| \begin{array}{l} F_{ma} = 4(0.8)T_u \\ F_{mb} = t_{fc}^{\ 2} F_{yc} \{3.14 + \frac{0.5c}{(m+n)}\} + \frac{4(0.8)T_u n}{(m+n)} \\ F_{mc} = t_{fc}^{\ 2} F_{yc}\ \{3.14 + (2n + c - d_h)/m\} \end{array} \right.$$

$F_t = 5/3\ \frac{(498)(12)}{(21.51 - 0.875)} = 482.7^k$

$F_{ma} = 4(0.8)(155.5) = 497.6^k$

$F_{mb} = (1.31)^2(36)\{3.14 + \frac{0.5(4\ 7/8)}{2.335 + 4.5}\} + \frac{4(0.8)(155.5)(4.5)}{2.335 + 4.5}$

$F_{mb} = 543.6^k$

$F_{mc} = (1.31)^2(36)\{3.14 + (2(4.5) + 4\ 7/8 - 1.5625)/2.335\}$

$F_{mc} = 519.8^k$

$F_{allow} = 497.6^k > 482.7^k$ O.K.

$\therefore$ No stiffeners required

Modified Krishnamurthy Procedure

Beam W21x111

Column W14x176

A36 Steel

A325 Bolts

$$p_f = \tfrac{1}{2}(g - t_{wc}) = \tfrac{1}{2}(5\tfrac{1}{2} - .830) = 2.335"$$

$$p_e = p_f - \frac{d_b}{4} - r_c = 2.335 - \frac{1\frac{1}{2}}{4} - (2 - 1.310 - 1/16) = 1.333"$$

$$b_s = 3.5 \times c = 3.5(4\ 7/8) = 17.063"$$

$$C_a = 1.13 \text{ A36 steel, A325 bolts}$$

$$C_b = (b_f/b_s)^{\frac{1}{2}} = 1$$

$$\alpha_m = C_a C_b\ (A_f/A_w)^{1/3}\ (p_e/d_b)^{\frac{1}{4}}$$

$$= 1.13(1)(1)^{1/3}(\frac{1.333}{1\frac{1}{2}})^{\frac{1}{4}} = 1.097$$

$$M_e = \frac{\alpha_m p_e F_t}{4} = \frac{1.097(1.333)(3/5)(482.7)}{4} = 105.9"^k$$

$$t_{fc} = \left[\frac{6M_e}{b_s F_b}\right]^{\frac{1}{2}} = \left[\frac{6(105.9)}{(17.063)(.75)(36)}\right]^{\frac{1}{2}} = 1.174"$$

Actual $t_{fc} = 1.310" > 1.174"$

$\therefore$ No stiffeners required.

Mann and Morris[6] suggest that for most designs if the column flange thickness is greater than the bolt diameter the column flange will be adequate. To investigate this contention, a table was developed (Appendix D) comparing required column flange thickness by the Mann & Morris procedure to require bolt diameter for end-plate connections for various combinations of beams and columns. The end-plate and bolt diameter were sized using the design procedure in the 8th edition AISC Manual of Steel Construction[10] assuming the full capacity of the beam is realized. All 18 in. deep or larger economy sections from the Allowable Stress Design Selection Table in the Manual were used. Columns were chosen based upon the actual flange thickness as follows: Only columns with flange thickness greater than or equal to the end-plate thickness were chosen. The maximum column size was limited to those in which the flange thickness was less than or equal to the bolt diameter + 1/4 in. or the end-plate thickness + 1/4 in. The bolt gage was taken as the end-plate width less 1-1/4 in. edge distance on each side but not greater than 5-1/2 in.

As can be seen from Table D.1, the required column flange thickness is less than the required bolt diameter for most cases. The maximum unconservative error is 8.2%.

From these results, it is recommended that as a "rule of thumb" stiffeners be required in the tension region only if the column flange thickness is less than the required bolt diameter found from the end-plate design procedure in the 8th edition AISC Manual of Steel Construction as long as, the geometric limitations stated above are met.

3.5.2 Column Tension Web Strength

Although column tension web strength was not adequately assessed in this research to provide firm recommendations, it is suggested that either the method suggested by Witteeven *et al.*[7] or Granstrom[20] be used pending further testing. Because the Granstrom procedure is somewhat simpler to use, its use is suggested. For consistency with the notation used in the Mann and Morris procedure for determining column flange strength, Equation 69 is rewritten as

$$F_t \leq 2W_{eff}\, t_{wc}\, F_{yc} \tag{85}$$

where

$$W_{eff} = C + 4\,\{m - \tfrac{2}{3}(k - t_{fc})\} + 1.25n'.$$

CHAPTER IV

SUMMARY AND CONCLUSIONS

As discussed in the introduction, the purpose of this study was to review past works on beam-to-column moment connections and to develop design equations to determine the maximum allowable capacity of columns in both the compression and tension region of end-plate connections.

