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Teoman Peköz

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research report

TEOMAN PEKÖZ

DEVELOPMENT OF A
UNIFIED APPROACH
TO THE DESIGN
OF COLD-FORMED
STEEL MEMBERS

REPORT SG 86-4
MAY 1986

DRAFT
FOR REVIEW PURPOSES
ONLY



Committee of Sheet Steel Producers
American Iron and Steel Institute
1000 16th Street, NW
Washington, DC 20036

DEVELOPMENT OF A UNIFIED APPROACH TO
THE DESIGN OF COLD-FORMED STEEL MEMBERS

by

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May 27, 1986

Report SG 86-4



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PREFACE

This report, "Development of a Unified Approach to the Design of Cold-Formed Steel Members," is reference 50 of the draft Commentary on the May 1986 Edition of the Specification for the Design of Cold-Formed Steel Structural Members published by the American Iron and Steel Institute. It is being issued as a draft research report pending public review of the Specification, which is from June 1 to July 1, 1986.

The unified design approach developed in this report includes the treatments of compression elements, columns, beams and beam columns. The approach covers sections with compression elements that are locally stable as well as those in the post-local buckling range at overall failure. The overall failure modes include flexural, lateral, and torsional-flexural. The unified design approach developed was checked on the basis of analytical and parametric studies. The results of carefully conducted tests were also used in the verification of the proposed approach.

American Iron and Steel Institute

June 1986

TABLE OF CONTENTS

| | Page |
|---|------|
| Chapter 1 Introduction | 1 |
| Chapter 2 Behavior of Compression Elements | 3 |
| 2.1 Introduction | 3 |
| 2.2 Stiffened Elements | 4 |
| 2.2.1 Effective Widths for Deflection Calculations | 7 |
| 2.2.2 Effective Widths for Perforated Elements | 10 |
| 2.3 Webs | 11 |
| 2.4 Unstiffened Elements | 12 |
| 2.4.1 Uniform Stress | 12 |
| 2.4.2 Unstiffened Elements with Stress Gradient | 14 |
| 2.5 Compression Elements with Edge and Intermediate Stiffeners | 14 |
| Chapter 3 Locally Unstable Open Section Columns and Beam Columns | 19 |
| 3.1 Introduction | 19 |
| 3.2 Columns | 19 |
| 3.3 Beam-Columns | 20 |
| Chapter 4 Interaction of Local and Flexural Column Buckling. . . | 23 |
| 4.1 Introduction | 23 |
| 4.2 Comparison of the Q-factor Approach with the Iterative Approach | 25 |
| 4.3 Comparison of the Iterative Approach with the Unified Approach | 26 |
| Chapter 5 Interaction of Local and Torsional-Flexural Column Buckling. | 28 |
| 5.1 Introduction | 28 |
| 5.2 Parametric Studies | 28 |
| 5.3 Experimental Verification. | 29 |

| | | |
|------------|--|----|
| Chapter 6 | Interaction of Local and Lateral Beam Buckling | 30 |
| 6.1 | Introduction | 30 |
| 6.2 | Lateral Buckling of Locally Stable Beams | 30 |
| 6.3 | Lateral Buckling of Locally Unstable Beams . . . | 31 |
| 6.4 | Experimental Verification. | 31 |
| 6.5 | Development of a Design Approach for Flexural Members | 32 |
| Chapter 7 | Biaxially Loaded Locally Unstable Open Section Beam Columns | 34 |
| 7.1 | Introduction | 34 |
| 7.2 | Parametric Studies | 34 |
| 7.3 | Correlation with Test Results. | 36 |
| Chapter 8 | Design of Angles | 37 |
| 8.1 | Introduction | 37 |
| 8.2 | Test Specimens | 37 |
| 8.3 | Correlation of Observed and Calculated Results . | 38 |
| 8.4 | Conclusions. | 40 |
| Chapter 9 | Design of Columns with Circular Perforations | 41 |
| 9.1 | Introduction | 41 |
| 9.2 | Stub Column Test Results | 41 |
| 9.3 | Column Test Results. | 42 |
| | 9.3.1 Calculation Procedures | 42 |
| | 9.3.2 Correlation of Observed and Calculated Results | 44 |
| 9.4 | Buckling Load of Nonprismatic Sections | 44 |
| 9.5 | Limitations of the Effective Width Equations . . | 47 |
| 9.6 | Conclusions. | 47 |
| Chapter 10 | Stub Column Tests and Evaluation of Stub Column Test Results. | 49 |
| 10.1 | Introduction | 49 |

| | | |
|------------|---|----|
| 10.2 | Evaluation of Test Results | 49 |
| | 10.2.1. Test Where Axial Shortening is Accurately Measured. | 49 |
| | 10.2.2. Tests Where Axial Shortening is Not Measured | 51 |
| 10.3 | Effect of Stub Column Length | 53 |
| 10.4 | Use of the Results of Multiple Tests in Design. | 55 |
| 10.5 | Interpolations | 56 |
| | 10.5.1. Interpolations Between Thickness Tested | 56 |
| | 10.5.2. Extrapolation of Yield Strength. | 56 |
| | 10.5.3. Extrapolation of Thickness | 58 |
| 10.6 | Use of Axial Shortening Measurements in Design | 58 |
| 10.7 | Summary of Evaluation Procedure | 60 |
| 10.8 | Specimen Preparation Tolerances and Testing Rates | 61 |
| 10.9 | Further Studies Needed | 61 |
| Appendix A | Draft of Possible Stub Column Test and Evaluation Procedures | 63 |
| References | | 75 |
| Tables | | 79 |
| 2.3-1 | Evaluation of Test Results | 79 |
| 2.4.1-1 | Dimensions of Test Specimen Sections | 81 |
| 3.3-1 | Locally Stable Beam-Columns Evaluation of Test Results Hat Sections | 82 |
| 3.3-2 | Locally Stable Beam-Columns Evaluation of Test Results Lipped Channel Sections. | 83 |
| 3.3-3 | Locally Stable Beam-Columns Evaluation of Test Results Lipped Channel Sections. | 84 |
| 3.3-4 | Locally Stable Beam-Columns Evaluation of Test Results Lipped Channel Sections. | 85 |

| | | |
|---------|--|-----|
| 4.3-1 | Sections for Parametric Studies | 86 |
| 7.3-1 | Locally Unstable Beam-Columns Evaluation of Test Results Lipped Channel Sections | 87 |
| 7.3-2 | Locally Unstable Beam-Columns Evaluation of Test Results Lipped Channel Sections | 88 |
| 7.3-3 | Locally Unstable Beam-Columns Evaluation of Test Results Lipped Channel Sections. | 89 |
| 7.3-4 | Locally Unstable Beam-Columns Evaluation of Test Results Lipped Channel Sections. | 90 |
| 7.3-5 | Locally Unstable Beam-Columns Evaluation of Test Results Lipped Channel Sections. | 91 |
| 8.2-1 | Cross Sectional Dimensions. | 92 |
| 8.2-2 | Cross Sectional Dimensions. | 93 |
| 8.3-1 | Effect of Initial Sweep on Lipped Channel Columns. | 94 |
| 9.1-1 | Stub Column Test Results. | 95 |
| 9.1-2 | Column Test Specimens | 96 |
| 9.3.1-1 | Evaluation of Column Test Results (All test results) | 97 |
| 9.3.1-2 | Evaluation of Column Test Results (Specimens without perforations) | 98 |
| 9.3.1-3 | Evaluation of Column Test Results (Specimens with one perforation at midheight) | 99 |
| 9.3.1-4 | Evaluation of Column Test Results (Specimens with multiple perforations) | 100 |
| 9.4-1 | Nonprismatic Columns Buckling Load Ratios . . . | 101 |
| 9.4-2 | Nonprismatic Columns Buckling Load Ratios . . . | 102 |
| 9.4-3 | Parameters for Column Specimens | 103 |

| | | |
|------------------|---|-----|
| 9.4-4 | Parameters and Results of Calculations. (All columns) | 104 |
| 9.4-5 | Parameters and Results of Calculations. (Unperforated columns) | 105 |
| 9.4-6 | Parameters and Results of Calculations. (Perforated columns) | 106 |
| Figures. | | 107 |
| 2.2-1 | Plot of Eq. 2.2-1 and -2. | 107 |
| 2.2-2 | Stress f in Eq. 2.2-2 | 108 |
| 2.2.1-1 | Subultimate Behavior of Stiffened Compression Elements | 109 |
| 2.2.1-2 | Comparison of Some Subultimate Behavior Predictions of Stiffened Compression Elements . | 110 |
| 2.2.1-3 | Ratio, R , of the Effective Widths Calculated According to Eqs. 2.2.1-4 Through -6 to Those Calculated According to Eq. 2.2-2 | 111 |
| 2.2.1-4 | Possible Design Aids for Webs | 113 |
| 2.2.1-5 | Subultimate Behavior of Beams with Unstiffened Flanges | 114 |
| 2.3-1 | Web Effective Widths. | 115 |
| 2.3-2 | Limiting Values of λ and w/t Above Which the Web is not Fully Effective. | 116 |
| 2.4.1-1 | Values of Q -factor Observed in Tests and According to the 1980 AISI Specification. . . . | 117 |
| 2.4.1-2 | Column Specimen Sections. | 118 |
| 2.4.1-3 | Some Stub Column Test Results on Sections With Unstiffened Flanges. | 119 |
| 2.4.1-4 | Maximum Out-of-Plane Deflections of Unstiffened Compression Elements. | 120 |
| 2.4.1-5 | Ratio, R , of the Effective Width Given by Eq. 2.2-2 with $k = .43$ to that Implied in the 1980 AISI Specification. | 121 |
| 2.5-1 | Sections for the Numerical Examples 4 and 5 of the 1980 AISI Cold-Formed Steel Design Manual . | 122 |

| | | |
|--------|--|-----|
| 2.5-2 | Edge Stiffener Design Criteria. | 123 |
| 2.5-3 | Compression Element with Edge Stiffener | 124 |
| 3.3-1 | Torsional-Flexural Behavior for Axial Loads Along the Axis of Symmetry. | 125 |
| 3.3-2 | Cross Sectional Geometry. | 126 |
| 4.2-1 | Correlation of Observed and Calculated Failure Loads for Section S-1 with $w/t = 57.2$ | 127 |
| 4.2-2 | Correlation of Observed and Calculated Failure Loads for Section S-2 with $w/t = 83.0$ | 128 |
| 4.2-3 | Correlation of Observed and Calculated Failure Loads for Section S-3 with $w/t = 117.4$ | 129 |
| 4.2-4 | Correlation of Observed and Calculated Failure Loads for Section S-4 with $w/t = 151.8$ | 130 |
| 4.2-5 | Correlation of Observed and Calculated Failure Loads for Section U-1 with $w/t = 16.2$ | 131 |
| 4.2-6 | Correlation of Observed and Calculated Failure Loads for Section U-2 with $w/t = 20.5$ | 132 |
| 4.2-7 | Correlation of Observed and Calculated Failure Loads for Section U-3 with $w/t = 24.8$ | 133 |
| 4.2-8 | Correlation of Observed and Calculated Failure Loads for Section U-4 with $w/t = 29.1$ | 134 |
| 4.2-9 | Correlation of Calculated and Observed Failure Loads for Section LC-1 with $w/t = 57.5$ | 135 |
| 4.2-10 | Correlation of Calculated and Observed Failure Loads for Section LC-II with $w/t = 50.5$ | 136 |
| 4.2-11 | Correlation of Calculated and Observed Failure Loads for Section LC-III with $w/t = 42.0$ | 137 |
| 4.2-12 | Correlation of Calculated and Observed Failure Loads for Section LC-IV with $w/t = 35.0$ | 138 |
| 4.2-13 | Correlation of Calculated and Observed Failure Loads for Section LC-V with $w/t = 29.5$ | 139 |
| 4.3-1 | C-Section Column Curves - Flexural Buckling | 140 |
| 4.3-2 | C-Section Column Curve Comparisons - Flexural Buckling | 141 |

| | | |
|--------|--|-----|
| 4.3-3 | C-Section Column Curve Comparisons - Flexural Buckling | 142 |
| 4.3-4 | Tube Column Curves - Flexural Buckling. | 143 |
| 4.3-5 | Tube Column Curve Comparisons | 144 |
| 4.3-6 | Tube Column Curve Comparisons | 145 |
| 4.3-7 | Tube Column Curve Comparisons | 146 |
| 5.2-1a | C-Section Column Curves - Torsional-Flexural Buckling | 147 |
| 5.2-1b | Hat Section Column Curves - Torsional-Flexural Buckling | 148 |
| 6.2-1 | Lateral Buckling Moment About the Symmetry Axis - Section C1-1. | 149 |
| 6.2-2 | Lateral Buckling Moment About the Symmetry Axis - Section C2-1. | 150 |
| 6.2-3 | Ratio of the Lateral Buckling Moment According to the 1980 AISI Specification to that According to the Torsional-Flexural Buckling Theory - C-Sections. | 151 |
| 7.1-1 | Section Geometry and Generalized Effective Section | 152 |
| 7.3-1 | Correlation of Test Results with the Unified Approach for Beam Columns | 153 |
| 8.3-1a | Section 1 - Unified Approach with $e_x = 0$ | 154 |
| 8.3-1b | Section 1 - Unified Approach. | 154 |
| 8.3-1c | Section 1 - Unified Approach. | 155 |
| 8.3-1d | Section 1 - Unified Approach with $e_x = L/1000$ | 155 |
| 8.3-1e | Section 1 - Correlation with the 1980 AISI Approach and Other Approaches | 156 |
| 8.3-2a | Section 2 - Unified Approach with $e_x = 0$ | 157 |
| 8.3-2b | Section 2 - Unified Approach with $e_x = L/1000$ | 157 |
| 8.3-2c | Section 2 - Unified Approach. | 158 |
| 8.3-2d | Section 2 - Correlation with the 1980 AISI Approach and Other Approaches | 159 |

| | | |
|--------|---|-----|
| 8.3-3a | Section 3 - Unified Approach with $e_x = 0$ | 160 |
| 8.3-3b | Section 3 - Unified Approach with $e_x = L/1000$ | 160 |
| 8.3-3c | Section 3 - Unified Approach. | 161 |
| 8.3-3d | Section 3 - Correlation with the 1980 AISI Approach and Other Approaches | 162 |
| 8.3-4a | Section 4 - Unified Approach with $e_x = 0$ | 163 |
| 8.3-4b | Section 4 - Unified Approach with $e_x = L/1000$ | 163 |
| 8.3-4c | Section 4 - Unified Approach. | 164 |
| 8.3-5a | Section 5 - Unified Approach with $e_x = 0$ | 165 |
| 8.3-5b | Section 5 - Unified Approach with $e_x = L/1000$ | 165 |
| 8.3-5c | Section 5 - Unified Approach. | 166 |
| 9.1-1 | Dimensions of the Lipped Channels Tested. | 167 |
| 9.4-1 | Dimensions of Columns Used in the Parametric Study on Nonprismatic Columns | 168 |
| 9.4-2 | Ratios of Observed Test Results to Calculated Test Results | 169 |
| 9.4-3 | Ratios of Test Results to Calculated Results. | 170 |
| 9.4-4 | Ratios of Test Results to Calculated Results. | 171 |
| 9.4-5a | Ratios of Test Results to Calculated Results. | 172 |
| 9.4-5b | Ratios of Test Results to Calculated Results. | 172 |
| 9.4-6a | Ratios of Test Results to Calculated Results. | 173 |
| 9.4-6b | Ratios of Test Results to Calculated Results. | 173 |
| 9.4-7a | Ratios of Test Results to Calculated Results. | 174 |
| 9.4-7b | Ratios of Test Results to Calculated Results. | 174 |
| 10.1 | Simulated Stub Column Test Results 8"x8" Square Tube of .05" Thickness. | 175 |
| 10.2 | Simulated Stub Column Test Results 4"x4" Square Tube of .05" Thickness. | 176 |
| 10.3 | Simulated Stub Column Test Results 8"x8" Square Tube of .05" Thickness. | 177 |

| | | |
|-------|---|-----|
| 10-4 | Simulated Stub Column Test Results 16"x16" Square Tube of .05" Thickness. | 178 |
| 10.5 | Simulated Stub Column Test Results 4"x4" Square Tube of .05" Thickness. | 179 |
| 10.6 | Simulated Stub Column Test Results 8"x8" Square Tube of .05" Thickness. | 180 |
| 10.7 | Simulated Stub Column Test Results 16"x16" Square Tube of .05" Thickness. | 181 |
| 10.8 | Actual Stub Column Test Results Specimen. | 182 |
| 10.9 | Actual Stub Column Test REsults Specimen slc/2 60/90 | 183 |
| 10.10 | Simulated Stub Column Test Results. | 184 |
| 10.11 | Simulated Stub Column Test Results 4"x4" Square Tube. | 185 |
| 10.12 | Simulated Stub Column Test Results. | 186 |
| 10.13 | Extrapolation for Yield Stress from $F_y = 35$ ksi, $w/t = 20$ | 187 |
| 10.14 | Extrapolation for Yield Stress from $F_y = 35$ ksi, $w/t = 100$ | 187 |
| 10.15 | Extrapolation for Yield Stress from $F_y = 50$ ksi, $w/t = 20$ | 188 |
| 10.17 | Extrapolation for Yield Stress from $F_y = 50$ ksi, $w/t = 100$ | 189 |
| 10.18 | Extrapolation for Yield Stress from $F_y = 50$ ksi, $w/t = 500$ | 189 |
| 10.19 | Extrapolation for Yield Stress from $F_y = 65$ ksi, $w/t = 100$ | 190 |
| 10.20 | Extrapolation for Yield Stress from $F_y = 65$ ksi, $w/t = 500$ | 190 |
| 10.21 | Extrapolation for Thickness from $F_y = 55$ ksi, $W/t = 20$ | 191 |
| 10.22 | Extrapolation for Thickness from $F_y = 35$ ksi, $w/t = 100$ | 191 |
| 10.23 | Extrapolation for Thickness from $F_y = 55$ ksi, $w/t = 500$ | 192 |

| | | |
|-------|---|-----|
| 10.24 | Extrapolation for Thickness from $F_y = 55$ ksi, w/t = 20. | 192 |
| 10.25 | Extrapolation for thickness from $F_y = 55$ ksi, w/t + 100 | 193 |
| 10.26 | Hypothetical Perforation Patterns and Stub Column Length | 194 |
| 10.27 | Test No. 1 - 2 5/16 in Square Tube, .048 in Thick $F_y = 67.6$ ksi on Flats. | 195 |
| 10.28 | Test No. 1. | 196 |
| 10.29 | Test No. 1. | 196 |
| 10-30 | Test No. 2 - 2 5/16 in Square Tube, .063 in Thick, $F_y = 67.5$ ksi on Flats | 197 |
| 10-31 | Test No. 1 - 2 5/16 in Square Tube, 0.48 in Thick Observed and Calculated Test Results. . . | 198 |
| 10-33 | Test No. 2 - 2 5/16 in Square Tube, .063 in Thick Observed and Calculated Test Results. . . | 199 |
| A1 | Hypothetical Perforation Patterns and Stub Column Length. It is Assumed that the Perforations Shown in the Web of a C-Section and the Width of the Web is W. | 200 |

CORRELATION OF THE 1986 AISI
SPECIFICATION AND THIS REPORT

| The 1986 AISI Specification Section: | Discussed in this report in Section: |
|---|---|
| B2.1 | 2.2.1 |
| B2.2 | 2.2.2, 9.1 through .2 |
| B2.3 | 2.3 |
| B3.1 | 2.4.1 |
| B3.2 | 2.4.2 |
| B4 | 2.5 |
| B5 | 2.5 |
| C3.1 | 6.1 through .2, 6.5 |
| C3.1.1 | 6.5 |
| C3.1.2 | 6.2 through .5 |
| C4 | 3.2, 4.1 through .3 9.1 through .6 |
| C4(c) | 8.1 through .4 |
| C4.1 | 3.2, 4.1 through .3 |
| C4.2 | 3.2, 5.1 through .3 |
| C5 | 3.3, 7.1 through .3 8.1 through .4 |

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CHAPTER 1

INTRODUCTION

Studies to develop a unified approach to the design of cold-formed steel members will be discussed in this report. This approach has been the basis of many of the changes introduced in the 1986 Edition of the Specification for the Design of Cold-Formed Steel Structural Members published by American Iron and Steel Institute. This document will be referred to in this report as the 1986 AISI Specification.

The unified approach pertains to two general areas of design. These two areas are the design of compression elements and the design of members. For the design of compression elements, a generalized effective width approach is developed for stiffened and unstiffened compression elements, compression elements with intermediate or edge stiffeners and webs. For the design of members, the approach covers design of columns, beams and beam-columns. The failure modes for members include flexural, lateral and torsional-flexural buckling. The approach treats members that are made up of elements that may or may not be in the post-buckling range. This approach encompassing both compression elements and members will be referred to in this report as the unified approach.

The 1980 AISI Specification has a mixed approach for compression elements and webs. Stiffened elements are treated by an effective width approach while unstiffened elements and webs are treated by an allowable stress approach. The accuracy and the consistency of this approach has been studied by the researchers at Cornell University and elsewhere. The unified approach developed in this report treats all compression elements by a generalized effective width approach in a consistent manner.

The unified approach is based on the data and conclusions reached in several research projects carried out and reported by DeWolf, Peköz and Winter (1973, 1974), Kalyanaraman, Peköz and Winter (1972, 1977), Kalyanaraman and Peköz (1978), Desmond, Peköz and Winter (1978, 1981a), Desmond, Peköz and Winter (1978, 1981b), Mulligan and Peköz (1983, 1984), Loh and Peköz (1985), Cohen and Peköz (1987 - to be published), Weng and Peköz (1986 - to be published) and the extensive parametric studies carried out by the author as discussed in this report. In each phase of the research, rigorous analytical models were developed and compared with the results of several hundreds of experiments conducted within the project as well as those conducted elsewhere, for example, those of Thomasson (1978), LaBoube and Yu (1978) and Loughlan (1979). The study carried out by the author for the development of the unified approach reported here used the results of tests on 49 locally stable columns and beam-columns, 140 locally unstable beams, 213 locally unstable stub columns, columns, and beam-columns, more than 60 angle section columns and 46 perforated columns and stub columns.

In the implementation of the unified approach in the 1986 AISI Specification several new features were introduced in format as well as the technical content. The changes in format include the determination of allowable loads by applying a factor of safety to the calculated nominal load-carrying capacity rather than through the use of allowable stresses.

This feature will simplify conversion to a load and resistance factor design approach. Another change in format is the formulation of all non-dimensionalized equations that can be used with any consistent system of units.

CHAPTER 2

BEHAVIOR OF COMPRESSION ELEMENTS

2.1 INTRODUCTION

The approach developed in this chapter unifies the design procedures for stiffened and unstiffened compression elements, compression elements with intermediate or edge stiffeners as well as webs of flexural members. The 1980 AISI Specification treated these elements with mixed approaches involving effective widths and allowable average stresses. In some instances the 1980 AISI Specification was overly conservative thus putting cold-formed steel members at an unnecessary disadvantage. The unified approach for compression elements will make some cold-formed steel products more competitive and improve the procedures for calculations.

The effective width equation used in cold-formed steel design at the present time is a slightly modified form of the one derived by Winter (1947) for elements supported by webs along the two longitudinal edges. In the unified approach the behavior of compression elements and webs with various edge support conditions is represented by an effective width approach.

The effective width approach has been extended in Kalyanaraman, Peköz and Winter (1972, 1977); Kalyanaraman and Peköz (1978); Desmond, Peköz and Winter (1978, 1981a, 1981b) to unstiffened elements and to elements with an edge stiffener or an intermediate stiffener of any size. The application of the approach to webs was studied by Mulligan and Peköz (1983, 1984); Cohen and Peköz (1987) and other researchers. In the course of the studies conducted by the author for the preparation of

the 1986 AISI Specification some modifications were introduced to improve accuracy or to simplify the expressions.

2.2 STIFFENED ELEMENTS

Cold-formed steel sections are, in general, made up of thin plate elements. It is well known that the post-buckling behavior of longitudinally compressed elements supported by webs on longitudinal edges can be represented by an effective width approach. The effective width equation proposed by Winter (1947) and later modified by him slightly, serves as the basis of the AISI Specification for the Design of Cold-Formed Steel Members (1980). The effective width, b , can be written in a nondimensional form as

$$b = \rho w \quad \text{Eq. 2.2-1}$$

where w is the actual flat element width and ρ is a reduction factor determined as follows

$$\rho = (1 - 0.22/\lambda) / \lambda \quad \text{Eq. 2.2-2}$$

where

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \quad \text{Eq. 2.2-3}$$

F_{cr} is the plate buckling stress calculated as

$$F_{cr} = \frac{k\pi^2 E}{12(1 - \mu^2)(w/t)^2} \quad \text{Eq. 2.2-4}$$

λ can be written as

$$\lambda = \frac{1.052}{\sqrt{k}} \left(\frac{w}{t}\right) \sqrt{\frac{f}{E}} \quad \text{Eq. 2.2-5}$$

where t is the element thickness, f is the actual stress in the element, E is the modulus of elasticity, μ is Poisson's ratio, and k is the plate buckling coefficient.

For a fully effective element $w = b$ and $\rho = 1$. Substituting $\rho = 1$ into Eq. 2.2-2, the values of λ for which an element is fully effective can be found. These values are 0.327 and 0.673. The value of ρ cannot exceed unity. For values of λ between .327 and .673, the value of ρ determined from Eq. 2.2-2 is larger than unity. Therefore, $\lambda = .673$ is the limiting value of λ below which an element is fully effective.

Section B2.1 of the 1986 AISI Specification gives the effective width expression in the above form. This equation can be shown to be the same as the effective width equation for deflections in the 1980 AISI Specification. The form of the effective width equation in Eqs. 2.2-1 and -2 were used for two reasons. It is in nondimensional form and hence can be used with any consistent unit system, and identifying the plate buckling coefficient k explicitly shows the consistent nature of the treatment for all types of elements.

The stress, f , to be used for the element being considered depends on the way the load carrying capacity is determined. The load carrying capacity calculation may be based on either the initial yield or in some cases the reaching of the ultimate inelastic moment determined according to the procedures of Reck, Peköz and Winter (1975) provided certain conditions are met.

If the initial yield is the basis of design, then f is the yield stress if the yielding initiates in the element being considered, as seen in Fig. 2.2-2a. If the yielding does not initiate in the element being considered as shown in Fig. 2.2-2b, then the stress f should be calculated on the basis of the effective section when yielding initiates. This, in general, results in an iterative procedure.

If the load carrying capacity is determined on the basis of the ultimate strength, then the stress f is to be determined according to Reck, Peköz and Winter (1975). In this case the stress f is equal to the yield stress, F_y , as shown in Figs. 2.2-2c and -2d, except for unusual cases such as the one shown in Fig. 2.2-2e. Further considerations for the selection of f in computations for design of beams are discussed in Chapter 6; those for columns are explained in Chapters 4 and 5.

The unified approach involves the determination of the effective widths of elements with all edge conditions and stress gradients using Eqs. 2.2-1 and -2. Special conditions of these cases are accounted for by using the appropriate plate buckling coefficient, k . For unstiffened and stiffened elements, the plate buckling coefficients of 0.43 and 4 are used, respectively. These coefficients are for the cases when the longitudinal edge or edges are simply supported. It is, in general, conservative to take these values of k . As discussed in Mulligan and Peköz (1983, 1984) it is also possible that for exceptional cases such as for sections having large width-to-depth ratios -- for example, channel section columns (without stiffening lips) having ratios in excess of 6 -- the unstiffened element may destabilize the stiffened flange. For this reason the conservative values of k were judged to be appropriate. Further information on this subject can be found in Mulligan and Peköz (1983, 1984).

The 1980 AISI Specification contains two types of effective width equations, one for deflection calculations and one for load carrying capacity calculations. The 1986 AISI Specification contains only one effective width equation for load capacity and deflection calculations since the factor of safety is applied to the nominal load capacity rather

than the yield stress. Although the above equation can be used conservatively for deflection calculations, an alternate, more accurate expression is discussed in the next section.

The 1980 AISI Specification also gives two sets of effective width equations, one for flanges of square and rectangular tubes and another one for all other sections. Since the difference between the results that can be obtained between these two sets is quite small, the 1986 Specification stipulates only one effective width equation.

2.2.1 EFFECTIVE WIDTHS FOR DEFLECTION CALCULATIONS

The effective width equation in the 1986 AISI Specification has been shown to be very satisfactory for determining ultimate loads. As was observed by Winter (1947), the above equation underestimates effective widths at loads below ultimate, namely in calculating deflections. The same conclusion was reached by Kalyanaraman, Peköz and Winter (1972, 1977), Kalyanaraman and Peköz (1978), Desmond, Peköz and Winter (1978, 1981a, 1981b), Mulligan and Peköz (1983, 1984) as well as by Dawson and Walker (1972), Graves-Smith (1969) and Thomasson (1978).

The subultimate behavior, namely the deflections and stresses at service loads, can be studied by casting the above effective width equation into a more convenient format as follows. The average stress, F_{av} , on the full section area, A , can be written as

$$F_{av} = (b/w) F_{max} \quad \text{Eq. 2.2.1-1}$$

where F_{max} is the maximum edge stress, which is taken to be uniform over the effective area. Using Eqs. 2.2-2 and 2.2-3 and taking $f = F_{max}$, Eq. 2.2.1-1 can be rewritten as

$$F_{av} / F_{cr} = \lambda^{-0.22} \quad \text{Eq. 2.2.1-2}$$

when λ is greater than 0.673. When λ is equal to or less than 0.673, the element is fully effective, namely $b = w$, and Eq. 2.2.1-1 becomes

$$F_{av}/F_{cr} = \lambda^2 \quad \text{Eq. 2.2.1-3}$$

Test results can be compared with the equations in the above form. For a test the stress, F_{av} , for a compressed element can be calculated based on the full section and the applied moment. F_{cr} and λ can be calculated using Eq. 2.2-4 and -5, respectively. Then the test results can be plotted having as the vertical axis F_{av}/F_{cr} and as the horizontal axis λ as done in Fig. 2.2.1-1. In this figure, the curve designated Mulligan and Peköz (1983, 1984) is typical of many test observations. This curve was derived so that the slope at $\lambda = 0.673$ would be equal to that for the curve for a fully effective section. The curve also has zero slope as it intersects the curve for Eq. 2.2-2. The resulting equation for the curve given in Mulligan and Peköz (1983, 1984) is a rather complicated one. Based on regression analysis, the two straight lines designated Weng and Peköz (1986) were obtained to fit the curve of Mulligan and Peköz (1983, 1984). Based on these two straight lines, Weng and Peköz (1986) derive the following equations for λ for determining effective widths for deflection calculations:

$$\rho = 1 \text{ when } \lambda \leq 0.673 \quad \text{Eq. 2.2.1-4}$$

$$\rho = (1.358 - .461/\lambda)/\lambda \text{ when } 0.673 < \lambda < \lambda_c \quad \text{Eq. 2.2.1-5}$$

$$\rho = 0.41 + 0.59 \sqrt{F_y/f} - .22/\lambda)/\lambda \text{ when } \lambda \geq \lambda_c \quad \text{Eq. 2.2.1-6}$$

where

$$\lambda_c = 0.256 + 0.328(w/t) \sqrt{F_y/E} \quad \text{Eq. 2.2.1-7}$$

Additional supporting evidence for the above expressions can be found in Thomasson (1978). The expressions of Thomasson are compared

with the results of analytic studies of Graves-Smith (1969) in Fig. 2.2.1-2. Thomasson concludes that though the analytical curves of Graves-Smith agree well with his curves qualitatively, the analytical solution fails to compare well with the test results. In contrast, expressions of Thomasson agree very well with his test results. The expressions of Mulligan and Peköz (1983, 1984) and consequently the expressions of Weng and Peköz (1986) agree well with those of Thomasson.

Section B.2.1 of the 1986 AISI Specification gives Eqs. 2.2.1-4 through -6 for an alternative approach for calculating deflections. The use of these equations may result in significant benefits in the calculation of deflections. Deflection requirements for cold-formed steel members frequently do govern the calculated load capacities. The difference between the results obtained with above equations and the results obtained using Eq. 2.2-2 depend on the stress at which the effective width is calculated, the w/t ratio, yield stress and the section geometry. The value of ρ is plotted against λ in Fig. 2.2.1-4a. Whether Eq. 2.2.1-5 or -6 governs can be determined with the aid of Fig. 2.2.1-4b. These figures can be used as design aids.

The ratios of the effective widths calculated using the Eqs. 2.2.1-4 through -6 which are the equations given as an alternate procedure in the 1986 AISI Specification, to those calculated using Eq. 2.2-2 which is the equation for deflection calculation in the 1980 AISI Specification, are plotted in Fig. 2.2.1-3 versus λ/λ_y where λ_y is the value of λ when $f = F_y$. In these figures it is seen that the increase in the calculated effective widths can be up to about 30 percent. Moments of inertia used for calculating deflections for most sections depend almost linearly on the effective width of the flange. Thus it is possible to have load

capacities as determined by deflection requirements to increase by up to about 30 percent.

The 1986 AISI Specification contains more accurate deflection determination effective width equations discussed above, for stiffened elements only. For other types of elements Eq. 2.2-2 is stipulated. However, Kalyanaraman, Peköz and Winter (1972, 1977), Kalyanaraman and Peköz (1978), Desmond, Peköz and Winter (1978, 1981a, 1981b), Mulligan and Peköz (1983, 1984) found that Eq. 2.2-2 can be conservative also for unstiffened elements and for elements with an edge or intermediate stiffener. Examples of the behavior of beams with unstiffened flanges are illustrated in Fig. 2.2.1-5.

2.2.2 EFFECTIVE WIDTHS FOR PERFORATED ELEMENTS

The post buckling behavior of elements containing circular perforations can also be represented by an effective width approach. The approach used in the 1986 AISI Specification is based on Ortiz-Colberg and Peköz (1981). This approach involves the modification of Eqs. 2.2-1 and -2 to calculate the effective width b as follows:

$$b = w[1 - (0.22/\lambda) - (0.8d_h/w)]/\lambda \quad \text{Eq. 2.2.2-1}$$

where d_h is the perforation diameter and λ is as defined in Eq. 2.2-5. The upper limit for effective width thus calculated is taken conservatively as the net effective width ($w - d_h$). The applicability of the above experimentally obtained equation is limited by the range of the values of the parameters involved in the experiments. These limits are specified in the 1986 Specification Section B2.2a. The limitations and other considerations for perforated elements and some of the supporting experimental evidence are discussed in Chapter 9.

2.3 WEBS

The unified approach developed here includes the treatment of webs with an effective width approach as well. The development in this section is based on the test results reported in LaBoube and Yu (1978), Cohen and Peköz (1987), Kallsner (1977), Johnson (1976), He (1981) and van Neste (1983). All the test results were evaluated in Cohen and Peköz (1987) using the proposed procedure for webs connected to stiffened, unstiffened and partially stiffened compression flanges. Some statistical data on the correlation are given in Table 2.3-1. To unify the treatment of webs and compression flanges, it was decided the generalized effective width equations in Section B2.3 of the 1986 AISI Specification would be used for webs rather than the allowable stress approach of the 1980 AISI Specification.

An illustration of the effective width approach is given in Fig. 2.3-1. For a web with stresses f_1 and f_2 , as shown in the figure, the effective widths b_2 and b_1 are determined as follows:

$$b_2 = b_e / 2 \quad \text{Eq. 2.3-1}$$

$$b_1 = b_e / (1 - \psi)$$

where $\psi = f_2/f_1$ and b_e is equal to b determined according to the Eqs. 2.2-1 through -5 by taking the theoretical value of k determined as follows:

$$k = 4 + 2(1 - \psi)^3 + 2(1 + \psi) \quad \text{Eq. 2.3-3}$$

f_2 and f_1 have to be compression (+) and tension (-), respectively. The case of both f_1 and f_2 being compression is rather unusual except in columns and beam-columns. However, for columns the stresses are assumed to be uniform in the web and for beam-columns the axial load and the

moments are treated separately and, thus, there is no need for provisions for webs for the case when f_1 is compression. Nevertheless, the above equations may give conservative results for compressive f_1 .

The flat width w used in the equation λ is the flat width of the web. The limiting values of w/t and λ above which the web is not fully effective can be obtained from Fig. 2.3-2. This figure can be used as design aids as well.

As will be discussed in Chapter 7, if lateral buckling is involved, the stress f_2 is to be determined on the basis of full section. The stress f_1 is to be determined on the basis of effective section taking f_2 as fixed.

2.4 UNSTIFFENED ELEMENTS

2.4.1 UNIFORM STRESS

An example of the motivation for the extension of the effective width concept to unstiffened elements can be seen in Fig. 2.4.1-1 taken from DeWolf, Peköz and Winter (1973). In this figure, the results of some stub column tests on specimens designated S and U are plotted. The cross-sectional geometry is illustrated in Fig. 2.4.1-2 and the dimensions are given in Table 2.4.1-1. It is seen that the value of Q , namely the ratio of the effective area at failure to the full area as defined in the 1980 AISI Specification, is conservative and satisfactory for stiffened elements, whereas the effective width implied in the 1980 AISI Specification is grossly conservative for unstiffened elements. The shortcomings of the Q -factor for column design will be discussed further in Chapter 4.

The extension of the concept of effective widths to unstiffened elements has been tried previously by Winter (1947) and others. Winter

proposed the following effective width equation for such elements:

$$\rho = 1.19(1 - 0.30/\lambda)/\lambda \quad \text{Eq. 2.4.1-1}$$

where λ is as defined in Eq. 2.2-3. This equation was not used previously because of lack of extensive experimental verification and the concern for out-of-plane distortions. In the research reported in Kalyanaraman, Peköz and Winter (1972, 1977) and Kalyanaraman and Peköz (1978) the applicability of the effective width concept to unstiffened elements with uniform stress was studied in depth. The results of some of the tests used in this study are plotted in Fig. 2.4.1-3. In this figure, both Eq. 2.4.1-1 and Eq. 2.2-2 are plotted with a buckling coefficient of .43. It is seen that Eq. 2.2-2 gives a conservative lower bound to the test results. Therefore, Eq. 2.2-2 is also used with the proper plate buckling coefficient for unstiffened elements. The correlation of other test data with the unified approach predictions is summarized in Table 2.3-1.

The out-of-plane deformations in unstiffened elements was studied by Kalyanaraman, Peköz and Winter (1972, 1977) and Kalyanaraman and Peköz (1978). An idea about the out-of-plane deformations can be obtained from Fig. 2.4.1-4 taken from Kalyanaraman, Peköz and Winter (1972). The results of theoretical calculations and test results on sections having unstiffened elements with flat width to thickness ratios up to 60 are presented in this figure. Maximum amplitude of the out-of-plane deformations at failure divided by the thickness can approach 2 as the flat-width to thickness ratio approaches 60. However at service loads the deformations may be significantly less as the figure indicates.

The unified approach incorporated into Section B3.1 of the 1986 AISI Specification results in a significant increase in the calculated

load carrying capacity of unstiffened elements and sections having such elements. For purposes of comparison the 1980 AISI Specification approach of allowable stress can be cast into an effective width equation. In Fig. 2.4.1-5 the ratio of the effective width to the effective width calculated according to the unified approach implied by the 1980 AISI Specification is plotted against flat width to thickness ratios. It is seen that the benefit can be well in excess of 100 percent.

2.4.2 UNSTIFFENED ELEMENTS WITH STRESS GRADIENT

There is very little conclusive data available on this subject. This subject is being studied at Cornell University. It was decided to recommend that the unstiffened compression element with a stress gradient be treated as a uniformly compressed unstiffened element until further information becomes available. In this approach, the stress f is assumed to be the maximum stress in the element.

2.5 COMPRESSION ELEMENTS WITH EDGE AND INTERMEDIATE STIFFENERS

The motivation for the extension of the effective width concept to elements with edge and intermediate stiffeners is obvious when one considers the fact that for an element with a stiffener slightly smaller than the minimum required by the 1980 AISI Specification, the stiffener is completely ignored and the calculated capacity as a result can be only a small percentage of the actual capacity. Again, further examples of the improvements obtained by the unified approach that have been incorporated into Section B4 of the 1986 AISI Specification are given below.

Illustrative Examples 4 and 5 of the 1980 AISI Cold-Formed Steel Design Manual Part IV can be used to show the shortcomings of 1980 AISI Specification provisions. The cross-sectional dimensions of the two

sections considered are shown in Fig. 2.5-1. Except for the intermediate stiffener these sections are identical. The intermediate stiffener added in Example 5 results in an increase of 60 percent over the section without the intermediate stiffener. If the intermediate stiffener is slightly smaller than the one shown in the example, the load capacity according to the 1980 AISI Specification drops by 60 percent. However the unified approach of the 1986 AISI Specification shows only a slight loss in capacity. Thus the unified approach for this case will result in a significant improvement.

A more significant increase in the predicted capacity according to the 1986 AISI Specification can be obtained for the case of edge stiffened sections. For example, for a 0.06-inch thick lipped C-section 7 inches deep, 1.5 inches wide with 0.48-inch long lips made of 50 ksi steel the allowable major axis bending moment is 38.1 kip-in. If the lips are 0.47 inches long the allowable moment drops down to 14.3 kip-in. However the proposed approach would predict an insignificant drop in the calculated capacity, thus resulting in a dramatic 166 percent increase in the calculated design moment compared to that calculated according to the 1980 AISI Specification.

It may be noted when the edge stiffener has a moment of inertia slightly more than the minimum required by the 1980 AISI Specification, the unified approach may give capacities up to 10 percent lower. Test results have shown that for these cases the 1980 AISI Specification could be unconservative up to about 10 percent.

In both of the above examples the proposed version gives a better representation of the actual performance by far. In cases where the neutral axis is closer to the compression flange than the tension flange

the increase in the calculated capacity may be less.

Stiffeners present two problems to be considered. First is evaluating the adequacy and effectiveness of the stiffener in restraining the element to be stiffened. Second is how the stiffener itself should be considered in calculating the overall section properties such as the moment of inertia, area and the section modulus.

The effective width approach given in the 1986 AISI Specification for compression elements with an edge or intermediate stiffener is based on the work reported in Desmond, Peköz and Winter (1978, 1981a, 1981b).

The provisions of Section B4.2 of the 1986 AISI Specification for elements with edge stiffeners are illustrated in Fig. 2.5-2. In this figure qualitative plots of the longitudinal stresses in the elements are also illustrated. The cross-sectional notation is shown in Fig. 2.5-3. In Section B4.2 three cases are identified. The first case is for elements that do not need an edge stiffener; namely, for elements that are fully effective as an unstiffened element. For this case the longitudinal stresses are uniform except for the case when the edge stiffener is so long ($D/w > 0.25$) as to destabilize the flange to which it is attached. There is no provision for this exceptional case in the 1986 AISI Specification.

The second case is when the element can be fully effective as a stiffened element when it has an adequate stiffener ($I_s > I_a$) that is not too long ($D/w < 0.25$). The expressions for the moment of inertia for an adequate stiffener were derived in Desmond, Peköz and Winter (1978, 1981a, 1981b). Again the stiffener can be so long ($D/w > 0.25$) that it can destabilize the flange to which it is attached. For this case the provisions reflect the decrease in the efficiency of the element by

reducing the plate buckling coefficient.

The third case is when the element is not fully effective even with an adequate stiffener. Again, for this case the special considerations discussed above, such as the effect of a stiffener that is too long, are applicable.

The correlation of some of the available test data with those predicted by the unified approach is summarized in Table 2.3-1. Further correlations can be found in Desmond, Peköz and Winter (1978).

Similar considerations except for the local instability of the stiffener are valid for elements with intermediate stiffeners. Local instability of the stiffener is of a different nature in intermediate stiffeners. Due to lack of sufficient data, the design of element with locally unstable intermediate stiffeners is not included in the unified approach. Further information on the design of elements with intermediate stiffeners can be found in Desmond, Peköz and Winter (1978, 1981a and 1981b).

In the formulation of Desmond, Peköz and Winter (1978, 1981a, 1981b) for the edge stiffeners made of a simple lip, the effective portion of the edge stiffener and the corner was taken as the stiffener. Their data base for this decision was rather limited. The evidence collected since this work, including the hand calculations and computer parametric studies carried out by the author and J. M. Cohen, supported taking the flat portion of the lip as the stiffener and ignoring the round corner in evaluating the adequacy of the stiffener. The results obtained with this approach are summarized in Table 2.3-1. Taking just the effective part of the stiffener for this purpose would be too conservative because it implies using the effective width equation for

determining the flexural stiffness of the stiffener about its axis parallel to the plane of the flange. The slight weakening effect of the stress gradient in the stiffener due to local buckling would be offset in the computations by ignoring the corner radius. It may also be noted that if the corner radius is large, its beneficial effect is doubtful, and if the corner is sharp, the corner is negligible. The effects of large corner radii are being studied at Cornell University at the present time.

The parametric studies on beams and beam-columns, as discussed in Chapter 4 show that the effective stiffener properties need to be used in calculating the overall section properties of the member. For a plain lip stiffener which is less than adequate, a simplification to the computation of the overall section properties was obtained by further reducing the effective width of the stiffener. This reduction is needed to represent the reduction of the stress in the stiffener as seen in Fig. 2.5-1 for such cases. Again, the result of this simplification was checked against the test results and was found to be satisfactory.

The available data for the intermediate stiffeners is not yet studied from the point of view of ignoring the transition corner between the stiffener and the element to be stiffened. However, it seemed reasonable to base the stiffener properties on the actual stiffener rather than the effective stiffener area in judging the stiffener adequacy and effects.

The topic of multiple stiffeners was not covered in the research reported in Desmond, Peköz and Winter (1978, 1981a, 1981b). Therefore, the 1986 AISI Specification has the same provisions as the 1980 AISI Specification for the case of multiple stiffeners.

CHAPTER 3

LOCALLY STABLE OPEN SECTION COLUMNS AND BEAM-COLUMNS

3.1 INTRODUCTION

This chapter deals with thin-walled doubly and singly symmetric open sections subjected to biaxial loading. The formulation also applies to tubular sections. Though singly symmetric open section cold-formed steel members are frequently subjected to biaxial loading, general design provisions did not exist in the 1980 AISI Specification. The design problem is often complicated because the plate elements making up such sections may buckle locally below loads causing overall failure. The subject is relevant to several practical applications including thin-walled square or rectangular tubes, end wall columns in metal buildings, many typical industrial storage rack columns and, possibly, purlins in the end bays of metal buildings. Sections with locally buckled plate elements will be covered in Chapter 7.

3.2 COLUMNS

At present research is under way at Cornell University on the flexural behavior of cold-formed steel columns. It was decided by the AISI Specification Advisory Committee to wait for the completion of this work before any changes are made in the provisions for flexural buckling of columns. Therefore, the equations for flexural buckling in Section C4 of the 1986 AISI Specification are the same as those in Section 3.6.1.1 of the previous Specification. However, in format the provisions for flexural and torsional-flexural buckling are unified as follows. First, the elastic flexural and torsional-flexural buckling stresses are found. Then, the smaller of these stresses is designated F_e . If F_e is greater

than $F_y/2$ then the inelastic buckling stress, F_n , is found by the usual equation

$$F_n = F_y (1 - F_y/4F_e) \quad \text{Eq. 3.2-1}$$

A simplified equation is introduced in the 1986 AISI Specification for torsional-flexural buckling under concentric loading. This simplified equation is based on Peköz and Winter (1969). In that reference the following simple interaction equation was derived

$$\frac{1}{P_{tfo}} = \frac{1}{P_{ex}} + \frac{1}{P_t} \quad \text{Eq. 3.2-2}$$

where P_{tfo} is the concentric torsional-flexural buckling load, P_{ex} is the concentric buckling load about the x axis which, is the axis of symmetry, and P_t is the torsional buckling load. This equation was further studied by Loh and Peköz (1985). This equation is expressed in terms of stresses and proposed for adoption in Section C4.2 of the 1986 AISI Specification.

3.3 BEAM-COLUMNS

For sections with fully effective plate elements, the studies by the author Peköz (1979) show that interaction equations can be used. This approach was adopted in the RMI Specification for Design, Testing and Utilization of Industrial Storage Racks (1979). The validity of the approach was further confirmed in Loh and Peköz (1985) on the basis of extensive analytical and experimental studies. The interaction equation for failure between the inflection points is

$$\frac{P}{P_0} + \frac{M_x C}{M_{x0} (1 - P/P_x)} + \frac{M_y C}{M_{y0} (1 - P/P_y)} = 1 \quad \text{Eq. 3.3-1}$$

P , M_x and M_y are the axial force and the moments about the x and y axes, respectively, due to the applied loading. P_0 is the failure load in the absence of any moment. M_{x0} is the failure moment for bending

about the x axis in the absence of an axial load or bending about the y axis. Similarly, M_{y0} is for bending about the y axis. P_0 , M_{x0} and M_{y0} are determined considering both flexural and torsional-flexural buckling. C_{mx} and C_{my} are corrections to reflect the moment gradient in the member. P_x and P_y are the flexural-buckling loads about the x and y axes, respectively.

The above interaction equation expressed in terms of allowable loads and moments is the basis of Eq. C5-1 of the 1986 AISI Specification. Equations C5-2 and -3 were adopted from the previous Specification.

The use of the above equation for axial loading in the plane of symmetry of singly symmetric sections simplifies the provisions of the 1980 AISI Specification for this case. The approximation involved in this simplification is shown qualitatively in Fig. 3.3-1.

The ultimate loads observed in tests with various axial load eccentricities are compared with those predicted by Eq. 3.3-1 in Tables 3.3-1 through -4. The cross sectional notation is illustrated in Fig. 3.3-2. The comparison is made by calculating the ultimate load by the two approaches. P_{na} is obtained by disregarding any inelastic reserve capacity, while P_{nb} is calculated considering inelastic reserve capacity. The correlation is considered to be satisfactory. For eccentricities of load between the centroid and the shear center, the approach is seen to be quite conservative as one can see from Fig. 3.3-1. However, it is considered desirable to use the interaction equation above because it is simpler than the provisions of the 1980 AISI Specification.

The above interaction equation will be shown in Chapter 7 to be applicable to the case of locally unstable beam-columns if certain modifications are made in the definitions of various terms. Thus it was

possible to unify the approaches for locally stable and unstable beam-columns.

Further studies and design formulations for angle sections and perforated columns will be discussed in Chapters 8 and 9, respectively.

CHAPTER 4

INTERACTION OF LOCAL AND FLEXURAL COLUMN BUCKLING

4.1 INTRODUCTION

The effect of local buckling on overall buckling behavior has been studied in several research projects at Cornell University and elsewhere. It has been shown repeatedly that the Q -factor approach may not be in good agreement with the test results. On the basis of tests and analytical studies, DeWolf, Peköz and Winter (1973, 1974) and Falyanaraman, Peköz and Winter (1972, 1977) conclude that the overall buckling load can be calculated using the effective radius of gyration and the effective area, both calculated at the overall buckling stress. This results in an iterative procedure because the buckling stress depends on the effective section properties which in turn depend on the buckling stress. As will be discussed in Chapter 5, the iterative approach has been extended in Loh and Peköz (1985) to the treatment of torsional-flexural buckling.

The approach of Canadian Specification for Cold-Formed Steel Structural Members (1984) formulated by John Springfield for flexural buckling was tried by the author for a variety of sections and failure modes. This approach is very similar to the one proposed by Peköz (1979) for perforated columns, and beam-columns subject to torsional-flexural buckling. For these members, the allowable stress is found for an unperforated column and the allowable load is found by multiplying this stress by the net area. This approach was adopted by the Rack Manufacturers Institute (RMI) Specification for Design, Testing and Utilization of Industrial Storage Racks (1979).

The studies summarized below show that proposed approach approximates the iterative approach very closely for flexural, torsional-flexural and

lateral buckling and will be used for all these types of behavior. For this reason this approach will be referred to in the rest of this report as the unified approach. This is also the approach adopted in the 1986 AISI Specification.

The unified approach consists of the following steps:

- a) The elastic flexural buckling stress, F_e , is calculated for the full unreduced section:

$$F_e = \pi^2 E / (KL/r)^2 \quad \text{Eq. 4.1-1}$$

- b) Then the (nominal) failure stress, F_n , is determined:

$$F_n = F_e \quad \text{if } F_n \leq F_y/2 \quad \text{Eq. 4.1-2}$$

$$F_n = F_y (1 - F_y/4F_e) \quad \text{if } F_n > F_y/2 \quad \text{Eq. 4.1-3}$$

- c) and the ultimate column load, P_n , is calculated as

$$P_n = A_e F_n \quad \text{Eq. 4.1-4}$$

where A_e is the effective area computed at stress F_n .

The remarkable accuracy of the proposed approach can be explained as follows: The reduction in the value of the radius of gyration resulting from local buckling is rather small. For small slenderness ratios where the column buckling stresses are high compared to the yield stress, the buckling stress is quite insensitive to the changes in the radius of gyration. However, the effective area is influenced directly and significantly by local buckling. For small stresses, namely large slenderness ratios, the local buckling is not significant. Therefore, for both cases the behavior is well represented by ignoring the reduction in the effective radius of gyration, but taking into account the reduction in the effective area in finding the ultimate load of the column.

For locally buckled C and other singly symmetric sections, concentric

axial loading with respect to the centroid of the effective section is an exceptional situation. The centroid of the effective section depends on the magnitude of loading. The location of the centroid moves as the load is increased. However, the allowable concentric loading is important as a parameter in the interaction equation.

4.2 COMPARISON OF THE Q-FACTOR APPROACH WITH THE ITERATIVE APPROACH

Some results taken from DeWolf, Peköz and Winter (1973, 1974) and Kalyanaraman, Peköz and Winter (1972, 1977) are illustrated in Figs. 4.2-1 to -13. A comparison of the experimentally observed ultimate values and the ultimate values computed according to the Q-factor approach of the 1980 AISI Specification can be made on the basis of these figures. The specimens designated U and S were discussed in Chapter 2. The dimensions for these sections were given in Fig. 2.4.1-2 and Table 2.4.1-1. In these figures ultimate loads are predicted using the Q-factor determined according to the 1980 AISI Specification, (Q_{AISI}), as well as the experimentally determined Q-factor (Q_{test}).

It is seen in Fig. 4.2-3 and -4 that the Q-factor approach can be unconservative for S-sections even when the experimentally determined value of Q is used. On the other hand, as can be seen in Figs. 4.2-5 through -13, the predicted ultimate loads for U-sections and LC-sections that contain unstiffened flanges are grossly conservative when the Q-values are computed according to the 1980 AISI Specification. The tests on LC-sections which are basically the same shape as U-sections are reported by Kalyanaraman, Peköz and Winter (1972). The correlation is better when experimentally determined values of Q are used. However, as mentioned above, even the experimentally observed Q-values give unsatisfactory results for S-sections. In contrast the iterative approach gives satisfactory results for S-, U- and LC-sections.

The iterative approach though satisfactory for S-, U- and LC- sections is tedious to use. This was the motivation for adopting the unified approach that does not involve iteration and that is shown to agree well with the iterative approach.

4.3 COMPARISON OF THE ITERATIVE APPROACH WITH THE UNIFIED APPROACH

The dimensions of several sections used in evaluating the proposed approach are given in Table 4.3-1. Sections C1-1, C2-1 and C3-1 are the same as those used by Loh and Peköz (1985). The results of the computations are given in Figs. 4.3-1 and -2. The ratios of the ultimate load to the yield load (P_n/P_y) versus the slenderness ratio based on full section weak axis radius of gyration (KL/r_y) are plotted in Fig. 4.3-1. Curve 0 is the curve for the ultimate load if the section is assumed to be fully effective. Curve 1 is for the present AISI Specification approach with the Q -values calculated according to Chapter 2, namely effective widths are used for unstiffened elements as well. Curve 2 is for the iterative approach described above. The iterative approach agrees best with the test results and is to be taken as the most accurate approach against which the other approaches will be judged. Curve 3 is for the approach proposed here. The ratios, R , of the ratio (P_n/P_y) calculated by different procedures to that obtained by the iterative approach, R_2 , are plotted in Fig. 4.3-2.

It is seen that the agreement between the proposed approach and the iterative approach is excellent. It is seen further that for the sections considered, the present AISI approach with the Q -values calculated according to Chapter 2 may give results that are up to about 35 percent unconservative. If Q -values were calculated according to the 1980 AISI Specification, the errors would be even larger.

The results of computations on other C-sections that have rather high

width to thickness ratios and exaggerated geometries are given in Fig. 4.3-3 for the sake of demonstration. Here the notation is the same as in Fig. 4.3-2. The same trend is also evident in this figure. Here the unconservatism of the 1980 AISI Specification approach with the Q -values calculated according to Chapter 2 is up to 80 percent. Again, if Q -values were calculated according to the present AISI Specification, the errors would be even larger.

Comparisons similar to those of Figs. 4.3-1 and -2 are presented for tubular sections in Figs. 4.3-4 through -7. Here again the correlation between the iterative approach and the unified approach is excellent. Again, the 1980 AISI approach can be very unconservative. For example the 1980 AISI approach can give results that are more than 60 percent unconservative for Tube T1-1, which has quite typical dimensions.

