# Load and Resistance Factor Design Specification for Structural Steel Buildings

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# PREFACE

The AISC *Load and Resistance Factor Design (LRFD) Specification for Structural Steel Buildings* is based on reliability theory. As have all AISC Specifications, this LRFD Specification has been based upon past successful usage, advances in the state of knowledge, and changes in design practice. The LRFD Specification has been developed as a consensus document to provide a uniform practice in the design of steel-framed buildings. The intention is to provide design criteria for routine use and not to provide specific criteria for infrequently encountered problems which occur in the full range of structural design. Providing definitive provisions to cover all cases would make the LRFD Specification too cumbersome for routine design usage.

The LRFD Specification is the result of the deliberations of a committee of structural engineers with wide experience and high professional standing, representing a wide geographical distribution throughout the U.S. The committee includes approximately equal numbers of engineers in private practice and code agencies, engineers involved in research and teaching, and engineers employed by steel fabricating and producing companies.

In order to avoid reference to proprietary steels which may have limited availability, only those steels which can be identified by ASTM specifications are approved under this Specification. However, some steels covered by ASTM specifications, but subject to more costly manufacturing and inspection techniques than deemed essential for structures covered by this Specification, are not listed, even though they may provide all the necessary characteristics for reliable usage in structural applications. Approval of such steels in lieu of less expensive steels is left to the owner's representative.

The Appendices to this Specification are an integral part of the Specification.

A non-mandatory Commentary has been prepared to provide background for the Specification provisions and the user is encouraged to consult it.

The principal changes incorporated in this edition of the Specification include:

- Updated web crippling design provisions.
- Recommendations for the use of heavy rolled shapes and welded members made up of thick plates.
- Updated provisions for slender web girders and unsymmetric members.
- Revised provisions for built-up compression members.
- Improved $C_b$ equation.
- Provisions for slip-critical joints designed at factored loads.
- Reorganization and expansion of material on stability of unbraced frames.
- Reorganization and expansion of Chapters F and K.

- Alternative fillet-weld design strength.
- Addition of beam-web opening provisions.

The reader is cautioned that professional judgment must be exercised when data or recommendations in the Specification are applied. The publication of the material contained herein is not intended as a representation or warranty on the part of the American Institute of Steel Construction, Inc.—or any other person named herein—that this information is suitable for general or particular use, or freedom from infringement of any patent or patents. Anyone making use of this information assumes all liability arising from such use. The design and detailing of steel structures is within the expertise of professional individuals who are competent by virtue of education, training, and experience for the application of engineering principles and the provisions of this specification to the design and/or detailing of a particular structure.

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# TABLE OF CONTENTS

**SYMBOLS** . . . 17

**A. GENERAL PROVISIONS** . . . 25

**A1. Scope** . . . 25

**A2. Limits of Applicability** . . . 25

1. Structural Steel Defined . . . 25
2. Types of Construction . . . 25

**A3. Material** . . . 26

1. Structural Steel . . . 26
2. Steel Castings and Forgings . . . 28
3. Bolts, Washers, and Nuts . . . 28
4. Anchor Bolts and Threaded Rods . . . 28
5. Filler Metal and Flux for Welding . . . 29
6. Stud Shear Connectors . . . 29

**A4. Loads and Load Combinations** . . . 29

1. Loads, Load Factors, and Load Combinations . . . 30
2. Impact . . . 30
3. Crane Runway Horizontal Forces . . . 31

**A5. Design Basis** . . . 31

1. Required Strength at Factored Loads . . . 31
2. Limit States . . . 31
3. Design for Strength . . . 32
4. Design for Serviceability and Other Considerations . . . 32

**A6. Referenced Codes and Standards** . . . 32

**A7. Design Documents** . . . 33

1. Plans . . . 33
2. Standard Symbols and Nomenclature . . . 33
3. Notation for Welding . . . 33

**B. DESIGN REQUIREMENTS** . . . 34

**B1. Gross Area** . . . 34

**B2. Net Area** . . . 34

**B3. Effective Net Area for Tension Members** . . . 34

**B4. Stability** . . . 35

**B5. Local Buckling** . . . 36

1. Classification of Steel Sections . . . 36
2. Design by Plastic Analysis . . . 36
3. Slender-Element Compression Sections . . . 37

**B6. Bracing at Supports** . . . 37

**B7. Limiting Slenderness Ratios** . . . 37

**B8. Simple Spans** . . . 37

**B9. End Restraint** . . . 37

**B10. Proportions of Beams and Girders** . . . 37

**C. FRAMES AND OTHER STRUCTURES** . . . . . 41

**C1. Second Order Effects** . . . . . 41

**C2. Frame Stability** . . . . . 41

1. Braced Frames . . . . . 41
2. Unbraced Frames . . . . . 41

**D. TENSION MEMBERS** . . . . . 44

**D1. Design Tensile Strength** . . . . . 44

**D2. Built-Up Members** . . . . . 44

**D3. Pin-Connected Members and Eyebars** . . . . . 45

**E. COLUMNS AND OTHER COMPRESSION MEMBERS** . . . . . 47

**E1. Effective Length and Slenderness Limitations** . . . . . 47

1. Effective Length . . . . . 47
2. Design by Plastic Analysis . . . . . 47

**E2. Design Compressive Strength for Flexural Buckling** . . . . . 47

**E3. Design Compressive Strength for Flexural-Torsional Buckling** . . . . . 48

**E4. Built-Up Members** . . . . . 48

**E5. Pin-Connected Compression Members** . . . . . 51

**F. BEAMS AND OTHER FLEXURAL MEMBERS** . . . . . 52

**F1. Design for Flexure** . . . . . 52

1. Yielding . . . . . 52
2. Lateral-Torsional Buckling . . . . . 53

**F2. Design for Shear** . . . . . 56

**F3. Web-Tapered Members (see Appendix F3)** . . . . . 56

**F4. Beams and Girders with Web Openings** . . . . . 57

**G. PLATE GIRDERS** . . . . . 58

**H. MEMBERS UNDER COMBINED FORCES AND TORSION** . . . . . 59

**H1. Symmetric Members Subject to Bending and Axial Force** . . . . . 59

1. Doubly and Singly Symmetric Members in Flexur and Tension . . . . . 59
2. Doubly and Singly Symmetric Members in Flexure and Compression . . 60

**H2. Unsymmetric Members and Members Under Torsion and Combined Torsion, Flexure, Shear, and/or Axial Force** . . . . . 60

**H3. Alternative Interaction Equations for Members Under Combined Stress (see Appendix H3)** . . . . . 60

**I. COMPOSITE MEMBERS** . . . . . 61

**I1. Design Assumptions** . . . . . 61

**I2. Compression Members** . . . . . 62

1. Limitations . . . . . 62
2. Design Strength . . . . . 63
3. Columns with Multiple Steel Shapes . . . . . 63
4. Load Transfer . . . . . 63

**I3. Flexural Members** . . . . . 64

1. Effective Width . . . . . 64
2. Strength of Beams with Shear Connectors . . . . . 64

3. Strength of Concrete-Encased Beams . . . . . . . . . . . . . . . . . . . . 64
4. Strength During Construction . . . . . . . . . . . . . . . . . . . . . . . 65
5. Formed Steel Deck . . . . . . . . . . . . . . . . . . . . . . . . . . . . 65
6. Design Shear Strength . . . . . . . . . . . . . . . . . . . . . . . . . . 66

**I4. Combined Compression and Flexure** . . . . . . . . . . . . . . . . . . . 66

**I5. Shear Connectors** . . . . . . . . . . . . . . . . . . . . . . . . . . . . 67

1. Materials . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 67
2. Horizontal Shear Force . . . . . . . . . . . . . . . . . . . . . . . . . 67
3. Strength of Stud Shear Connectors . . . . . . . . . . . . . . . . . . . . 68
4. Strength of Channel Shear Connectors . . . . . . . . . . . . . . . . . . 68
5. Required Number of Shear Connectors . . . . . . . . . . . . . . . . . . 68
6. Shear Connector Placement and Spacing . . . . . . . . . . . . . . . . . 68

**I6. Special Cases** . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 69

**J. CONNECTIONS, JOINTS, AND FASTENERS** . . . . . . . . . . . . . . . . . . 70

**J1. General Provisions** . . . . . . . . . . . . . . . . . . . . . . . . . . . 70

1. Design Basis . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 70
2. Simple Connections . . . . . . . . . . . . . . . . . . . . . . . . . . . 70
3. Moment Connections . . . . . . . . . . . . . . . . . . . . . . . . . . . 70
4. Compression Members with Bearing Joints . . . . . . . . . . . . . . . . 70
5. Splices in Heavy Sections . . . . . . . . . . . . . . . . . . . . . . . . 70
6. Beam Copes and Weld Access Holes . . . . . . . . . . . . . . . . . . . 71
7. Minimum Strength of Connections . . . . . . . . . . . . . . . . . . . . 71
8. Placement of Welds and Bolts . . . . . . . . . . . . . . . . . . . . . . 72
9. Bolts in Combination with Welds . . . . . . . . . . . . . . . . . . . . 72
10. High-Strength Bolts in Combination with Rivets . . . . . . . . . . . . . 72
11. Limitations on Bolted and Welded Connections . . . . . . . . . . . . . . 72

**J2. Welds** . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 73

1. Groove Welds . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 73
2. Fillet Welds . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 74
3. Plug and Slot Welds . . . . . . . . . . . . . . . . . . . . . . . . . . . 76
4. Design Strength . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 77
5. Combination of Welds . . . . . . . . . . . . . . . . . . . . . . . . . . 77
6. Matching Weld Metal . . . . . . . . . . . . . . . . . . . . . . . . . . . 77
7. Mixed Weld Metal . . . . . . . . . . . . . . . . . . . . . . . . . . . . 79
8. Preheat for Heavy Shapes . . . . . . . . . . . . . . . . . . . . . . . . 79

**J3. Bolts and Threaded Parts** . . . . . . . . . . . . . . . . . . . . . . . . 79

1. High-Strength Bolts . . . . . . . . . . . . . . . . . . . . . . . . . . . 79
2. Size and Use of Holes . . . . . . . . . . . . . . . . . . . . . . . . . . 79
3. Minimum Spacing . . . . . . . . . . . . . . . . . . . . . . . . . . . . 80
4. Minimum Edge Distance . . . . . . . . . . . . . . . . . . . . . . . . . 81
5. Maximum Spacing and Edge Distance . . . . . . . . . . . . . . . . . . . 82
6. Design Tension or Shear Strength . . . . . . . . . . . . . . . . . . . . 83
7. Combined Tension and Shear in Bearing-Type Connections . . . . . . . . 83
8. High-Strength Bolts in Slip-Critical Connections . . . . . . . . . . . . . 83
9. Combined Tension and Shear in Slip-Critical Connections . . . . . . . . 84
10. Bearing Strength at Bolt Holes . . . . . . . . . . . . . . . . . . . . . 85
11. Long Grips . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 87

**J4. Design Rupture Strength** . . . 87
1. Shear Rupture Strength . . . 87
2. Tension Rupture Strength . . . 87
3. Block Shear Rupture Strength . . . 87
**J5. Connecting Elements** . . . 88
1. Eccentric Connections . . . 88
2. Design Strength of Elements in Tension . . . 88
3. Other Connecting Elements . . . 88
**J6. Fillers** . . . 89
**J7. Splices** . . . 89
**J8. Bearing Strength** . . . 89
**J9. Column Bases and Bearing on Concrete** . . . 90
**J10. Anchor Bolts and Embedments** . . . 90
**K. CONCENTRATED FORCES, PONDING, AND FATIGUE** . . . 91
**K1. Flanges and Webs with Concentrated Forces** . . . 91
1. Design Basis . . . 91
2. Local Flange Bending . . . 91
3. Local Web Yielding . . . 92
4. Web Crippling . . . 92
5. Sidesway Web Buckling . . . 93
6. Compression Buckling of the Web . . . 94
7. Panel-Zone Web Shear . . . 95
8. Unframed Ends of Beams and Girders . . . 96
9. Additional Stiffener Requirements for Concentrated Forces . . . 96
10. Additional Doubler Plate Requirements for Concentrated Forces . . . 96
**K2. Ponding** . . . 97
**K3. Fatigue** . . . 97
**L. SERVICEABILITY DESIGN CONSIDERATIONS** . . . 98
**L1. Camber** . . . 98
**L2. Expansion and Contraction** . . . 98
**L3. Deflections, Vibrations, and Drift** . . . 98
1. Deflections . . . 98
2. Floor Vibrations . . . 98
3. Drift . . . 98
**L4. Connection Slip** . . . 99
**L5. Corrosion** . . . 99
**M. FABRICATION, ERECTION, AND QUALITY CONTROL** . . . 100
**M1. Shop Drawings** . . . 100
**M2. Fabrication** . . . 100
1. Cambering, Curving, and Straightening . . . 100
2. Thermal Cutting . . . 100
3. Planing of Edges . . . 100
4. Welded Construction . . . 101
5. Bolted Construction . . . 101
6. Compression Joints . . . 101

7. Dimensional Tolerances . . . . . . . . . . . . . . . . . . . . . . 101
8. Finish of Column Bases . . . . . . . . . . . . . . . . . . . . . . 101

**M3. Shop Painting** . . . . . . . . . . . . . . . . . . . . . . . . 102
1. General Requirements . . . . . . . . . . . . . . . . . . . . . . 102
2. Inaccessible Surfaces . . . . . . . . . . . . . . . . . . . . . . 102
3. Contact Surfaces . . . . . . . . . . . . . . . . . . . . . . . . 102
4. Finished Surfaces . . . . . . . . . . . . . . . . . . . . . . . . 102
5. Surfaces Adjacent to Field Welds . . . . . . . . . . . . . . . . 102

**M4. Erection** . . . . . . . . . . . . . . . . . . . . . . . . . . . 102
1. Alignment of Column Bases . . . . . . . . . . . . . . . . . . . 102
2. Bracing . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 102
3. Alignment . . . . . . . . . . . . . . . . . . . . . . . . . . . . 103
4. Fit of Column Compression Joints and Base Plates . . . . . . . . 103
5. Field Welding . . . . . . . . . . . . . . . . . . . . . . . . . . 103
6. Field Painting . . . . . . . . . . . . . . . . . . . . . . . . . . 103
7. Field Connections . . . . . . . . . . . . . . . . . . . . . . . . 103

**M5. Quality Control** . . . . . . . . . . . . . . . . . . . . . . . 103
1. Cooperation . . . . . . . . . . . . . . . . . . . . . . . . . . . 103
2. Rejections . . . . . . . . . . . . . . . . . . . . . . . . . . . . 103
3. Inspection of Welding . . . . . . . . . . . . . . . . . . . . . . 104
4. Inspection of Slip-Critical High-Strength Bolted Connections . . . . . . 104
5. Identification of Steel . . . . . . . . . . . . . . . . . . . . . 104

**APPENDICES**

**B. DESIGN REQUIREMENTS** . . . . . . . . . . . . . . . . . . . . . 105

**B5. Local Buckling** . . . . . . . . . . . . . . . . . . . . . . . . 105
1. Classification of Steel Sections . . . . . . . . . . . . . . . . 105
3. Slender-Element Compression Sections . . . . . . . . . . . . . . 105

**E. COLUMNS AND OTHER COMPRESSION MEMBERS** . . . . . . . . . . . . 109

**E3. Design Compressive Strength for Flexural-Torsional Buckling** . . . . . . 109

**F. BEAMS AND OTHER FLEXURAL MEMBERS** . . . . . . . . . . . . . . . 111

**F1. Design for Flexure** . . . . . . . . . . . . . . . . . . . . . . 111

**F2. Design for Shear** . . . . . . . . . . . . . . . . . . . . . . . 113
2. Design Shear Strength . . . . . . . . . . . . . . . . . . . . . . 113
3. Transverse Stiffeners . . . . . . . . . . . . . . . . . . . . . . 113

**F3. Web-Tapered Members** . . . . . . . . . . . . . . . . . . . . . . 118
1. General Requirements . . . . . . . . . . . . . . . . . . . . . . 118
2. Design Tensile Strength . . . . . . . . . . . . . . . . . . . . . 118
3. Design Compressive Strength . . . . . . . . . . . . . . . . . . . 118
4. Design Flexural Strength . . . . . . . . . . . . . . . . . . . . . 119
5. Design Shear Strength . . . . . . . . . . . . . . . . . . . . . . 120
6. Combined Flexure and Axial Force . . . . . . . . . . . . . . . . 120

**G. PLATE GIRDERS** . . . . . . . . . . . . . . . . . . . . . . . . . 122

**G1. Limitations** . . . . . . . . . . . . . . . . . . . . . . . . . . 122

**G2. Design Flexural Strength** . . . . . . . . . . . . . . . . . . . 122

**G3. Design Shear Strength with Tension Field Action** . . . . . . . . . 124

**G4. Transverse Stiffeners** . . . . . 125

**G5. Flexure-Shear Interaction** . . . . . 125

**H. MEMBERS UNDER COMBINED FORCES AND TORSION** . . . . . 127

**H3. Alternative Interaction Equations for Members Und27 Combined Stress** . . . . . 127

**J. CONNECTIONS, JOINTS, AND FASTENERS** . . . . . 129

**J2. Welds** . . . . . 129

4. Design Strength . . . . . 129

**J3. Bolts and Threaded Parts** . . . . . 130

8. High-Strength Bolts in Slip-Critical Connections . . . . . 130
9. Combined Tension and Shear in Slip-Critical Connections . . . . . 130

**K. CONCENTRATED FORCES, PONDING, AND FATIGUE** . . . . . 132

**K2. Ponding** . . . . . 132

**K3. Fatigue** . . . . . 135

1. Loading Conditions; Type and Location of Material . . . . . 135
2. Design Stress Range . . . . . 135
3. Design Strength of Bolts in Tension . . . . . 135

**NUMERICAL VALUES** . . . . . 144

**COMMENTARY**

**A. GENERAL PROVISIONS** . . . . . 162

**A1. Scope** . . . . . 162

**A2. Limits of Applicability** . . . . . 162

2. Types of Construction . . . . . 162

**A3. Material** . . . . . 163

1. Structural Steel . . . . . 163
3. Bolts, Washers, and Nuts . . . . . 164
4. Anchor Bolts and Threaded Rods . . . . . 165
5. Filler Metal and Flux for Welding . . . . . 165

**A4. Loads and Load Combinations** . . . . . 166

1. Loads, Load Factors, and Load Combinations . . . . . 166
2. Impact . . . . . 167

**A5. Design Basis** . . . . . 167

1. Required Strength at Factored Loads . . . . . 167
2. Limit States . . . . . 167
3. Design for Strength . . . . . 168
4. Design for Serviceability and Other Considerations . . . . . 171

**B. DESIGN REQUIREMENTS** . . . . . 172

**B2. Net Area** . . . . . 172

**B3. Effective Net Area for Tension Members** . . . . . 172

**B5. Local Buckling** . . . . . 174

**B7. Limiting Slenderness Ratios** . . . . . 177

**C. FRAMES AND OTHER STRUCTURES** . . . . . 179

**C1. Second Order Effects** . . . . . 179

**C2. Frame Stability** . . . . . . . . . . 183
**D. TENSION MEMBERS** . . . . . . . . . . 191
**D1. Design Tensile Strength** . . . . . . . . . . 191
**D2. Built-Up Members** . . . . . . . . . . 191
**D3. Pin-Connected Members and Eyebars** . . . . . . . . . . 191
**E. COLUMNS AND OTHER COMPRESSION MEMBERS** . . . . . . . . . . 192
**E1. Effective Length and Slenderness Limitations** . . . . . . . . . . 192
1. Effective Length . . . . . . . . . . 192
2. Design by Plastic Analysis . . . . . . . . . . 192
**E2. Design Compressive Strength for Flexural Buckling** . . . . . . . . . . 192
**E3. Design Compressive Strength for Flexural-Torsional Buckling** . . . . . . 193
**E4. Built-Up Members** . . . . . . . . . . 194
**F. BEAMS AND OTHER FLEXURAL MEMBERS** . . . . . . . . . . 195
**F1. Design for Flexure** . . . . . . . . . . 195
1. Yielding . . . . . . . . . . 195
2. Lateral-Torsional Buckling . . . . . . . . . . 195
**F2. Design for Shear** . . . . . . . . . . 199
**F4. Beams and Girders with Web Openings** . . . . . . . . . . 200
**H. MEMBERS UNDER COMBINED FORCES AND TORSION** . . . . . . . . . . 201
**H1. Symmetric Members Subject to Bending and Axial Force** . . . . . . . . . . 201
**H2. Unsymmetric Members and Members Under Torsion and Combined Torsion, Flexure, Shear, and/or Axial Force** . . . . . . . . . . 202
**I. COMPOSITE MEMBERS** . . . . . . . . . . 203
**I1. Design Assumptions** . . . . . . . . . . 203
**I2. Compression Members** . . . . . . . . . . 204
1. Limitations . . . . . . . . . . 204
2. Design Strength . . . . . . . . . . 205
3. Columns with Multiple Steel Shapes . . . . . . . . . . 205
4. Load Transfer . . . . . . . . . . 205
**I3. Flexural Members** . . . . . . . . . . 205
1. Effective Width . . . . . . . . . . 205
2. Strength of Beams with Shear Connectors . . . . . . . . . . 206
3. Strength of Concrete-Encased Beams . . . . . . . . . . 209
4. Strength During Construction . . . . . . . . . . 210
5. Formed Steel Deck . . . . . . . . . . 210
6. Design Shear Strength . . . . . . . . . . 212
**I4. Combined Compression and Flexure** . . . . . . . . . . 213
**I5. Shear Connectors** . . . . . . . . . . 213
1. Materials . . . . . . . . . . 213
2. Horizontal Shear Force . . . . . . . . . . 214
3. Strength of Stud Shear Connectors . . . . . . . . . . 214
4. Strength of Channel Shear Connectors . . . . . . . . . . 214
6. Shear-Connector Placement and Spacing . . . . . . . . . . 214
**I6. Special Cases** . . . . . . . . . . 215

**J. CONNECTIONS, JOINTS, AND FASTENERS** . . . 216
**J1. General Provisions** . . . 216
5. Splices in Heavy Sections . . . 216
8. Placement of Welds and Bolts . . . 217
9. Bolts in Combination with Welds . . . 217
10. High-Strength Bolts in Combination with Rivets . . . 219
**J2. Welds** . . . 219
1. Groove Welds . . . 219
2. Fillet Welds . . . 219
4. Design Strength . . . 222
5. Combination of Welds . . . 223
7. Mixed Weld Metal . . . 223
**J3. Bolts and Threaded Parts** . . . 223
1. High-Strength Bolts . . . 223
2. Size and Use of Holes . . . 224
3. Minimum Spacing . . . 224
4. Minimum Edge Distance . . . 224
5. Maximum Spacing and Edge Distance . . . 225
6. Design Tension or Shear Strength . . . 225
7. Combined Tension and Shear in Bearing-Type Connections . . . 226
8. High-Strength Bolts in Slip-Critical Connections . . . 226
10. Bearing Strength at Bolt Holes . . . 227
11. Long Grips . . . 228
**J4. Design Rupture Strength** . . . 228
**J5. Connecting Elements** . . . 229
2. Design Strength of Connecting Elements in Tension . . . 229
**J6. Fillers** . . . 229
**J8. Bearing Strength** . . . 230
**J9. Column Bases and Bearing on Concrete** . . . 230
**J10. Anchor Bolts and Embedments** . . . 230
**K. CONCENTRATED FORCES, PONDING, AND FATIGUE** . . . 231
**K1. Flanges and Webs with Concentrated Forces** . . . 231
1. Design Basis . . . 231
2. Local Flange Bending . . . 231
3. Local Web Yielding . . . 231
4. Web Crippling . . . 232
5. Sidesway Web Buckling . . . 232
6. Compression Buckling of the Web . . . 233
7. Panel-Zone Web Shear . . . 234
**K2. Ponding** . . . 236
**L. SERVICEABILITY DESIGN CONSIDERATIONS** . . . 239
**L1. Camber** . . . 240
**L2. Expansion and Contraction** . . . 240
**L3. Deflections, Vibration, and Drift** . . . 240
1. Deflections . . . 240

2. Floor Vibration . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 240
3. Drift . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 241

**L5. Corrosion** . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 241

**M. FABRICATION, ERECTION, AND QUALITY CONTROL** . . . . . . . . . . . 242

**M2. Fabrication** . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 242

1. Cambering, Curving, and Straightening . . . . . . . . . . . . . . . . . . 242
2. Thermal Cutting . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 242
5. Bolted Construction . . . . . . . . . . . . . . . . . . . . . . . . . . . 242

**M3. Shop Painting** . . . . . . . . . . . . . . . . . . . . . . . . . . . . 243

5. Surfaces Adjacent to Field Welds . . . . . . . . . . . . . . . . . . . . 243

**M4. Erection** . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 243

4. Fit of Column Compression Joints and Base Plates . . . . . . . . . . . . 243
5. Field Welding . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 243

**APPENDIX B. DESIGN REQUIREMENTS** . . . . . . . . . . . . . . . . . . . . 244

**B5. Local Buckling** . . . . . . . . . . . . . . . . . . . . . . . . . . . . 244

1. Classification of Steel Sections . . . . . . . . . . . . . . . . . . . . 244

**APPENDIX E. COLUMNS AND OTHER COMPRESSION MEMBERS** . . . . . . 245

**E3. Design Compressive Strength for Flexural-Torsional Buckling** . . . . . . 245

**APPENDIX F. BEAMS AND OTHER FLEXURAL MEMBERS** . . . . . . . . . . . 246

**F1. Design for Flexure** . . . . . . . . . . . . . . . . . . . . . . . . . . 246

**F3. Web-Tapered Members** . . . . . . . . . . . . . . . . . . . . . . . . . 247

1. General Requirements . . . . . . . . . . . . . . . . . . . . . . . . . . 247
3. Design Compressive Strength . . . . . . . . . . . . . . . . . . . . . . 247
4. Design Flexural Strength . . . . . . . . . . . . . . . . . . . . . . . . 248

**APPENDIX G. PLATE GIRDERS** . . . . . . . . . . . . . . . . . . . . . . . 249

**G2. Design Flexural Strength** . . . . . . . . . . . . . . . . . . . . . . . 249

**APPENDIX H. MEMBERS UNDER COMBINED FORCES AND TORSION** . . . 250

**H3. Alternative Interaction Equations for Members Under Combined Stress** . . . . . . . . . . . . . . . . . . . . . . . . . . 250

**APPENDIX J. CONNECTIONS, JOINTS, AND FASTENERS** . . . . . . . . . . . 251

**J2. Welds** . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 251

4. Design Strength . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 251

**APPENDIX K. CONCENTRATED FORCES, PONDING, AND FATIGUE** . . . . 254

**K3. Fatigue** . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 254

**REFERENCES** . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 257

**GLOSSARY** . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 267

# Symbols

The section number in parentheses after the definition of a symbol refers to the section where the symbol is first defined.

| | |
|---|---|
| $A$ | Cross-sectional area, in.$^2$ (F1.2) |
| $A_B$ | Loaded area of concrete, in.$^2$ (I2.4) |
| $A_b$ | Nominal body area of a fastener, in.$^2$ (J3.7) |
| $A_c$ | Area of concrete, in.$^2$ (I2.2) |
| $A_c$ | Area of concrete slab within effective width, in.$^2$ (I5.2) |
| $A_D$ | Area of an upset rod based on the major diameter of its threads, in.$^2$ (J3.6) |
| $A_e$ | Effective net area, in.$^2$ (B3) |
| $A_f$ | Area of flange, in.$^2$ (Appendix F3) |
| $A_{fe}$ | Effective tension flange area, in.$^2$ (B10) |
| $A_{fg}$ | Gross area of flange, in.$^2$ (B10) |
| $A_{fn}$ | Net area of flange, in.$^2$ (B10) |
| $A_g$ | Gross area, in.$^2$ (A5) |
| $A_{gt}$ | Gross area subject to tension, in.$^2$ (J4.3) |
| $A_{gv}$ | Gross area subject to shear, in.$^2$ (J4.3) |
| $A_n$ | Net area, in.$^2$ (B2) |
| $A_{nt}$ | Net area subject to tension, in.$^2$ (J4.2) |
| $A_{nv}$ | Net area subject to shear, in.$^2$ (J4.1) |
| $A_{pb}$ | Projected bearing area, in.$^2$ (J8.1) |
| $A_r$ | Area of reinforcing bars, in.$^2$ (I2.2) |
| $A_s$ | Area of steel cross section, in.$^2$ (I2.2) |
| $A_{sc}$ | Cross-sectional area of stud shear connector, in.$^2$ (I5.3) |
| $A_{sf}$ | Shear area on the failure path, in.$^2$ (D3) |
| $A_w$ | Web area, in.$^2$ (F2.1) |
| $A_1$ | Area of steel bearing concentrically on a concrete support, in.$^2$ (J9) |
| $A_2$ | Total cross-sectional area of a concrete support, in.$^2$ (J9) |
| $B$ | Factor for bending stress in tees and double angles (F1.2) |
| $B$ | Factor for bending stress in web-tapered members, in., defined by Equations A-F3-8 through A-F3-11 (Appendix F3) |
| $B_1, B_2$ | Factors used in determining $M_u$ for combined bending and axial forces when first-order analysis is employed (C1) |
| $C_{PG}$ | Plate-girder coefficient (Appendix G2) |
| $C_b$ | Bending coefficient dependent on moment gradient (F1.2a) |
| $C_m$ | Coefficient applied to bending term in interaction formula for prismatic members and dependent on column curvature caused by applied moments (C1) |

| | |
|---|---|
| $C'_m$ | Coefficient applied to bending term in interaction formula for tapered members and dependent on axial stress at the small end of the member (Appendix F3) |
| $C_p$ | Ponding flexibility coefficient for primary member in a flat roof (K2) |
| $C_s$ | Ponding flexibility coefficient for secondary member in a flat roof (K2) |
| $C_v$ | Ratio of "critical" web stress, according to linear buckling theory, to the shear yield stress of web material (Appendix G3) |
| $C_w$ | Warping constant, in.$^6$ (F1.2) |
| $D$ | Outside diameter of circular hollow section, in. (Appendix B5.3) |
| $D$ | Dead load due to the weight of the structural elements and permanent features on the structure (A4.1) |
| $D$ | Factor used in Equation A-G4-2, dependent on the type of transverse stiffeners used in a plate girder (Appendix G4) |
| $E$ | Modulus of elasticity of steel ($E$ = 29,000 ksi) (E2) |
| $E$ | Earthquake load (A4.1) |
| $E_c$ | Modulus of elasticity of concrete, ksi (I2.2) |
| $E_m$ | Modified modulus of elasticity, ksi (I2.2) |
| $F_{BM}$ | Nominal strength of the base material to be welded, ksi (J2.4) |
| $F_{EXX}$ | Classification number of weld metal (minimum specified strength), ksi (J2.4) |
| $F_L$ | Smaller of $(F_{yf} - F_r)$ or $F_{yw}$, ksi (F1.2) |
| $F_{b\gamma}$ | Flexural stress for tapered members defined by Equations A-F3-4 and A-F3-5 (Appendix F3) |
| $F_{cr}$ | Critical stress, ksi (E2) |
| $F_{crft}$, $F_{cry}$, $F_{crz}$ | Flexural-torsional buckling stresses for double-angle and tee-shaped compression members, ksi (E3) |
| $F_e$ | Elastic buckling stress, ksi (Appendix E3) |
| $F_{ex}$ | Elastic flexural buckling stress about the major axis, ksi (Appendix E3) |
| $F_{ey}$ | Elastic flexural buckling stress about the minor axis, ksi (Appendix E3) |
| $F_{ez}$ | Elastic torsional buckling stress, ksi (Appendix E3) |
| $F_{my}$ | Modified yield stress for composite columns, ksi (I2.2) |
| $F_n$ | Nominal shear rupture strength, ksi (J4) |
| $F_r$ | Compressive residual stress in flange (10 ksi for rolled; 16.5 ksi for welded), ksi (Table B5.1) |
| $F_{s\gamma}$ | Stress for tapered members defined by Equation A-F3-6, ksi (Appendix F3) |
| $F_u$ | Specified minimum tensile strength of the type of steel being used, ksi (B10) |
| $F_w$ | Nominal strength of the weld electrode material, ksi (J2.4) |
| $F_{w\gamma}$ | Stress for tapered members defined by Equation A-F3-7, ksi (Appendix F3) |
| $F_y$ | Specified minimum yield stress of the type of steel being used, ksi. As used in this Specification, "yield stress" denotes either the specified minimum yield point (for those steels that have a yield point) or specified yield strength (for those steels that do not have a yield point) (A5) |
| $F_{yf}$ | Specified minimum yield stress of the flange, ksi (Table B5.1) |
| $F_{yr}$ | Specified minimum yield stress of reinforcing bars, ksi (I2.2) |
| $F_{yst}$ | Specified minimum yield stress of the stiffener material, ksi (Appendix G4) |

| | |
|---|---|
| $F_{yw}$ | Specified minimum yield stress of the web, ksi (Table B5.1) |
| $G$ | Shear modulus of elasticity of steel, ksi ($G$ = 11,200) (F1.2) |
| $H$ | Horizontal force, kips (C1) |
| $H$ | Flexural constant (E3) |
| $H_s$ | Length of stud connector after welding, in. (I3.5) |
| $I$ | Moment of inertia, in.$^4$ (F1.2) |
| $I_d$ | Moment of inertia of the steel deck supported on secondary members, in.$^4$ (K2) |
| $I_p$ | Moment of inertia of primary members, in.$^4$ (K2) |
| $I_s$ | Moment of inertia of secondary members, in.$^4$ (K2) |
| $I_{st}$ | Moment of inertia of a transverse stiffener, in.$^4$ (Appendix G4) |
| $I_{yc}$ | Moment of inertia about y axis referred to compression flange, or if reverse curvature bending referred to smaller flange, in.$^4$ (Appendix F1) |
| $J$ | Torsional constant for a section, in.$^4$ (F1.2) |
| $K$ | Effective length factor for prismatic member (B7) |
| $K_z$ | Effective length factor for torsional buckling (Appendix E3) |
| $K_\gamma$ | Effective length factor for a tapered member (Appendix F3) |
| $L$ | Story height, in. (C1) |
| $L$ | Length of connection in the direction of loading, in. (B3) |
| $L$ | Live load due to occupancy and moveable equipment (A4.1) |
| $L_b$ | Laterally unbraced length; length between points which are either braced against lateral displacement of compression flange or braced against twist of the cross section, in. (F1.2) |
| $L_c$ | Length of channel shear connector, in. (I5.4) |
| $L_e$ | Edge distance, in. (J3.10) |
| $L_p$ | Limiting laterally unbraced length for full plastic bending capacity, uniform moment case ($C_b$ = 1.0), in. (F1.2) |
| $L_p$ | Column spacing in direction of girder, ft (K2) |
| $L_{pd}$ | Limiting laterally unbraced length for plastic analysis, in. (F1.2) |
| $L_r$ | Limiting laterally unbraced length for inelastic lateral-torsional buckling, in. (F1.2) |
| $L_r$ | Roof live load (A4.1) |
| $L_s$ | Column spacing perpendicular to direction of girder, ft (K2) |
| $M_A$ | Absolute value of moment at quarter point of the unbraced beam segment, kip-in. (F1.2) |
| $M_B$ | Absolute value of moment at centerline of the unbraced beam segment, kip-in. (F1.2) |
| $M_C$ | Absolute value of moment at three-quarter point of the unbraced beam segment, kip-in. (F1.2) |
| $M_{cr}$ | Elastic buckling moment, kip-in. (F1.2) |
| $M_{lt}$ | Required flexural strength in member due to lateral frame translation only, kip-in. (C1) |
| $M_{max}$ | Absolute value of maximum moment in the unbraced beam segment, kip-in. (F1.2) |
| $M_n$ | Nominal flexural strength, kip-in. (F1.1) |
| $M'_{nx}$, $M'_{ny}$ | Flexural strength defined in Equations A-H3-7 and A-H3-8 for use in alternate interaction equations for combined bending and axial force, kip-in. (Appendix H3) |