From the test results on the compression region it was found that a considerable liberalization could be made of the existing design recommendation which was formulated from welded beam-to-column connection tests. Use of this additional column web capacity, will eliminate the need for a substantial number of column web stiffeners, thus significantly reducing fabrication costs of columns used with moment end-plate connections. However, when this increased capacity is used, adequate lateral bracing of the column web/flange must be provided as demonstrated by the failure modes encountered in test the test program.

The Mann and Morris [6] procedure to evaluate column flange strength was shown to be adequate for end-plate connections designed using the Krishnamurthy procedure found in the 8th edition AISC Manual of Steel Construction [10]. It is recommended that this procedure be used to evaluate column flange strength at moment end-plate beam-to-column connections.

From the literature study and comparison with some test data, it is recommended, pending further study, that the procedure suggested by Granstrom[20] be used to estimate column web strength in the tension region of four-bolt end-plate connections.

Although a thorough study of the column compression and tension regions at moment end-plate connections was reported here, it must be emphasized that all tests were conducted using only a portion of the entire connection and that column axial loads were not a parameter in the test matrices.

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APPENDIX A

Nomenclature

NOMENCLATURE

a = distance from center of bolts to prying action force (in.) or weld dimension (in.)

A_b = bolt cross sectional area (sq. in.)

A_f = area of the beam flange (sq. in.)

A_{st} = stiffener area required (sq. in.)

b = distance from face of tee-stub stem to center of bolt (in.)

b_e = end-plate width (in.)

b_{eff} = equation 47

b_f = beam flange width (in.)

b_{fc} = column flange width (in.)

b_m = equation 51 or 52

b_s = effective column flange length in the modified Krishnamurthy procedure (in.)

C = residual contact force as shown in Figure 4b (kips)

c = bolt pitch (in.)

d = projection of the end-plate beyond the compression flange of the beam but not greater than t_e (in.); beam depth (in.) or beam depth (in.)

d_c = column web depth clear of fillets (in.)

d_b = bolt diameter (in.)

d_h = bolt hole diameter (in.)

e = distance from the center of bolts to the face of the column web minus 2/3 of the column fillet radius minus 1/4 of the bolt diameter (in.)

F_b = allowable bending stress (ksi)

F_{bx} = strong axis allowable bending stress (ksi)

F_{ms} = equation 66

F_{mz} = equation 67

F_t = tension force applied by the beam flange (kips)

$\overline{F}_t$ = force associated with the development of a yield line on each side of the column web (kips)

F_{yb} = yield stress of beam (ksi)

F_{yc} = yield stress of column (ksi)

F_{yp} = yield stress of the end-plate (ksi)

F_{yst} = yield stress of stiffener (ksi)

g = bolt gage (in.)

k = column "k" distance (in.)

M = applied beam end moment (in-kips) or allowable moment capacity (in-kips)

m = distance from face of column web to center of bolts (in.)

$\overline{M}$ = permissable moment on the effective column flange length, b_{eff} (in-kips)

M_b = actual beam moment (in-kips)

M_e = effective plate bending moment (in.-kips)

m_e = distance between two yield lines in the end-plate (in.)

M_p = plastic moment capacity per unit length of the column flange (in.-kips)

n = distance from center of bolts to edge of end-plate (in.)

n_b = number of bolts in a transverse line (in.)

n' = distance from center of bolts to edge of column flange (in.)

P_{bf} = the computed force delivered by the flange multiplied by 5/3 when the computed force is due to live and dead load only, or 4/3 when the computed force is due to live and dead load in conjunction with wind or earthquake forces (kips).

P_e = equation 77

P_f = distance from centerline of bolts to nearer surface of the beam tension flange (in.)

P_{max} = maximum force the column web is capable of resisting (kips)

Q = prying action force (kips)

Q' = sum of the beam flange thickness and twice the end-plate thickness (in.)

r_c = column fillet radius (in.)

S_x = strong axis section modulus (in.^{3})

T = load tributary to each bolt (kips)

t_e = end-plate thickness (in.)

t_{fb} = beam flange thickness (in.)

t_{fc} = column flange thickness (in.)

t_s = stiffener thickness (in.)

T_u = maximum bolt force (kips)

t_w = weld leg dimension (in.)

t_{wc} = column web thickness (in.)

r_c = fillet between the flange and the web of the column (in.)

v = distance from face of horizontal stiffeners to center of bolts (in.)

w = tee stem length per bolt row (in.)

W_{eff} = effective length of column flange (in.)

w_t = fillet weld throat size or reinforcement of groove weld, in.

α = ratio between the moment per unit width at the center-line of the bolt line and the flange moment at the web face.

Δ = maximum transverse displacement of the column flange (in.)

δ = ratio of the net area at the bolt line and the gross area at the web face of the flange.