CHAPTER 5

INTERACTION OF LOCAL AND TORSIONAL-FLEXURAL COLUMN BUCKLING

5.1 INTRODUCTION

Extensive studies on post-local buckling torsional-flexural behavior of open sections has been reported in Loh and Peköz (1985). An analytical model for this behavior was developed on the basis of the torsional-flexural theory for the effective section, and the theory was confirmed by correlating the results with the test results. The approach involves iterations and, hence, is referred to as the iterative approach in the discussion below. Again, in this case, the concentric load is not a typical load for a locally buckled section, but the concentric ultimate load is a parameter used in the proposed interaction equations.

The approach proposed for calculating the allowable load of a locally buckled column subject to torsional-flexural buckling is exactly the same for those subject to flexural buckling in Chapter 4. It consists simply of determining the torsional-flexural buckling stress for the fully effective section and multiplying this stress by the effective area at the buckling stress calculated to obtain the ultimate load. Thus, in Eqs. 4.1-1 through -4, only the determination of F_e changes. For torsional-flexural buckling, F_e is determined according to the torsional-flexural buckling theory for the full unreduced section.

5.2 PARAMETRIC STUDIES

Some of the studies conducted are presented in Fig. 5.2-1a and 5.2-1b. The sections used in these parametric studies are described in Table 4.3-1. The designation of the curves is the same as those in the figures of Chapter 4. The iterative curves were taken from Loh and Peköz

(1985). The results for Sections C1-1 and C2-1 are shown in Fig. 5.2-1a. The results for Section H3-LU-3 of Loh and Peköz (1985) are shown in Fig. 5.2-1b. This section is a hat-section with very pronounced torsional-flexural behavior. The dimensions are the same as those of Section C3-1 except that the lips are turned outwards to form a hat section.

The correlation between the proposed approach and the iterative approach is seen to be excellent. The results of the approach proposed here is much better than those of the simplified approach of Loh and Peköz (1985).

5.3 EXPERIMENTAL VERIFICATION

As mentioned above, a concentric loading, with respect to the centroid of the effective section, is an exceptional situation. When the elements buckle locally, the location of the neutral axis changes. Therefore, all the members that were tested as pin-ended columns were actually beam-columns. The experimental verification for these members is described in Chapter 7. Several of the tests reported in Chapter 7 are with axial loading at the centroid of the full section.

CHAPTER 6

INTERACTION OF LOCAL AND LATERAL BEAM BUCKLING

6.1 INTRODUCTION

The unified approach for calculating the effect of local buckling on lateral buckling is consistent with that for columns. First, the elastic lateral buckling stress, F_e , is calculated on the basis of the torsional-flexural buckling theory for the full unreduced section using the equations of Peköz and Winter (1969). Then the failure stress, F_n , is determined using Eqs. 4.1-2 and -3. The lateral buckling moment is determined by multiplying F_n by the effective section modulus calculated for an outer fiber stress of F_n .

Further details will be discussed and developed in Section 6.5.

6.2 LATERAL BUCKLING OF LOCALLY STABLE BEAMS

Expressions for lateral buckling moments for singly symmetric sections derived on the basis of the torsional-flexural theory can be found in several references. Lateral buckling possibility exists for bending about both principal axes of a singly symmetric section (Peköz and Winter 1969). Then the failure stress F_n is determined using Eqs. 4.1-2 and -3. The lateral buckling moment is determined by multiplying F_n by the effective section modulus calculated for an outer fiber stress of F_n .

The author proposed the use of the equations derived on the basis of the torsional-flexural buckling theory for the RMI Specification (1979). Furthermore, the author proposed to use the same inelastic buckling correction for lateral buckling as the one used for columns. These expressions are specified to be used to determine the parameters for the

interaction equations for biaxial bending in the present RMI Specification.

Comparisons of the AISI and the RMI approaches are given in Figs. 6.2-1 and 6.2-2 taken from Loh and Peköz (1985) for Sections C1-1 and C2-1 of Table 4.3-1. Basically, the primary reason for the difference between the two approaches is the way the inelastic correction is applied. Comparisons of the elastic lateral buckling moments according to the AISI Specification and those computed according to the torsional-flexural buckling theory can be seen in Fig. 6.2-3. The two approaches give very similar results when the section bends about the strong axis as in the case of C1-1. The correlation is still acceptable for C2-1 but not as good. The two moments of inertia are close to each other for this section. For Section C3-1 the 1980 AISI approach predicts results too low compared to those from the torsional-flexural buckling theory. For this section the bending is about the weak axis. The buckling load for pure bending even in the plane of the weak axis is needed as a parameter for the beam-column interaction equations.

6.3 LATERAL BUCKLING OF LOCALLY UNSTABLE BEAMS

The unified approach described in Section 6.1 was tried for several sections and the results were virtually identical with those of the analytical approach developed in Loh and Peköz (1985) on the basis of torsional-flexural buckling theory.

6.4 EXPERIMENTAL VERIFICATION

There are no direct test data on the lateral buckling of cold-formed steel beams. However some data exist on the behavior of sections loaded axially by eccentric loading in Peköz (1967) and Loh and Peköz (1985). The data, which are admittedly indirect show that the RMI Specification

approach that is now incorporated into the unified approach and hence the 1986 AISI Specification, is quite satisfactory.

6.5 DEVELOPMENT OF A DESIGN APPROACH FOR FLEXURAL MEMBERS

The load carrying capacity of a beam subjected to pure bending should be checked for the section strength and lateral buckling. The section strength can be defined as either reaching the yield stress on the effective section or reaching the failure yield strain as defined and discussed by Reck, Peköz and Winter (1975). These two definitions may, in certain cases, give the same result. For example, for a section with a compression flange that cannot sustain more than the yield strain and, at the same time, yields in compression first, the initial yield also means the reaching of the failure strain.

The 1986 AISI Specification uses a factor of safety of 1.67 against section failure and lateral buckling. The factor of safety is applied to the nominal moment capacity or the calculated failure moment M_n , which is taken as the smaller of the nominal section moment capacity or the nominal lateral buckling moment.

The nominal section moment is determined either by defining the failure moment as the moment that will cause initial yield in the effective section or the moment at which compressive yield strain in the effective section is reached. According to the 1980 AISI Specification, when the latter definition is used, the allowable moment is the smaller of 0.75 times the nominal moment determined for initial yield or 0.6 times the moment that causes the reaching of the compressive failure strain. Since in the 1986 AISI Specification, a factor of safety of 1.67 is applied to the nominal moment, the nominal moment should be taken as the smaller of $1.25 (= 0.75/0.60)$ times the nominal moment determined for

initial yield or the nominal moment determined for reaching of the compressive failure strain, M_u .

As discussed in Section 6.1 above, according to the unified approach, the nominal lateral buckling moment, M_c , is calculated assuming the section to be fully effective. M_c can be either the elastic lateral buckling moment, M_e , or the inelastic buckling moment depending on the ratio of M_e to the yield moment, M_y , for the full section. The lateral buckling stress, M_c/S_f , is determined on the basis of the full section modulus, S_f , for the extreme compression fiber. The nominal lateral buckling moment is determined by multiplying the lateral buckling stress, M_c/S_f , by the effective section modulus, S_c , for the fiber in question. The effective section modulus is determined with the effective widths formulated in Chapter 2. It is assumed that the stress in the fiber in question remains fixed and the stresses in other fibers in the section can be determined on the basis of the effective section.

CHAPTER 7

BIAXIALLY LOADED LOCALLY UNSTABLE OPEN SECTION BEAM-COLUMNS

7.1 INTRODUCTION

The use of interaction equations for cold-formed steel beam-columns was studied extensively and extended to locally unstable sections in Loh and Peköz (1985). Though the interaction equation of Loh and Peköz (1985) is in the same form as Eq. 3.1-1, the definition of several terms of the interaction equation have been changed for the unified approach.

The unified approach for beam-columns developed here and adopted by the 1986 AISI Specification is basically different from that of Loh and Peköz (1985). It involves the use of Eq. 3.1-1 for singly or doubly symmetric open sections and closed tubes with some of the terms redefined to account for locally buckled plate elements. P_0 is determined as described above for locally unstable torsional-flexural buckling. M_{x0} and M_{y0} are determined by the approach described above for lateral buckling. All eccentricities (for example e_x in Fig. 7.1-1) are taken with respect to the centroid of the effective section for the axial load alone. The parameters P_x and P_y are the elastic buckling loads for the full unreduced section.

The unified approach for beam-columns, which is the basis of Section C5 of the 1986 AISI Specification, was confirmed by theory and tests on simply supported locally unstable C, channel and hat section beam-columns. Correlation for angle sections is discussed in Chapter 8.

7.2 PARAMETRIC STUDIES

The unified approach for beam-columns had several points to be resolved and clarified. This was achieved by means of parametric

studies and comparisons with the test results and the results of rigorous solutions.

A point to be resolved was the determination of the applied moments. The moment of the axial loads was stated to be taken about the centroidal axes of the effective section. There are several possibilities for calculating the effective section. A possible approach would be to use the effective section for the axial load and the moments. However, fortunately taking the effective section as that under axial load alone in combination with the other assumptions gave very satisfactory results.

Another point to be resolved was the determination of the critical load terms in the amplification factors. These factors are used to amplify the beam moments to determine the beam-column moments. Beam moments are determined ignoring the axial loads. The beam-column moment at a point is the sum of the beam moment and the increase in the moment due to the axial load. This increase can be determined by multiplying the axial load by the deflection at the point in question. It is customary to determine this deflection on the basis of elastic behavior. The critical load in the magnification factor is, therefore, the elastic buckling load about the bending axis.

The computation of the buckling load contained in the amplification factor should include the effects of local buckling on the stiffness. This would involve an iterative procedure. Several different schemes were tried. These schemes in combination with various ways of calculating the eccentricities required parametric studies and comparisons with the test results. The simplest approach, namely using the buckling load for the full section, gave the most

satisfactory results.

7.3 CORRELATION WITH TEST RESULTS

The correlation of the test results with those computed according to the unified approach is presented in Tables 7.3-1 through -5. In these tables, P_n and P_{uexp} are the calculated and observed failure loads, respectively. Considering the complex nature of the problem, the correlation is seen to be satisfactory. An exceptional case is the correlation with test results of Loughlan (1979). In this case the unified approach appears to be too conservative. The details of his test could not be verified.

The correlation of the calculated results with those observed is summarized in Fig. 7.3-1. This figure presents the results of all the tests with loads with uniaxial or biaxial eccentricities. The figure on the left illustrates the presentation of the results. In this figure R_p , R_x and R_y represent the first, second and the third terms of Eq. 3.3-1. Equation 3.3-1 defines the plane ABC. For a given test, the observed values of P , M_x and M_y are substituted into the equation and a point with the resulting R_p , R_x and R_y values is plotted. The results that fall outside the volume OABC indicate that the proposed interaction equation is conservative for those cases. This three-dimensional situation is represented in the right figure in Fig. 7.3-1 in two dimensions by plotting the projections of the test points on the R_p - R_o plane. Thus, from geometry R_o is equal to $0.707 (R_x + R_y)$. The points that fall outside the area OAD in the figure on the right show that the Eq. 3.3-1 is conservative. The few points that fall within this area have been mostly explained and the approach is judged satisfactory.

CHAPTER 8

DESIGN OF ANGLES

8.1 INTRODUCTION

In this chapter the applicability of the unified approach to the design of angle section columns and beam-columns is studied. Due to time constraints, the study reported here has been brief. Several extensions and further verifications are possible. The verification here has been based on the test results reported in Madugula, Prabhu, and Temple (1984) and Wilhoit, Zandonini and Zavellani (1983). The results of 41 tests from Madugula, Prabhu and Temple (1984) were used in the study. The tests of Madugula, Prabhu and Temple (1984) include tests on 16 hot rolled angles.

8.2 TEST SPECIMENS

Both references report nominal dimensions and section properties calculated from actual dimensions. The cross-sectional dimensions and yield stresses are presented in Tables 8.2-1 and -2. Center line length of the angle leg is designated "b." The actual dimensions were not stated clearly in Madugula, Prabhu and Temple (1984). The stated actual dimensions in Wilhoit, Zandonini and Zavellani (1983) did not give the section properties stated. This could have been due to the effect of rounded corners in calculating the section properties. Therefore, an angle leg dimension was calculated for each section from the given section properties as indicated in the tables of Section 8.3.

All the sections were loaded concentrically. It should be noted that all the sections except Section 2 have moderate w/t ratios and hence are not very sensitive to local buckling.

8.3 CORRELATION OF OBSERVED AND CALCULATED RESULTS

A computer program was prepared using the unified design approach. This program was capable of calculating ultimate loads for an angle section loaded in the plane of symmetry. By introducing an eccentricity in the plane of symmetry, the effect of sweep (initial out-of-straightness) is studied. The sweep in the plane of symmetry towards the corner of the section has the most influence as opposed to sweep in other directions.

The observed and calculated ultimate loads of Section 1 are plotted in Figs. 8.3-1a through -1e. Some of the tests on Section 1 were carried out with some restraint to force the direction of deflection towards the corner of the column section. The calculated results when the column is assumed to be perfectly straight, namely $e_x = 0$, are shown in Fig. 8.3-1a. It is seen that the calculated results provide an upper bound and, hence, are unconservative. Torsional-flexural buckling governs in the initial rather flat portion up to a slenderness ratio of about 60. Beyond this flexural buckling about the minor axis governs.

Wilhoit, Zandonini, and Zavellani (1983) mentions the presence of sweep and applies an analytical model assuming a sweep equal to $L/1000$. The effect of various eccentricities of the axial load or sweep values, as indicated by e_x , are studied in Figs. 8.3-1b and -1c.

It is seen in Figs. 8.3-1b and -1d that the assumption of an initial sweep of $L/1000$ leads to estimates of a lower bound of the test results. The following ratio is plotted for various sweep values for Section 1 in Fig. 8.3-1c for Section 1.

$$\text{Ratio} = \frac{\text{Ultimate load calculated assuming } e_x = 0}{\text{Ultimate load calculated assuming an } e_x}$$

As will be discussed similar plots are provided for other sections as well. It is seen in Fig. 8.3-1c that a sweep of L/1000 causes a lowering of the ultimate load of up to 30 percent.

To compare the sensitivity of the angle sections to sweep with the sensitivity of other sections, a study reported in Table 8.3-1 was carried out for a lipped channel with sweep perpendicular to the plane of symmetry. The two sections compared buckled flexurally and torsional-flexurally when loaded concentrically. The results show that the angle sections are much more sensitive to sweep than the lipped sections studied in Table 8.3-1. Studies, as the one shown in this table, need to be carried out for other sections.

Since the angle sections are very sensitive to initial sweep it appears prudent to consider a sweep in the design of angle sections. This conclusion is further supported in the plots for other sections as well.

The comparison of the test results on Section 1 with those obtained from the 1980 AISI Specification are given in Fig. 8.3-1e. This figure is taken from Madugula, Prabhu and Temple (1984). It is seen that the 1980 AISI Specification is unconservative for some of the test specimens.

The results of studies on Section 2 are plotted in Figs. 8.3-2a through -2d. This particular section had a high w/t ratio and, hence, was more susceptible to local buckling effects. The assumption of a perfectly straight column does not seem to lead to unconservative results. However, the assumption of a sweep in design still seems prudent.

The results of tests on Section 3, which is a hot rolled angle, are

presented in Figs. 8.3-3a through -3d. Again, the assumption of an initial sweep in design appears needed.

The results of tests on Sections 4 and 5 are reported in Wilhoit, Zandonini and Zavellani (1983). The initial out-of-straightnesses for the specimens were measured but not reported. The test results are compared with those calculated in Figs. 8.3-4a through -4c and -5a through -5c for Sections 4 and 5, respectively. It is quite possible that these specimens were more straight than others. However the assumption of an initial sweep does not seem to lead to overly conservative results.

8.4 CONCLUSIONS

The results of the above brief and limited study appear to support the use of the unified approach outlined. Due to the particular sensitivity of angle sections to initial sweep, it appears prudent to consider an initial sweep in design. The magnitude of the initial sweep equal to $L/1000$ gave reasonable results for the specimens considered in this study.

Based on this study, Sections D4c and D5 of the 1986 AISI Specification require the use of an out-of-straightness of $L/1000$ for the design of angle sections.

A more extensive verification for sections with large flat width to thickness ratios is needed. The application of the unified approach to biaxially loaded angle sections, though appearing justified intuitively, also needs to be verified.

CHAPTER 9

DESIGN OF COLUMNS WITH CIRCULAR PERFORATIONS

9.1 INTRODUCTION

This chapter presents the results of a brief study on the design of perforated cold-formed steel columns using the unified design approach. Some limited data reported in Ortiz-Colberg and Peköz (1981) are used in this study. The provisions of the 1986 AISI Specifications are thus based on rather limited data, some engineering judgement and intuition.

The brief study reported in Ortiz-Colberg and Peköz (1981) included stub and long column tests on lipped channel sections with and without circular perforations. The dimensions of the stub and long columns used in the study are given in Tables 9.1-1 and -2, respectively. Obviously, the variation of several possible parameters would have resulted in much more than the 46 members used in the study. A few of the results reported in Ortiz-Colberg and Peköz (1981) were not used because they looked obviously defective.

9.2 STUB COLUMN TEST RESULTS

The ratio of the observed stub column ultimate load to that calculated using the effective width equations of Chapter 2 for perforated plate elements is designated R and given in Table 9.1-1. The mean and standard deviation of R values are 1.072 and 0.086, respectively. This excellent correlation should not be too surprising since the formulation was based partly on the same data set.

9.3 COLUMN TEST RESULTS

9.3.1 CALCULATION PROCEDURES

The results of the application of the unified approach is presented in Tables 9.3.1-1 through -4. The following are the different procedures tried in calculating failure loads:

Procedure_1:

The unified approach procedure is applied assuming the axial load to be applied at the centroid of the full cross section. In the evaluation of the test results this assumption is not correct because the load was centered such that the strains around the cross section did not deviate from the average by more than 5 percent at about 25 percent of the expected ultimate load. This means that the point of application of the load was between the centroid of the full cross section and the centroid of the effective section at the failure load. It is possible to estimate the location of the load in the tests from a calculation of the effective section centroid location at 25 percent of the ultimate load. This would be rather approximate, and the time constraint of the present study did not permit such calculations.

The ratio of the ultimate loads observed to those calculated, assuming the load to be at the centroid of the full section, is designated R_1 in all the tables. It is seen that since the load was not applied at the centroid of the full section the calculated results are significantly below the observed results.

Procedure_2:

The unified approach is applied assuming the load to be applied at the centroid of the effective section at ultimate load. The ratio

of the observed load to that calculated according to this procedure is designated R2 in the tables.

Procedure_3:

The unified approach is applied ignoring the perforations and assuming the load to be applied at the centroid of the full section. The ratio of the observed load to that calculated according to this procedure is designated R3 in the tables.

Procedure_4:

The unified approach is applied ignoring the perforation and assuming the load to be applied at the centroid of the effective section at ultimate load. The ratio of the observed load to that calculated according to this procedure is designated R4 in the tables.

Procedure_5:

Procedures 1 and 2 assume that when there is a perforation, the section properties are changed throughout the section even if the perforation is at one point. Even for multiple perforations the reduction in the section properties are limited to small segments of the column. Procedure 5 involves weighting the calculations for perforated and unperforated columns depending on the number and size of perforations. This is done according to the formula

$$P = P_o - (P_o - P_p) \frac{dn}{L} \quad \text{Eq. 9.3.1-1}$$

where P_o and P_p are the ultimate loads calculated for unperforated and perforated columns, respectively, assuming the load to be applied at the centroid of the effective section at ultimate; d is the diameter of each perforation; n is the number of perforations; L is the column length. The ratio of the observed load to that

calculated according to Eq. 9.3.1-1 is designated R5.

9.3.2 CORRELATION OF OBSERVED AND CALCULATED RESULTS

As discussed above, it appears more reasonable to assume that the load is applied close to the centroid of the effective section. Thus the values of R2, R4 and R5 show better correlation than R1 and R3. Procedure 5 is quite a bit more complicated than procedures 2 and 4. For these reasons, the rest of the study will deal with Procedures 2 and 4.

For most of the sections without multiple perforations there is very little or no difference between R2, R4 and R5. This is due to the fact that for the sections without multiple perforations only a small portion of the length of the column has reduced moment of inertia and area due to the perforation. Since there are only nine columns with multiple perforations, an analytical parametric study was carried out. This study is described in the next section.

9.4 BUCKLING LOAD OF NONPRISMATIC SECTIONS

The effect of perforations on a column may be idealized as the reduction of section properties at the regions of perforations. An estimate of the magnitude of this effect can be obtained treating the column as a nonprismatic member with reduced moment of inertia at the regions of perforations. For a column with multiple perforations along its length, this would result in an idealized model with multiple regions of reduced moment of inertia.

An upper bound estimate of the effect of perforations was obtained by concentrating all the regions of reduced moment of inertia along a region at midheight of the column. The length of this region at midheight was taken equal to the sum of the lengths of individual regions along the length of the column.

An analytical model for a column with varying moment of inertia as described above, can be found in Timoshenko and Gere (1961). The transcendental equation for the flexural buckling of a column as shown in Fig. 9.4-1a, can be derived as

$$\tan \left[\frac{\pi r}{2} (1 - RL) \right] \tan \left[\frac{\pi r}{2} (RI)(RL) \right] \frac{1}{RI} = 0 \quad \text{Eq. 9.4-1}$$

In this equation $RI = \frac{I_1}{I_2}$ Eq. 9.4-2

I_1 is the moment of inertia of the effective section without perforation, and I_2 is the moment of inertia of the section with perforation. The regions of I_1 and I_2 are illustrated in Fig. 9.4-1a

$$RL = \frac{a}{L} \quad \text{Eq. 9.4-3}$$

L is the length of the column, and a is the length of the region of reduced moment of inertia taken as (nd) . The number of perforations along the length of a column is designated n .

$$r = \frac{P}{P_1} \quad \text{Eq. 9.4-4}$$

The term r is the ratio of the buckling load for the column of Fig. 9.4-1a divided by that for the column of Fig. 9.4-1b. P_1 is the buckling load of a column with moment of inertia equal to I_1 as shown in Fig. 9.4-1. P_1 can be expressed as

$$P_1 = \frac{\pi^2 E I_1}{L^2} \quad \text{Eq. 9.4-5}$$

P is the buckling load of the nonprismatic column.

The buckling load P can be written as

$$P = r P_1 \quad \text{Eq. 9.4-6}$$

The term r indicates the reduction in the value of the elastic buckling load due to the presence of perforations. As described above

the concentration of the perforations at midheight should overestimate the effect of perforations.

A computer program was written to determine the value of P using the above transcendental equation. The results are presented in Table 9.4-1. It is seen that for a wide range of values of the ratios RI and RL the column buckling load is quite insensitive to the presence of perforations. It should also be noted that the discussion so far has been limited to elastic buckling. For inelastic buckling, namely overall buckling stresses more than one-half of the yield stress, the effect of perforations on the buckling stress is even more insignificant.

The accuracy of Eq. 9.3.1-1 is studied in Table 9.4-2. The buckling load ratio given in this figure is the buckling load determined according to the transcendental equation Eq. 9.4-1 divided by the buckling load determined using Eq. 9.3.1-1.

It is instructive to examine the ratios RI and RL of the test specimens, which are listed in Table 9.4-3. In this table RI is the ratio of the moment of inertia of the effective section through a perforation, and the moment of inertia of the full unreduced section. It is seen that most of the sections without multiple perforations have RL and RI ratios such that there should be little difference between perforated and unperforated columns. The values of $R2$, $R4$ and $R5$ as well as RI and RL are tabulated in Tables 9.4-4 through -6. The values of these parameters are plotted in Figures 9.4-2 through 9.4-7.

In Tables 9.4-4 through 9.4-6 and Fig. 9.4-6a, it is seen that when RL is less than 0.015, the value of $R4$ is not in general below one except when the column is unperforated. If $R4$ is not below one, the capacity of the column can be calculated ignoring the perforations. The values of $R4$

below one for unperforated columns is attributed to test data scatter. However it appears prudent and conservative to account for the perforations in accordance with the unified approach when RL is larger than 0.015. The value of RL is easy to calculate.

A similar demarcation can be formulated for the value of RI . However, the calculation of RI is almost as tedious as the calculation of the capacity using the unified approach. Therefore such a formulation would not result in a significant design simplification.

9.5 LIMITATIONS OF THE EFFECTIVE WIDTH EQUATIONS

The effective width equations for perforated plate elements given in Chapter 2 were obtained from tests on lipped-channel sections reported in Ortiz-Colberg and Peköz (1981) with the following ranges of dimensions:

d/w from 0 to 0.50 where w is the flat width and d is the hole diameter.

w/t up to 64.

spacing of holes greater than $0.50w$ and $3d$.

These should be the range of applicability of the effective width equations given in Chapter 2 for perforated plate elements.

9.6 CONCLUSIONS

Based on the limited test data and the studies above, it has been shown that the design formulation given in this chapter can be used until further studies are completed. The limitations stipulated in Section 9.5 above are included in Section B2.2 of the 1986 AISI Specification. Section C4 of the 1986 AISI Specification contains a provision for perforated columns that allows one to ignore perforations if the number of holes in the effective length region times the hole diameter divided by the

effective length does not exceed 0.015. This provision is based on the discussion of Section 9.4 above.

effective length does not exceed 0.015. This provision is based on the discussion of Section 9.4 above.

CHAPTER 10

STUB COLUMN TESTS AND EVALUATION OF STUB COLUMN TEST RESULTS

10.1 INTRODUCTION

The unified approach necessitates an expression for the effective area, A_e , as a function of the stress, f , on the effective area. The stress f is taken as F_n in calculating column strength. When A_e cannot be calculated, such as when the column has dimensions or geometry outside the range of applicability of the generalized effective width equations of the 1986 AISI Specification, a functional relation between f and A_e can be obtained by stub column tests as described in this chapter.

A draft for test and evaluation procedures is given in Appendix A.

10.2 EVALUATION OF TEST RESULTS

10.2.1 TESTS WHERE AXIAL SHORTENING IS ACCURATELY MEASURED

An expression for the relationship between the stress on the effective section versus the effective area for axially loaded members can be obtained directly by measuring the axial shortening versus the axial load in a stub column test. From these measurements, an expression for the relationship between the effective area at that stress, A_e , and the stress, f , on the effective area can be derived as described below. In finding the ultimate load for an actual column, f is to be taken as F_n which is the column buckling stress calculated on the basis of full unreduced cross-section.

Denoting the axial load on the stub column, P , the effective area at this load is

$$A_e = P/f$$

Eq. 10-1

The axial shortening, D , at load P can be expressed as

$$D = f L/E \quad \text{Eq. 10-2}$$

where L is the length of the stub column and E is the modulus of elasticity.

It is usually assumed that at the ultimate stub column load, P_u , the stress on the effective section is equal to the yield stress, F_y . The effective area and the axial shortening at P_u will be designated A_{eu} and D_u , respectively. Thus

$$P_u = A_{eu} F_y \quad \text{Eq. 10-3}$$

and

$$D_u = F_y L/E \quad \text{Eq. 10-4}$$

From Eqs. 10-2 and -4

$$D/D_u = f/F_y \quad \text{Eq. 10-5}$$

and

$$f = F_y D/D_u \quad \text{Eq. 10-6}$$

Substituting Eq. 10-6 into Eq. 10-1

$$A_e = P D_u / (F_y D) \quad \text{Eq. 10-7}$$

If axial shortening measurements are taken and F_y is known, then all the terms of Eq. 10-7 are known, and the effective area, A_e , at each stress level, f , and the axial load level, P , can be determined. A_e should not be taken larger than the full unreduced area of the section, A .

Even though the axial shortening is measured accurately, if the evaluation of multiple stub column tests and any interpolations are needed (as given in Eq. 10-13, -15, -16 and -17 below), the approach derived in the following section is definitely more convenient. For this reason it is given as the primary procedure in Appendix A. The

approach of the next section underestimates effective areas and hence is conservative. An alternate procedure for using the measured axial shortening is given in the Section 10.6.

10.2.2 TESTS WHERE AXIAL SHORTENING IS NOT MEASURED

In this section, expressions for A_e versus f will be derived for the case when the axial shortening is not measured and only the ultimate load is measured (as was required according to the 1980 AISI Specification). These equations can be used for the cases where axial measurements are taken and the results of multiple tests are to be evaluated.

It can be observed that for f equal to zero A_e should be equal to the full unreduced area A . At ultimate load the effective area is equal to A_{eu} . A_{eu} should not be taken larger than A . A_e may be expressed as

$$A_e = A - (A - A_{eu}) (f/F_y)^n \quad \text{Eq. 10-8}$$

The writer carried out rather extensive parametric studies to determine a proper expression for the exponent n in the above equation. The following is a brief description of these parametric studies.

For most sections the effective area, A_e , can be calculated analytically using the unified effective width approach described in the 1986 AISI Specification. Using these calculated results an expression for n in Eq. 10-8 was developed. In this study three square thin-walled tubes were used. These sections give a conservative assessment of the approach of Eq. 10-8 since all the plate elements are partially effective. An expression for n based on this type of section would be conservative for sections that have both partially effective and fully effective plate elements.

Figure 10-1 shows the plots of calculated P/P_u versus D/D_u curves for a 8"x8" square tube with .05-inch wall thickness. Curve A is obtained using the effective width equation of Section B2.1b Procedure (1) of the 1986 AISI Specification. Curve B is obtained using the effective width equations of Procedure (2). As expected the approach of Procedure (1) is conservative. Thus in the subsequent plots Procedure (1) is used.

In Figs. 10-2 through -4 the results using Eq. 10-8 are plotted for 4"x4", 8"x8" and 16"x16" columns each having 0.05-inch-thick walls. In each of these figures the curves for $n = 1/3, 1/5$, and the curve obtained using the present effective width equation for stiffened elements (Curve A - simulated stub column test result) is plotted. These plots were just three of many obtained trying different approaches.

If the calculated curve shows a larger axial shortening for a given axial load, it means that the calculated results are conservative compared to the simulated test results. It is seen that for sections having larger width to thickness values, smaller values of n are required to get better agreement between the curve A and the curve calculated according to

Eq. 10-8. Consequently, it was decided to have

$$n = A_{eu}/A \qquad \text{Eq. 10-9}$$

The results of using n according to Eq. 10-9 in Eq. 10-8 are plotted in Figs. 10-5 through -7. As seen in these figures the agreement with curve A is excellent.

Figures 10-8 and -9 show comparison of the results obtained by Eqs. 10-8 and 10-9 with two stub column test results reported in Mulligan and Peköz (1983). Figure 10-8 shows that the Eqs. 10-8 and -9 are slightly

unconservative but quite satisfactory for the Test Specimen CLC/1 60X30. However, Fig. 10-9 shows that Eqs. 10-8 and -9 are quite conservative for the test specimen CLC/2 60X90.

It may be noted that Eq. 10-8 can be written in terms of the Q factor presently used in the 1980 AISI Specification as follows:

$$A_e/A = 1 - (1 - Q)(f/F_y)^Q \quad \text{Eq. 10-10}$$

The value of A_e for any f can be calculated using the presently available product properties tables listing values of Q .

If the axial shortening measurements are taken and Eq. 10-8 or -10 is used in the evaluation of multiple stub column test results, then the procedure developed in Section 10.4 may be used.

10.3 EFFECT OF STUB COLUMN LENGTH

Another question that needs to be resolved is the evaluation of test results when the length of the column is 3 to 4 times the largest dimension of the column cross-section (minimum length requirement) and when this length is larger than 20 times the least radius of gyration of the section, r_{\min} (maximum length requirement). In the opinion of the writer it is more important to meet the minimum length requirement rather than the maximum length requirement.

When the length is larger than $20 r_{\min}$, then the test results can be evaluated as follows: Having the length larger than $20 r_{\min}$ is expected to lower the overall buckling load. In this case, the critical stress F_n for the full section of the column tested can be determined according to the proposed specification provisions using the full unreduced section and taking the effective length factor as 0.5. The effective area A_p in

the test at the ultimate load P_u can be calculated as

$$A_e = F_u / F_n \quad \text{Eq. 10-11}$$

If the overall column buckling effects were to be reduced by lateral restraints, the effective area at ultimate load would be A_{eu} . The value of A_{eu} can be determined from Eq. 10-8 on the basis of A_e calculated according to Eq. 10-11. Thus with F_u substituted for f

$$A_{eu} = A - (A - A_e) / (F_n / F_y)^n \quad \text{Eq. 10-12}$$

The value of n depends on A_{eu} and has to be determined by trial and error. As a first try n can be taken as 0.2 and A_{eu} can be determined. This value of A_{eu} can now be used in Eq. 10-9 to determine n . With this value of n a new value of A_{eu} can be determined. The process should be repeated until the assumed value of n equals the calculated value of A_{eu}/A . With the converged-upon value of A_{eu} , Eq. 10-8 can be used to obtain A_e at any stress level f . The above minimum length requirements are for unperforated columns. For perforated columns, since there can be a wide variety of configurations, general guidelines should be used carefully and with consideration of special conditions. The applicability of the unified approach and the use of stub columns for results in the unified approach for columns with significant perforations have not yet been verified. In the meantime, some guidelines have been obtained mostly based on engineering intuition. The following are the guidelines given in Appendix A.

"For perforated columns with the largest dimension of the cross section W , the following guidelines will be used in deciding the length of the specimen to be used:

If a complete perforation pattern (single or multiple group of perforations) has a length along the column less than W then the specimen length may be chosen with the guidelines for unperforated columns (as in Figs. 10-26b, c, g).

If a complete perforation pattern (single or multiple group of perforations) has a length along the column larger than W (as in Figs. 10-26d, e, f) then the specimen length should be not less than 3 times the perforation pattern length if the ends are not welded and 4 times the perforation pattern length if the ends are welded. In this case it is desirable to have the more significant perforation at midheight of the stub column (compare Figs. 10-26d and e).

If the perforations are such that a cut cannot be made without going through a perforation as in Fig. 10-26i, then a special section may be fabricated as in Fig. 10-26j. In some instances it is conceivable to have tests carried out even when the cut goes through a perforation to evaluate performance in the actual member if the member has a similar configuration in actual application."

10.4 USE OF THE RESULTS OF MULTIPLE TESTS IN DESIGN

The results of several tests are to be used to determine the expression for calculating A_e for a stress level f . One needs to consider how this can be achieved for design purposes. Several approaches come to mind. These may include the following:

1. Use of the lowest or a statistically weighted value of A_{eu} obtained in a series of tests in Eq. 10-8. Obviously this would necessitate the use of Eq. 10-8, which is conservative compared to the use of plots of stub column axial shortening versus axial load.
2. If one wants to use such plots then one may use a lower envelope to the different curves obtained in different tests.
3. Another approach again for the case of using plots would be a weighted curve obtained from the results of several tests.

The first possibility is adopted in the procedures outlined in Appendix A. Using this approach it is possible to carry out various interpolations outlined below.

The procedure for choosing the value of A_{eu} is in general agreement with the present AISI Specification Section 6.2. According to this

section, three specimens are tested first. If the results of any test does not differ from the mean of all the tests by more than 10 percent then the mean value is used. If the deviation from the mean is more than 10 percent then three more tests are conducted. The average of the three lowest are regarded as the result of the series of tests.

It is possible to carry out evaluations of the multiple test results using the equations derived in this report directly from the plots of A_{eu} versus f . This procedure would be much more complicated.

10.5 INTERPOLATIONS

10.5.1 INTERPOLATIONS BETWEEN THICKNESSES TESTED

Using the first approach of Section 10.4, it is possible to carry out various interpolations. For example, where a series of sections with identical cross-sectional dimensions and hole dimensions and locations are produced in a variety of thicknesses, with t_1 and t_2 being the minimum and maximum thicknesses, the value of A_{eu} for a given thickness t can be calculated as follows:

$$\frac{A_{eu}}{A} = \frac{A_{eu1}}{A_1} + \frac{(A_{eu2}/A_2) - (A_{eu1}/A_1)}{t_1 - t_2} (t_1 - t) \quad \text{Eq. 10-13}$$

where A_1 and A_2 are the full unreduced areas of the sections with minimum and maximum thicknesses. A_{eu1} and A_{eu2} are the effective areas of the sections with minimum and maximum thicknesses at ultimate stub column loads.

Some results of the parametric studies on the validity of Eq. 10-13 are presented in Fig. 10-10. This figure shows that a straight line drawn between any two points for two different thicknesses would underestimate the value of A_{eu} between the two points. This leads to a conservative estimate of the A_e versus f relationship. These curves were

drawn for a 4"x4" tubular section. Because the thicknesses are varied over a wide range, the results are typical of other tubular sections as well. Since the square tube gets affected by local buckling significantly, other sections with combinations of effective and partially effective plate elements should behave in a similar manner. The same reasoning is used in the interpolations and extrapolations studied below.

10.5.2 EXTRAPOLATION FOR YIELD STRENGTH

In using the above interpolation equation as well as in other situations, a procedure is needed for adjusting the test results for variations in the yield strength. Making straight-line interpolations between two yield strengths for a given thickness can be studied on the basis of Figs. 10-11 and -12. It is seen that straight-line interpolations would be unconservative. In Fig. 10-12 it is seen that the following equation with $m = 0.8$ gives a good approximation of the A_{eu} values:

$$A_{eu} = A_{eu1} - (A_{eu1} - A_{eu2})[(F_y - F_{y1}) / (F_{y2} - F_{y1})]^m \quad \text{Eq. 10-14}$$

where A_{eu1} and A_{eu2} are the values for the same section with yield stresses F_{y1} and F_{y2} , respectively. Straight line with $m = 1$ gives unconservative results.

A more typical situation is to determine the value of A_{eu} for a yield stress F_y from a test where A_{eut} was obtained on a specimen with yield stress F_{yt} . On the basis of a rather extensive numerical study, the following procedure was obtained. The results of this parametric study are presented in Figs. 10-13 through -20. For yield strength F_y smaller than F_{yt} , the value of A_{eu} can be taken as the lower of the two values obtained from Eqs. 10-15 and -16:

$$A_{eu}/A = 1 - (1 - A_{eut}/A)(F_y/F_{yt}) \quad \text{Eq. 10-15}$$

$$A_{eu}/A = (A_{eut}/A)(F_{yt}/F_y)^{0.4} \quad \text{Eq. 10-16}$$

The variation in the yield strength between 30 ksi and 65 ksi appears to be acceptable. Positive errors indicate conservative estimates of test results.

10.5.3 EXTRAPOLATION FOR THICKNESS

It is possible that multiple specimens tested may have small differences in thickness. In that event it is reasonable to adjust the test results linearly provided that the differences in the thicknesses are of the order of 20 percent. For example, if the nominal thickness is T_n and the tested specimen with an observed A_{eut} has thickness T_t , then the A_{eu} for the nominal thickness can be calculated as

$$A_{eu}/A = (A_{eut}/A)(T_n/T_t) \quad \text{Eq. 10-17}$$

The accuracy of extrapolations using Eq. 10-17 can be seen in Figs. 10-21 through -25. Positive errors indicate conservative estimates of test results.

10.6 USE OF AXIAL SHORTENING MEASUREMENTS IN DESIGN

The procedure of using Eq. 10-8 with Eq. 10-9 can be conservative, and a procedure enabling the use of observed axial shortening values when measured is desirable. The following is a development of a procedure for this purpose. The procedure will be illustrated through use of two examples.

Two examples of test results are plotted in Figs. 10-27 through -30 along with the result of calculations according to Eqs. 10-8 and -9 as well as the equations developed below. It is seen that Eqs. 10-8 and -9 predict reduction of the effective area starting with zero stress. This

was intentionally devised to make the approach conservative for a wide range of section geometries. It is seen in these figures that a function as

$$A_e/A = 1 - (1 - A_{eu}/A)[f - f_o]/(F_y - f_o)]^b \quad \text{Eq. 10-18}$$

can be fitted through the test data consisting of n number of readings taken from which f and A_e/A can be calculated for reading number i by Eqs. 10-6 and -7 as f_i and A_{ei}/A . In this equation f_o is the value of stress above which the section is not fully effective. The exponent b can be determined by a least squares approach as

$$b = \frac{\sum_{i=1}^n (X_i)(Y_i) - a \sum_{i=1}^n X_i}{\sum_{i=1}^n (X_i)^2} \quad \text{Eq. 10-19}$$

where n is the number of measurements taken

$$X_i = \ln [(f_i - f_o)/(F_y - f_o)]$$

$$Y_i = \ln (1 - A_{ei}/A)$$

ln designates natural logarithm

The error in various equations can be seen in Fig. 10-27 for one of the tests. For the same test, the error is magnified through the use of different scales in Figs. 10-28 and -29. Similar information is presented in Fig. 10-30 for another test. There the fitted curve is identical with the observed curve.

It is not possible to express a simple averaging process for obtaining a design curve. The number of tests may be decided upon the basis of A_{eu} , but the values may be selected as the lowest of the values obtained from multiple tests for each value of f.

It is interesting to compare the values of A_e obtained in the two

tests with those calculated in Figs. 10-31 through -34. It is seen that the effective width equations of the 1986 AISI Specification Section B2.1b Procedure (1) are quite conservative at subultimate loads. The effective width equations of Procedure (2) for deflection calculations give closer and very satisfactory estimate of subultimate effective areas. Since the corner radii were not specified, the corners were assumed to be sharp. This could have led to the slight underestimate seen in the figures. It may be noted that the vertical scales in Figs. 10-31 and -33 are convenient for seeing the percentage of error, whereas the scales in Figs. 10-32 and -34 exaggerate the error.

10.7 SUMMARY OF EVALUATION PROCEDURE

For a single thickness and configuration either three or six tests are carried out in accordance with the 1986 AISI Specification. The results of these multiple tests are adjusted to the nominal thickness and yield stress using Eqs. 10-15 through -17. The means and deviations used in deciding whether three or six tests are to be carried out will be based on the adjusted values of A_{eu} for variations in yield stress and thickness. The resulting A_{eu} value is a property of the section that may be tabulated for different sections. Extrapolations of this value for properties varying, as discussed above, from those of the test specimens appear reasonable.

If the same section configuration (including perforation patterns and dimensions if any) is used for different thicknesses, a series of tests can be carried out and evaluated as described in the preceding paragraph for the thinnest and the thickest sections. Then interpolations may be carried out using Eq. 10-13 for intermediate thicknesses.

10.8 SPECIMEN PREPARATION TOLERANCES AND TESTING RATES

The study presented by Peköz (1979b) and other experience at Cornell University indicates that for industrial purposes the specimens can be saw cut to a tolerance of ± 0.001 inch. If such tolerances cannot be maintained because of the shape of the cross-section, the ends may be machined as flat as possible and bearing plates welded to each end.

The application of the load at a rate of about 1.5 ksi per minute appears to be satisfactory. This rate was used in the tests reported by Peköz (1979b).

10.9 FURTHER STUDIES NEEDED

Using the approaches developed in this report, the author has carried out a statistical study of the determination of the number of tests required and the evaluation of the test results on the basis of the number of tests. This study will be published in the near future.

To a large degree, mostly analytical approaches were used to develop the approaches given in this report. It is very desirable to verify these approaches experimentally. Some work is in progress at Cornell University on this subject.

The applicability of the unified approach and the use of the stub column test results for members with significant perforations need to be studied.

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A P P E N D I X A

DRAFT OF POSSIBLE STUB COLUMN TEST
AND EVALUATION PROCEDURES

A1. OBJECTIVES OF STUB COLUMN TESTS

The primary objective of the stub column test of prismatic cold-formed steel sections is to determine the variation of the effective area of a compression member with the stress on the effective area when it cannot be calculated. The results of the procedure described below also include the effects of residual stresses and work hardening due to cold-forming. In the 1986 AISI Specification, the ultimate column load is determined by multiplying the column buckling stress based on full unreduced section by the effective area calculated at that stress.

A secondary objective is the determination of the extent of local deformations with axial load when the appearance of the member is important.

A2. TEST SPECIMEN AND ITS PREPARATION

A2.1 GENERAL

If possible, stub column specimens shall be cut from a member formed by the same procedure as used or contemplated for use in the prototype. If this is not possible, the amount of cold work in producing the specimens shall be less than that for the contemplated prototype. If the prototype is made by a forming procedure different than that used to fabricate the specimens, it is advisable to confirm the test results later by using specimens cut from the commercially produced member.

A2.2 LENGTH OF SPECIMEN

To insure proper representation of local buckling and post-buckling behavior the following minimum lengths shall be used:

- If end plates are not welded, the length of the specimen shall be not less than 3 times the largest dimension of the section.

- If end plates are welded, the length of the specimen shall be not less than 4 times the largest dimension of the section.
- If the member has a repeating perforation pattern the minimum length of column shall be chosen so that the local buckling and post-buckling behavior is properly represented.

For perforated columns with the largest dimension of the cross section W , the following guidelines will be used in deciding the length of the specimen to be used:

If a complete perforation pattern (single or multiple group of perforations) has a length along the column less than W , the specimen length may be chosen with the guidelines for unperforated columns (as in Figs. A1b, c, g).

If a complete perforation pattern (single or multiple group of perforations) has a length along the column larger than W (as in Figs. A1d, e, f), then the specimen length should be not less than 3 times the perforation pattern length if the ends are not welded and 4 times the perforation pattern length if the ends are welded. In this case it is desirable to have the more significant perforation at midheight of the stub column (compare Figs. A1d and e).

If the perforations are such that a cut cannot be made without going through a perforation as in Fig. A1i, a special section may be fabricated as in Fig. A1j. Tests may be carried out with the cut going through a perforation if the member has a similar configuration in actual application and if the effect of such an end condition is to be evaluated.

It is desirable to have the length of the specimen not more than 20 times the least radius of gyration. For members with perforations, the least radius of gyration of the net section shall be used. This

requirement is to ensure that the overall column-buckling behavior will not affect the stub column behavior. If this requirement cannot be met because of the requirement for minimum length, then the influence of the overall column behavior shall be accounted for as described in Section A4.2.

A2.3 PREPARATION OF THE TEST SPECIMEN

The stub column specimen shall be cut from the member without causing annealing that would change the properties of the stub column significantly. The specimens can be saw cut to a tolerance of + or - 0.002 inch out of the plane of the ends. Careful saw cutting should give such a tolerance if the plate elements are not too slender. If the plate elements are very slender, the vibrations of the section during saw cutting can be reduced by clamping wooden blocks to the section. The end planes shall be perpendicular to the axis of the specimen.

If such tolerances cannot be maintained because of the shape of the cross section, the ends shall be machined as flat as possible and bearing plates shall be welded to each end. (The above tolerance and the requirement regarding the welded end plates are based on the study reported in Appendix B.) A welding procedure that minimizes residual stresses (such as sequential welding, with time allowed between short segments of welding to allow cooling, etc.) shall be used. The resulting weld should be continuous around the section. The welded end plates shall be at least 1/2-inch thick and ground to plane a tolerance of .0015 inch.

A2.4 NUMBER OF TESTS

The number of tests shall be in accordance with the AISI Specification.

A2.5 TESTING APPARATUS

The tests shall be performed in a testing machine complying with the requirements prescribed in ASTM Methods E4, Verification of Testing Machines.

A2.6 INSTRUMENTATION

Displacement measuring devices with 0.0001-inch divisions shall be used to measure the axial shortening. For this purpose the displacement measuring device shall be supported firmly on the inner surface of one end plate and bear against the inner surface of the other end plate.

Local distortions of the stub column may be measured with devices with 0.001-inch divisions.

A2.7 TEST SET-UP AND ALIGNMENT

A quarter inch layer of gypsum based capping compound (such as Hydrostone) of sufficient viscosity shall be placed on the center of the testing machine table. A ground end plate shall be placed on top of the capping compound if no end plates are welded. If end plates are welded the specimen will be seated directly on the capping compound. The capping compound should cover completely the underside of the plate.

The specimen shall be placed on top of the base plate and centered with respect to the axis of the test machine. Levels shall be used to align the specimen, vertically by pressing on the base plate. Before the capping compound hardens, the other ground end plate shall be placed on the top of the specimen and a layer of capping compound shall be spread entirely over the top plate. If end plates are welded, the capping compound shall be placed directly on the top end plate. There shall be no direct contact between the machine head and the stub

column. While the capping compound hardens some load may be developed on the stub column.

Testing shall not be started until after the capping compound hardens (after about 40 minutes). The test shall begin with the load developed as a result of the hardening process of the capping compound.

A2.8 TEST PROCEDURE

After alignment as described above, the axial load shall be applied in increments not exceeding one-tenth of the estimated yield or local buckling load. If axial shortening measurements are taken, they shall be recorded at each load increment after the readings stabilize. It is a good practice to plot the results and record all observations as the test progresses.

A3. PRESENTATION OF TEST DATA

The report on each test shall contain the following:

- A description of the test specimen with all relevant dimensions including base material thickness exclusive of coating
- A description of the steel used
- A sketch of the test setup showing all relevant details
- The results of the tension tests according to ASTM A370 on specimens taken from the flats of the section or from the virgin material
- The magnitude of the maximum load observed and description of any events during the test such as the appearance of local buckles
- A plot and a table of axial shortening values versus axial load if such readings are taken
- Plot of local distortions if measured.

A4. EVALUATION OF TEST RESULTS

The effective area A_e or the effective area at ultimate load A_{eu} determined in any of the subsections of this section shall not be taken larger than the unreduced area A .

A4.1 LENGTH SHORTER THAN 20 TIMES r_{min}

If the length of the column is not longer than 20 times minimum radius of gyration of the section, r_{min} (net minimum section for perforated sections), the effective area A_e for a given stress f on the effective section shall be calculated as follows:

$$A_e = A - (A - A_{eu}) (f/F_y)^n \quad \text{Eq. A4.1-1}$$

where

$$n = A_{eu}/A \quad \text{Eq. A4.1-2}$$

A = full unreduced area of an unperforated section and the net minimum area of a perforated section.

Alternatively, the value of A_e can be calculated on the basis plots of observed axial shortening versus stress on the effective area as described in Section A4.4.

A4.2 LENGTH LONGER THAN 20 TIMES r_{min}

When the length is larger than 20 r_{min} , the test results can be evaluated using the proposed provisions and the following procedure shall be used:

1. Determine failure stress F_n using the Specification
2. Determine A_e

$$A_e = P_u / F_n \quad \text{Eq. A4.2-1}$$

3. Calculate A_{eu} with the following equation:

$$A_{eu} = A - (A - A_e) / (F_n / F_y)^n \quad \text{Eq. A4.2-2}$$

$$\text{where } n = A_{eu} / A \quad \text{Eq. A4.2-3}$$

For the first try take $n = .2$

4. Using the value of A_{eu} obtained determine a new value of n and A_{eu} by Eqs. A4.2-3 and -2, respectively
5. Repeat step 3 until the value of A_{eu} used in Eq. A4.2-3 equals the value of A_{eu} calculated by Eq. A4.2-2
6. With the value of A_{eu} converged upon in step 5 use Eq. A4.1-1 of Section A4.1 to obtain A_e for any value of f .

A4.3 EVALUATION OF MULTIPLE COLUMN TEST RESULTS

For a given geometry and thickness the value of the number of tests to be carried out shall be determined in accordance with the present AISI Specification Section 6.2. According to this section, first three specimens are tested. The values of A_{eu} determined for any test does not differ from the mean of all the tests by more than 10 percent then the mean value A_{eu} is used. If the deviation from the mean is more than 10 percent three more tests are conducted. Then the average of the three lowest are regarded as the result A_{eu} .

The results of multiple tests shall be adjusted to the nominal thickness and yield stress using Sections A4.3.2 and .3 below. The means and deviations used in deciding whether three or six tests are to be carried out will be based on the values of A_{eu} adjusted for variations in yield stress and thickness. The resulting A_{eu} value is property of the section that may be tabulated for different sections. Extrapolations of this value for properties varying from those of the test specimens may be carried out according to Sections A4.3.2 and .3.

If the same section configuration (including perforation patterns and dimensions if any) is used for different thicknesses, a series of tests can be carried out and evaluated for the thinnest and the thickest sections. Then interpolations may be carried out using Section A4.3.1 for intermediate thicknesses.

A4.3.1 INTERPOLATION FOR THICKNESSES NOT TESTED

For sections with identical cross-sectional dimensions and hole dimensions and locations produced in a variety of thicknesses, A_{eu} for a thickness t can be determined by interpolation from the results of tests on specimens with the minimum and maximum thicknesses, t_1 and t_2 , respectively, as follows:

$$\frac{A_{eu}}{A} = \frac{A_{eu1}}{A_1} + \frac{\frac{A_{eu2}}{A_2} - \frac{A_{eu1}}{A_1}}{(t_1 - t_2)} (t_1 - t) \quad \text{Eq. A4.3.1-1}$$

where A_1 and A_2 are the full unreduced areas of the sections with minimum and maximum thicknesses. A_{eu1} and A_{eu2} are the effective areas of the sections with minimum and maximum thicknesses at ultimate stub column loads.

A4.3.2 EXTRAPOLATION FOR YIELD STRESS

If a specimen with yield stress F_{yt} is tested and A_{eu} equal to A_{eut} is obtained, A_{eu} for a given F_y can be calculated using the following procedure:

For yield strength F_y smaller than F_{yt} , the value of A_{eu} can be taken as the lower of the two values obtained from Eqs. A4.3.2-1 and -2:

$$\frac{A_{eu}}{A} = 1 - (1 - \frac{A_{eut}}{A}) (F_y / F_{yt}) \quad \text{Eq. A4.3.2-1}$$

$$\frac{A_{eu}}{A} = (\frac{A_{eut}}{A}) (F_{yt} / F_y)^{0.4} \quad \text{Eq. 4.3.2-2}$$

For yield strength F_y larger than F_{yt} , the value of A_{eu} can be obtained from Eqs. A4.3.2-2.

A4.3.3 EXTRAPOLATION FOR THICKNESS

If a specimen of thickness t_t is tested and A_{eu} equal to A_{eut} is obtained, the value of A_{eu} for a thickness t varying by not more than 20 percent from t_t can be calculated by the following equation:

$$A_{eu}/A = (A_{eut}/A)(t/t_t) \quad \text{Eq. A4.3.3-1}$$

A4.4 USE OF AXIAL SHORTENING MEASUREMENTS IN DESIGN

At any axial, P , in a stub column test, the effective area, A_e , and the stress on the effective area, f , can be calculated using the equations below. In all cases the value of A_e or the value of A_{eu} at ultimate load should not be taken greater than A , full unreduced area of the section.

$$A_e = P D_u / F_y D \quad \text{Eq. A1}$$

$$f = F_y D / D_u \quad \text{Eq. A2}$$

where F_y is the yield stress of the material, D is the axial shortening at load P and D_u is the axial shortening at ultimate load.

If n number of readings are taken A_e and f for the reading number i , A_{ei} and f_i , respectively, can be calculated using Eqs. A1 and A2. The following expression can be used for determining A_e for any stress f :

$$A_e/A = 1 - (1 - A_{eu}/A)[(f - f_o)/(F_y - f_o)]^b$$

where f_o is the value of stress above which the section is not fully

effective. The exponent b can be determined by a least squares approach as

$$b = \frac{\sum_{i=1}^n (X_i)(Y_i) - a \sum_{i=1}^n X_i}{\sum_{i=1}^n (X_i)^2}$$

where $X_i = \ln [(f_i - f_o)/(F_y - f_o)]$

$$Y_i = \ln (1 - A_{ei}/A)$$

$$a = \ln (1 - A_{eu}/A)$$

ln designates natural logarithm

It is not possible to express a simple averaging process for obtaining a design curve. The number of tests may be decided upon the basis of A_{eu} , but the values should be selected as the lowest of the values obtained from multiple tests for each value of f.

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TABLES AND FIGURES

TABLE 2.3-1
EVALUATION OF TEST RESULTS

| Comp. Flange (1) | Web (2) | Test on (3) | Sources (4) | No. (5) | Mean | C.O.V. |
|------------------------|------------|-------------------|----------------|------------|----------|----------|
| U | F | S | A,B | 14 | 1.242 | .052 |
| U | F | B | A | 16 | 1.201 | .081 |
| E | F | S | C | 28 | 1.013 | .060 |
| E | F | F | C | 4 | 1.130 | .093 |
| ES | F | B | D | 8 | 1.146 | .046 |
| U | P | S | E | 8 | 1.114 | .064 |
| U | P | S | F | 12 | .970 | .077 |
| U | P | S | J | 7 | 1.015 | .044 |
| U | P | B | G | 23 (6) | .951 | .105 |
| UW | P | B | H | 9 | .954 | .026 |
| ST | P | B | K | 16 | 1.032 | .067 |
| ST | P | B | I | 14 | .932 | .061 |
| ST | P | B | L | 54 | .891 (7) | .106 (7) |
| ST | P | B | L | 54 | .988 (8) | .060 (8) |

(1) Compression flange
U Unstiffened
E Edge stiffened
ES Edge stiffened with a sloping stiffener
ST Stiffened
UW Unstiffened (built-up I section by welding)

(2) Web
F Fully effective
P Partially effective

(3) Tests on
S Stub Columns
B Beams

(4) Sources
A. Kalyanaraman, Peköz and Winter (1972)
B. DeWolf, Peköz and Winter (1973)
C. Desmond, Peköz and Winter (1978)
D. Cohen and Peköz (1987)
E. Mulligan and Peköz (1983)
F. Chilver (1953)
G. LaBoube and Yu (1978)
H. Johnson (1976)
I. Kalsner (1977)
J. Peköz (1977)
K. LaBoube (1983)
L. van Neste (1983)

(5) Number of tests

(6) Only specimens without calculated inelastic reserve strength considered.