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|---|---|
| $M_{nt}$ | Required flexural strength in member assuming there is no lateral translation of the frame, kip-in. (C1) |
| $M_p$ | Plastic bending moment, kip-in. (F1.1) |
| $M_p'$ | Moment defined in Equations A-H3-5 and A-H3-6, for use in alternate interaction equations for combined bending and axial force, kip-in. (Appendix H3) |
| $M_r$ | Limiting buckling moment, $M_{cr}$, when $\lambda = \lambda_r$ and $C_b = 1.0$, kip-in. (F1.2) |
| $M_u$ | Required flexural strength, kip-in. (C1) |
| $M_y$ | Moment corresponding to onset of yielding at the extreme fiber from an elastic stress distribution ($= F_y S$ for homogeneous sections), kip-in. (F1.1) |
| $M_1$ | Smaller moment at end of unbraced length of beam or beam-column, kip-in. |
| $M_2$ | Larger moment at end of unbraced length of beam or beam-column, kip-in. |
| $N$ | Length of bearing, in. (K1.3) |
| $N_r$ | Number of stud connectors in one rib at a beam intersection (I3.5) |
| $P_{e1}$, $P_{e2}$ | Elastic Euler buckling load for braced and unbraced frame, respectively, kips (C1) |
| $P_n$ | Nominal axial strength (tension or compression), kips (D1) |
| $P_p$ | Bearing load on concrete, kips (J9) |
| $P_u$ | Required axial strength (tension or compression), kips (Table B5.1) |
| $P_y$ | Yield strength, kips (Table B5.1) |
| $Q$ | Full reduction factor for slender compression elements (Appendix E3) |
| $Q_a$ | Reduction factor for slender stiffened compression elements (Appendix B5) |
| $Q_n$ | Nominal strength of one stud shear connector, kips (I5) |
| $Q_s$ | Reduction factor for slender unstiffened compression elements (Appendix B5.3) |
| $R$ | Load due to initial rainwater or ice exclusive of the ponding contribution (A4.1) |
| $R_{PG}$ | Plate girder bending strength reduction factor (Appendix G) |
| $R_e$ | Hybrid girder factor (Appendix F1) |
| $R_n$ | Nominal strength (A5.3) |
| $R_v$ | Web shear strength, kips (K1.7) |
| $S$ | Elastic section modulus, in.$^3$ (F1.2) |
| $S$ | Spacing of secondary members, ft (K2) |
| $S$ | Snow load (A4.1) |
| $S_x'$ | Elastic section modulus of larger end of tapered member about its major axis, in.$^3$ (Appendix F3) |
| $S_{eff}$ | Effective section modulus about major axis, in.$^3$ (Appendix F1) |
| $S_{xt}$, $S_{xc}$ | Elastic section modulus referred to tension and compression flanges, respectively, in.$^3$ (Appendix F1) |
| $T$ | Tension force due to service loads, kips (J3.9) |
| $T_b$ | Specified pretension load in high-strength bolt, kips (J3.9) |
| $T_u$ | Required tensile strength due to factored loads, kips (Appendix J3.9b) |
| $U$ | Reduction coefficient, used in calculating effective net area (B3) |
| $V_n$ | Nominal shear strength, kips (F2.2) |

| | |
|---|---|
| $V_u$ | Required shear strength, kips (Appendix G4) |
| $W$ | Wind load (A4.1) |
| $X_1$ | Beam buckling factor defined by Equation F1-8 (F1.2) |
| $X_2$ | Beam buckling factor defined by Equation F1-9 (F1.2) |
| $Z$ | Plastic section modulus, in.$^3$ (F1.1) |
| $a$ | Clear distance between transverse stiffeners, in. (Appendix F2.2) |
| $a$ | Distance between connectors in a built-up member, in. (E4) |
| $a$ | Shortest distance from edge of pin hole to edge of member measured parallel to direction of force, in. (D3) |
| $a_r$ | Ratio of web area to compression flange area (Appendix G2) |
| $a'$ | Weld length, in. (B10) |
| $b$ | Compression element width, in. (B5.1) |
| $b_e$ | Reduced effective width for slender compression elements, in. (Appendix B5.3) |
| $b_{eff}$ | Effective edge distance, in. (D3) |
| $b_f$ | Flange width, in. (B5.1) |
| $c_1, c_2, c_3$ | Numerical coefficients (I2.2) |
| $d$ | Nominal fastener diameter, in. (J3.3) |
| $d$ | Overall depth of member, in. (B5.1) |
| $d$ | Pin diameter, in. (D3) |
| $d$ | Roller diameter, in. (J8.2) |
| $d_L$ | Depth at larger end of unbraced tapered segment, in. (Appendix F3) |
| $d_b$ | Beam depth, in. (K1.7) |
| $d_c$ | Column depth, in. (K1.7) |
| $d_o$ | Depth at smaller end of unbraced tapered segment, in. (Appendix F3) |
| $e$ | Base of natural logarithm = 2.71828. . . |
| $f$ | Computed compressive stress in the stiffened element, ksi (Appendix B5.3) |
| $f_{b1}$ | Smallest computed bending stress at one end of a tapered segment, ksi (Appendix F3) |
| $f_{b2}$ | Largest computed bending stress at one end of a tapered segment, ksi (Appendix F3) |
| $f_c'$ | Specified compressive strength of concrete, ksi (I2.2) |
| $f_o$ | Stress due to $1.2D + 1.2R$, ksi (Appendix K2) |
| $f_{un}$ | Required normal stress, ksi (H2) |
| $f_{uv}$ | Required shear stress, ksi (H2) |
| $f_v$ | Required shear stress due to factored loads in bolts or rivets, ksi (J3.7) |
| $g$ | Transverse center-to-center spacing (gage) between fastener gage lines, in. (B2) |
| $h$ | Clear distance between flanges less the fillet or corner radius for rolled shapes; and for built-up sections, the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used, in. (B5.1) |
| $h$ | Distance between centroids of individual components perpendicular to the member axis of buckling, in. (E4) |
| $h_c$ | Twice the distance from the centroid to the following: the inside face of the compression flange less the fillet or corner radius, for rolled shapes; the nearest line of fasteners at the compression flange or the inside faces |

| | |
|---|---|
| | of the compression flange when welds are used, for built-up sections, in., (B5.1) |
| $h_r$ | Nominal rib height, in. (I3.5) |
| $h_s$ | Factor used in Equation A-F3-6 for web-tapered members (Appendix F3) |
| $h_w$ | Factor used in Equation A-F3-7 for web-tapered members (Appendix F3) |
| $j$ | Factor defined by Equation A-F2-4 for minimum moment of inertia for a transverse stiffener (Appendix F2.3) |
| $k$ | Distance from outer face of flange to web toe of fillet, in. (K1.3) |
| $k_v$ | Web plate buckling coefficient (Appendix F2.2) |
| $l$ | Laterally unbraced length of member at the point of load, in. (B7) |
| $l$ | Length of bearing, in. (J8.2) |
| $l$ | Length of connection in the direction of loading, in. (B3) |
| $l$ | Length of weld, in. (B3) |
| $m$ | Ratio of web to flange yield stress or critical stress in hybrid beams (Appendix G2) |
| $r$ | Governing radius of gyration, in. (B7) |
| $r_{To}$ | For the smaller end of a tapered member, the radius of gyration, considering only the compression flange plus one-third of the compression web area, taken about an axis in the plane of the web, in. (Appendix F3.4) |
| $r_i$ | Minimum radius of gyration of individual component in a built-up member, in. (E4) |
| $r_{ib}$ | Radius of gyration of individual component relative to centroidal axis parallel to member axis of buckling, in. (E4) |
| $r_m$ | Radius of gyration of the steel shape, pipe, or tubing in composite columns. For steel shapes it may not be less than 0.3 times the overall thickness of the composite section, in. (I2) |
| $\bar{r}_o$ | Polar radius of gyration about the shear center, in. (E3) |
| $r_{ox}$ , $r_{oy}$ | Radius of gyration about x and y axes at the smaller end of a tapered member, respectively, in. (Appendix F3.3) |
| $r_x$, $r_y$ | Radius of gyration about x and y axes, respectively, in. (E3) |
| $r_{yc}$ | Radius of gyration about y axis referred to compression flange, or if reverse curvature bending, referred to smaller flange, in. (Appendix F1) |
| $s$ | Longitudinal center-to-center spacing (pitch) of any two consecutive holes, in. (B2) |
| $t$ | Thickness of connected part, in. (D3) |
| $t_f$ | Flange thickness, in. (B5.1) |
| $t_f$ | Flange thickness of channel shear connector, in. (I5.4) |
| $t_w$ | Web thickness of channel shear connector, in. (I5.4) |
| $t_w$ | Web thickness, in. (B5.3) |
| $w$ | Plate width; distance between welds, in. (B3) |
| $w$ | Unit weight of concrete, lbs/cu ft. (I2) |
| $w_r$ | Average width of concrete rib or haunch, in. (I3.5) |
| $x$ | Subscript relating symbol to strong axis bending |
| $x_o$, $y_o$ | Coordinates of the shear center with respect to the centroid, in. (E3) |
| $\bar{x}$ | Connection eccentricity, in. (B3) |
| $y$ | Subscript relating symbol to weak axis bending |
| $z$ | Distance from the smaller end of tapered member used in Equation A-F3-1 for the variation in depth, in. (Appendix F3) |

| | |
|---|---|
| $\alpha$ | Separation ratio for built-up compression members = $\frac{h}{2r_{ib}}$ (E4) |
| $\Delta_{oh}$ | Translation deflection of the story under consideration, in. (C1) |
| $\gamma$ | Depth tapering ratio (Appendix F3). Subscript for tapered members (Appendix F3) |
| $\zeta$ | Exponent for alternate beam-column interaction equation (Appendix H3) |
| $\eta$ | Exponent for alternate beam-column interaction equation (Appendix H3) |
| $\lambda_c$ | Column slenderness parameter (C1) |
| $\lambda_e$ | Equivalent slenderness parameter (Appendix E3) |
| $\lambda_{eff}$ | Effective slenderness ratio defined by Equation A-F3-2 (Appendix F3) |
| $\lambda_p$ | Limiting slenderness parameter for compact element (B5.1) |
| $\lambda_r$ | Limiting slenderness parameter for noncompact element (B5.1) |
| $\phi$ | Resistance factor (A5.3) |
| $\phi_b$ | Resistance factor for flexure (F1) |
| $\phi_c$ | Resistance factor for compression (A5) |
| $\phi_c$ | Resistance factor for axially loaded composite columns (I2.2) |
| $\phi_{sf}$ | Resistance factor for shear on the failure path (D3) |
| $\phi_t$ | Resistance factor for tension (D1) |
| $\phi_v$ | Resistance factor for shear (F2.2) |

# CHAPTER A

# GENERAL PROVISIONS

## A1. SCOPE

The *Load and Resistance Factor Design Specification for Structural Steel Buildings* governs the design, fabrication, and erection of steel-framed buildings. As an alternative, the AISC *Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design* is permitted.

## A2. LIMITS OF APPLICABILITY

### 1. Structural Steel Defined

As used in this Specification, the term *structural steel* refers to the steel elements of the structural steel frame essential to the support of the required loads. Such elements are enumerated in Section 2.1 of the AISC *Code of Standard Practice for Steel Buildings and Bridges*. For the design of cold-formed steel structural members, whose profiles contain rounded corners and slender flat elements, the provisions of the American Iron and Steel Institute *Load and Resistance Factor Design Specification for the Design of Cold-Formed Steel Structural Members* are recommended.

### 2. Types of Construction

Two basic types of construction and associated design assumptions are permissible under the conditions stated herein, and each will govern in a specific manner the strength of members and the types and strength of their connections.

Type FR (fully restrained), commonly designated as "rigid-frame" (continuous frame), assumes that connections have sufficient rigidity to maintain the angles between intersecting members.

Type PR (partially restrained) assumes that connections have insufficient rigidity to maintain the angles between intersecting members.

The type of construction assumed in the design shall be indicated on the design documents. The design of all connections shall be consistent with the assumption.

Type PR construction under this Specification depends upon a predictable proportion of full end restraint. When a portion of the full end restraint of members is used in the design for strength of the connected members or for the stability of the structure as a whole, the capacity of the connections to provide the needed restraint shall be documented in the technical literature or established by analytical or empirical means.

When the connection restraint is ignored, commonly designated "simple framing," it is assumed that for the transmission of gravity loads the ends of the beams and girders are connected for shear only and are free to rotate. For "simple framing" the following requirements apply:

(1) The connections and connected members shall be adequate to resist the factored gravity loads as "simple beams."

(2) The connections and connected members shall be adequate to resist the factored lateral loads.

(3) The connections shall have sufficient inelastic rotation capacity to avoid overload of fasteners or welds under combined factored gravity and lateral loading.

Type PR construction may necessitate some inelastic, but self-limiting, deformation of a structural steel part.

## A3. MATERIAL

### 1. Structural Steel

### 1a. ASTM Designations

Material conforming to one of the following standard specifications is approved for use under this Specification:

Structural Steel, ASTM A36
Pipe, Steel, Black and Hot-Dipped, Zinc-Coated Welded and Seamless ASTM A53, Gr. B
High-Strength Low-Alloy Structural Steel, ASTM A242
Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes, ASTM A500
Hot-Formed Welded and Seamless Carbon Steel Structural Tubing, ASTM A501
High-Yield-Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding, ASTM A514
High-Strength Carbon-Manganese Steel of Structural Quality, ASTM A529
Steel, Sheet and Strip, Carbon, Hot-Rolled, Structural Quality, ASTM A570, Gr. 40, 45, and 50
High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality, ASTM A572
High-Strength Low-Alloy Structural Steel with 50 ksi Minimum Yield Point to 4-in. Thick, ASTM A588
Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance, ASTM A606
Steel, Sheet and Strip, High-Strength, Low-Alloy, Columbium or Vanadium, or Both, Hot-Rolled and Cold-Rolled, ASTM A607
Hot-Formed Welded and Seamless High-Strength Low-Alloy Structural Tubing, ASTM A618
Structural Steel for Bridges, ASTM A709
Quenched and Tempered Low-Alloy Structural Steel Plate with 70 ksi Minimum Yield Strength to 4-in. Thick, ASTM A852

Certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6 or A568, as applicable, shall constitute sufficient evidence of conformity with one of the above ASTM standards. If requested, the fabricator shall provide an affidavit stating that the structural steel furnished meets the requirements of the grade specified.

### 1b. Unidentified Steel

Unidentified steel, if surface conditions are acceptable according to criteria contained in ASTM A6, may be used for unimportant members or details, where the precise physical properties and weldability of the steel would not affect the strength of the structure.

### 1c. Heavy Shapes

For ASTM A6 Group 4 and 5 rolled shapes to be used as members subject to primary tensile stresses due to tension or flexure, toughness need not be specified if splices are made by bolting. If such members are spliced using complete-joint penetration welds, the steel shall be specified in the contract documents to be supplied with Charpy V-Notch testing in accordance with ASTM A6, Supplementary Requirement S5. The impact test shall meet a minimum average value of 20 ft-lbs. absorbed energy at +70°F and shall be conducted in accordance with ASTM A673 with the following exceptions:

(1) The center longitudinal axis of the specimens shall be located as near as practical to midway between the inner flange surface and the center of the flange thickness at the intersection with the web mid-thickness.

(2) Tests shall be conducted by the producer on material selected from a location representing the top of each ingot or part of an ingot used to produce the product represented by these tests.

For plates exceeding 2-in. thick used for built-up cross-sections with bolted splices and subject to primary tensile stresses due to tension or flexure, material toughness need not be specified. If such cross-sections are spliced using complete-joint penetration welds, the steel shall be specified in the contract documents to be supplied with Charpy V-Notch testing in accordance with ASTM A6, Supplementary Requirement S5. The impact test shall be conducted by the producer in accordance with ASTM A673, Frequency P, and shall meet a minimum average value of 20 ft-lbs. absorbed energy at +70°F.

The above supplementary requirements also apply when complete-joint penetration welded joints through the thickness of ASTM A6 Group 4 and 5 shapes and built-up cross sections with thickness exceeding two inches are used in connections subjected to primary tensile stress due to tension or flexure of such members. The requirements need not apply to ASTM A6 Group 4 and 5 shapes and built-up members with thickness exceeding two inches to which members other than ASTM A6 Group 4 and 5 shapes and built-up members are connected by complete-joint penetration welded joints through the thickness of the thinner material to the face of the heavy material.

Additional requirements for joints in heavy rolled and built-up members are given in Sections J1.5, J1.6, J2, and M2.2.

## 2. Steel Castings and Forgings

Cast steel shall conform to one of the following standard specifications:

Mild-to-Medium-Strength Carbon-Steel Castings for General Applications, ASTM A27, Gr. 65-35
High-Strength Steel Castings for Structural Purposes, ASTM A148 Gr. 80-50

Steel forgings shall conform to the following standard specification:

Steel Forgings Carbon and Alloy for General Industrial Use, ASTM A668

Certified test reports shall constitute sufficient evidence of conformity with standards.

## 3. Bolts, Washers, and Nuts

Steel bolts, washers, and nuts shall conform to one of the following standard specifications:

Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service, ASTM A194
Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength, ASTM A307
Structural Bolts, Steel, Heat-Treated, 120/105 ksi Minimum Tensile Strength, ASTM A325
Quenched and Tempered Steel Bolts and Studs, ASTM A449
Heat-Treated Steel Structural Bolts, 150 ksi Min. Tensile Strength, ASTM A490
Carbon and Alloy Steel Nuts, ASTM A563
Hardened Steel Washers, ASTM F436

A449 bolts are permitted to be used only in connections requiring bolt diameters greater than 1½-in. and shall not be used in slip-critical connections.

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards.

## 4. Anchor Bolts and Threaded Rods

Anchor bolt and threaded rod steel shall conform to one of the following standard specifications:

Structural Steel, ASTM A36
Alloy Steel and Stainless Steel Bolting Materials for High-Temperature Service, ASTM A193
Quenched and Tempered Alloy Steel Bolts, Studs and Other Externally Threaded Fasteners, ASTM A354
High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality, ASTM A572
High-Strength Low-Alloy Structural Steel with 50,000 psi Minimum Yield Point to 4-in. Thick, ASTM A588
High-Strength Nonheaded Steel Bolts and Studs, ASTM A687

Threads on bolts and rods shall conform to the Unified Standard Series of ANSI B18.1 and shall have Class 2A tolerances.

Steel bolts conforming to other provisions of Section A3.3 are permitted as anchor bolts. A449 material is acceptable for high-strength anchor bolts and threaded rods of any diameter.

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards.

### 5. Filler Metal and Flux for Welding

Welding electrodes and fluxes shall conform to one of the following specifications of the American Welding Society:

Specification for Carbon Steel Electrodes for Shield Metal Arc Welding, AWS A5.1
Specification for Low-Alloy Steel Covered Arc Welding Electrodes, AWS A5.5
Specification for Carbon Steel Electrodes and Fluxes for Submerged Arc Welding, AWS A5.17
Specification for Carbon Steel Filler Metals for Gas Shielded Arc Welding, AWS A5.18
Specification for Carbon Steel Electrodes for Flux Cored Arc Welding, AWS A5.20
Specification for Low-Alloy Steel Electrodes and Fluxes for Submerged Arc Welding, AWS A5.23
Specification for Low-Alloy Steel Filler Metals for Gas Shielded Arc Welding, AWS A5.28
Specification for Low-Alloy Steel Electrodes for Flux Cored Arc Welding, AWS A5.29

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards. Electrodes (filler metals) that are suitable for the intended application shall be selected. Weld metal notch toughness is generally not critical for building construction.

### 6. Stud Shear Connectors

Steel stud shear connectors shall conform to the requirements of *Structural Welding Code—Steel*, AWS D1.1.

Manufacturer's certification shall constitute sufficient evidence of conformity with the code.

## A4. LOADS AND LOAD COMBINATIONS

The nominal loads shall be the minimum design loads stipulated by the applicable code under which the structure is designed or dictated by the conditions involved. In the absence of a code, the loads and load combinations shall be those stipulated in the American Society of Civil Engineers Standard *Minimum Design Loads for Buildings and Other Structures*, ASCE 7. For design purposes, the loads stipulated by the applicable code shall be taken as nominal loads. For ease of reference, the more common ASCE load combinations are listed in the following section.

Seismic design of buildings assigned to the higher risk Seismic Performance Categories defined in the AISC *Seismic Provisions for Structural Steel Buildings* shall comply with that document. Seismic design not covered by the AISC

*Seismic Provisions for Structural Steel Buildings* shall be in accordance with this Specification.

## 1. Loads, Load Factors, and Load Combinations

The following nominal loads are to be considered:

$D$ : dead load due to the weight of the structural elements and the permanent features on the structure
$L$ : live load due to occupancy and moveable equipment
$L_r$ : roof live load
$W$ : wind load
$S$ : snow load
$E$ : earthquake load determined in accordance with Part I of the AISC *Seismic Provisions for Structural Steel Buildings*
$R$ : load due to initial rainwater or ice exclusive of the ponding contribution

The required strength of the structure and its elements must be determined from the appropriate critical combination of factored loads. The most critical effect may occur when one or more loads are not acting. The following load combinations and the corresponding load factors shall be investigated:

$$1.4D \tag{A4-1}$$

$$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R) \tag{A4-2}$$

$$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.8W) \tag{A4-3}$$

$$1.2D + 1.3W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R) \tag{A4-4}$$

$$1.2D \pm 1.0E + 0.5L + 0.2S \tag{A4-5}$$

$$0.9D \pm (1.3W \text{ or } 1.0E) \tag{A4-6}$$

*Exception:* The load factor on $L$ in combinations A4-3, A4-4, and A4-5 shall equal 1.0 for garages, areas occupied as places of public assembly, and all areas where the live load is greater than 100 psf.

## 2. Impact

For structures carrying live loads which induce impact, the assumed nominal live load shall be increased to provide for this impact in combinations A4-2 and A4-3.

If not otherwise specified, the increase shall be:

For supports of elevators and elevator machinery . . . . . . . . . . . . . . . . . . 100%
For supports of light machinery, shaft or motor driven, not less than . . . . 20%
For supports of reciprocating machinery or power driven units,
not less than . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 50%
For hangers supporting floors and balconies . . . . . . . . . . . . . . . . . . . . . . 33%
For cab-operated traveling crane support girders and their
connections . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 25%
For pendant-operated traveling crane support girders and their
connections . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 10%

### 3. Crane Runway Horizontal Forces

The nominal lateral force on crane runways to provide for the effect of moving crane trolleys shall be a minimum of 20 percent of the sum of weights of the lifted load and of the crane trolley, but exclusive of other parts of the crane. The force shall be assumed to be applied at the top of the rails, acting in either direction normal to the runway rails, and shall be distributed with due regard for lateral stiffness of the structure supporting the rails.

The nominal longitudinal force shall be a minimum of 10 percent of the maximum wheel loads of the crane applied at the top of the rail, unless otherwise specified.

## A5. DESIGN BASIS

### 1. Required Strength at Factored Loads

The required strength of structural members and connections shall be determined by structural analysis for the appropriate factored load combinations given in Section A4.

Design by either elastic or plastic analysis is permitted, except that design by plastic analysis is permitted only for steels with specified yield stresses not exceeding 65 ksi and is subject to provisions of Sections B5.2, C2, E1.2, F1.2d, H1, and I1.

Beams and girders composed of compact sections, as defined in Section B5.1, and satisfying the unbraced length requirements of Section F1.2d (including composite members) which are continuous over supports or are rigidly framed to columns may be proportioned for nine-tenths of the negative moments produced by gravity loading at points of support, provided that the maximum positive moment is increased by one-tenth of the average negative moments. This reduction is not permitted for hybrid beams, members of A514 steel, or moments produced by loading on cantilevers. If the negative moment is resisted by a column rigidly framed to the beam or girder, the one-tenth reduction may be used in proportioning the column for combined axial force and flexure, provided that the axial force does not exceed $\phi_c$ times $0.15A_gF_y$,

where

$A_g$ = gross area, in.$^2$
$F_y$ = specified minimum yield stress, ksi
$\phi_c$ = resistance factor for compression

### 2. Limit States

LRFD is a method of proportioning structures so that no applicable limit state is exceeded when the structure is subjected to all appropriate factored load combinations.

Strength limit states are related to safety and concern maximum load carrying capacity. Serviceability limit states are related to performance under normal service conditions. The term "resistance" includes both strength limit states and serviceability limit states.

### 3. Design for Strength

The design strength of each structural component or assemblage must equal or exceed the required strength based on the factored loads. The design strength $\phi R_n$ for each applicable limit state is calculated as the nominal strength $R_n$ multiplied by a resistance factor $\phi$.

The required strength is determined for each applicable load combination as stipulated in Section A4.

Nominal strengths $R_n$ and resistance factors $\phi$ are given in Chapters D through K.

### 4. Design for Serviceability and Other Considerations

The overall structure and the individual members, connections, and connectors shall be checked for serviceability. Provisions for design for serviceability are given in Chapter L.

## A6. REFERENCED CODES AND STANDARDS

The following documents are referenced in this Specification:

American National Standards Institute
ANSI B18.1-72

American Society of Civil Engineers
ASCE 7-88

American Society for Testing and Materials

| | | |
|---|---|---|
| ASTM A6-91b | ASTM A27-87 | ASTM A36-91 |
| ASTM A53-88 | ASTM A148-84 | ASTM A193-91 |
| ASTM A194-91 | ASTM A242-91a | ASTM A307-91 |
| ASTM A325-91c | ASTM A354-91 | ASTM A449-91a |
| ASTM A490-91 | ASTM A500-90a | ASTM A501-89 |
| ASTM A502-91 | ASTM A514-91 | ASTM A529-89 |
| ASTM A563-91c | ASTM A570-91 | ASTM A572-91 |
| ASTM A588-91a | ASTM A606-91a | ASTM A607-91 |
| ASTM A618-90a | ASTM A668-85a | ASTM A687-89 |
| ASTM A709-91 | ASTM A852-91 | ASTM C33-90 |
| ASTM C330-89 | ASTM F436-91 | |

American Welding Society

| | | |
|---|---|---|
| AWS D1.1-92 | AWS A5.1-91 | AWS A5.5-81 |
| AWS A5.17-89 | AWS A5.18-79 | AWS A5.20-79 |
| AWS A5.23-90 | AWS A5.28-79 | AWS A5.29-80 |

Research Council on Structural Connections
*Load and Resistance Factor Design Specification for Structural Joints Using ASTM A325 or A490 Bolts,* 1988

American Iron and Steel Institute
*Load and Resistance Factor Design Specification for Cold-Formed Steel Members,* 1991

American Institute of Steel Construction, Inc.
*Code of Standard Practice for Steel Buildings and Bridges,* 1992
*Seismic Provisions for Structural Steel Buildings,* 1992
*Specification for Load and Resistance Factor Design of Single-Angle Members,* 1993

## A7. DESIGN DOCUMENTS

### 1. Plans

The design plans shall show a complete design with sizes, sections, and relative locations of the various members. Floor levels, column centers and offsets shall be dimensioned. Drawings shall be drawn to a scale large enough to show the information clearly.

Design documents shall indicate the type or types of construction as defined in Section A2.2 and include the required strengths (moments and forces) if necessary for preparation of shop drawings.

Where joints are to be assembled with high-strength bolts, the design documents shall indicate the connection type (i.e., snug-tight bearing, fully-tensioned bearing, direct tension, or slip-critical).

Camber of trusses, beams, and girders, if required, shall be specified in the design documents. The requirements for stiffeners and bracing shall be shown in the design documents.

### 2. Standard Symbols and Nomenclature

Welding and inspection symbols used on plans and shop drawings shall be the American Welding Society symbols. Welding symbols for special requirements not covered by AWS is permitted to be used provided a complete explanation thereof is shown in the design documents.

### 3. Notation for Welding

Weld lengths called for in the design documents and on the shop drawings shall be the net effective lengths.

# CHAPTER B

# DESIGN REQUIREMENTS

This chapter contains provisions which are common to the Specification as a whole.

## B1. GROSS AREA

The gross area $A_g$ of a member at any point is the sum of the products of the thickness and the gross width of each element measured normal to the axis of the member. For angles, the gross width is the sum of the widths of the legs less the thickness.

## B2. NET AREA

The net area $A_n$ of a member is the sum of the products of the thickness and the net width of each element computed as follows:

In computing net area for tension and shear, the width of a bolt hole shall be taken as $\frac{1}{16}$-in. greater than the nominal dimension of the hole.

For a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters or slot dimensions as provided in Section J3.2, of all holes in the chain, and adding, for each gage space in the chain, the quantity $s^2/4g$

where

$s$ = longitudinal center-to-center spacing (pitch) of any two consecutive holes, in.
$g$ = transverse center-to-center spacing (gage) between fastener gage lines, in.

For angles, the gage for holes in opposite adjacent legs shall be the sum of the gages from the back of the angles less the thickness.

In determining the net area across plug or slot welds, the weld metal shall not be considered as adding to the net area.

## B3. EFFECTIVE NET AREA FOR TENSION MEMBERS

The effective net area for tension members shall be determined as follows:

1. When a tension load is transmitted directly to each of the cross-sectional elements by fasteners or welds, the effective net area $A_e$ is equal to the net area $A_n$.

2. When a tension load is transmitted by bolts or rivets through some but not

all of the cross-sectional elements of the member, the effective net area $A_e$ shall be computed as:

$$A_e = AU \tag{B3-1}$$

where

$A$ = area as defined below
$U$ = reduction coefficient
$= 1 - (\bar{x}/L) \leq 0.9$ or as defined in B3c or B3d (B3-2)
$\bar{x}$ = connection eccentricity, in.
$L$ = length of connection in the directions of loading, in.