σ = the average stress at the toe of the column fillet (ksi)

θ = equation 72

APPENDIX B

Critical Column Web Stresses For
Combination of W-Beam Sections
and W14 Column Sections

Table B.1
Critical Column Web Stress for No End-Plate
W14 Column Section, A36 Steel

Beam	90	99	109	120	132	145	159	176	193	211	233	257	283
W18 x 35	41.4	36.0	30.8										
W18 x 40	48.2	42.0	35.9	30.8									
W18 x 46	55.0	47.9	41.0	35.1	31.0								
W18 x 50	62.3	54.2	46.4	39.8	35.1	32.2							
W18 x 55	-	-	-	43.6	38.5	35.3	30.2						
W18 x 60	-	-	-	-	41.8	38.3	32.8	27.7					
W18 x 76	-	-	-	-	-	-	44.5	37.6	33.1	28.5			
W18 x 86	-	-	-	-	-	-	49.9	42.2	37.2	32.0			
W18 x 106	-	-	-	-	-	-	-	-	44.6	38.4	33.5		
W21 x 44	50.0	43.5	37.1	31.8									
W21 x 50	-	-	42.4	36.4	32.1								
W21 x 62	-	-	-	-	42.6	39.1	33.5	28.3					
W21 x 68	-	-	-	-	-	42.7	36.6	30.9					
W24 x 55	-	-	45.3	38.9	34.3	31.5							
W24 x 62	-	-	-	44.0	38.8	35.6	30.4						
W24 x 68	-	-	-	-	45.7	41.9	35.8	30.3					
W24 x 76	-	-	-	-	-	47.3	40.4	34.2	30.1				
W24 x 84	-	-	-	-	-	-	44.5	37.6	33.1	28.5			
W24 x 94	-	-	-	-	-	-	-	42.0	37.0	31.9	27.8		
W27 x 84	-	-	-	-	55.9	51.2	43.8	37.0	32.6	28.1			
W27 x 94	-	-	-	-	-	-	49.2	41.6	36.7	31.6	27.5		
W30 x 99	-	-	-	-	-	-	-	-	36.9	31.8	27.6		
W30 x 108	-	-	-	-	-	-	-	-	40.6	34.9	30.4	25.8	
W30 x 116	-	-	-	-	-	-	-	-	44.2	38.0	33.1	28.1	
W30 x 124	-	-	-	-	-	-	-	-	-	40.7	35.5	30.1	25.7
W33 x 118	-	-	-	-	-	-	-	-	44.2	38.0	33.1	28.1	
W33 x 130	-	-	-	-	-	-	-	-	-	42.5	37.0	31.4	26.8

Note: If stress (ksi) shown is greater than 36, a stiffener is required.
-Web crippling stress is much greater than 36 ksi.

Table B.2
Critical Column Web Stress With End-Plate (1:1)
W14 Column Section, A36 Steel

Beam	90	99	109	120	132	145	159	176	193	211	233	257	283
W18x35	27.4	24.1											
W18x40	30.3	26.8	23.5										
W18x46	34.7	30.7	27.0	23.4									
W18x50	39.2	34.7	30.5	26.5									
W18x55	40.9	36.2	31.9	27.7									
W18x60	42.4	37.6	33.2	28.9									
W18x76	-	-	44.9	39.1	35.0	32.5	28.5						
W18x86	-	-	-	42.2	37.8	35.2	30.9	26.7					
W18x106	-	-	-	-	-	40.8	35.8	31.0	28.0	24.6			
W21x44	33.0	29.2											
W21x50	35.9	31.7	27.9										
W21x62	45.3	40.1	35.3	30.7									
W21x68	-	41.9	37.0	32.2									
W24x55	38.3	33.8	29.7										
W24x62	43.4	38.3	33.7	29.3									
W24x68	-	-	39.7	34.5	30.8								
W24x76	-	-	40.9	35.6	31.9								
W24x84	-	-	-	39.2	35.1	32.6							
W24x94	-	-	-	42.1	37.8	35.1	30.8						
W27x84	-	-	-	38.5	34.5	32.0	28.0						
W27x94	-	-	-	41.6	37.3	34.7	30.4						
W30x99	-	-	-	43.6	39.0	36.3	31.8						
W30x108	-	-	-	-	41.3	38.4	33.7	29.1					
W30x116	-	-	-	-	-	41.8	36.7	31.8					
W33x118	-	-	-	-	44.9	41.8	36.7	31.7					
W33x130	-	-	-	-	-	45.0	39.6	34.3	30.9				

Note: If stress (ksi) shown is greater than 36, a stiffener is required.

-Web crippling stress is much greater than 36.

Table B.3
Critical Column Web Stress with End-Plate ($2\frac{1}{2}$:1)
W14 Column Section, A36 Steel