(7) Sections had large corner radii. Actual corner radius used in calculations.

(8) Same sections as in (7). Corner radii assumed equal to thickness.

TABLE 2.4.1-1
DIMENSIONS OF TEST SPECIMEN SECTIONS¹

| Specimen ² | W (in) | W (in) | T (in) | Width-Thickness ³ Single Thickness Element | Width-Thickness Double Thickness Element |
|--------------------------|-----------|-----------|-----------|---|--|
| (a) Stiffened Sections | | | | | |
| S-1 | 2.0 | 3.5 | 0.058 | 57.2 | 16.7 |
| S-2 | 2.0 | 5.0 | 0.058 | 83.0 | 16.7 |
| S-3 | 2.0 | 7.0 | 0.058 | 117.4 | 16.7 |
| S-4 | 2.0 | 9.0 | 0.058 | 151.8 | 16.7 |
| (b) Unstiffened Sections | | | | | |
| U-1 | 1.0 | 3.0 | 0.058 | 16.2 | 24.8 |
| U-2 | 1.25 | 3.0 | 0.058 | 20.5 | 24.8 |
| U-3 | 1.5 | 3.0 | 0.058 | 24.8 | 24.8 |
| U-4 | 1.75 | 3.0 | 0.058 | 29.1 | 24.8 |

¹ See Fig. 2.4-1 for cross-sectional notation

² Inside corner radii of all specimens were extremely small and may be considered as zero.

³ Width based on distance to inside of adjoining element.

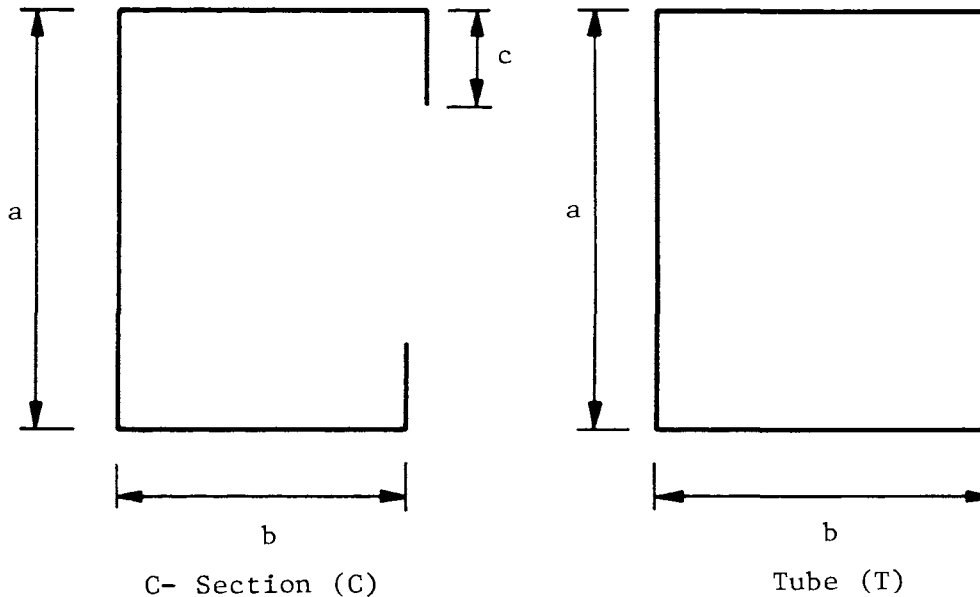
TABLE 3.3-4
LOCALLY STABLE BEAM-COLUMNS
EVALUATION OF TEST RESULTS
Lipped Channel Sections from Loh and Peköz (1985)

| Specimen | w ₁ (in) | w ₂ (in) | w ₃ (in) | t (in) | L (in) | r _i (in) | f _y (ksi) | e _x (in) | e _y (in) | $\frac{P_{uexp}}{P_{na}}$ | $\frac{P_{uexp}}{P_{nb}}$ |
|------------------------------|------------------------|------------------------|------------------------|-----------|-----------|------------------------|-------------------------|------------------------|------------------------|---------------------------|---------------------------|
| LC9-LS-1 | 3.396 | 2.334 | .604 | .104 | 39.5 | .201 | 58.2 | 1.50 | 2.00 | 1.18 | 1.08 |
| LC9-LS-2 | 3.396 | 2.334 | .604 | .104 | 52.8 | .201 | 58.2 | 1.50 | 2.00 | 1.13 | 1.08 |
| LC9-LS-3 | 3.396 | 2.334 | .604 | .104 | 64.8 | .201 | 58.2 | 1.50 | 2.00 | 1.05 | 1.00 |
| LC9-LS-4\$ | 3.395 | 2.395 | .603 | .105 | 62.6 | .192 | 65.3 | -2.00 | 2.50 | 0.91 | 0.85 |
| LC10-LS-1* | 3.899 | 2.899 | .949 | .101 | 39.2 | .235 | 54.9 | -2.00 | 2.50 | 1.36 | 1.31 |
| LC10-LS-2* | 3.899 | 2.899 | .949 | .101 | 51.2 | .235 | 54.9 | -2.00 | 2.50 | 1.33 | 1.27 |
| LC10-LS-3 | 3.899 | 2.899 | .949 | .101 | 63.2 | .235 | 54.9 | -2.00 | 2.50 | 1.30 | 1.24 |
| LC11-LS-1 | 2.924 | 2.924 | .712 | .076 | 39.0 | .205 | 48.8 | 2.00 | 2.50 | 1.28 | 1.27 |
| LC11-LS-2 | 2.924 | 2.924 | .712 | .076 | 49.5 | .205 | 48.8 | 2.50 | 2.00 | 1.27 | 1.25 |
| LC11-LS-3 | 2.924 | 2.924 | .712 | .076 | 61.3 | .205 | 48.8 | 2.00 | 2.50 | 1.16 | 1.14 |
| Mean | | | | | | | | | | 1.23 | 1.18 |
| Standard Deviation | | | | | | | | | | 0.10 | 0.11 |
| Coefficient of Variation (%) | | | | | | | | | | 8 | 9 |

\$ Premature failure of welds joining end plates to column. Excluded from statistical evaluation.

* Premature failure of welds joining end plates to column. Included in statistical evaluation.

TABLE 4.3-1
SECTIONS FOR PARAMETRIC STUDIES



| Section | a (in) | b (in) | c (in) | t (in) | F _y (ksi) | a/t | b/t |
|---------|-----------|-----------|-----------|-----------|-------------------------|--------|--------|
| C1-1 | 8.886 | 3.374 | 1.029 | .061 | 62.9 | 145.67 | 55.31 |
| C1-2 | 17.772 | 6.748 | 1.029 | .061 | 62.9 | 291.34 | 110.62 |
| C2-1 | 4.000 | 4.000 | .750 | .050 | 50.0 | 80. | 80. |
| C2-2 | 8.000 | 8.000 | .750 | .050 | 50.0 | 160. | 160. |
| C3-1 | 2.593 | 5.025 | .793 | .046 | 33.0 | 56.37 | 109.24 |
| C3-1 | 5.186 | 10.060 | .793 | .046 | 33.0 | 112.74 | 218.98 |
| T1-1 | 8.886 | 3.374 | | .061 | 62.9 | 145.67 | 55.31 |
| T1-2 | 17.772 | 6.748 | | .061 | 62.9 | 291.34 | 110.62 |
| T1-3 | 26.658 | 10.122 | | .061 | 62.9 | 937.02 | 165.93 |
| T2-1 | 4.000 | 4.000 | | .050 | 50.0 | 80. | 80. |
| T2-2 | 8.000 | 8.000 | | .050 | 50.0 | 160. | 160. |
| T2-3 | 12.000 | 12.000 | | .050 | 50.0 | 320. | 320. |
| T3-1 | 2.593 | 5.025 | | .046 | 33.0 | 56.37 | 109.24 |
| T3-2 | 5.186 | 10.050 | | .046 | 33.0 | 112.11 | 218.48 |
| T3-3 | 7.779 | 15.075 | | .046 | 33.0 | 169.11 | 372.72 |

TABLE 7.3-1
 LOCALLY UNSTABLE BEAM-COLUMNS
 EVALUATION OF TEST RESULTS
 Lipped Channel Sections of Mulligan and Peköz (1983)

| Specimen | w_1 (in) | w_2 (in) | w_3 (in) | t (in) | L (in) | r_1 (in) | f_y (ksi) | e_x^* (in) | e_y^* (in) | P_{uexp} P_n |
|------------------------------|---------------|---------------|---------------|-------------|-------------|---------------|----------------|-----------------|-----------------|---------------------|
| GM1 | 6.119 | 3.148 | .691 | .045 | 63.0 | .109 | 32.4 | 0. | 0. | 1.186 |
| GM2 | 5.096 | 3.147 | .649 | .045 | 75.0 | .107 | 32.0 | 0. | 0. | 1.323 |
| GM3 | 6.134 | 3.146 | .648 | 0.46 | 121.1 | .109 | 32.0 | 0. | 0. | 1.189 |
| GM4 | 6.072 | 3.152 | .672 | .045 | 121.0 | .111 | 32.0 | 0. | 0. | 1.247 |
| GM5 | 6.116 | 3.121 | .696 | .048 | 75.0 | .116 | 32.5 | 0. | 0. | 1.323 |
| GM6 | 9.072 | 3.154 | .658 | .045 | 72.0 | .107 | 32.6 | 0. | 0. | 1.321 |
| GM7 | 9.088 | 3.149 | .666 | .045 | 96.1 | .111 | 32.4 | 0. | 0. | 1.291 |
| GM8 | 9.097 | 3.150 | .662 | .044 | 118.0 | .106 | 32.4 | 0. | 0. | 1.271 |
| GM9 | 9.030 | 3.151 | .702 | .048 | 96.0 | .116 | 33.1 | 0. | 0. | 1.399 |
| GM10 | 4.468 | 4.432 | .754 | .048 | 99.2 | .118 | 34.3 | 0. | 0. | 0.983 |
| GM11 | 8.694 | 4.448 | .761 | .048 | 75.1 | .114 | 31.8 | 0. | 0. | 1.256 |
| GM12 | 8.725 | 4.428 | .742 | .048 | 99.1 | .118 | 35.4 | 0. | 0. | 1.211 |
| GM13 | 8.718 | 4.440 | .729 | .048 | 99.2 | .118 | 33.9 | 0. | 0. | 1.213 |
| GM14 | 6.125 | 3.124 | .687 | .047 | 75.0 | .121 | 31.8 | 0.54 | 0. | 1.136 |
| GM15 | 6.108 | 3.134 | .704 | .048 | 75.7 | .118 | 31.8 | 0.53 | 0. | 0.927 |
| GM16 | 6.100 | 3.135 | .688 | .048 | 75.7 | .118 | 31.8 | 0.98 | 0. | 1.318 |
| GM17 | 6.092 | 3.143 | .690 | .048 | 75.0 | .116 | 32.5 | 0.21 | 0. | 1.153 |
| GM18 | 8.671 | 4.436 | .742 | .048 | 99.0 | .105 | 33.1 | 0.52 | 0. | 0.983 |
| Mean | | | | | | | | | | 1.207 |
| Standard Deviation | | | | | | | | | | 0.131 |
| Coefficient of Variation (%) | | | | | | | | | | 10.85 |
| Number of Specimens | | | | | | | | | | 18 |

* With respect to the centroid of the full section.

TABLE 7.3-2
 LOCALLY UNSTABLE BEAM-COLUMNS
 EVALUATION OF TEST RESULTS
 Lipped Channel Sections of Thomasson (1978)

| Specimen | w ₁ (in) | w ₂ (in) | w ₃ (in) | t (in) | L (in) | f _y (ksi) | e _x * (in) | e _y * (in) | $\frac{P_{uexp}}{P_n}$ |
|------------------------------|------------------------|------------------------|------------------------|-----------|-----------|-------------------------|--------------------------|--------------------------|------------------------|
| A71 | 11.775 | 3.949 | .771 | .025 | 105.9 | 56.7 | 0. | 0. | 1.029 |
| A74 | 11.795 | 3.965 | .811 | .025 | 105.9 | 57.3 | 0. | 0. | 1.029 |
| A75 | 11.775 | 3.957 | .787 | .025 | 105.9 | 57.7 | 0. | 0. | 0.985 |
| A76 | 11.814 | 3.944 | .795 | .026 | 105.9 | 41.8 | 0. | 0. | 0.972 |
| A101 | 11.803 | 3.957 | .795 | .037 | 105.9 | 67.3 | 0. | 0. | 1.082 |
| A102 | 11.803 | 3.957 | .787 | .037 | 105.9 | 66.7 | 0. | 0. | 1.031 |
| A103 | 11.783 | 3.961 | .771 | .037 | 105.9 | 66.7 | 0. | 0. | 1.094 |
| A104 | 11.742 | 9.921 | .768 | .038 | 105.9 | 68.9 | 0. | 0. | 0.964 |
| A151 | 11.783 | 3.937 | .799 | .057 | 105.9 | 55.4 | 0. | 0. | 1.402 |
| A152 | 11.814 | 3.937 | .795 | .056 | 105.9 | 55.0 | 0. | 0. | 1.316 |
| A153 | 11.806 | 3.929 | .819 | .054 | 105.9 | 57.3 | 0. | 0. | 1.364 |
| A154 | 11.835 | 3.952 | .921 | .055 | 105.9 | 57.0 | 0. | 0. | 1.347 |
| A156 | 11.785 | 3.926 | .803 | .055 | 105.9 | 55.3 | 0. | 0. | 1.322 |
| Mean | | | | | | | | | 1.149 |
| Standard Deviation | | | | | | | | | 0.171 |
| Coefficient of Variation (%) | | | | | | | | | 14.84 |
| Number of Specimens | | | | | | | | | 13 |

* With respect to the centroid of the full section.

TABLE 7.3-3
LOCALLY UNSTABLE BEAM-COLUMNS
EVALUATION OF TEST RESULTS
Lipped Channel Sections of Loh and Peköz (1985)

| Specimen | w ₁ (in) | w ₂ (in) | w ₃ (in) | t (in) | L (in) | r _i (in) | f _y (ksi) | e _x * (in) | e _y * (in) | $\frac{P_{uexp}}{P_n}$ |
|------------------------------|------------------------|------------------------|------------------------|-----------|-----------|------------------------|-------------------------|--------------------------|--------------------------|------------------------|
| LC1-LU-1 | 6.870 | 2.461 | .710 | .073 | 76.0 | .240 | 55.9 | 0. | 2.1 | 1.155 |
| LC1-LU-2 | 6.870 | 2.461 | .710 | .073 | 76.0 | .240 | 55.9 | 0. | 12.0 | 1.037 |
| LC1-LU-3 | 6.870 | 2.461 | .710 | .073 | 76.0 | .240 | 55.9 | 0. | 6.0 | 1.202 |
| LC2-LU-1 | 9.086 | 3.136 | .684 | .050 | 99.9 | .128 | 35.1 | 0. | 6.0 | 1.158 |
| LC2-LU-2 | 9.071 | 3.129 | .717 | .050 | 99.9 | .128 | 35.8 | 0. | 9.0 | 1.048 |
| LC3-LU-1 | 7.958 | 3.900 | .624 | .058 | 99.9 | .098 | 43.4 | 0. | 4.0 | 0.905 |
| LC3-LU-2 | 8.001 | 3.919 | .620 | .058 | 99.9 | .098 | 44.4 | 0. | 8.0 | 1.006 |
| LC3-LU-3 | 7.930 | 3.927 | .631 | .058 | 99.9 | .098 | 43.2 | 0. | 4.0 | 0.962 |
| LC4-LU-1 | 8.881 | 3.379 | 1.023 | .061 | 99.9 | .283 | 62.1 | 0. | 12.0 | 1.107 |
| LC4-LU-2 | 8.881 | 3.379 | 1.023 | .061 | 99.9 | .283 | 62.1 | 0. | 18.0 | 0.989 |
| LC4-LU-3 | 8.873 | 3.383 | 1.026 | .058 | 99.9 | .301 | 62.9 | 0. | 6.0 | 1.108 |
| LC5-LU-1 | 7.882 | 3.399 | 1.028 | .061 | 99.9 | .283 | 58.5 | 0. | 4.0 | 1.181 |
| LC5-LU-2 | 7.882 | 3.399 | 1.028 | .061 | 99.9 | .283 | 58.5 | 0. | 8.0 | 1.151 |
| LC5-LU-3 | 7.884 | 3.394 | 1.032 | .062 | 99.8 | .297 | 58.6 | 0. | 6.0 | 1.200 |
| LC5-LU-4 | 7.884 | 3.394 | 1.032 | .062 | 99.8 | .297 | 58.6 | 0. | 10.0 | 1.101 |
| LC6-LU-1 | 9.351 | 3.436 | 1.222 | .090 | 99.9 | .269 | 71.7 | 0. | 5.0 | 1.131 |
| LC6-LU-2 | 9.351 | 3.436 | 1.222 | .090 | 99.8 | .269 | 71.7 | 0. | 10.0 | 1.098 |
| Mean | | | | | | | | | | 1.091 |
| Standard Deviation | | | | | | | | | | 0.087 |
| Coefficient of Variation (%) | | | | | | | | | | 7.95 |
| Number of Specimens | | | | | | | | | | 17 |

* With respect to the centroid of the full section.

TABLE 7.3-4
LOCALLY UNSTABLE BEAM-COLUMNS
EVALUATION OF TEST RESULTS
Lipped Channel Sections of Loh and Peköz (1985)

| Specimen | w ₁ (in) | w ₂ (in) | w ₃ (in) | t (in) | L (in) | r _i (in) | f _y (ksi) | e _x * (in) | e _y * (in) | $\frac{P_{uexp}}{P_n}$ |
|------------------------------|------------------------|------------------------|------------------------|-----------|-----------|------------------------|-------------------------|--------------------------|--------------------------|------------------------|
| LC8-LU-1 | 8.887 | 3.379 | 1.031 | .060 | 98.9 | .284 | 61.2 | -1.0 | 2.0 | 1.206 |
| LC8-LU-2 | 8.887 | 3.379 | 1.031 | .060 | 99.2 | .284 | 61.2 | -1.0 | 2.0 | 1.222 |
| LC8-LU-3 | 8.886 | 3.374 | 1.029 | .061 | 98.9 | .298 | 62.9 | -1.0 | 4.0 | 1.217 |
| LC8-LU-4 | 8.886 | 3.374 | 1.029 | .061 | 98.9 | .298 | 62.9 | -1.0 | 4.0 | 1.195 |
| LC8-LU-5 | 8.879 | 3.375 | 1.021 | .060 | 99.1 | .299 | 63.2 | -1.0 | 6.0 | 1.177 |
| LC8-LU-6 | 8.879 | 3.375 | 1.021 | .060 | 98.7 | .299 | 63.2 | -1.0 | 6.0 | 1.235 |
| LC9-LU-1 | 7.874 | 3.400 | 1.031 | .062 | 93.4 | .282 | 69.9 | 0.38 | 3.94 | 1.011 |
| LC9-LU-2 | 7.874 | 3.400 | 1.031 | .062 | 93.1 | .282 | 69.9 | 0.38 | 6.0 | 0.905 |
| LC9-LU-3 | 7.885 | 3.404 | 1.031 | .062 | 93.1 | .251 | 70.3 | 0.38 | 6.0 | 0.945 |
| LC9-LU-4 | 7.885 | 3.404 | 1.031 | .062 | 93.1 | .251 | 70.3 | 0.63 | 3.94 | 0.922 |
| LC10-LU-1 | 9.375 | 3.439 | 1.221 | .089 | 98.9 | .286 | 70.6 | 0. | 5.5 | 1.144 |
| LC10-LU-2 | 9.375 | 3.439 | 1.221 | .089 | 98.9 | .286 | 70.6 | 0. | 5.5 | 1.154 |
| Mean | | | | | | | | | | 1.111 |
| Standard Deviation | | | | | | | | | | 0.127 |
| Coefficient of Variation (%) | | | | | | | | | | 11.45 |
| Number of Specimens | | | | | | | | | | 12 |

* With respect to the centroid of the full section.

TABLE 7.3-5
LOCALLY UNSTABLE BEAM-COLUMNS
EVALUATION OF TEST RESULTS
Lipped Channel Sections of Loughlan (1979)

| Specimen | w ₁ (in) | w ₂ (in) | w ₃ (in) | t (in) | L (in) | f _y (ksi) | e _x * (in) | e _y * (in) | $\frac{P_{uexp}}{P_n}$ |
|------------------------------|------------------------|------------------------|------------------------|-----------|-----------|-------------------------|--------------------------|--------------------------|------------------------|
| L1 | 3.998 | 1.991 | .742 | .032 | 75. | 35.1 | -0.29 | 0. | 1.022 |
| L2 | 3.981 | 1.988 | .757 | .031 | 51. | 35.1 | -0.29 | 0. | 1.078 |
| L3 | 4.045 | 2.471 | 1.001 | .031 | 75. | 35.1 | -0.40 | 0. | 0.987 |
| L4 | 4.014 | 2.472 | 1.000 | .032 | 63. | 35.1 | -0.40 | 0. | 0.961 |
| L5 | 4.001 | 2.480 | 1.013 | .031 | 51. | 35.1 | -0.41 | 0. | 1.065 |
| L6 | 5.034 | 1.992 | .735 | .031 | 75. | 35.1 | -0.07 | 0. | 1.138 |
| L7 | 4.976 | 1.987 | .742 | .031 | 63. | 35.1 | -0.07 | 0. | 1.103 |
| L8 | 4.980 | 1.991 | .753 | .031 | 51. | 35.1 | -0.07 | 0. | 1.125 |
| L9 | 5.031 | 2.469 | 1.000 | .031 | 75. | 35.1 | -0.18 | 0. | 1.115 |
| L10 | 5.015 | 2.481 | 1.004 | .031 | 63. | 35.1 | -0.19 | 0. | 1.130 |
| L11 | 4.990 | 2.477 | 1.007 | .032 | 51. | 35.1 | -0.19 | 0. | 1.052 |
| L12 | 5.981 | 1.998 | .746 | .032 | 75. | 35.1 | -0.18 | 0. | 1.071 |
| L13 | 5.992 | 1.992 | .774 | .032 | 63. | 35.1 | -0.18 | 0. | 1.044 |
| L14 | 5.974 | 1.987 | .749 | .032 | 51. | 35.1 | -0.18 | 0. | 1.088 |
| L15 | 6.061 | 2.472 | .998 | .031 | 75. | 34.1 | 0. | 0. | 1.157 |
| L16 | 6.061 | 2.476 | 1.008 | .032 | 63. | 35.1 | 0. | 0. | 1.109 |
| L17 | 5.985 | 2.480 | 1.006 | .031 | 51. | 35.1 | 0. | 0. | 1.163 |
| L18 | 7.009 | 1.976 | .742 | .031 | 75. | 35.1 | -0.22 | 0. | 1.124 |
| L19 | 6.976 | 1.987 | .745 | .031 | 63. | 35.1 | -0.22 | 0. | 1.129 |
| L20 | 6.995 | 1.988 | .771 | .032 | 51. | 35.1 | -0.22 | 0. | 1.090 |
| L21 | 7.020 | 2.478 | .992 | .031 | 75. | 35.1 | -0.16 | 0. | 1.036 |
| L22 | 7.021 | 2.480 | .996 | .031 | 63. | 35.1 | -0.16 | 0. | 1.139 |
| L23 | 6.950 | 2.481 | 1.017 | .031 | 51. | 35.1 | -0.16 | 0. | 1.030 |
| L24 | 5.997 | 1.941 | .699 | .064 | 75. | 33.8 | 0. | 0. | 1.638 |
| L25 | 5.994 | 2.430 | .967 | .065 | 75. | 33.8 | -0.08 | 0. | 1.669 |
| L26 | 5.986 | 2.428 | .974 | .066 | 63. | 33.8 | -0.08 | 0. | 1.594 |
| L27 | 5.960 | 2.434 | .978 | .065 | 51. | 33.8 | -0.08 | 0. | 1.563 |
| L28 | 7.010 | 1.940 | .708 | .064 | 75. | 33.8 | -0.11 | 0. | 1.562 |
| L29 | 6.998 | 1.944 | .707 | .065 | 63. | 33.8 | -0.11 | 0. | 1.550 |
| L30 | 6.933 | 1.933 | .712 | .065 | 51. | 33.8 | -0.11 | 0. | 1.561 |
| L31 | 7.017 | 2.435 | .969 | .064 | 75. | 33.8 | 0. | 0. | 1.710 |
| L32 | 7.006 | 2.426 | .975 | .064 | 63. | 33.8 | 0. | 0. | 1.622 |
| L33 | 6.958 | 2.441 | .980 | .064 | 51. | 33.8 | 0. | 0. | 1.636 |
| Mean | | | | | | | | | 1.244 |
| Standard Deviation | | | | | | | | | 0.251 |
| Coefficient of Variation (%) | | | | | | | | | 20.17 |
| Number of Specimens | | | | | | | | | 33 |

* With respect to the centroid of the full section.

TABLE 8.2-1
CROSS SECTIONAL DIMENSIONS
Specimens of Madugula, Prabhu and Temple (1984)

| Spec. No. | b nom (in) | t nom (in) | r _y (in) | b used (in) | w/t | F _y (ksi) |
|--------------|---------------|---------------|------------------------|----------------|--------|-------------------------|
| 1 | 1.952751 | .15748 | .3684 | 1.803 | 9.449 | 67.56 |
| 2 | 2.728341 | .11811 | .5331 | 2.611 | 20.107 | 67.56 |
| 3 | 1.980311 | .16929 | .3871 | 1.896 | 9.200 | 59.38 |

Notes: b_{nom} is the nominal leg length given in the paper
r_y is the least radius gyration given in the paper
b_{used} is the leg dimension calculated from the value of
r_y given in the paper assuming sharp corners. This
value is used in all calculations
w is the flat width calculated as b-2t
Sections 1 and 2 are cold formed. Section 3 is hot rolled

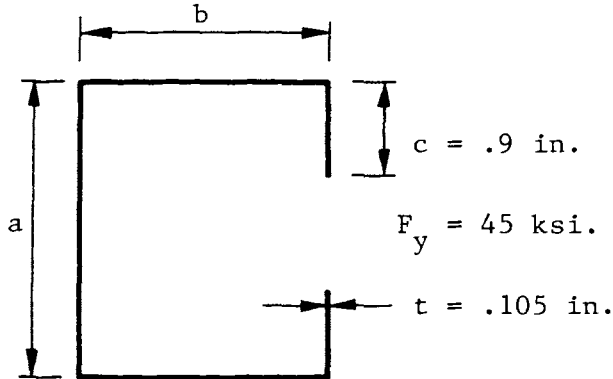
Dimensions are given to as many significant figures as
given in the paper

TABLE 8.2-2
CROSS SECTIONAL DIMENSIONS
Specimens of Wilhoit, Zandonini and Zavellani (1983)

| Spec. No. | b _{nom} (in) | t (in) | I _y (in) | b _{used} (in) | b/t (in) | F _y (ksi) |
|--------------|--------------------------|-----------|------------------------|---------------------------|-------------|-------------------------|
| 4 | 2.5591 | .1575 | .1723 | 2.359 | 10.59 | 54.68 |
| 5 | 1.7717 | .1181 | .0419 | 1.620 | 11.26 | 53.54 |

Notes: b_{nom} is the nominal leg length given in the paper
I_y is the last moment of inertia given in the paper
b_{used} is the leg dimension calculated from the value
of r_y given in the paper assuming sharp corners. This
value is used in all calculations
b is the flat width calculated based on the information
given in the paper
The dimensions are converted from metric units.

TABLE 8.3-1
EFFECT OF INITIAL SWEEP ON LIPPED CHANNEL COLUMNS



| a x b (in) | Length (in) | $e_y = 0$ | $e_y = L/1000$ | $\frac{P_{ult1}}{P_{ult2}}$ |
|---------------|----------------|-------------------|-------------------|-----------------------------|
| | | P_{ult1} (k) | P_{ult2} (k) | |
| 3 x 1 | 60 | 19.33 | 18.42 | 1.05 |
| | 100 | 7.66 | 7.35 | 1.04 |
| 3 x 3 | 60 | 32.17 | 30.70 | 1.05 |
| | 100 | 14.73 | 14.12 | 1.04 |

Note: e_y is the maximum amplitude of sweep perpendicular to the plane of symmetry

**TABLE 9.1-1
STUB COLUMN TEST RESULTS**

| Spec. No. | F _y (ksi) | d (in) | P _u (k) | R |
|------------------|-------------------------|-----------|-----------------------|-------|
| 1 | 48.50 | .00 | 15.10 | 1.019 |
| 2 | 47.10 | .00 | 14.80 | 1.024 |
| 3 | 49.60 | .50 | 14.50 | 1.006 |
| 4 | 47.10 | .75 | 14.15 | 1.054 |
| 5 | 49.60 | 1.04 | 14.05 | 1.028 |
| 6 | 51.55 | 1.25 | 13.80 | .996 |
| 7 | 48.50 | 1.50 | 12.65 | .992 |
| 8 | 51.55 | 1.75 | 13.60 | 1.035 |
| 9 | 49.60 | .50 | 15.50 | 1.075 |
| 10 | 48.50 | 1.04 | 14.68 | 1.097 |
| 11 | 48.50 | 1.50 | 14.50 | 1.137 |
| 12 | 47.00 | .00 | 27.90 | 1.274 |
| 13 | 47.60 | 1.04 | 24.60 | 1.159 |
| 14 | 47.60 | 1.50 | 24.00 | 1.214 |
| Mean* | | | | 1.072 |
| St. Dev.* | | | | 0.086 |

All sections assumed to have the average overall dimensions shown in Fig. 9.1-1.
Specimens 1-11 t = .05 in, Specimens 12-14 t = 0.076 in

*Excluding result on specimen 12 which is obviously in error.

TABLE 9.1-2
COLUMN TEST SPECIMENS

| Spec. No. | F _y (ksi) | d (in) | F _y (ksi) | s (*) | s (*) |
|--------------|-------------------------|-----------|-------------------------|-----------|-----------|
| 1 | 63.00 | .50 | 51.60 | 10.05 | 6.00 |
| 2 | 63.00 | .50 | 45.70 | 8.50 | 1.00 |
| 3 | 27.00 | 1.00 | 42.90 | 11.35 | 1.00 |
| 4 | 27.00 | .00 | 44.90 | 12.40 | .00 |
| 5 | 27.00 | 1.00 | 42.90 | 11.55 | 6.00 |
| 6 | 63.00 | 1.00 | 46.10 | 8.50 | 1.00 |
| 7 | 63.00 | 1.50 | 45.50 | 8.45 | 1.00 |
| 8 | 63.00 | 1.00 | 41.90 | 7.05 | 6.00 |
| 9 | 39.00 | 1.00 | 43.80 | 9.40 | 1.00 |
| 10 | 38.90 | 1.50 | 42.30 | 10.10 | 1.00 |
| 11 | 39.00 | .00 | 43.80 | 8.65 | .00 |
| 12 | 27.00 | .00 | 41.90 | 11.90 | .00 |
| 13 | 63.00 | .00 | 42.90 | 8.00 | .00 |
| 14 | 39.10 | .50 | 42.90 | 9.60 | 1.00 |
| 15 | 45.00 | .00 | 48.30 | 22.40 | .00 |
| 16 | 51.00 | 1.00 | 48.10 | 17.20 | 1.00 |
| 17 | 51.10 | 1.50 | 48.10 | 15.00 | 1.00 |
| 18 | 45.00 | 1.00 | 47.60 | 18.20 | 2.00 |
| 19 | 27.00 | 1.50 | 51.50 | 21.20 | 1.00 |
| 20 | 45.00 | 1.00 | 47.60 | 19.00 | 8.00 |
| 21 | 45.00 | 1.00 | 44.50 | 15.85 | 4.00 |
| 22 | 45.00 | 1.50 | 46.70 | 20.00 | 1.00 |
| 23 | 45.00 | 1.50 | 46.70 | 15.85 | 6.00 |
| 24 | 45.00 | 1.00 | 44.50 | 16.20 | 3.00 |
| 25 | 62.50 | .00 | 48.30 | 13.44 | .00 |
| 26 | 45.00 | 1.00 | 45.80 | 19.10 | 1.00 |
| 27 | 27.00 | 1.00 | 48.30 | 21.90 | 1.00 |
| 28 | 27.00 | 1.00 | 42.30 | 22.40 | 1.00 |
| 29 | 27.00 | .00 | 42.30 | 22.00 | .00 |
| 30 | 27.00 | .00 | 42.30 | 22.40 | .00 |
| 31 | 45.00 | 1.00 | 46.70 | 18.10 | 6.00 |
| 32 | 63.00 | 1.00 | 47.90 | 13.30 | 1.00 |

(*) s = 0 for column without perforations
s = 1 for column with one perforation at midheight
s > 1 spacing of the perforations along the length of the column in inches

All sections assumed to have the average overall dimensions shown in Fig. 9.1-1.
Specimens 1-14 t = .05 in, Specimens 15-32 t = .076 in

TABLE 9.3.1-1
EVALUATION OF COLUMN TEST RESULTS
(All test results)

| Spec. No. | R1 | R2 | R3 | R4 | R5 |
|--------------|-------|-------|-------|-------|-------|
| 1 | 1.692 | 1.275 | 1.525 | 1.202 | 1.208 |
| 2 | 1.469 | 1.091 | 1.317 | 1.028 | 1.028 |
| 3 | 1.458 | 1.038 | 1.186 | .936 | .939 |
| 4 | 1.258 | .986 | 1.258 | .986 | .986 |
| 5 | 1.484 | 1.056 | 1.207 | .952 | .970 |
| 6 | 1.628 | 1.160 | 1.315 | 1.026 | 1.028 |
| 7 | 1.808 | 1.237 | 1.311 | 1.022 | 1.027 |
| 8 | 1.391 | .984 | 1.113 | .869 | .887 |
| 9 | 1.318 | .922 | 1.069 | .829 | .831 |
| 10 | 1.613 | 1.077 | 1.170 | .912 | .917 |
| 11 | .984 | .763 | .984 | .763 | .763 |
| 12 | 1.262 | 1.000 | 1.262 | 1.000 | 1.000 |
| 13 | 1.257 | .980 | 1.257 | .980 | .980 |
| 14 | 1.229 | .906 | 1.105 | .859 | .860 |
| 15 | 1.299 | 1.155 | 1.299 | 1.155 | 1.155 |
| 16 | 1.570 | 1.138 | 1.080 | .969 | .951 |
| 17 | 1.611 | 1.089 | .943 | .846 | .852 |
| 18 | 1.518 | 1.105 | 1.060 | .946 | 1.021 |
| 19 | 1.584 | 1.095 | .999 | .880 | .890 |
| 20 | 1.584 | 1.154 | 1.107 | .988 | 1.007 |
| 21 | 1.372 | 1.005 | .945 | .858 | .983 |
| 22 | 1.983 | 1.342 | 1.172 | 1.052 | 1.059 |
| 23 | 1.571 | 1.064 | .929 | .834 | .855 |
| 24 | 1.402 | 1.027 | .965 | .877 | .925 |
| 25 | .961 | .961 | .961 | .961 | .961 |
| 26 | 1.627 | 1.189 | 1.127 | 1.016 | 1.019 |
| 27 | 1.471 | 1.103 | 1.071 | .956 | .960 |
| 28 | 1.661 | 1.261 | 1.183 | 1.085 | 1.091 |
| 29 | 1.162 | 1.066 | 1.162 | 1.066 | 1.066 |
| 30 | 1.183 | 1.085 | 1.183 | 1.085 | 1.085 |
| 31 | 1.525 | 1.112 | 1.061 | .952 | .977 |
| 32 | 1.545 | 1.150 | .966 | .966 | .968 |
| MEAN | 1.453 | 1.080 | 1.134 | .964 | .975 |
| ST. DEV. | .223 | .116 | .140 | .098 | .094 |

TABLE 9.3.1-2
EVALUATION OF COLUMN TEST RESULTS
(Specimens without perforations)

| Spec. No. | R1 | R2 | R3 | R4 | R5 |
|--------------|-------|-------|-------|-------|-------|
| 4 | 1.258 | .986 | 1.258 | .986 | .986 |
| 11 | .984 | .763 | .984 | .763 | .763 |
| 12 | 1.262 | 1.000 | 1.262 | 1.000 | 1.000 |
| 13 | 1.257 | .980 | 1.257 | .980 | .980 |
| 15 | 1.299 | 1.155 | 1.299 | 1.155 | 1.155 |
| 29 | 1.162 | 1.066 | 1.162 | 1.066 | 1.066 |
| 30 | 1.183 | 1.085 | 1.183 | 1.085 | 1.085 |
| MEAN | 1.171 | .999 | 1.171 | .999 | .999 |
| ST. DEV. | .130 | .116 | .130 | .116 | .116 |

TABLE 9.3.1-3
EVALUATION OF COLUMN TEST RESULTS
(Specimens with one perforation at midheight)

| Spec. No. | R1 | R2 | R3 | R4 | R5 |
|--------------|-------|-------|-------|-------|-------|
| 2 | 1.469 | 1.091 | 1.317 | 1.028 | 1.028 |
| 3 | 1.458 | 1.038 | 1.186 | .936 | .939 |
| 6 | 1.628 | 1.160 | 1.315 | 1.026 | 1.028 |
| 7 | 1.808 | 1.160 | 1.315 | 1.026 | 1.028 |
| 9 | 1.318 | .922 | 1.069 | .829 | .831 |
| 10 | 1.613 | 1.077 | 1.170 | .912 | .917 |
| 14 | 1.228 | .906 | 1.105 | .859 | .860 |
| 16 | 1.570 | 1.138 | 1.080 | .969 | .971 |
| 17 | 1.611 | 1.089 | .943 | .846 | .852 |
| 19 | 1.584 | 1.095 | .999 | .880 | .890 |
| 22 | 1.983 | 1.342 | 1.172 | 1.052 | 1.059 |
| 26 | 1.627 | 1.189 | 1.127 | 1.016 | 1.019 |
| 27 | 1.471 | 1.103 | 1.071 | .956 | .960 |
| 28 | 1.661 | 1.261 | 1.183 | 1.085 | 1.091 |
| 32 | 1.545 | 1.150 | .966 | .966 | .968 |
| MEAN | 1.572 | 1.120 | 1.134 | .959 | .963 |
| ST. DEV. | .181 | .116 | .120 | .080 | .080 |

TABLE 9.3.1-4
EVALUATION OF COLUMN TEST RESULTS
(Specimens with multiple perforations)

| Spec. No. | R1 | R2 | R3 | R4 | R5 |
|--------------|-------|-------|-------|-------|-------|
| 1 | 1.692 | 1.275 | 1.525 | 1.202 | 1.208 |
| 5 | 1.484 | 1.056 | 1.207 | .952 | .970 |
| 8 | 1.391 | .984 | 1.113 | .869 | .887 |
| 18 | 1.518 | 1.105 | 1.060 | .946 | 1.021 |
| 20 | 1.584 | 1.154 | 1.107 | .988 | 1.007 |
| 21 | 1.372 | 1.005 | .945 | .858 | .893 |
| 23 | 1.571 | 1.064 | .929 | .834 | .885 |
| 24 | 1.402 | 1.027 | .965 | .877 | .925 |
| 31 | 1.525 | 1.112 | 1.061 | .952 | .977 |
| MEAN | 1.504 | 1.087 | 1.101 | .942 | .975 |
| ST. DEV. | .105 | .089 | .183 | .111 | .102 |

TABLE 9.4-1
NONPRISMATIC COLUMNS
BUCKLING LOAD RATIOS (*)

| RI | RL | .10 | .20 | .30 | .40 | .50 | .60 |
|------|----|------|------|------|------|------|------|
| 1.00 | | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| 1.10 | | .98 | .96 | .95 | .93 | .92 | .92 |
| 1.20 | | .96 | .93 | .90 | .88 | .86 | .85 |
| 1.30 | | .94 | .89 | .86 | .83 | .80 | .79 |
| 1.40 | | .93 | .86 | .82 | .78 | .75 | .73 |
| 1.50 | | .91 | .84 | .78 | .74 | .71 | .69 |
| 1.60 | | .89 | .81 | .75 | .70 | .67 | .65 |

(*) Ratio of the buckling load for the nonprismatic column shown in Fig. 9.4-2a to that for the prismatic column shown in Fig. 9.4-2b.

**TABLE 9.4-2
NONPRISMATIC COLUMNS
BUCKLING LOAD RATIOS (*)**

| RI | RL | .10 | .20 | .30 | .40 | .50 | .60 |
|------|----|------|------|------|------|------|------|
| 1.00 | | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| 1.10 | | .99 | .98 | .97 | .97 | .97 | .97 |
| 1.20 | | .98 | .96 | .95 | .94 | .94 | .94 |
| 1.30 | | .97 | .94 | .92 | .91 | .91 | .91 |
| 1.40 | | .95 | .92 | .89 | .88 | .88 | .89 |
| 1.50 | | .94 | .89 | .87 | .85 | .85 | .86 |
| 1.60 | | .93 | .87 | .84 | .82 | .82 | .84 |

(*) Ratio of the buckling load for the nonprismatic column shown in Fig. 9.4-2a determined by the transcendental equation to that obtained by the approximate equation.

TABLE 9.4-3
PARAMETERS FOR COLUMN SPECIMENS

| Spec. | RI | RL |
|-------|-------|------|
| 1 | 1.057 | .087 |
| 2 | 1.057 | .008 |
| 3 | 1.122 | .037 |
| 4 | 1.000 | .000 |
| 5 | 1.122 | .185 |
| 6 | 1.131 | .016 |
| 7 | 1.228 | .024 |
| 8 | 1.131 | .175 |
| 9 | 1.123 | .026 |
| 10 | 1.212 | .039 |
| 11 | 1.000 | .000 |
| 12 | 1.000 | .000 |
| 13 | 1.000 | .000 |
| 14 | 1.055 | .013 |
| 15 | 1.000 | .000 |
| 16 | 1.150 | .020 |
| 17 | 1.272 | .029 |
| 18 | 1.146 | .511 |
| 19 | 1.246 | .056 |
| 20 | 1.146 | .133 |
| 21 | 1.148 | .267 |
| 22 | 1.266 | .033 |
| 23 | 1.266 | .267 |
| 24 | 1.148 | .356 |
| 25 | 1.000 | .000 |
| 26 | 1.147 | .022 |
| 27 | 1.140 | .037 |
| 28 | 1.144 | .037 |
| 29 | 1.000 | .000 |
| 30 | 1.000 | .000 |
| 31 | 1.147 | .178 |
| 21 | 1.157 | .016 |

TABLE 9.4-4
PARAMETERS AND RESULTS OF CALCULATIONS
ALL COLUMNS

| SP. NO. | RI | RL | R2 | R4 | R5 |
|---------|-------|------|-------|-------|-------|
| 1 | 1.057 | .087 | 1.275 | 1.202 | 1.208 |
| 2 | 1.057 | .008 | 1.091 | 1.028 | 1.028 |
| 3 | 1.122 | .037 | 1.038 | .936 | .939 |
| 4 | 1.000 | .000 | .986 | .986 | .986 |
| 5 | 1.222 | .185 | 1.056 | .952 | .970 |
| 6 | 1.131 | .016 | 1.160 | 1.026 | 1.028 |
| 7 | 1.228 | .024 | 1.237 | 1.002 | 1.027 |
| 8 | 1.131 | .175 | .984 | .869 | .887 |
| 9 | 1.123 | .026 | .922 | .829 | .831 |
| 10 | 1.212 | .039 | 1.077 | .912 | .917 |
| 11 | 1.000 | .000 | .763 | .763 | .763 |
| 12 | 1.000 | .000 | 1.000 | 1.000 | 1.000 |
| 13 | 1.000 | .000 | .980 | .980 | .980 |
| 14 | 1.055 | .013 | .906 | .859 | .860 |
| 15 | 1.000 | .000 | 1.155 | 1.155 | 1.155 |
| 16 | 1.150 | .020 | 1.138 | .969 | .971 |
| 17 | 1.272 | .029 | 1.089 | .846 | .852 |
| 18 | 1.146 | .511 | 1.105 | .946 | 1.021 |
| 19 | 1.246 | .056 | 1.095 | .880 | .890 |
| 20 | 1.146 | .133 | 1.154 | .988 | 1.007 |
| 21 | 1.148 | .267 | 1.005 | .858 | .893 |
| 22 | 1.266 | .033 | 1.342 | 1.052 | 1.059 |
| 23 | 1.266 | .267 | 1.064 | .834 | .885 |
| 24 | 1.148 | .356 | 1.027 | .877 | .925 |
| 25 | 1.000 | .000 | .961 | .961 | .961 |
| 26 | 1.147 | .022 | 1.189 | 1.016 | 1.019 |
| 27 | 1.140 | .037 | 1.103 | .956 | .960 |
| 28 | 1.144 | .037 | 1.261 | 1.085 | 1.091 |
| 29 | 1.000 | .000 | 1.066 | 1.066 | 1.066 |
| 30 | 1.000 | .000 | 1.085 | 1.085 | 1.085 |
| 31 | 1.147 | .178 | 1.112 | .952 | .977 |
| 32 | 1.157 | .016 | 1.150 | .966 | .968 |

TABLE 9.4-5
PARAMETERS AND RESULTS OF CALCULATIONS
UNPERFORATED COLUMNS

| SP. NO. | RI | RL | R2 | R4 | R5 |
|---------|-------|------|-------|-------|-------|
| 4 | 1.000 | .000 | .986 | .986 | .986 |
| 11 | 1.000 | .000 | .763 | .763 | .763 |
| 12 | 1.000 | .000 | 1.000 | 1.000 | 1.000 |
| 13 | 1.000 | .000 | .980 | .980 | .980 |
| 15 | 1.000 | .000 | 1.155 | 1.155 | 1.155 |
| 25 | 1.000 | .000 | .961 | .961 | .961 |
| 29 | 1.000 | .000 | 1.066 | 1.066 | 1.066 |
| 30 | 1.000 | .000 | 1.085 | 1.085 | 1.085 |

TABLE 9.4-6
PARAMETERS AND RESULTS OF CALCULATIONS

PERFORATED COLUMNS

| SP. NO. | RI | RL | R2 | R4 | R5 |
|---------|-------|------|-------|-------|-------|
| 1 | 1.057 | .087 | 1.275 | 1.202 | 1.208 |
| 2 | 1.057 | .008 | 1.091 | 1.028 | 1.028 |
| 3 | 1.122 | .027 | 1.038 | .936 | .939 |
| 5 | 1.122 | .185 | 1.056 | .952 | .970 |
| 6 | 1.131 | .016 | 1.160 | 1.026 | 1.028 |
| 7 | 1.228 | .024 | 1.237 | 1.022 | 1.027 |
| 8 | 1.131 | .175 | .984 | .869 | .887 |
| 9 | 1.123 | .026 | .922 | .829 | .831 |
| 10 | 1.212 | .039 | 1.077 | .912 | .917 |
| 14 | 1.055 | .013 | .906 | .859 | .860 |
| 16 | 1.150 | .020 | 1.138 | .969 | .971 |
| 17 | 1.272 | .029 | 1.089 | .846 | .852 |
| 18 | 1.146 | .511 | 1.105 | .946 | .021 |
| 19 | 1.246 | .056 | 1.095 | .880 | .890 |
| 20 | 1.146 | .133 | 1.154 | .988 | 1.007 |
| 21 | 1.148 | .267 | 1.005 | .858 | .893 |
| 22 | 1.266 | .033 | 1.342 | 1.052 | 1.059 |
| 23 | 1.266 | .267 | 1.064 | .834 | .885 |
| 24 | 1.148 | .356 | 1.027 | .877 | .925 |
| 26 | 1.147 | .022 | 1.189 | 1.016 | 1.019 |
| 27 | 1.140 | .037 | 1.103 | .956 | .960 |
| 28 | 1.144 | .037 | 1.261 | 1.085 | 1.091 |
| 31 | 1.147 | .178 | 1.112 | .952 | .977 |
| 32 | 1.157 | .016 | 1.150 | .966 | .968 |

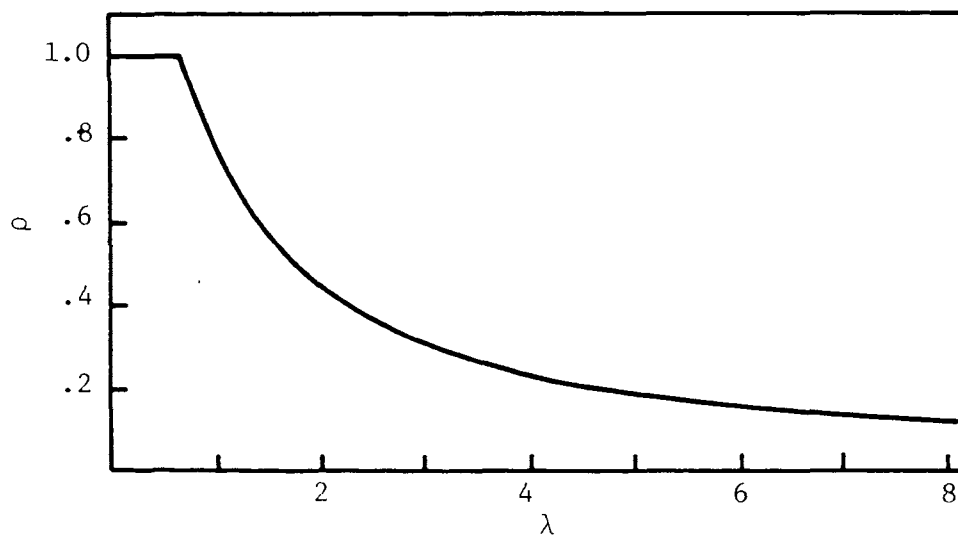


Fig. 2.2-1 Plot of Eq. 2.2-1 and -2

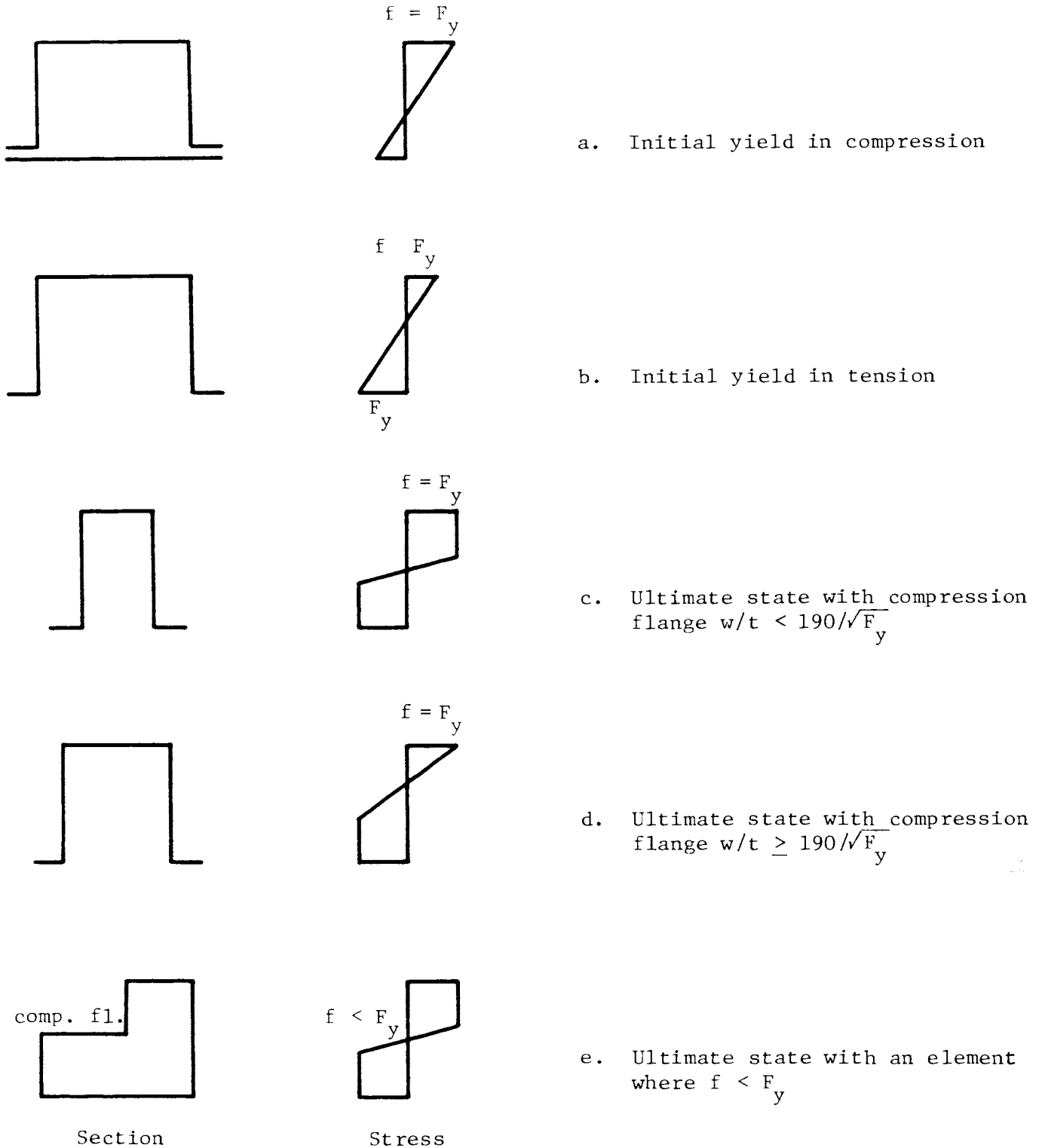


Fig. 2.2-2 Stress f in Eq. 2.2-2

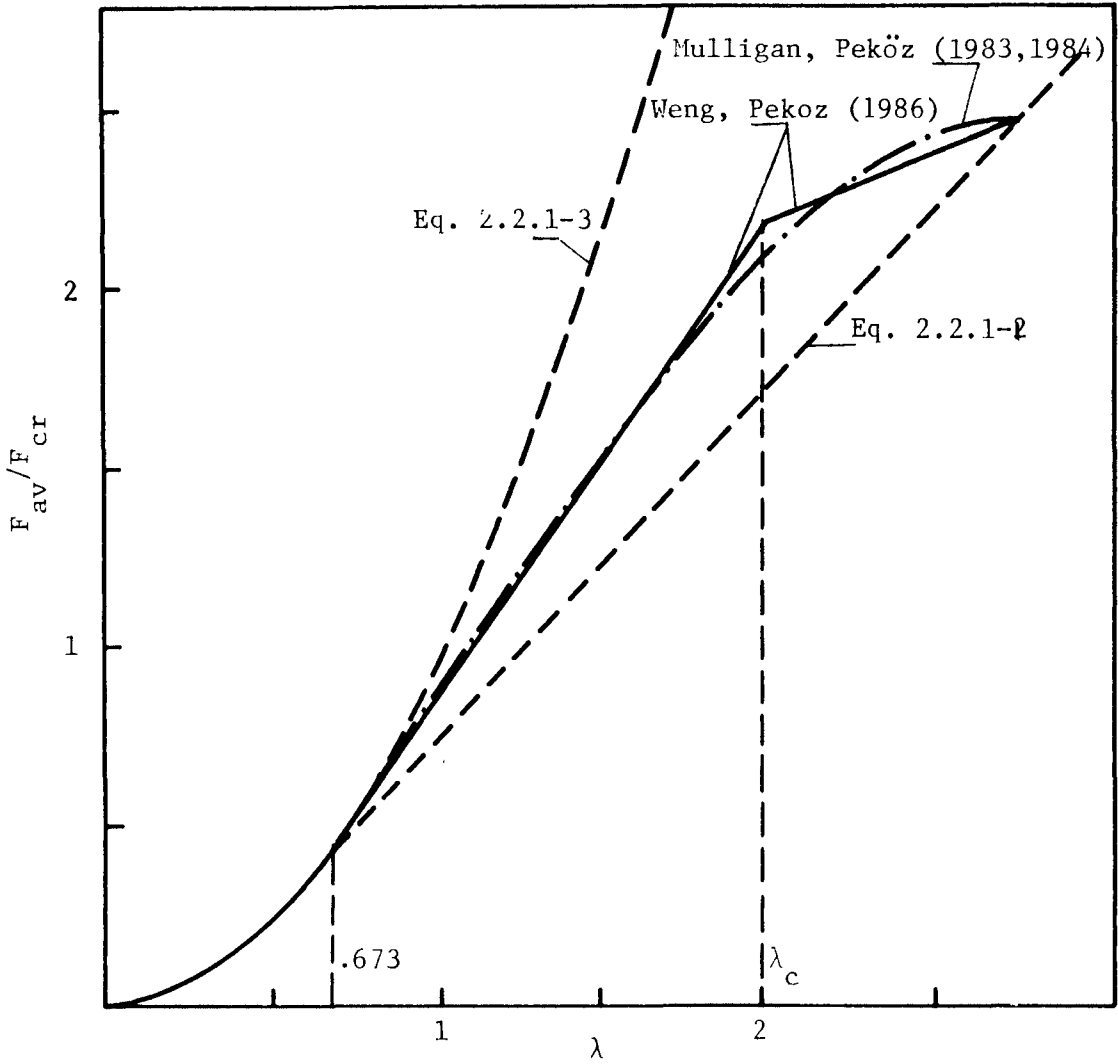


Fig. 2.2.1-1 Subultimate behavior of stiffened compression elements

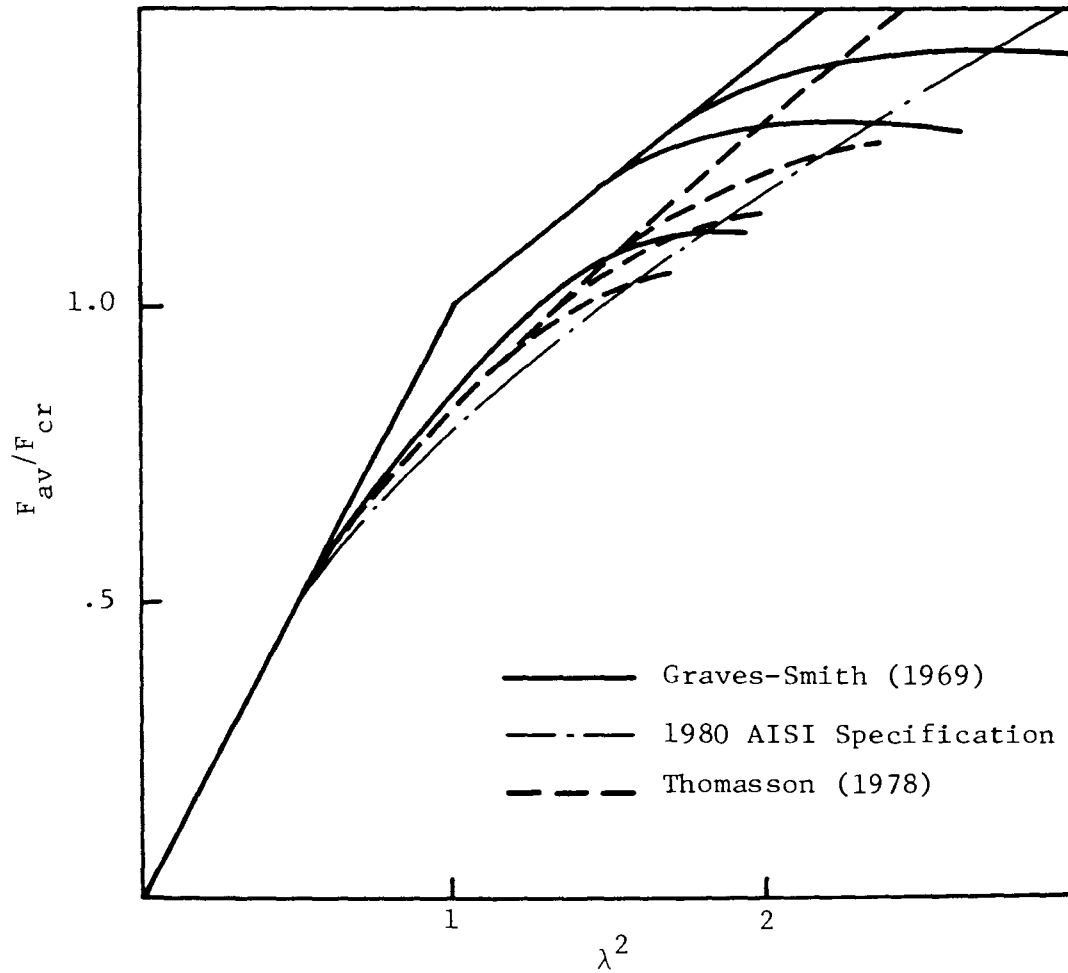


Fig. 2.2.1-2 Comparison of some subultimate behavior predictions of stiffened compression elements