Larger values of $U$ are permitted to be used when justified by tests or other rational criteria.

(a) When the tension load is transmitted only by bolts or rivets:

$A = A_n$
= net area of member, in.$^2$

(b) When the tension load is transmitted only by longitudinal welds to other than a plate member or by longitudinal welds in combination with transverse welds:

$A = A_g$
= gross area of member, in.$^2$

(c) When the tension load is transmitted only by transverse welds:

$A$ = area of directly connected elements, in.$^2$
$U$ = 1.0

(d) When the tension load is transmitted to a plate by longitudinal welds along both edges at the end of the plate for $l \geq w$:

$A$ = area of plate, in.$^2$

For $l \geq 2w$ .......................................... $U = 1.00$
For $2w > l \geq 1.5w$ .................................. $U = 0.87$
For $1.5w > l \geq w$ .................................... $U = 0.75$

where

$l$ = length of weld, in.
$w$ = plate width (distance between welds), in.

For effective area of connecting elements, see Section J5.2.

## B4. STABILITY

General stability shall be provided for the structure as a whole and for each of its elements.

Consideration shall be given to the significant effects of the loads on the deflected shape of the structure and its individual elements.

## B5. LOCAL BUCKLING

### 1. Classification of Steel Sections

Steel sections are classified as compact, noncompact, or slender-element sections. For a section to qualify as compact, its flanges must be continuously connected to the web or webs and the width-thickness ratios of its compression elements must not exceed the limiting width-thickness ratios $\lambda_p$ from Table B5.1. If the width-thickness ratio of one or more compression elements exceeds $\lambda_p$, but does not exceed $\lambda_r$, the section is noncompact. If the width-thickness ratio of any element exceeds $\lambda_r$ from Table B5.1, the section is referred to as a slender-element compression section.

For unstiffened elements which are supported along only one edge parallel to the direction of the compression force, the width shall be taken as follows:

(a) For flanges of I-shaped members and tees, the width $b$ is half the full-flange width, $b_f$.

(b) For legs of angles and flanges of channels and zees, the width $b$ is the full nominal dimension.

(c) For plates, the width $b$ is the distance from the free edge to the first row of fasteners or line of welds.

(d) For stems of tees, $d$ is taken as the full nominal depth.

For stiffened elements which are supported along two edges parallel to the direction of the compression force, the width shall be taken as follows:

(a) For webs of rolled or formed sections, $h$ is the clear distance between flanges less the fillet or corner radius at each flange; $h_c$ is twice the distance from the centroid to the inside face of the compression flange less the fillet or corner radius.

(b) For webs of built-up sections, $h$ is the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used, and $h_c$ is twice the distance from the centroid to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used.

(c) For flange or diaphragm plates in built-up sections, the width $b$ is the distance between adjacent lines of fasteners or lines of welds.

(d) For flanges of rectangular hollow structural sections, the width $b$ is the clear distance between webs less the inside corner radius on each side. If the corner radius is not known, the width may be taken as the total section width minus three times the thickness.

For tapered flanges of rolled sections, the thickness is the nominal value halfway between the free edge and the corresponding face of the web.

### 2. Design by Plastic Analysis

Design by plastic analysis is permitted when flanges subject to compression involving hinge rotation and all webs have a width-thickness ratio less than or equal to the limiting $\lambda_p$ from Table B5.1. For circular hollow sections see Footnote d of Table B5.1.

Design by plastic analysis is subject to the limitations in Section A5.1.

### 3. Slender-Element Compression Sections

For the flexural design of I-shaped sections, channels and rectangular or circular sections with slender flange elements, see Appendix F1. For other shapes in flexure or members in axial compression that have slender compression elements, see Appendix B5.3. For plate girders with slender web elements, see Appendix G.

## B6. BRACING AT SUPPORTS

At points of support for beams, girders and trusses, restraint against rotation about their longitudinal axis shall be provided.

## B7. LIMITING SLENDERNESS RATIOS

For members in which the design is based on compression, the slenderness ratio $Kl / r$ preferably should not exceed 200.

For members in which the design is based on tension, the slenderness ratio $l / r$ preferably should not exceed 300. The above limitation does not apply to rods in tension. Members in which the design is dictated by tension loading, but which may be subject to some compression under other load conditions, need not satisfy the compression slenderness limit.

## B8. SIMPLE SPANS

Beams, girders and trusses designed on the basis of simple spans shall have an effective length equal to the distance between centers of gravity of the members to which they deliver their end reactions.

## B9. END RESTRAINT

When designed on the assumption of full or partial end restraint due to continuous, semicontinuous, or cantilever action, the beams, girders, and trusses, as well as the sections of the members to which they connect, shall be designed to carry the factored forces and moments so introduced, as well as all other factored forces, without exceeding the design strengths prescribed in Chapters D through K, except that some inelastic but self-limiting deformation of a part of the connection is permitted.

## B10. PROPORTIONS OF BEAMS AND GIRDERS

Rolled or welded shapes, plate girders and cover-plated beams shall, in general, be proportioned by the moment of inertia of the gross section. No deduction shall be made for bolt or rivet holes in either flange provided that

$$0.75F_u A_{fn} \geq 0.9F_y A_{fg} \tag{B10-1}$$

where $A_{fg}$ is the gross flange area and $A_{fn}$ is the net flange area calculated in

# TABLE B5.1
## Limiting Width-Thickness Ratios for Compression Elements

| | Description of Element | Width Thick-ness Ratio | Limiting Width-Thickness Ratios | |
|---|---|---|---|---|
| | | | $\lambda_p$ (compact) | $\lambda_r$ (non compact) |
| Unstiffened Elements | Flanges of I-shaped rolled beams and channels in flexure | $b/t$ | $65/\sqrt{F_y}$ [c] | $141/\sqrt{F_y - 10}$ |
| | Flanges of I-shaped hybrid or welded beams in flexure | $b/t$ | $65/\sqrt{F_{yf}}$ | $\dfrac{162}{\sqrt{(F_{yf} - 16.5)/k_c}}$ [f] |
| | Flanges projecting from built-up compression members | $b/t$ | NA | $109/\sqrt{F_y/k_c}$ [f] |
| | Outstanding legs of pairs of angles in continuous contact, flanges of channels in axial compression; angles and plates projecting from beams or compression members | $b/t$ | NA | $95/\sqrt{F_y}$ |
| | Legs of single angle struts; legs of double angle struts with separators; unstiffened elements, i.e., supported along one edge | $b/t$ | NA | $76/\sqrt{F_y}$ |
| | Stems of tees | $d/t$ | NA | $127/\sqrt{F_y}$ |

accordance with the provisions of Sections B1 and B2 and $F_u$ is the specified minimum tensile strength.

If

$$0.75F_u A_{fn} < 0.9F_y A_{fg} \tag{B10-2}$$

the member flexural properties shall be based on an effective tension flange area $A_{fe}$

$$A_{fe} = \frac{5}{6}\frac{F_u}{F_y}A_{fn} \tag{B10-3}$$

Hybrid girders may be proportioned by the moment of inertia of their gross section, subject to the applicable provisions in Appendix G1, provided they are not required to resist an axial force greater than $\phi_b$ times $0.15F_{yf}A_g$, where $F_{yf}$ is the specified yield stress of the flange material and $A_g$ is the gross area. No limit is placed on the web stresses produced by the applied bending moment for which a hybrid girder is designed, except as provided in Section K3 and Appendix K3. To qualify as hybrid girders, the flanges at any given section shall have the same cross-sectional area and be made of the same grade of steel.

# TABLE B5.1 (cont.) Limiting Width-Thickness Ratios for Compression Elements

| | Description of Element | Width Thickness Ratio | Limiting Width-Thickness Ratios: $\lambda_p$ (compact) | Limiting Width-Thickness Ratios: $\lambda_r$ (non compact) |
|---|---|---|---|---|
| Stiffened Elements | Flanges of square and rectangular box and hollow structural sections of uniform thickness subject to bending or compression; flange cover plates and diaphragm plates between lines of fasteners or welds | $b/t$ | $190/\sqrt{F_y}$ | $238/\sqrt{F_y}$ |
| | Unsupported width of cover plates perforated with a succession of access holes [b] | $b/t$ | NA | $317/\sqrt{F_y}$ |
| | Webs in flexural compression [a] | $h/t_w$ | $640/\sqrt{F_y}$ [c] | $970/\sqrt{F_y}$ [g] |
| | Webs in combined flexural and axial compression | $h/t_w$ | for $P_u/\phi_b P_y \le 0.125$ [c] $\frac{640}{\sqrt{F_y}}\left(1-\frac{2.75P_u}{\phi_b P_y}\right)$ <br> for $P_u/\phi_b P_y > 0.125$ [c] $\frac{191}{\sqrt{F_y}}\left(2.33-\frac{P_u}{\phi_b P_y}\right) \ge \frac{253}{\sqrt{F_y}}$ | [g] $\frac{970}{\sqrt{F_y}}\left(1-0.74\frac{P_u}{\phi_b P_y}\right)$ |
| | All other uniformly compressed stiffened elements, i.e., supported along two edges | $b/t$ <br> $h/t_w$ | NA | $253/\sqrt{F_y}$ |
| | Circular hollow sections <br> In axial compression <br> In flexure | $D/t$ | [d] <br> NA <br> $2,070/F_y$ | <br> $3,300/F_y$ <br> $8,970/F_y$ |

[a] For hybrid beams, use the yield strength of the flange $F_{yf}$ instead of $F_y$

[b] Assumes net area of plate at widest hole.

[c] Assumes an inelastic rotation capacity of 3. For structures in zones of high seismicity, a greater rotation capacity may be required.

[d] For plastic design use $1,300/F_y$.

[e] $F_r$ = compressive residual stress in flange
= 10 ksi for rolled shapes
= 16.5 ksi for welded shapes

[f] $k_c = \frac{4}{\sqrt{h/t_w}}$ but not less than $0.35 \le k_c \le 0.763$

[g] For members with unequal flanges, see Appendix B5.1.
$F_y$ is the specified minimum yield stress of the type of steel being used.

Flanges of welded beams or girders may be varied in thickness or width by splicing a series of plates or by the use of cover plates.

The total cross-sectional area of cover plates of bolted or riveted girders shall not exceed 70 percent of the total flange area.

High-strength bolts, rivets, or welds connecting flange to web, or cover plate to flange, shall be proportioned to resist the total horizontal shear resulting from the bending forces on the girder. The longitudinal distribution of these bolts, rivets, or intermittent welds shall be in proportion to the intensity of the shear. However, the longitudinal spacing shall not exceed the maximum permitted for compression or tension members in Section E4 or D2, respectively. Bolts, rivets, or welds connecting flange to web shall also be proportioned to transmit to the web any loads applied directly to the flange, unless provision is made to transmit such loads by direct bearing.

Partial length cover plates shall be extended beyond the theoretical cutoff point and the extended portion shall be attached to the beam or girder by high-strength bolts in a slip-critical connection, rivets, or fillet welds. The attachment shall be adequate, at the applicable design strength given in Sections J2.2, J3.8, or K3 to develop the cover plate's portion of the flexural design strength in the beam or girder at the theoretical cutoff point.

For welded cover plates, the welds connecting the cover plate termination to the beam or girder in the length $a'$, defined below, shall be adequate, at the applicable design strength, to develop the cover plate's portion of the design strength in the beam or girder at the distance $a'$ from the end of the cover plate. The length $a'$, measured from the end of the cover plate, shall be:

(a) A distance equal to the width of the cover plate when there is a continuous weld equal to or larger than three-fourths of the plate thickness across the end of the plate and continuous welds along both edges of the cover plate in the length $a'$.

(b) A distance equal to one and one-half times the width of the cover plate when there is a continuous weld smaller than three-fourths of the plate thickness across the end of the plate and continuous welds along both edges of the cover plate in the length $a'$.

(c) A distance equal to two times the width of the cover plate when there is no weld across the end of the plate, but continuous welds along both edges of the cover plate in the length $a'$.

# CHAPTER C

# FRAMES AND OTHER STRUCTURES

This chapter contains general requirements for stability of the structure as a whole.

## C1. SECOND ORDER EFFECTS

Second order ($P\Delta$) effects shall be considered in the design of frames.

In structures designed on the basis of plastic analysis, the required flexural strength $M_u$ shall be determined from a second-order plastic analysis that satisfies the requirements of Section C2. In structures designed on the basis of elastic analysis, $M_u$ for beam-columns, connections, and connected members shall be determined from a second-order elastic analysis or from the following approximate second-order analysis procedure:

$$M_u = B_1 M_{nt} + B_2 M_{lt} \qquad \text{(C1-1)}$$

where

$M_{nt}$ = required flexural strength in member assuming there is no lateral translation of the frame, kip-in.

$M_{lt}$ = required flexural strength in member as a result of lateral translation of the frame only, kip-in.

$$B_1 = \frac{C_m}{(1 - P_u / P_{e1})} \geq 1 \qquad \text{(C1-2)}$$

$P_{e1} = A_g F_y / \lambda_c^2$ where $\lambda_c$ is the slenderness parameter, in which the effective $K$ in the plane of bending shall be determined in accordance with Section C2.1, for the braced frame.

$$\lambda_c = \frac{Kl}{r\pi}\sqrt{\frac{F_y}{E}}$$

$P_u$ = required axial compressive strength for the member under consideration, kips

$C_m$ = a coefficient based on elastic first-order analysis assuming no lateral translation of the frame whose value shall be taken as follows:

(a) For compression members not subject to transverse loading between their supports in the plane of bending,

$$C_m = 0.6 - 0.4(M_1 / M_2) \qquad \text{(C1-3)}$$

where $M_1 / M_2$ is the ratio of the smaller to larger moments at the ends of that portion of the member unbraced in the plane of bending under consideration. $M_1 / M_2$ is positive when the member is bent in reverse curvature, negative when bent in single curvature.

(b) For compression members subjected to transverse loading between their supports, the value of $C_m$ shall be determined either by rational analysis or by the use of the following values:

For members whose ends are restrained. . . . . . . . . . . . . . . . . . . . $C_m = 0.85$
For members whose ends are unrestrained. . . . . . . . . . . . . . . . . . $C_m = 1.00$

$$B_2 = \frac{1}{1 - \Sigma P_u \left( \dfrac{\Delta_{oh}}{\Sigma HL} \right)} \tag{C1-4}$$

or

$$B_2 = \frac{1}{1 - \dfrac{\Sigma P_u}{\Sigma P_{e2}}} \tag{C1-5}$$

$\Sigma P_u$ = required axial strength of all columns in a story, kips
$\Delta_{oh}$ = lateral inter-story deflection, in.
$\Sigma H$ = sum of all story horizontal forces producing $\Delta_{oh}$, kips
$L$ = story height, in.
$P_{e2}$ = $A_g F_y / \lambda_c^2$, kips, where $\lambda_c$ is the slenderness parameter, in which the effective length factor $K$ in the plane of bending shall be determined in accordance with Section C2.2, for the unbraced frame.

## C2. FRAME STABILITY

### 1. Braced Frames

In trusses and frames where lateral stability is provided by diagonal bracing, shear walls, or equivalent means, the effective length factor $K$ for compression members shall be taken as unity, unless structural analysis shows that a smaller value may be used.

The vertical bracing system for a braced multistory frame shall be determined by structural analysis to be adequate to prevent buckling of the structure and to maintain the lateral stability of the structure, including the overturning effects of drift, under the factored loads given in Section A4.

The vertical bracing system for a multistory frame may be considered to function together with in-plane shear-resisting exterior and interior walls, floor slabs, and roof decks, which are properly secured to the structural frames. The columns, girders, beams, and diagonal members, when used as the vertical bracing system, may be considered to comprise a vertically cantilevered simply connected truss in the analyses for frame buckling and lateral stability. Axial deformation of all members in the vertical bracing system shall be included in the lateral stability analysis.

In structures designed on the basis of plastic analysis, the axial force in these members caused by factored gravity plus factored horizontal loads shall not exceed $0.85\phi_c$ times $A_g F_y$.

Girders and beams included in the vertical bracing system of a braced multistory frame shall be proportioned for axial force and moment caused by concurrent factored horizontal and gravity loads.

## 2. Unbraced Frames

In frames where lateral stability depends upon the bending stiffness of rigidly connected beams and columns, the effective length factor $K$ of compression members shall be determined by structural analysis. The destabilizing effects of gravity loaded columns whose simple connections to the frame do not provide resistance to lateral loads shall be included in the design of the moment-frame columns. Stiffness reduction adjustment due to column inelasticity is permitted.

Analysis of the required strength of unbraced multistory frames shall include the effects of frame instability and column axial deformation under the factored loads given in Section A4.

In structures designed on the basis of plastic analysis, the axial force in the columns caused by factored gravity plus factored horizontal loads shall not exceed $0.75\phi_c$ times $A_g F_y$.

# CHAPTER D

# TENSION MEMBERS

This chapter applies to prismatic members subject to axial tension caused by static forces acting through the centroidal axis. For members subject to combined axial tension and flexure, see Section H1.1. For threaded rods, see Section J3. For block shear rupture strength at end connections of tension members, see Section J4.3. For the design tensile strength of connecting elements, see Section J5.2. For members subject to fatigue, see Section K3.

## D1. DESIGN TENSILE STRENGTH

The design strength of tension members $\phi_t P_n$ shall be the lower value obtained according to the limit states of yielding in the gross section and fracture in the net section.

(a) For yielding in the gross section:

$$\phi_t = 0.90$$
$$P_n = F_y A_g \tag{D1-1}$$

(b) For fracture in the net section:

$$\phi_t = 0.75$$
$$P_n = F_u A_e \tag{D1-2}$$

where

$A_e$ = effective net area, in.$^2$
$A_g$ = gross area of member, in.$^2$
$F_y$ = specified minimum yield stress, ksi
$F_u$ = specified minimum tensile strength, ksi
$P_n$ = nominal axial strength, kips

When members without holes are fully connected by welds, the effective net section used in Equation D1-2 shall be as defined in Section B3. When holes are present in a member with welded-end connections, or at the welded connection in the case of plug or slot welds, the net section through the holes shall be used in Equation D1-2.

## D2. BUILT-UP MEMBERS

For limitations on the longitudinal spacing of connectors between elements in continuous contact consisting of a plate and a shape or two plates, see Section J3.5.

The longitudinal spacing of connectors between components should preferably limit the slenderness ratio in any component between the connectors to 300.

Either perforated cover plates or tie plates without lacing are permitted to be used on the open sides of built-up tension members. Tie plates shall have a length not less than two-thirds the distance between the lines of welds or fasteners connecting them to the components of the member. The thickness of such tie plates shall not be less than one-fiftieth of the distance between these lines. The longitudinal spacing of intermittent welds or fasteners at tie plates shall not exceed six inches. The spacing of tie plates shall be such that the slenderness ratio of any component in the length between tie plates should preferably not exceed 300.

## D3. PIN-CONNECTED MEMBERS AND EYEBARS

The pin diameter shall not be less than seven-eighths times the eyebar body width.

The pin-hole diameter shall not be more than 1/32-in. greater than the pin diameter.

For steels having a yield stress greater than 70 ksi, the hole diameter shall not exceed five times the plate thickness and the width of the eyebar body shall be reduced accordingly.

In pin-connected members, the pin hole shall be located midway between the edges of the member in the direction normal to the applied force. For pin-connected members in which the pin is expected to provide for relative movement between connected parts while under full load, the diameter of pin hole shall not be more than 1/32-in. greater than the diameter of the pin. The width of the plate beyond the pin hole shall be not less than the effective width on either side of the pin hole.

In pin-connected plates other than eyebars, the minimum net area beyond the bearing end of the pin hole, parallel to the axis of the member, shall not be less than two-thirds of the net area required for strength across the pin hole.

The design strength of a pin-connected member $\phi P_n$ shall be the lowest value of the following limit states:

(a) Tension on the net effective area:

$$\phi = \phi_t = 0.75 \qquad \text{(D3-1)}$$
$$P_n = 2tb_{eff} F_u$$

(b) Shear on the effective area:

$$\phi_{sf} = 0.75 \qquad \text{(D3-2)}$$
$$P_n = 0.6A_{sf}F_u$$

(c) For bearing on the projected area of the pin, see Section J8.1.

(d) For yielding in the gross section, use Equation D1-1.

where

$a$ = shortest distance from edge of the pin hole to the edge of the member measured parallel to the direction of the force, in.
$A_{sf}$ = $2t(a + d/2)$, in.$^2$
$b_{eff}$ = $2t + 0.63$, but not more than the actual distance from the edge of the hole to the edge of the part measured in the direction normal to the applied force, in.
$d$ = pin diameter, in.
$t$ = thickness of plate, in.

The corners beyond the pin hole are permitted to be cut at 45° to the axis of the member, provided the net area beyond the pin hole, on a plane perpendicular to the cut, is not less than that required beyond the pin hole parallel to the axis of the member.

The design strength of eyebars shall be determined in accordance with Section D1 with $A_g$ taken as the cross-sectional area of the body.

Eyebars shall be of uniform thickness, without reinforcement at the pin holes, and have circular heads whose periphery is concentric with the pin hole.

The radius of transition between the circular head and the eyebar body shall be not less than the head diameter.

The width of the body of the eyebars shall not exceed eight times its thickness.

The thickness of less than ½-in. is permissible only if external nuts are provided to tighten pin plates and filler plates into snug contact. The width $b$ from the hole edge to the plate edge perpendicular to the direction of applied load shall be greater than two-thirds and, for the purpose of calculation, not more than three-fourths times the eyebar body width.

# CHAPTER E

# COLUMNS AND OTHER COMPRESSION MEMBERS

This chapter applies to compact and non-compact prismatic members subject to axial compression through the centroidal axis. For members subject to combined axial compression and flexure, see Section H1.2. For members with slender compression elements, see Appendix B5.3. For tapered members, see Appendix F3. For single-angle members, see AISC *Specification for Load and Resistance Design of Single-Angle Members*.

## E1. EFFECTIVE LENGTH AND SLENDERNESS LIMITATIONS

### 1. Effective Length

The effective length factor $K$ shall be determined in accordance with Section C2.

### 2. Design by Plastic Analysis

Design by plastic analysis, as limited in Section A5.1, is permitted if the column slenderness parameter $\lambda_c$ does not exceed $1.5K$.

## E2. DESIGN COMPRESSIVE STRENGTH FOR FLEXURAL BUCKLING

The design strength for flexural buckling of compression members whose elements have width-thickness ratios less than $\lambda_r$ from Section B5.1 is $\phi_c P_n$:

$$\phi_c = 0.85$$

$$P_n = A_g F_{cr} \tag{E2-1}$$

(a) For $\lambda_c \leq 1.5$

$$F_{cr} = (0.658^{\lambda_c^2})F_y \tag{E2-2}$$

(b) For $\lambda_c > 1.5$

$$F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right]F_y \tag{E2-3}$$

where

$$\lambda_c = \frac{Kl}{r\pi}\sqrt{\frac{F_y}{E}} \tag{E2-4}$$

$A_g$ = gross area of member, in.$^2$
$F_y$ = specified yield stress, ksi
$E$ = modulus of elasticity, ksi

$K$ = effective length factor
$l$ = laterally unbraced length of member, in.
$r$ = governing radius of gyration about the axis of buckling, in.

For members whose elements do not meet the requirements of Section B5.1, see Appendix B5.3.

## E3. DESIGN COMPRESSIVE STRENGTH FOR FLEXURAL-TORSIONAL BUCKLING

The design strength for flexural-torsional buckling of double-angle and tee-shaped compression members whose elements have width-thickness ratios less than $\lambda_r$ from Section B5.1 is $\phi_c P_n$:

$$\phi_c = 0.85$$

$$P_n = A_g F_{crft}$$

$$F_{crft} = \left(\frac{F_{cry} + F_{crz}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{cry}F_{crz}H}{(F_{cry} + F_{crz})^2}}\,\right] \qquad \text{(E3-1)}$$

where:

$F_{crz}$ $= \dfrac{GJ}{A\bar{r}_o^2}$

$\bar{r}_o$ = polar radius of gyration about shear center, in. (see Equation A-E3-8)

$H$ $= 1 - \left(\dfrac{x_o^2 + y_o^2}{\bar{r}_o^2}\right)$

$x_o, y_o$ = coordinate of shear center with respect to the centroid, in.
$x_o$ = 0 for double-angle and tee-shaped members (y-axis of symmetry)

$F_{cry}$ is determined according to Section E2 for flexural buckling about the y-axis of symmetry for $\lambda_c = \dfrac{Kl}{r_y \pi}\sqrt{\dfrac{F_y}{E}}$.

For double-angle and tee-shaped members whose elements do not meet the requirements of Section B5.1, see Appendix B5.3 to determine $F_{cry}$ for use in Equation E3-1.

Other singly symmetric and unsymmetric columns, and doubly symmetric columns, such as cruciform or built-up columns, with very thin walls shall be designed for the limit states of flexural-torsional and torsional buckling in accordance with Appendix E3.

## E4. BUILT-UP MEMBERS

At the ends of built-up compression members bearing on base plates or milled surfaces, all components in contact with one another shall be connected by a weld having a length not less than the maximum width of the member or by bolts spaced longitudinally not more than four diameters apart for a distance equal to 1½ times the maximum width of the member.

Along the length of built-up compression members between the end connections required above, longitudinal spacing for intermittent welds, bolts, or rivets shall be adequate to provide for the transfer of the required forces. For limitations on the longitudinal spacing of connectors between elements in continuous contact consisting of a plate and a shape or two plates, see Section J3.5. Where a component of a built-up compression member consists of an outside plate, the maximum spacing shall not exceed the thickness of the thinner outside plate times $127/\sqrt{F_y}$, nor 12 inches, when intermittent welds are provided along the edges of the components or when fasteners are provided on all gage lines at each section. When fasteners are staggered, the maximum spacing on each gage line shall not exceed the thickness of the thinner outside plate times $190/\sqrt{F_y}$ nor 18 inches.

Individual components of compression members composed of two or more shapes shall be connected to one another at intervals, $a$, such that the effective slenderness ratio $Ka/r_i$ of each of the component shapes, between the connectors, does not exceed three-fourths times the governing slenderness ratio of the built-up member. The least radius of gyration $r_i$ shall be used in computing the slenderness ratio of each component part. The end connection shall be welded or fully tensioned bolted with clean mill scale or blasted cleaned faying surfaces with Class A coatings.

The design strength of built-up members composed of two or more shapes shall be determined in accordance with Section E2 and Section E3 subject to the following modification. If the buckling mode involves relative deformations that produce shear forces in the connectors between individual shapes, $Kl/r$ is replaced by $(Kl/r)_m$ determined as follows:

(a) For intermediate connectors that are snug-tight bolted:

$$\left(\frac{Kl}{r}\right)_m = \sqrt{\left(\frac{Kl}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \qquad \text{(E4-1)}$$

(b) For intermediate connectors that are welded or fully-tensioned bolted:

$$\left(\frac{Kl}{r}\right)_m = \sqrt{\left(\frac{Kl}{r}\right)_o^2 + 0.82\frac{\alpha^2}{(1+\alpha^2)}\left(\frac{a}{r_{ib}}\right)^2} \qquad \text{(E4-2)}$$

where

$\left(\frac{Kl}{r}\right)_o$ = column slenderness of built-up member acting as a unit

$\left(\frac{Kl}{r}\right)_m$ = modified column slenderness of built-up member

$\frac{a}{r_i}$ = largest column slenderness of individual components

$\frac{a}{r_{ib}}$ = column slenerness of individual components relative to its centroidal axis parallel to axis of buckling

$a$ = distance between connectors, in.
$r_i$ = minimum radius of gyration of individual component, in.
$r_{ib}$ = radius of gyration of individual component relative to its centroidal axis parallel to member axis of buckling, in.
$\alpha$ = separation ratio = $h/2r_{ib}$
$h$ = distance between centroids of individual components perpendicular to the member axis of buckling, in.

Open sides of compression members built up from plates or shapes shall be provided with continuous cover plates perforated with a succession of access holes. The unsupported width of such plates at access holes, as defined in Section B5.1, is assumed to contribute to the design strength provided that:

(1) The width-thickness ratio conforms to the limitations of Section B5.1.

(2) The ratio of length (in direction of stress) to width of hole shall not exceed two.

(3) The clear distance between holes in the direction of stress shall be not less than the transverse distance between nearest lines of connecting fasteners or welds.

(4) The periphery of the holes at all points shall have a minimum radius of 1½-in.

As an alternative to perforated cover plates, lacing with tie plates is permitted at each end and at intermediate points if the lacing is interrupted. Tie plates shall be as near the ends as practicable. In main members providing design strength, the end tie plates shall have a length of not less than the distance between the lines of fasteners or welds connecting them to the components of the member. Intermediate tie plates shall have a length not less than one-half of this distance. The thickness of tie plates shall be not less than one-fiftieth of the distance between lines of welds or fasteners connecting them to the segments of the members. In welded construction, the welding on each line connecting a tie plate shall aggregate not less than one-third the length of the plate. In bolted and riveted construction, the spacing in the direction of stress in tie plates shall be not more than six diameters and the tie plates shall be connected to each segment by at least three fasteners.

Lacing, including flat bars, angles, channels, or other shapes employed as lacing, shall be so spaced that $l/r$ of the flange included between their connections shall not exceed the governing slenderness ratio for the member as a whole. Lacing shall be proportioned to provide a shearing strength normal to the axis of the member equal to two percent of the compressive design strength of the member. The $l/r$ ratio for lacing bars arranged in single systems shall not exceed 140. For double lacing this ratio shall not exceed 200. Double lacing bars shall be joined at the intersections. For lacing bars in compression, $l$ is permitted to be taken as the unsupported length of the lacing bar between welds or fasteners connecting it to the components of the built-up member for single lacing, and 70 percent of that distance for double lacing. The inclination of lacing bars to the axis of the member shall preferably be not less than 60° for single lacing and

45° for double lacing. When the distance between the lines of welds or fasteners in the flanges is more than 15 inches, the lacing shall preferably be double or be made of angles.

For additional spacing requirements, see Section J3.

## E5. PIN-CONNECTED COMPRESSION MEMBERS

Pin connections of pin-connected compression members shall conform to the requirements of Section D3 except Equations D3-1 and D3-2 do not apply.

# CHAPTER F

# BEAMS AND OTHER FLEXURAL MEMBERS

This chapter applies to compact and noncompact prismatic members subject to flexure and shear. For members subject to combined flexure and axial force, see Section H1. For members subject to fatigue, see Section K4. For members with slender compression elements, see Appendix B5. For web-tapered members, see Appendix F3. For members with slender web elements (plate girders), see Appendix G. For single-angle members, the AISC *Specification for Load and Resistance Factor Design of Single-Angle Members* is applicable.

## F1. DESIGN FOR FLEXURE

The nominal flexural strength $M_n$ is the lowest value obtained according to the limit stress of: (a) yielding; (b) lateral-torsional buckling; (c) flange local buckling; and (d) web local buckling. For laterally braced compact beams with $L_b \leq L_p$, only the limit state of yielding is applicable. For unbraced compact beams and noncompact tees and double angles, only the limit states of yielding and lateral-torsional buckling are applicable. The lateral-torsional buckling limit state is not applicable to members subject to bending about the minor axis, or to square or circular shapes.

This section applies to homogeneous and hybrid shapes with at least one axis of symmetry and which are subject to simple bending about one principal axis. For simple bending, the beam is loaded in a plane parallel to a principal axis that passes through the shear center or the beam is restrained against twisting at load points and supports. Only the limit states of yielding and lateral-torsional buckling are considered in this section. The lateral-torsional buckling provisions are limited to doubly symmetric shapes, channels, double angles, and tees. For lateral-torsional buckling of other singly symmetric shapes and for the limit states of flange local buckling and web local buckling of noncompact or slender-element sections, see Appendix F1. For unsymmetric shapes and beams subject to torsion combined with flexure, see Section H2. For biaxial bending, see Section H1.

### 1. Yielding

The flexural design strength of beams, determined by the limit state of yielding, is $\phi_b M_n$:

$$\phi_b = 0.90$$

$$M_n = M_p \tag{F1-1}$$

where

$M_p$ = plastic moment ($= F_y Z \leq 1.5 M_y$ for homogeneous sections), kip-in.
$M_y$ = moment corresponding to onset of yielding at the extreme fiber from an elastic stress distribution ($= F_y S$ for homogeneous section and $F_{yf} S$ for hybrid sections), kip-in.

## 2. Lateral-Torsional Buckling

This limit state is only applicable to members subject to major axis bending. The flexural design strength, determined by the limit state of lateral-torsional buckling, is $\phi_b M_n$:

$\phi_b$ = 0.90
$M_n$ = nominal strength determined as follows:

## 2a. Doubly Symmetric Shapes and Channels with $L_b \leq L_r$

The nominal flexural strength is:

$$M_n = C_b \left[ M_p - (M_p - M_r) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p \tag{F1-2}$$

where:

$L_b$ = distance between points braced against lateral displacement of the compression flange, or between points braced to prevent twist of the cross section, in.