Beam	90	99	109	120	132	145	159	176	193	211	233	257	283
W18x35	34.4	30.1	26.0	22.4									
W18x40	39.0	34.2	29.7	25.6									
W18x46	-	39.1	33.9	29.3	26.0								
W18x50	-	-	38.4	33.1	29.4	27.1							
W18x55	-	-	41.0	35.4	31.5	29.1							
W18x60	-	-	-	37.7	33.5	30.9	26.8						
W18x76	-	-	-	-	45.4	41.9	36.3	31.0					
W18x86	-	-	-	-	-	46.1	40.0	34.2	30.5				
W18x106	-	-	-	-	-	-	-	40.4	36.0	31.4	27.6		
W21x44	41.5	36.3	31.4	27.1									
W21x50	46.2	40.5	35.1	30.3									
W21x62	-	-	45.4	39.3	34.9	32.2							
W21x68	-	-	-	42.0	37.4	34.5	29.9						
W24x55	-	43.2	37.5	32.3	28.7								
W24x62	-	-	42.4	36.6	32.5	30.0							
W24x68	-	-	-	43.1	38.8	35.3	30.5						
W24x76	-	-	-	-	41.3	38.2	33.1	28.2					
W24x84	-	-	-	-	-	42.0	36.4	31.1	27.7				
W24x94	-	-	-	-	-	-	39.9	34.1	30.4				
W27x84	-	-	-	-	-	41.3	35.8	30.6	27.2				
W27x94	-	-	-	-	-	-	39.5	33.8	30.1	26.1			
W30x99	-	-	-	-	-	-	40.5	34.6	30.8				
W30x108	-	-	-	-	-	-	43.7	37.3	33.3	28.9			
W30x116	-	-	-	-	-	-	-	40.7	36.3	31.5	27.7		
W30x124	-	-	-	-	-	-	-	42.8	38.2	33.2	29.2		
W33x118	-	-	-	-	-	-	-	40.7	36.2	31.5	27.6		
W33x130	-	-	-	-	-	-	-	44.6	39.8	34.6	30.4		

Note: If stress (ksi) shown is greater than 36, a stiffener is required.
-Web crippling stress is much greater than 36.

APPENDIX C

Required Column Flange Thickness
from
Various Methods

Table C.1
Required Column Flange Thickness
$F_{bx} = 0.66F_y$

Beam	Column	Beam End Moment (ft-k)	End-Plate Width (in)	End-Plate Thickness (in)	Bolt Diameter (in)	Weld: Fillet (in) or Groove	Weld: Groove Reinforcement (in)	Required Flange Thickness (in): Mann & Morris	Witeveen et al	Fisher & Struik	Granstrom	Krishnamurthy
W18x35	W14x48	115.2	7	3/4	7/8	3/8		.823	1.72	1.290	.518	.734
W18x40	W14x53	136.8	7	7/8	7/8	7/16		.894	1.272	1.403	.747	.793
W18x46	W14x53	157.6	7	7/8	1	1/2		.947	1.341	1.475	.631	.777
W18x50	W14x61	177.8	9	7/8	1	1/2		1.015	1.502	1.621	.870	1.037
W18x55	W14x68	196.6	9	1	1	Groove	3/16	1.064	1.575	1.701	1.028	1.085
W18x60	W14x74	216	9	1 1/8	1 1/8	Groove	3/16	1.101	1.624	1.752	.867	1.046
W18x65	W14x74	234	9	1 1/8	1 1/8	Groove	3/16	1.144	1.688	1.821	1.005	1.087
W18x71	W14x74	254	9	1 1/8	1 1/8	Groove	1/4	1.185	1.743	1.884	1.143	1.113
W18x76	W14x120	292	13	1 1/8	1 1/4	Groove	3/16	1.119	1.818	2.018	1.032	1.160
W18x86	W14x132	332	13	1 1/4	1 3/8	Groove	1/4	1.182	1.917	2.125	.984	1.170
W18x97	W14x145	376	13	1 3/8	1 3/8	Groove	1/4	1.248	1.995	2.244	1.191	1.214
W18x106	W14x159	408	13	1 3/8	1 1/2	Groove	1/4	1.283	2.034	2.300	1.065	1.155
W18x119	W14x176	462	13	1 1/2	1 1/2	Groove	5/16	1.349	2.131	2.428	1.285	1.184
W21x44	W14x48	163.2	7 ½	3/4	7/8	3/8		.928	1.325	1.445	.829	.915
W21x50	W14x53	189	8	7/8	1	7/16		1.006	1.433	1.548	.754	.998
W21x57	W14x61	222	8	1	1	Groove	3/16	1.087	1.481	1.673	.975	1.071
W21x62	W14x68	254	9 ½	1	1 1/8	1/2		1.096	1.679	1.772	.896	1.076
W21x68	W14x82	280	10	1 1/8	1 1/8	Groove	3/16	1.119	1.784	1.848	1.053	1.093
W21x73	W14x82	302	10	1 1/8	1 1/4	Groove	3/16	1.156	1.834	1.898	.882	1.071
W21x83	W14x82	342	10	1 1/4	1 1/4	Groove	1/4	1.223	1.939	2.005	1.110	1.121
W21x93	W14x90	384	10	1 3/8	1 3/8	Groove	1/4	1.287	1.877	2.089	.968	1.175
W21x101	W14x132	454	14½	1 3/8	1 1/2	Groove	1/4	1.222	2.075	2.370	1.021	1.188

Note: Underlined values of required flange thickness are less than the actual column flange thickness.