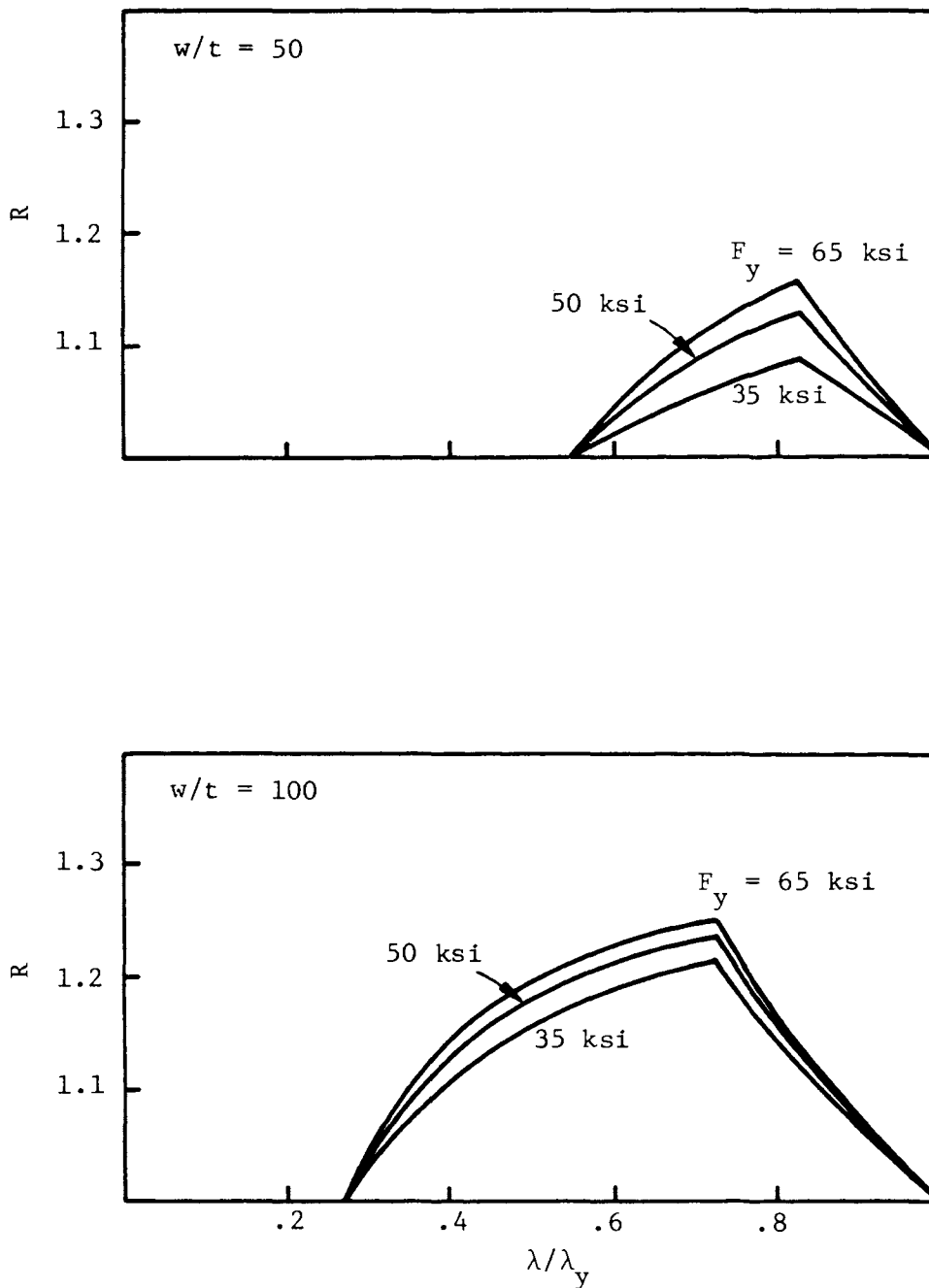


Fig. 2.2.1-3 Ratio, R , of the effective widths calculated according to Eqs. 2.2.1-4 through -6 to those calculated according to Eq. 2.2-2

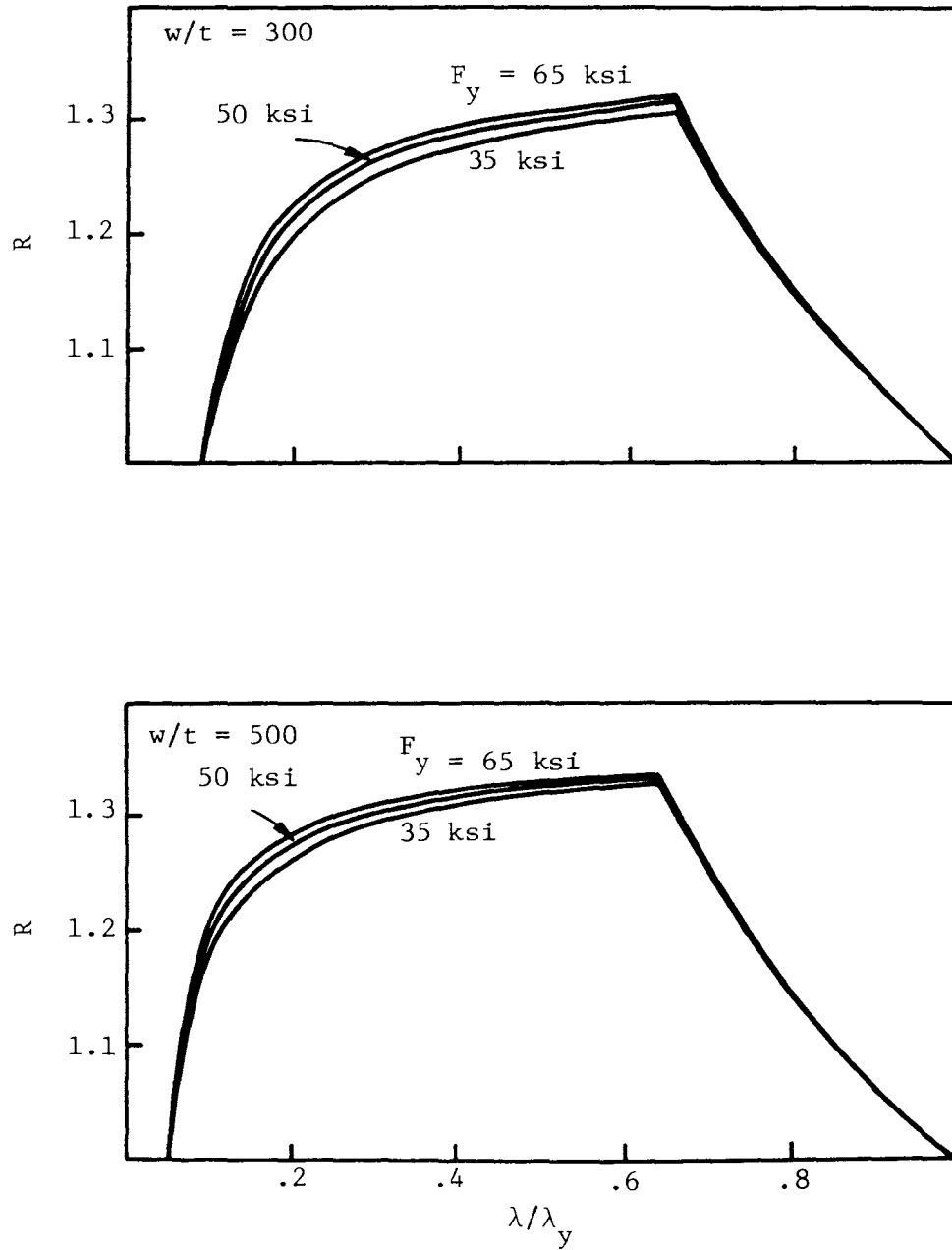


Fig. 2.2.1-3 (cont.)

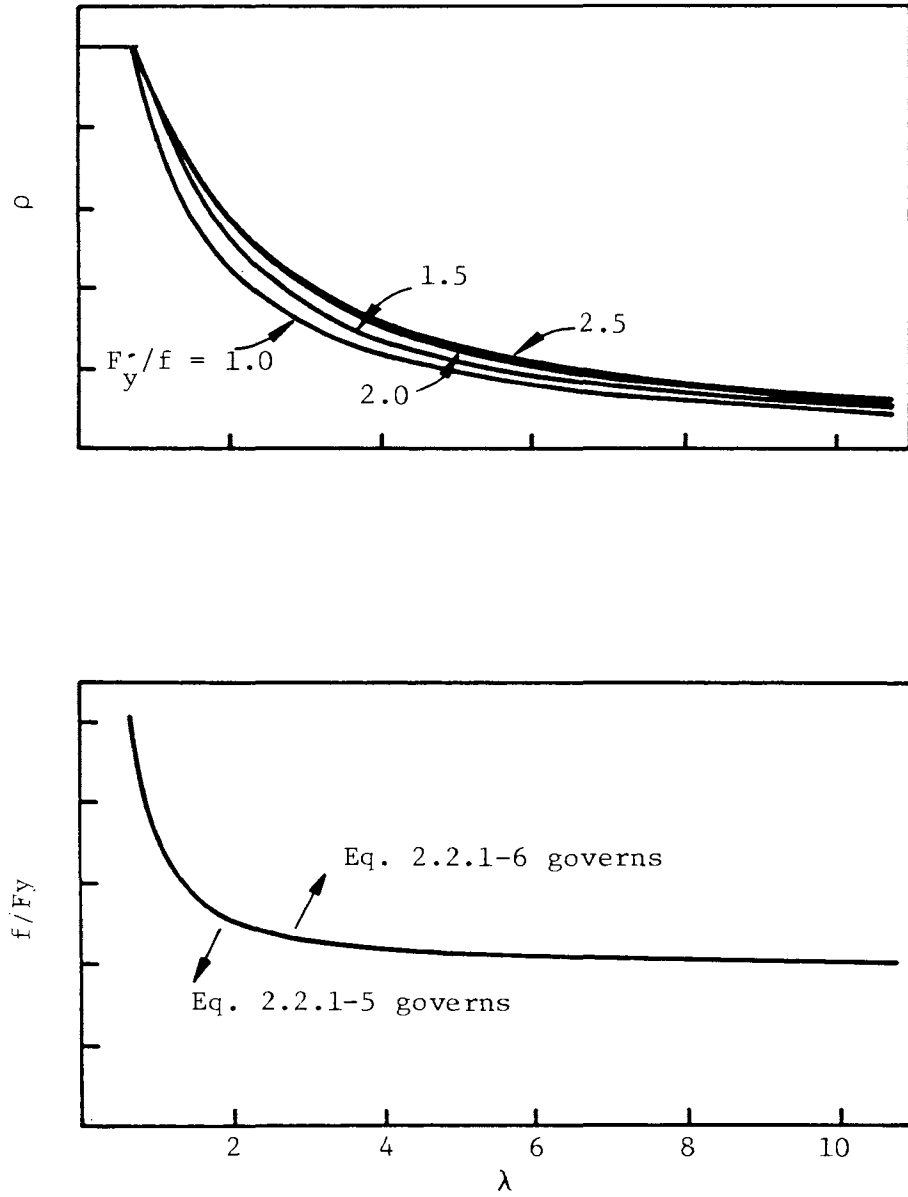


Fig. 2.2.1-4 Possible design aids for webs

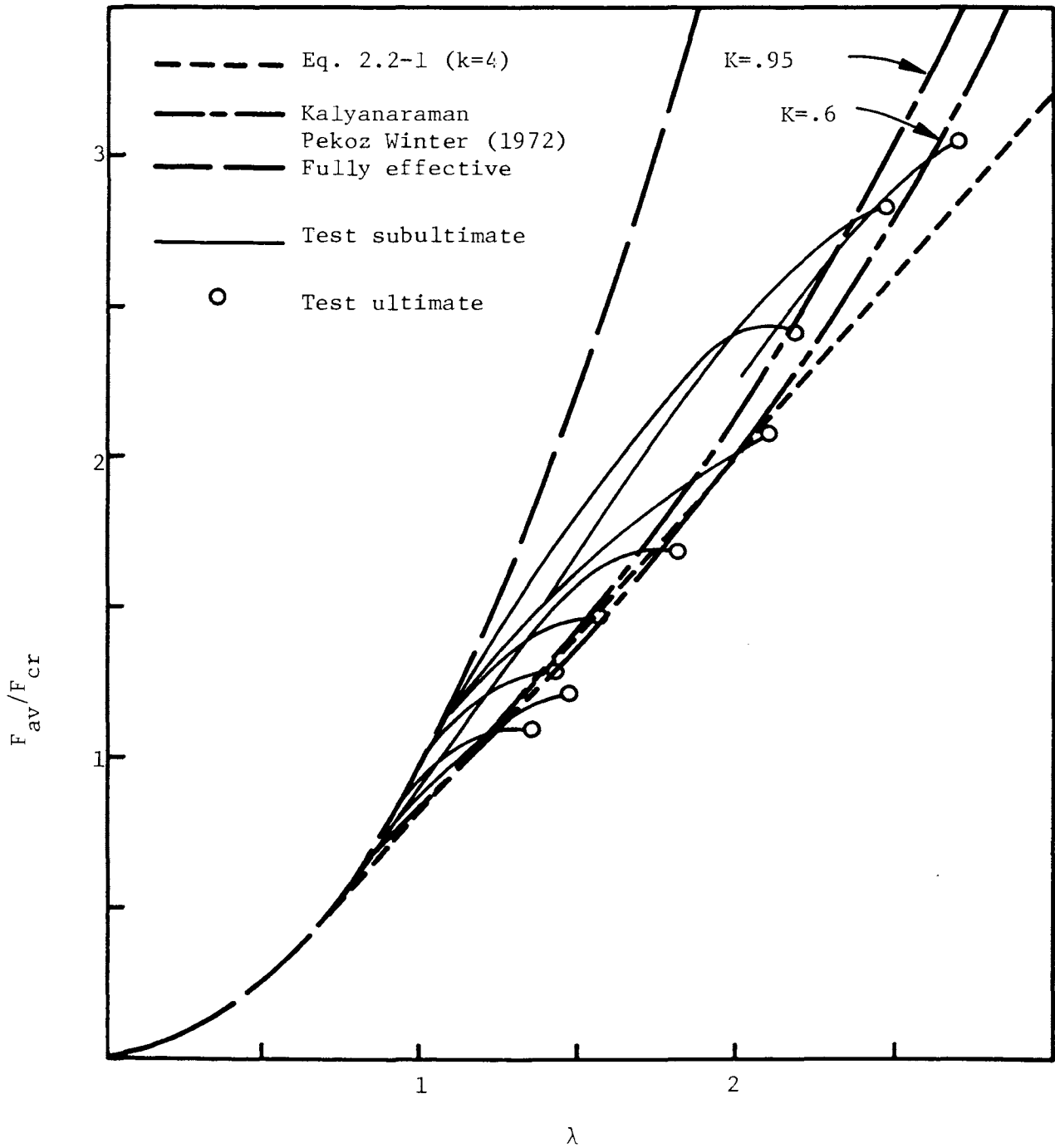
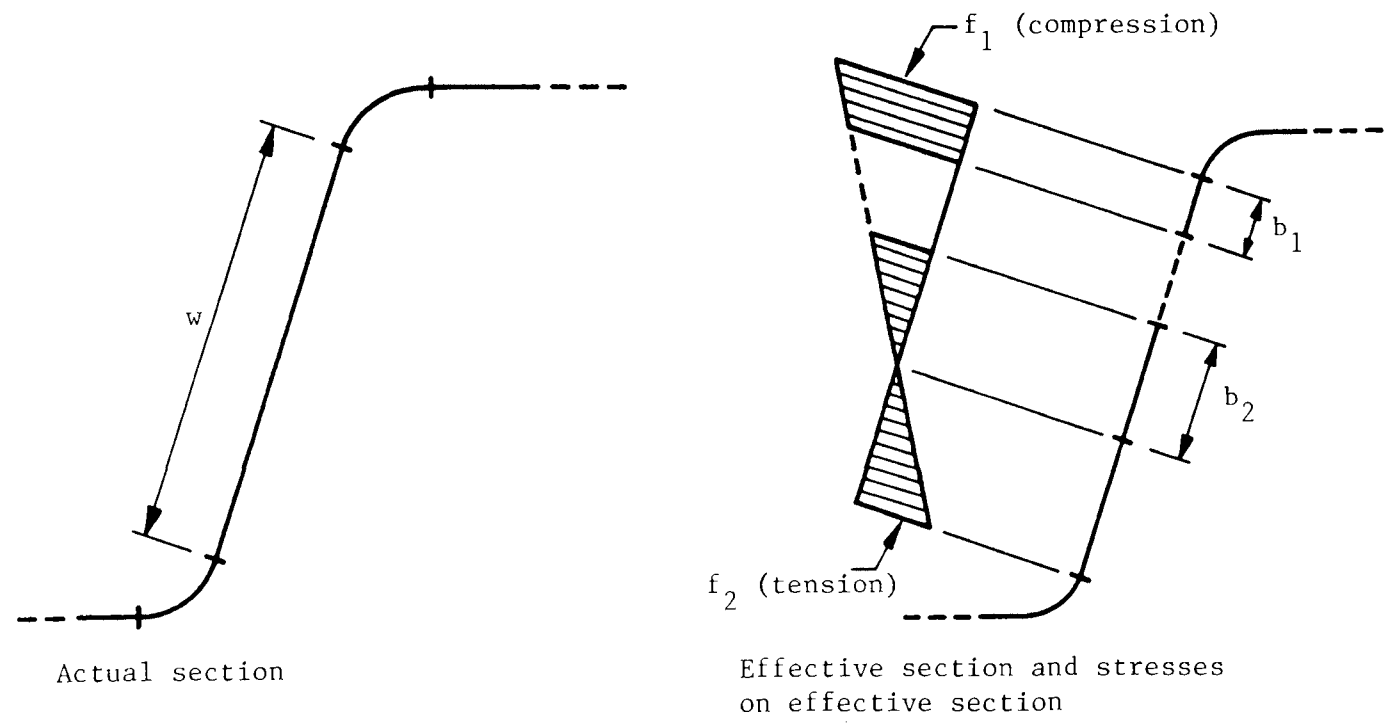


Fig. 2.2.1- 5 Subultimate behavior of beams with unstiffened flanges



Actual section

Effective section and stresses
on effective section

Fig. 2.3-1 Web effective widths

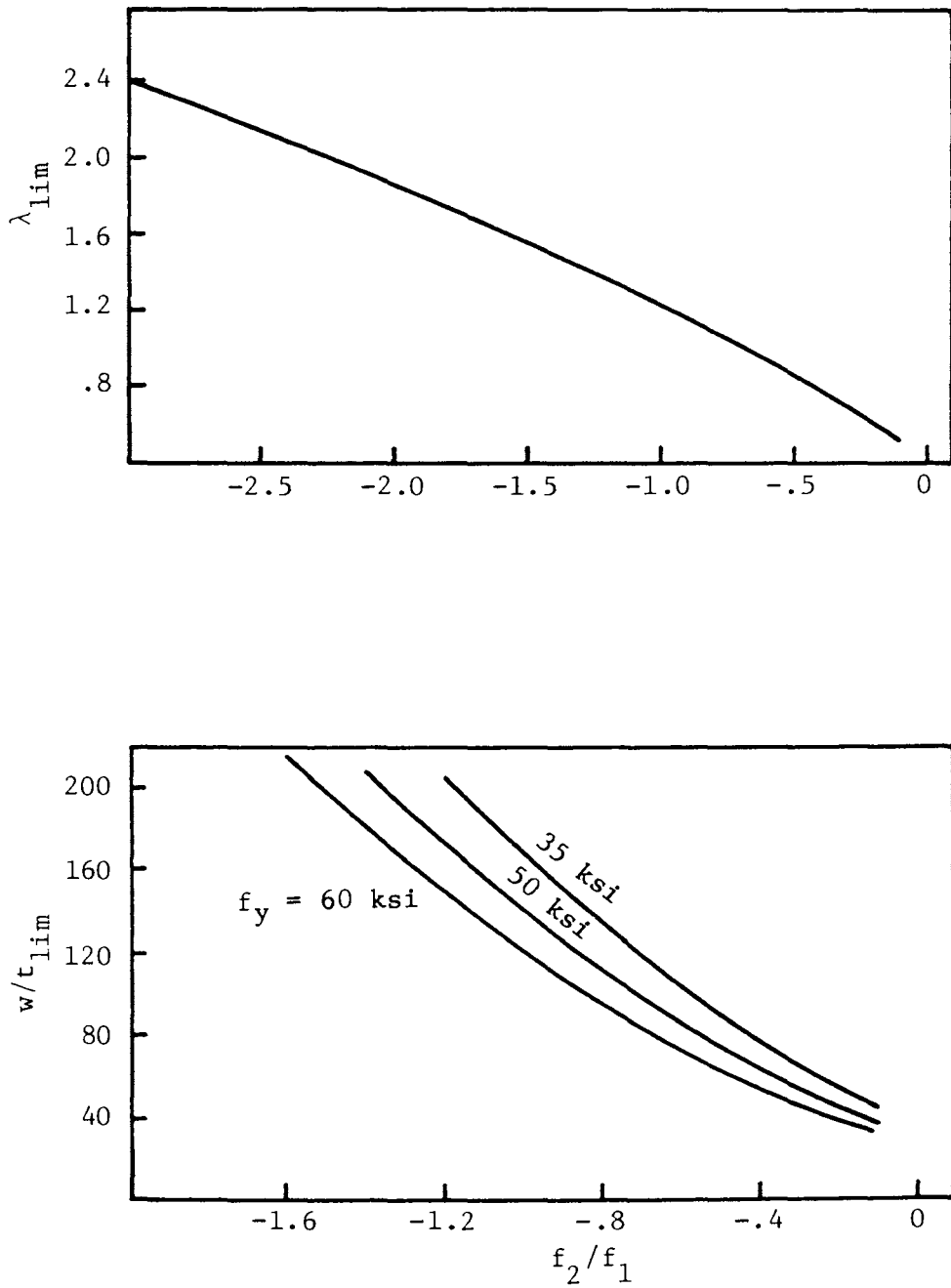


Fig. 2.3-2 Limiting values of λ and w/t above which the web is not fully effective

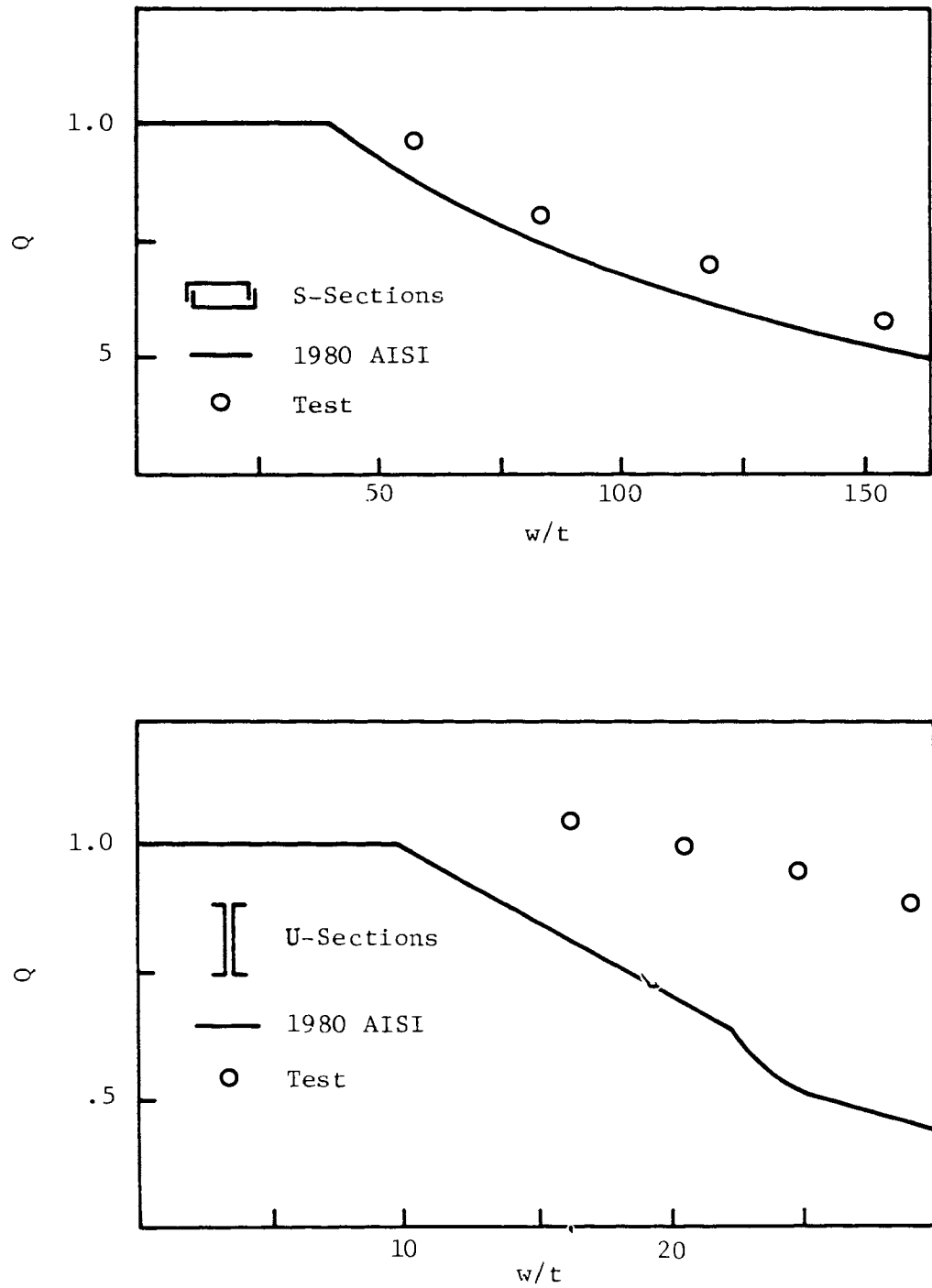


Fig. 2.4.1-1 Values of Q-factor observed in tests and according to the 1980 AISI Specification

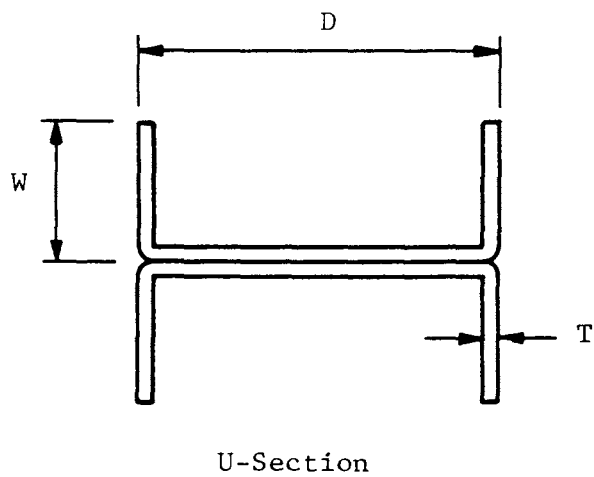
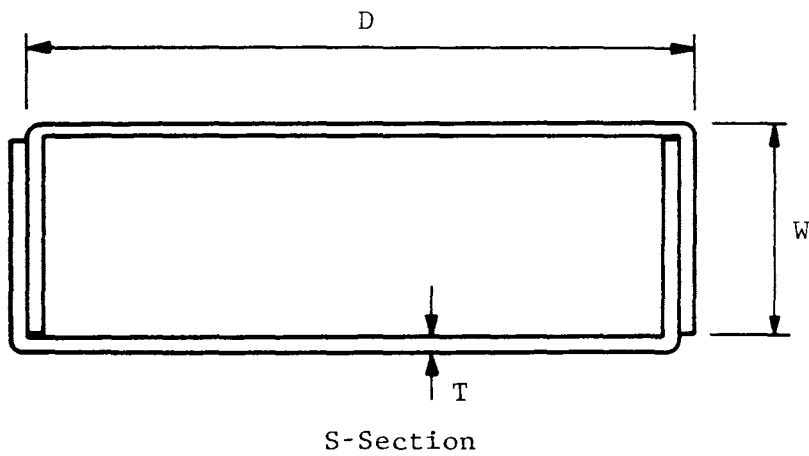


Fig. 2.4.1-2 Column specimen sections

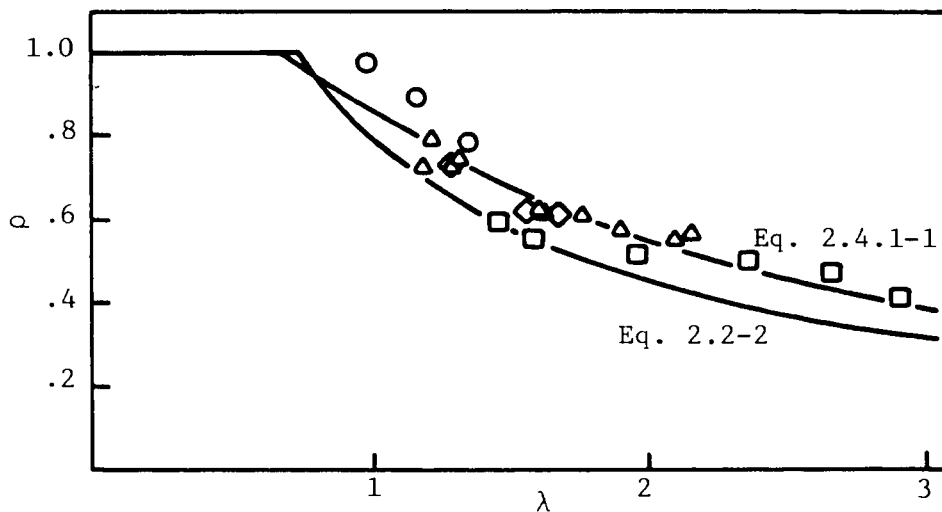


Fig. 2.4.1-3 Some stub column test results on sections with unstiffened flanges

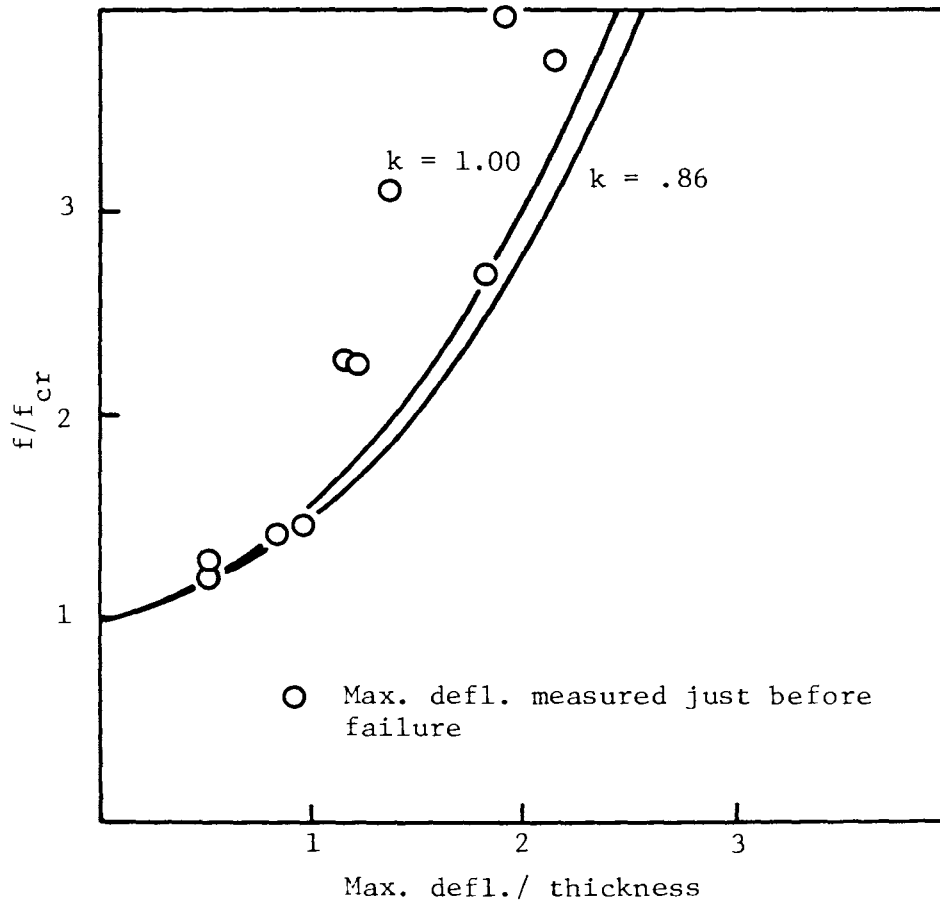


Fig. 2.4.1-4 Maximum out-of-plane deflections of unstiffened compression elements

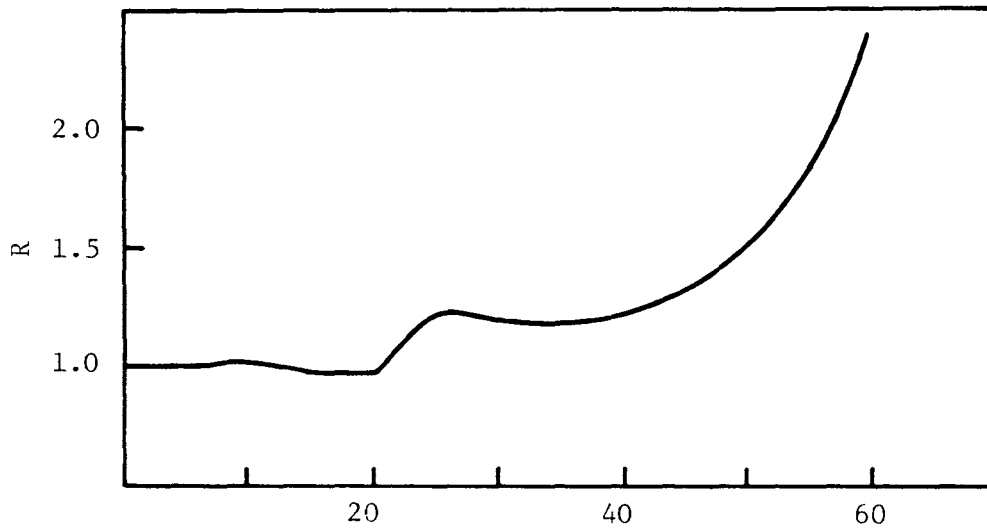


Fig. 2.4.1-5 Ratio, R, of the effective width given by Eq. 2.2-2 with $k = 4.3$ to that implied in the 1980 AISI Specification

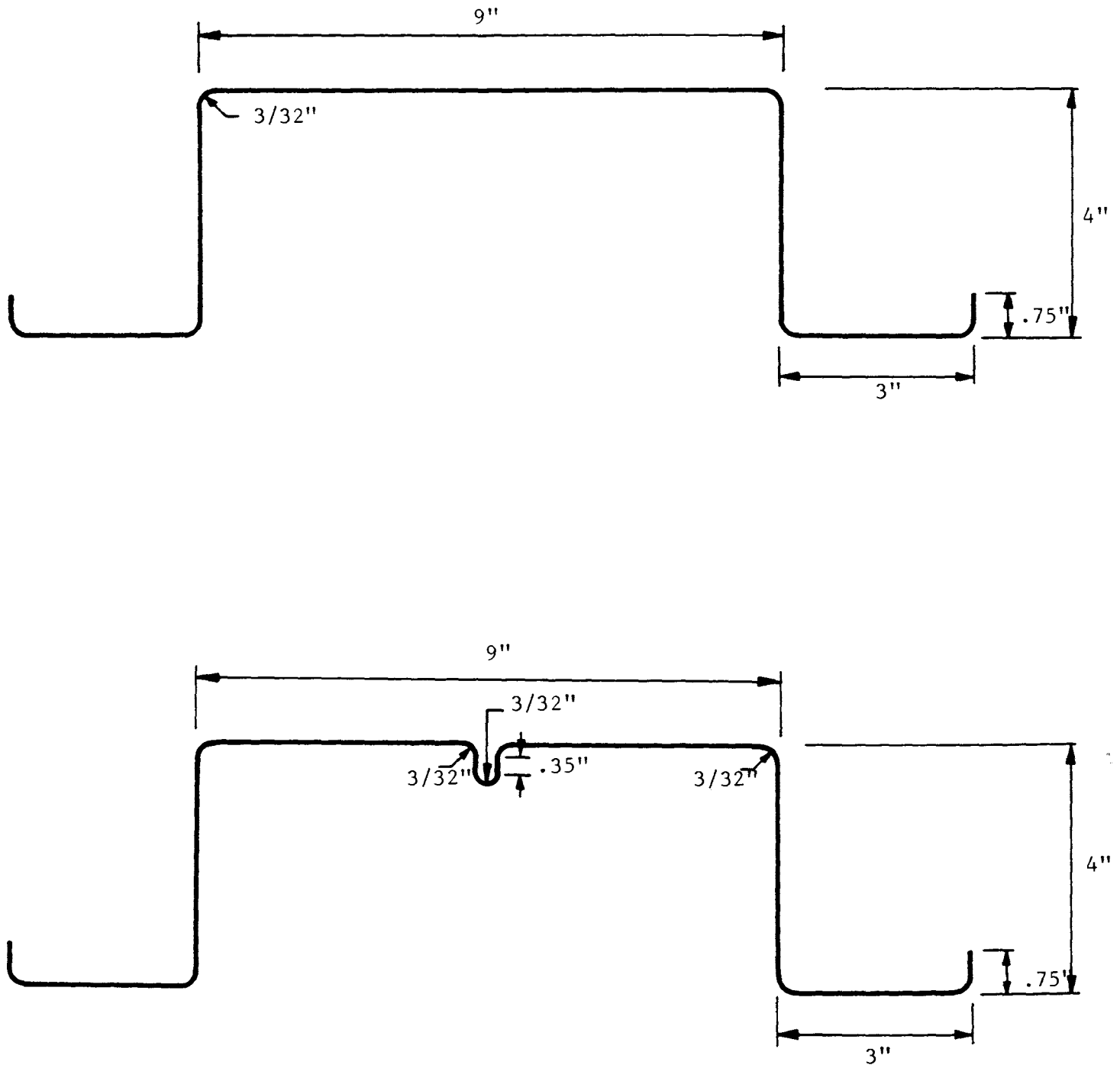


Fig. 2.5-1 Sections for the Numerical Examples 4 and 5 of the 1980 AISI Cold-Formed Steel Design Manual

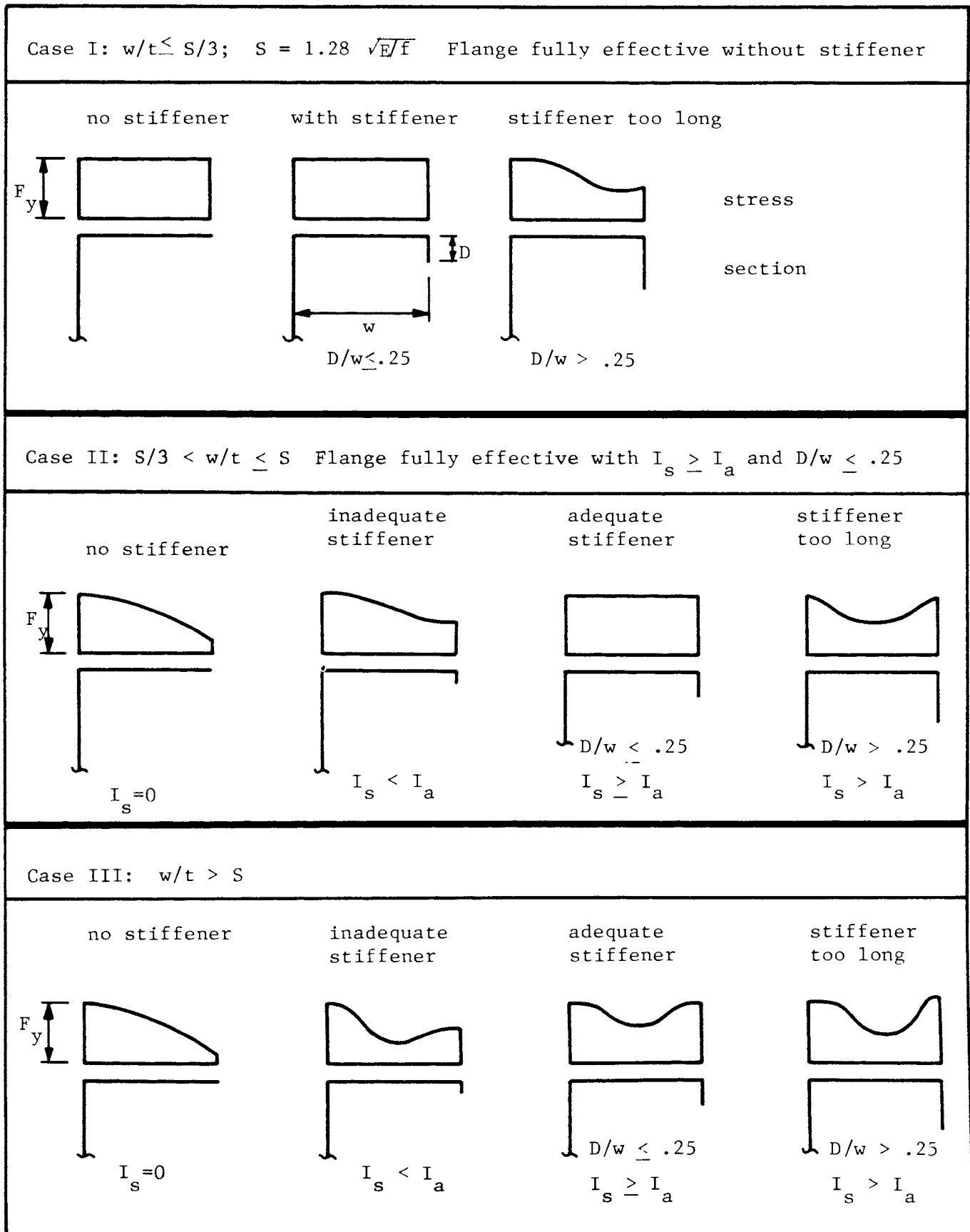


Fig. 2.5-2. Edge stiffener design criteria

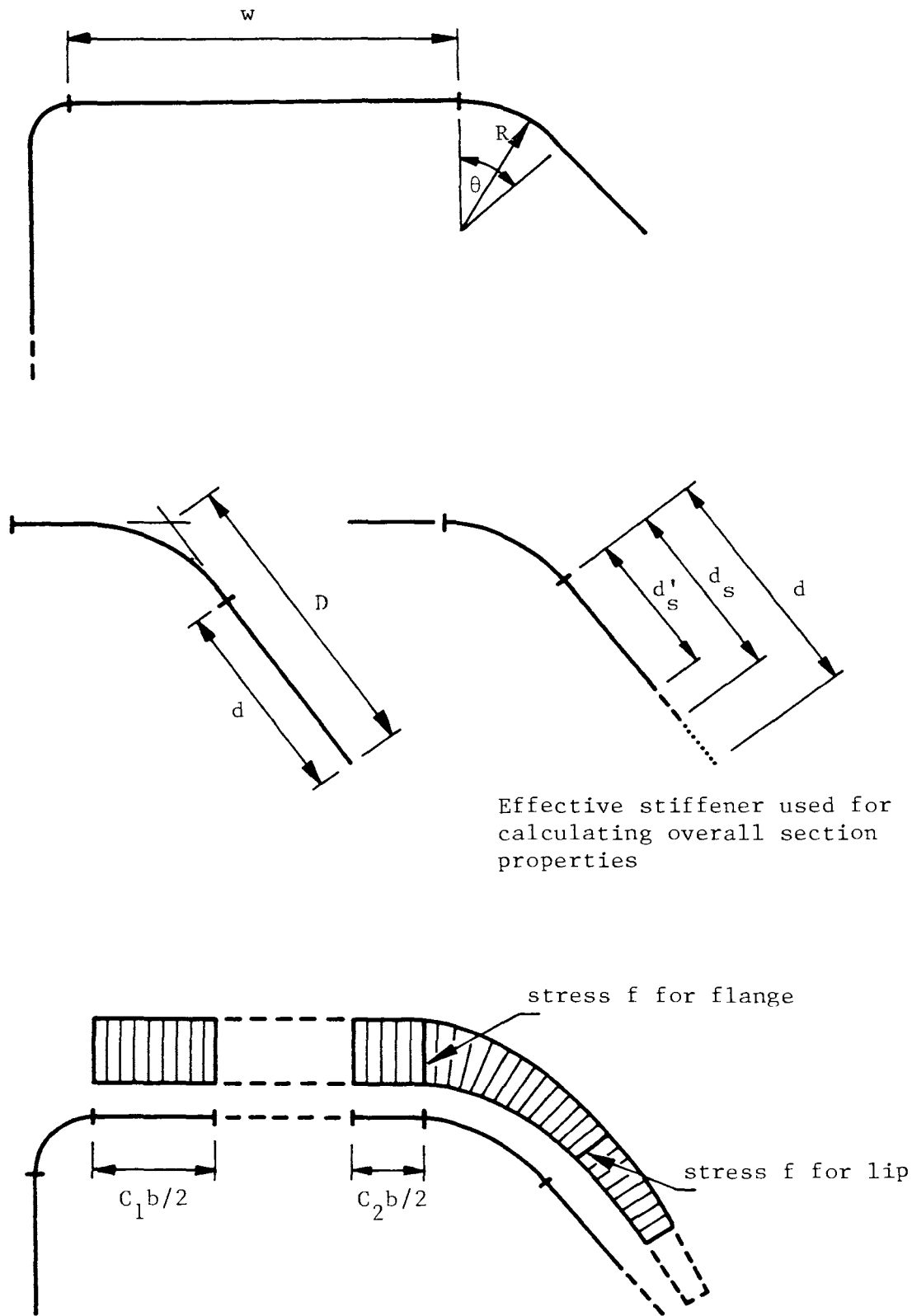


Fig. 2.5-3 Compression element with edge stiffener

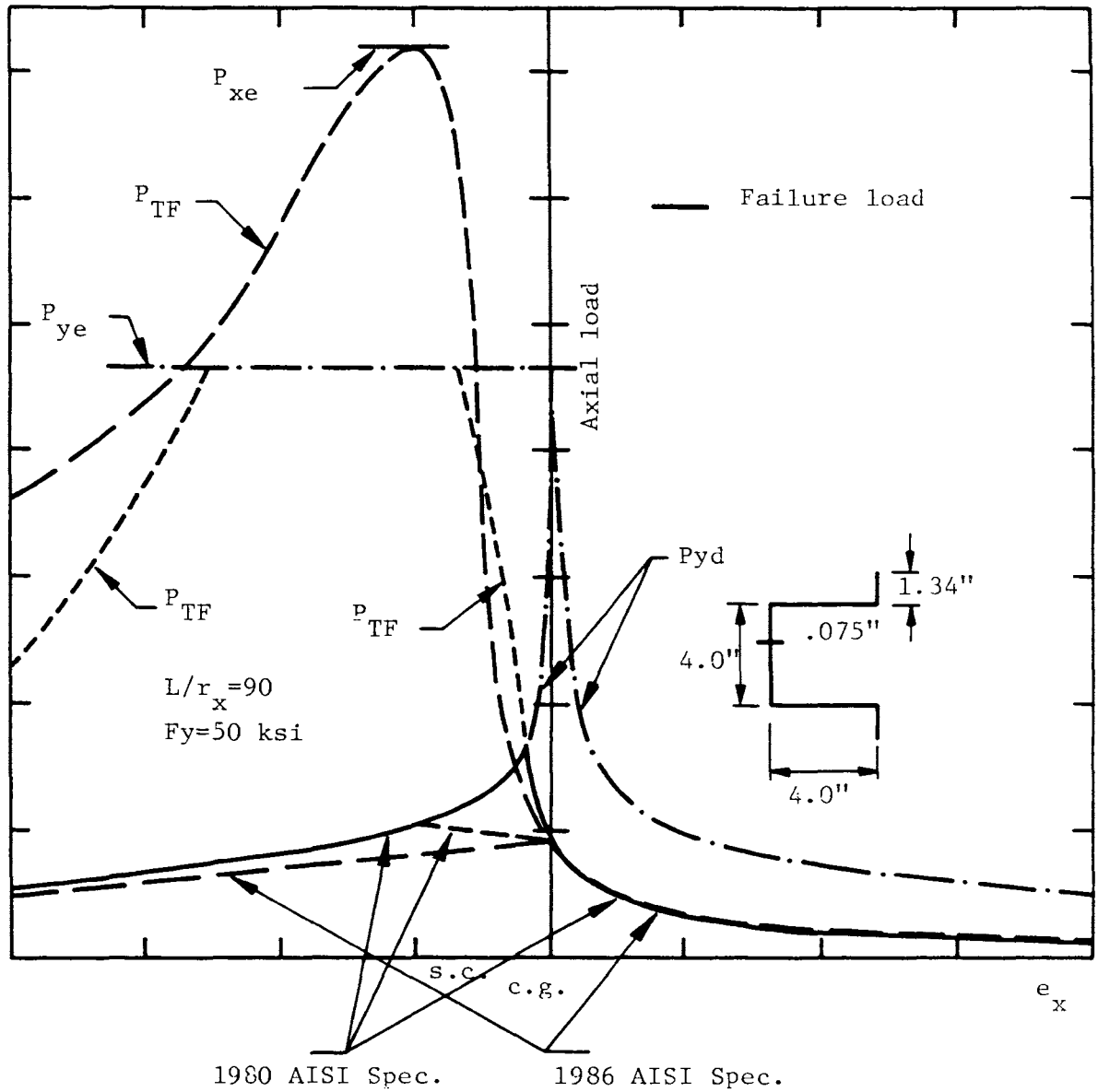


Fig. 3.3-1 Torsional-flexural behavior for axial loads along the axis of symmetry