In the above equation, $C_b$ is a modification factor for non-uniform moment diagrams where, when both ends of the beam segment are braced:

$$C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3M_A + 4M_B + 3M_C} \tag{F1-3}$$

where

$M_{max}$ = absolute value of maximum moment in the unbraced segment, kip-in.
$M_A$ = absolute value of moment at quarter point of the unbraced segment
$M_B$ = absolute value of moment at centerline of the unbraced beam segment
$M_C$ = absolute value of moment at three-quarter point of the unbraced beam segment

$C_b$ is permitted to be conservatively taken as 1.0 for all cases. For cantilevers or overhangs where the free end is unbraced, $C_b$ = 1.0.

The limiting unbraced length for full plastic bending capacity, $L_p$, shall be determined as follows.

(a) For I-shaped members including hybrid sections and channels:

$$L_p = \frac{300 r_y}{\sqrt{F_{yf}}} \tag{F1-4}$$

(b) For solid rectangular bars and box sections:

$$L_p = \frac{3{,}750 r_y}{M_p} \sqrt{JA} \tag{F1-5}$$

where

$A$ = cross-sectional area, in.$^2$
$J$ = torsional constant, in.$^4$

The limiting laterally unbraced length $L_r$ and the corresponding buckling moment $M_r$ shall be determined as follows:

(a) For doubly symmetric I-shaped members and channels:

$$L_r = \frac{r_y X_1}{F_L} \sqrt{1 + \sqrt{1 + X_2 F_L^2}} \tag{F1-6}$$

$$M_r = F_L S_x \tag{F1-7}$$

where

$$X_1 = \frac{\pi}{S_x} \sqrt{\frac{EGJA}{2}} \tag{F1-8}$$

$$X_2 = 4 \frac{C_w}{I_y} \left( \frac{S_x}{GJ} \right)^2 \tag{F1-9}$$

$S_x$ = section modulus about major axis, in.$^3$
$E$ = modulus of elasticity of steel (29,000 ksi)
$G$ = shear modulus of elasticity of steel (11,200 ksi)
$F_L$ = smaller of $(F_{yf} - F_r)$ or $F_{yw}$
$F_r$ = compressive residual stress in flange; 10 ksi for rolled shapes, 16.5 ksi for welded shapes
$F_{yf}$ = yield stress of flange, ksi
$F_{yw}$ = yield stress of web, ksi
$I_y$ = moment of inertia about y-axis, in.$^4$
$C_w$ = warping constant, in.$^6$

Equations F1-4 and F1-6 are conservatively based on $C_b = 1.0$.

(b) For solid rectangular bars and box sections:

$$L_r = \frac{57{,}000 r_y \sqrt{JA}}{M_r} \tag{F1-10}$$

$$M_r = F_{yf} S_x \tag{F1-11}$$

## 2b. Doubly Symmetric Shapes and Channels with $L_b > L_r$

The nominal flexural strength is:

$$M_n = M_{cr} \le M_p \tag{F1-12}$$

where $M_{cr}$ is the critical elastic moment, determined as follows:

(a) For doubly symmetric I-shaped members and channels:

$$M_{cr} = C_b \frac{\pi}{L_b} \sqrt{E I_y G J + \left(\frac{\pi E}{L_b}\right)^2 I_y C_w} \tag{F1-13}$$

$$= \frac{C_b\, S_x X_1 \sqrt{2}}{L_b / r_y} \sqrt{1 + \frac{X_1^2 X_2}{2(L_b / r_y)^2}}$$

(b) For solid rectangular bars and symmetric box sections:

$$M_{cr} = \frac{57{,}000\, C_b \sqrt{JA}}{L_b / r_y} \tag{F1-14}$$

## 2c. Tees and Double Angles

For tees and double-angle beams loaded in the plane of symmetry:

$$M_n = M_{cr} = \frac{\pi \sqrt{E I_y G J}}{L_b} \left[B + \sqrt{1 + B^2}\right] \tag{F1-15}$$

where

$M_n \leq 1.5 M_y$ for stems in tension

$M_n \leq 1.0 M_y$ for stems in compression

$$B = \pm 2.3 (d / L_b) \sqrt{I_y / J} \tag{F1-16}$$

The plus sign for $B$ applies when the stem is in tension and the minus sign applies when the stem is in compression. If the tip of the stem is in compression anywhere along the unbraced length, use the negative value of $B$.

## 2d. Unbraced Length for Design by Plastic Analysis

Design by plastic analysis, as limited in Section A5.1, is permitted for a compact section member bent about the major axis when the laterally unbraced length $L_b$ of the compression flange adjacent to plastic hinge locations associated with the failure mechanism does not exceed $L_{pd}$, determined as follows:

(a) For doubly symmetric and singly symmetric I-shaped members with the compression flange equal to or larger than the tension flange (including hybrid members) loaded in the plane of the web

$$L_{pd} = \frac{[3{,}600 + 2{,}200\, (M_1 / M_2)]\, r_y}{F_y} \tag{F1-17}$$

where

$F_y$ = specified minimum yield stress of the compression flange, ksi

$M_1$ = smaller moment at end of unbraced length of beam, kip-in.

$M_2$ = larger moment at end of unbraced length of beam, kip-in.

$r_y$ = radius of gyration about minor axis, in.

$(M_1 / M_2)$ is positive when moments cause reverse curvature and negative for single curvature

(b) For solid rectangular bars and symmetric box beams

$$L_{pd} = \frac{5{,}000 + 3{,}000\,(M_1 / M_2)}{F_y}\, r_y \geq 3{,}000 r_y / F_y \qquad \text{(F1-18)}$$

There is no limit on $L_b$ for members with circular or square cross sections nor for any beam bent about its minor axis.

In the region of the last hinge to form, and in regions not adjacent to a plastic hinge, the flexural design strength shall be determined in accordance with Section F1.2.

## F2. DESIGN FOR SHEAR

This section applies to unstiffened webs of singly or doubly symmetric beams, including hybrid beams, and channels subject to shear in the plane of the web. For the design shear strength of webs with stiffeners, see Appendix F2 or Appendix G3. For shear in the weak direction of the shapes above, pipes, and unsymmetric sections, see Section H2. For web panels subject to high shear, see Section K1.7. For shear strength at connections, see Sections J4 and J5.

### 1. Web Area Determination

The web area $A_w$ shall be taken as the overall depth $d$ times the web thickness $t_w$.

### 2. Design Shear Strength

The design shear strength of unstiffened webs, with $h / t_w \leq 260$, is $\phi_v V_n$, where

$\phi_v$ = 0.90
$V_n$ = nominal shear strength defined as follows

For $h / t_w \leq 418 / \sqrt{F_{yw}}$

$$V_n = 0.6 F_{yw} A_w \qquad \text{(F2-1)}$$

For $418 / \sqrt{F_{yw}} < h / t_w \leq 523 / \sqrt{F_{yw}}$

$$V_n = 0.6 F_{yw} A_w (418 / \sqrt{F_{yw}}) / (h / t_w) \qquad \text{(F2-2)}$$

For $523 / \sqrt{F_{yw}} < h / t_w \leq 260$

$$V_n = (132{,}000 A_w) / (h / t_w)^2 \qquad \text{(F2-3)}$$

The general design shear strength of webs with or without stiffeners is given in Appendix F2.2 and an alternative method utilizing tension field action is given in Appendix G3.

### 3. Transverse Stiffeners

See Appendix F2.3.

## F3. WEB-TAPERED MEMBERS

See Appendix F3.

## F4. BEAMS AND GIRDERS WITH WEB OPENINGS

The effect of all web openings on the design strength of steel and composite beams shall be determined. Adequate reinforcement shall be provided when the required strength exceeds the net strength of the member at the opening.

# CHAPTER G

# PLATE GIRDERS

I-shaped plate girders shall be distinguished from I-shaped beams on the basis of the web slenderness ratio $h / t_w$ . When this value is greater than $\lambda_r$ the provisions of Appendices G1 and G2 shall apply for design flexural strength. For $h / t_w \leq \lambda_r$, the provisions of Chapter F or Appendix F shall apply for design flexural strength. For girders with unequal flanges, see Appendix B5.1.

The design shear strength and transverse stiffener design shall be based on either Section F2 (without tension-field action) or Appendix G3 (with tension-field action). For girders with unequal flanges, see Appendix B5.1.

# CHAPTER H

# MEMBERS UNDER COMBINED FORCES AND TORSION

This chapter applies to prismatic members subject to axial force and flexure about one or both axes of symmetry, with or without torsion, and torsion only. For web-tapered members, see Appendix F3.

## H1. SYMMETRIC MEMBERS SUBJECT TO BENDING AND AXIAL FORCE

### 1. Doubly and Singly Symmetric Members in Flexure and Tension

The interaction of flexure and tension in symmetric shapes shall be limited by Equations H1-1a and H1-1b.

(a) For $\dfrac{P_u}{\phi P_n} \geq 0.2$

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \qquad \text{(H1-1a)}$$

(b) For $\dfrac{P_u}{\phi P_n} < 0.2$

$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \qquad \text{(H1-1b)}$$

where

$P_u$ = required tensile strength, kips
$P_n$ = nominal tensile strength determined in accordance with Section D1, kips
$M_u$ = required flexural strength determined in accordance with Section C1, kip-in.
$M_n$ = nominal flexural strength determined in accordance with Section F1, kip-in.
$x$ = subscript relating symbol to strong axis bending
$y$ = subscript relating symbol to weak axis bending.
$\phi$ = $\phi_t$ = resistance factor for tension (see Section D1)
$\phi_b$ = resistance factor for flexure = 0.90

A more detailed analysis of the interaction of flexure and tension is permitted in lieu of Equations H1-1a and H1-1b.

### 2. Doubly and Singly Symmetric Members in Flexure and Compression

The interaction of flexure and compression in symmetric shapes shall be limited by Equations H1-1a and H1-1b where

$P_u$ = required compressive strength, kips
$P_n$ = nominal compressive strength determined in accordance with Section E2, kips
$M_u$ = required flexural strength determined in accordance with Section C1 kip-in.
$M_n$ = nominal flexural strength determined in accordance with Section F1, kip-in.
$x$ = subscript relating symbol to strong axis bending
$y$ = subscript relating symbol to weak axis bending.
$\phi$ = $\phi_c$ = resistance factor for compression, = 0.85 (see Section E2)
$\phi_b$ = resistance factor for flexure = 0.90

## H2. UNSYMMETRIC MEMBERS AND MEMBERS UNDER TORSION AND COMBINED TORSION, FLEXURE, SHEAR, AND/OR AXIAL FORCE

The design strength $\phi F_y$ of the member shall equal or exceed the required strength expressed in terms of the normal stress $f_{un}$ or the shear stress $f_{uv}$, determined by elastic analysis for the factored loads:

(a) For the limit state of yielding under normal stress:

$$f_{un} \leq \phi F_y \qquad \text{(H2-1)}$$
$$\phi = 0.90$$

(b) For the limit state of yielding under shear stress:

$$f_{uv} \leq 0.6 \phi F_y \qquad \text{(H2-2)}$$
$$\phi = 0.90$$

(c) For the limit state of buckling:

$$f_{un} \text{ or } f_{uv} \leq \phi_c F_{cr}, \text{ as applicable} \qquad \text{(H2-3)}$$
$$\phi_c = 0.85$$

Some constrained local yielding is permitted adjacent to areas which remain elastic.

## H3. ALTERNATIVE INTERACTION EQUATIONS FOR MEMBERS UNDER COMBINED STRESS

See Appendix H3.

# CHAPTER I

# COMPOSITE MEMBERS

This chapter applies to composite columns composed of rolled or built-up structural steel shapes, pipe or tubing, and structural concrete acting together and to steel beams supporting a reinforced concrete slab so interconnected that the beams and the slab act together to resist bending. Simple and continuous composite beams with shear connectors and concrete-encased beams, constructed with or without temporary shores, are included.

## I1. DESIGN ASSUMPTIONS

*Force Determination.* In determining forces in members and connections of a structure that includes composite beams, consideration shall be given to the effective sections at the time each increment of load is applied.

*Elastic Analysis.* For an elastic analysis of continuous composite beams without haunched ends, it is permissible to assume that the stiffness of a beam is uniform throughout the beam length. The stiffness is permitted to be computed using the moment of inertia of the composite transformed section in the positive moment region.

*Plastic Analysis.* When plastic analysis is used, the strength of flexural composite members shall be determined from plastic stress distributions.

*Plastic Stress Distribution for Positive Moment.* If the slab in the positive moment region is connected to the steel beam with shear connectors, a concrete stress of $0.85f_c'$ is permitted to be assumed uniformly distributed throughout the effective compression zone. Concrete tensile strength shall be neglected. A uniformly distributed steel stress of $F_y$ shall be assumed throughout the tension zone and throughout the compression zone in the structural steel section. The net tensile force in the steel section shall be equal to the compressive force in the concrete slab.

*Plastic Stress Distribution for Negative Moment.* If the slab in the negative moment region is connected to the steel beam with shear connectors, a tensile stress of $F_{yr}$ shall be assumed in all adequately developed longitudinal reinforcing bars within the effective width of the concrete slab. Concrete tensile strength shall be neglected. A uniformly distributed steel stress of $F_y$ shall be assumed throughout the tension zone and throughout the compression zone in the structural steel section. The net compressive force in the steel section shall be equal to the total tensile force in the reinforcing steel.

*Elastic Stress Distribution.* When a determination of elastic stress distribution is required, strains in steel and concrete shall be assumed directly proportional to the distance from the neutral axis. The stress shall equal strain times modulus of elasticity for steel, $E$, or modulus of elasticity for concrete, $E_c$. Concrete tensile strength shall be neglected. Maximum stress in the steel shall not exceed $F_y$. Maximum compressive stress in the concrete shall not exceed $0.85f_c'$ where $f_c'$ is the specified compressive strength of the concrete. In composite hybrid beams, the maximum stress in the steel flange shall not exceed $F_{yf}$ but the strain in the web may exceed the yield strain; the stress shall be taken as $F_{yw}$ at such locations.

*Fully Composite Beam.* Shear connectors are provided in sufficient numbers to develop the maximum flexural strength of the composite beam. For elastic stress distribution it shall be assumed that no slip occurs.

*Partially Composite Beam.* The shear strength of shear connectors governs the flexural strength of the partially composite beam. Elastic computations such as those for deflections, fatigue, and vibrations shall include the effect of slip.

*Concrete-Encased Beam.* A beam totally encased in concrete cast integrally with the slab may be assumed to be interconnected to the concrete by natural bond, without additional anchorage, provided that: (1) concrete cover over beam sides and soffit is at least two inches; (2) the top of the beam is at least 1½-in. below the top and two inches above the bottom of the slab; and (3) concrete encasement contains adequate mesh or other reinforcing steel to prevent spalling of concrete.

*Composite Column.* A steel column fabricated from rolled or built-up steel shapes and encased in structural concrete or fabricated from steel pipe or tubing and filled with structural concrete shall be designed in accordance with Section I2.

## I2. COMPRESSION MEMBERS

### 1. Limitations

To qualify as a composite column, the following limitations shall be met:

(1) The cross-sectional area of the steel shape, pipe, or tubing shall comprise at least four percent of the total composite cross section.

(2) Concrete encasement of a steel core shall be reinforced with longitudinal load carrying bars, longitudinal bars to restrain concrete, and lateral ties. Longitudinal load carrying bars shall be continuous at framed levels; longitudinal restraining bars may be interrupted at framed levels. The spacing of ties shall be not greater than two-thirds of the least dimension of the composite cross section. The cross-sectional area of the transverse and longitudinal reinforcement shall be at least 0.007 sq. in. per inch of bar spacing. The encasement shall provide at least 1½-in. of clear cover outside of both transverse and longitudinal reinforcement.

(3) Concrete shall have a specified compressive strength $f_c'$ of not less than 3 ksi nor more than 8 ksi for normal weight concrete and not less than 4 ksi for light weight concrete.

(4) The specified minimum yield stress of structural steel and reinforcing bars

used in calculating the strength of a composite column shall not exceed 55 ksi.

(5) The minimum wall thickness of structural steel pipe or tubing filled with concrete shall be equal to $b\sqrt{F_y / 3E}$ for each face of width $b$ in rectangular sections and $D\sqrt{F_y / 8E}$ for circular sections of outside diameter $D$.

## 2. Design Strength

The design strength of axially loaded composite columns is $\phi_c P_n$,

where

$\phi_c$ = 0.85
$P_n$ = nominal axial compressive strength determined from Equations E2-1 through E2-4 with the following modifications:

(1) $A_s$ = gross area of steel shape, pipe, or tubing, in.$^2$ (replaces $A_g$)

$r_m$ = radius of gyration of the steel shape, pipe, or tubing except that for steel shapes it shall not be less than 0.3 times the overall thickness of the composite cross section in the plane of buckling, in. (replaces $r$)

(2) Replace $F_y$ with modified yield stress $F_{my}$ from Equation I2-1 and replace $E$ with modified modulus of elasticity $E_m$ from Equation I2-2.

$$F_{my} = F_y + c_1 F_{yr}(A_r / A_s) + c_2 f_c'(A_c / A_s) \tag{I2-1}$$

$$E_m = E + c_3 E_c (A_c / A_s) \tag{I2-2}$$

where

$A_c$ = area of concrete, in.$^2$
$A_r$ = area of longitudinal reinforcing bars, in.$^2$
$A_s$ = area of steel, in.$^2$
$E$ = modulus of elasticity of steel, ksi
$E_c$ = modulus of elasticity of concrete. $E_c$ is permitted to be computed from $E_c = w^{1.5}\sqrt{f_c'}$ where $w$, the unit weight of concrete, is expressed in lbs./cu. ft and $f_c'$ is expressed in ksi.
$F_y$ = specified minimum yield stress of steel shape, pipe, or tubing, ksi
$F_{yr}$ = specified minimum yield stress of longitudinal reinforcing bars, ksi
$f_c'$ = specified compressive strength of concrete, ksi
$c_1, c_2, c_3$ = numerical coefficients. For concrete-filled pipe and tubing: $c_1 = 1.0$, $c_2 = 0.85$, and $c_3 = 0.4$; for concrete encased shapes $c_1 = 0.7$, $c_2 = 0.6$, and $c_3 = 0.2$

## 3. Columns with Multiple Steel Shapes

If the composite cross section includes two or more steel shapes, the shapes shall be interconnected with lacing, tie plates, or batten plates to prevent buckling of individual shapes before hardening of concrete.

## 4. Load Transfer

The portion of the design strength of axially loaded composite columns resisted by concrete shall be developed by direct bearing at connections. When the supporting concrete area is wider than the loaded area on one or more sides and otherwise restrained against lateral expansion on the remaining sides, the maximum design strength of concrete shall be $1.7\phi_c f_c' A_B$,

where

$\phi_c$ = 0.60
$A_B$ = loaded area

## I3. FLEXURAL MEMBERS

### 1. Effective Width

The effective width of the concrete slab on each side of the beam center-line shall not exceed:

(a) one-eighth of the beam span, center to center of supports;

(b) one-half the distance to the center-line of the adjacent beam; or

(c) the distance to the edge of the slab.

### 2. Strength of Beams with Shear Connectors

The positive design flexural strength $\phi_b M_n$ shall be determined as follows:

(a) For $h / t_w \leq 640 / \sqrt{F_{yf}}$:

$\phi_b = 0.85$; $M_n$ shall be determined from the plastic stress distribution on the composite section.

(b) For $h / t_w > 640 / \sqrt{F_{yf}}$:

$\phi_b = 0.90$; $M_n$ shall be determined from the superposition of elastic stresses, considering the effects of shoring.

The negative design flexural strength $\phi_b M_n$ shall be determined for the steel section alone, in accordance with the requirements of Chapter F.

Alternatively, the negative design flexural strength $\phi_b M_n$ shall be computed with: $\phi_b = 0.85$ and $M_n$ determined from the plastic stress distribution on the composite section, provided that:

(1) Steel beam is an adequately braced compact section, as defined in Section B5.

(2) Shear connectors connect the slab to the steel beam in the negative moment region.

(3) Slab reinforcement parallel to the steel beam, within the effective width of the slab, is properly developed.

### 3. Strength of Concrete-Encased Beams

The design flexural strength $\phi_b M_n$ shall be computed with $\phi_b = 0.90$ and $M_n$ determined from the superposition of elastic stresses, considering the effects of shoring.

Alternatively, the design flexural strength $\phi_b M_n$ shall be computed with $\phi_b = 0.90$ and $M_n$ determined from the plastic stress distribution on the steel section alone.

**4. Strength During Construction**

When temporary shores are not used during construction, the steel section alone shall have adequate strength to support all loads applied prior to the concrete attaining 75 percent of its specified strength $f_c'$. The design flexural strength of the steel section shall be determined in accordance with the requirements of Section F1.

**5. Formed Steel Deck**

**5a. General**

The design flexural strength $\phi_b M_n$ of composite construction consisting of concrete slabs on formed steel deck connected to steel beams shall be determined by the applicable portions of Section I3.2, with the following modifications.

This section is applicable to decks with nominal rib height not greater than three inches. The average width of concrete rib or haunch $w_r$ shall be not less than two inches, but shall not be taken in calculations as more than the minimum clear width near the top of the steel deck. See Section I3.5c for additional restrictions.

The concrete slab shall be connected to the steel beam with welded stud shear connectors ¾-in. or less in diameter (AWS D1.1). Studs shall be welded either through the deck or directly to the steel beam. Stud shear connectors, after installation, shall extend not less than 1½-in. above the top of the steel deck.

The slab thickness above the steel deck shall be not less than two inches.

**5b. Deck Ribs Oriented Perpendicular to Steel Beam**

Concrete below the top of the steel deck shall be neglected in determining section properties and in calculating $A_c$ for deck ribs oriented perpendicular to the steel beams.

The spacing of stud shear connectors along the length of a supporting beam shall not exceed 36 inches.

The nominal strength of a stud shear connector shall be the value stipulated in Section I5 multiplied by the following reduction factor:

$$\frac{0.85}{\sqrt{N_r}}(w_r / h_r)\,[(H_s / h_r) - 1.0] \leq 1.0 \qquad \text{(I3-1)}$$

where

$h_r$ = nominal rib height, in.
$H_s$ = length of stud connector after welding, in., not to exceed the value $(h_r + 3)$ in computations, although actual length may be greater
$N_r$ = number of stud connectors in one rib at a beam intersection, not to exceed three in computations, although more than three studs may be installed

$w_r$ = average width of concrete rib or haunch (as defined in Section I3.5a), in.

To resist uplift, steel deck shall be anchored to all supporting members at a spacing not to exceed 18 inches. Such anchorage shall be provided by stud connectors, a combination of stud connectors and arc spot (puddle) welds, or other devices specified by the designer.

### 5c. Deck Ribs Oriented Parallel to Steel Beam

Concrete below the top of the steel deck may be included in determining section properties and shall be included in calculating $A_c$ in Section I5.

Steel deck ribs over supporting beams may be split longitudinally and separated to form a concrete haunch.

When the nominal depth of steel deck is 1½-in. or greater, the average width $w_r$ of the supported haunch or rib shall be not less than two inches for the first stud in the transverse row plus four stud diameters for each additional stud.

The nominal strength of a stud shear connector shall be the value stipulated in Section I5, except that when $w_r / h_r$ is less than 1.5, the value from Section I5 shall be multiplied by the following reduction factor:

$$0.6(w_r / h_r)[(H_s / h_r) - 1.0] \leq 1.0 \qquad \text{(I3-2)}$$

where $h_r$ and $H_s$ are as defined in Section I3.5b and $w_r$ is the average width of concrete rib or haunch as defined in Section I3.5a.

### 6. Design Shear Strength

The design shear strength of composite beams shall be determined by the shear strength of the steel web, in accordance with Section F2.

## I4. COMBINED COMPRESSION AND FLEXURE

The interaction of axial compression and flexure in the plane of symmetry on composite members shall be limited by Section H1.2 with the following modifications:

$M_n$ = nominal flexural strength determined from plastic stress distribution on the composite cross section except as provided below, kip-in.

$P_{e1}$, $P_{e2}$ = $A_s F_{my} / \lambda_c^2$ elastic buckling load, kips

$F_{my}$ = modified yield stress, ksi, see Section I2

$\phi_b$ = resistance factor for flexure from Section I3

$\phi_c$ = resistance factor for compression = 0.85

$\lambda_c$ = column slenderness parameter defined by Equation E2-4 as modified in Section I2.2

When the axial term in Equations H1-1a and H1-1b is less than 0.3, the nominal flexural strength $M_n$ shall be determined by straight line transition between the nominal flexural strength determined from the plastic distribution on the composite cross sections at $(P_u / \phi_c P_n) = 0.3$ and the flexural strength at $P_u = 0$ as determined in Section I3. If shear connectors are required at $P_u = 0$, they shall be provided whenever $P_u / \phi_c P_n$ is less than 0.3.

## I5. SHEAR CONNECTORS

This section applies to the design of stud and channel shear connectors. For connectors of other types, see Section I6.

### 1. Materials

Shear connectors shall be headed steel studs not less than four stud diameters in length after installation, or hot rolled steel channels. The stud connectors shall conform to the requirements of Section A3.6. The channel connectors shall conform to the requirements of Section A3. Shear connectors shall be embedded in concrete slabs made with ASTM C33 aggregate or with rotary kiln produced aggregates conforming to ASTM C330, with concrete unit weight not less than 90 pcf.

### 2. Horizontal Shear Force

Except for concrete-encased beams as defined in Section I1, the entire horizontal shear at the interface between the steel beam and the concrete slab shall be assumed to be transferred by shear connectors. For composite action with concrete subject to flexural compression, the total horizontal shear force between the point of maximum positive moment and the point of zero moment shall be taken as the smallest of the following: (1) $0.85 f_c' A_c$; (2) $A_s F_y$; and (3) $\Sigma Q_n$;

where

$f_c'$ = specified compressive strength of concrete, ksi
$A_c$ = area of concrete slab within effective width, in.$^2$
$A_s$ = area of steel cross section, in.$^2$
$F_y$ = minimum specified yield stress, ksi
$\Sigma Q_n$ = sum of nominal strengths of shear connectors between the point of maximum positive moment and the point of zero moment, kips

For hybrid beams, the yield force shall be computed separately for each component of the cross section; $A_s F_y$ of the entire cross section is the sum of the component yield forces.

In continuous composite beams where longitudinal reinforcing steel in the negative moment regions is considered to act compositely with the steel beam, the total horizontal shear force between the point of maximum negative moment and the point of zero moment shall be taken as the smaller of $A_r F_{yr}$ and $\Sigma Q_n$;

where

$A_r$ = area of adequately developed longitudinal reinforcing steel within the effective width of the concrete slab, in.$^2$
$F_{yr}$ = minimum specified yield stress of the reinforcing steel, ksi
$\Sigma Q_n$ = sum of nominal strengths of shear connectors between the point of maximum negative moment and the point of zero moment, kips

## 3. Strength of Stud Shear Connectors

The nominal strength of one stud shear connector embedded in a solid concrete slab is

$$Q_n = 0.5A_{sc}\sqrt{f_c' E_c} \leq A_{sc}F_u \tag{I5-1}$$

where

$A_{sc}$ = cross-sectional area of a stud shear connector, in.$^2$
$f_c'$ = specified compressive strength of concrete, ksi
$F_u$ = minimum specified tensile strength of a stud shear connector, ksi
$E_c$ = modulus of elasticity of concrete, ksi

For stud shear connector embedded in a slab on a formed steel deck, refer to Section I3 for reduction factors given by Equations I3-1 and I3-2 as applicable. The reduction factors apply only to $0.5A_{sc}\sqrt{f_c' E_c}$ term in Equation I5-1.

## 4. Strength of Channel Shear Connectors

The nominal strength of one channel shear connector embedded in a solid concrete slab is

$$Q_n = 0.3(t_f + 0.5t_w)L_c\sqrt{f_c' E_c} \tag{I5-2}$$

where

$t_f$ = flange thickness of channel shear connector, in.
$t_w$ = web thickness of channel shear connector, in.
$L_c$ = length of channel shear connector, in.

## 5. Required Number of Shear Connectors

The number of shear connectors required between the section of maximum bending moment, positive or negative, and the adjacent section of zero moment shall be equal to the horizontal shear force as determined in Section I5.2 divided by the nominal strength of one shear connector as determined from Section I5.3 or Section I5.4.

## 6. Shear Connector Placement and Spacing

Unless otherwise specified shear connectors required each side of the point of maximum bending moment, positive or negative, shall be distributed uniformly between that point and the adjacent points of zero moment. However, the number of shear connectors placed between any concentrated load and the nearest point of zero moment shall be sufficient to develop the maximum moment required at the concentrated load point.

Except for connectors installed in the ribs of formed steel decks, shear connectors shall have at least one inch of lateral concrete cover. Unless located over the web, the diameter of studs shall not be greater than 2.5 times the thickness of the flange to which they are welded. The minimum center-to-center spacing of stud connectors shall be six diameters along the longitudinal axis of the supporting composite beam and four diameters transverse to the longitudinal axis of the supporting composite beam, except that within the ribs of formed steel decks the center-to-center spacing may be as small as four diameters in any direction. The maximum center-to-center spacing of shear connectors shall not exceed eight times the total slab thickness. Also see Section I3.5b.

## I6. SPECIAL CASES

When composite construction does not conform to the requirements of Section I1 through Section I5, the strength of shear connectors and details of construction shall be established by a suitable test program.

# CHAPTER J

# CONNECTIONS, JOINTS, AND FASTENERS

This chapter applies to connecting elements, connectors, and the affected elements of the connected members subject to static loads. For connections subject to fatigue, see Appendix K3.

## J1. GENERAL PROVISIONS

### 1. Design Basis

Connections consist of affected elements of connected members (e.g. beam webs), connecting elements (e.g., gussets, angles, brackets), and connectors (welds, bolts, rivets). These components shall be proportioned so that their design strength equals or exceeds the required strength determined by structural analysis for factored loads acting on the structure or a specified proportion of the strength of the connected members, whichever is appropriate.

### 2. Simple Connections

Except as otherwise indicated in the design documents, connections of beams, girders, or trusses shall be designed as flexible, and are permitted to ordinarily be proportioned for the reaction shears only. Flexible beam connections shall accommodate end rotations of unrestrained (simple) beams. To accomplish this, some inelastic but self-limiting deformation in the connection is permitted.

### 3. Moment Connections

End connections of restrained beams, girders, and trusses shall be designed for the combined effect of forces resulting from moment and shear induced by the rigidity of the connections.

### 4. Compression Members with Bearing Joints

When columns bear on bearing plates or are finished to bear at splices, there shall be sufficient connectors to hold all parts securely in place.

When compression members other than columns are finished to bear, the splice material and its connectors shall be arranged to hold all parts in line and shall be proportioned for 50 percent of the required strength of the member.

All compression joints shall be proportioned to resist any tension developed by the factored loads specified by load combination A4-6.

### 5. Splices in Heavy Sections

This paragraph applies to ASTM A6 Group 4 and 5 rolled shapes, or shapes built-up by welding plates more than two inches thick together to form the cross section, and where the cross section is to be spliced and subject to primary tensile stresses due to tension or flexure. When the individual elements of the cross section are spliced prior to being joined to form the cross section in accordance with AWS D1.1, Article 3.4.6, the applicable provisions of AWS D1.1 apply in lieu of the requirements of this section. When tensile forces in these sections are to be transmitted through splices by complete-joint-penetration groove welds, material notch-toughness requirements as given in Section A3.1c, weld access hole details as given in Section J1.6, welding preheat requirements as given in Section J2.8, and thermal-cut surface preparation and inspection requirements as given in Section M2.2 apply.

At tension splices in ASTM A6 Group 4 and 5 shapes and built-up members of material more than two inches thick, weld tabs and backing shall be removed and the surfaces ground smooth.

When splicing ASTM A6 Group 4 and 5 rolled shapes or shapes built-up by welding plates more than two inches thick to form a cross section, and where the section is to be used as a primary compression member, all weld access holes required to facilitate groove welding operations shall satisfy the provisions of Section J1.6.

Alternatively, splicing of such members subject to compression, including members which are subject to tension due to wind or seismic loads, shall be accomplished using splice details which do not induce large weld shrinkage strains; for example partial-joint-penetration flange groove welds with fillet-welded surface lap plate splices on the web, bolted lap plate splices, or combination bolted/fillet-welded lap plate splices.