Table C.1
Required Column Flange Thickness
$F_{bx} = 0.66F_y$
Continued

Beam	Column	Beam End Moment (ft-k)	End-Plate Width (in)	End-Plate Thickness (in)	Bolt Diameter (in)	Weld: Fillet (in) or Groove	Weld: Groove Reinforcement (in)	Required Flange Thickness (in): Mann & Morris	Witeveen et al	Fisher & Struik	Granstrom	Krishnamurthy
W21x111	W14x176	498	14½	1 3/8	1 1/2	Groove	1/4	1.262	2.075	2.370	1.159	1.174
W24x55	W14x61	228	8½	7/8	1	7/16		1.022	1.452	1.603	.821	1.039
W24x62	W14x68	262	8½	7/8	1	1/2		1.086	1.544	1.706	1.038	1.089
W24x68	W14x99	308	10½	7/8	1 1/8	1/2		1.091	1.657	1.831	.937	1.150
W24x76	W14x132	352	10½	1 1/8	1 1/4	Groove	3/16	1.146	1.730	1.924	.859	1.107
W24x84	W14x145	392	10½	1 1/8	1 1/4	Groove	1/4	1.205	1.793	2.027	1.028	1.156
W24x94	W14x159	444	10½	1 1/4	1 3/8	Groove	1/4	1.264	1.868	2.120	.964	1.114
W24x104	W14x145	516	15	1 1/4	1 1/2	Groove	3/16	1.210	2.031	2.281	1.012	1.194
W24x117	W14x176	582	15	1 3/8	1 1/2	Groove	1/4	1.265	2.106	2.406	1.219	1.192
W27x84	W14x132	426	11½	1 1/8	1 1/4	Groove	3/16	1.158	1.804	2.009	.998	1.167
W27x94	W14x145	486	11½	1 1/4	1 3/8	Groove	3/16	1.223	1.872	2.109	.939	1.152
W27x102	W14x159	534	12	1 1/4	1 3/8	Groove	1/4	1.251	1.935	2.197	1.097	1.159
W30x99	W14x132	538	12½	1 1/8	1 3/8	Groove	3/16	1.190	1.903	2.109	.955	1.161
W30x116	W14x159	658	12½	1 1/4	1 1/2	Groove	1/4	1.295	2.024	2.292	1.036	1.160
W30x124	W14x176	710	12½	1 3/8	1 1/2	Groove	1/4	1.331	2.074	2.366	1.168	1.163
W33x118	W14x145	718	13½	1 1/4	1 1/2	Groove	3/16	1.263	2.041	2.292	1.032	1.200
W33x130	W14x159	812	13½	1 3/8	1 1/2	Groove	1/4	1.331	2.139	2.422	1.260	1.226
W36x135	W14x159	878	14	1 1/4	1 1/2	Groove	1/4	1.316	2.142	2.425	1.265	1.228

Note: Underlined values of required flange thickness are less than the actual column flange thickness.

Table C.2
Required Column Flange Thickness
$F_{bx} = 3/4 \times 0.66F_y$

Beam	Column	Beam End Moment (ft-k)	End Plate Width (in)	End-plate Thickness (in)	Bolt Diameter (in)	Weld: Fillet (in) or Groove	Weld: Groove Reinforcement (in)	Required Flange Thickness (in): Mann & Morris	Witeveen et al	Fisher & Struik	Granstrom	Krishnamurthy
W18x35	W14x48	86.4	7	5/8	3/4	3/8		.718	1.027	1.132	.480	.687
W18x40	W14x53	102.6	7	5/8	3/4	7/16		.774	1.108	1.223	.676	.726
W18x46	W14x53	118.2	7	3/4	7/8	1/2		.825	1.174	1.293	.537	.722
X18x50	W14x61	133.4	9	3/4	7/8	1/2		.883	1.312	1.419	.745	.958
W18x55	W14x68	147.5	9	7/8	7/8	Groove	3/16	.925	1.376	1.489	.885	1.003
W18x60	W14x74	162	9	1	1	Groove	3/16	.958	1.419	1.533	.700	.961
W18x65	W14x74	175.5	9	1	1	Groove	3/16	.995	1.474	1.593	.824	.999
W18x71	W14x74	190.5	9	1	1	Groove	1/4	1.031	1.527	1.649	.948	1.022
W18x76	W14x120	219	13	1	1 1/8	Groove	3/16	.972	1.586	1.765	.817	1.063
W18x86	W14x132	249	13	1	1 1/8	Groove	1/4	1.031	1.683	1.877	1.018	1.131
W18x97	W14x145	282	13	1 1/8	1 1/4	Groove	1/4	1.084	1.739	1.963	.929	1.109
W18x106	W14x159	306	13	1 1/8	1 1/4	Groove	1/4	1.119	1.786	2.030	1.051	1.109
W18x119	W14x176	346.5	13	1 1/4	1 3/8	Groove	5/16	1.172	1.858	2.122	.986	1.078
W21x44	W14x48	122.4	7½	3/4	3/4	3/8		.808	1.159	1.266	.751	.853
W21x50	W14x53	141.8	8	3/4	7/8	7/16		.875	1.253	1.355	.641	.922
W21x57	W14x61	166.5	8	7/8	7/8	Groove	3/16	.941	1.288	1.457	.834	.971
W21x62	W14x68	190.5	9½	3/4	1	1/2		.953	1.467	1.551	.726	.990
W21x68	W14x82	210	10	7/8	1	Groove	3/16	.974	1.558	1.617	.868	1.005
W21x73	W14x82	226.5	10	1	1	Groove	3/16	1.010	1.616	1.677	.987	1.042
W21x83	W14x82	256.5	10	1	1 1/8	Groove	1/4	1.063	1.694	1.754	.880	1.026
W21x93	W14x90	288	10	1 1/8	1 1/8	Groove	1/4	1.123	1.648	1.845	1.006	1.130
W21x101	W14x132	340.5	14½	1 1/8	1 1/4	Groove	1/4	1.065	1.803	2.002	1.016	1.140

Note: Underlined values of required flange thickness are less than the actual column flange thickness.