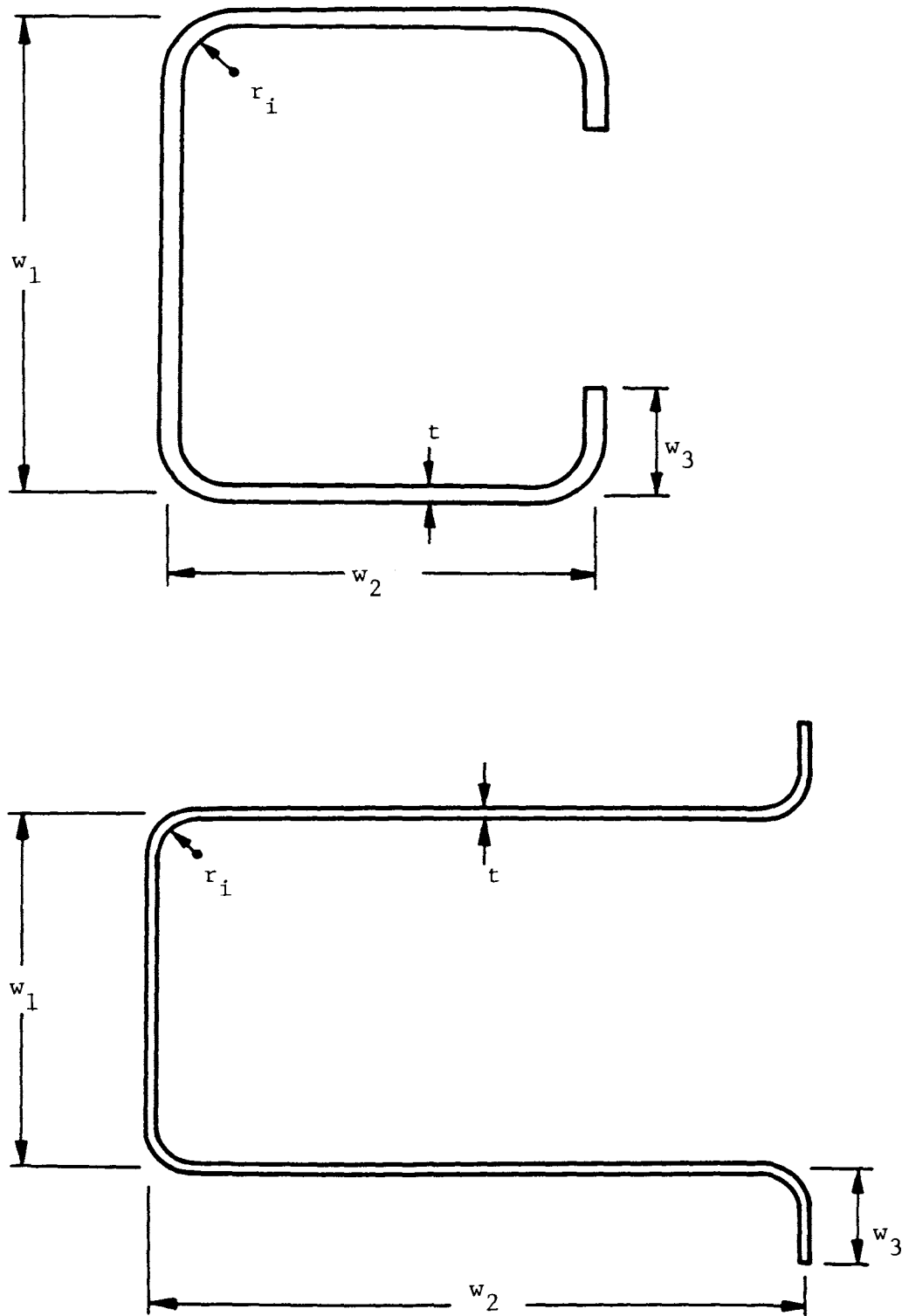


Fig. 3.3-2 Cross sectional geometry

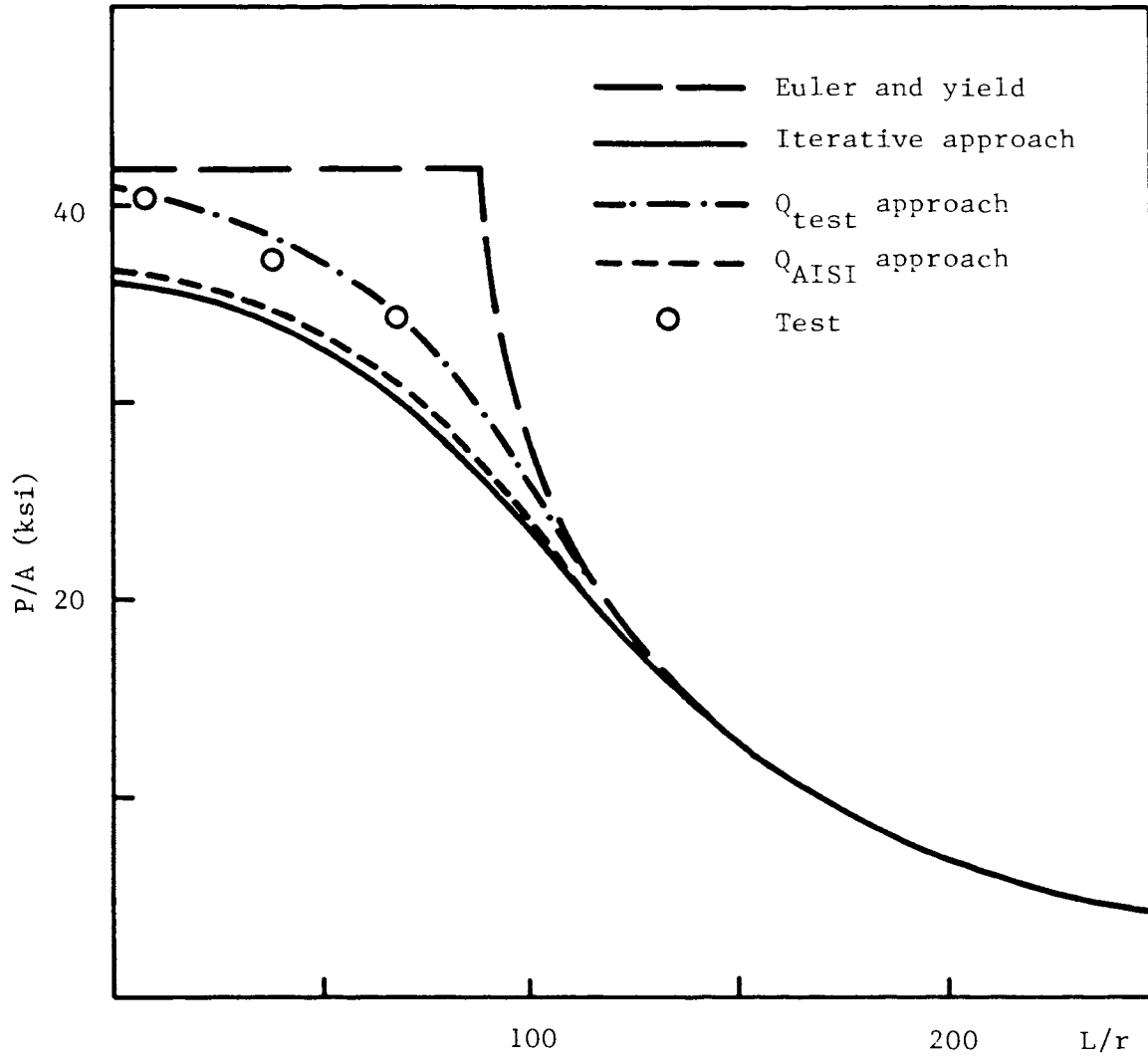


Fig. 4.2-1 Correlation of observed and calculated failure loads for Section S-1 with $w/t = 57.2$

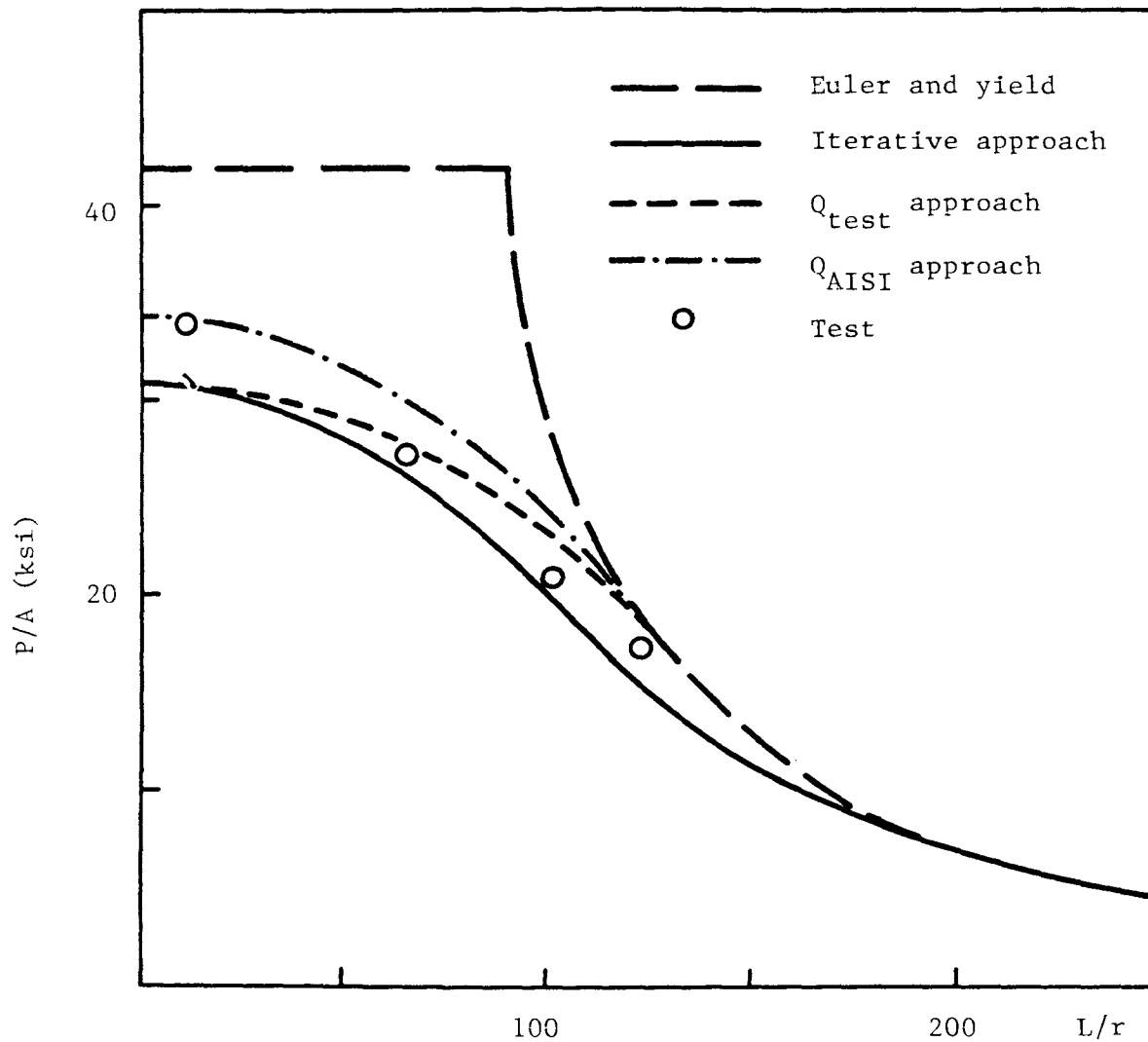


Fig. 4.2-2 Correlation of observed and calculated failure loads for Section S-2 with $w/t = 83.0$

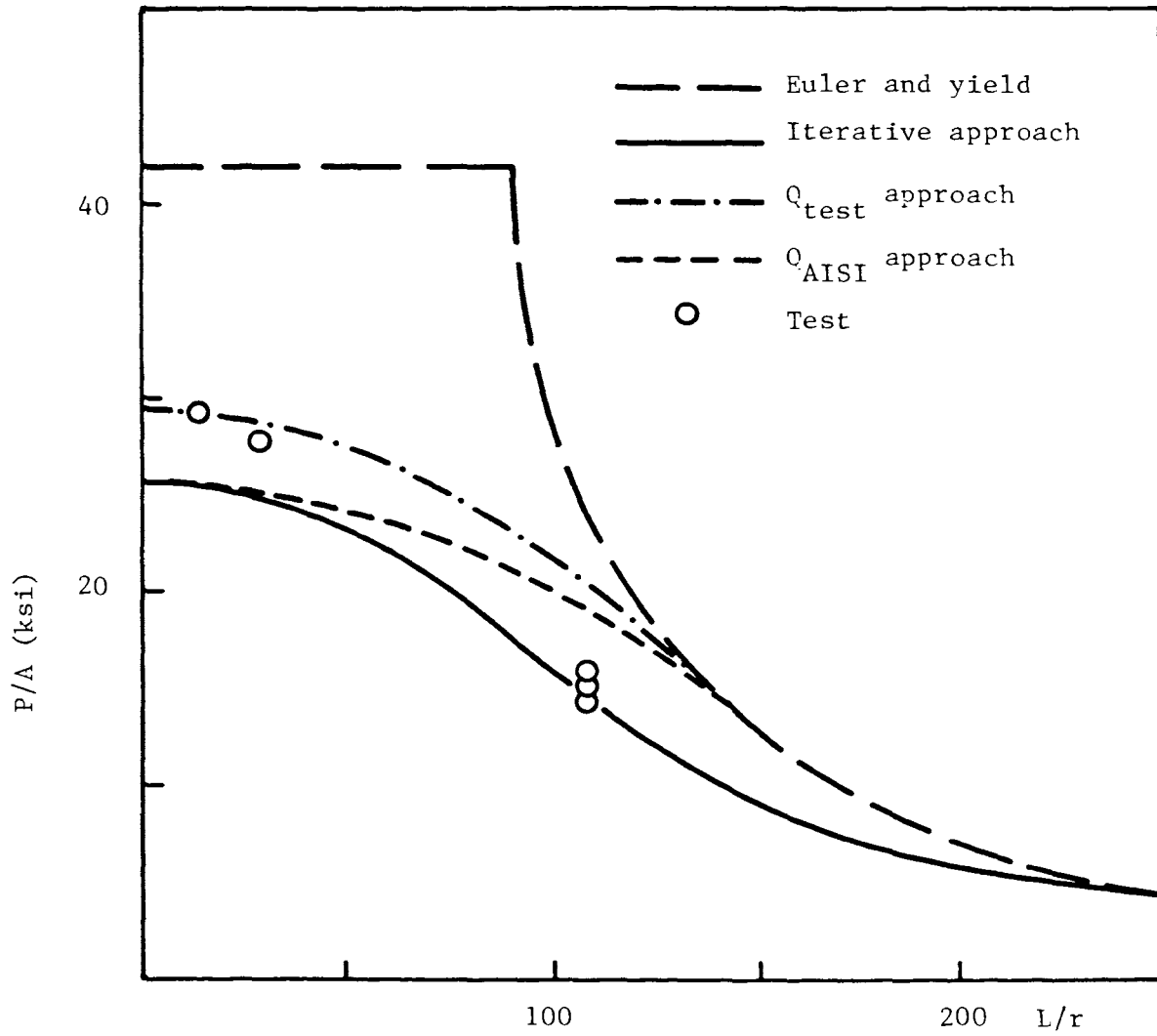


Fig. 4.2-3 Correlation of observed and calculated failure loads for Section S-3 with $w/t = 117.4$

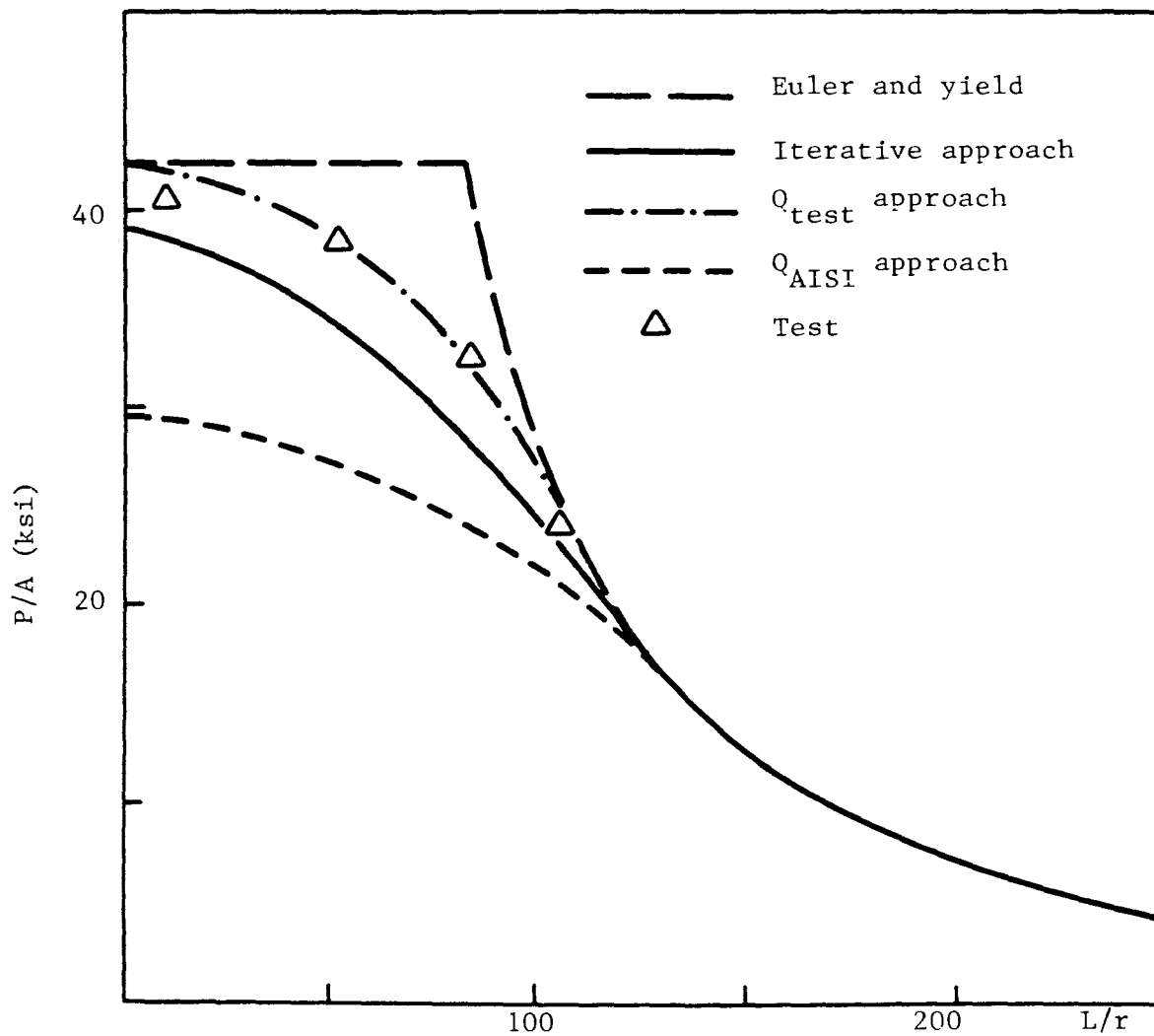


Fig. 4.2-6 Correlation of observed and calculated failure loads for
Section U-2 with $w/t = 20.5$

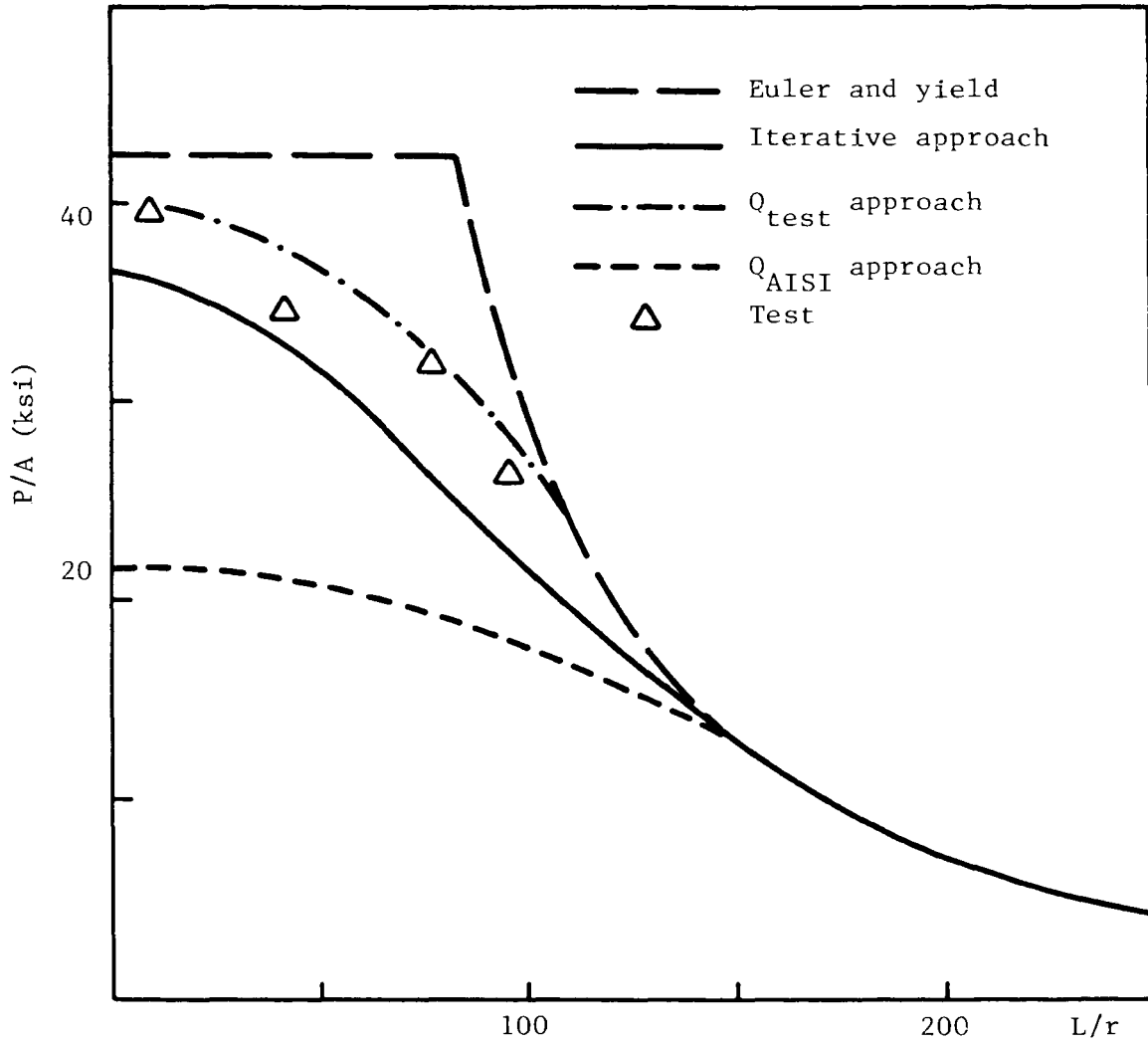


Fig. 4.2-7 Correlation of observed and calculated failure loads for
Section U-3 with $w/t = 24.8$

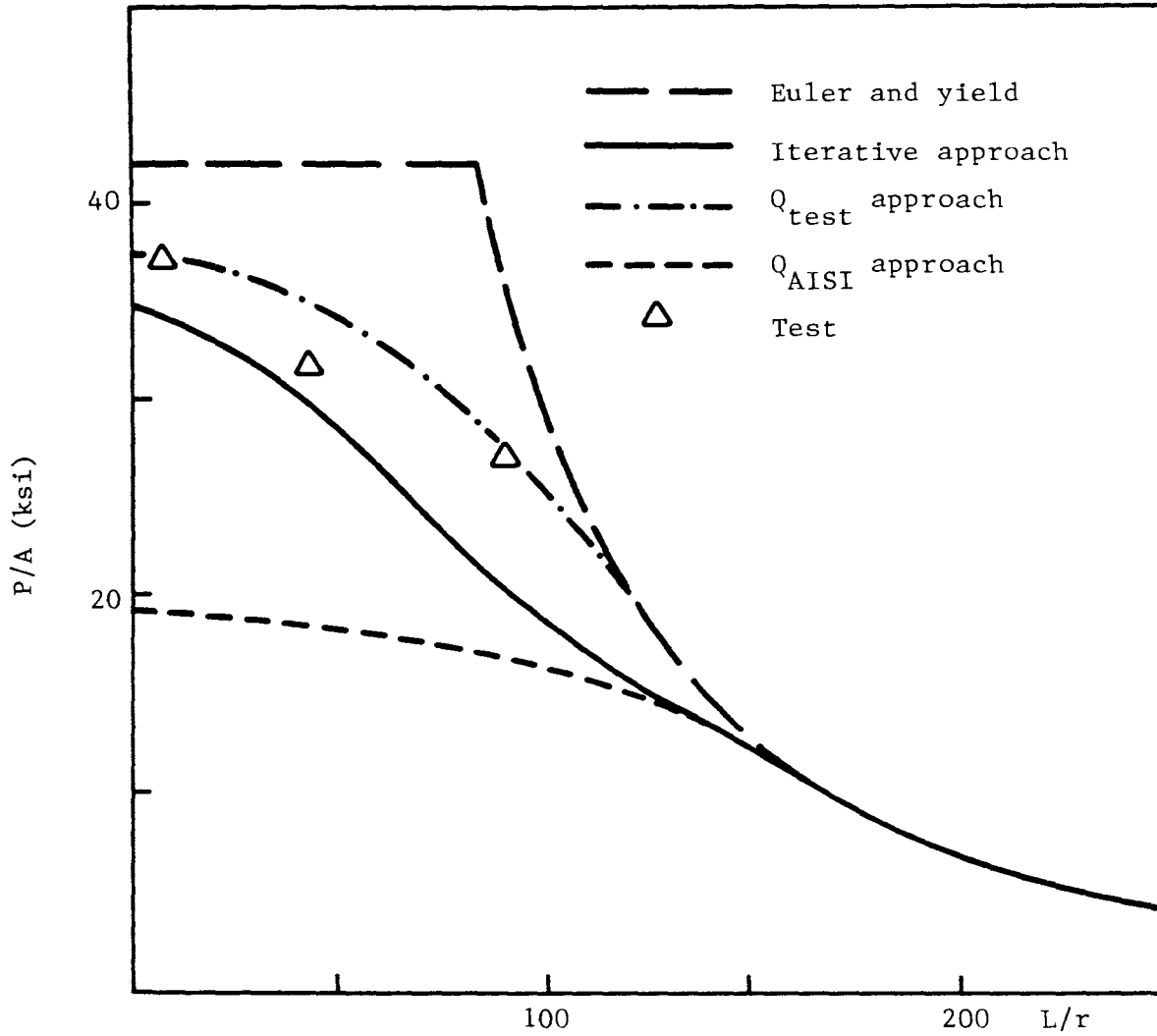


Fig. 4.2-8 Correlation of observed and calculated failure loads for Section U-4 with $w/t = 29.1$

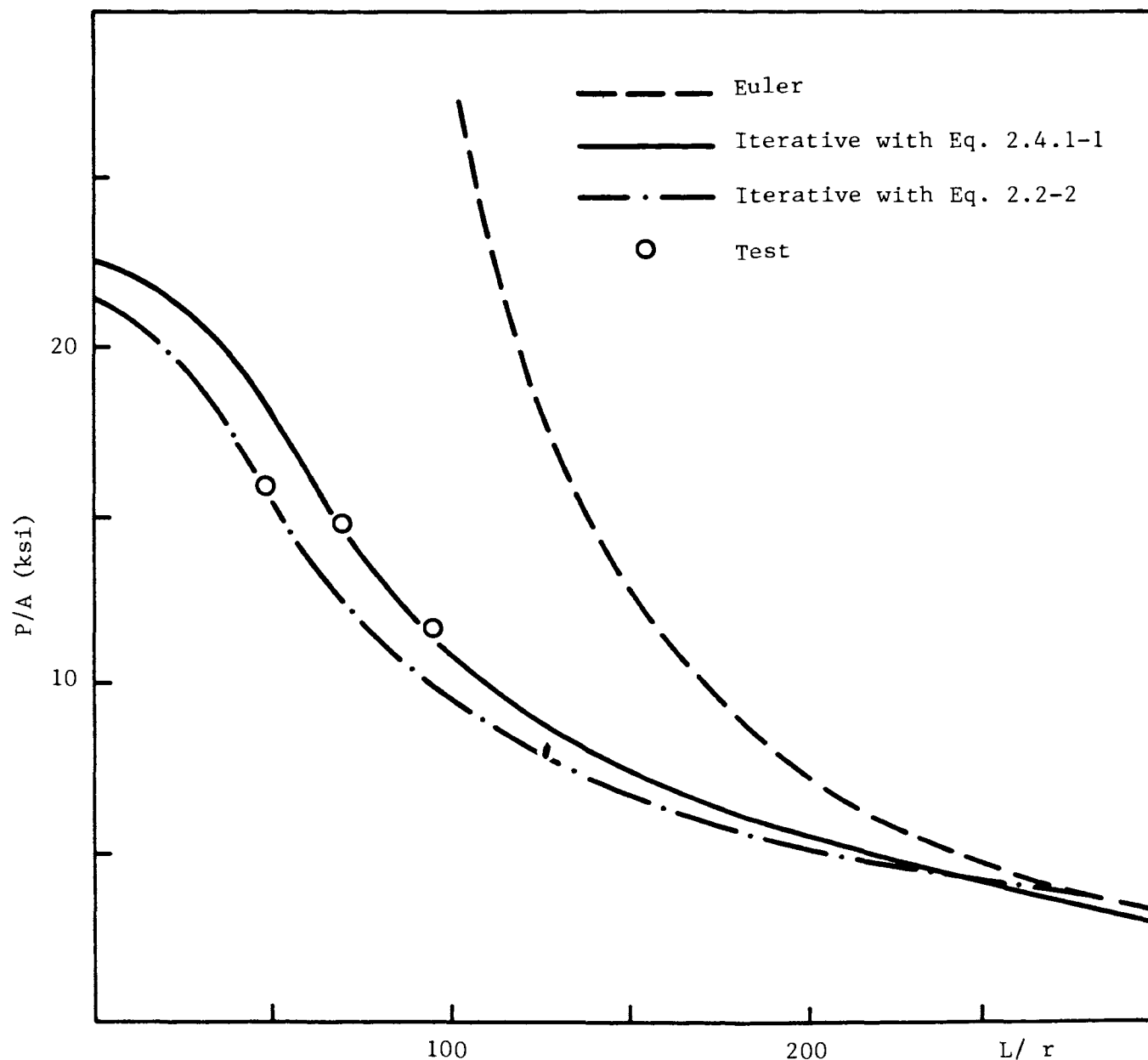


Fig. 4.2-9 Correlation of calculated and observed failure loads for Section LC-1 with $w/t = 57.5$

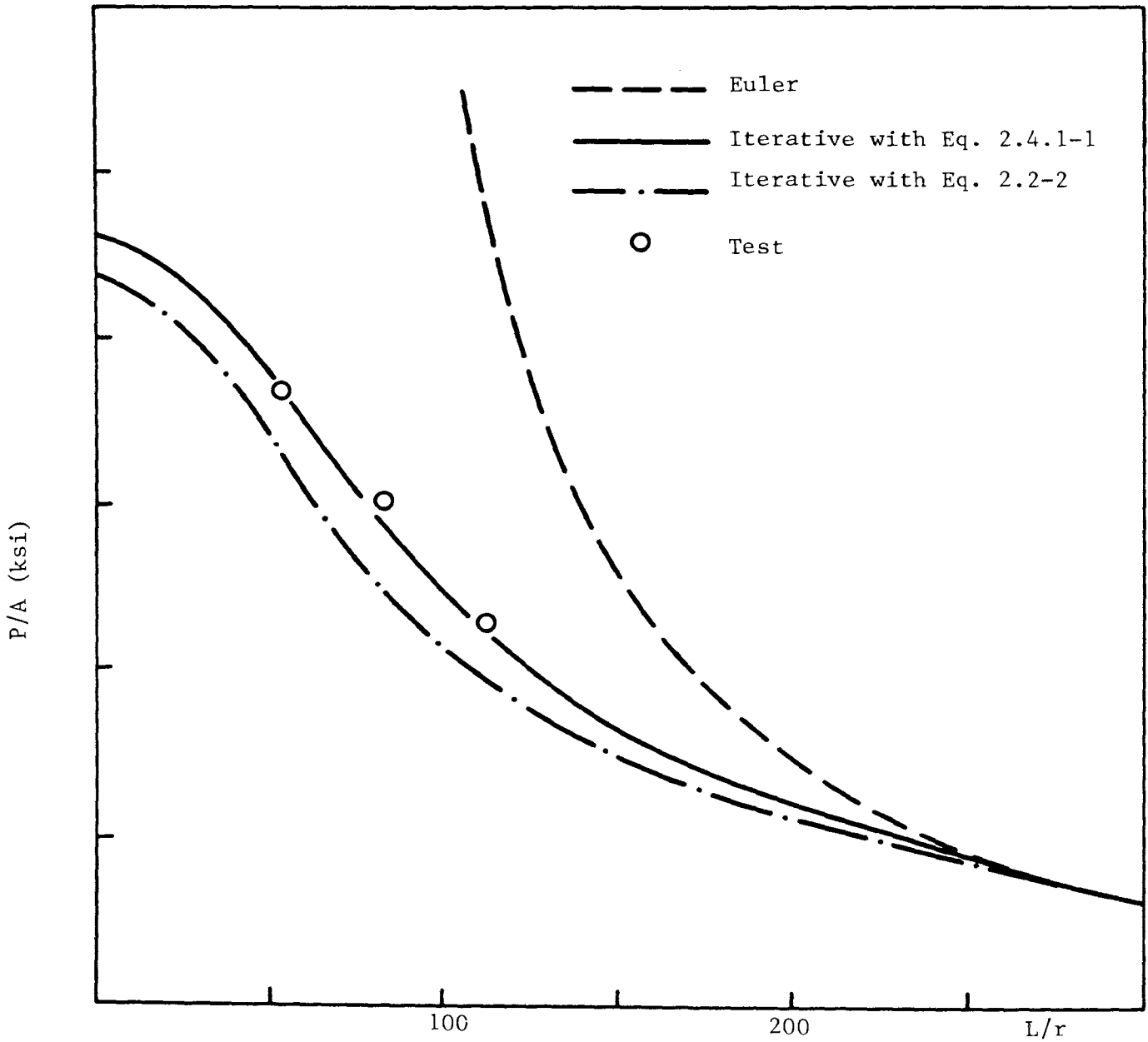


Fig. 4.2-10 Correlation of calculated and observed failure loads for Section LC-II with $w/t = 50.5$

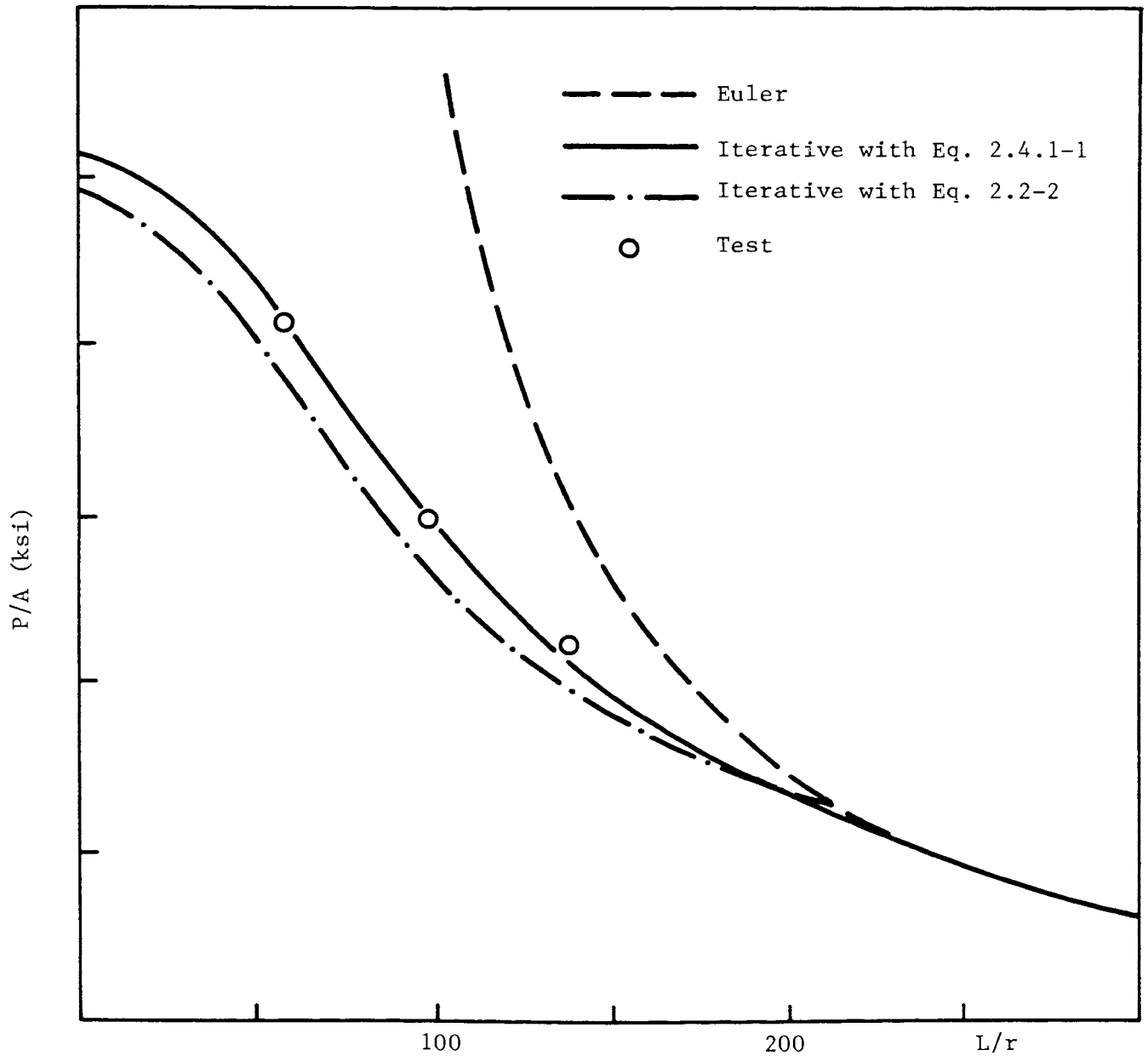


Fig. 4.2-11 Correlation of calculated and observed failure loads for Section LC-III with $w/t = 42.0$

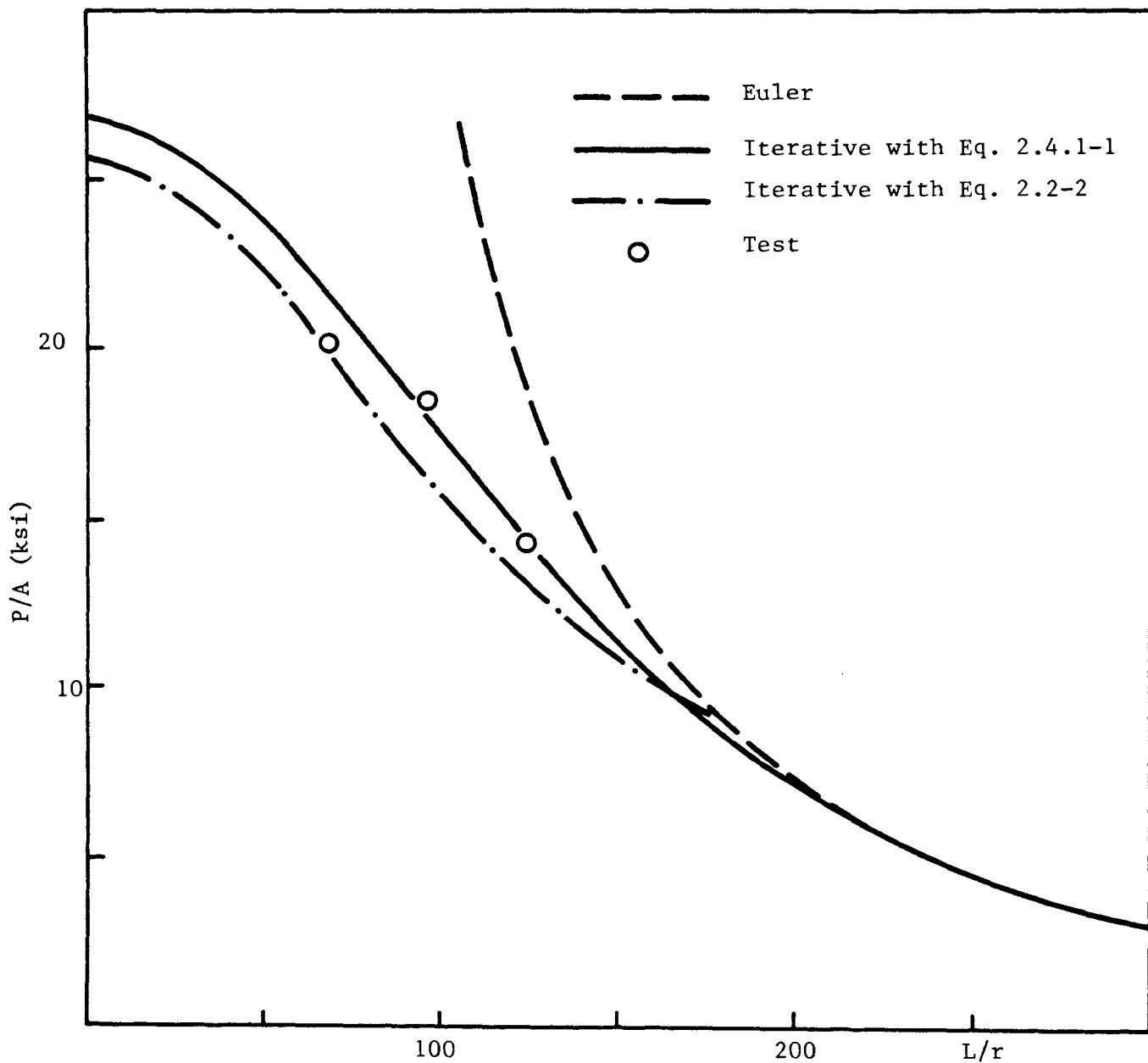


Fig. 4.2-12 Correlation of calculated and observed failure loads for Section LC-IV with $w/t = 35.0$

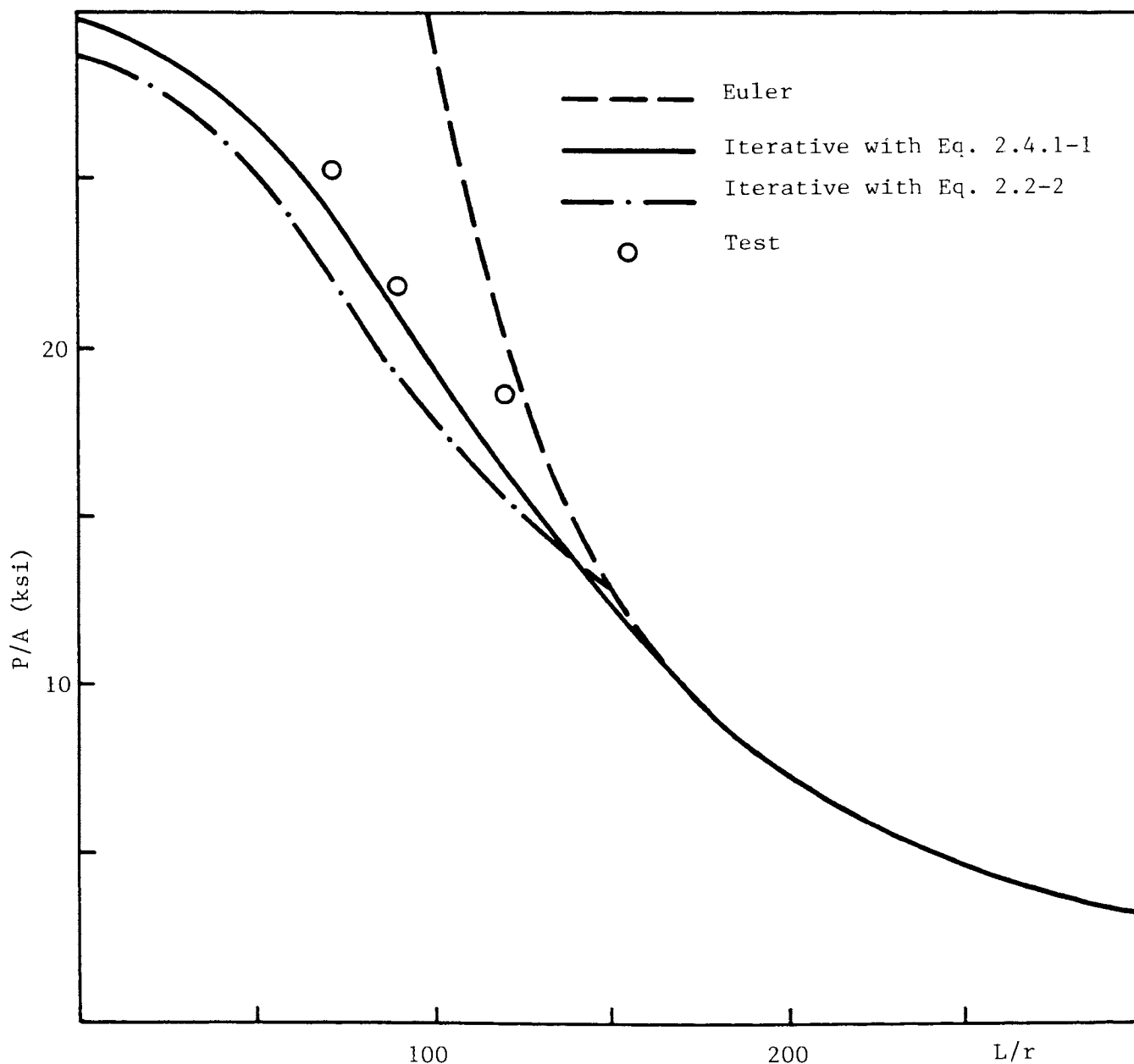


Fig. 4.2-13 Correlation of calculated and observed failure loads for Section LC-V with $w/t = 29.5$