## 6. Beam Copes and Weld Access Holes

All weld access holes required to facilitate welding operations shall have a length from the toe of the weld preparation not less than 1½ times the thickness of the material in which the hole is made. The height of the access hole shall be adequate for deposition of sound weld metal in the adjacent plates and provide clearance for weld tabs for the weld in the material in which the hole is made, but not less than the thickness of the material. In hot-rolled shapes and built-up shapes, all beam copes and weld access holes shall be shaped free of notches and sharp re-entrant corners except, when fillet web-to-flange welds are used in built-up shapes, access holes are permitted to terminate perpendicular to the flange.

For ASTM A6 Group 4 and 5 shapes and built-up shapes of material more than two inches thick, the thermally cut surfaces of beam copes and weld access holes shall be ground to bright metal and inspected by either magnetic particle or dye penetrant methods prior to deposition of splice welds. If the curved transition portion of weld access holes and beam copes are formed by predrilled or sawed holes, that portion of the access hole or cope need not be ground. Weld access holes and beam copes in other shapes need not be ground nor inspected by dye penetrant or magnetic particle methods.

## 7. Minimum Strength of Connections

Except for lacing, sag rods, or girts, connections providing design strength shall be designed to support a factored load not less than 10 kips.

## 8. Placement of Welds and Bolts

Groups of welds or bolts at the ends of any member which transmit axial force into that member shall be sized so that the center of gravity of the group coincides with the center of gravity of the member, unless provision is made for the eccentricity. The foregoing provision is not applicable to end connections of statically-loaded single angle, double angle, and similar members.

## 9. Bolts in Combination with Welds

In new work, A307 bolts or high-strength bolts proportioned as bearing-type connections shall not be considered as sharing the load in combination with welds. Welds, if used, shall be proportioned for the entire force in the connection. In slip-critical connections, high-strength bolts are permitted to be considered as sharing the load with the welds.

In making welded alterations to structures, existing rivets and high-strength bolts tightened to the requirements for slip-critical connections are permitted to be utilized for carrying loads present at the time of alteration and the welding need only provide the additional design strength required.

## 10. High-Strength Bolts in Combination with Rivets

In both new work and alterations, in connections designed as slip-critical connections in accordance with the provisions of Section J3, high-strength bolts are permitted to be considered as sharing the load with rivets.

## 11. Limitations on Bolted and Welded Connections

Fully tensioned high-strength bolts (see Table J3.1) or welds shall be used for the following connections:

Column splices in all tier structures 200 ft or more in height.

Column splices in tier structures 100 to 200 ft in height, if the least horizontal dimension is less than 40 percent of the height.

Column splices in tier structures less than 100 ft in height, if the least horizontal dimension is less than 25 percent of the height.

Connections of all beams and girders to columns and of any other beams and girders on which the bracing of columns is dependent, in structures over 125 ft in height.

In all structures carrying cranes of over five-ton capacity: roof-truss splices and connections of trusses to columns, column splices, column bracing, knee braces, and crane supports.

Connections for supports of running machinery, or of other live loads which produce impact or reversal of stress.

Any other connections stipulated on the design plans.

In all other cases connections are permitted to be made with A307 bolts or snug-tight high-strength bolts.

For the purpose of this section, the height of a tier structure shall be taken as the vertical distance from the curb level to the highest point of the roof beams in the case of flat roofs, or to the mean height of the gable in the case of roofs having a rise of more than $2\frac{2}{3}$ in 12. Where the curb level has not been established, or where the structure does not adjoin a street, the mean level of the adjoining land shall be used instead of curb level. It is permissible to exclude penthouses in computing the height of structure.

## J2. WELDS

All provisions of the American Welding Society *Structural Welding Code Steel,*, AWS D1.1, apply under this specification, except Chapter 10—Tubular Structures, which is outside the scope of this specification, and except that the provisions of the listed AISC LRFD Specification Sections apply under this Specification in lieu of the cited AWS Code provisions as follows:

AISC Section J1.5 and J1.6 in lieu of AWS Section 3.2.5
AISC Section J2.2 in lieu of AWS Section 2.3.2.4
AISC Table J2.5 in lieu of AWS Table 8.1
AISC Table A-K3.2 in lieu of AWS Section 2.5
AISC Section K3 and Appendix K3 in lieu of AWS Chapter 9
AISC Section M2.2 in lieu of AWS Section 3.2.2

### 1. Groove Welds

### 1a. Effective Area

The effective area of groove welds shall be considered as the effective length of the welds times the effective throat thickness.

The effective length of a groove weld shall be the width of the part joined.

The effective throat thickness of a complete-joint-penetration groove weld shall be the thickness of the thinner part joined.

The effective throat thickness of a partial-joint-penetration groove weld shall be as shown in Table J2.1.

The effective throat thickness of flare groove weld when flush to the surface of a bar or 90° bend in formed section shall be as shown in Table J2.2. Random sections of production welds for each welding procedure, or such test sections as may be required by design documents, shall be used to verify that the effective throat is consistently obtained.

Larger effective throat thicknesses than those in Table J2.2 are permitted, provided the fabricator can establish by qualification the consistent production of such larger effective throat thicknesses. Qualification shall consist of sectioning the weld normal to its axis, at mid-length and terminal ends. Such sectioning shall be made on a number of combinations of material sizes representative of the range to be used in the fabrication or as required by the designer.

### 1b. Limitations

The minimum effective throat thickness of a partial-joint-penetration groove weld shall be as shown in Table J2.3. Weld size is determined by the thicker of the two parts joined, except that the weld size need not exceed the thickness of the thinnest part joined when a larger size is required by calculated strength. For this exception, particular care shall be taken to provide sufficient preheat for soundness of the weld.

## TABLE J2.1
## Effective Throat Thickness of Partial-Penetration Groove Welds

| Welding Process | Welding Position | Included Angle at Root of Groove | Effective Throat Thickness |
|---|---|---|---|
| Shielded metal arc<br>Submerged arc | All | J or U joint | Depth of chamfer |
| Gas metal arc | | Bevel or V joint ≥ 60º | |
| Flux-cored arc | | Bevel or V joint < 60º<br>but ≥ 45º | Depth of chamfer<br>minus $\frac{1}{8}$-in. |

## TABLE J2.2
## Effective Throat Thickness of Flare Groove Welds

| Type of Weld | Radius ($R$) of Bar or Bend | Effective Throat Thickness |
|---|---|---|
| Flare bevel groove | All | $\frac{5}{16}R$ |
| Flare V-groove | All | $\frac{1}{2}R$ |
| [a] Use $\frac{3}{8}R$ for Gas Metal Arc Welding (except short circuiting transfer process) when $R \geq 1$ in. | | |

## TABLE J2.3
## Minimum Effective Throat Thickness of Partial-Joint-Penentration Groove Welds

| Material Thickness of Thicker Part Joined (in.) | Minimum Effective Throat Thickness[a] (in.) |
|---|---|
| To $\frac{1}{4}$ inclusive | $\frac{1}{8}$ |
| Over $\frac{1}{4}$ to $\frac{1}{2}$ | $\frac{3}{16}$ |
| Over $\frac{1}{2}$ to $\frac{3}{4}$ | $\frac{1}{4}$ |
| Over $\frac{3}{4}$ to $1\frac{1}{2}$ | $\frac{5}{16}$ |
| Over $1\frac{1}{2}$ to $2\frac{1}{4}$ | $\frac{3}{8}$ |
| Over $2\frac{1}{4}$ to 6 | $\frac{1}{2}$ |
| Over 6 | $\frac{5}{8}$ |
| [a] See Section J2. | |

## 2. Fillet Welds

### 2a. Effective Area

The effective area of fillet welds shall be as defined in American Welding Society Code D1.1 Article 2.3.2, except 2.3.2.4. The effective throat thickness of a fillet weld shall be the shortest distance from the root of the joint to the face of the diagrammatic weld, except that for the fillet welds made by the submerged

# TABLE J2.4
## Minimum Size of Fillet Welds[b]

| Material Thickness of Thicker Part Joined (in.) | Minimum Size of Fillet Weld[a] (in.) |
|---|---|
| To ¼ inclusive | ⅛ |
| Over ¼ to ½ | 3/16 |
| Over ½ to ¾ | ¼ |
| Over ¾ | 5/16 |

[a] Leg dimension of fillet welds. Single pass welds must be used.
[b] See Section J2.2b for maximum size of fillet welds.

arc process, the effective throat thickness shall be taken equal to the leg size for ⅜-in. and smaller fillet welds, and equal to the theoretical throat plus 0.11-in. for fillet welds over ⅜-in.

For fillet welds in holes and slots, the effective length shall be the length of the centerline of the weld along the center of the plane through the throat. In the case of overlapping fillets, the effective area shall not exceed the nominal cross-sectional area of the hole or slot, in the plane of the faying surface.

### 2b. Limitations

The *minimum size of fillet welds* shall be not less than the size required to transmit calculated forces nor the size as shown in Table J2.4 which is based upon experiences and provides some margin for uncalculated stress encountered during fabrication, handling, transportation, and erection. These provisions do not apply to fillet weld reinforcements of partial- or complete-joint-penetration welds.

The *maximum size of fillet welds* of connected parts shall be:

(a) Along edges of material less than ¼-in. thick, not greater than the thickness of the material.

(b) Along edges of material ¼-in. or more in thickness, not greater than the thickness of the material minus 1/16-in., unless the weld is especially designated on the drawings to be built out to obtain full-throat thickness. In the as-welded condition, the distance between the edge of the base metal and the toe of the weld is permitted to be less than 1/16-in. provided the weld size is clearly verifiable.

(c) For flange-web welds and similar connections, the actual weld size need not be larger than that required to develop the web capacity, and the requirements of Table J2.4 need not apply.

The *minimum effective length of fillet welds* designed on the basis of strength shall be not less than four times the nominal size, or else the size of the weld shall be considered not to exceed ¼ of its effective length. If longitudinal fillet welds are used alone in end connections of flat-bar tension members, the length of each fillet weld shall be not less than the perpendicular distance between them. The transverse spacing of longitudinal fillet welds used in end connections of tension members shall comply with Section B3.

The *maximum effective length of fillet welds* loaded by forces parallel to the weld, such as lap splices, shall not exceed 70 times the fillet weld leg. A uniform stress distribution may be assumed throughout the maximum effective length.

*Intermittent fillet welds* may be used to transfer calculated stress across a joint or faying surfaces when the strength required is less than that developed by a continuous fillet weld of the smallest permitted size, and to join components of built-up members. The effective length of any segment of intermittent fillet welding shall be not less than four times the weld size, with a minimum of 1½-in.

*In lap joints,* the minimum amount of lap shall be five times the thickness of the thinner part joined, but not less than one inch. Lap joints joining plates or bars subjected to axial stress shall be fillet welded along the end of both lapped parts, except where the deflection of the lapped parts is sufficiently restrained to prevent opening of the joint under maximum loading.

*Fillet welds terminations* shall not be at the extreme ends or sides of parts or members. They shall be *either* returned continuously around the ends or sides, respectively for a distance of not less than two times the nominal weld size *or* shall terminate not less than the nominal weld size from the sides or ends except as follows. For details and structural elements such as brackets, beam seats, framing angles, and simple end plates which are subject to cyclic (fatigue) out-of-plane forces and/or moments of frequency and magnitude that would tend to initiate a progressive failure of the weld, fillet welds *shall be returned* around the side or end for a distance not less than two times the nominal weld size. For framing angles and simple end-plate connections which depend upon flexibility of the outstanding legs for connection flexibility, if end returns are used, their length shall not exceed four times the nominal size of the weld. Fillet welds which occur on opposite sides of a common plane shall be interrupted at the corner common to both welds. End returns shall be indicated on the design and detail drawings.

*Fillet welds in holes or slots* may be used to transmit shear in lap joints or to prevent the buckling or separation of lapped parts and to join components of built-up members. Such fillet welds may overlap, subject to the provisions of Section J2. Fillet welds in holes or slots are not to be considered plug or slot welds.

## 3. Plug and Slot Welds

### 3a. Effective Area

The effective shearing area of plug and slot welds shall be considered as the nominal cross-sectional area of the hole or slot in the plane of the faying surface.

### 3b. Limitations

Plug or slot welds are permitted to be used to transmit shear in lap joints or to prevent buckling of lapped parts and to join component parts of built-up members.

The diameter of the holes for a plug weld shall not be less than the thickness of the part containing it plus $^5/_{16}$-in., rounded to the next larger odd $^1/_{16}$-in., nor greater than the minimum diameter plus $^1/_8$-in. or $2^1/_4$ times the thickness of the weld.

The minimum center-to-center spacing of plug welds shall be four times the diameter of the hole.

The length of slot for a slot weld shall not exceed 10 times the thickness of the weld. The width of the slot shall be not less than the thickness of the part containing it plus $^5/_{16}$-in. rounded to the next larger odd $^1/_{16}$-in., nor shall it be larger than $2^1/_4$ times the thickness of the weld. The ends of the slot shall be semicircular or shall have the corners rounded to a radius of not less than the thickness of the part containing it, except those ends which extend to the edge of the part.

The minimum spacing of lines of slot welds in a direction transverse to their length shall be four times the width of the slot. The minimum center-to-center spacing in a longitudinal direction on any line shall be two times the length of the slot.

The thickness of plug or slot welds in material $^5/_8$-in. or less in thickness shall be equal to the thickness of the material. In material over $^5/_8$-in. thick, the thickness of the weld shall be at least one-half the thickness of the material but not less than $^5/_8$-in.

## 4. Design Strength

The design strength of welds shall be the lower value of $\phi F_{BM} A_{BM}$ and $\phi F_w A_w$, when applicable. The values of $\phi$, $F_{BM}$, and $F_w$ and limitations thereon are given in Table J2.5,

where

$F_{BM}$ = nominal strength of the base material, ksi
$F_w$ = nominal strength of the weld electrode, ksi
$A_{BM}$ = cross-sectional area of the base material, in.$^2$
$A_w$ = effective cross-sectional area of the weld, in.$^2$
$\phi$ = resistance factor

Alternatively, fillet welds loaded in-plane are permitted to be designed in accordance with Appendix J2.4.

## 5. Combination of Welds

If two or more of the general types of welds (groove, fillet, plug, slot) are combined in a single joint, the design strength of each shall be separately computed with reference to the axis of the group in order to determine the design strength of the combination.

## 6. Matching Weld Metal

The choice of electrode for use with complete-joint-penetration groove welds subject to tension normal to the effective area shall comply with the requirements for matching weld metals given in AWS D1.1.

# TABLE J2.5
# Design Strength of Welds

| Types of Weld and Stress [a] | Material | Resistance Factor $\phi$ | Nominal Strength $F_{BM}$ or $F_w$ | Required Weld Strength Level [b,c] |
|---|---|---|---|---|
| **Complete-Joint-Penetration Groove Weld** | | | | |
| Tension normal to effective area | Base | 0.90 | $F_y$ | Matching weld must be used. |
| Compression normal to effective area<br>Tension or compression parallel to axis of weld | Base | 0.90 | $F_y$ | Weld metal with a strength level equal to or less than matching weld metal is permitted to be used. |
| Shear on effective area | Base<br>Weld electrode | 0.90<br>0.80 | $0.60F_y$<br>$0.60F_{EXX}$ | |
| **Partial-Joint-Penetration Groove Weld** | | | | |
| Compression normal to effective area<br>Tension or compression parallel to axis of weld [d] | Base | 0.90 | $F_y$ | Weld metal with a strength level equal to or less than matching weld metal is permitted to be used. |
| Shear parallel to axis of weld | Base<br>Weld electrode | 0.75 | [e]<br>$0.60F_{EXX}$ | |
| Tension normal to effective area | Base<br>Weld electrode | 0.90<br>0.80 | $F_y$<br>$0.60F_{EXX}$ | |
| **Fillet Welds** | | | | |
| Shear on effective area | Base<br>Weld electrode | 0.75 | [f]<br>$0.60F_{EXX}$ | Weld metal with a strength level equal to or less than matching weld metal is permitted to be used. |
| Tension or compression parallel to axis of weld [d] | Base | 0.90 | $F_y$ | |
| **Plug or Slot Welds** | | | | |
| Shear parallel to faying surfaces (on effective area) | Base<br>Weld electrode | 0.75 | [e]<br>$0.60F_{EXX}$ | Weld metal with a strength level equal to or less than matching weld metal is permitted to be used. |

[a] For definition of effective area, see Section J2.
[b] For matching weld metal, see Table 4.1, AWS D1.1.
[c] Weld metal one strength level stronger than matching weld metal is permitted.
[d] Fillet welds and partial-joint-penetration groove welds joining component elements of built-up members, such as flange-to-web connections, are not required to be designed with to the tensile or compressive stress in these elements parallel to the axis of the welds.
[e] The design of connected material is governed by Sections J4 and J5.
[f] For alternative design strength, see Appendix J2.4.

## 7. Mixed Weld Metal

When notch-toughness is specified, the process consumables for all weld metal, tack welds, root pass, and subsequent passes deposited in a joint shall be compatible to assure notch-tough composite weld metal.

## 8. Preheat for Heavy Shapes

For ASTM A6 Group 4 and 5 shapes and welded built-up members made of plates more than two inches thick, a preheat equal to or greater than 350°F shall be used when making groove-weld splices.

# J3. BOLTS AND THREADED PARTS

## 1. High-Strength Bolts

Use of high-strength bolts shall conform to the provisions of the *Load and Resistance Factor Design Specification for Structural Joints Using ASTM A325 or A490 Bolts*, as approved by the Research Council on Structural Connections, except as otherwise provided in this Specification.

If required to be tightened to more than 50 percent of their minimum specified tensile strength, A449 bolts in tension and bearing-type shear connections shall have an ASTM F436 hardened washer installed under the bolt head, and the nuts shall meet the requirements of ASTM A563. When assembled, all joint surfaces, including those adjacent to the washers, shall be free of scale, except tight mill scale. Except as noted below, all A325 and A490 bolts shall be tightened to a bolt tension not less than that given in Table J3.1. Tightening shall be done by any of the following methods: turn-of-nut method, a direct tension indicator, calibrated wrench, or alternative design bolt.

Bolts in connections not subject to tension loads, where slip can be permitted and where loosening or fatigue due to vibration or load fluctuations are not design considerations, need only to be tightened to the snug-tight condition. The snug-tight condition is defined as the tightness attained by either a few impacts of an impact wrench or the full effort of a worker with an ordinary spud wrench that brings the connected plies into firm contact. The nominal strength value given in Table J3.2 for bearing-type connections shall be used for bolts tightened to the snug-tight condition. Bolts tightened only to the snug-tight condition shall be clearly identified on the design and erection drawings.

When A490 bolts over one inch in diameter are used in slotted or oversize holes in external plies, a single hardened washer conforming to ASTM F436, except with $\frac{5}{16}$-in. minimum thickness, shall be used in lieu of the standard washer.

In slip-critical connections in which the direction of loading is toward an edge of a connected part, adequate bearing strength at factored load shall be provided based upon the applicable requirements of Section J3.10.

## 2. Size and Use of Holes

In slip-critical connections in which the direction of loading is toward edge of connected part, adequate bearing capacity at factored load shall be provided based upon the applicable requirements of Section J3.10.

The *maximum sizes* of holes for rivets and bolts are given in Table J3.3, except

# TABLE J3.1
## Minimum Bolt Tension, kips*

| Bolt Size, in. | A325 Bolts | A490 Bolts |
|---|---|---|
| ½ | 12 | 15 |
| ⅝ | 19 | 24 |
| ¾ | 28 | 35 |
| ⅞ | 39 | 49 |
| 1 | 51 | 64 |
| 1⅛ | 56 | 80 |
| 1¼ | 71 | 102 |
| 1⅜ | 85 | 121 |
| 1½ | 103 | 148 |

* Equal to 0.70 of minimum tensile strength of bolts, rounded off to nearest kip, as specified in ASTM specifications for A325 and A490 bolts with UNC threads.

that larger holes, required for tolerance on location of anchor bolts in concrete foundations, are allowed in column base details.

*Standard holes* shall be provided in member-to-member connections, unless oversized, short-slotted, or long-slotted holes in bolted connections are approved by the designer. Finger shims up to ¼-in. are permitted in slip-critical connections designed on the basis of standard holes without reducing the nominal shear strength of the fastener to that specified for slotted holes.

*Oversized holes* are allowed in any or all plies of slip-critical connections, but they shall not be used in bearing-type connections. Hardened washers shall be installed over oversized holes in an outer ply.

*Short-slotted holes* are allowed in any or all plies of slip-critical or bearing-type connections. The slots are permitted to be used without regard to direction of loading in slip-critical connections, but the length shall be normal to the direction of the load in bearing-type connections. Washers shall be installed over short-slotted holes in an outer ply; when high-strength bolts are used, such washers shall be hardened.

*Long-slotted holes* are allowed in only one of the connected parts of either a slip-critical or bearing-type connection at an individual faying surface. Long-slotted holes are permitted to be used without regard to direction of loading in slip-critical connections, but shall be normal to the direction of load in bearing-type connections. Where long-slotted holes are used in an outer ply, plate washers, or a continuous bar with standard holes, having a size sufficient to completely cover the slot after installation, shall be provided. In high-strength bolted connections, such plate washers or continuous bars shall be not less than 5⁄16-in. thick and shall be of structural grade material, but need not be hardened. If hardened washers are required for use of high-strength bolts, the hardened washers shall be placed over the outer surface of the plate washer or bar.

## 3. Minimum Spacing

The distance between centers of standard, oversized, or slotted holes, shall not

# TABLE J3.2
# Design Strength of Fasteners

| Description of Fasteners | Tensile Strength | | Shear Strength in Bearing-type Connections | |
|---|---|---|---|---|
| | Resistance Factor $\phi$ | Nominal Strength, ksi | Resistance Factor $\phi$ | Nominal Strength, ksi |
| A307 bolts | 0.75 | 45 [a] | 0.75 | 24 [b,e] |
| A325 bolts, when threads are not excluded from shear planes | | 90 [d] | | 48 [e] |
| A325 bolts, when threads are excluded from shear planes | | 90 [d] | | 60 [e] |
| A490 bolts, when threads are not excluded from shear planes | | 113 [d] | | 60 [e] |
| A490 bolts, when threads are excluded from shear planes | | 113 [d] | | 75 [e] |
| Threaded parts meeting the requirements of Sect. A3, when threads are not excluded from shear planes | | $0.75F_u$ [a,c] | | $0.40F_u$ |
| Threaded parts meeting the requirements of Sect. A3, when threads are excluded from shear planes | | $0.75F_u$ [a,c] | | $0.50F_u$ [a,c] |
| A502, Gr. 1, hot-driven rivets | | 45 [a] | | 25 [e] |
| A502, Gr. 2 & 3, hot-driven rivets | | 60 [a] | | 33 [e] |

[a] Static loading only.
[b] Threads permitted in shear planes.
[c] The nominal tensile strength of the threaded portion of an upset rod, based upon the cross-sectional area at its major thread diameter, $A_D$ shall be larger than the nominal body area of the rod before upsetting times $F_y$.
[d] For A325 and A490 bolts subject to tensile fatigue loading, see Appendix K3.
[e] When bearing-type connections used to splice tension members have a fastener pattern whose length, measured parallel to the line of force, exceeds 50 in., tabulated values shall be reduced by 20 percent.

be less than 2⅔ times the nominal diameter of the fastener; a distance of $3d$ is preferred. Refer to Section J3.10 for bearing strength requirement.

## 4. Minimum Edge Distance

The distance from the center of a standard hole to an edge of a connected part shall not be less than either the applicable value from Table J3.4, or as required

## TABLE J3.3
## Nominal Hole Dimensions

| Bolt Diameter | Hole Dimensions: Standard (Dia.) | Oversize (Dia.) | Short-slot (Width × Length) | Long-slot Width × Length) |
|---|---|---|---|---|
| 1/2 | 9/16 | 5/8 | 9/16 × 11/16 | 9/16 × 1 1/4 |
| 5/8 | 11/16 | 13/16 | 11/16 × 7/8 | 11/16 × 1 9/16 |
| 3/4 | 13/16 | 15/16 | 13/16 × 1 | 13/16 × 1 7/8 |
| 7/8 | 15/16 | 1 1/16 | 15/16 × 1 1/8 | 15/16 × 2 3/16 |
| 1 | 1 1/16 | 1 1/4 | 1 1/16 × 1 5/16 | 1 1/16 × 2 1/2 |
| ≥1 1/8 | $d$ + 1/16 | $d$ + 5/16 | ($d$ + 1/16) × ($d$ + 3/8) | ($d$ + 1/16) × (2.5 × $d$) |

## TABLE J3.4
## Minimum Edge Distance,[a] in.
## (Center of Standard Hole[b] to Edge of Connected Part)

| Nominal Rivet or Bolt Diameter (in.) | At Sheared Edges | At Rolled Edges of Plates, Shapes or Bars, or Gas Cut Edges [c] |
|---|---|---|
| 1/2 | 7/8 | 3/4 |
| 5/8 | 1 1/8 | 7/8 |
| 3/4 | 1 1/4 | 1 |
| 7/8 | 1 1/2 [d] | 1 1/8 |
| 1 | 1 3/4 [d] | 1 1/4 |
| 1 1/8 | 2 | 1 1/2 |
| 1 1/4 | 2 1/4 | 1 5/8 |
| Over 1 1/4 | 1 3/4 × Diameter | 1 1/4 × Diameter |

[a] Lesser edge distances are permitted to be used provided Equations from J3.10, as appropriate, are satisfied.
[b] For oversized or slotted holes, see Table J3.8.
[c] All edge distances in this column are permitted to be reduced 1/8-in. when the hole is at a point where stress does not exceed 25 percent of the maximum design strength in the element.
[d] These are permitted to be 1 1/4-in. at the ends of beam connection angles and shear end plates.

in Section J3.10. The distance from the center of an oversized or slotted hole to an edge of a connected part shall be not less than that required for a standard hole to an edge of a connected part plus the applicable increment $C_2$ from Table J3.8. Refer to Section J3.10 for bearing strength requirement.

### 5. Maximum Spacing and Edge Distance

The maximum distance from the center of any bolt or rivet to the nearest edge of parts in contact shall be 12 times the thickness of the connected part under consideration, but shall not exceed six inches. The longitudinal spacing of connectors between elements in continuous contact consisting of a plate and a shape or two plates shall be as follows:

(a) For painted members or unpainted members not subject to corrosion, the spacing shall not exceed 24 times the thickness of the thinner plate or 12 inches.

(b) For unpainted members of weathering steel subject to atmospheric corrosion, the spacing shall not exceed 14 times the thickness of the thinner plate or seven inches.

## 6. Design Tension or Shear Strength

The design tension or shear strength of a high-strength bolt or threaded part is $\phi F_n A_b$

where

$\phi$ = resistance factor tabulated in Table J3.2
$F_n$ = nominal tensile strength $F_t$, or shear strength, $F_v$, tabulated in Table J3.2, ksi
$A_b$ = nominal unthreaded body area of bolt or threaded part (for upset rods, see Footnote c, Table J3.2), in.$^2$

The applied load shall be the sum of the factored loads and any tension resulting from prying action produced by deformation of the connected parts.

## 7. Combined Tension and Shear in Bearing-Type Connections

The design strength of a bolt or rivet subject to combined tension and shear is $\phi F_t A_b$, where $\phi$ is 0.75 and the nominal tension stress $F_t$ shall be computed from the equations in Table J3.5 as a function of $f_v$, the required shear stress produced by the factored loads. The design shear strength $\phi F_v$, tabulated in Table J3.2, shall equal or exceed the shear stress, $f_v$.

## 8. High-Strength Bolts in Slip-Critical Connections

The design for shear of high-strength bolts in slip-critical connections shall be in accordance with either Section J3.8a or J3.8b and checked for bearing in accordance with J3.2 and J3.10.

## 8a. Slip-Critical Connections Designed at Service Loads

The design resistance to shear of a bolt in a slip-critical connection is $\phi F_v A_b$,

where

$\phi$ = 1.0 for standard, oversized, short-slotted, and long-slotted holes when the long slot is perpendicular to the line of force
$\phi$ = 0.85 for long-slotted holes when the long slot is parallel to the line of force
$F_v$ = nominal slip-critical shear resistance tabulated in Table J3.6, ksi

The design resistance to shear shall equal or exceed the shear on the bolt due to service loads. When the loading combination includes wind loads in addition to dead and live loads, the total shear on the bolt due to combined load effects, at service load, may be multiplied by 0.75.

The values for $F_v$ in Table J3.6 are based on Class A (slip coefficient 0.33), clean mill scale and blast cleaned surfaces with class A coatings. When specified by

## TABLE J3.5
## Tension Stress Limit ($F_t$), ksi
## Fasteners in Bearing-type Connections

| Description of Fasteners | Threads Included in the Shear Plane | Threads Excluded from the Shear Plane |
|---|---|---|
| A307 bolts | $59 - 1.9f_v \leq 45$ | |
| A325 bolts | $117 - 1.9f_v \leq 90$ | $117 - 1.5f_v \leq 90$ |
| A490 bolts | $147 - 1.9f_v \leq 113$ | $147 - 1.5f_v \leq 113$ |
| Threaded parts A449 bolts over 1½ diameter | $0.98F_u - 1.9f_v \leq 0.75F_u$ | $0.98F_u - 1.5f_v \leq 0.75F_u$ |
| A502 Gr.1 rivets | $59 - 1.8f_v \leq 45$ | |
| A502 Gr.2 rivets | $78 - 1.8f_v \leq 60$ | |

## TABLE J3.6
## Slip-Critical Nominal Resistance to Shear, ksi, of High-Strength Bolts[a]

| Type of Bolt | Nominal Resistance to Shear | | |
|---|---|---|---|
| | Standard Size Holes | Oversized and Short-slotted Holes | Long-slotted Holes |
| A325 | 17 | 15 | 12 |
| A490 | 21 | 18 | 15 |
| [a] For each shear plane. | | | |

the designer, the nominal slip resistance for connections having special faying surface conditions are permitted to be adjusted to the applicable values in the RCSC Load and Resistance Factor Design Specification.

Finger shims up to ¼-in. are permitted to be introduced into slip-critical connections designed on the basis of standard holes without reducing the design shear stress of the fastener to that specified for slotted holes.

### 8b. Slip-Critical Connections Designed at Factored Loads

See Appendix J3.8b.

### 9. Combined Tension and Shear in Slip-Critical Connections

The design of slip-critical connections subject to tensile forces shall be in accordance with either Sections J3.9a and J3.8a or Sections J3.9b and J3.8b.

### 9a. Slip-Critical Connections Designed at Service Loads

The design resistance to shear of a bolt in a slip-critical connection subject to a tensile force $T$ due to service loads shall be computed according to Section J3.8a multiplied by the following reduction factor,

$$\left(1 - \frac{T}{T_b}\right)$$

where

$T_b$ = minimum bolt pre-tension from Table J3.1

## 9b. Slip-Critical Connections Designed at Factored Loads

See Appendix J3.9b.

## 10. Bearing Strength at Bolt Holes

The design bearing strength at bolt holes is $\phi R_n$, where

$\phi$ = 0.75
$R_n$ = nominal bearing strength

Bearing strength shall be checked for both bearing-type and slip-critical connections. The use of oversize holes and short- and long-slotted holes parallel to the line of force is restricted to slip-critical connections per Section J3.2.

In the following sections:

$L_e$ = distance (in.) along the line of force from the edge of the connected part to the center of a standard hole or the center of a short- and long-slotted hole perpendicular to the line of force. For oversize holes and short- and long-slotted holes parallel to the line of force, $L_e$ shall be increased by the increment $C_2$ of Table J3.8.
$s$ = distance (in.) along the line of force between centers of standard holes, or between centers of short- and long-slotted holes perpendicular to the line of force. For oversize holes and short- and long-slotted holes parallel to the line of force, $s$ shall be increased by the spacing increment $C_1$ of Table J3.7.
$d$ = diameter of bolt, in.
$F_u$ = specified minimum tensile strength of the critical part, ksi
$t$ = thickness of the critical connected part, in. For countersunk bolts and rivets, deduct one-half the depth of the countersink.

(a) When $L_e \geq 1.5d$ and $s \geq 3d$ and there are two or more bolts in line of force:

For standard holes; short and long-slotted holes perpendicular to the line of force; oversize holes in slip-critical connections; and long and short-slotted holes in slip-critical connections when the line of force is parallel to the axis of the hole:

When deformation around the bolt holes is a design consideration

$$R_n = 2.4dtF_u \quad \text{(J3-1a)}$$

When deformation around the bolt holes is not a design consideration, for the bolt nearest the edge

$$R_n = L_e t F_u \leq 3.0dtF_u \quad \text{(J3-1b)}$$

## TABLE J3.7
## Values of Spacing Increment $C_1$, in.

| Nominal Diameter of Fastener | Oversize Holes | Slotted Holes | | |
|---|---|---|---|---|
| | | Perpendicular to Line of Force | Parallel to Line of Force | |
| | | | Short-slots | Long-slots [a] |
| ≤7/8 | 1/8 | 0 | 3/16 | $1\frac{1}{2}d - \frac{1}{16}$ |
| 1 | 3/16 | 0 | 1/4 | 1 7/16 |
| ≥1 1/8 | 1/4 | 0 | 5/16 | $1\frac{1}{2}d - \frac{1}{16}$ |

[a] When length of slot is less than maximum allowed in Table J3.5, $C_1$ are permitted to be reduced by the difference between the maximum and actual slot lengths.