Table C.2
Required Column Flange Thickness
$F_{bx} = 3/4 \times 0.66F_y$
Continued

Beam	Column	Beam End Moment (ft-k)	End-Plate Width (in)	End-Plate Thickness (in)	Bolt Diameter (in)	Weld: Fillet (in or Groove	Weld: Groove Reinforcement (in)	Required Flange Thickness (in): Mann & Morris	Witeveen et al	Fisher & Struik	Granstrom	Krishnamurthy
W21x111	W14x176	373.5	14½	1 1/8	1 3/8	Groove	1/4	1.097	1.809	2.073	.869	1.071
W24x55	W14x61	171	8½	3/4	7/8	7/16		.889	1.268	1.404	.701	.961
W24x62	W14x68	196.5	8½	3/4	7/8	1/2		.945	1.349	1.494	.894	1.006
W24x68	W14x99	231	10½	3/4	1	1/2		.949	1.445	1.603	.767	1.058
W24x76	W14x132	264	10½	7/8	1	Groove	3/16	1.000	1.520	1.701	.945	1.076
W24x84	W14x145	294	10½	1	1 1/8	Groove	1/4	1.047	1.564	1.773	.816	1.059
W24x94	W14x159	333	10½	1	1 1/8	Groove	1/4	1.104	1.640	1.873	.996	1.081
W24x104	W14x145	387	15	1	1 1/4	Groove	3/16	1.054	1.782	2.015	1.007	1.149
W24x117	W14x176	436.5	15	1 1/8	1 3/8	Groove	1/4	1.099	1.836	2.104	.924	1.087
W27x84	W14x132	319.5	11½	7/8	1 1/8	Groove	3/16	1.007	1.573	1.757	.787	1.070
W27x94	W14x145	364.5	11½	1	1 1/8	Groove	3/16	1.067	1.644	1.864	.977	1.113
W27x102	W14x159	400.5	12	1 1/8	1 1/4	Groove	1/4	1.088	1.687	1.921	.845	1.059
W30x99	W14x132	403.5	12½	1	1 1/8	Groove	3/16	1.038	1.671	1.864	.993	1.122
W30x116	W14x159	493.5	12½	1 1/8	1 1/4	Groove	1/4	1.129	1.777	2.024	1.026	1.116
W30x124	W14x176	532.5	12½	1 1/8	1 3/8	Groove	1/4	1.157	1.809	2.069	.878	1.060
W33x118	W14x145	538.5	13½	1	1 1/4	Groove	3/16	1.101	1.791	2.024	1.024	1.155
W33x130	W14x159	609	13½	1 1/8	1 3/8	Groove	1/4	1.157	1.865	2.117	.959	1.118
W36x135	W14x159	658.5	14	1 1/8	1 3/8	Groove	1/4	1.143	1.867	2.120	.964	1.119

Note: Underlined values of required flange thickness are less than the actual column flange thickness.