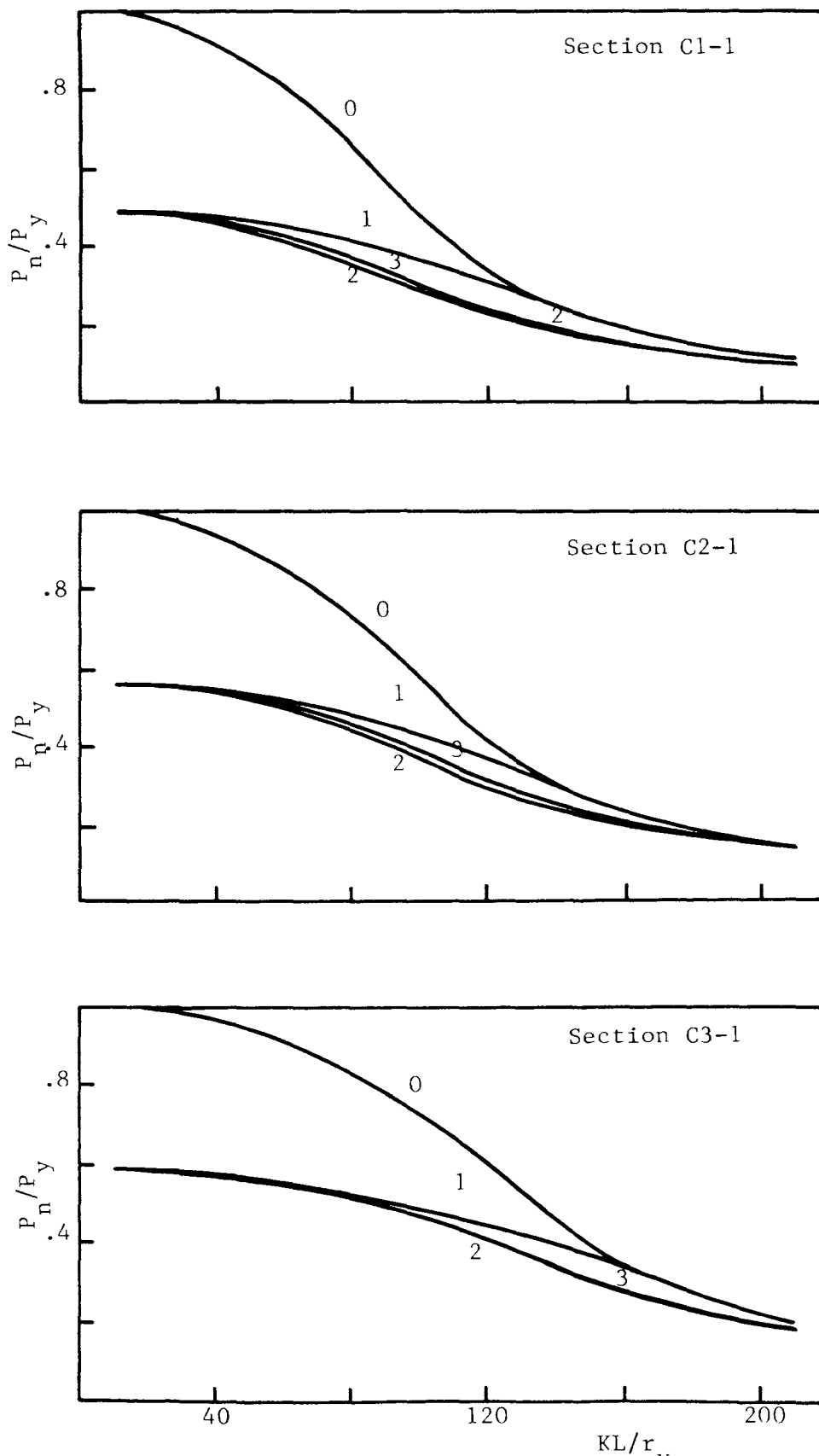


Fig. 4.3-1 C-Section column curves - flexural buckling

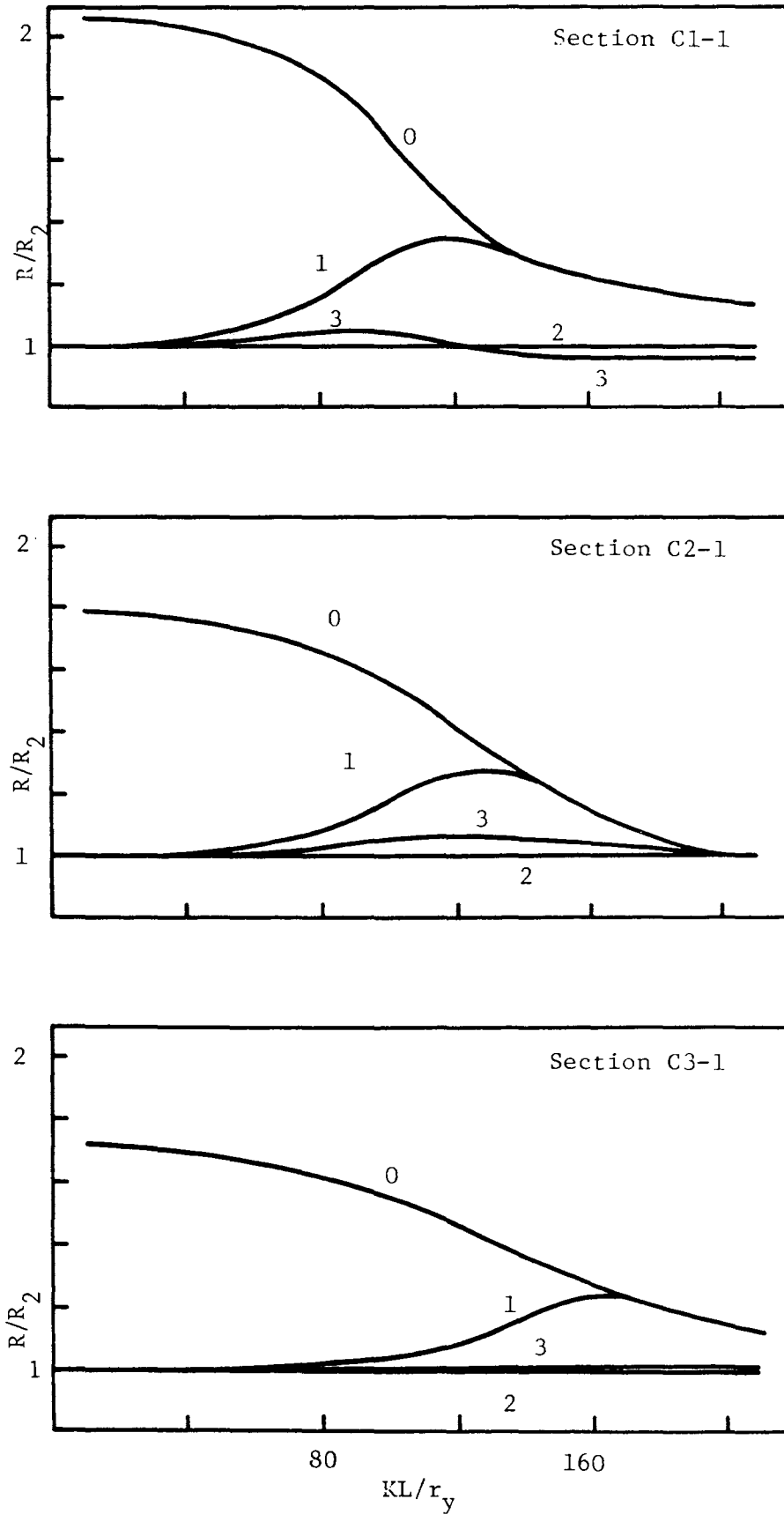


Fig. 4.3-2 C-section column curve comparisons - flexural buckling

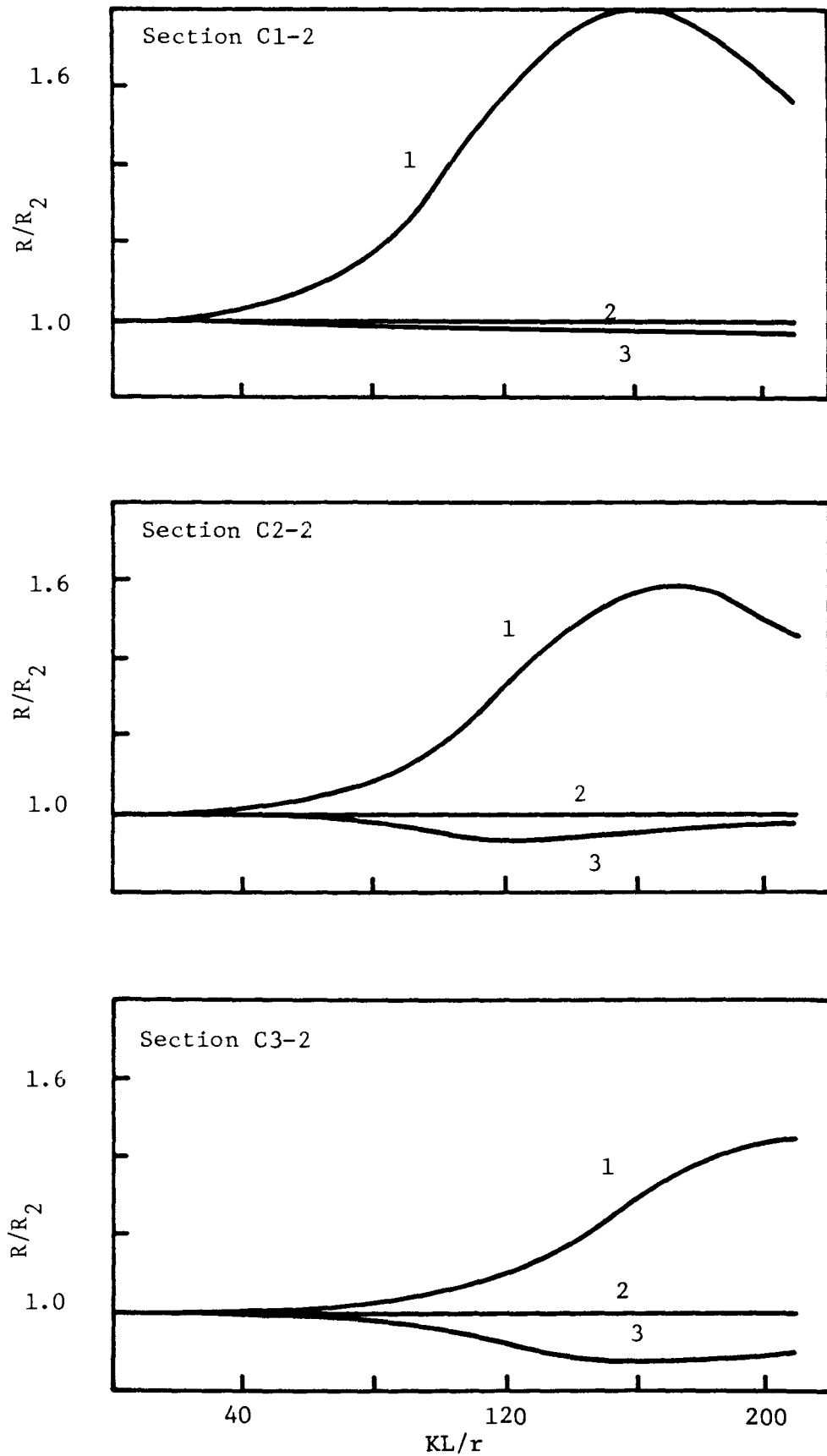


Fig. 4.3-3 C-section column curve comparisons - flexural buckling

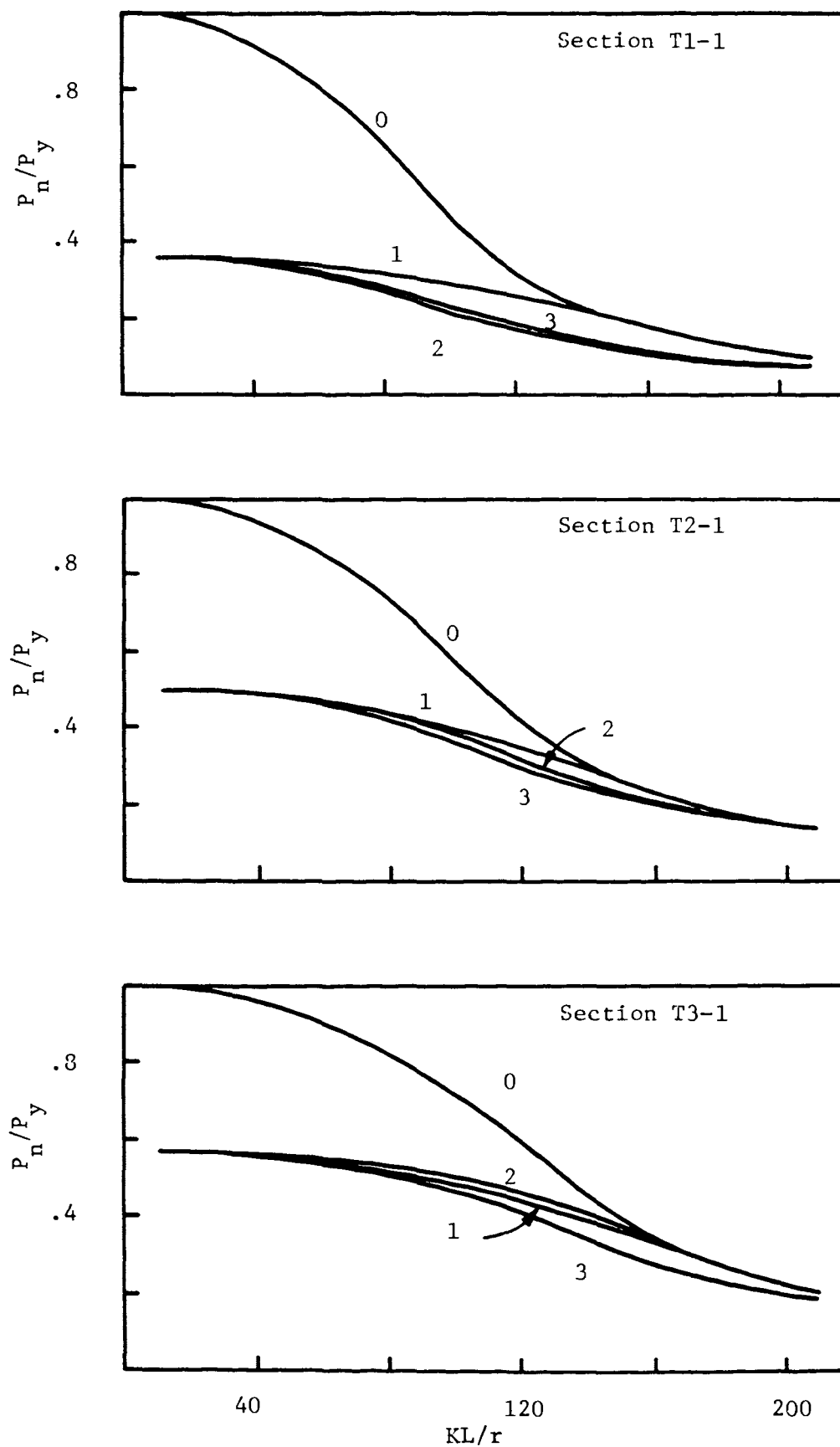


Fig. 4.3-4 Tube column curves - flexural buckling

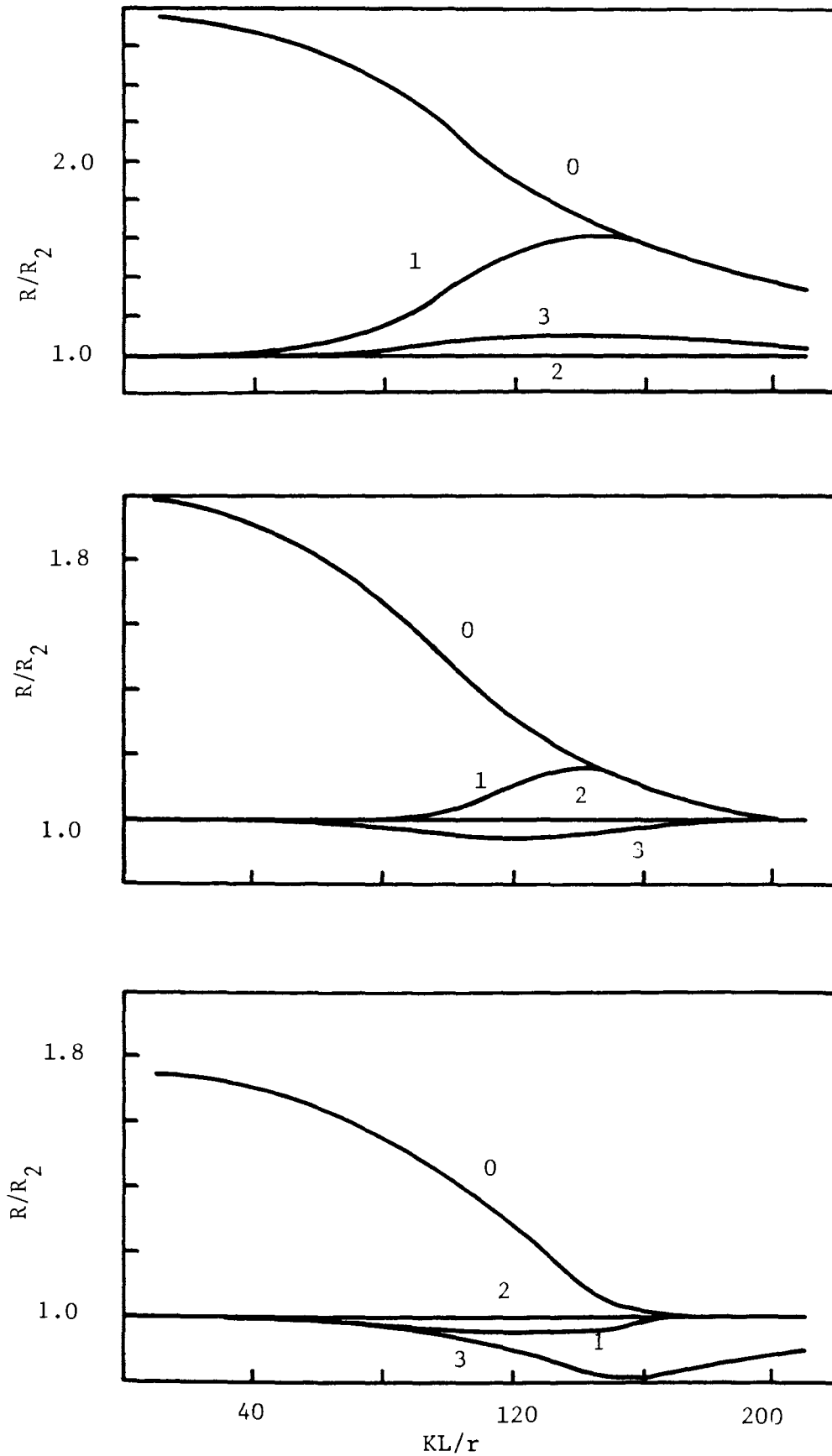


Fig. 4.3-5 Tube column curve comparisons

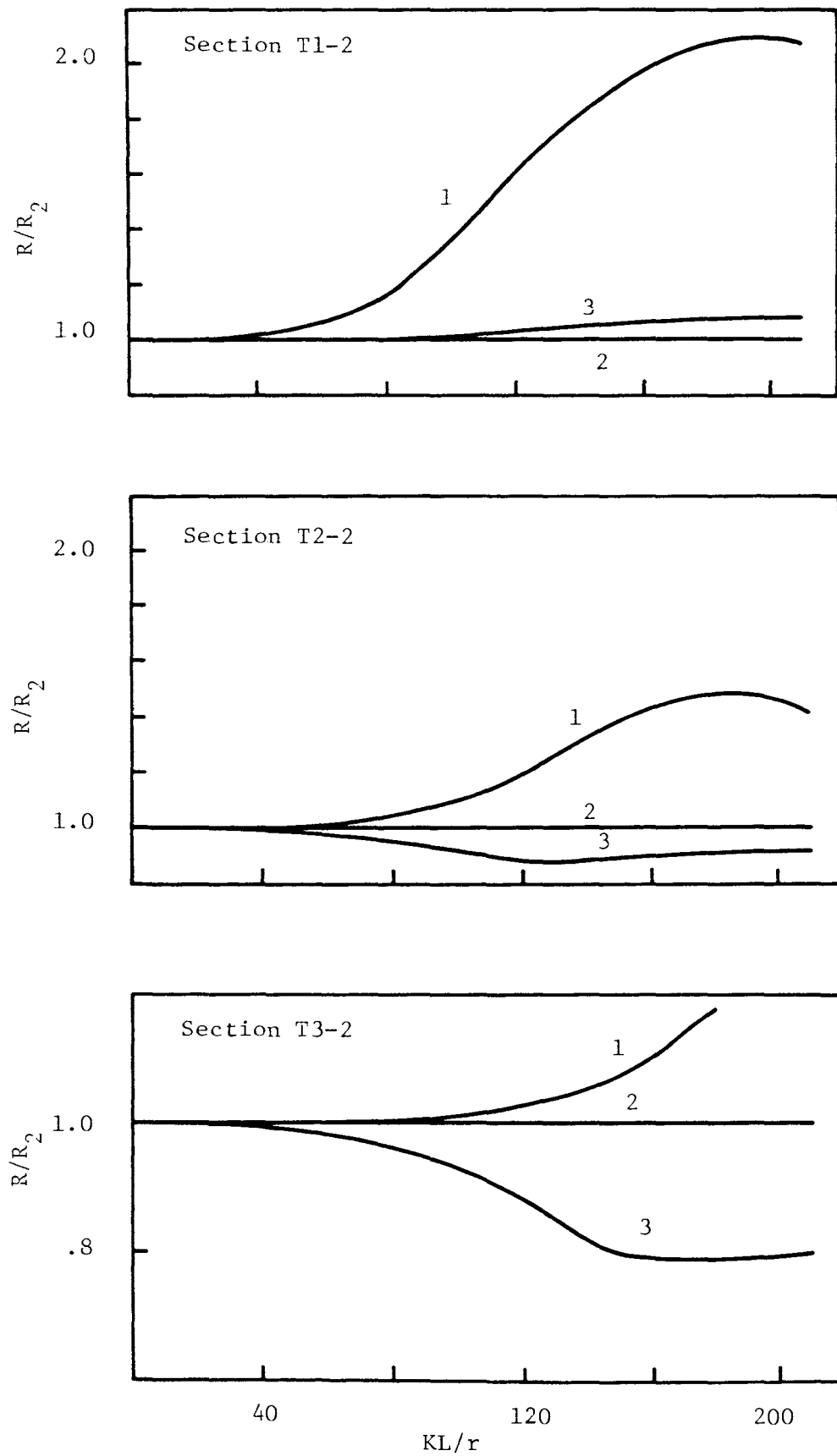


Fig. 4.3-6 Tube column curve comparisons

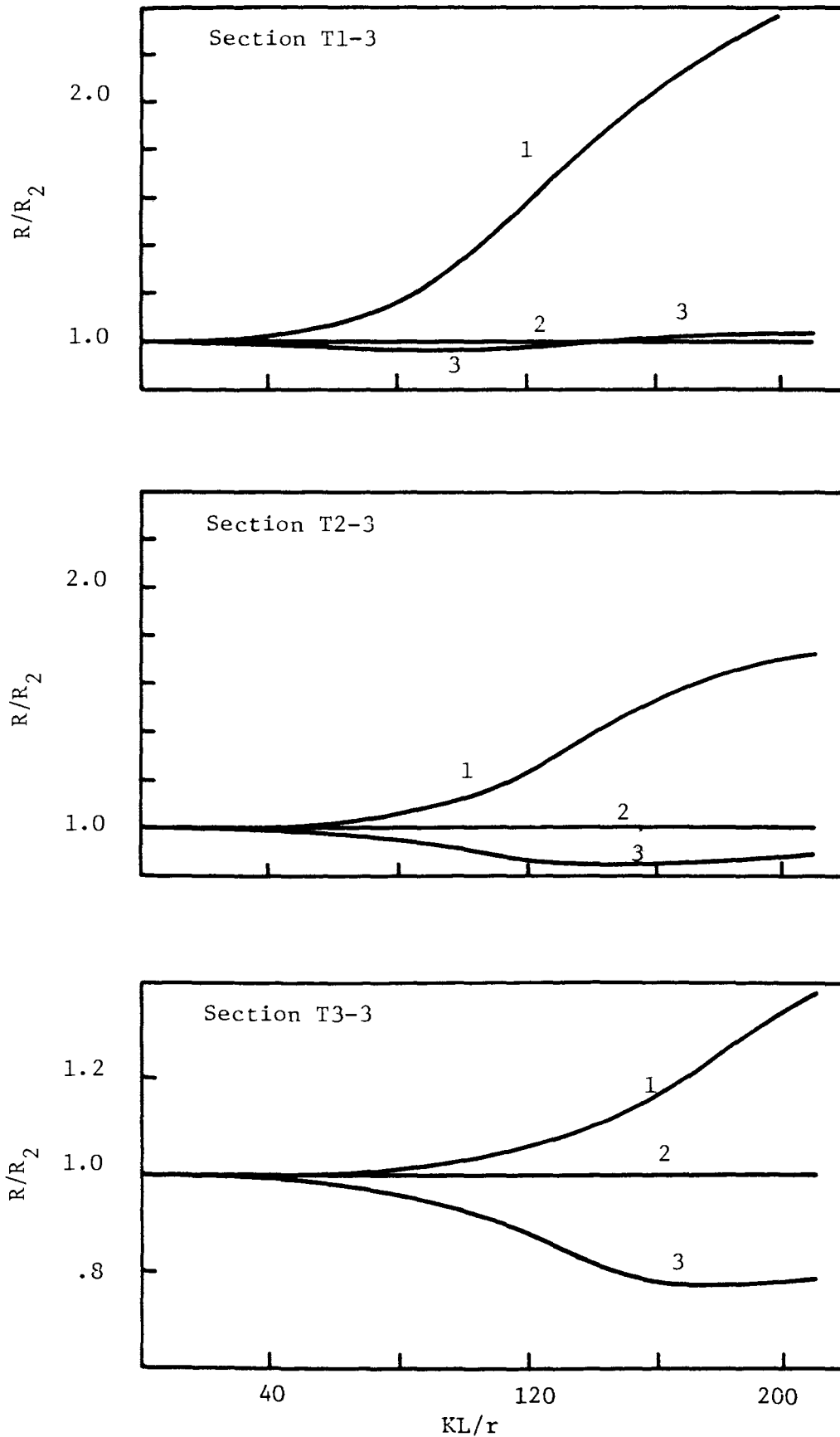


Fig. 4.3-7 Tube column curve comparisons

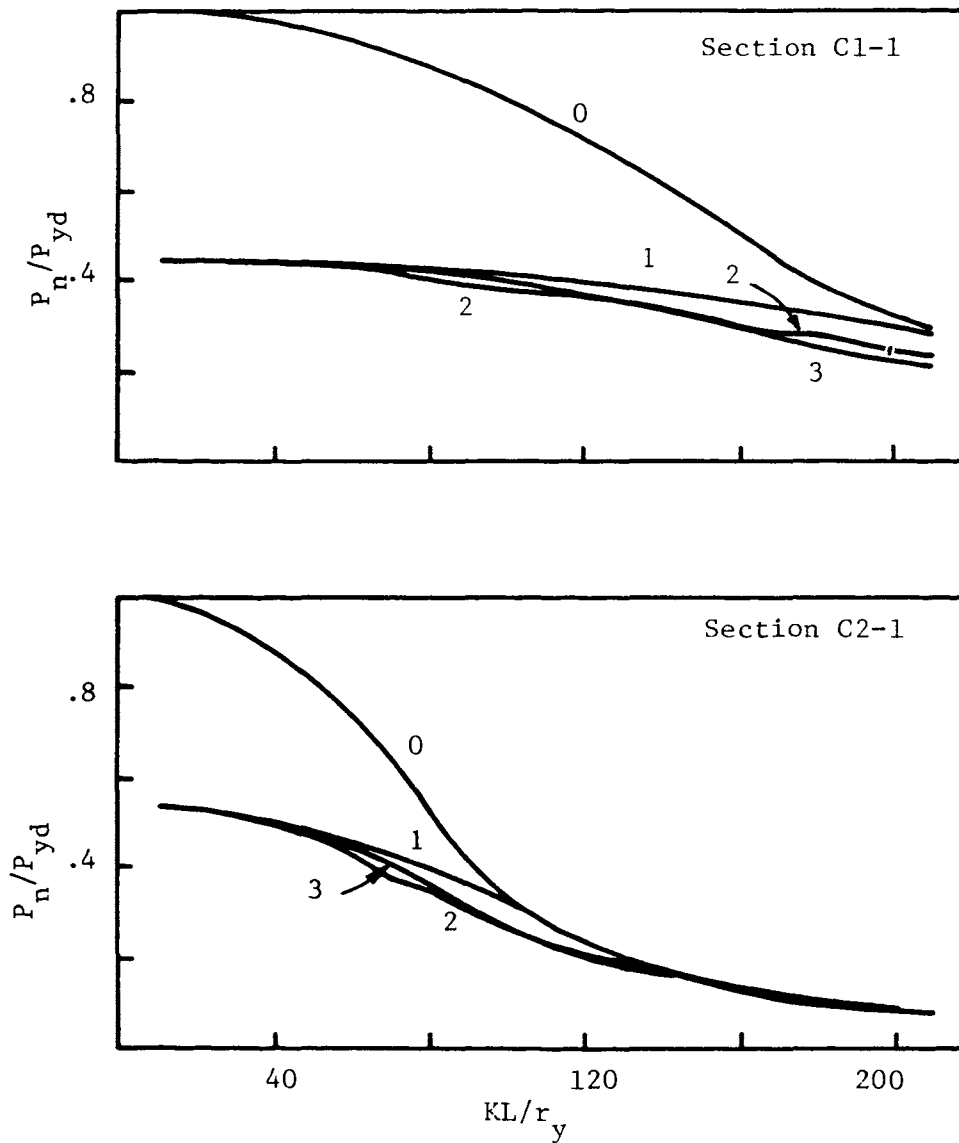


Fig. 5.2-1a C-section column curves - torsional-flexural buckling

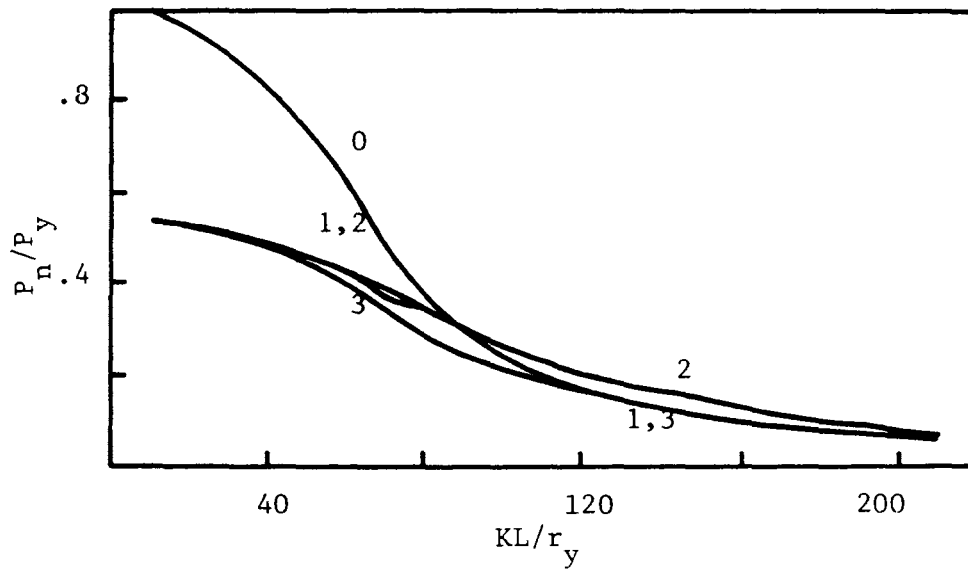


Fig. 5.2-1 b Hat section column curves - torsional-flexural buckling

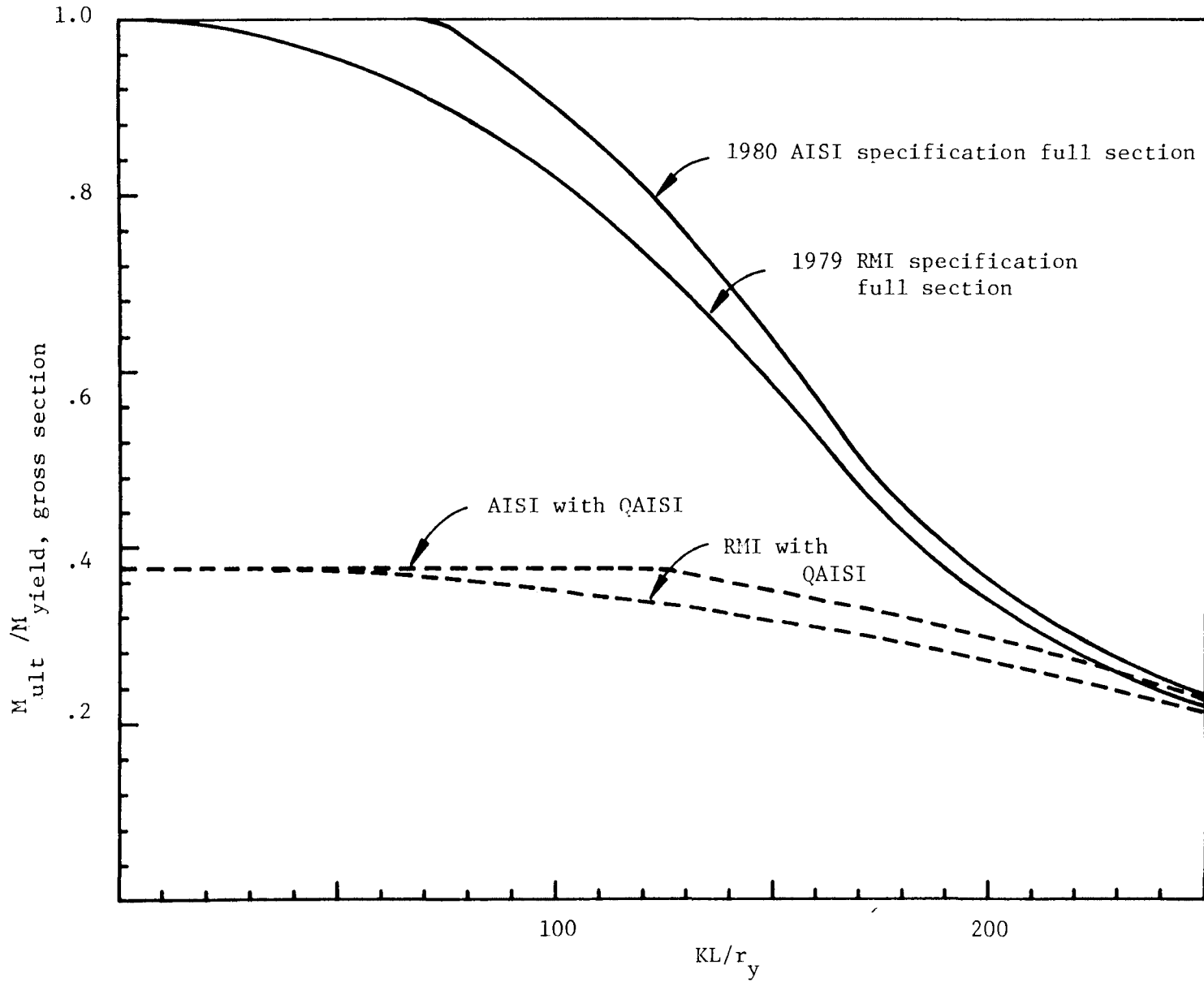


Fig. 6.2-1 Lateral buckling moment about the symmetry axis - Section C1-1

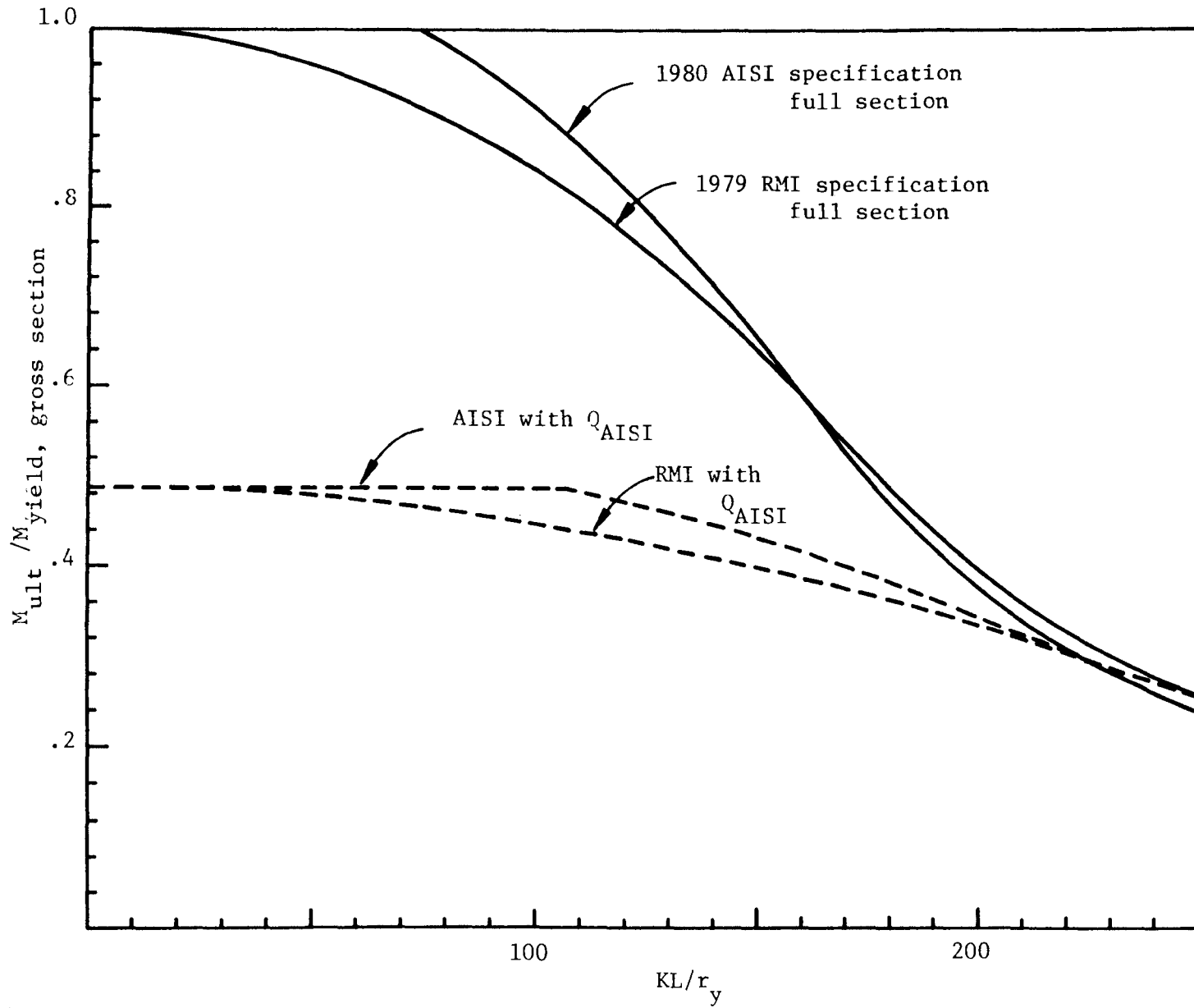


Fig. 6.2-2 Lateral buckling moment about the symmetry axis - Section C2-1

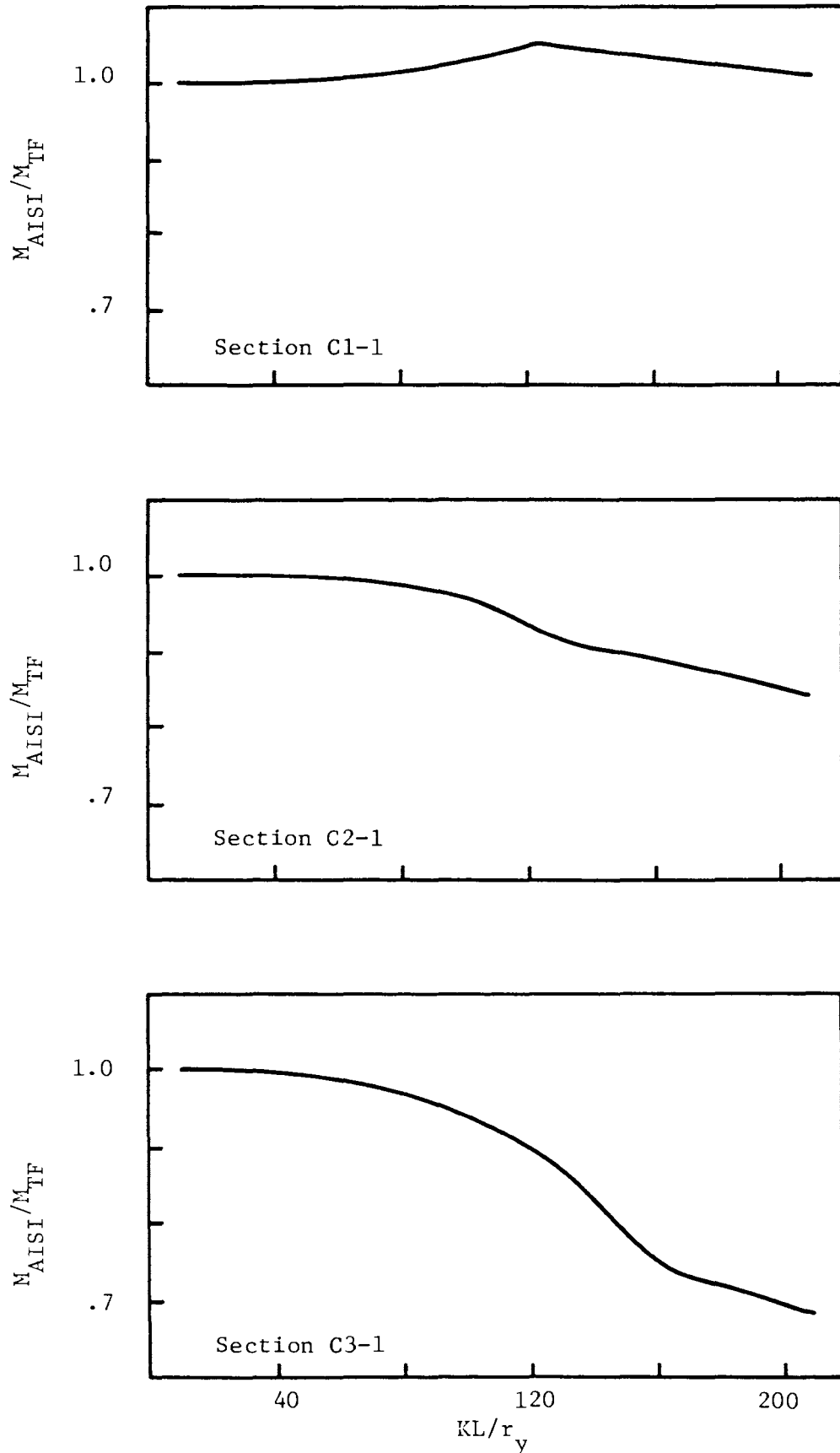


Fig. 6.2-3 Ratio of the lateral buckling moment according to the 1980 AISI specification to that according to the torsional-flexural buckling theory - C-Sections

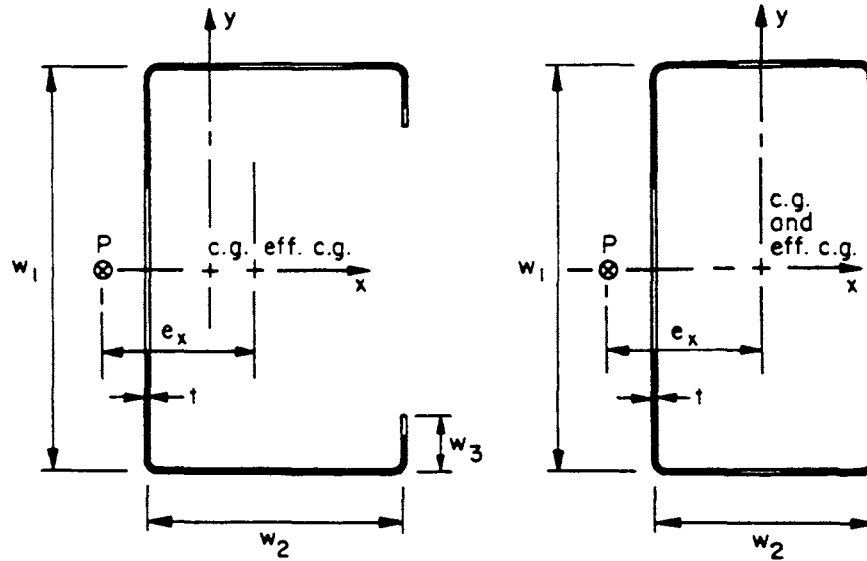


Fig. 7.1-1 Section geometry and generalized effective section

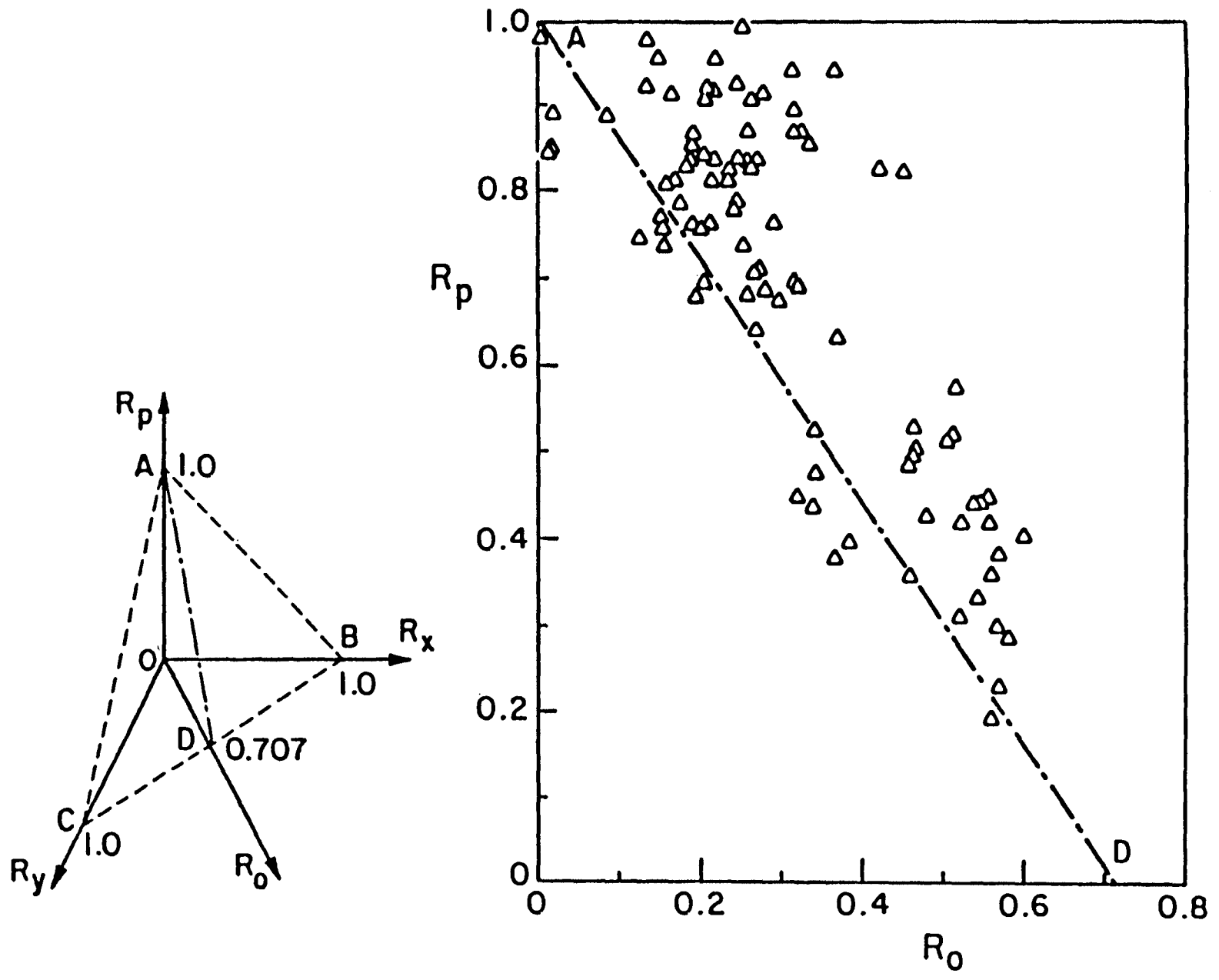


Fig. 7.3-1 Correlation of test results with the unified approach for beam columns

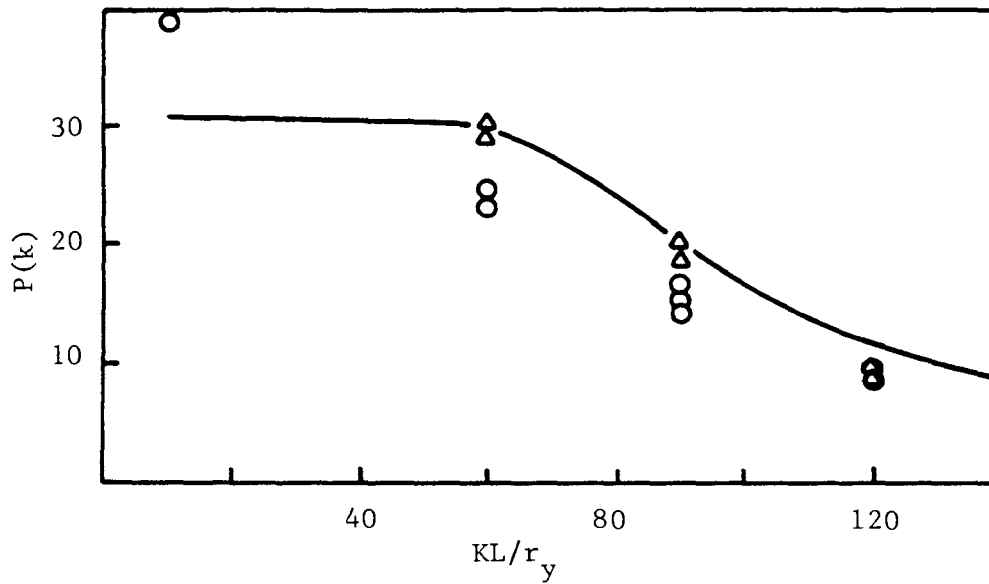


Fig. 8.3-1a Section 1 - Unified approach with $e_x = 0$
 ○ Columns deflecting towards the corner of the section
 △ Columns deflecting towards the tips of flanges

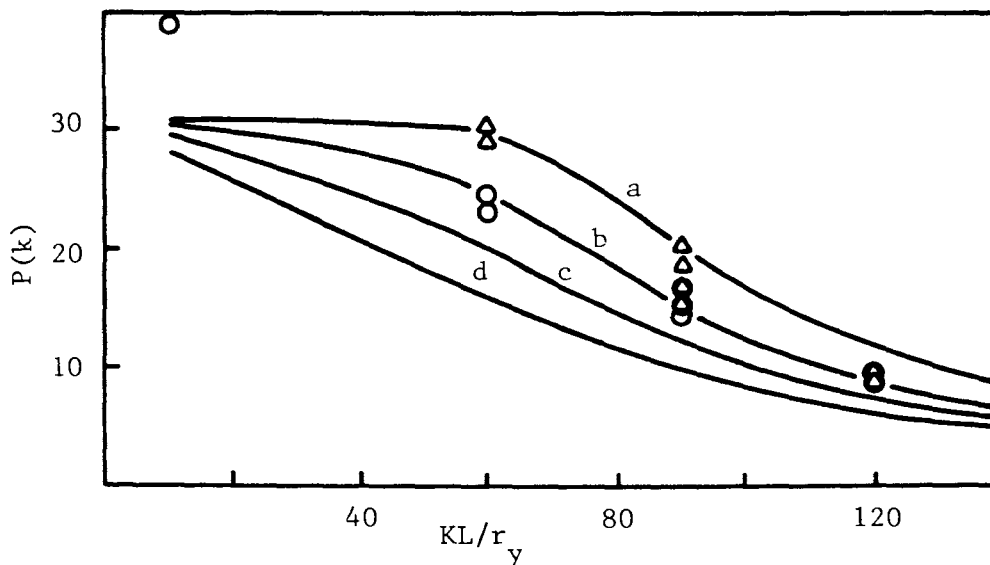


Fig. 8.3-1b Section 1 - Unified approach
 Curve a: $e_x = 0$
 Curve b: $e_x = L/1000$
 Curve c: $e_x = L/500$
 Curve d: $e_x = 3L/1000$
 ○ Columns deflecting towards the corner of the section
 △ Columns deflecting towards the tips of flanges

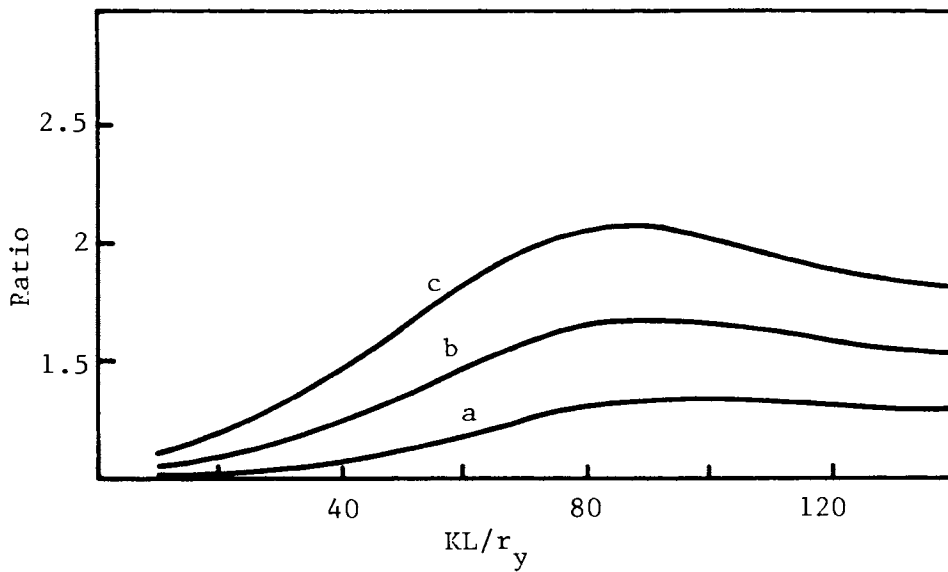


Fig. 8.3-1c Section 1 - Unified approach

Curve a: $(P_{ult} \text{ for } e_x = 0) / (P_{ult} \text{ for } e_x = L/1000)$
 Curve b: $(P_{ult} \text{ for } e_x = 0) / (P_{ult} \text{ for } e_x = L/500)$
 Curve c: $(P_{ult} \text{ for } e_x = 0) / (P_{ult} \text{ for } e_x = 2L/1000)$

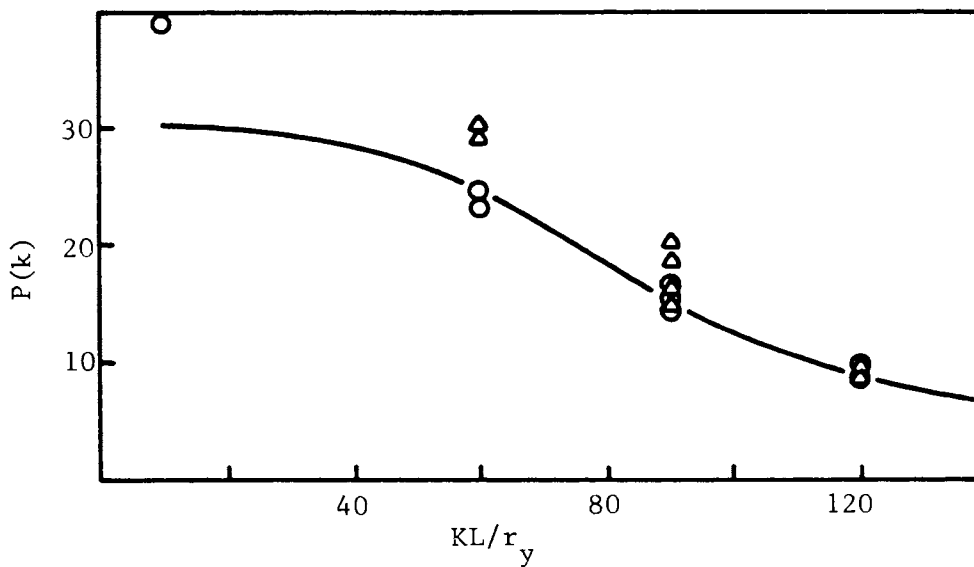


Fig. 8.3-1d Section 1 - Unified approach with $e_x = L/1000$

○ Columns deflecting towards the corner of the section
 △ Columns deflecting towards the tips of flanges

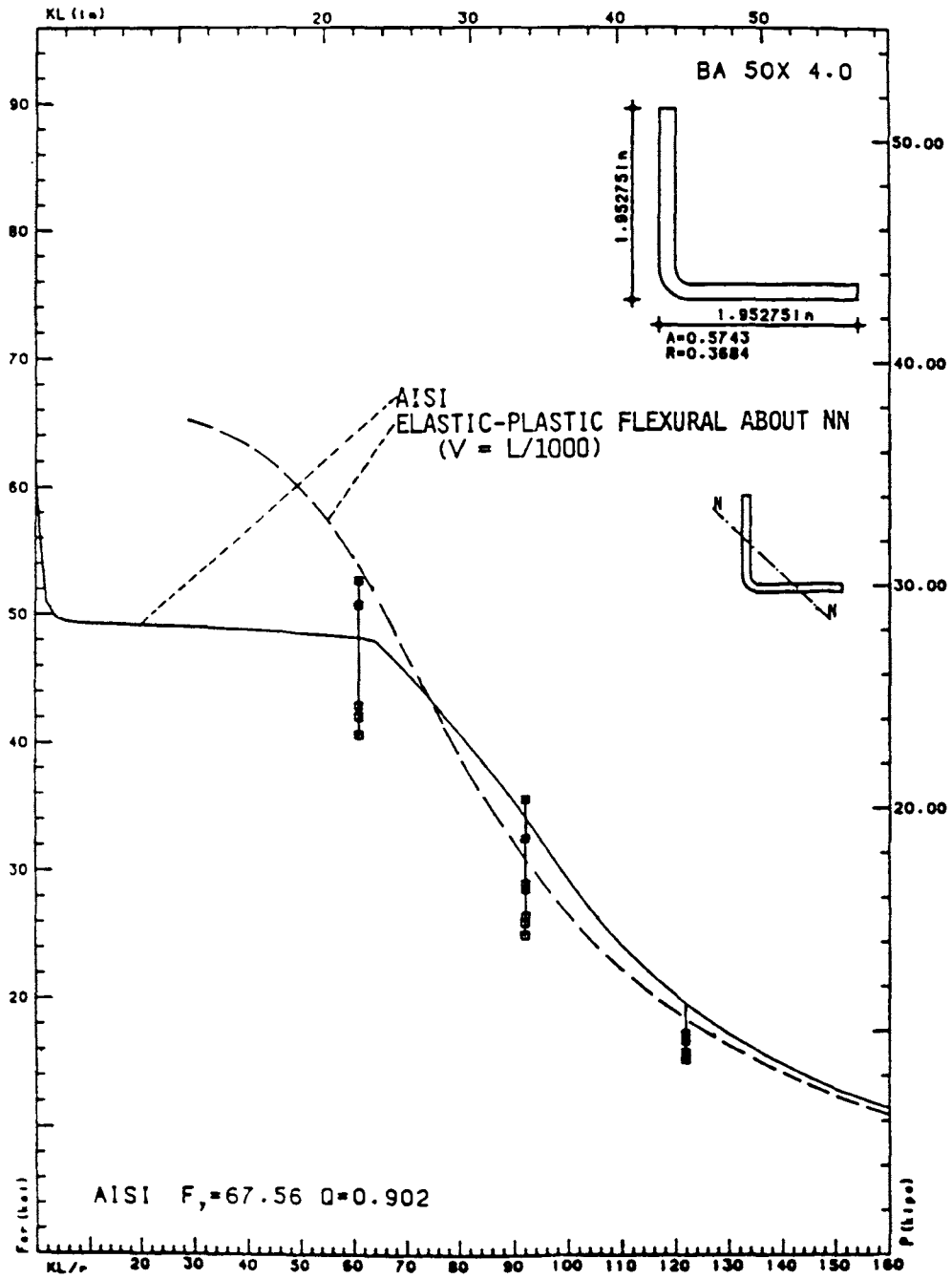


Fig. 8.3-1e Section 1 - Correlation with the 1980 AISI approach and other approaches

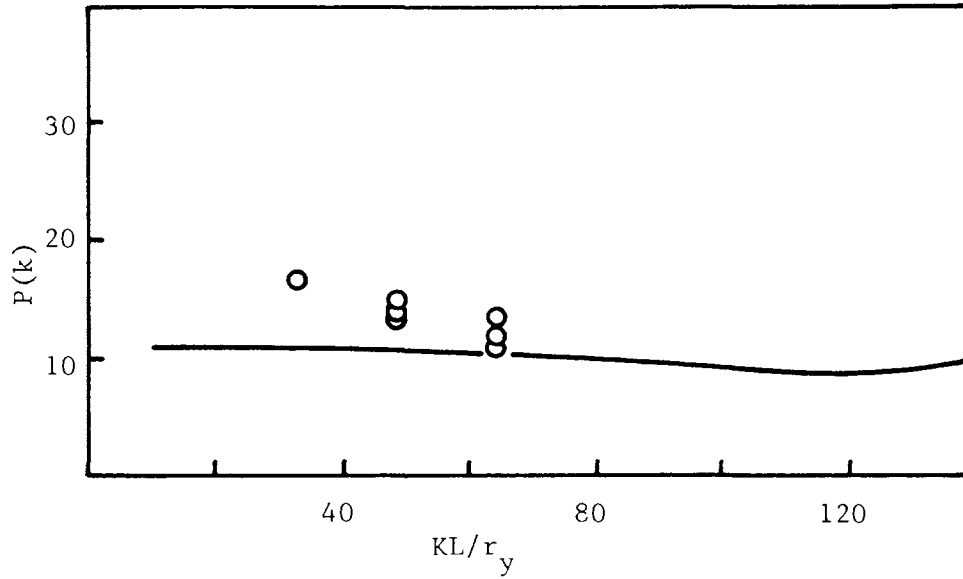


Fig. 8.3-2a Section 2 - Unified approach with $e_x = 0$

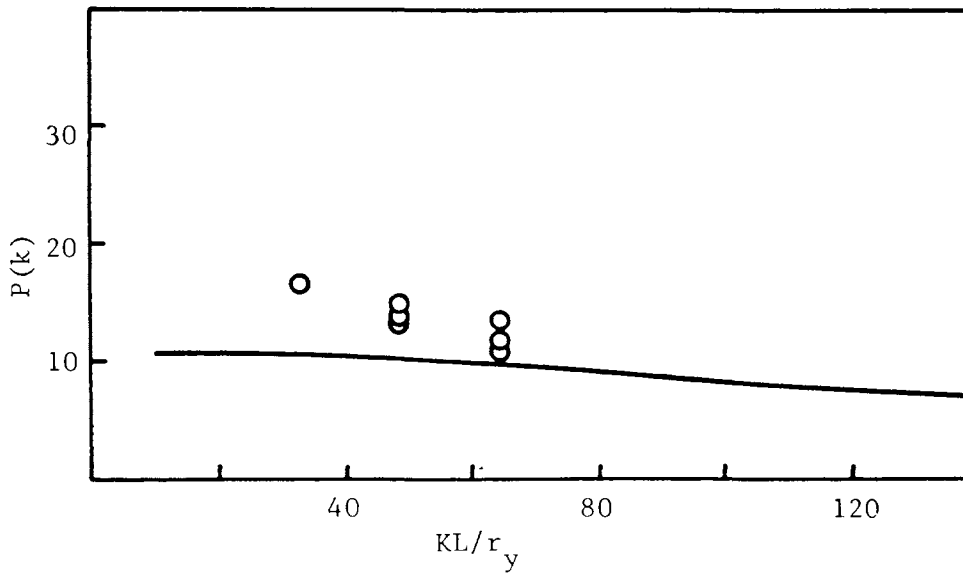


Fig. 8.3-2b Section 2 - Unified approach with $e_x = L/1000$

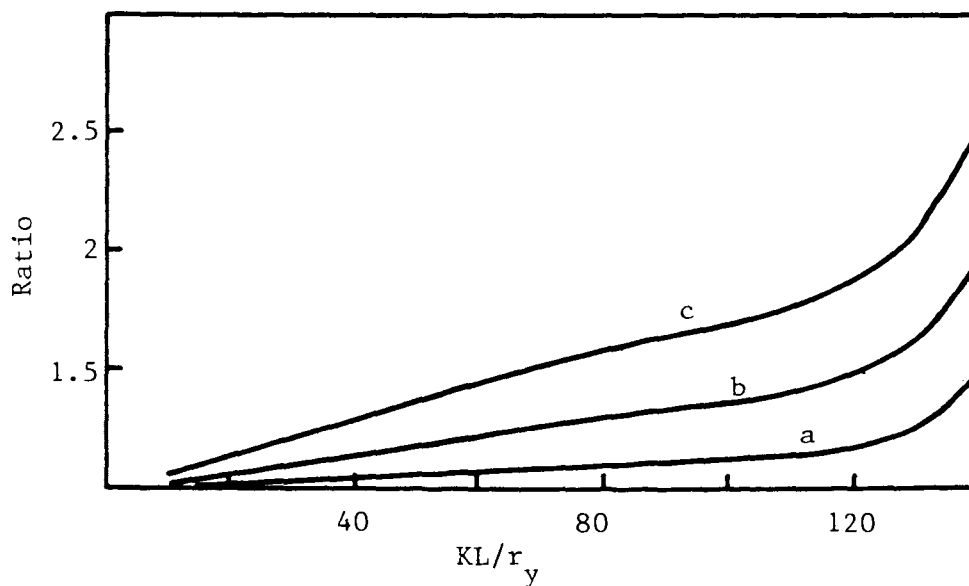


Fig. 8.3-2c Section 2 - Unified approach

Curve a: $(P_{ult} \text{ for } e_x = 0) / (P_{ult} \text{ for } e_x = L/1000)$

Curve b: $(P_{ult} \text{ for } e_x = 0) / (P_{ult} \text{ for } e_x = L/500)$

Curve c: $(P_{ult} \text{ for } e_x = 0) / (P_{ult} \text{ for } e_x = 2L/1000)$

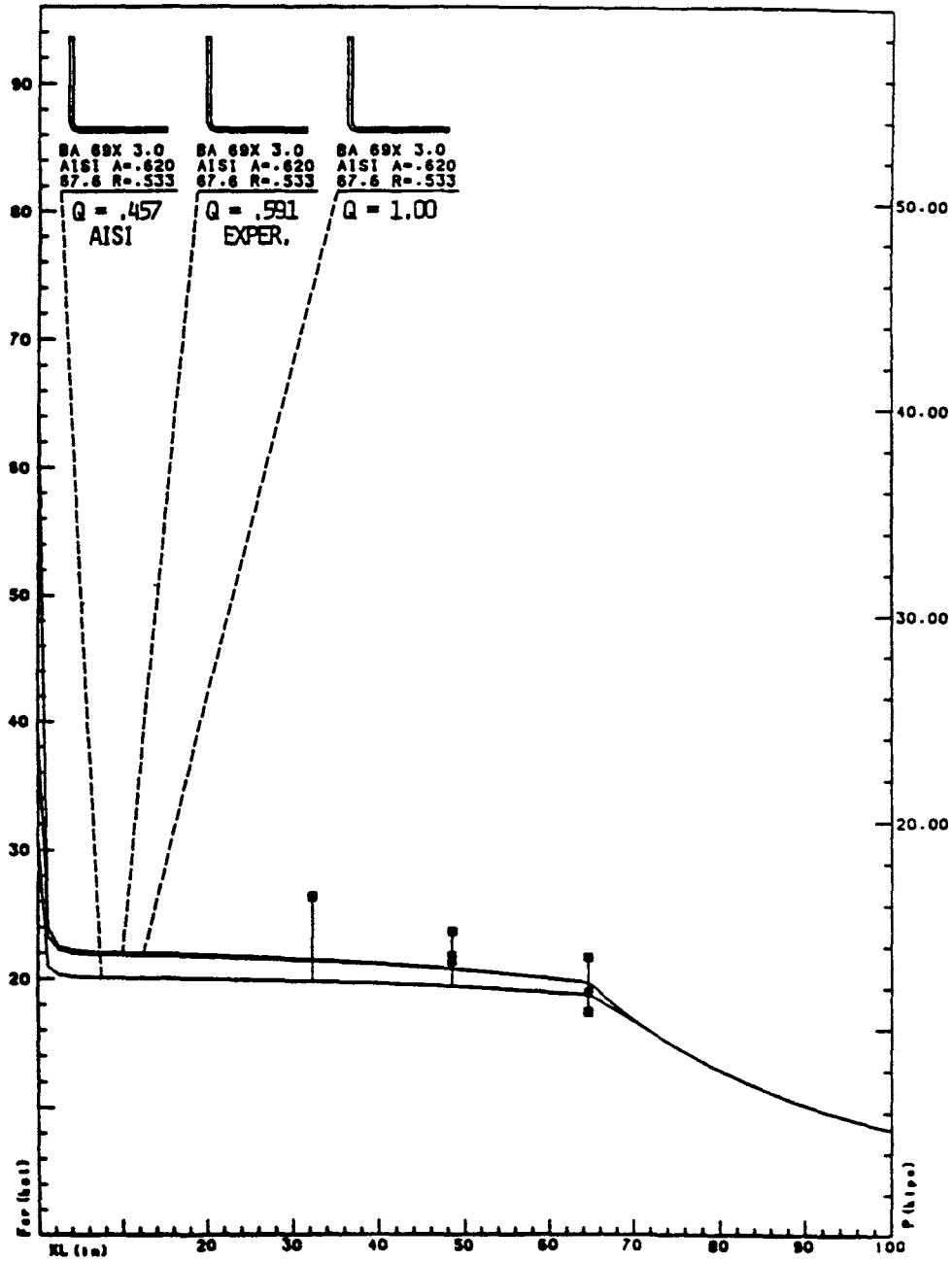


Fig. 8.3-2d Section 2 - Correlation with the 1980 AISI approach and other approaches