## TABLE J3.8
## Values of Edge Distance Increment $C_2$, in.

| Nominal Diameter of Fastener (in.) | Oversized Holes | Slotted Holes | | |
|---|---|---|---|---|
| | | Long Axis Perpendicular to Edge | | Long Axis Parallel to Edge |
| | | Short Slots | Long Slots [a] | |
| ≤7/8 | 1/16 | 1/8 | $\frac{3}{4}d$ | 0 |
| 1 | 1/8 | 1/8 | | |
| ≥1 1/8 | 1/8 | 3/16 | | |

[a] When length of slot is less than maximum allowable (see Table J3.5), $C_2$ are permitted to be reduced by one-half the difference between the maximum and actual slot lengths.

and for the remaining bolts

$$R_n = (s - d/2)tF_u \leq 3.0dtF_u \qquad \text{(J3-1c)}$$

For long-slotted bolt holes perpendicular to the line of force:

$$R_n = 2.0dtF_u \qquad \text{(J3-1d)}$$

(b) When $L_e < 1.5d$ or $s < 3d$ or for a single bolt in the line of force:

For standard holes; short and long-slotted holes perpendicular to the line of force; oversize holes in slip-critical connections; and long and short-slotted holes in slip-critical connections when the line of force is parallel to the axis of the hole:

For a single bolt hole or the bolt hole nearest the edge when there are two or more bolt holes in the line of force

$$R_n = L_e tF_u \leq 2.4dtF_u \qquad \text{(J3-2a)}$$

For the remaining bolt holes

$$R_n = (s - d/2)tF_u \leq 2.4dtF_u \tag{J3-2b}$$

For long-slotted bolt holes perpendicular to the line of force:

For a single bolt hole or the bolt hole nearest the edge where there are two or more bolt holes in the line of force

$$R_n = L_e tF_u \leq 2.0dtF_u \tag{J3-2c}$$

For the remaining bolt holes

$$R_n = (s - d/2)tF_u \leq 2.0dtF_u \tag{J3-2d}$$

## 11. Long Grips

A307 bolts providing design strength, and for which the grip exceeds five diameters, shall have their number increased one percent for each additional $1/16$-in. in the grip.

# J4. DESIGN RUPTURE STRENGTH

## 1. Shear Rupture Strength

The design strength for the limit state of rupture along a shear failure path in the affected elements of connected members shall be taken as $\phi R_n$

where

$\phi = 0.75$
$R_n = 0.6F_u A_{nv}$ (J4-1)
$A_{nv}$ = net area subject to shear, in.$^2$

## 2. Tension Rupture Strength

The design strength for the limit state of rupture along a tension path in the affected elements of connected members shall be taken as $\phi R_n$

where

$\phi = 0.75$
$R_n = F_u A_{nt}$ (J4-2)
$A_{nt}$ = net area subject to tension, in.$^2$

## 3. Block Shear Rupture Strength

Block shear is a limit state in which the resistance is determined by the sum of the shear strength on a failure path(s) and the tensile strength on a perpendicular segment. It shall be checked at beam end connections where the top flange is coped and in similar situations, such as tension members and gusset plates. When ultimate rupture strength on the net section is used to determine the resistance on one segment, yielding on the gross section shall be used on the perpendicular segment. The block shear rupture design strength, $\phi R_n$, shall be determined as follows:

(a) When $F_u A_{nt} \geq 0.6F_u A_{nv}$:

$$\phi R_n = \phi[0.6F_y A_{gv} + F_u A_{nt}] \tag{J4-3a}$$

(b) When $0.6F_uA_{nv} > F_uA_{nt}$:

$$\phi R_n = \phi[0.6F_uA_{nv} + F_yA_{gt}] \tag{J4-3b}$$

where

$\phi$ = 0.75
$A_{gv}$ = gross area subject to shear, in.$^2$
$A_{gt}$ = gross area subject to tension, in.$^2$
$A_{nv}$ = net area subjected to shear, in.$^2$
$A_{nt}$ = net area subjected to tension, in.$^2$

## J5. CONNECTING ELEMENTS

This section applies to the design of connecting elements, such as plates, gussets, angles, brackets, and the panel zones of beam-to-column connections.

### 1. Eccentric Connections

Intersecting axially stressed members shall have their gravity axis intersect at one point, if practicable; if not, provision shall be made for bending and shearing stresses due to the eccentricity. Also see Section J1.8.

### 2. Design Strength of Connecting Elements in Tension

The design strength, $\phi R_n$, of welded, bolted, and riveted connecting elements statically loaded in tension (e.g., splice and gusset plates) shall be the lower value obtained according to limit states of yielding, rupture of the connecting element, and block shear rupture.

(a) For tension yielding of the connecting element:

$$\phi = 0.90$$
$$R_n = A_gF_y \tag{J5-1}$$

(b) For tension rupture of the connecting element:

$$\phi = 0.75$$
$$R_n = A_nF_u \tag{J5-2}$$

where $A_n$ is the net area, not to exceed $0.85A_g$.

(c) For block shear rupture of connecting elements, see Section J4.3.

### 3. Other Connecting Elements

For all other connecting elements, the design strength, $\phi R_n$, shall be determined for the applicable limit state to ensure that the design strength is equal to or greater than the required strength, where $R_n$ is the nominal strength appropriate to the geometry and type of loading on the connecting element. For shear yielding of the connecting element:

$$\phi = 0.90$$
$$R_n = 0.60A_gF_y \tag{J5-3}$$

If the connecting element is in compression an appropriate limit state analysis shall be made.

## J6. FILLERS

In welded construction, any filler ¼-in. or more in thickness shall extend beyond the edges of the splice plate and shall be welded to the part on which it is fitted with sufficient weld to transmit the splice plate load, applied at the surface of the filler. The welds joining the splice plate to the filler shall be sufficient to transmit the splice plate load and shall be long enough to avoid overloading the filler along the toe of the weld. Any filler less than ¼-in. thick shall have its edges made flush with the edges of the splice plate and the weld size shall be the sum of the size necessary to carry the splice plus the thickness of the filler plate.

When bolts or rivets carrying loads pass through fillers thicker than ¼-in., except in connections designed as slip-critical connections, the fillers shall be extended beyond the splice material and the filler extension shall be secured by enough bolts or rivets to distribute the total stress in the member uniformly over the combined section of the member and the filler, or an equivalent number of fasteners shall be included in the connection. Fillers between ¼-in. and ¾-in. thick, inclusive, need not be extended and developed, provided the design shear strength of the bolts is reduced by the factor, $0.4(t - 0.25)$, where $t$ is the total thickness of the fillers, up to ¾-in.

## J7. SPLICES

Groove-welded splices in plate girders and beams shall develop the full strength of the smaller spliced section. Other types of splices in cross sections of plate girders and beams shall develop the strength required by the forces at the point of splice.

## J8. BEARING STRENGTH

The strength of surfaces in bearing is $\phi R_n$, where

$\phi = 0.75$
$R_n$ is defined below for the various types of bearing

(a) For milled surfaces, pins in reamed, drilled, or bored holes, and ends of fitted bearing stiffeners,

$$R_n = 1.8 F_y A_{pb} \tag{J8-1}$$

where

$F_y$ = specified minimum yield stress, ksi
$A_{pb}$ = projected bearing area, in.$^2$

(b) For expansion rollers and rockers,

If $d \leq 25$ in.,

$$R_n = 1.2(F_y - 13)ld / 20 \tag{J8-2}$$

If $d > 25$ in.,

$$R_n = 6.0(F_y - 13)l\sqrt{d}/20 \tag{J8-3}$$

where

$d$ = diameter, in.
$l$ = length of bearing, in.

## J9. COLUMN BASES AND BEARING ON CONCRETE

Proper provision shall be made to transfer the column loads and moments to the footings and foundations.

In the absence of code regulations, design bearing loads on concrete may be taken as $\phi_c P_p$:

(a) On the full area of a concrete support

$$P_p = 0.85 f_c' A_1 \tag{J9-1}$$

(b) On less than the full area of a concrete support

$$P_p = 0.85 f_c' A_1 \sqrt{A_2 / A_1} \tag{J9-2}$$

where

$\phi_c = 0.60$
$A_1$ = area of steel concentrically bearing on a concrete support, in.
$A_2$ = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, in.$^2$
$\sqrt{A_2/A_1} \leq 2$

## J10. ANCHOR BOLTS AND EMBEDMENTS

Anchor bolts and embedments shall be designed in accordance with American Concrete Institute or Prestressed Concrete Institute criteria. If the load factors and combinations given in Section A4.1 are used, a reduction in the $\phi$ factors specified by ACI shall be made based on the ratio of load factors given in Section A4.1 and in ACI.

# CHAPTER K

# CONCENTRATED FORCES, PONDING, AND FATIGUE

This chapter covers member strength design considerations pertaining to concentrated forces, ponding, and fatigue.

## K1. FLANGES AND WEBS WITH CONCENTRATED FORCES

### 1. Design Basis

Sections K1.2 through K1.7 apply to single and double concentrated forces as indicated in each Section. A single concentrated force is tensile or compressive. Double concentrated forces, one tensile and one compressive, form a couple on the same side of the loaded member.

Transverse stiffeners are required at locations of concentrated tensile forces in accordance with Section K1.2 for the flange limit state of local bending, and at unframed ends of beams and girders in accordance with Section K1.8. Transverse stiffeners or doubler plates are required at locations of concentrated forces in accordance with Sections K1.3 through K1.6 for the web limit states of yielding, crippling, sidesway buckling, and compression buckling. Doubler plates or diagonal stiffeners are required in accordance with Section K1.7 for the web limit state of panel-zone shear.

Transverse stiffeners and diagonal stiffeners required by Sections K1.2 through K1.8 shall also meet the requirements of Section K1.9. Doubler plates required by Sections K1.3 through K1.6 shall also meet the requirements of Section K1.10.

### 2. Local Flange Bending

This Section applies to both tensile single-concentrated forces and the tensile component of double-concentrated forces.

A pair of transverse stiffeners extending at least one-half the depth of the web shall be provided adjacent to a concentrated tensile force centrally applied across the flange when the required strength of the flange exceeds $\phi R_n$, where

$$\phi = 0.90$$
$$R_n = 6.25 t_f^2 F_{yf} \qquad \text{(K1-1)}$$

where

$F_{yf}$ = specified minimum yield stress of the flange, ksi
$t_f$ = thickness of the loaded flange, in.

If the length of loading measured across the member flange is less than $0.15b$, where $b$ is the member flange width, Equation K1-1 need not be checked.

When the concentrated force to be resisted is applied at a distance from the member end that is less than $10t_f$, $R_n$ shall be reduced by 50 percent.

When transverse stiffeners are required, they shall be welded to the loaded flange to develop the welded portion of the stiffener. The weld connecting transverse stiffeners to the web shall be sized to transmit the unbalanced force in the stiffener to the web. Also, see Section K1.9.

## 3. Local Web Yielding

This Section applies to single-concentrated forces and both components of double-concentrated forces.

Either a pair of transverse stiffeners or a doubler plate, extending at least one-half the depth of the web, shall be provided adjacent to a concentrated tensile or compressive force when the required strength of the web at the toe of the fillet exceeds $\phi R_n$, where

$\phi = 1.0$

and $R_n$ is determined as follows:

(a) When the concentrated force to be resisted is applied at a distance from the member end that is greater than the depth of the member $d$,

$$R_n = (5k + N)F_{yw}\, t_w \tag{K1-2}$$

(b) When the concentrated force to be resisted is applied at a distance from the member end that is less than or equal to the depth of the member $d$,

$$R_n = (2.5k + N)F_{yw}t_w \tag{K1-3}$$

In Equations K1-2 and K1-3, the following definitions apply:

$F_{yw}$ = specified minimum yield stress of the web, ksi
$N$ = length of bearing (not less than $k$ for end beam reactions), in.
$k$ = distance from outer face of the flange to the web toe of the fillet, in.
$t_w$ = web thickness, in.

When required for a tensile force normal to the flange, transverse stiffeners shall be welded to the loaded flange to develop the connected portion of the stiffener. When required for a compressive force normal to the flange, transverse stiffeners shall either bear on or be welded to the loaded flange to develop the force transmitted to the stiffener. The weld connecting transverse stiffeners to the web shall be sized to transmit the unbalanced force in the stiffener to the web. Also, see Section K1.9.

Alternatively, when doubler plates are required, see Section K1.10.

## 4. Web Crippling

This Section applies to both compressive single-concentrated forces and the compressive component of double-concentrated forces.

Either a transverse stiffener, a pair of transverse stiffeners, or a doubler plate, extending at least one-half the depth of the web, shall be provided adjacent to a concentrated compressive force when the required strength of the web exceeds $\phi R_n$, where

$\phi = 0.75$

and $R_n$ is determined as follows:

(a) When the concentrated compressive force to be resisted is applied at a distance from the member end that is greater than or equal to $d/2$,

$$R_n = 135 t_w^2 \left[ 1 + 3 \left( \frac{N}{d} \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{F_{yw} t_f}{t_w}} \quad \text{(K1-4)}$$

(b) When the concentrated compressive force to be resisted is applied at a distance from the member end that is less than $d/2$,

For $N/d \leq 0.2$,

$$R_n = 68 t_w^2 \left[ 1 + 3 \left( \frac{N}{d} \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{F_{yw} t_f}{t_w}} \quad \text{(K1-5a)}$$

For $N/d > 0.2$,

$$R_n = 68 t_w^2 \left[ 1 + \left( \frac{4N}{d} - 0.2 \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{F_{yw} t_f}{t_w}} \quad \text{(K1-5b)}$$

In Equations K1-4 and K1-5, the following definitions apply:

$d$ = overall depth of the member, in.
$t_f$ = flange thickness, in.

When transverse stiffeners are required, they shall either bear on or be welded to the loaded flange to develop the force transmitted to the stiffener. The weld connecting transverse stiffeners to the web shall be sized to transmit the unbalanced force in the stiffener to the web. Also, see Section K1.9.

Alternatively, when doubler plates are required, see Section K1.10.

## 5. Sidesway Web Buckling

This Section applies only to compressive single-concentrated forces applied to members where relative lateral movement between the loaded compression flange and the tension flange is not restrained at the point of application of the concentrated force.

The design strength of the web is $\phi R_n$, where

$\phi = 0.85$

and $R_n$ is determined as follows:

(a) If the compression flange is restrained against rotation:

for $(h / t_w) / (l / b_f) \leq 2.3$,

$$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[1 + 0.4\left(\frac{h / t_w}{l / b_f}\right)^3\right] \tag{K1-6}$$

for $(h / t_w) / (l / b_f) > 2.3$, the limit state of sidesway web buckling does not apply.

When the required strength of the web exceeds $\phi R_n$, local lateral bracing shall be provided at the tension flange or either a pair of transverse stiffeners or a doubler plate, extending at least one-half the depth of the web, shall be provided adjacent to the concentrated compressive force.

When transverse stiffeners are required, they shall either bear on or be welded to the loaded flange to develop the full applied force. The weld connecting transverse stiffeners to the web shall be sized to transmit the force in the stiffener to the web. Also, see Section K1.9.

Alternatively, when doubler plates are required, they shall be sized to develop the full applied force. Also, see Section K1.10.

(b) If the compression flange is *not* restrained against rotation:

for $(h / t_w) / (l / b_f) \leq 1.7$,

$$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[0.4\left(\frac{h / t_w}{l / b_f}\right)^3\right] \tag{K1-7}$$

for $(h / t_w) / (l / b_f) > 1.7$, the limit state of sidesway web buckling does not apply.

When the required strength of the web exceeds $\phi R_n$, local lateral bracing shall be provided at both flanges at the point of application of the concentrated forces.

In Equations K1-6 and K1-7, the following definitions apply:

$l$ = largest laterally unbraced length along either flange at the point of load, in.
$b_f$ = flange width, in.
$t_w$ = web thickness, in.
$h$ = clear distance between flanges less the fillet or corner radius for rolled shapes; distance between adjacent lines of fasteners or the clear distance between flanges when welds are used for built-up shapes, in.
$C_r$ = 960,000 when $M_u < M_y$ at the location of the force, ksi
= 480,000 when $M_u \geq M_y$ at the location of the force, ksi

## 6. Compression Buckling of the Web

This Section applies to a pair of compressive single-concentrated forces or the compressive components in a pair of double-concentrated forces, applied at both flanges of a member at the same location.

Either a single transverse stiffener, or pair of transverse stiffeners, or a doubler plate, extending the full depth of the web, shall be provided adjacent to concentrated compressive forces at both flanges when the required strength of the web exceeds $\phi R_n$, where

$$\phi = 0.90$$

and

$$R_n = \frac{4{,}100 t_w^3 \sqrt{F_{yw}}}{h} \tag{K1-8}$$

When the pair of concentrated compressive forces to be resisted is applied at a distance from the member end that is less than $d/2$, $R_n$ shall be reduced by 50 percent.

When transverse stiffeners are required, they shall either bear on or be welded to the loaded flange to develop the force transmitted to the stiffener. The weld connecting transverse stiffeners to the web shall be sized to transmit the unbalanced force in the stiffener to the web. Also, see Section K1.9.

Alternatively, when doubler plates are required, see Section K1.10.

## 7. Panel-Zone Web Shear

Either doubler plates or diagonal stiffeners shall be provided within the boundaries of the rigid connection of members whose webs lie in a common plane when the required strength exceeds $\phi R_v$, where

$$\phi = 0.90$$

and $R_v$ is determined as follows:

(a) When the effect of panel-zone deformation on frame stability is *not* considered in the analysis,

For $P_u \le 0.4P_y$

$$R_v = 0.60 F_y d_c t_w \tag{K1-9}$$

For $P_u > 0.4P_y$

$$R_v = 0.60 F_y d_c t_w \left(1.4 - \frac{P_u}{P_y}\right) \tag{K1-10}$$

(b) When frame stability, including plastic panel-zone deformation, is considered in the analysis:

For $P_u \le 0.75P_y$

$$R_v = 0.60 F_y d_c t_w \left(1 + \frac{3b_{cf} t_{cf}^2}{d_b d_c t}\right) \tag{K1-11}$$

For $P_u > 0.75P_y$

$$R_v = 0.60 F_y d_c t_w \left(1 + \frac{3b_{cf} t_{cf}^2}{d_b d_c t_w}\right)\left(1.9 - \frac{1.2P_u}{P_y}\right) \tag{K1-12}$$

In Equations K1-9 through K1-12, the following definitions apply:

$t_w$ = column web thickness, in.
$b_{cf}$ = width of column flange, in.
$t_{cf}$ = thickness of the column flange, in.
$d_b$ = beam depth, in.
$d_c$ = column depth, in.
$F_y$ = yield strength of the column web, in.
$P_y$ = $F_y A$, axial yield strength of the column, in.
$A$ = column cross-sectional area, in.

When doubler plates are required, they shall meet the criteria of Section F2 and shall be welded to develop the proportion of the total shear force which is to be carried.

Alternatively, when diagonal stiffeners are required, the weld connecting diagonal stiffeners to the web shall be sized to transmit the stiffener force caused by unbalanced moments to the web. Also, see Section K1.9.

## 8. Unframed Ends of Beams and Girders

At unframed ends of beams and girders not otherwise restrained against rotation about their longitudinal axes, a pair of transverse stiffeners, extending the full depth of the web, shall be provided. Also, see Section K1.9.

## 9. Additional Stiffener Requirements for Concentrated Forces

Transverse and diagonal stiffeners shall also comply with the following criteria:

(1) The width of each stiffener plus one-half the thickness of the column web shall not be less than one-third of the width of the flange or moment connection plate delivering the concentrated force.

(2) The thickness of a stiffener shall not be less than one-half the thickness of the flange or moment connection plate delivering the concentrated load, and not less than its width times $\sqrt{F_y}\,/\,95$.

Full depth transverse stiffeners for compressive forces applied to a beam or plate girder flange shall be designed as axially compressed members (columns) in accordance with the requirements of Section E2, with an effective length of $0.75h$, a cross section composed of two stiffeners and a strip of the web having a width of $25t_w$ at interior stiffeners and $12t_w$ at the ends of members.

The weld connecting bearing stiffeners to the web shall be sized to transmit the excess web shear force to the stiffener. For fitted bearing stiffeners, see Section J8.1.

## 10. Additional Doubler Plate Requirements for Concentrated Forces

Doubler plates required by Sections K1.3 through K1.6 shall also comply with the following criteria:

(1) The thickness and extent of the doubler plate shall provide the additional material necessary to equal or exceed the strength requirements.

(2) The doubler plate shall be welded to develop the proportion of the total force transmitted to the doubler plate.

## K2. PONDING

The roof system shall be investigated by structural analysis to assure adequate strength and stability under ponding conditions, unless the roof surface is provided with sufficient slope toward points of free drainage or adequate individual drains to prevent the accumulation of rainwater.

The roof system shall be considered stable and no further investigation is needed if:

$$C_p + 0.9C_s \leq 0.25 \tag{K2-1}$$

$$I_d \geq 25(S^4)10^{-6} \tag{K2-2}$$

where

$$C_p = \frac{32 L_s L_p^4}{10^7 I_p}$$

$$C_s = \frac{32 S L_s^4}{10^7 I_s}$$

$L_p$ = column spacing in direction of girder (length of primary members), ft
$L_s$ = column spacing perpendicular to direction of girder (length of secondary members), ft
$S$ = spacing of secondary members, ft
$I_p$ = moment of inertia of primary members, in.$^4$
$I_s$ = moment of inertia of secondary members, in.$^4$
$I_d$ = moment of inertia of the steel deck supported on secondary members, in.$^4$ per ft

For trusses and steel joists, the moment of inertia $I_s$ shall be decreased 15 percent when used in the above equation. A steel deck shall be considered a secondary member when it is directly supported by the primary members.

See Appendix K2 for an alternate determination of flat roof framing stiffness.

## K3. FATIGUE

Few members or connections in conventional buildings need to be designed for fatigue, since most load changes in such structures occur only a small number of times or produce only minor stress fluctuations. The occurrence of full design wind or earthquake loads is too infrequent to warrant consideration in fatigue design. However, crane runways and supporting structures for machinery and equipment are often subject to fatigue loading conditions.

Members and their connections subject to fatigue loading shall be proportioned in accordance with the provisions of Appendix K3 for service loads.

# CHAPTER L

# SERVICEABILITY DESIGN CONSIDERATIONS

This chapter is intended to provide design guidance for serviceability considerations.

Serviceability is a state in which the function of a building, its appearance, maintainability, durability, and comfort of its occupants are preserved under normal usage. The general design requirement for serviceability is given in Section A5.4. Limiting values of structural behavior to ensure serviceability (e.g., maximum deflections, accelerations, etc.) shall be chosen with due regard to the intended function of the structure. Where necessary, serviceability shall be checked using realistic loads for the appropriate serviceability limit state.

## L1. CAMBER

If any special camber requirements are necessary to bring a loaded member into proper relation with the work of other trades, as for the attachment of runs of sash, the requirements shall be set forth in the design documents.

Beams and trusses detailed without specified camber shall be fabricated so that after erection any camber due to rolling or shop assembly shall be upward. If camber involves the erection of any member under a preload, this shall be noted in the design documents.

## L2. EXPANSION AND CONTRACTION

Adequate provision shall be made for expansion and contraction appropriate to the service conditions of the structure.

## L3. DEFLECTIONS, VIBRATION, AND DRIFT

### 1. Deflections

Deformations in structural members and structural systems due to service loads shall not impair the serviceability of the structure.

### 2. Floor Vibration

Vibration shall be considered in designing beams and girders supporting large areas free of partitions or other sources of damping where excessive vibration due to pedestrian traffic or other sources within the building is not acceptable.

### 3. Drift

Lateral deflection or drift of structures due to code-specified wind or seismic loads shall not cause collision with adjacent structures nor exceed the limiting values of such drifts which may be specified or appropriate.

## L4. CONNECTION SLIP

For the design of slip-critical connections see Sections J3.8 and J3.9.

## L5. CORROSION

When appropriate, structural components shall be designed to tolerate corrosion or shall be protected against corrosion that may impair the strength or serviceability of the structure.

# CHAPTER M

# FABRICATION, ERECTION, AND QUALITY CONTROL

This chapter provides requirements for shop drawings, fabrication, shop painting, erection, and quality control.

## M1. SHOP DRAWINGS

Shop drawings giving complete information necessary for the fabrication of the component parts of the structure, including the location, type, and size of all welds, bolts, and rivets, shall be prepared in advance of the actual fabrication. These drawings shall clearly distinguish between shop and field welds and bolts and shall clearly identify slip-critical high-strength bolted connections.

Shop drawings shall be made in conformity with good practice and with due regard to speed and economy in fabrication and erection.

## M2. FABRICATION

### 1. Cambering, Curving, and Straightening

Local application of heat or mechanical means is permitted to be used to introduce or correct camber, curvature, and straightness. The temperature of heated areas, as measured by approved methods, shall not exceed 1,100°F for A514 and A852 steel nor 1,200°F for other steels.

### 2. Thermal Cutting

Thermally cut edges shall meet the requirements of AWS 3.2.2 with the exception that thermally cut free edges which will be subject to calculated static tensile stress shall be free of round bottom gouges greater than $\frac{3}{16}$-in. deep and sharp V-shaped notches. Gouges greater than $\frac{3}{16}$-in. deep and notches shall be removed by grinding or repaired by welding.

Re-entrant corners, except re-entrant corners of beam copes and weld access holes, shall meet the requirements of AWS 3.2.4. If other specified contour is required it must be shown on the contract documents.

Beam copes and weld access holes shall meet the geometrical requirements of Section J1.6. For beam copes and weld access holes in ASTM A6 Group 4 and 5 shapes and welded built-up shapes with material thickness greater than two inches, a preheat temperature of not less than 150°F shall be applied prior to thermal cutting.

### 3. Planing of Edges

Planing or finishing of sheared or thermally cut edges of plates or shapes is not required unless specifically called for in the design documents or included in a stipulated edge preparation for welding.

### 4. Welded Construction

The technique of welding, the workmanship, appearance, and quality of welds and the methods used in correcting nonconforming work shall be in accordance with AWS D1.1 except as modified in Section J2.

### 5. Bolted Construction

All parts of bolted members shall be pinned or bolted and rigidly held together during assembly. Use of a drift pin in bolt holes during assembly shall not distort the metal or enlarge the holes. Poor matching of holes shall be cause for rejection.

If the thickness of the material is not greater than the nominal diameter of the bolt plus ⅛-in., the holes are permitted to be punched. If the thickness of the material is greater than the nominal diameter of the bolt plus ⅛-in., the holes shall be either drilled or sub-punched and reamed. The die for all sub-punched holes, and the drill for all sub-drilled holes, shall be at least 1/16-in. smaller than the nominal diameter of the bolt. Holes in A514 steel plates over ½-in. thick shall be drilled.

Fully inserted finger shims, with a total thickness of not more than ¼-in. within a joint, are permitted in joints without changing the design strength (based upon hole type) for the design of connections. The orientation of such shims is independent of the direction of application of the load.

The use of high-strength bolts shall conform to the requirements of the RCSC *Load and Resistance Factor Design Specification for Structural Joints Using ASTM A325 or A490 Bolts.*

### 6. Compression Joints

Compression joints which depend on contact bearing as part of the splice strength shall have the bearing surfaces of individual fabricated pieces prepared by milling, sawing, or other suitable means.

### 7. Dimensional Tolerances

Dimensional tolerances shall be in accordance with the AISC *Code of Standard Practice.*

### 8. Finish of Column Bases

Column bases and base plates shall be finished in accordance with the following requirements:

(1) Steel bearing plates two inches or less in thickness are permitted without milling, provided a satisfactory contact bearing is obtained. Steel bearing plates over two inches but not over four inches in thickness are permitted to be straightened by pressing or, if presses are not available, by milling for all bearing surfaces (except as noted in subparagraphs 2 and 3 of this section), to obtain a satisfactory contact bearing. Steel bearing plates over four inches

in thickness shall be milled for all bearing surfaces (except as noted in subparagraphs 2 and 3 of this section).

(2) Bottom surfaces of bearing plates and column bases which are grouted to ensure full bearing contact on foundations need not be milled.

(3) Top surfaces of bearing plates need not be milled when full-penetration welds are provided between the column and the bearing plate.

## M3. SHOP PAINTING

### 1. General Requirements

Shop painting and surface preparation shall be in accordance with the provisions of the AISC *Code of Standard Practice*.

Shop paint is not required unless specified by the contract documents.

### 2. Inaccessible Surfaces

Except for contact surfaces, surfaces inaccessible after shop assembly shall be cleaned and painted prior to assembly, if required by the design documents.

### 3. Contact Surfaces

Paint is permitted unconditionally in bearing-type connections. For slip-critical connections, the faying surface requirements shall be in accordance with the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, paragraph 3(b).

### 4. Finished Surfaces

Machine-finished surfaces shall be protected against corrosion by a rust-inhibitive coating that can be removed prior to erection, or which has characteristics that make removal prior to erection unnecessary.

### 5. Surfaces Adjacent to Field Welds

Unless otherwise specified in the design documents, surfaces within two inches of any field weld location shall be free of materials that would prevent proper welding or produce objectionable fumes during welding.

## M4. ERECTION

### 1. Alignment of Column Bases

Column bases shall be set level and to correct elevation with full bearing on concrete or masonry.

### 2. Bracing

The frame of steel skeleton buildings shall be carried up true and plumb within the limits defined in the AISC *Code of Standard Practice*. Temporary bracing shall be provided, in accordance with the requirements of the *Code of Standard Practice*, wherever necessary to support all loads to which the structure may be subjected, including equipment and the operation of same. Such bracing shall be left in place as long as required for safety.

### 3. Alignment

No permanent bolting or welding shall be performed until the adjacent affected portions of the structure have been properly aligned.

### 4. Fit of Column Compression Joints and Base Plates

Lack of contact bearing not exceeding a gap of $\frac{1}{16}$-in., regardless of the type of splice used (partial-joint-penetration groove welded, or bolted), is permitted. If the gap exceeds $\frac{1}{16}$-in., but is less than $\frac{1}{4}$-in., and if an engineering investigation shows that sufficient contact area does not exist, the gap shall be packed out with non-tapered steel shims. Shims need not be other than mild steel, regardless of the grade of the main material.

### 5. Field Welding

Shop paint on surfaces adjacent to joints to be field welded shall be wire brushed if necessary to assure weld quality.

Field welding of attachments to installed embedments in contact with concrete shall be done in such a manner as to avoid excessive thermal expansion of the embedment which could result in spalling or cracking of the concrete or excessive stress in the embedment anchors.

### 6. Field Painting

Responsibility for touch-up painting, cleaning, and field painting shall be allocated in accordance with accepted local practices, and this allocation shall be set forth explicitly in the design documents.

### 7. Field Connections

As erection progresses, the structure shall be securely bolted or welded to support all dead, wind, and erection loads.

## M5. QUALITY CONTROL

The fabricator shall provide quality control procedures to the extent that the fabricator deems necessary to assure that all work is performed in accordance with this Specification. In addition to the fabricator's quality control procedures, material and workmanship at all times may be subject to inspection by qualified inspectors representing the purchaser. If such inspection by representatives of the purchaser will be required, it shall be so stated in the design documents.

### 1. Cooperation

As far as possible, all inspection by representatives of the purchaser shall be made at the fabricator's plant. The fabricator shall cooperate with the inspector, permitting access for inspection to all places where work is being done. The purchaser's inspector shall schedule this work for minimum interruption to the work of the fabricator.

### 2. Rejections

Material or workmanship not in reasonable conformance with the provisions of this Specification may be rejected at any time during the progress of the work.

The fabricator shall receive copies of all reports furnished to the purchaser by the inspection agency.

## 3. Inspection of Welding

The inspection of welding shall be performed in accordance with the provisions of AWS D1.1 except as modified in Section J2.

When visual inspection is required to be performed by AWS certified welding inspectors, it shall be so specified in the design documents.

When nondestructive testing is required, the process, extent, and standards of acceptance shall be clearly defined in the design documents.

## 4. Inspection of Slip-Critical High-Strength Bolted Connections

The inspection of slip-critical high-strength bolted connections shall be in accordance with the provisions of the RCSC *Load and Resistance Factor Design Specification for Structural Joints Using ASTM A325 or A490 Bolts.*

## 5. Identification of Steel

The fabricator shall be able to demonstrate by a written procedure and by actual practice a method of material application and identification, visible at least through the "fit-up" operation, of the main structural elements of a shipping piece.