Table C.3
Required Column Flange Thickness
$F_{bx} = 1/2 \times 0.66F_y$

						Weld		Required Flange Thickness (in)				
Beam	Column	Beam End Moment (ft-k)	End-Plate Width (in)	End-Plate Thickness (in)	Bolt Diameter (in)	Fillet (in) or Groove	Groove Reinforcement (in)	Mann & Morris	Witeveen et al	Fisher & Struik	Granstrom	Krishnamurthy
W18x35	W14x48	57.6	7	1/2	3/4	3/8		.586	.838	.925	.269	.561
W18x40	W14x53	68.4	7	5/8	3/4	7/16		.632	.905	.998	.289	.593
W18x46	W14x53	78.8	7	5/8	3/4	1/2		.677	.969	1.069	.355	.635
W18x50	W14x61	88.9	9	5/8	3/4	1/2		.724	1.081	1.171	.527	.839
W18x55	W14x68	98.3	9	3/4	3/4	Groove	3/16	.759	1.133	1.229	.648	.878
W18x60	W14x74	108	9	3/4	3/4	Groove	3/16	.789	1.179	1.279	.757	.896
W18x65	W14x74	117	9	3/4	7/8	Groove	3/16	.816	1.214	1.315	.536	.869
W18x71	W14x74	127	9	3/4	7/8	Groove	1/4	.845	1.257	1.360	.645	.888
W18x76	W14x120	146	13	3/4	7/8	Groove	3/16	.799	1.313	1.472	.784	.981
W18x86	W14x132	166	13	7/8	1	Groove	1/4	.845	1.384	1.549	.668	.980
W18x97	W14x145	188	13	7/8	1	Groove	1/4	.891	1.440	1.635	.826	1.015
W18x106	W14x159	204	13	7/8	1	Groove	1/4	.920	1.478	1.691	.928	1.014
W18x119	W14x176	231	13	1	1 1/8	Groove	3/16	.964	1.538	1.767	.827	.980
W21x44	W14x48	81.6	7½	1/2	3/4	3/8		.660	.947	1.034	.312	.696
W21x50	W14x53	94.5	8	5/8	3/4	7/16		.718	1.033	1.118	.431	.807
W21x57	W14x61	111	8	3/4	3/4	Groove	3/16	.772	1.161	1.202	.604	.849
W21x62	W14x68	127	9½	5/8	3/4	1/2		.785	1.219	1.294	.779	.924
W21x68	W14x82	140	10	3/4	7/8	Groove	3/16	.798	1.283	1.334	.575	.874
W21x73	W14x82	151	10	3/4	7/8	Groove	3/16	.828	1.331	1.384	.679	.907
W21x83	W14x82	171	10	3/4	7/8	Groove	1/4	.865	1.407	1.462	.848	.946
W21x93	W14x90	192	10	1½	1	Groove	1/4	.921	1.364	1.521	.667	1.048
W21x101	W14x132	227	14½	7/8	1	Groove	1/4	.875	1.493	1.668	.899	1.043
W21x111	W14x176	249	14½	7/8	1 1/8	Groove	1/4	.901	1.498	1.726	.729	.976

Note: Underlined values of required flange thickness are less than the actual column flange thickness.

Table C.3
Required Column Flange Thickness
$F_{bx} = 1/2 \times 0.66F_y$
Continued

Beam	Column	Beam End Moment (ft-k)	End-Plate Width (in)	End-Plate Thickness (in)	Bolt Diameter (in)	Weld: Fillet (in) or Groove	Weld: Groove Reinforcement (in)	Required Flange Thickness (in): Mann & Morris	Witeveen et al	Fisher & Struik	Granstrom	Krishnamurthy
W24x55	W14x61	114	8½	1/2	3/4	7/16		.729	1.045	1.159	.489	.843
W24x62	W14x68	131	8½	5/8	3/4	1/2		.775	1.112	1.233	.655	.881
W24x68	W14x99	154	10½	5/8	7/8	1/2		.778	1.189	1.322	.498	.921
W24x76	W14x132	176	10½	3/4	7/8	Groove	3/16	.820	1.248	1.400	.657	.936
W24x84	W14x145	196	10½	3/4	7/8	Groove	1/4	.862	1.295	1.478	.781	.978
W24x94	W14x159	222	10½	7/8	1	Groove	1/4	.905	1.348	1.545	.656	.937
W24x104	W14x145	258	15	7/8	1	Groove	3/16	.866	1.476	1.679	.891	1.053
W24x117	W14x176	291	15	7/8	1 1/8	Groove	1/4	.903	1.520	1.752	.775	.991
W27x84	W14x132	213	11½	3/4	7/8	Groove	3/16	.828	1.303	1.465	.760	.990
W27x94	W14x145	243	11½	3/4	1	Groove	3/16	.874	1.352	1.537	.635	.965
W27x102	W14x159	267	12	7/8	1	Groove	1/4	.895	1.397	1.601	.757	.970
W30x99	W14x132	269	12½	3/4	1	Groove	3/16	.850	1.374	1.537	.646	.972
W30x116	W14x159	329	12½	7/8	1	Groove	1/4	.929	1.471	1.686	.907	1.022
W30x124	W14x176	355	12½	7/8	1 1/8	Groove	1/4	.952	1.497	1.723	.736	.965
W33x118	W14x145	359	13½	3/4	1	Groove	3/16	.905	1.483	1.689	.906	1.059
W33x130	W14x159	406	13½	7/8	1 1/8	Groove	1/4	.951	1.544	1.763	.804	1.018
W36x135	W14x159	439	14	7/8	1 1/8	Groove	1/4	.939	1.546	1.766	.808	1.019

Note: Underlined values of required flange thickness are less than the actual column flange thickness.

APPENDIX D

Required Column Flange Thickness using Mann and Morris Procedure

Table D.1
Required Column Flange Thickness versus Required Bolt Diameter

Beam	Column	Column Flange Thickness t_{fc} (in.)	End-Plate Thickness t_e (in.)	Bolt Diameter d_b (in.)	Required Column Flange Thickness (in.)
W18x35	W14x68	.720	3/4	7/8	.820
W18x35	W14x74	.785	3/4	7/8	.819
W18x35	W14x82	.855	3/4	7/8	.816
W18x35	W14x99	.780	3/4	7/8	.817
W18x35	W14x109	.860	3/4	7/8	.815
W18x35	W14x120	.940	3/4	7/8	.813
W18x40	W14x82	.855	7/8	7/8	.888
W18x40	W14x109	.860	7/8	7/8	.887
W18x40	W14x120	.940	7/8	7/8	.884
W18x40	W14x132	1.030	7/8	7/8	.881
W18x40	W14x145	1.090	7/8	7/8	.879
W18x50	W14x82	.855	7/8	1	.713
W18x50	W14x109	.860	7/8	1	.713
W18x50	W14x120	.940	7/8	1	.711
W18x50	W14x132	1.030	7/8	1	.709
W18x50	W14x145	1.090	7/8	1	.708
W18x55	W14x132	1.030	1	1	1.053
W18x55	W14x145	1.090	1	1	1.051
W18x55	W14x159	1.190	1	1	1.048
W21x44	W14x68	.720	7/8	7/8	.925
W21x44	W14x74	.785	7/8	7/8	.924
W21x44	W14x82	.855	7/8	7/8	.921