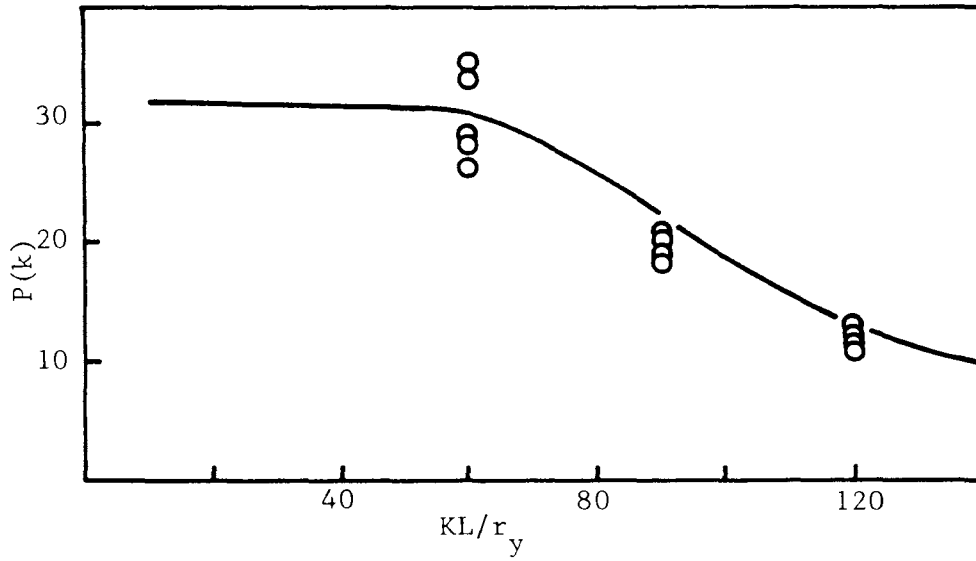


Fig. 8.3-3a Section 3 - Unified approach with $e_x = 0$

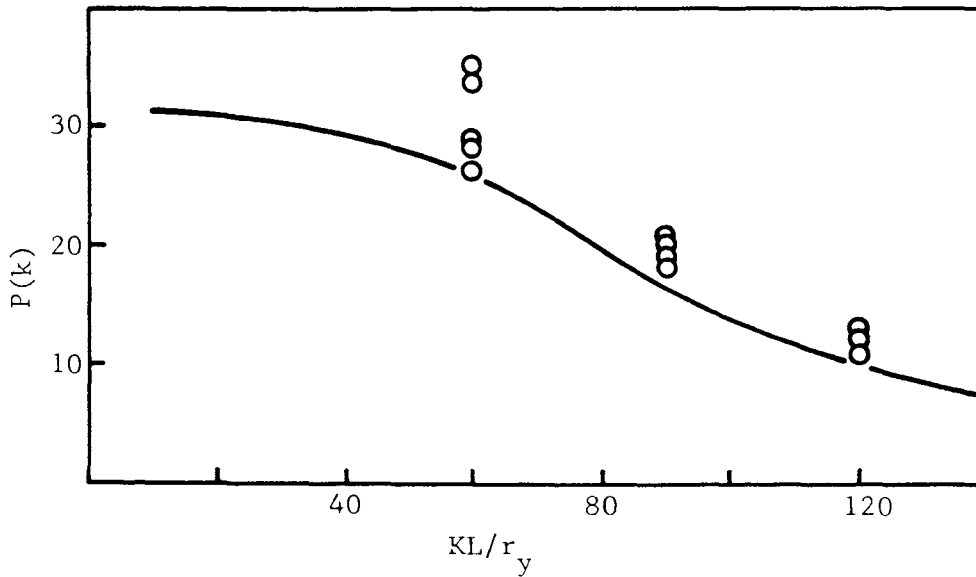


Fig. 8.3-3b Section 3 - Unified approach with $e_x = L/1000$

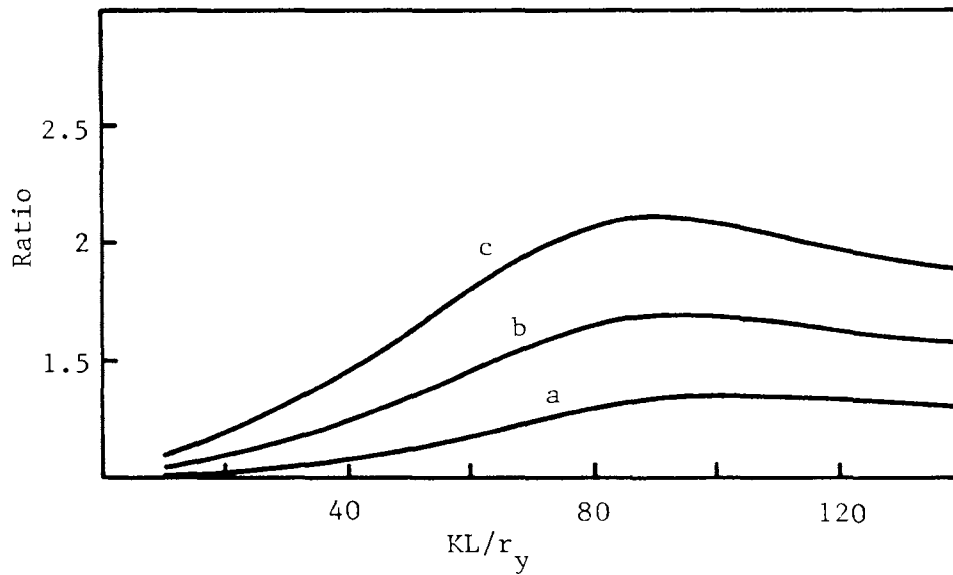


Fig. 8.3-3c Section 3 - Unified approach

Curve a: $(P_{ult} \text{ for } e_x = 0) / (P_{ult} \text{ for } e_x = L/1000)$

Curve b: $(P_{ult} \text{ for } e_x = 0) / (P_{ult} \text{ for } e_x = L/500)$

Curve c: $(P_{ult} \text{ for } e_x = 0) / (P_{ult} \text{ for } e_x = 2L/1000)$

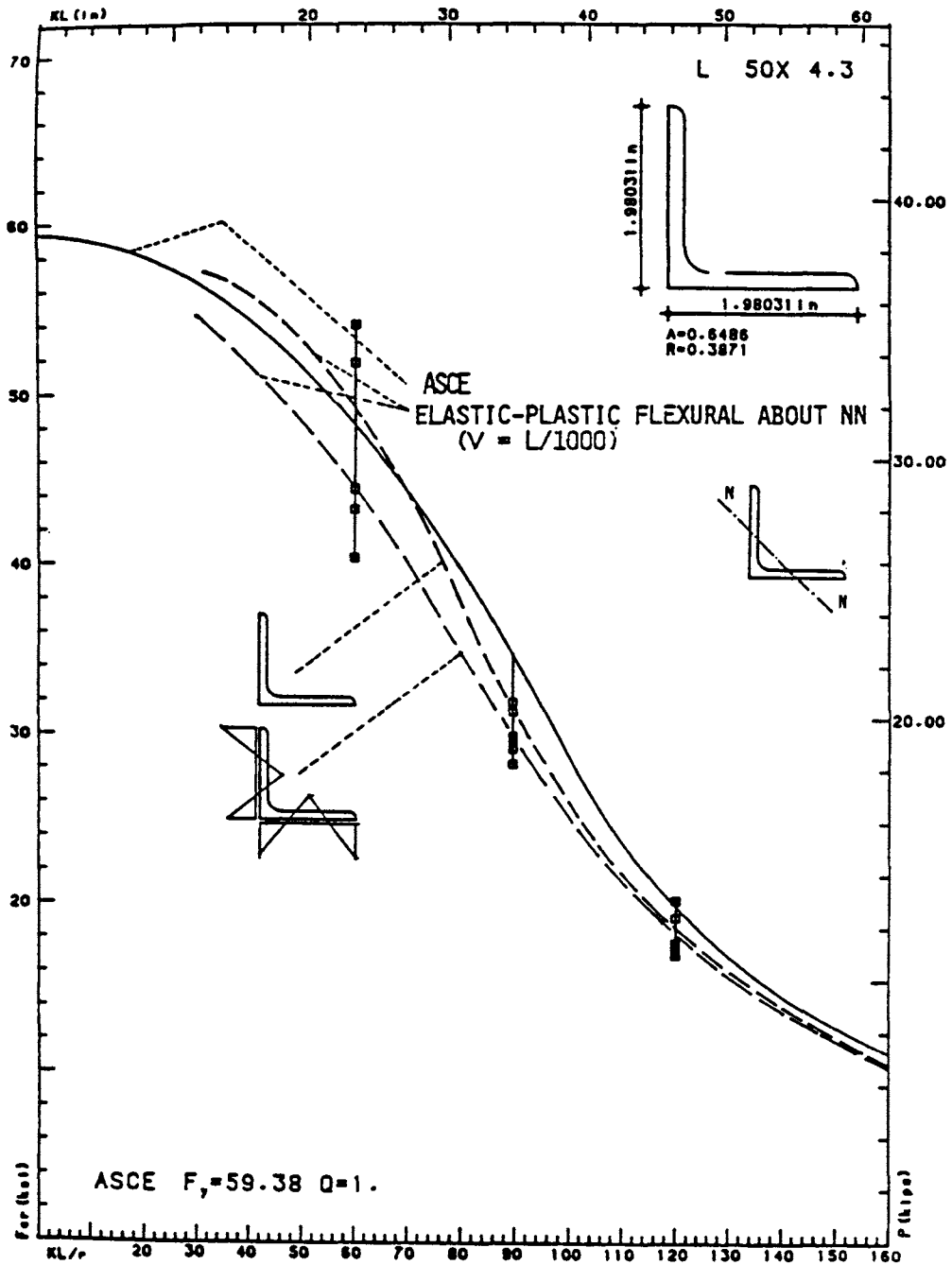


Fig. 8.2-3d Section 3 - Correlation with the 1980 AISI approach and other approaches

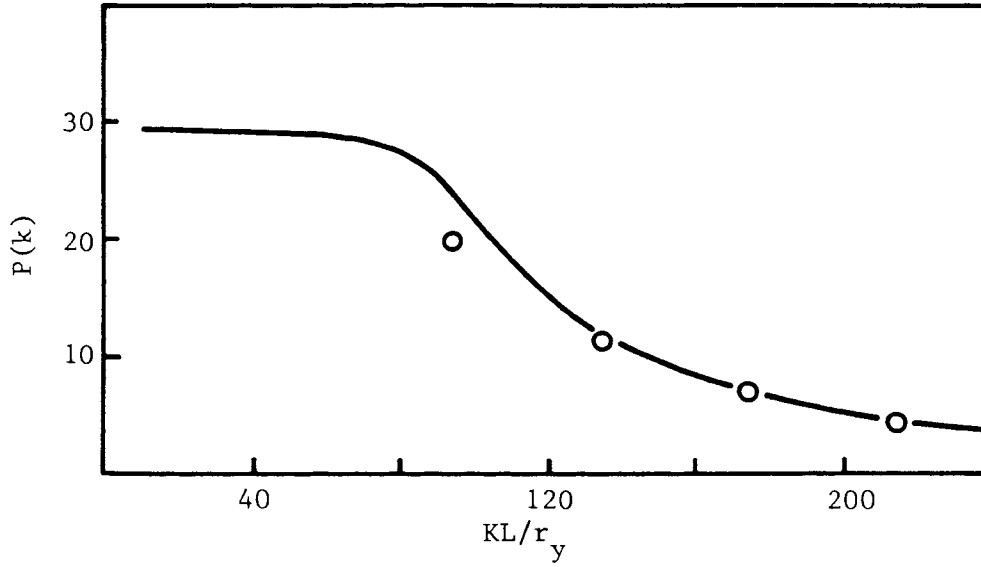


Fig. 8.3-4a Section 4 - Unified approach with $e_x = 0$

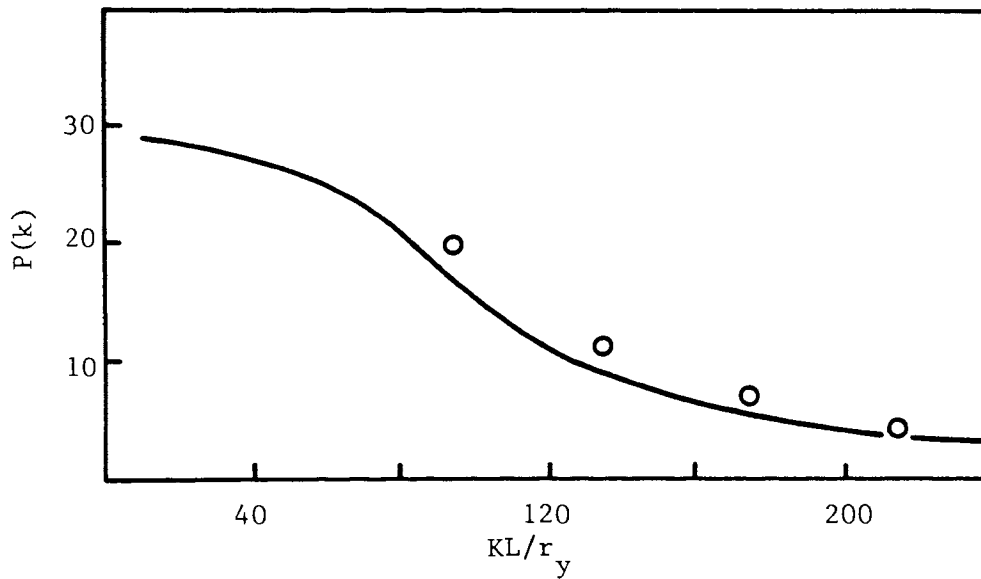


Fig. 8.3-4b Section 4 - Unified approach with $e_x = L/1000$

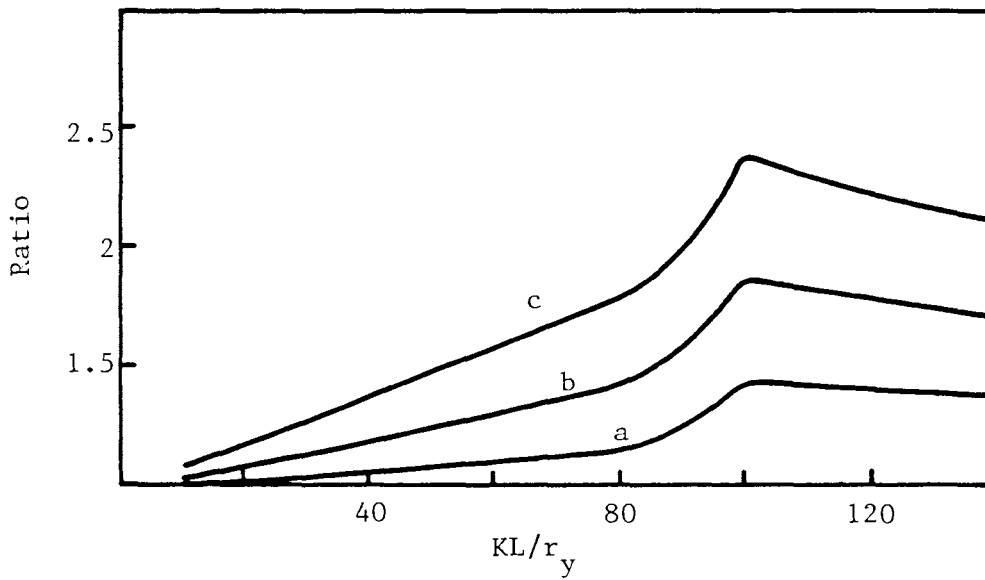


Fig. 8.3--4c Section 4 - Unified approach

- Curve a: $(P_{ult} \text{ for } e_x = 0) / (P_{ult} \text{ for } e_x = L/1000)$
Curve b: $(P_{ult} \text{ for } e_x = 0) / (P_{ult} \text{ for } e_x = L/500)$
Curve c: $(P_{ult} \text{ for } e_x = 0) / (P_{ult} \text{ for } e_x = 2L/1000)$

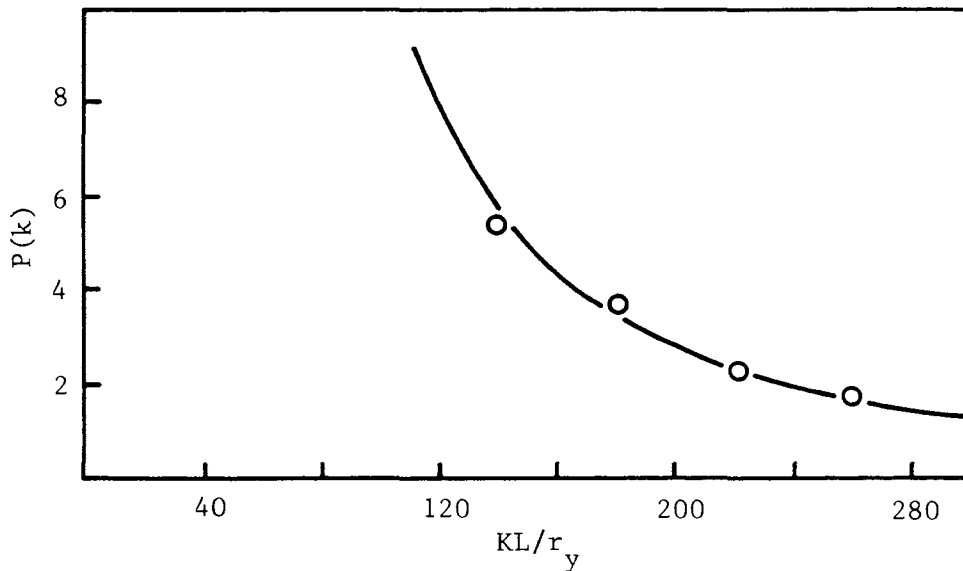


Fig. 8.3-5a Section 5 - Unified approach with $e_x = 0$

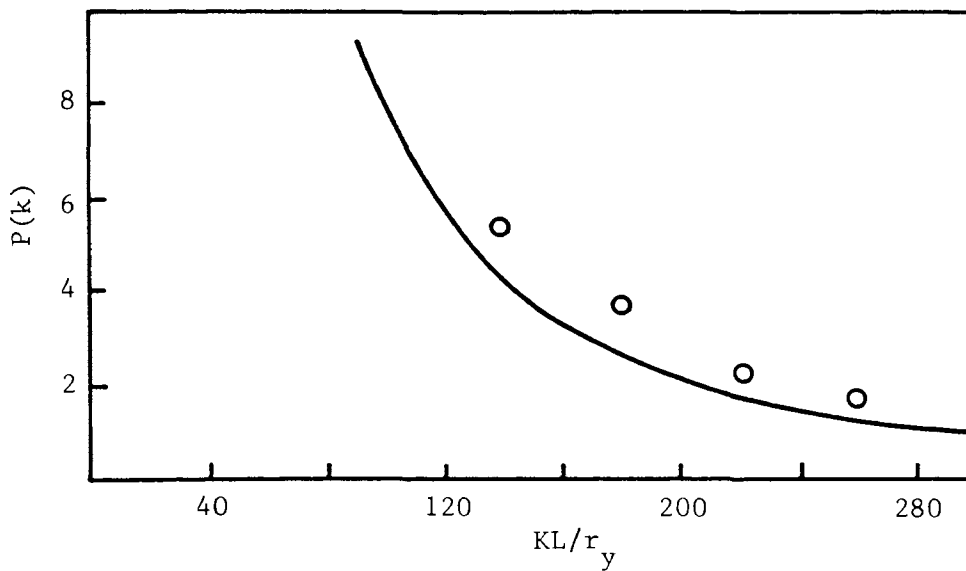


Fig. 8.3-5b Section 5 - Unified approach with $e_x = L/1000$

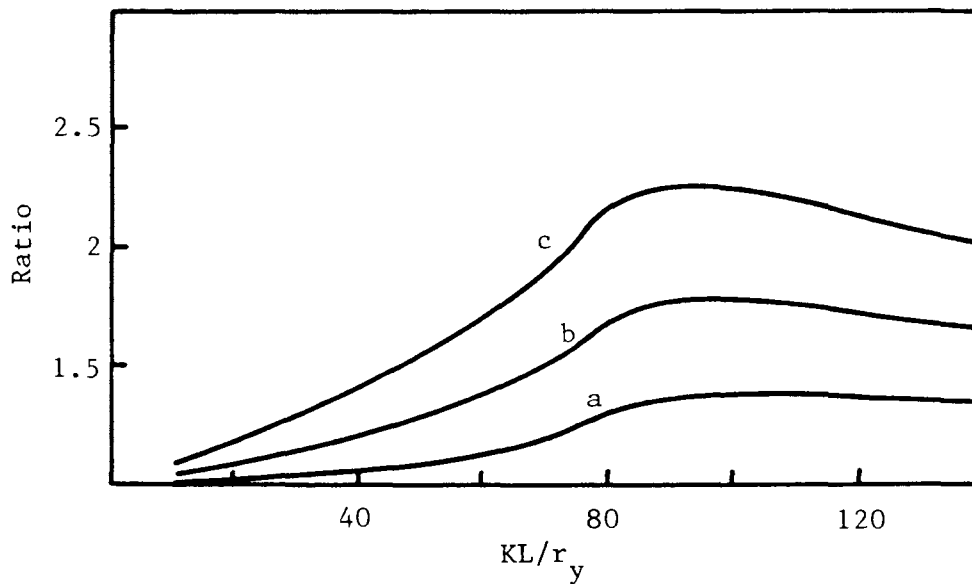


Fig. 8.3-5c Section 5 - Unified approach

Curve a: $(P_{ult} \text{ for } e_x = 0) / (P_{ult} \text{ for } e_x = L/1000)$
Curve b: $(P_{ult} \text{ for } e_x = 0) / (P_{ult} \text{ for } e_x = L/500)$
Curve c: $(P_{ult} \text{ for } e_x = 0) / (P_{ult} \text{ for } e_x = 2L/1000)$

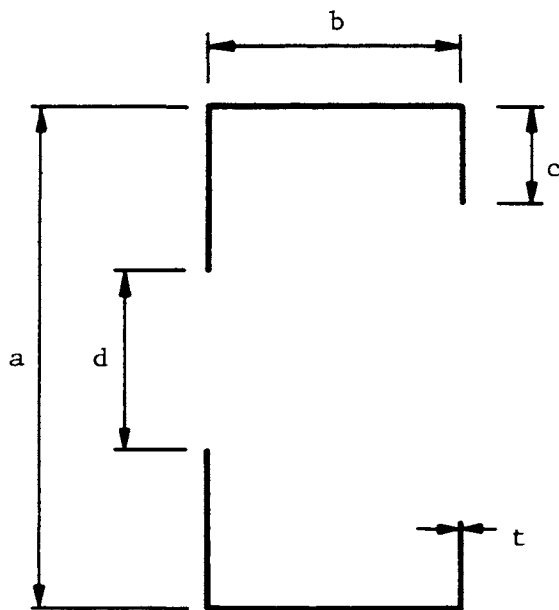


Fig. 9.1-1 Dimensions of the lipped channels tested

$a=3.5$ in $b=1.555$ in $c=0.512$ in

Assumed corner radius = 0.15 in

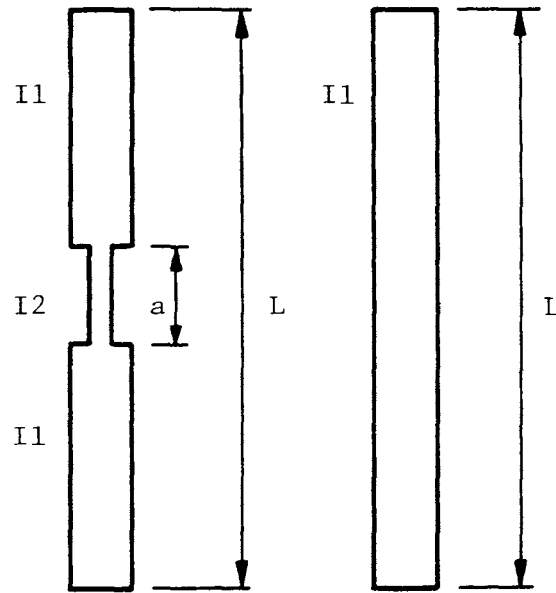


Fig. 9.4-1 Dimensions of columns used in the parametric study on nonprismatic columns

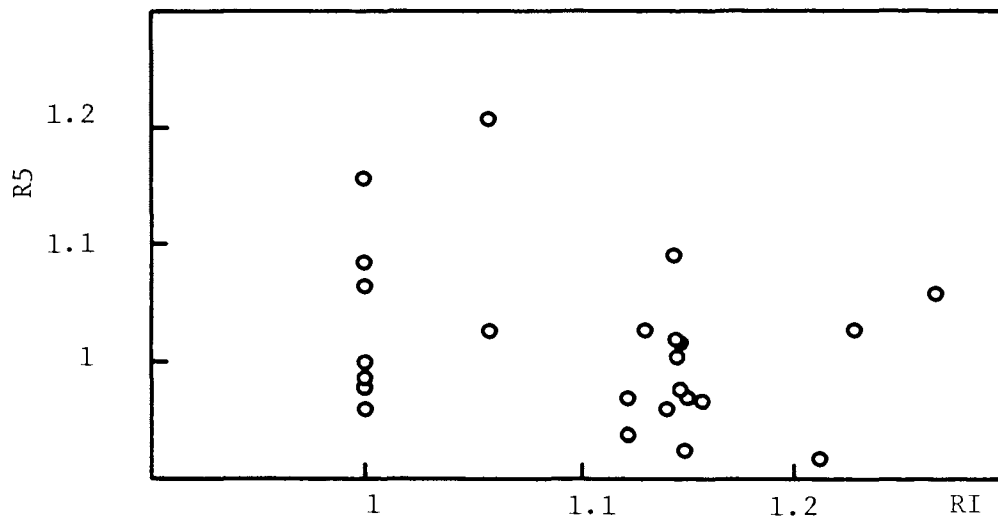


Fig. 9.4-2 Ratios of observed test results to calculated test results

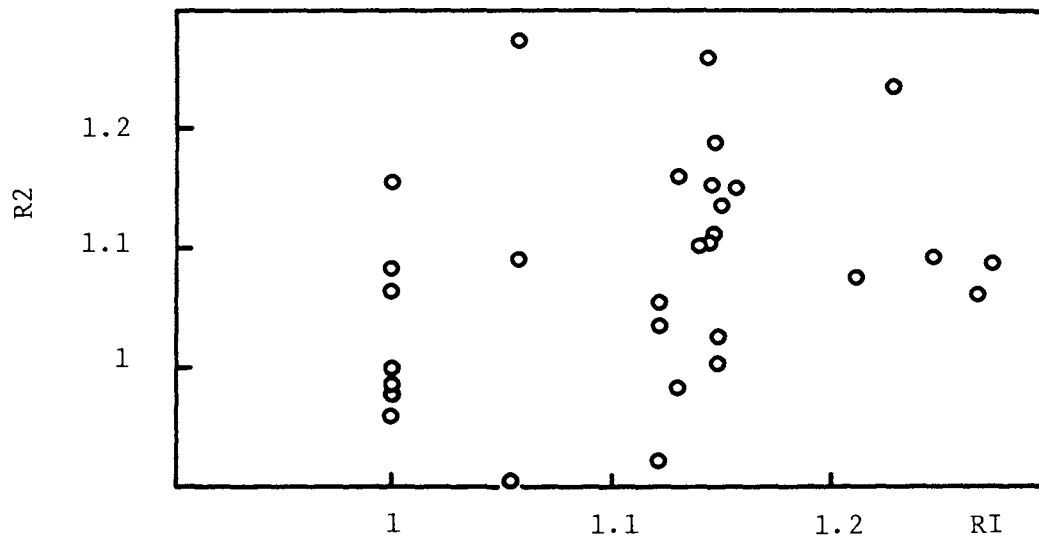


Fig. 9.4-3 Ratios of test results to calculated results

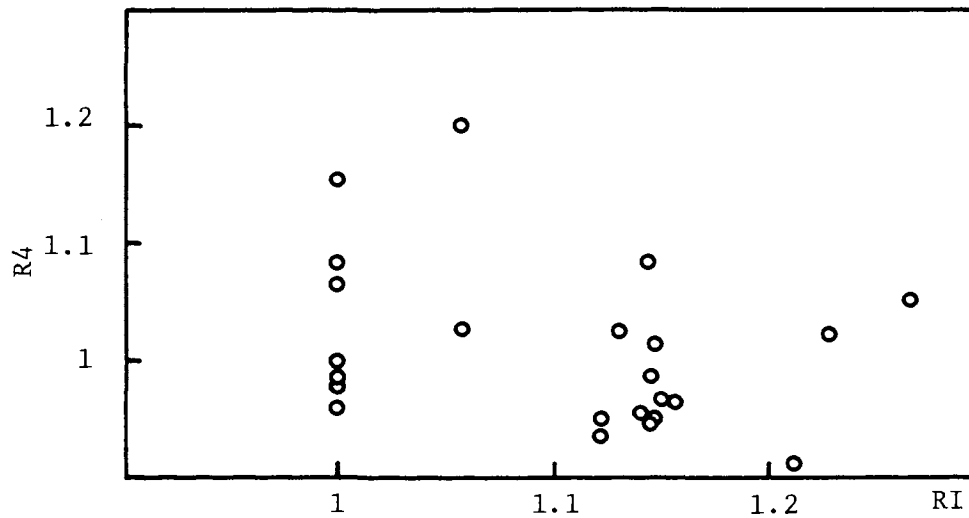


Fig. 9.4-4 Ratios of test results to calculated results

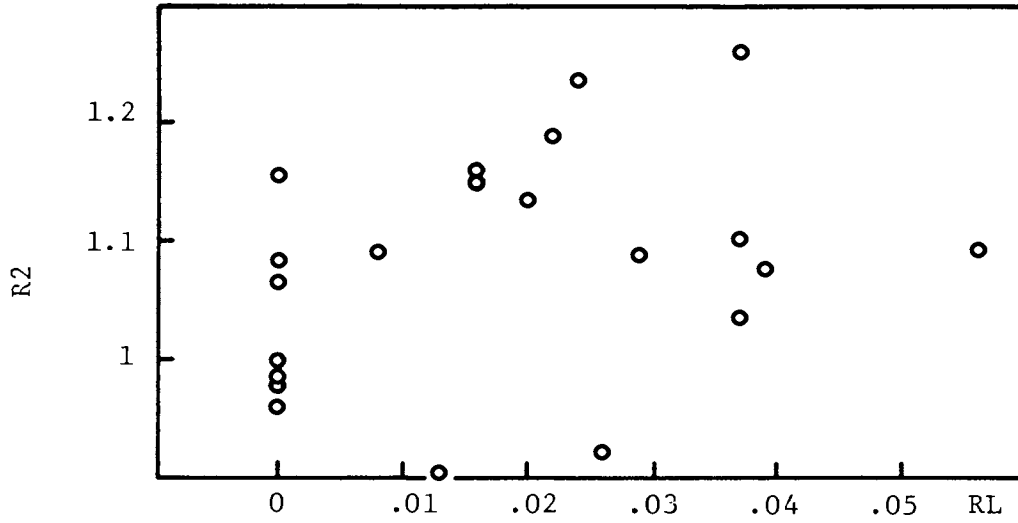


Fig. 9.4-5a Ratios of test results to calculated results

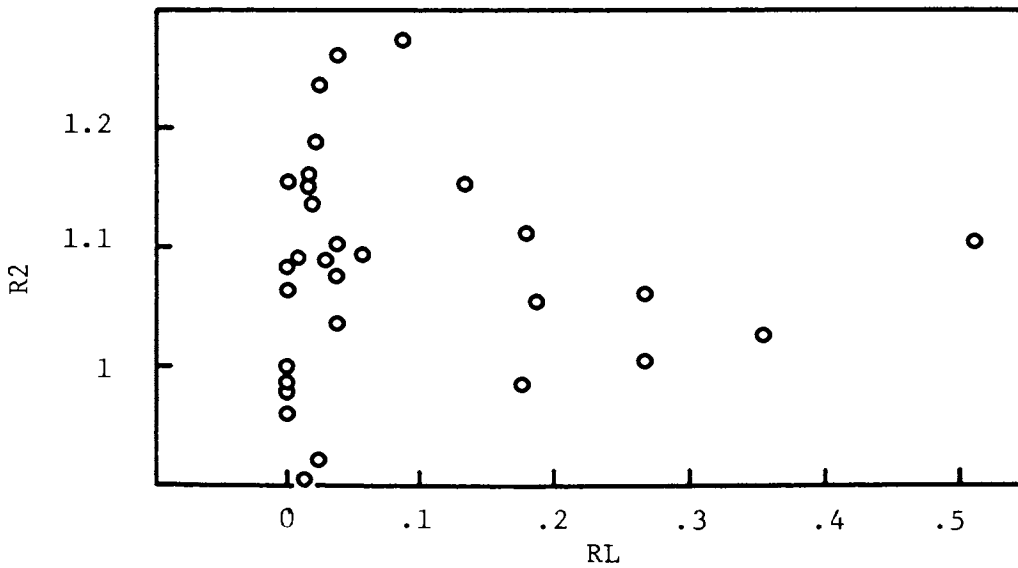


Fig. 9.4-5b Ratios of test results to calculated results

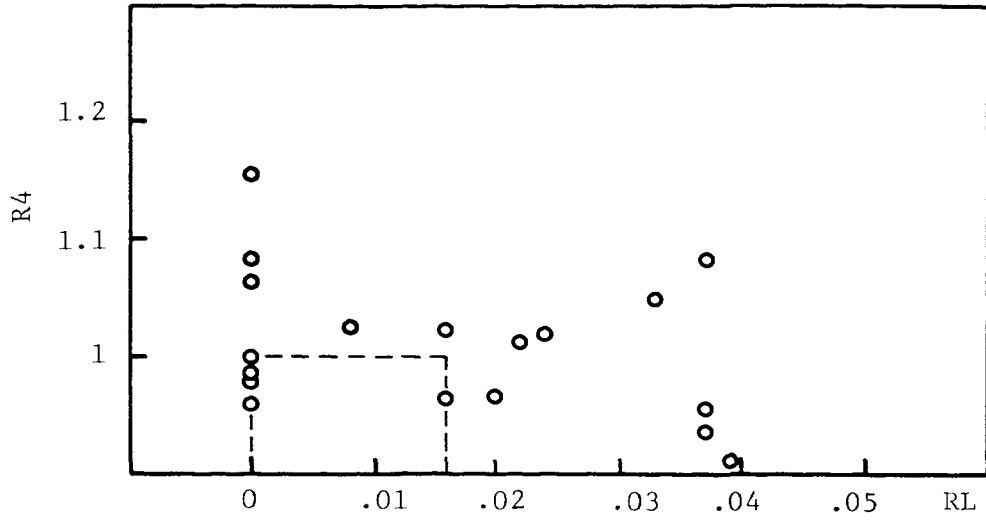


Fig. 9.4-6a Ratios of test results to calculated results

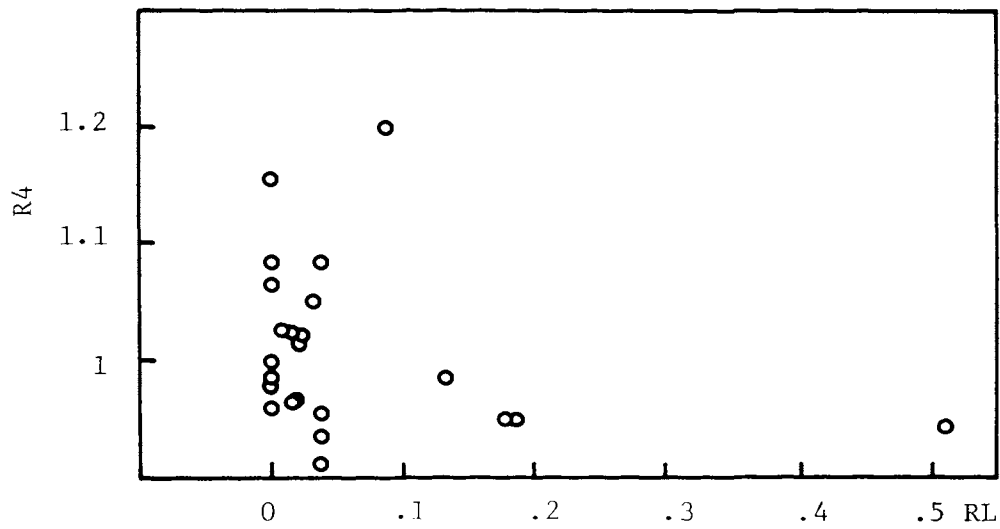


Fig. 9.4-6b Ratios of test results to calculated results

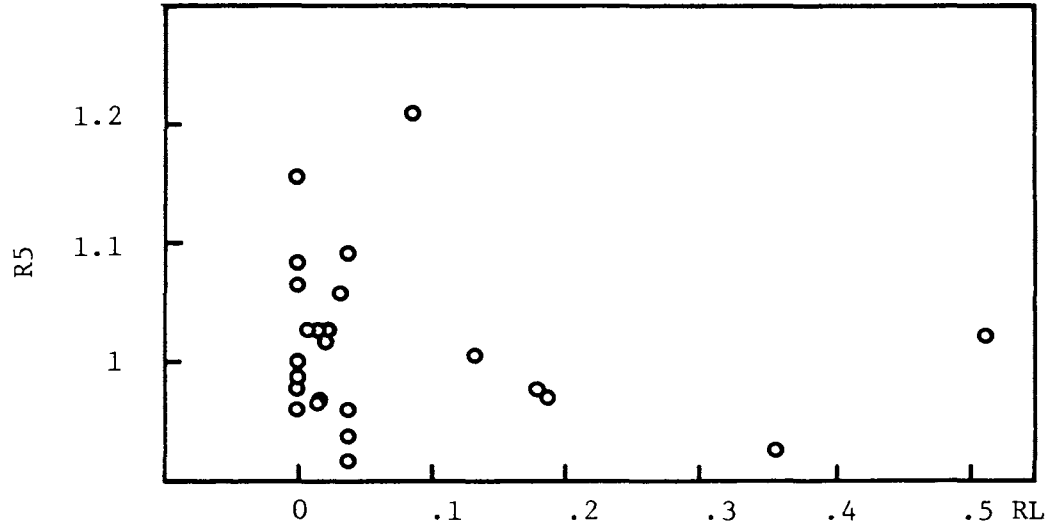


Fig. 9.4-7a Ratios test results to calculated results

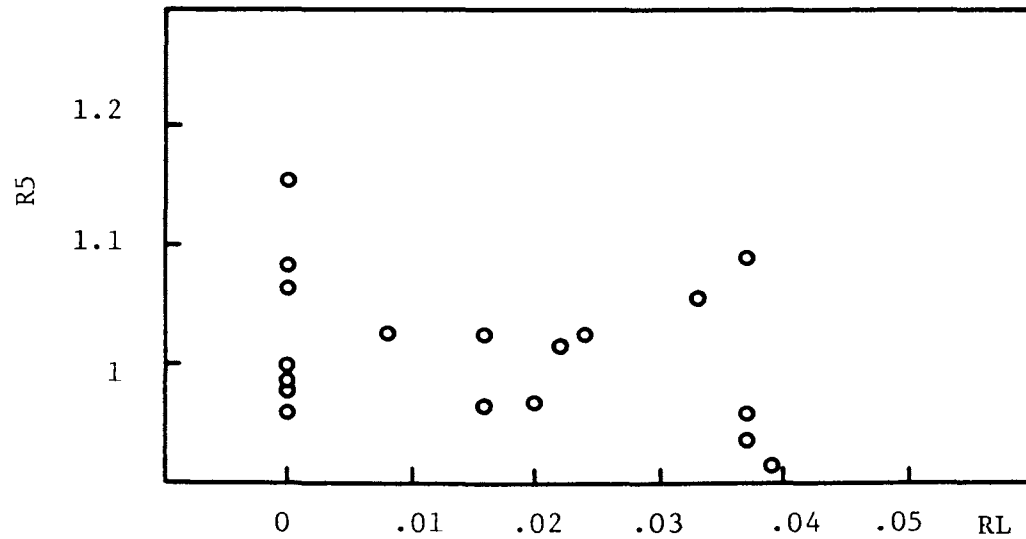


Fig. 9.4-7b Ratios of test results to calculated results

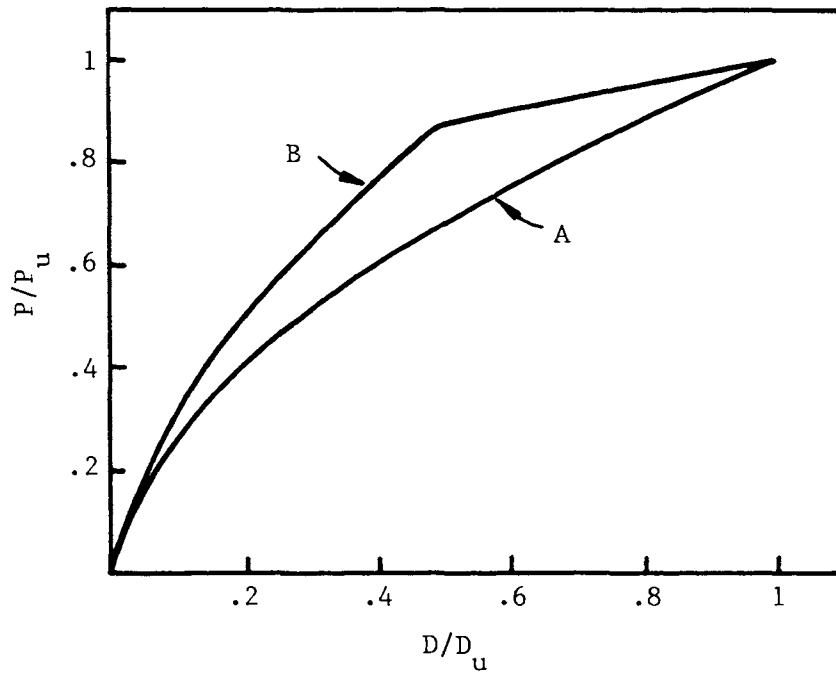


Fig. 10-1 Simulated stub column test results
8"x8" square tube of .05" thickness
Curve A - using Procedure (1)
Curve B - using Procedure (2)

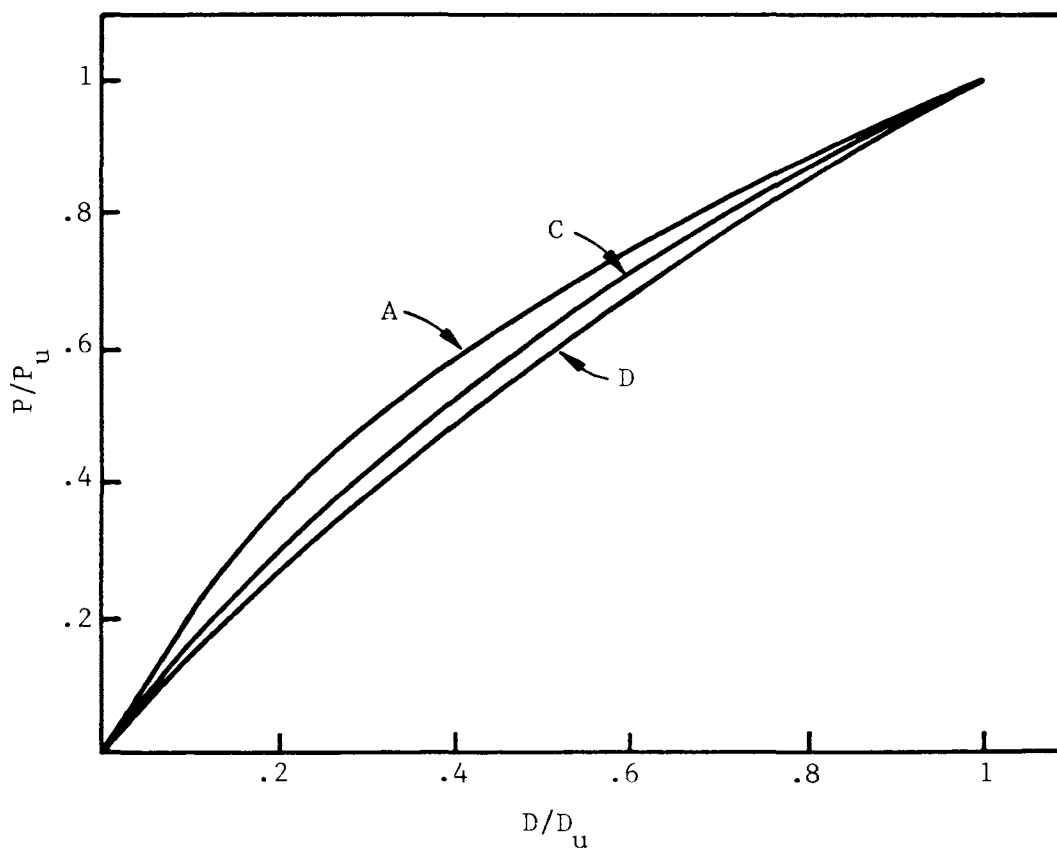


Fig. 10-2 Simulated stub column test results 4"x4" square tube
of .05" thickness
Curve A - using Procedure (1)
Curve C - Eq. 10-8 with $n = 1/3$
Curve D - Eq. 10-8 with $n = 1/5$

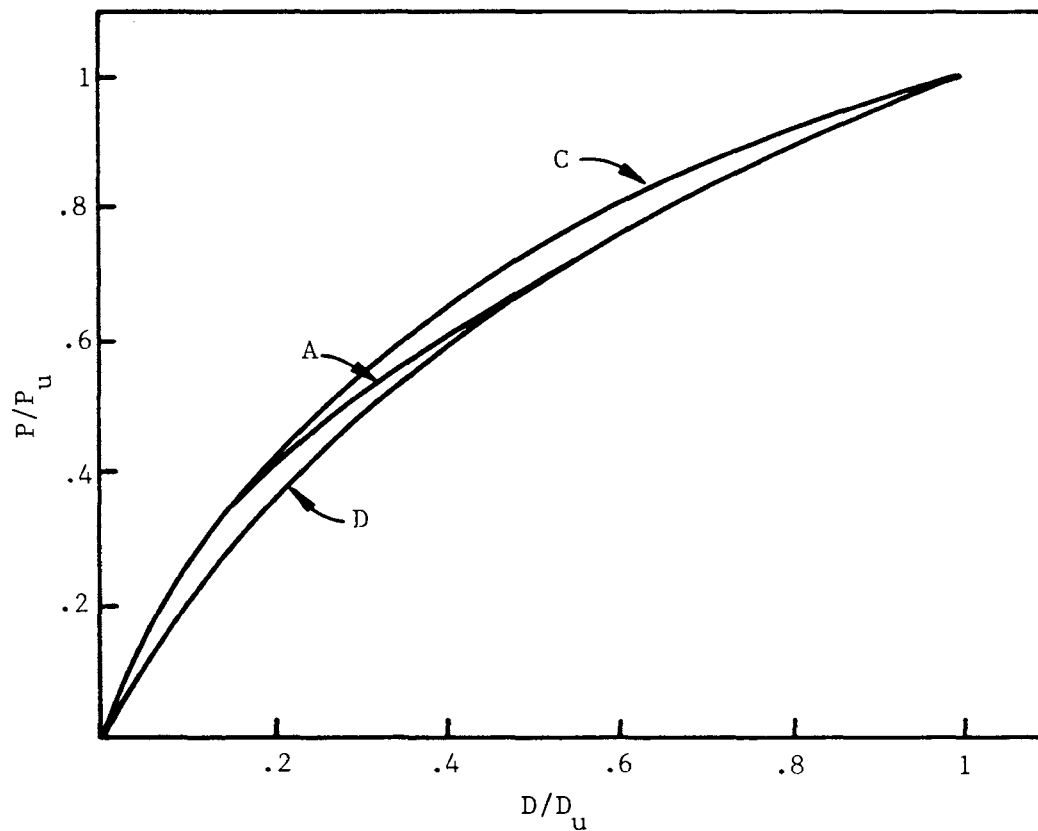


Fig. 10-3 Simulated stub column test results 8"x8" square tube of .05" thickness

- Curve A - using Procedure (1)
- Curve C - Eq. 10-8 with $n = 1/3$
- Curve D - Eq. 10-8 with $n = 1/5$

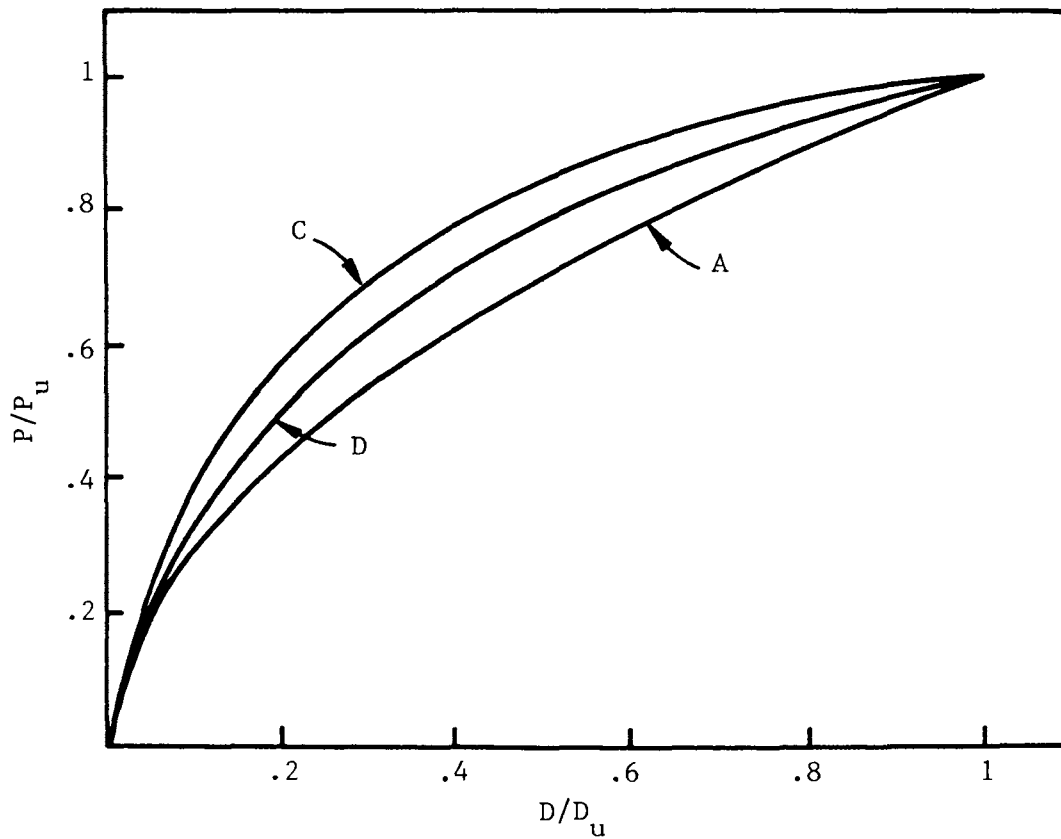


Fig. 10-4 Simulated stub column test results 16"x16" square tube of .05" thickness
Curve A - using Procedure (1)
Curve C - Eq. 10-8 with $n = 1/3$
Curve D - Eq. 10-8 with $n = 1/5$

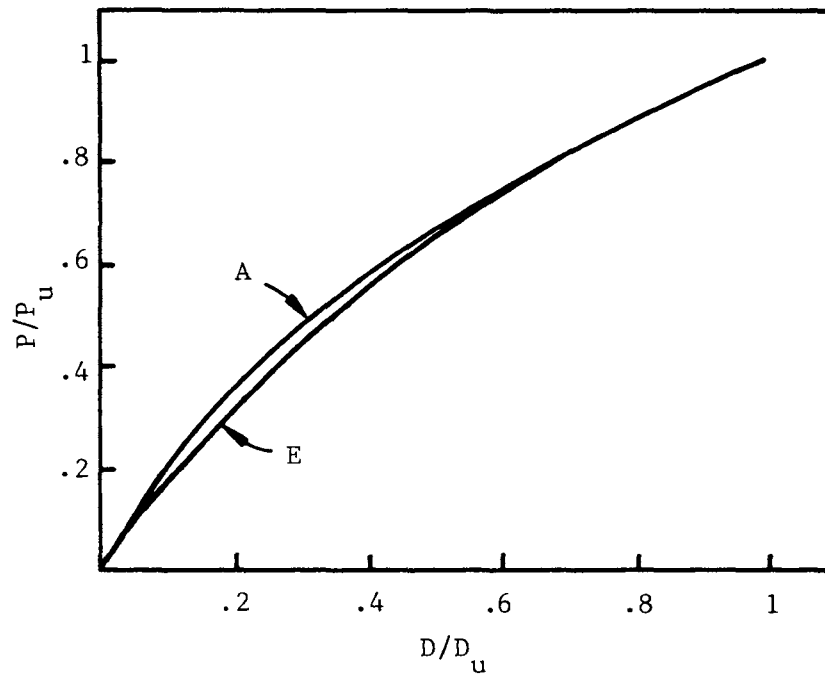


Fig. 10-5 Simulated stub column test results 4"x4"
square tube of .05" thickness
Curve A - using Procedure (1)
Curve E - Eq. 10-8 with $n = A_{eu} / A$ (Eq. 10-9)

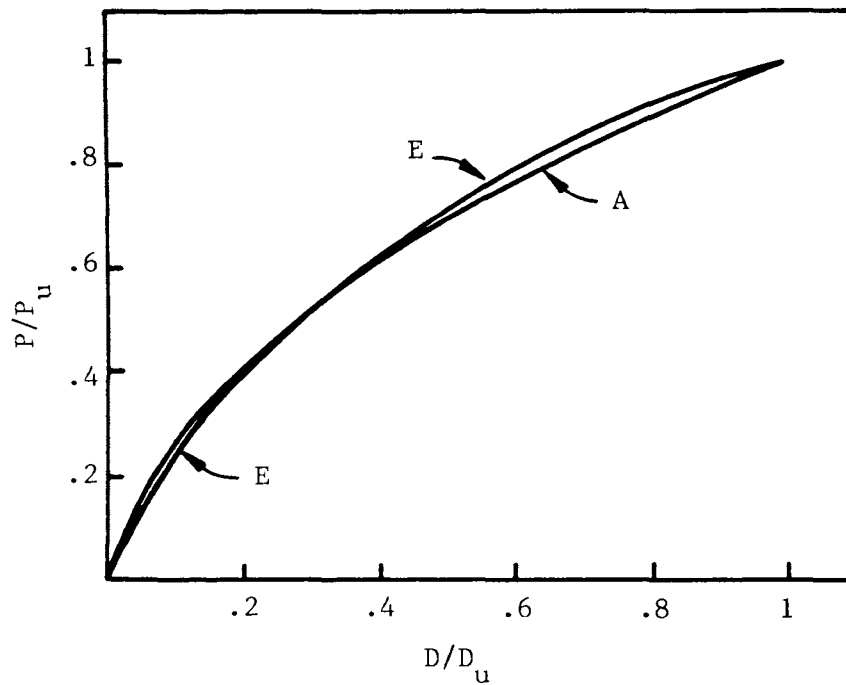


Fig. 10-6 Simulated stub column test results 8"x8"
square tube of .05" thickness
Curve A - using Procedure (1)
Curve E - Eq. 10-8 with $n = A_{eu} / A$ (Eq. 10-9)

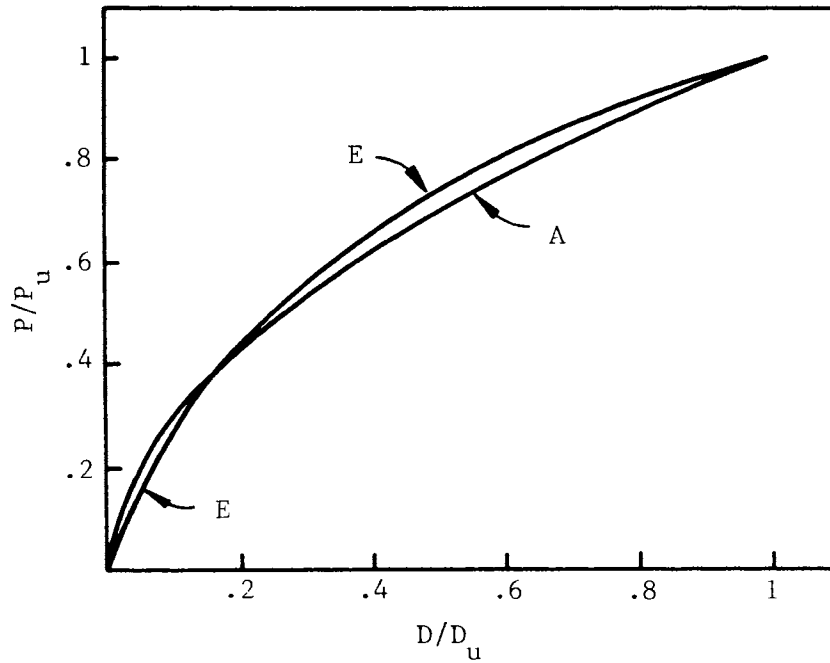


Fig. 10-7 Simulated stub column test results 16"x16"
square tube of .05" thickness
Curve A - using Procedure (1)
Curve E - Eq. 10-8 with $n = A_{eu} / A$ (Eq. 10-9)