The identification method shall be capable of verifying proper material application as it relates to:

(1) Material specification designation

(2) Heat number, if required

(3) Material test reports for special requirements.

# APPENDIX B

# DESIGN REQUIREMENTS

Appendix B5.1 provides an expanded definition of limiting width-thickness ratio for webs in combined flexure and axial compression. Appendix B5.3 applies to the design of members containing slender compression elements.

## B5. LOCAL BUCKLING

### 1. Classification of Steel Sections

For members with unequal flanges and with webs in combined flexural and axial compression, $\lambda_r$ for the limit state of web local buckling is

$$\lambda_r = \frac{253}{\sqrt{F_y}}\left[1 + 2.83\left(\frac{h}{h_c}\right)\left(1 - \frac{P_u}{\phi_b P_y}\right)\right] \qquad \text{(A-B5-1)}$$

$$\frac{3}{4} \leq \frac{h}{h_c} \leq \frac{3}{2}$$

For members with unequal flanges with webs subjected to flexure only, $\lambda_r$ for the limit state of web local buckling is

$$\lambda_r = \frac{253}{\sqrt{F_y}}\left[1 + 2.83\left(\frac{h}{h_c}\right)\right] \qquad \text{(A-B5-2)}$$

$$\frac{3}{4} \leq \frac{h}{h_c} \leq \frac{3}{2}$$

where $\lambda_r$, $h$, and $h_c$ are as defined in Section B5.1.

These substitutions shall be made in Appendices F and G when applied to members with unequal flanges. If the compression flange is larger than the tension flange, $\lambda_r$ shall be determined using Equation A-B5-1, A-B5-2, or Table B5.1.

### 3. Slender-Element Compression Sections

Axially loaded members containing elements subject to compression which have a width-thickness ratio in excess of the applicable $\lambda_r$ as stipulated in Section B5.1 shall be proportioned according to this Appendix. Flexural members with slender compression elements shall be designed in accordance with Appendices F and G. Flexural members with proportions not covered by Appendix F1 shall be designed in accordance with this Appendix.

### 3a. Unstiffened Compression Elements

The design strength of unstiffened compression elements whose width-thickness ratio exceeds the applicable limit $\lambda_r$ as stipulated in Section B5.1 shall be subject to a reduction factor $Q_s$. The value of $Q_s$ shall be determined by Equations A-B5-3 through A-B5-10, as applicable. When such elements comprise the compression flange of a flexural member, the maximum required bending stress shall not exceed $\phi_b F_y Q_s$, where $\phi_b = 0.90$. The design strength of axially loaded compression members shall be modified by the appropriate reduction factor $Q$, as provided in Appendix B5.3c.

(a) For single angles:

when $76.0/\sqrt{F_y} < b/t < 155/\sqrt{F_y}$:

$$Q_s = 1.340 - 0.00447(b/t)\sqrt{F_y} \tag{A-B5-3}$$

when $b/t > 155/\sqrt{F_y}$:

$$Q_s = 15{,}500/[F_y(b/t)^2] \tag{A-B5-4}$$

(b) For flanges, angles, and plates projecting from rolled beams or columns or other compression members:

when $95.0/\sqrt{F_y} < b/t < 176/\sqrt{F_y}$:

$$Q_s = 1.415 - 0.00437(b/t)\sqrt{F_y} \tag{A-B5-5}$$

when $b/t \geq 176/\sqrt{F_y}$:

$$Q_s = 20{,}000/[F_y(b/t)^2] \tag{A-B5-6}$$

(c) For flanges, angles and plates projecting from built-up columns or other compression members:

when $109/\sqrt{F_y/k_c} < b/t < 200/\sqrt{F_y/k_c}$:

$$Q_s = 1.415 - 0.00381(b/t)\sqrt{F_y/k_c} \tag{A-B5-7}$$

when $b/t \geq 200/\sqrt{F_y/k_c}$:

$$Q_s = 26{,}200k_c/[F_y(b/t)^2] \tag{A-B5-8}$$

The coefficient, $k_c$, shall be computed as follows:

(a) For I-shaped sections:

$$k_c = \frac{4}{\sqrt{h/t_w}},\ 0.35 \leq k_c \leq 0.763$$

where:

$h$ = depth of web, in.
$t_w$ = thickness of web, in.

(b) For other sections:

$k_c = 0.763$

(d) For stems of tees:

when $127/\sqrt{F_y} < b/t < 176/\sqrt{F_y}$:

$$Q_s = 1.908 - 0.00715(b/t)\sqrt{F_y} \qquad \text{(A-B5-9)}$$

when $b/t \geq 176/\sqrt{F_y}$:

$$Q_s = 20{,}000/[F_y(b/t)^2] \qquad \text{(A-B5-10)}$$

where

$b$ = width of unstiffened compression element as defined in Section B5.1, in.
$t$ = thickness of unstiffened element, in.
$F_y$ = specified minimum yield stress, ksi

## 3b. Stiffened Compression Elements

When the width-thickness ratio of uniformly compressed stiffened elements (except perforated cover plates) exceeds the limit $\lambda_r$ stipulated in Section B5.1, a reduced effective width $b_e$ shall be used in computing the design properties of the section containing the element.

(a) For flanges of square and rectangular sections of uniform thickness:

when $\dfrac{b}{t} \geq \dfrac{238}{\sqrt{f}}$:

$$b_e = \frac{326t}{\sqrt{f}}\left[1 - \frac{64.9}{(b/t)\sqrt{f}}\right] \qquad \text{(A-B5-11)}$$

otherwise $b_e = b$.

(b) For other uniformly compressed elements:

when $\dfrac{b}{t} \geq \dfrac{253}{\sqrt{f}}$:

$$b_e = \frac{326t}{\sqrt{f}}\left[1 - \frac{57.2}{(b/t)\sqrt{f}}\right] \qquad \text{(A-B5-12)}$$

otherwise $b_e = b$

where

$b$ = actual width of a stiffened compression element, as defined in Section B5.1, in.
$b_e$ = reduced effective width, in.
$t$ = element thickness, in.
$f$ = computed elastic compressive stress in the stiffened elements, based on the design properties as specified in Appendix B5.3c, ksi. If unstiffened elements are included in the total cross section, $f$ for the stiffened element must be such that the maximum compressive stress in the unstiffened element does not exceed $\phi_c F_{cr}$ as defined in Appendix B5.3c with $Q = Q_s$ and $\phi_c = 0.85$, or $\phi_b F_y Q_s$ with $\phi_b = 0.90$, as applicable.

(c) For axially loaded circular sections with diameter-to-thickness ratio $D/t$ greater than $3{,}300/F_y$ but less than $13{,}000/F_y$

$$Q = Q_a = \frac{1{,}100}{F_y(D/t)} + \frac{2}{3} \tag{A-B5-13}$$

where

$D$ = outside diameter, in.
$t$ = wall thickness, in.

### 3c. Design Properties

Properties of sections shall be determined using the full cross section, except as follows:

In computing the moment of inertia and elastic section modulus of flexural members, the effective width of uniformly compressed stiffened elements $b_e$, as determined in Appendix B5.3b, shall be used in determining effective cross-sectional properties.

For unstiffened elements of the cross section, $Q_s$ is determined from Appendix B5.3a. For stiffened elements of the cross section

$$Q_a = \frac{\text{effective area}}{\text{actual area}} \tag{A-B5-14}$$

where the effective area is equal to the summation of the effective areas of the cross section.

### 3d. Design Strength

For axially loaded compression members the gross cross-sectional area and the radius of gyration $r$ shall be computed on the basis of the actual cross section. The critical stress $F_{cr}$ shall be determined as follows:

(a) For $\lambda_c\sqrt{Q} \le 1.5$:

$$F_{cr} = Q(0.658^{Q\lambda_c^2})F_y \tag{A-B5-15}$$

(a) For $\lambda_c\sqrt{Q} > 1.5$:

$$F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right]F_y \tag{A-B5-16}$$

where

$$Q = Q_sQ_a \tag{A-B5-17}$$

Cross sections comprised of only unstiffened elements, $Q = Q_s$, ($Q_a$ = 1.0)

Cross sections comprised of only stiffened elements, $Q = Q_a$, ($Q_s$ = 1.0)

Cross sections comprised of both stiffened and unstiffened elements, Q = $Q_sQ_a$

# APPENDIX E

# COLUMNS AND OTHER COMPRESSION MEMBERS

This Appendix applies to the strength of doubly symmetric columns with thin plate elements, singly symmetric and unsymmetric columns for the limit states of flexural-torsional and torsional buckling.

## E3. DESIGN COMPRESSIVE STRENGTH FOR FLEXURAL-TORSIONAL BUCKLING

The strength of compression members determined by the limit states of torsional and flexural-torsional buckling is $\phi_c P_n$,

where

$\phi_c = 0.85$
$P_n$ = nominal resistance in compression, kips
$= A_g F_{cr}$ (A-E3-1)
$A_g$ = gross area of cross section, in.$^2$

The nominal critical stress $F_{cr}$ is determined as follows:

(a) For $\lambda_e\sqrt{Q} \leq 1.5$:

$$F_{cr} = Q(0.658^{Q\lambda_e^2})F_y \qquad \text{(A-E3-2)}$$

(b) For $\lambda_e\sqrt{Q} > 1.5$:

$$F_{cr} = \left[\frac{0.877}{\lambda_e^2}\right]F_y \qquad \text{(A-E3-3)}$$

where

$$\lambda_e = \sqrt{F_y / F_e} \qquad \text{(A-E3-4)}$$

$F_y$ = specified minimum yield stress of steel, ksi
$Q$ = 1.0 for elements meeting the width-thickness ratios $\lambda_r$ of Section B5.1
= $Q_sQ_a$ for elements not meeting the width-thickness ratios $\lambda_r$ of Section B5.1 and determined in accordance with the provisions of Appendix B5.3

The critical torsional or flexural-torsional elastic buckling stress $F_e$ is determined as follows:

(a) For doubly symmetric shapes:

$$F_e = \left[\frac{\pi^2 E C_w}{(K_z l)^2} + GJ\right]\frac{1}{I_x + I_y} \tag{A-E3-5}$$

(b) For singly symmetric shapes where $y$ is the axis of symmetry:

$$F_e = \frac{F_{ey} + F_{ez}}{2H}\left(1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{(F_{ey} + F_{ez})^2}}\right) \tag{A-E3-6}$$

(c) For unsymmetric shapes, the critical flexural-torsional elastic buckling stress $F_e$ is the lowest root of the cubic equation

$$(F_e - F_{ex})(F_e - F_{ey})(F_e - F_{ez}) - F_e^2(F_e - F_{ey})\left(\frac{x_o}{\bar{r}_o}\right)^2 - F_e^2(F_e - F_{ex})\left(\frac{y_o}{\bar{r}_o}\right)^2 = 0 \tag{A-E3-7}$$

where

$K_z$ = effective length factor for torsional buckling
$E$ = modulus of elasticity, ksi
$G$ = shear modulus, ksi
$C_w$ = warping constant, in.$^6$
$J$ = torsional constant, in.$^4$
$I_x$, $I_y$ = moment of inertia about the principal axes, in.$^4$
$x_o$, $y_o$ = coordinates of shear center with respect to the centroid, in.

$$\bar{r}_o^{\,2} = x_o^2 + y_o^2 + \frac{I_x + I_y}{A} \tag{A-E3-8}$$

$$H = 1 - \left(\frac{x_o^2 + y_o^2}{\bar{r}_o^{\,2}}\right) \tag{A-E3-9}$$

$$F_{ex} = \frac{\pi^2 E}{(K_x l / r_x)^2} \tag{A-E3-10}$$

$$F_{ey} = \frac{\pi^2 E}{(K_y l / r_y)^2} \tag{A-E3-11}$$

$$F_{ez} = \left(\frac{\pi^2 E C_w}{(K_z l)^2} + GJ\right)\frac{1}{A\bar{r}_o^{\,2}} \tag{A-E3-12}$$

where

$A$ = cross-sectional area of member, in.$^2$
$l$ = unbraced length, in.
$K_x$, $K_y$ = effective length factors in $x$ and $y$ directions
$r_x$, $r_y$ = radii of gyration about the principal axes, in.
$\bar{r}_o$ = polar radius of gyration about the shear center, in.

# APPENDIX F

# BEAMS AND OTHER FLEXURAL MEMBERS

Appendix F1 provides the design flexural strength of beams and girders. Appendix F2 provides the design shear strength of webs with and without stiffeners and requirements on transverse stiffeners. Appendix F3 applies to web-tapered members.

## F1. DESIGN FOR FLEXURE

The design strength for flexural members is $\phi_b M_n$ where $\phi_b = 0.90$ and $M_n$ is the nominal strength.

Table A-F1.1 provides a tabular summary of Equations F1-1 through F1-15 for determining the nominal flexural strength of beams and girders. For slenderness parameters of cross sections not included in Table A-F1.1, see Appendix B5.3. For flexural members with unequal flanges see Appendix B5.1 for the determination of $\lambda_r$ for the limit state of web local buckling.

The nominal flexural strength $M_n$ is the lowest value obtained according to the limit states of yielding: lateral-torsional buckling (LTB); flange local buckling (FLB); and web local buckling (WLB).

The nominal flexural strength $M_n$ shall be determined as follows for each limit state:

(a) For $\lambda \leq \lambda_p$:

$$M_n = M_p \tag{A-F1-1}$$

(b) For $\lambda_p < \lambda \leq \lambda_r$:

For the limit state of lateral-torsional buckling:

$$M_n = C_b \left[ M_p - (M_p - M_r)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right)\right] \leq M_p \tag{A-F1-2}$$

For the limit states of flange and web local buckling:

$$M_n = M_p - (M_p - M_r)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right) \tag{A-F1-3}$$

(c) For $\lambda > \lambda_r$:

For the limit state of lateral-torsional buckling and flange local buckling:

$$M_n = M_{cr} = SF_{cr} \leq M_p \tag{A-F1-4}$$

For design of girders with slender webs, the limit state of web local buckling is not applicable. See Appendix G2.

For $\lambda$ of the flange > $\lambda_r$ in shapes not included in Table A-F1.1, see Appendix B5.3.

For $\lambda$ of the web > $\lambda_r$, see Appendix G.

The terms used in the above equations are:

$M_n$ = nominal flexural strength, kip-in.
$M_p$ = $F_y Z$, plastic moment $\leq 1.5 F_y S$, kip-in.
$M_{cr}$ = buckling moment, kip-in.
$M_r$ = limiting buckling moment (equal to $M_{cr}$ when $\lambda = \lambda_r$), kip-in.
$\lambda$ = controlling slenderness parameter
= minor axis slenderness ratio $L_b / r_y$ for lateral-torsional buckling
= flange width-thickness ratio $b / t$ for flange local buckling as defined in Section B5.1
= web depth-thickness ratio $h / t_w$ for web local buckling as defined in Section B5.1
$\lambda_p$ = largest value of $\lambda$ for which $M_n = M_p$
$\lambda_r$ = largest value of $\lambda$ for which buckling is inelastic
$F_{cr}$ = critical stress, ksi
$C_b$ = Bending coefficient dependent on moment gradient, see Section F1.2a, Equation F1-3
$S$ = section modulus, in.$^3$
$L_b$ = laterally unbraced length, in.
$r_y$ = radius of gyration about minor axis, in.

The applicable limit states and equations for $M_p$, $M_r$, $F_{cr}$, $\lambda$, $\lambda_p$, and $\lambda_r$ are given in Table A-F1.1 for shapes covered in this Appendix. The terms used in the table are:

$A$ = cross-sectional area, in.$^2$
$F_L$ = smaller of $(F_{yf} - F_r)$ or $F_{yw}$, ksi
$F_r$ = compressive residual stress in flange
= 10 ksi for rolled shapes
= 16.5 ksi for welded shapes
$F_y$ = specified minimum yield strength, ksi
$F_{yf}$ = yield strength of the flange, ksi
$F_{yw}$ = yield strength of the web, ksi
$I_{yc}$ = moment of inertia of compression flange about $y$ axis or if reverse curvature bending, moment of inertia of smaller flange, in.$^4$
$J$ = torsional constant, in.$^4$
$R_e$ = see Appendix G2
$S_{eff}$ = effective section modulus about major axis, in.$^3$
$S_{xc}$ = section modulus of the outside fiber of the compression flange, in.$^3$
$S_{xt}$ = section modulus of the outside fiber of the tension flange, in.$^3$
$Z$ = plastic section modulus, in.$^3$
$b$ = flange width, in.
$d$ = overall depth, in.

$f$ = computed compressive stress in the stiffened element, ksi
$h$ = clear distance between flanges less the fillet or corner radius at each flange, in.
$r_{yc}$ = radius of gyration of compression flange about $y$ axis or if reverse curvature bending, smaller flange, in.
$t_f$ = flange thickness, in.
$t_w$ = web thickness, in.

## F2. DESIGN FOR SHEAR

### 2. Design Shear Strength

The design shear strength of stiffened or unstiffened webs is $\phi_v V_n$,

where

$\phi_v = 0.90$
$V_n$ = nominal shear strength defined as follows:

For $h / t_w \leq 187\sqrt{k_v / F_{yw}}$:

$$V_n = 0.6F_{yw}A_w \qquad \text{(A-F2-1)}$$

For $187\sqrt{k_v / F_{yw}} < h / t_w \leq 234\sqrt{k_v / F_{yw}}$:

$$V_n = 0.6F_{yw}A_w(187\sqrt{k_v / F_{yw}}) / (h / t_w) \qquad \text{(A-F2-2)}$$

For $h / t_w > 234\sqrt{k_v / F_{yw}}$:

$$V_n = A_w(26{,}400k_v) / (h / t_w)^2 \qquad \text{(A-F2-3)}$$

where

$k_v = 5 + 5 / (a / h)^2$
$= 5$ when $a / h > 3$ or $a / h > [260 / (h / t)]^2$
$a$ = distance between transverse stiffeners, in.
$h$ = for rolled shapes, the clear distance between flanges less the fillet or corner radius, in.
= for built-up welded sections, the clear distance between flanges, in.
= for built-up bolted or riveted sections, the distance between fastener lines, in.

### 3. Transverse Stiffeners

Transverse stiffeners are not required in plate girders where $h / t_w \leq 418 / \sqrt{F_{yw}}$, or where the required shear, $V_u$, as determined by structural analysis for the factored loads, is less than or equal to $0.6\phi_v A_w F_{yw} C_v$, where $C_v$ is determined for $k_v = 5$ and $\phi_v = 0.90$.

Transverse stiffeners used to develop the web design shear strength as provided in Appendix F2.2 shall have a moment of inertia about an axis in the web center for stiffener pairs or about the face in contact with the web plate for single stiffeners, which shall not be less than $at_w^3 j$, where

$$j = 2.5 / (a / h)^2 - 2 \geq 0.5 \qquad \text{(A-F2-4)}$$

# TABLE A-F1.1
# Nominal Strength Parameters

| Shape | Plastic Moment $M_p$ | Limit State of Buckling | Limiting Buckling Moment $M_r$ |
|---|---|---|---|
| Channels and doubly and singly symmetric I-shaped beams (including hybrid beams) bent about major axis [a] | $F_y Z_x$ [b] | LTB doubly symmetric members and channels | $F_L S_x$ |
| | | LTB singly symmetric members | $F_L S_{xc} \le F_{yt} S_{xt}$ |
| | | FLB | $F_L S_x$ |
| | | WLB | $R_e F_{yf} S_x$ |
| Channels and doubly and singly symmetric I-shaped members bent about minor axis [a] | $F_y Z_y$ | FLB | $F_y S_y$ |

NOTE: LTB applies only for strong axis bending.
[a] Excluding double angles and tees.
[b] Computed from fully plastic stress distribution for hybrid sections.

[c] $X_1 = \frac{\pi}{S_x}\sqrt{\frac{EGJA}{2}}$ $X_2 = 4\frac{C_w}{I_y}\left(\frac{S_x}{GJ}\right)^2$

[d] $\lambda_r = \frac{X_1}{F_L}\sqrt{1 + \sqrt{1 + X_2 F_L^2}}$

[e] $F_{cr} = \frac{M_{cr}}{S_{xc}}$, where $M_{cr} = \frac{57{,}000 C_b}{L_b}\sqrt{I_y J}\,[B_1 + \sqrt{(1 + B_2 + B_1^2)}] \le M_p$

where
$B_1 = 2.25[2(I_{yc}/I_y) - 1](h/L_b)\sqrt{(I_y/J)}$
$B_2 = 25(1 - I_{yc}/I_y)(I_{yc}/J)(h/L_b)^2$
$C_b = 1.0$ if $I_{yc}/I_y < 0.1$ or $I_{yc}/I_y > 0.9$.

# TABLE A-F1.1 (cont'd)
# Nominal Strength Parameters

| Critical Stress $F_{cr}$ | Slenderness Parameters | | | Limitations |
|---|---|---|---|---|
| | $\lambda$ | $\lambda_p$ | $\lambda_r$ | |
| $\frac{C_b X_1 \sqrt{2}}{\lambda}\sqrt{1+\frac{X_1^2 X_2}{2\lambda^2}}$ | $\frac{L_b}{r_y}$ | $\frac{300}{\sqrt{F_{yf}}}$ | [c, d] | Applicable for I-shaped members if $h / t_w \leq \lambda_r$ when $h / t_w > \lambda_r$ See Appendix G. |
| [e] | $\frac{L_b}{r_{yc}}$ | $\frac{300}{\sqrt{F_{yf}}}$ | Value of $\lambda$ for which $M_{cr}(C_b = 1) = M_r$ | |
| [f] | $\frac{b}{t}$ | $\frac{65}{\sqrt{F_{yf}}}$ | [g] | |
| Not applicable | $\frac{h}{t_w}$ | $\frac{640}{\sqrt{F_{yf}}}$ | $\lambda_r$ as defined in Section B5.1 | |
| Same as for major axis | | | | |

[f] $F_{cr} = \frac{20{,}000}{\lambda^2}$ for rolled shapes

$F_{cr} = \frac{26{,}200 k_c}{\lambda^2}$ for welded shapes

where
$k_c = 4 / \sqrt{h / t_w}$ and $0.35 \leq k_c \leq 0.763$

[g] $\lambda_r = \frac{141}{\sqrt{F_L}}$ for rolled shapes

$\lambda_r = \frac{162}{\sqrt{F_L / k_c}}$ for welded shapes

# TABLE A-F1.1 (cont'd)
# Nominal Strength Parameters

| **Shape** | **Plastic Moment $M_p$** | **Limit State of Buckling** | **Limiting Buckling Moment $M_r$** |
|---|---|---|---|
| Solid symmetric shapes, except rectangular bars, bent about major axis | $F_y Z_x$ | Not applicable | |
| Solid rectangular bars bent about major axis | $F_y Z_x$ | LTB | $F_y S_x$ |
| Symmetric box sections loaded in a plane of symmetry | $F_y Z$ | LTB | $F_{yf} S_{eff}$ |
| | | FLB | $F_L S_{eff}$ |
| | | WLB | Same as for I-shape |
| Circular tubes | $F_y Z$ | LTB | Not applicable |
| | | FLB | $M_n = \left(\dfrac{600}{D/t} + F_y\right) S$ [h] |
| | | WLB | Not applicable |

[h] This equation is to be used in place of Equation A-F1-4.

# TABLE A-F1.1 (cont'd)
# Nominal Strength Parameters

| Critical Stress $F_{cr}$ | Slenderness Parameters $\lambda$ | $\lambda_p$ | $\lambda_r$ | Limitations |
|---|---|---|---|---|
| Not applicable | | | | |
| $\dfrac{57{,}000 C_b \sqrt{JA}}{\lambda S_x}$ | $\dfrac{L_b}{r_y}$ | $\dfrac{3{,}750\sqrt{JA}}{M_p}$ | $\dfrac{57{,}000\sqrt{JA}}{M_r}$ | |
| $\dfrac{57{,}000 C_b \sqrt{JA}}{\lambda S_x}$ | $\dfrac{L_b}{r_y}$ | $\dfrac{3{,}750\sqrt{JA}}{M_p}$ | $\dfrac{57{,}000\sqrt{JA}}{M_r}$ | Applicable if $h / t_w \le 970 / \sqrt{F_{yf}}$ |
| $\dfrac{S_{eff}}{S_x} F_y$ [i] | $\dfrac{b}{t}$ | $\dfrac{190}{\sqrt{F_y}}$ | $\dfrac{238}{\sqrt{F_y}}$ | |
| Same as for I-shape | | | | |
| Not applicable | | | | |
| $\dfrac{9{,}570}{D/t}$ | $D/t$ | $\dfrac{2{,}070}{F_y}$ | $\dfrac{8{,}970}{F_y}$ | $D/t < \dfrac{13{,}000}{F_y}$ |
| Not applicable | | | | |

[i] $S_{eff}$ is the effective section modulus for the section with a compression flange $b_e$ defined in Appendix B5.3b

Intermediate stiffeners are permitted to be stopped short of the tension flange, provided bearing is not needed to transmit a concentrated load or reaction. The weld by which intermediate stiffeners are attached to the web shall be terminated not less than four times nor more than six times the web thickness from the near toe of the web-to-flange weld. When single stiffeners are used, they shall be attached to the compression flange, if it consists of a rectangular plate, to resist any uplift tendency due to torsion in the flange. When lateral bracing is attached to a stiffener, or a pair of stiffeners, these, in turn, shall be connected to the compression flange to transmit one percent of the total flange stress, unless the flange is composed only of angles.

Bolts connecting stiffeners to the girder web shall be spaced not more than 12 in. on center. If intermittent fillet welds are used, the clear distance between welds shall not be more than 16 times the web thickness nor more than 10 in.

## F3. WEB-TAPERED MEMBERS

The design of tapered members meeting the requirements of this section shall be governed by the provisions of Chapters D through H, except as modified by this Appendix.

### 1. General Requirements

In order to qualify under this Specification, a tapered member shall meet the following requirements:

(1) It shall possess at least one axis of symmetry which shall be perpendicular to the plane of bending if moments are present.

(2) The flanges shall be of equal and constant area.

(3) The depth shall vary linearly as

$$d = d_o\left(1 + \gamma \frac{z}{L}\right) \qquad \text{(A-F3-1)}$$

where

$d_o$ = depth at smaller end of member, in.
$d_L$ = depth at larger end of member, in.
$\gamma$ = $(d_L - d_o)/d_o \leq$ the smaller of $0.268(L/d_o)$ or 6.0
$z$ = distance from the smaller end of member, in.
$L$ = unbraced length of member measured between the center of gravity of the bracing members, in.

### 2. Design Tensile Strength

The design strength of tapered tension members shall be determined in accordance with Section D1.

### 3. Design Compressive Strength

The design strength of tapered compression members shall be determined in accordance with Section E2, using an effective slenderness parameter $\lambda_{eff}$ computed as follows:

$$\lambda_{eff} = \frac{S}{\pi}\sqrt{\frac{QF_y}{E}} \qquad \text{(A-F3-2)}$$

where

$S$ = $KL / r_{oy}$ for weak axis buckling and $K_\gamma L / r_{ox}$ for strong axis buckling
$K$ = effective length factor for a prismatic member
$K_\gamma$ = effective length factor for a tapered member as determined by a rational analysis
$r_{ox}$ = strong axis radius of gyration at the smaller end of a tapered member, in.
$r_{oy}$ = weak axis radius of gyration at the smaller end of a tapered member, in.
$F_y$ = specified minimum yield stress, ksi
$Q$ = reduction factor
= 1.0 if all elements meet the limiting width-thickness ratios $\lambda_r$ of Section B5.1
= $Q_sQ_a$, determined in accordance with Appendix B5.3, if any stiffened and/or unstiffened elements exceed the ratios $\lambda_r$ of Section B5.1
$E$ = modulus of elasticity for steel, ksi

The smallest area of the tapered member shall be used for $A_g$ in Equation E2-1.

## 4. Design Flexural Strength

The design flexural strength of tapered flexural members for the limit state of lateral-torsional buckling is $\phi_b M_n$, where $\phi_b$ = 0.90 and the nominal strength is

$$M_n = (5/3)S'_x F_{b\gamma} \qquad \text{(A-F3-3)}$$

where

$S_x'$ = the section modulus of the critical section of the unbraced beam length under consideration

$$F_{b\gamma} = \frac{2}{3}\left[1.0 - \frac{F_y}{6B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2}}\right]F_y \leq 0.60F_y \qquad \text{(A-F3-4)}$$

unless $F_{b\gamma} \leq F_y/3$, in which case

$$F_{b\gamma} = B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2} \qquad \text{(A-F3-5)}$$

In the preceding equations,

$$F_{s\gamma} = \frac{12 \times 10^3}{h_s L d_o / A_f} \qquad \text{(A-F3-6)}$$

$$F_{w\gamma} = \frac{170 \times 10^3}{(h_w L / r_{To})^2} \qquad \text{(A-F3-7)}$$

where

$h_s$ = factor equal to $1.0 + 0.0230\gamma\sqrt{Ld_o/A_f}$
$h_w$ = factor equal to $1.0 + 0.00385\gamma\sqrt{L/r_{To}}$
$r_{To}$ = radius of gyration of a section at the smaller end, considering only the

compression flange plus one-third of the compression web area, taken about an axis in the plane of the web, in.

$A_f$ = area of the compression flange, in.$^2$

and where $B$ is determined as follows:

(a) When the maximum moment $M_2$ in three adjacent segments of approximately equal unbraced length is located within the central segment and $M_1$ is the larger moment at one end of the three-segment portion of a member:

$$B = 1.0 + 0.37\left(1.0 + \frac{M_1}{M_2}\right) + 0.50\gamma\left(1.0 + \frac{M_1}{M_2}\right) \geq 1.0 \qquad \text{(A-F3-8)}$$

(b) When the largest computed bending stress $f_{b2}$ occurs at the larger end of two adjacent segments of approximately equal unbraced lengths and $f_{b1}$ is the computed bending stress at the smaller end of the two-segment portion of a member:

$$B = 1.0 + 0.58\left(1.0 + \frac{f_{b1}}{f_{b2}}\right) - 0.70\gamma\left(1.0 + \frac{f_{b1}}{f_{b2}}\right) \geq 1.0 \qquad \text{(A-F3-9)}$$

(c) When the largest computed bending stress $f_{b2}$ occurs at the smaller end of two adjacent segments of approximately equal unbraced length and $f_{b1}$ is the computed bending stress at the larger end of the two-segment portion of a member:

$$B = 1.0 + 0.55\left(1.0 + \frac{f_{b1}}{f_{b2}}\right) + 2.20\gamma\left(1.0 + \frac{f_{b1}}{f_{b2}}\right) \geq 1.0 \qquad \text{(A-F3-10)}$$

In the foregoing, $\gamma = (d_L - d_o) / d_o$ is calculated for the unbraced length that contains the maximum computed bending stress. $M_1 / M_2$ is considered as negative when producing single curvature. In the rare case where $M_1 / M_2$ is positive, it is recommended that it be taken as zero. $f_{b1} / f_{b2}$ is considered as negative when producing single curvature. If a point of contraflexure occurs in one of two adjacent unbraced segments, $f_{b1} / f_{b2}$ is considered as positive. The ratio $f_{b1} / f_{b2} \neq 0$.

(d) When the computed bending stress at the smaller end of a tapered member or segment thereof is equal to zero:

$$B = \frac{1.75}{1.0 + 0.25\sqrt{\gamma}} \qquad \text{(A-F3-11)}$$

where $\gamma = (d_L - d_o) / d_o$ is calculated for the unbraced length adjacent to the point of zero bending stress.

## 5. Design Shear Strength

The design shear strength of tapered flexural members shall be determined in accordance with Section F2.

## 6. Combined Flexure and Axial Force

For tapered members with a single web taper subject to compression and bending about the major axis, Equation H1-1 applies, with the following modifications: $P_n$ and $P_{ex}$ shall be determined for the properties of the smaller end, using appropriate effective length factors. $M_{nx}$, $M_u$, and $M_{px}$ shall be determined for the larger end; $M_{nx} = (5\,/\,3)\, S_x' F_{b\gamma}$, where $S_x'$ is the elastic section modulus of the larger end, and $F_{b\gamma}$ is the design flexural stress of tapered members. $C_{mx}$ is replaced by $C'_m$, determined as follows:

(a) When the member is subjected to end moments which cause single curvature bending and approximately equal computed moments at the ends:

$$C'_m = 1.0 + 0.1\left(\frac{P_u}{\phi_b P_{ex}}\right) + 0.3\left(\frac{P_u}{\phi_b P_{ex}}\right)^2 \qquad \text{(A-F3-12)}$$

(b) When the computed bending moment at the smaller end of the unbraced length is equal to zero:

$$C'_m = 1.0 - 0.9\left(\frac{P_u}{\phi_b P_{ex}}\right) + 0.6\left(\frac{P_u}{\phi_b P_{ex}}\right)^2 \qquad \text{(A-F3-13)}$$

When the effective slenderness parameter $\lambda_{eff} \geq 1.5$ and combined stress is checked incrementally along the length, the actual area and the actual section modulus at the section under investigation is permitted to be used.