Table D.1
Required Column Flange Thickness versus Required Bolt Diameter (continued)

Beam	Column	Column Flange Thickness t_{fc} (in.)	End-Plate Thickness t_e (in.)	Bolt Diameter d_b (in.)	Required Column Flange Thickness (in.)
W21x44	W14x99	.780	3/4	7/8	.922
W21x44	W14x109	.860	3/4	7/8	.921
W21x44	W14x120	.940	3/4	7/8	.918
W21x44	W14x132	1.030	3/4	7/8	.916
W21x44	W14x145	1.090	3/4	7/8	.914
W21x50	W14x82	.855	7/8	1	1.000
W21x50	W14x109	.860	7/8	1	1.000
W21x50	W14x120	.940	7/8	1	.997
W21x50	W14x132	1.030	7/8	1	.995
W21x50	W14x145	1.090	7/8	1	.994
W21x62	W14x132	1.030	1	1 1/8	1.084
W21x62	W14x145	1.090	1	1 1/8	1.082
W21x62	W14x159	1.190	1	1 1/8	1.079
W21x68	W14x159	1.190	1 1/8	1 1/8	1.106
W21x68	W14x176	1.310	1 1/8	1 1/8	1.101
W24x55	W14x82	.855	7/8	1	1.016
W24x55	W14x109	.860	7/8	1	1.015
W24x55	W14x120	.940	7/8	1	1.013
W24x55	W14x132	1.030	7/8	1	1.010
W24x55	W14x145	1.090	7/8	1	1.009
W24x62	W14x82	.855	7/8	1	1.082
W24x62	W14x109	.860	7/8	1	1.081

Table D.1
Required Column Flange Thickness versus Required Bolt Diameter (continued)

Beam	Column	Column Flange Thickness t_{fc} (in.)	End-Plate Thickness t_e (in.)	Bolt Diameter d_b (in.)	Required Column Flange Thickness (in.)
W24x62	W14x120	.940	7/8	1	1.078
W24x62	W14x132	1.030	7/8	1	1.076
W24x62	W14x145	1.090	7/8	1	1.074
W24x68	W14x82	.855	7/8	1 1/8	1.090
W24x68	W14x109	.860	7/8	1 1/8	1.089
W24x68	W14x120	.940	7/8	1 1/8	1.086
W24x68	W14x132	1.030	7/8	1 1/8	1.083
W24x68	W14x145	1.090	7/8	1 1/8	1.081
W24x76	W14x159	1.190	1 1/8	1 1/4	1.140
W24x76	W14x176	1.190	1 1/8	1 1/4	1.140
W24x84	W14x159	1.190	1 1/8	1 1/4	1.20
W24x84	W14x176	1.310	1 1/8	1 1/4	1.195
W24x94	W14x176	1.310	1 1/4	1 3/8	1.258
W24x94	W14x193	1.440	1 1/4	1 3/8	1.254
W27x84	W14x159	1.190	1 1/4	1 1/4	1.151
W27x84	W14x176	1.310	1 1/8	1 1/4	1.146
W27x94	W14x176	1.310	1 1/4	1 3/8	1.212
W27x94	W14x193	1.440	1 1/4	1 3/8	1.208
W30x99	W14x159	1.190	1 1/8	1 3/8	1.191
W30x99	W14x176	1.310	1 1/8	1 3/8	1.177
W30x108	W14x176	1.310	1 1/4	1 3/8	1.243
W30x108	W14x193	1.440	1 1/4	1 3/8	1.238

Table D.1
Required Column Flange Thickness versus Required Bolt Diameter (continued)

Beam	Column	Column Flange Thickness t_{fc} (in.)	End-Plate Thickness t_e (in.)	Bolt Diameter d_b(in.)	Required Column Flange Thickness (in.)
W30x116	W14x176	1.310	1 1/4	1 1/2	1.286
W30x116	W14x193	1.440	1 1/4	1 1/2	1.283
W33x118	W14x176	1.310	1 1/4	1 1/2	1.251
W33x118	W14x193	1.440	1 1/4	1 1/2	1.246
W33x130	W14x193	1.440	1 3/8	1 1/2	1.319
W33x130	W14x211	1.560	1 3/8	1 1/2	1.311
W36x135	W14x176	1.310	1 1/4	1 1/2	1.308
W36x135	W14x193	1.440	1 1/4	1 1/2	1.303