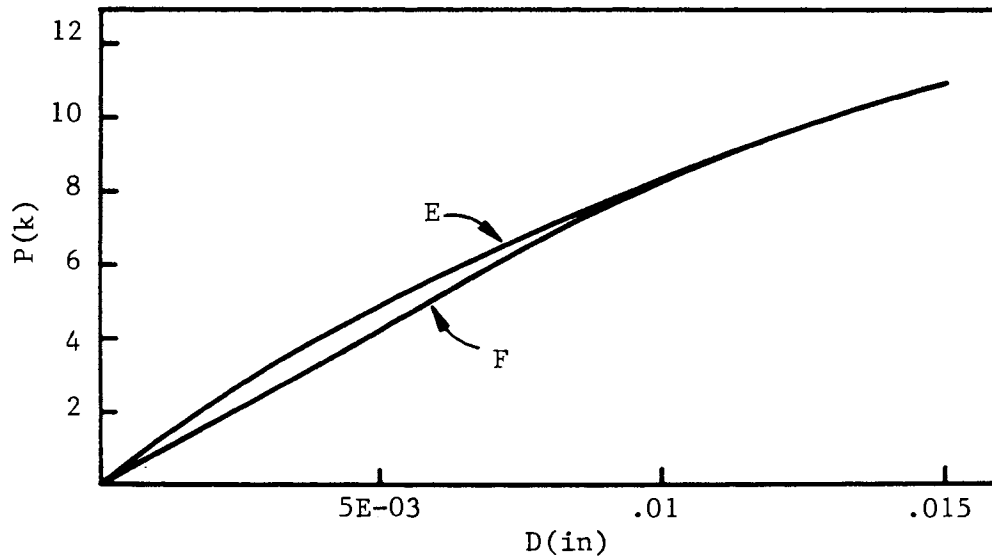


Fig. 10-8 Actual stub column test results specimen
slc/1 60x30
Curve E - Eq. 10-8 with $n = A_{eu} / A$ (Eq. 10-9)
Curve F - Actual test result

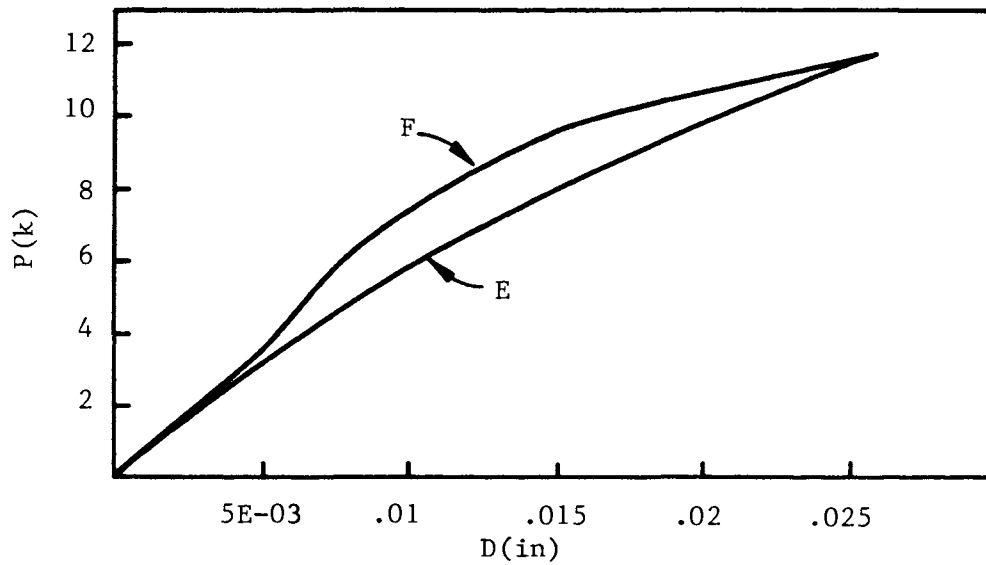


Fig. 10-9 Actual stub column test results specimen slc/2 60/90
Curve E - Eq. 10-8 with $n = A_{eu} / A$ (Eq. 10-9)
Curve F - Actual test result P_{eu}

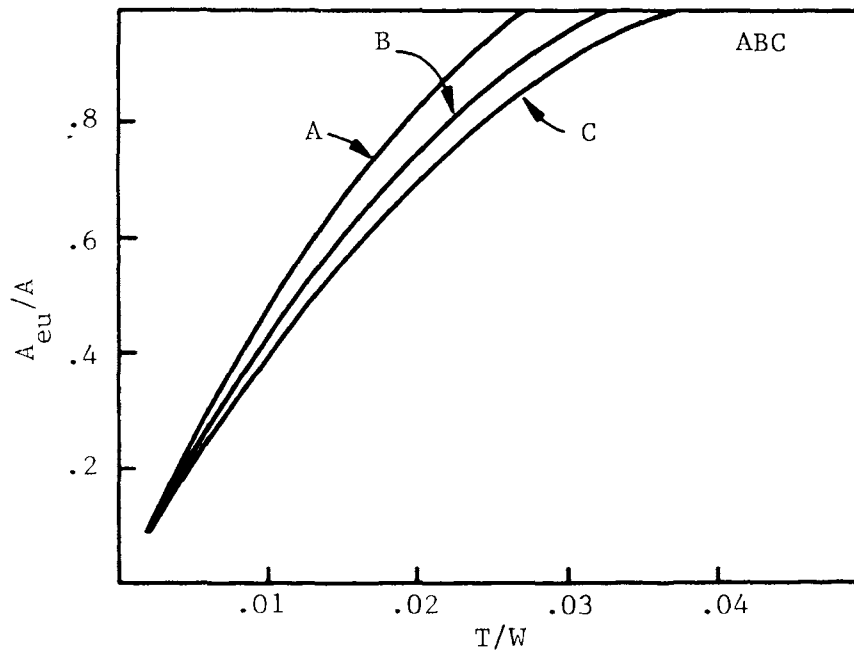


Fig. 10-10 Simulated stub column test results
4"x4" square tube
Curve A - $F_y = 35$ ksi
Curve B - $F_y = 48$ ksi
Curve C - $F_y = 60$ ksi

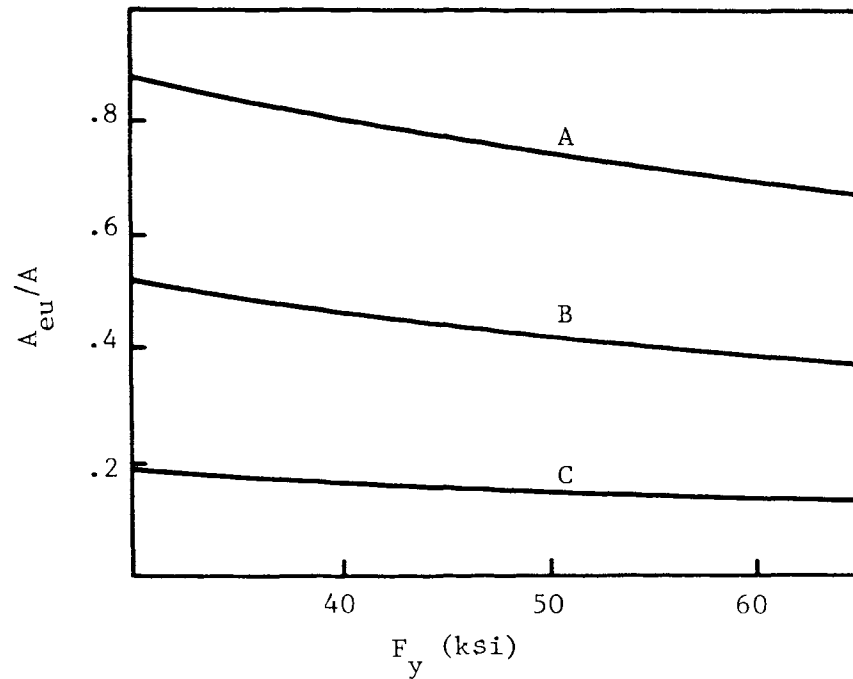


Fig. 10-11 Simulated stub column test results
4"x4" square tube
Curve A - $w/t = 50$
Curve B - $w/t = 100$
Curve C - $w/t = 300$

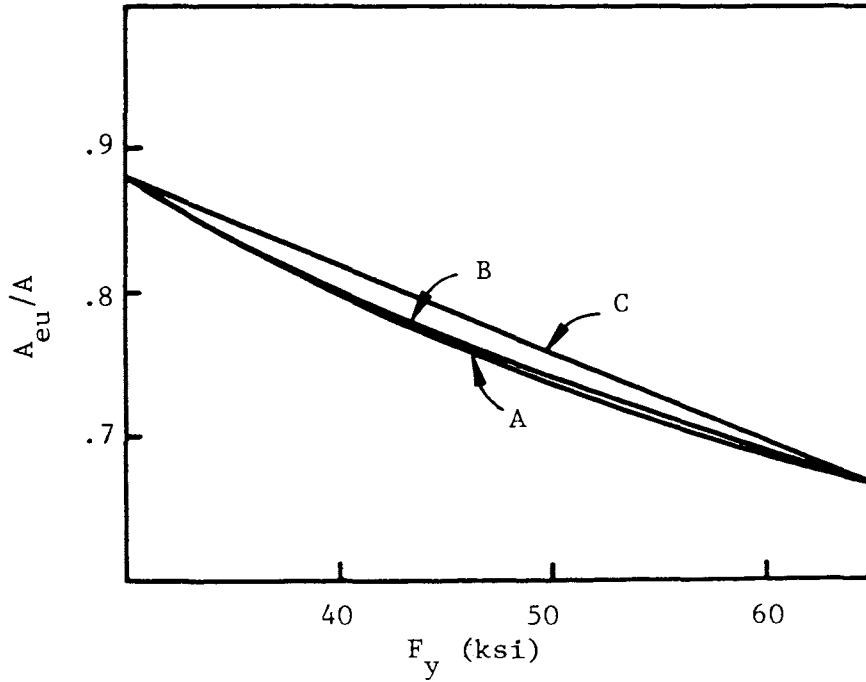


Fig. 10-12 Simulated stub column test results
Curve A - Eq. 14 with $m = 0.8$
Curve B - Simulated stub column test results
Curve C - Eq. 14 with $m = 1$

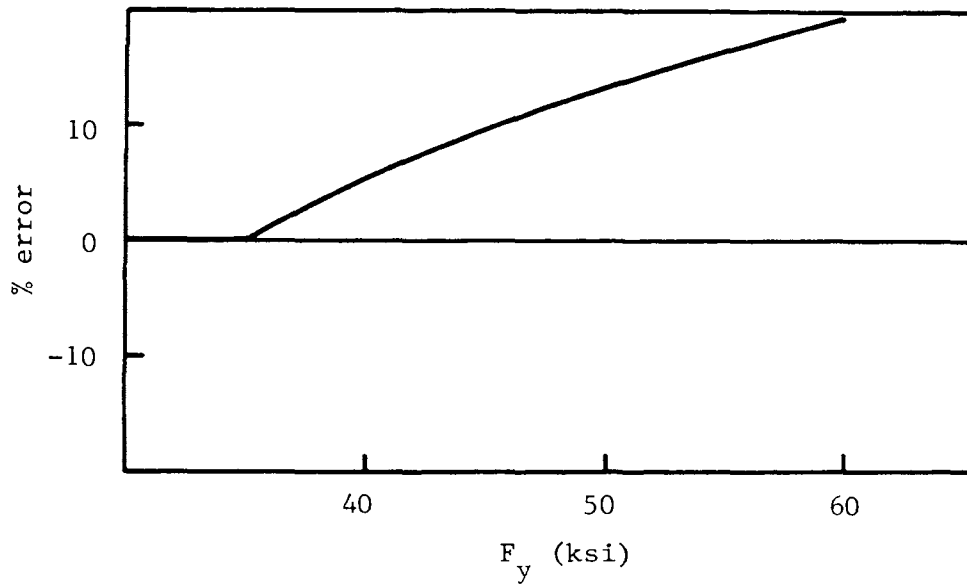


Fig. 10-13 Extrapolation for yield stress from $F_y = 35$ ksi,
 $w/t = 20$

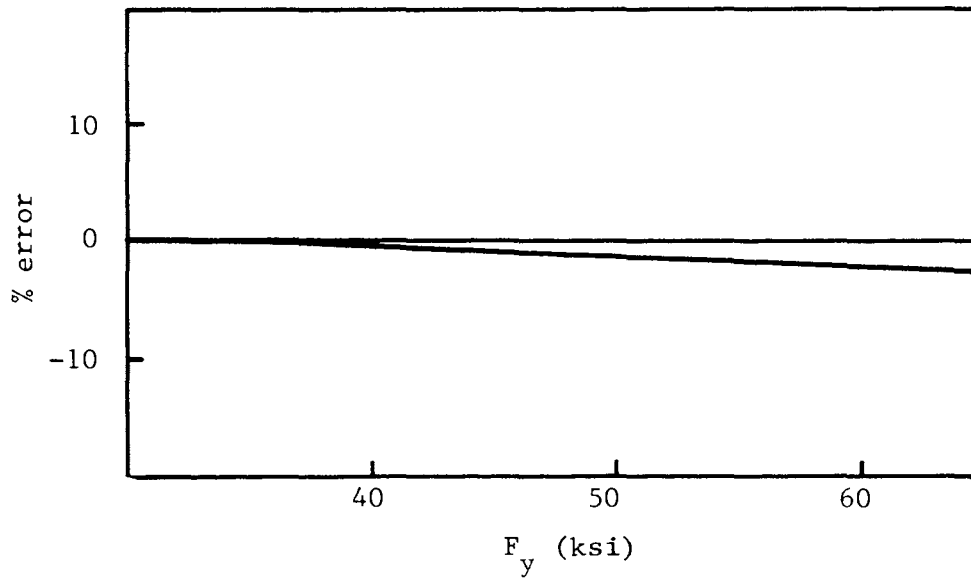


Fig. 10-14 Extrapolation for yield stress from $F_y = 35$ ksi,
 $w/t = 100$

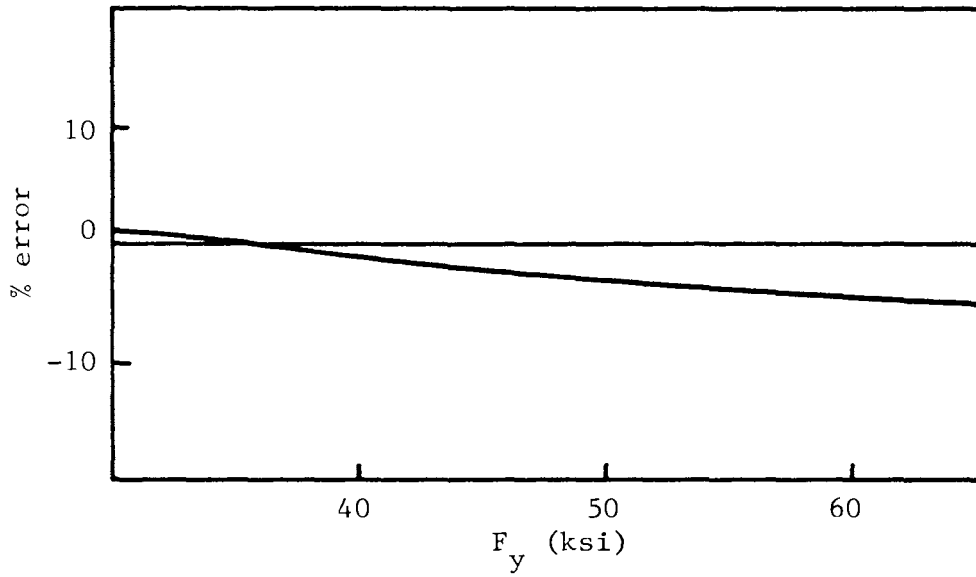


Fig. 10-15 Extrapolation for yield stress from $F_y = 35$ ksi,
 $w/t = 500$

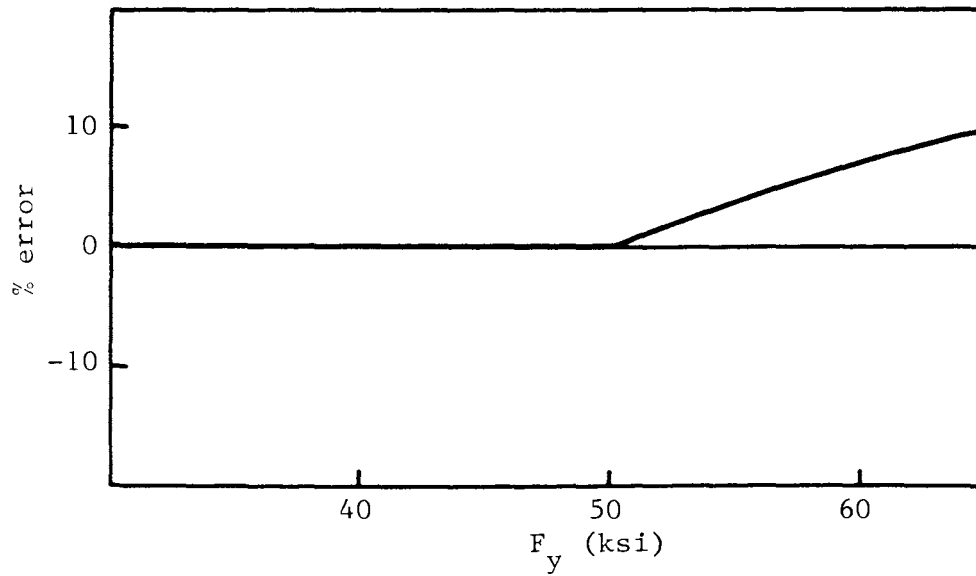


Fig. 10-16 Extrapolation for yield stress from $F_y = 50$ ksi,
 $w/t = 20$

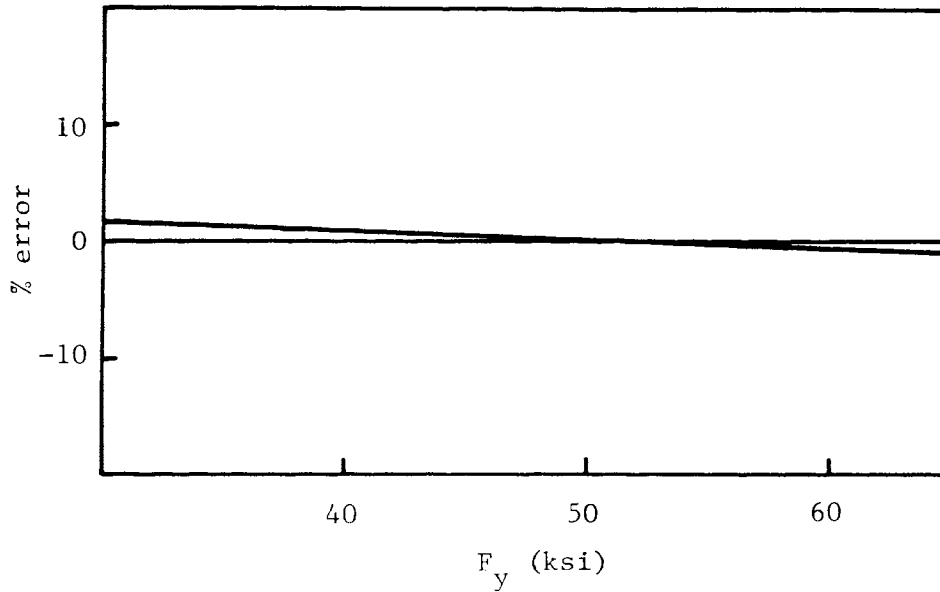


Fig. 10-17 Extrapolation for yield stress from $F_y = 50$ ksi,
 $w/t = 100$

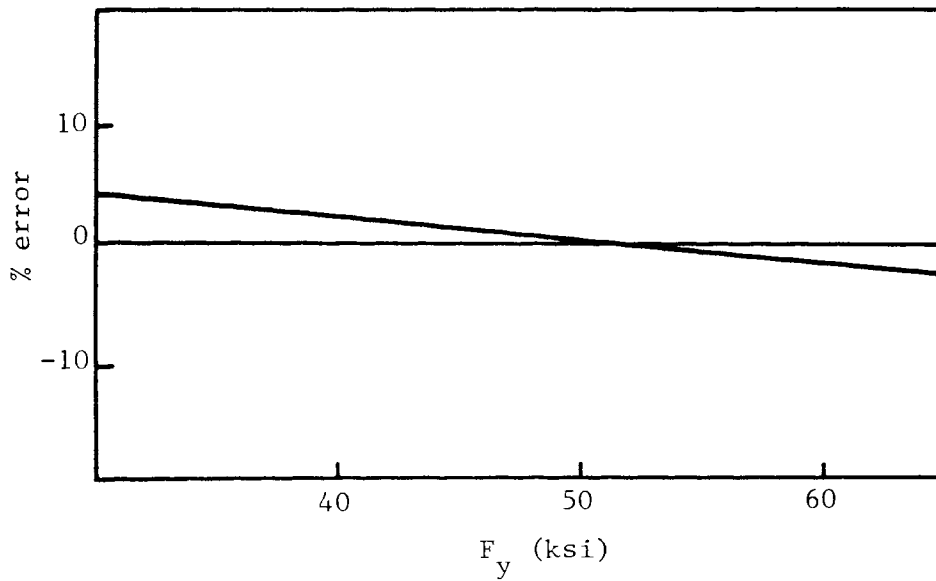


Fig. 10-18 Extrapolation for yield stress from $F_y = 50$ ksi,
 $w/t = 500$

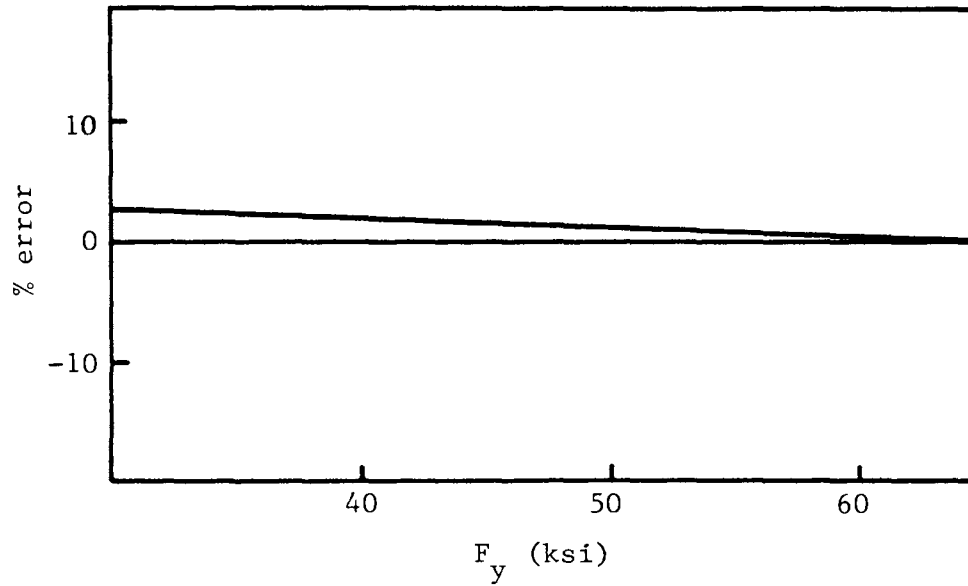


Fig. 10-19 Extrapolation for yield stress from $F_y = 65$ ksi,
 $w/t = 100$

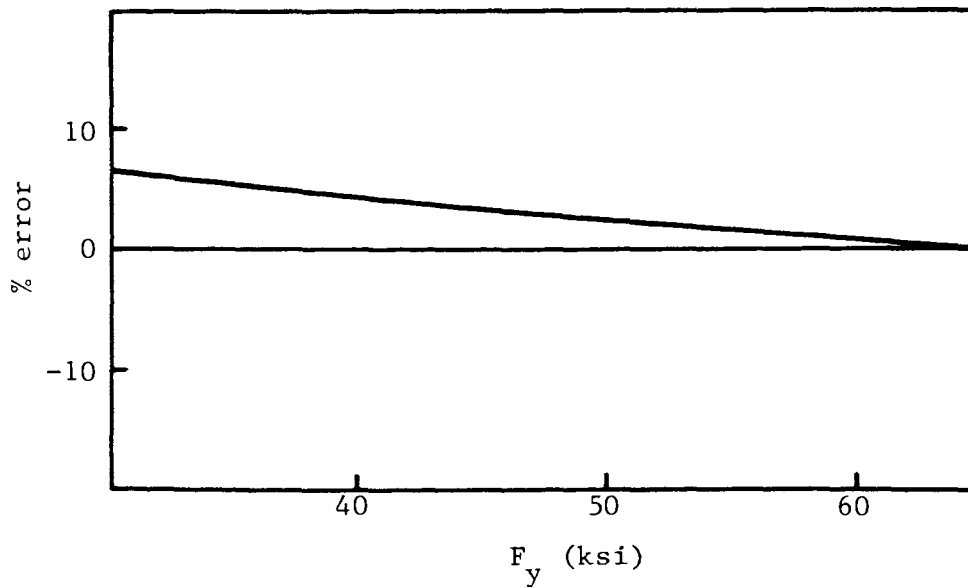


Fig. 10-20 Extrapolation for yield stress from $F_y = 65$ ksi,
 $w/t = 500$

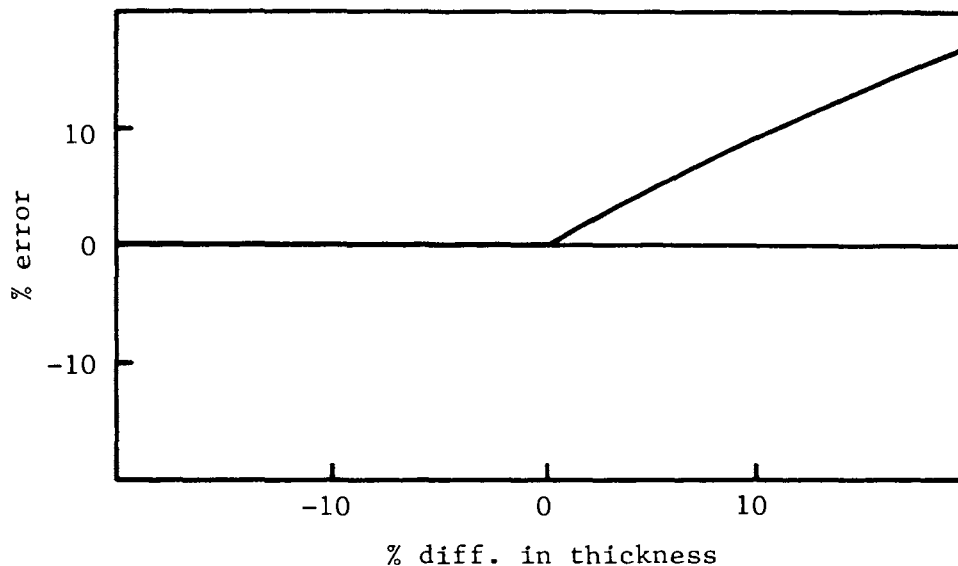


Fig. 10-21 Extrapolation for thickness from $F_y = 55$ ksi,
 $w/t = 20$

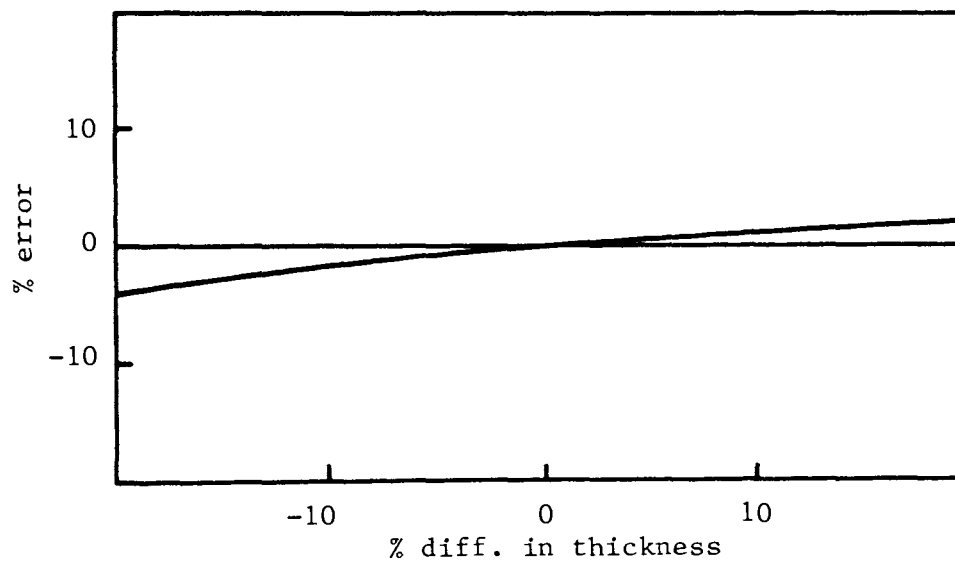


Fig. 10-22 Extrapolation for thickness from $F_y = 35$ ksi,
 $w/t = 100$

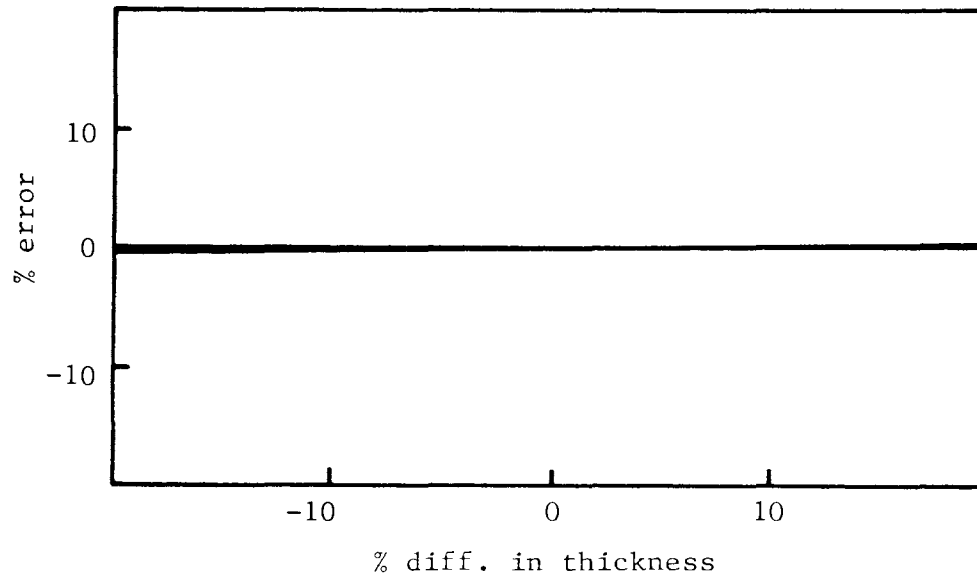


Fig. 10-23 Extrapolation for thickness from $F_y = 35$ ksi,
 $w/t = 500$

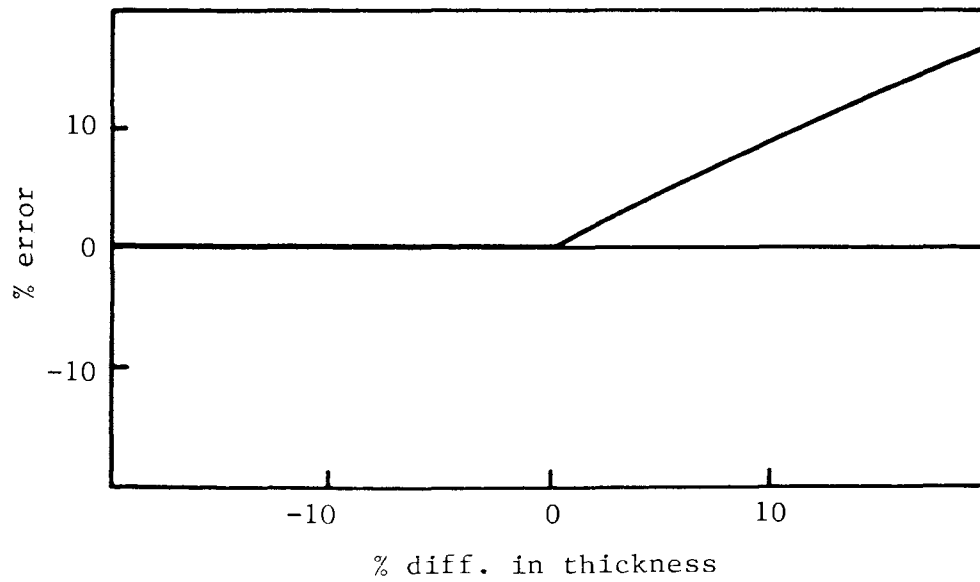


Fig. 10-24 Extrapolation for thickness from $F_y = 55$ ksi,
 $w/t = 20$

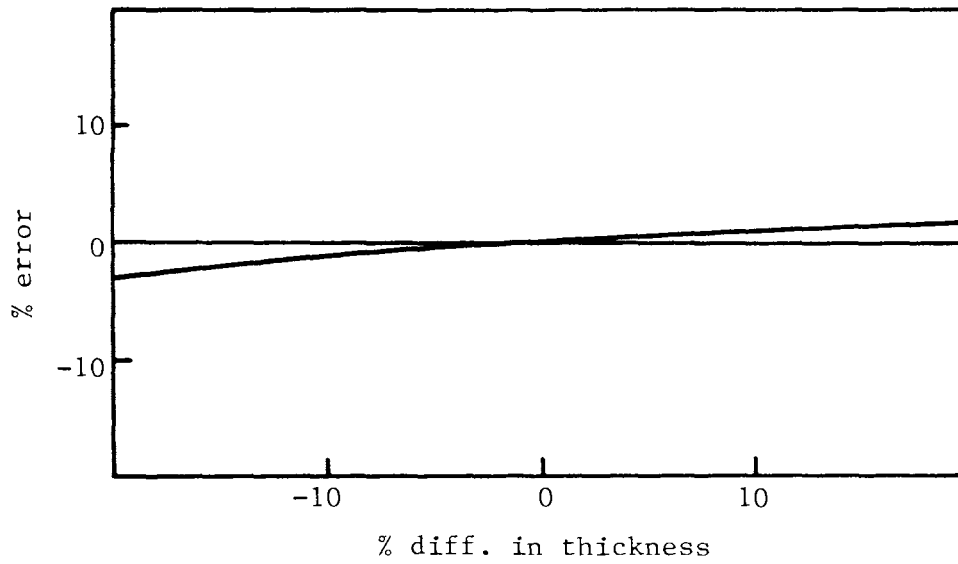


Fig. 10-25 Extrapolation for thickness from $F_y = 55$ ksi,
 $w/t = 100$

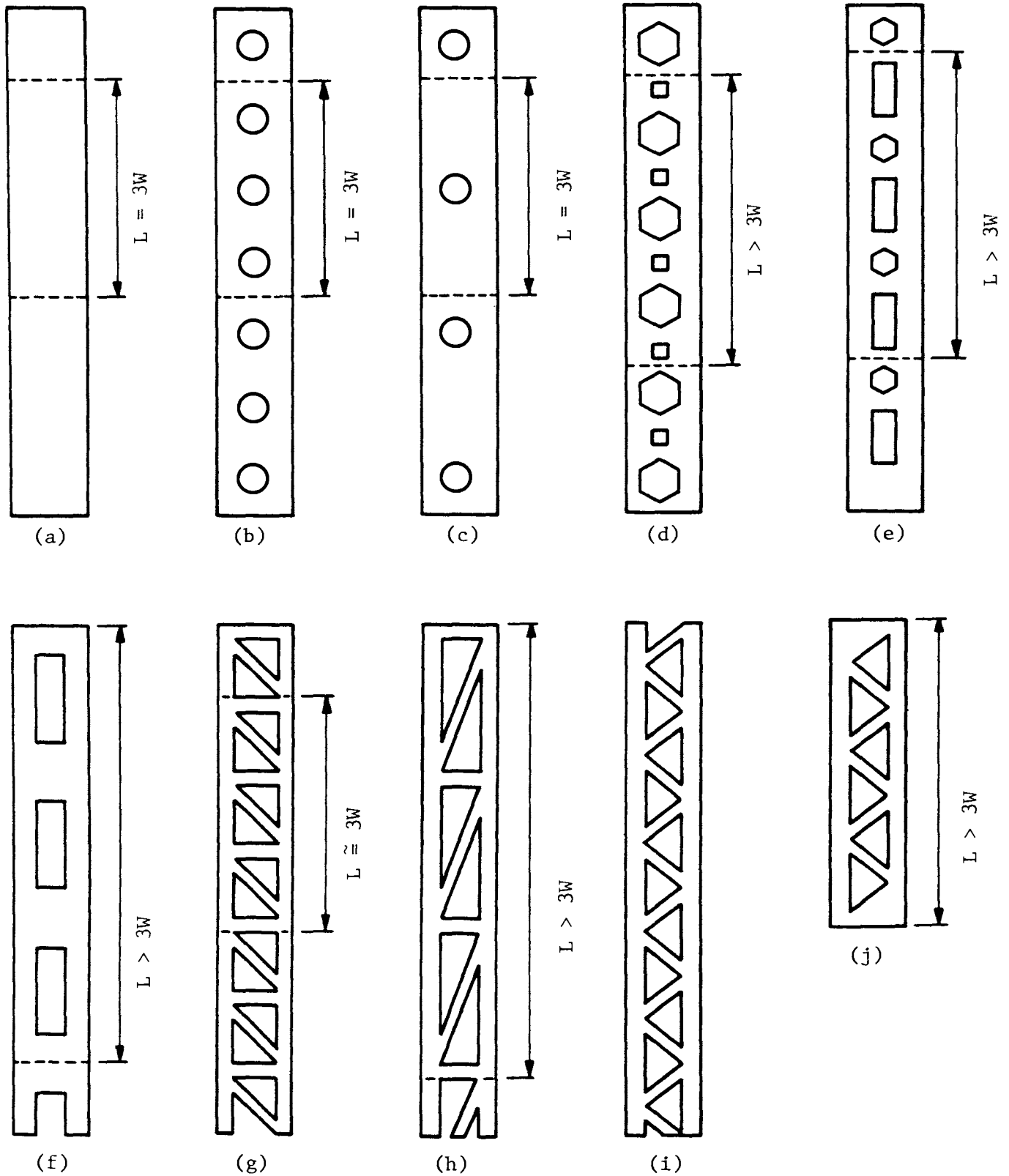


Fig. 10-26 Hypothetical perforation patterns and stub column length. It is assumed that the perforations shown are in the web of a C-Section and the width of the web is W .

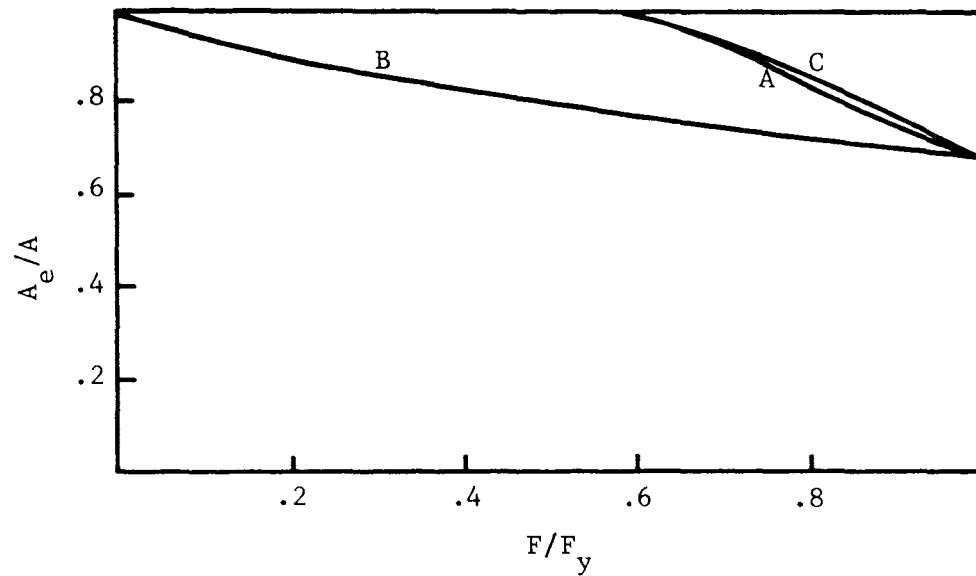


Fig. 10-27 Test no. 1 - 2 5/16 in square tube, .048 in thick
 $F_y = 67.6$ ksi on flats
Curve A - Measured in test
Curve B - Calculated by Eqs. 10-8 and -9
Curve C - Calculated by Eqs. 10-18 and -19

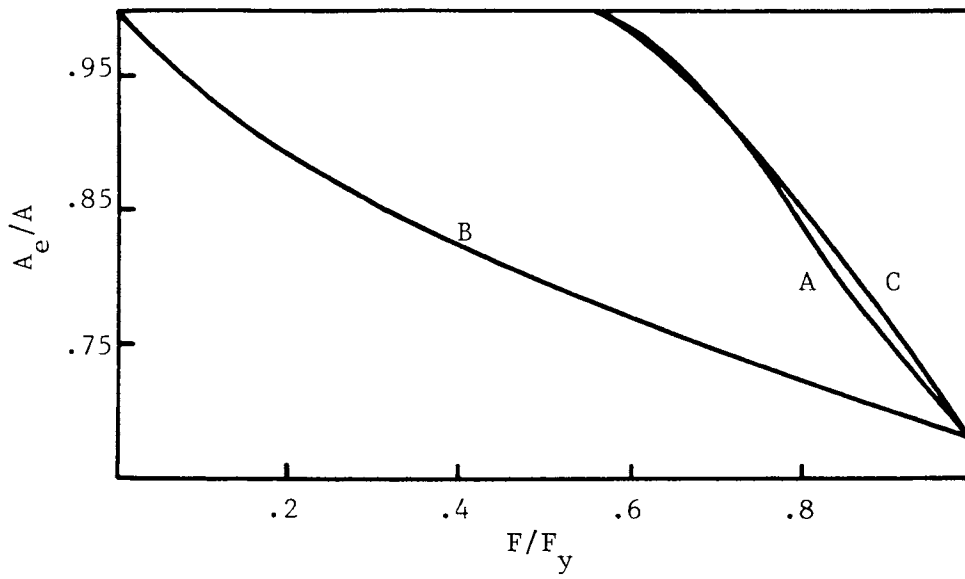


Fig. 10-28 Test no. 1
Fig. 10-27 drawn to a different scale

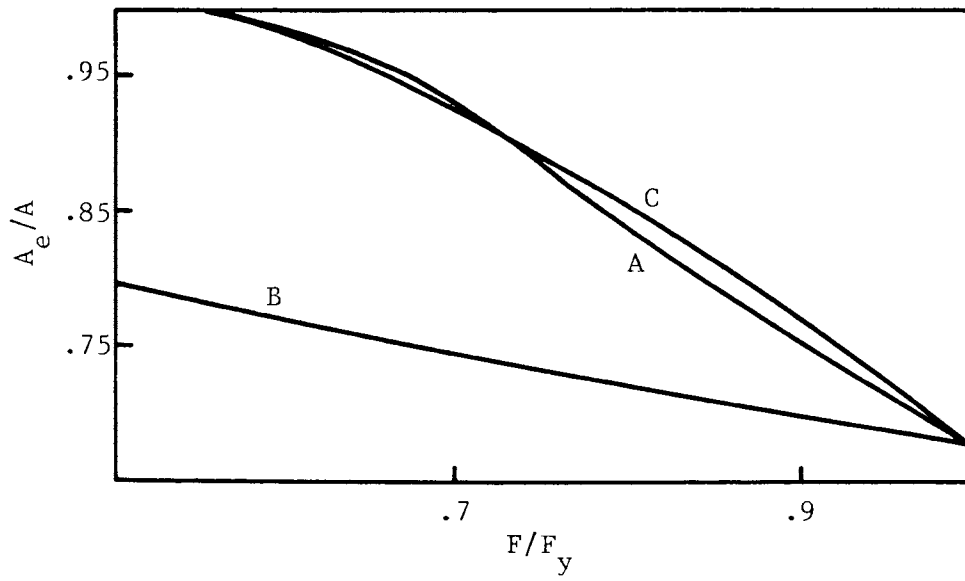


Fig. 10-29 Test no. 1
Fig. 10-27 drawn to a different scale

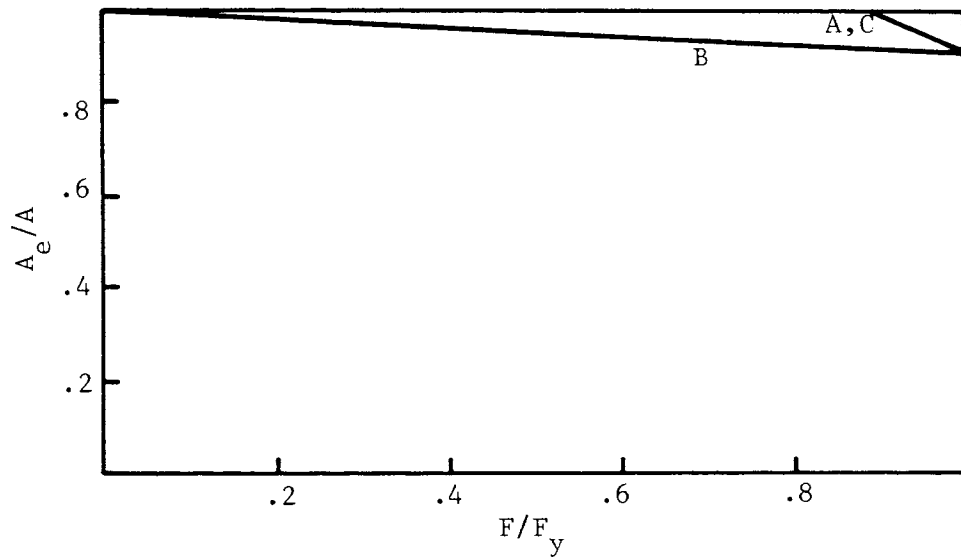


Fig. 10-30 Test no. 2 - 2 5/16 in square tube, .063 in thick, $F_y = 67.5$ ksi on flats
Curve A \bar{y} Measured in test
Curve B - Calculated by Eqs. 10-8 and -9
Curve C - Calculated by Eqs. 10-18 and -19

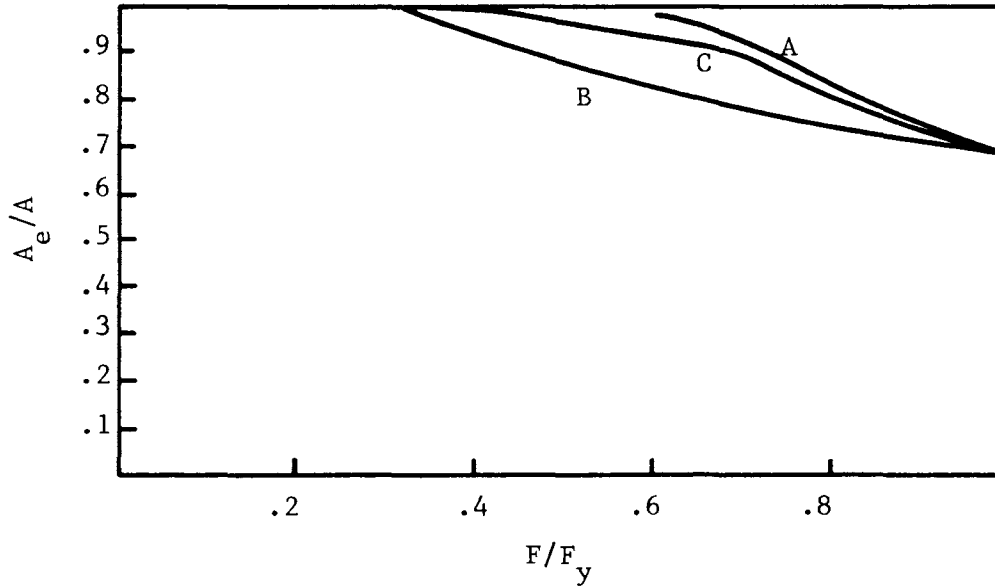


Fig. 10-31 Test no. 1 - 2 5/16 in square tube, 0.48 in thick
Observed and calculated test results
Curve A - Measured in test
Curve B - Calculated according to Procedure (1)
Curve C - Calculated according to Procedure (2)

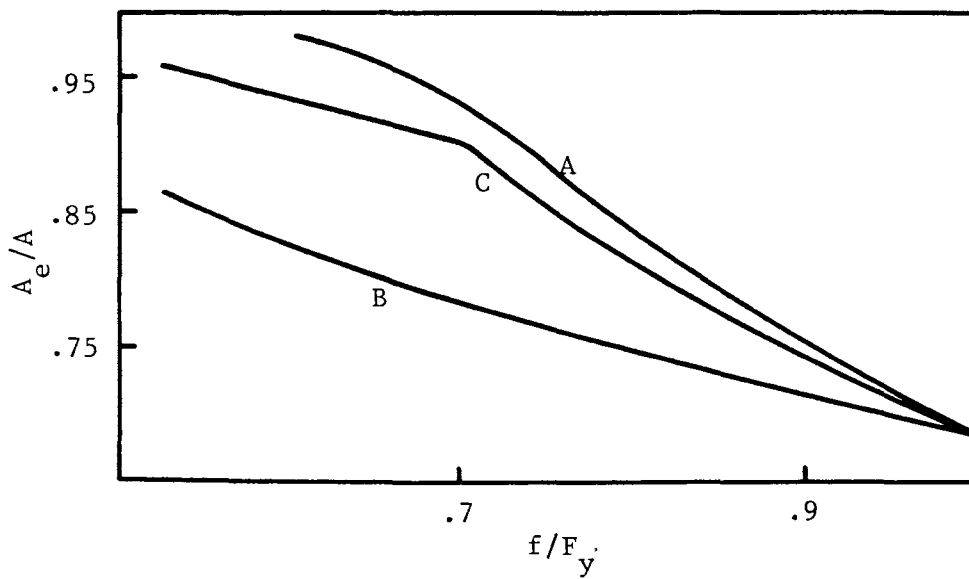


Fig. 10-32 Test no. 1
Fig. 10-31 drawn to a different scale

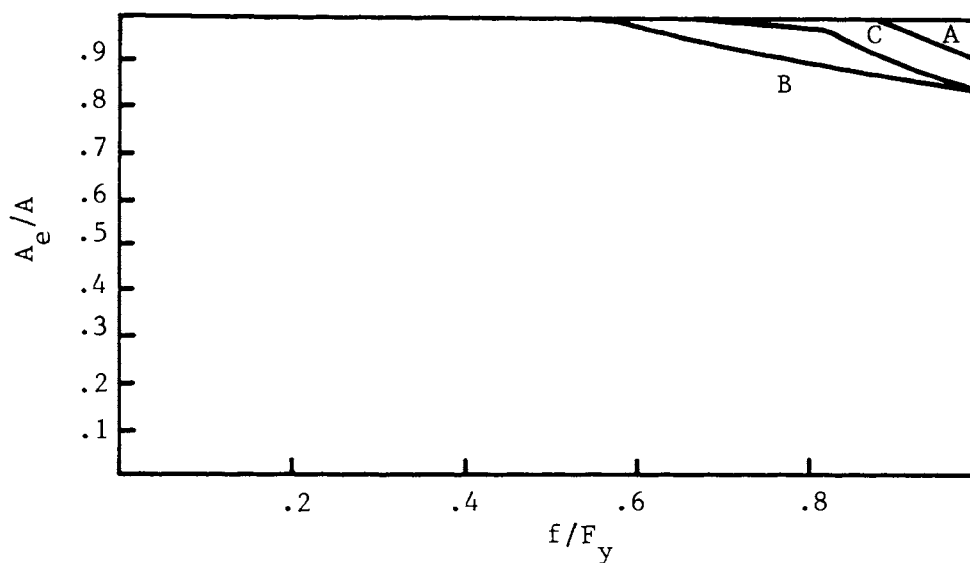


Fig. 10-33 Test no. 2 - 2 5/16 in square tube, .063 in thick
Observed and calculated test results
Curve A - Measured in test
Curve B - Calculated according to Procedure (1)
Curve C - Calculated according to Procedure (2)

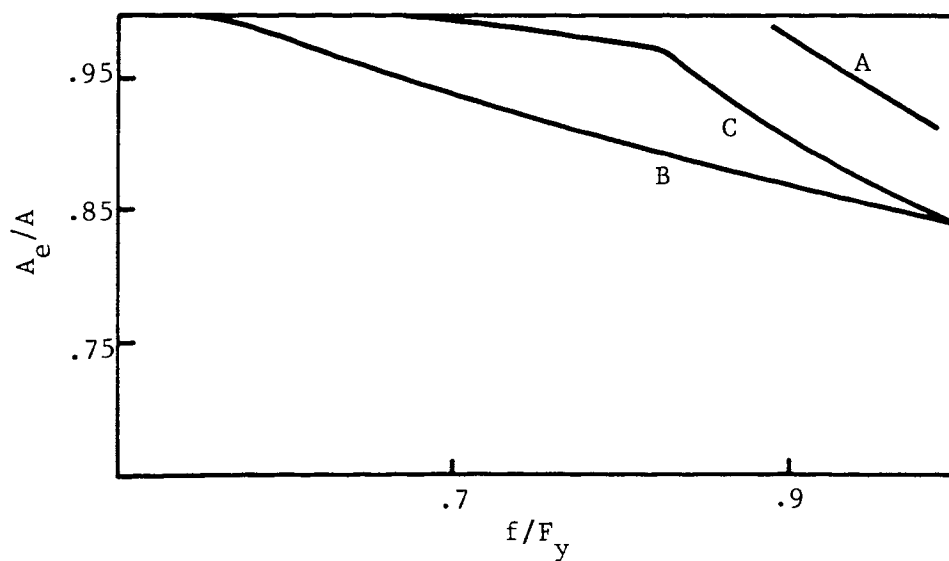


Fig. 10-34 Test no. 2
Fig. 10-33 drawn to a different scale

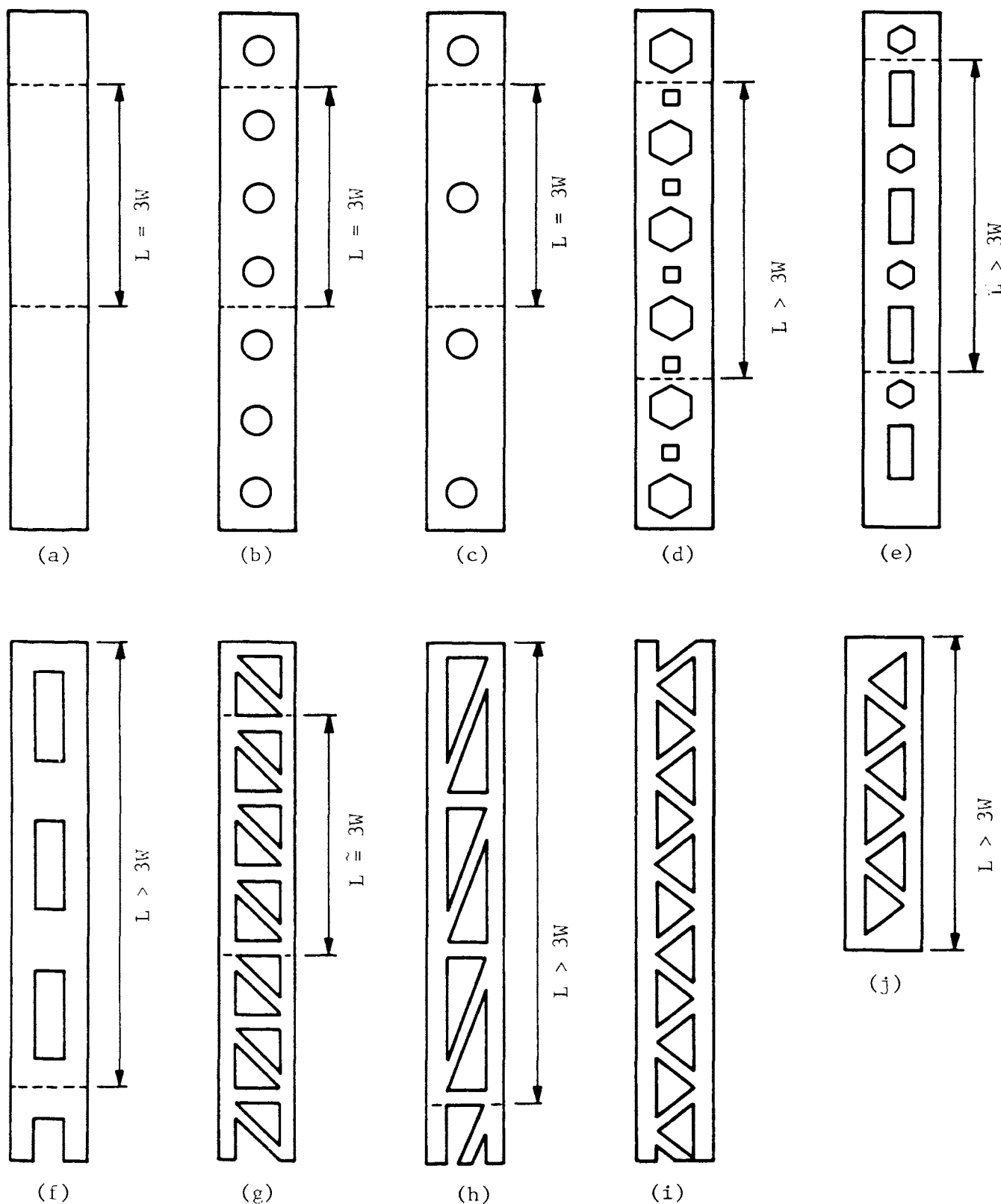


Fig. A1 Hypothetical perforation patterns and stub column length. It is assumed that the perforations shown are in the web of a C-Section and the width of the web is W.