# APPENDIX G

# PLATE GIRDERS

This appendix applies to I-shaped plate girders with slender webs.

## G1. LIMITATIONS

Doubly and singly symmetric single-web non-hybrid and hybrid plate girders loaded in the plane of the web shall be proportioned according to the provisions of this Appendix or Section F2, provided that the following limits are satisfied:

(a) For $\frac{a}{h} \leq 1.5$:

$$\frac{h}{t_w} \leq \frac{2{,}000}{\sqrt{F_{yf}}} \qquad \text{(A-G1-1)}$$

(b) For $\frac{a}{h} > 1.5$:

$$\frac{h}{t_w} \leq \frac{14{,}000}{\sqrt{F_{yf}\,(F_{yf} + 16.5)}} \qquad \text{(A-G1-2)}$$

where

$a$ = clear distance between transverse stiffeners, in.
$h$ = clear distance between flanges less the fillet or corner radius for rolled shapes; and for built-up sections, the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used, in.
$t_w$ = web thickness, in.
$F_{yf}$ = specified minimum yield stress of a flange, ksi

In unstiffened girders $h / t_w$ shall not exceed 260.

## G2. DESIGN FLEXURAL STRENGTH

The design flexural strength for plate girders with slender webs shall be $\phi_b M_n$, where $\phi_b = 0.90$ and $M_n$ is the lower value obtained according to the limit states of tension-flange yield and compression-flange buckling. For girders with unequal flanges, see Appendix B5.1 for the determination of $\lambda_r$ for the limit state of web local buckling.

(a) For tension-flange yield:

$$M_n = S_{xt} R_e F_{yt} \qquad \text{(A-G2-1)}$$

(b) For compression-flange buckling:

$$M_n = S_{xc} R_{PG} R_e F_{cr} \tag{A-G2-2}$$

where

$$R_{PG} = 1 - \frac{a_r}{1{,}200 + 300a_r}\left(\frac{h_c}{t_w} - \frac{970}{\sqrt{F_{cr}}}\right) \leq 1.0 \tag{A-G2-3}$$

$R_e$ = hybrid girder factor

$= \dfrac{12 + a_r(3m - m^3)}{12 + 2a_r} \leq 1.0$ (for non-hybrid girders, $R_e = 1.0$)

$a_r$ = ratio of web area to compression flange area ($\leq 10$)
$m$ = ratio of web yield stress to flange yield stress or to $F_{cr}$
$F_{cr}$ = critical compression flange stress, ksi
$F_{yt}$ = yield stress of tension flange, ksi
$S_{xc}$ = section modulus referred to compression flange, in.$^3$
$S_{xt}$ = section modulus referred to tension flange, in.$^3$
$h_c$ = twice the distance from the centroid to the nearest line of fasteners at the compression flange or the inside of the face of the compression flange when welds are used

The critical stress $F_{cr}$ to be used is dependent upon the slenderness parameters $\lambda$, $\lambda_p$, $\lambda_r$, and $C_{PG}$ as follows:

For $\lambda \leq \lambda_p$:

$$F_{cr} = F_{yf} \tag{A-G2-4}$$

For $\lambda_p < \lambda \leq \lambda_r$:

$$F_{cr} = C_b F_{yf}\left[1 - \frac{1}{2}\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right)\right] \leq F_{yf} \tag{A-G2-5}$$

For $\lambda > \lambda_r$:

$$F_{cr} = \frac{C_{PG}}{\lambda^2} \tag{A-G2-6}$$

In the foregoing, the slenderness parameter shall be determined for both the limit state of lateral-torsional buckling and the limit state of flange local buckling; the slenderness parameter which results in the lowest value of $F_{cr}$ governs.

(a) For the limit state of lateral-torsional buckling:

$$\lambda = \frac{L_b}{r_T} \tag{A-G2-7}$$

$$\lambda_p = \frac{300}{\sqrt{F_{yf}}} \tag{A-G2-8}$$

$$\lambda_r = \frac{756}{\sqrt{F_{yf}}} \tag{A-G2-9}$$

$$C_{PG} = 286{,}000 C_b \tag{A-G2-10}$$

where

$C_b$= see Section F1.2, Equation F1-3
$r_T$ = radius of gyration of compression flange plus one-third of the compression portion of the web, in.

(b) For the limit state of flange local buckling:

$$\lambda = \frac{b_f}{2t_f} \tag{A-G2-11}$$

$$\lambda_p = \frac{65}{\sqrt{F_{yf}}} \tag{A-G2-12}$$

$$\lambda_r = \frac{230}{\sqrt{F_{yf} / k_c}} \tag{A-G2-13}$$

$$C_{PG} = 26{,}200 k_c \tag{A-G2-14}$$

$$C_b = 1.0$$

where $k_c = 4 / \sqrt{h / t_w}$ and $0.35 \le k_c \le 0.763$.

The limit state of flexural web local buckling is not applicable.

## G3. DESIGN SHEAR STRENGTH WITH TENSION FIELD ACTION

The design shear strength with tension field action shall be $\phi_v V_n$, kips, where $\phi_v = 0.90$ and $V_n$ is determined as follows:

(a) For $h / t_w \le 187\sqrt{k_v / F_{yw}}$:

$$V_n = 0.6 A_w F_{yw} \tag{A-G3-1}$$

(b) For $h / t_w > 187\sqrt{k_v / F_{yw}}$:

$$V_n = 0.6 A_w F_{yw} \left( C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a / h)^2}} \right) \tag{A-G3-2}$$

where

$C_v$ = ratio of "critical" web stress, according to linear buckling theory, to the shear yield stress of web material

Also see Appendix G4 and G5.

For end-panels in non-hybrid plate girders, all panels in hybrid and web-tapered plate girders, and when $a / h$ exceeds 3.0 or $[260 / (h / t_w)]^2$, tension field action is not permitted and

$$V_n = 0.6 A_w F_{yw} C_v \tag{A-G3-3}$$

The web plate buckling coefficient $k_v$ is given as

$$k_v = 5 + \frac{5}{(a / h)^2} \tag{A-G3-4}$$

except that $k_v$ shall be taken as 5.0 if $a / h$ exceeds 3.0 or $[260 / (h / t_w)]^2$.

The shear coefficient $C_v$ is determined as follows:

(a) For $187\sqrt{\dfrac{k_v}{F_{yw}}} \le \dfrac{h}{t_w} \le 234\sqrt{\dfrac{k_v}{F_{yw}}}$:

$$C_v = \frac{187\sqrt{k_v / F_{yw}}}{h / t_w} \tag{A-G3-5}$$

(b) For $\dfrac{h}{t_w} > 234\sqrt{\dfrac{k_v}{F_{yw}}}$:

$$C_v = \frac{44{,}000 k_v}{(h / t_w)^2 F_{yw}} \tag{A-G3-6}$$

## G4. TRANSVERSE STIFFENERS

Transverse stiffeners are not required in plate girders where $h / t_w \le 418 / \sqrt{F_{yw}}$, or where the required shear $V_u$, as determined by structural analysis for the factored loads, is less than or equal to $0.6\phi_v A_w F_{yw} C_v$, where $C_v$ is determined for $k_v = 5$ and $\phi_v = 0.90$. Stiffeners may be required in certain portions of a plate girder to develop the required shear or to satisfy the limitations given in Appendix G1. Transverse stiffeners shall satisfy the requirements of Appendix F2.3.

When designing for tension field action, the stiffener area $A_{st}$ shall not be less than

$$\frac{F_{yw}}{F_{yst}}\left[0.15 D h t_w (1 - C_v)\frac{V_u}{\phi_v V_n} - 18 t_w^2\right] \ge 0 \tag{A-G4-1}$$

where

$F_{yst}$ = specified yield stress of the stiffener material, ksi
$D$ = 1 for stiffeners in pairs
= 1.8 for single angle stiffeners
= 2.4 for single plate stiffeners

$C_v$ and $V_n$ are defined in Appendix G3, and $V_u$ is the required shear at the location of the stiffener.

## G5. FLEXURE-SHEAR INTERACTION

For $0.6\phi V_n \le V_u \le \phi V_n$ ($\phi = 0.90$) and $0.75\phi M_n \le M_u \le \phi M_n$ ($\phi = 0.90$), plate girders with webs designed for tension field action shall satisfy the additional flexure-shear interaction criteria:

$$\frac{M_u}{\phi M_n} + 0.625 \frac{V_u}{\phi V_n} \leq 1.375 \tag{A-G5-1}$$

where $M_n$ is the nominal flexural strength of plate girders from Appendix G2 or Section F1, $\phi = 0.90$, and $V_n$ is the nominal shear strength from Appendix G3.

# APPENDIX H

# MEMBERS UNDER COMBINED FORCES AND TORSION

This appendix provides alternative interaction equations for biaxially loaded I-shaped members with $b_f / d \leq 1.0$ and box-shaped members.

## H3. ALTERNATIVE INTERACTION EQUATIONS FOR MEMBERS UNDER COMBINED STRESS

For biaxially loaded I-shaped members with $b_f / d \leq 1.0$ and box-shaped members in braced frames only, the use of the following interaction equations in lieu of Equations H1-1a and H1-1b is permitted. Both Equations A-H3-1 and A-H3-2 shall be satisfied.

$$\left(\frac{M_{ux}}{\phi_b M'_{px}}\right)^{\zeta} + \left(\frac{M_{uy}}{\phi_b M'_{py}}\right)^{\zeta} \leq 1.0 \qquad \text{(A-H3-1)}$$

$$\left(\frac{C_{mx} M_{ux}}{\phi_b M'_{nx}}\right)^{\eta} + \left(\frac{C_{my} M_{uy}}{\phi_b M'_{ny}}\right)^{\eta} \leq 1.0 \qquad \text{(A-H3-2)}$$

The terms in Equations A-H3-1 and A-H3-2 are determined as follows:

(a) For I-shaped members:

For $b_f / d < 0.5$:

$$\zeta = 1.0$$

For $0.5 \leq b_f / d \leq 1.0$:

$$\zeta = 1.6 - \frac{P_u / P_y}{2[\ln(P_u / P_y)]} \qquad \text{(A-H3-3)}$$

For $b_f / d < 0.3$:

$$\eta = 1.0$$

For $0.3 \leq b_f / d \leq 1.0$:

$$\eta = 0.4 + \frac{P_u}{P_y} + \frac{b_f}{d} \geq 1.0 \qquad \text{(A-H3-4)}$$

where

$b_f$ = flange width, in.
$d$ = member depth, in.
$C_m$ = coefficient applied to the bending term in interaction equation for

prismatic members and dependent on column curvature caused by applied moments, see Section C1.

$$M'_{px} = 1.2M_{px}[1 - (P_u / P_y)] \leq M_{px} \quad \text{(A-H3-5)}$$

$$M'_{py} = 1.2M_{py}[1 - (P_u / P_y)^2] \leq M_{py} \quad \text{(A-H3-6)}$$

$$M'_{nx} = M_{nx}\left(1 - \frac{P_u}{\phi_c P_n}\right)\left(1 - \frac{P_u}{P_{ex}}\right) \quad \text{(A-H3-7)}$$

$$M'_{ny} = M_{ny}\left(1 - \frac{P_u}{\phi_c P_n}\right)\left(1 - \frac{P_u}{P_{ey}}\right) \quad \text{(A-H3-8)}$$

(b) For box-section members:

$$\zeta = 1.7 - \frac{P_u / P_y}{\ln(P_u / P_y)} \quad \text{(A-H3-9)}$$

$$\eta = 1.7 - \frac{P_u / P_y}{\ln(P_u / P_y)} - a\lambda_x\left(\frac{P_u}{P_y}\right)^b > 1.1 \quad \text{(A-H3-10)}$$

For $P_u / P_y \leq 0.4$, $a = 0.06$, and $b = 1.0$;

For $P_u / P_y > 0.4$, $a = 0.15$, and $b = 2.0$;

$$M'_{px} = 1.2M_{px}[1 - P_u / P_y] \leq M_{px} \quad \text{(A-H3-11a)}$$

$$M'_{py} = 1.2M_{py}[1 - P_u / P_y] \leq M_{py} \quad \text{(A-H3-11b)}$$

$$M'_{nx} = M_{nx}\left(1 - \frac{P_u}{\phi_c P_n}\right)\left(1 - \frac{P_u}{P_{ex}}\frac{1.25}{(B/H)^{1/3}}\right) \quad \text{(A-H3-12)}$$

$$M'_{ny} = M_{ny}\left(1 - \frac{P_u}{\phi_c P_n}\right)\left(1 - \frac{P_u}{P_{ey}}\frac{1.25}{(B/H)^{1/2}}\right) \quad \text{(A-H3-13)}$$

where

$P_n$ = nominal compressive strength determined in accordance with Section E2, kips
$P_u$ = required axial strength, kips
$P_y$ = compressive yield strength $A_g F_y$, kips
$\phi_b$ = resistance factor for flexure = 0.90
$\phi_c$ = resistance factor for compression = 0.85
$P_e$ = Euler buckling strength $A_g F_y / \lambda_c^2$, where $\lambda_c$ is the column slenderness parameter defined by Equation E2-4, kips
$M_u$ = required flexural strength, kip-in.
$M_n$ = nominal flexural strength, determined in accordance with Section F1, kip-in.
$M_p$ = plastic moment $\leq 1.5F_y S$, kip-in.
$B$ = outside width of box section parallel to major principal axis x, in.
$H$ = outside depth of box section perpendicular to major principal axis x, in.

# APPENDIX J

# CONNECTIONS, JOINTS, AND FASTENERS

Appendix J2.4 provides the alternative design strength for fillet welds. Appendices J3.8 and J3.9 pertain to the design of slip-critical connections using factored loads.

## J2. WELDS

### 4. Design Strength

In lieu of the constant design strength for fillet welds given in Table J2.5, the following procedure is permitted.

(a) The design strength of a linear weld group loaded in-plane through the center of gravity is $\phi F_w A_w$:

$$F_w = 0.60 F_{EXX}(1.0 + 0.50 \sin^{1.5}\theta)$$

where

$\phi$ = 0.75
$F_w$ = nominal stress, ksi
$F_{EXX}$ = electrode classification number, i.e., minimum specified strength, ksi
$\theta$ = angle of loading measured from the weld longitudinal axis, degrees
$A_w$ = effective area of weld throat, in.$^2$

(b) The design strength of weld elements within a weld group that are loaded in-plane and analyzed using an instantaneous center of rotation method to maintain deformation compatibility and non-linear load deformation behavior of variable angle loaded welds is $\phi F_{wx} A_w$ and $\phi F_{wy} A_w$:

where

$F_{wx}$ = $\Sigma F_{wix}$
$F_{wy}$ = $\Sigma F_{wiy}$
$F_{wi}$ = $0.60 F_{EXX}(1.0 + 0.50 \sin^{1.5}\theta) f(p)$
$f(p)$ = $[p(1.9 - 0.9p)]^{0.3}$
$\phi$ = 0.75
$F_{wi}$ = nominal stress in any $i$th weld element, ksi
$F_{wix}$ = x component of stress $F_{wi}$
$F_{wiy}$ = y component of stress $F_{wi}$
$p$ = $\Delta_i / \Delta_m$, ratio of element i deformation to its deformation at maximum stress
$\Delta_m$ = $0.209(\theta + 2)^{-0.32} D$, deformation of weld element at maximum stress, in.

$\Delta_i$ = deformation of weld elements at intermediate stress levels, linearly proportioned to the critical deformation based on distance from the instantaneous center of rotation, $r_i$, in.
= $r_i \Delta_u / r_{crit}$

$\Delta_u$ = $1.087(\theta + 6)^{-0.65} D \leq 0.17D$, deformation of weld element at ultimate stress (fracture), usually in element furthest from instantaneous center of rotation, in.

$D$ = leg size of the fillet weld, in.

$r_{crit}$ = distance from instantaneous center of rotation to weld element with minimum $\Delta_u / r_i$ ratio

## J3. BOLTS AND THREADED PARTS

### 8. High-Strength Bolts in Slip-Critical Connections

### 8b. Slip-Critical Connections Designed at Factored Loads

It is permissible to proportion slip-critical connections at factored loads. The design slip resistance for use at factored loads, $\phi R_{str}$, shall equal or exceed the required force due to the factored loads, where:

$$R_{str} = 1.13 \mu T_m N_b N_s \tag{A-J3-1}$$

where:

$T_m$ = minimum fastener tension given in Table J3.1, kips
$N_b$ = number of bolts in the joint
$N_s$ = number of slip planes
$\mu$ = mean slip coefficient for Class A, B, or C surfaces, as applicable, or as established by tests

(a) For Class A surfaces (unpainted clean mill scale steel surfaces or surfaces with Class A coating on blast-cleaned steel), $\mu = 0.33$

(b) For Class B surfaces (unpainted blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel), $\mu = 0.50$

(c) For Class C surfaces (hot-dip galvanized and roughened surfaces), $\mu = 0.40$

$\phi$ = resistance factor

(a) For standard holes, $\phi = 1.0$

(b) For oversize and short-slotted holes, $\phi = 0.85$

(c) For long-slotted holes transverse to the direction of load, $\phi = 0.70$

(d) For long-slotted holes parallel to the direction of load, $\phi = 0.60$

### 9. Combined Tension and Shear in Slip-Critical Connections

### 9b. Slip-Critical Connections Designed at Factored Loads

When using factored loads as the basis for design of slip-critical connections subject to applied tension, $T$, that reduces the net clamping force, the slip resistance $\phi R_{str}$ according to Appendix J3.8b shall be multiplied by the following factor in which $T_u$ is the required tensile strength at factored loads:

$$[1 - T_u / (1.13 T_m N_b)] \quad \text{(A-J3-2)}$$

# APPENDIX K

# CONCENTRATED FORCES, PONDING, AND FATIGUE

Appendix K2 provides an alternative determination of roof stiffness. Appendix K4 pertains to members and connections due to fatigue loading.

## K2. PONDING

The provisions of this Appendix are permitted to be used when a more exact determination of flat roof framing stiffness is needed than that given by the provision of Section K2 that $C_p + 0.9C_s \leq 0.25$.

For any combination of primary and secondary framing, the stress index is computed as

$$U_p = \left(\frac{F_y - f_o}{f_o}\right)_p \text{ for the primary member} \quad \text{(A-K2-3)}$$

$$U_s = \left(\frac{F_y - f_o}{f_o}\right)_s \text{ for the secondary member} \quad \text{(A-K2-4)}$$

where

$f_o$ = the stress due to $1.2D + 1.2R$ ($D$ = nominal dead load, $R$ = nominal load due to rain water or ice exclusive of the ponding contribution)*

Enter Figure A-K2.1 at the level of the computed stress index $U_p$ determined for the primary beam; move horizontally to the computed $C_s$ value of the secondary beams and then downward to the abscissa scale. The combined stiffness of the primary and secondary framing is sufficient to prevent ponding if the flexibility constant read from this latter scale is more than the value of $C_p$ computed for the given primary member; if not, a stiffer primary or secondary beam, or combination of both, is required. In the above,

$$C_p = \frac{32L_sL_p^4}{10^7I_p}$$

$$C_s = \frac{32L_s^4}{10^7I_s}$$

* Depending upon geographic location, this loading should include such amount of snow as might also be present, although ponding failures have occurred more frequently during torrential summer rains when the rate of precipitation exceeded the rate of drainage runoff and the resulting hydraulic gradient over large roof areas caused substantial accumulation of water some distance from the eaves. A load factor of 1.2 shall be used for loads resulting from these phenomena.

where

$L_p$ = column spacing in direction of girder (length of primary members), ft
$L_s$ = column spacing perpendicular to direction of girder (length of secondary members), ft
$S$ = spacing of secondary members, ft
$I_p$ = moment of inertia of primary members, in.$^4$
$I_s$ = moment of inertia of secondary members, in.$^4$

A similar procedure must be followed using Figure A-K2.2.

Roof framing consisting of a series of equally spaced wall-bearing beams is considered as consisting of secondary members supported on an infinitely stiff

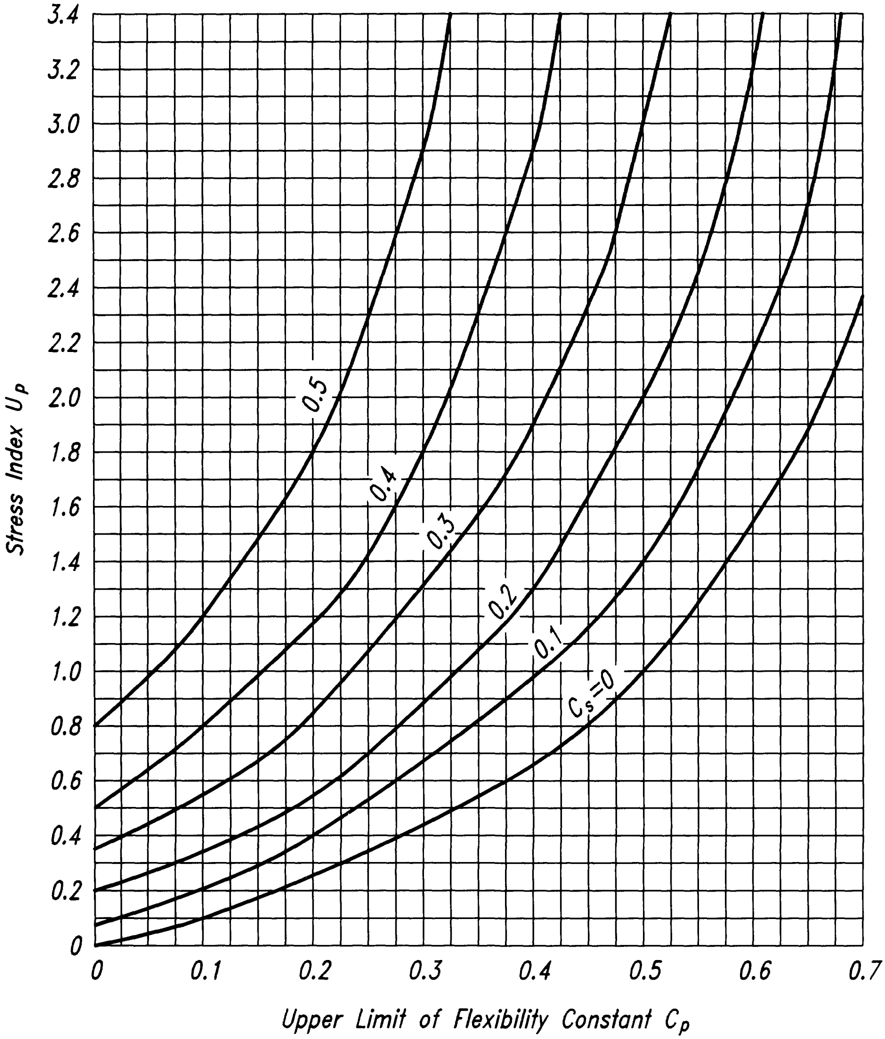



*Fig. A-K2.1. Limiting flexibility coefficient for the primary systems.*

primary member. For this case, enter Figure A-K2.2 with the computed stress index $U_s$. The limiting value of $C_s$ is determined by the intercept of a horizontal line representing the $U_s$ value and the curve for $C_p = 0$.

The ponding deflection contributed by a metal deck is usually such a small part of the total ponding deflection of a roof panel that it is sufficient merely to limit its moment of inertia (per foot of width normal to its span) to 0.000025 times the fourth power of its span length. However, the stability against ponding of a roof consisting of a metal roof deck of relatively slender depth-span ratio, spanning between beams supported directly on columns, may need to be checked. This can be done using Figure A-K2.1 or A-K2.2 using as $C_s$ the flexibility constant for a one-foot width of the roof deck ($S$ = 1.0).

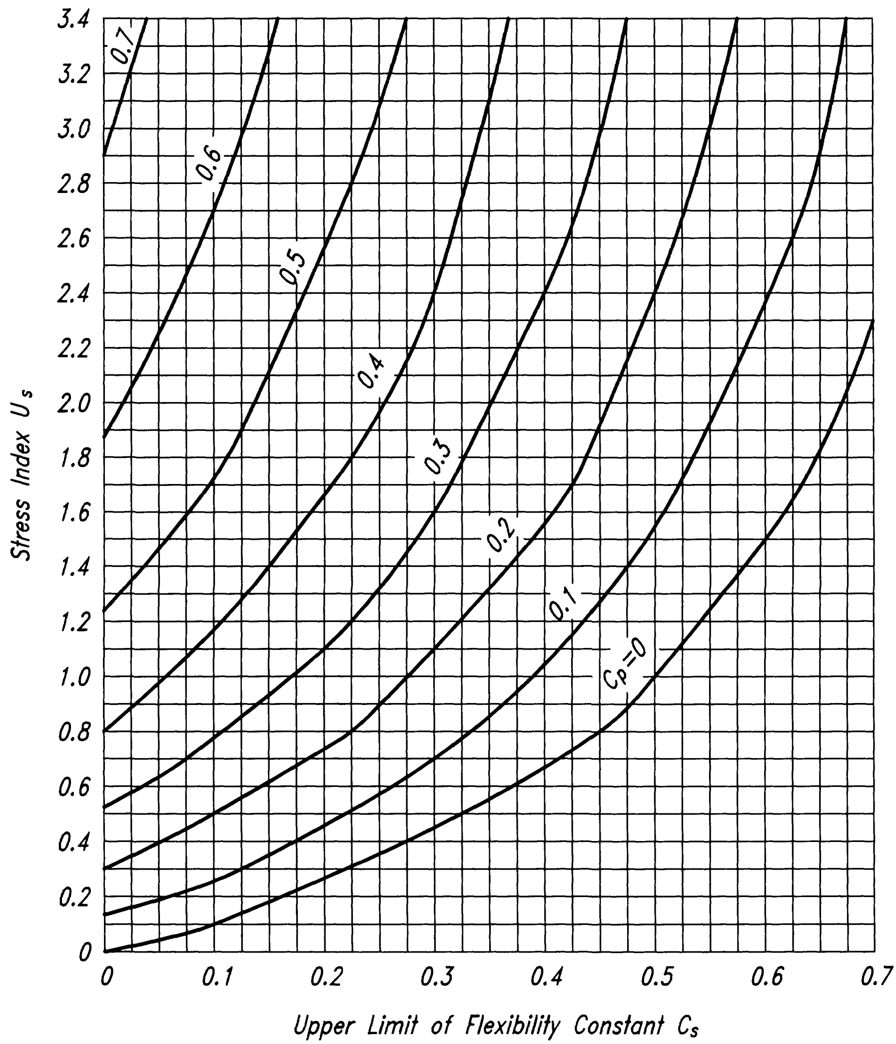



*Fig. A-K2.2. Limiting flexibility coefficient for the secondary systems.*

# TABLE A-K3.1
# Number of Loading Cycles

| **Loading Condition** | **From** | **To** |
|---|---|---|
| Loading Condition | From | To |
| 1 | 20,000 [a] | 100,000 [b] |
| 2 | 100,000 | 500,000 [c] |
| 3 | 500,000 | 2,000,000 [d] |
| 4 | Over 2,000,000 | |

[a] Approximately equivalent to two applications every day for 25 years.
[b] Approximately equivalent to 10 applications every day for 25 years.
[c] Approximately equivalent to 50 applications every day for 25 years.
[d] Approximately equivalent to 200 applications every day for 25 years.

Since the shear rigidity of the web system of steel joists and trusses is less than that of a solid plate, their moment of inertia shall be taken as 85 percent of their chords.

## K3. FATIGUE

Members and connections subject to fatigue loading shall be proportioned in accordance with the provisions of this Appendix.

Fatigue, as used in this Specification, is defined as the damage that may result in fracture after a sufficient number of fluctuations of stress. Stress range is defined as the magnitude of these fluctuations. In the case of a stress reversal, the stress range shall be computed as the numerical sum of maximum repeated tensile and compressive stresses or the sum of maximum shearing stresses of opposite direction at a given point, resulting from differing arrangement of live load.

### 1. Loading Conditions; Type and Location of Material

In the design of members and connections subject to repeated variation of live load, consideration shall be given to the number of stress cycles, the expected range of stress, and the type and location of member or detail.

Loading conditions shall be classified according to Table A-K3.1.

The type and location of material shall be categorized according to Table A-K3.2.

### 2. Design Stress Range

The maximum range of stress at service loads shall not exceed the design stress range specified in Table A-K3.3.

### 3. Design Strength of Bolts in Tension

When subject to tensile fatigue loading, fully tensioned A325 or A490 bolts shall be designed for the combined tensile design strength due to combined external and prying forces in accordance with Table A-K3.4.

# TABLE A-K3.2
# Type and Location of Material

| General Condition | Situation | Kind of Stress [a] | Stress Category (see Table A-K3.3) | Illustrative Example Nos. (see Fig. A-K3.1) [b] |
|---|---|---|---|---|
| Plain Material | Base metal with rolled or cleaned surface. Flame-cut edges with ANSI smoothness of 1,000 or less | T or Rev. | A | 1,2 |
| Built-up Members | Base metal and weld metal in members without attachments, built-up plates or shapes connected by continuous full-penetration groove welds or by continuous fillet welds parallel to the direction of applied stress | T or Rev. | B | 3,4,5,6 |
| | Base metal and weld metal in members without attachments, built-up plates, or shapes connected by full-penetration groove welds with backing bars not removed, or by partial-penetration groove welds parallel to the direction of applied stress | T or Rev. | B′ | 3,4,5,6 |
| | Base metal at toe of welds on girder webs or flanges adjacent to welded transverse stiffeners | T or Rev. | C | 7 |
| | Base metal at ends of partial length welded coverplates narrower than the flange having square or tapered ends, with or without welds across the ends or wider than flange with welds across the ends | | | |
| | Flange thickness ≤ 0.8 in. | T or Rev. | E | 5 |
| | Flange thickness > 0.8 in. | T or Rev. | E′ | 5 |
| | Base metal at end of partial length welded coverplates wider than the flange without welds across the ends | | E′ | 5 |

[a] "T" signifies range in tensile stress only; "Rev." signifies a range involving reversal of tensile or compressive stress; "S" signifies range in shear, including shear stress reversal.
[b] These examples are provided as guidelines and are not intended to exclude other reasonably similar situations.
[c] Allowable fatigue stress range for transverse partial-penetration and transverse fillet welds is a function of the effective throat, depth of penetration, and plate thickness. See Frank and Fisher, *Journal of the Structural Division*, Vol. 105 No. ST9, Sept. 1979.

# TABLE A-K3.2 (cont'd)
## Type and Location of Material

| **General Condition** | **Situation** | **Kind of Stress [a]** | **Stress Category (see Table A-K3.3)** | **Illustrative Example Nos. (see Fig. A-K3.1) [b]** |
|---|---|---|---|---|
| Groove Welds | Base metal and weld metal at full-penetration groove welded splices of parts of similar cross section ground flush, with grinding in the direction of applied stress and with weld soundness established by radiographic or ultrasonic inspection in accordance with the requirements of 9.25.2 or 9.25.3 of AWS D1.1 | T or Rev. | B | 10,11 |
| | Base metal and weld metal at full-penetration groove welded splices at transitions in width or thickness, with welds ground to provide slopes no steeper than 1 to 2½ with grinding in the direction of applied stress, and with weld soundness established by radiographic or ultrasonic inspection in accordance with the requirements of 9.25.2 or 9.25.3 of AWS D1.1 | | | |
| | A514 base metal | T or Rev. | B′ | 12,13 |
| | Other base metals | T or Rev. | B | 12,13 |
| | Base metal and weld metal at full-penetration groove welded splices, with or without transitions having slopes no greater than 1 to 2½ when reinforcement is not removed but weld soundness is established by radiographic or ultrasonic inspection in accordance with requirements of 9.25.2 or 9.25.3 of AWS D1.1 | T or Rev. | C | 10,11,12,13 |
| Partial-Penetration Groove Welds | Weld metal of partial-penetration transverse groove welds, based on effective throat area of the weld or welds | T or Rev. | F [c] | |
| Fillet-welded Connections | Base metal at intermittent fillet welds | T or Rev. | E | |

# TABLE A-K3.2 (cont'd)
## Type and Location of Material

| General Condition | Situation | Kind of Stress [a] | Stress Category (see Table A-K3.3) | Illustrative Example Nos. (see Fig. A-K3.1) [b] |
|---|---|---|---|---|
| Fillet-welded Connections (Continued) | Base metal at junction of axially loaded members with fillet-welded end connections. Welds shall be disposed about the axis of the member so as to balance weld stresses<br>$b \leq 1$ in.<br>$b > 1$ in. | <br>T or Rev.<br>T or Rev. | <br>E<br>E′ | <br>17,18<br>17,18 |
| | Base metal at members connected with transverse fillet welds<br>$b \leq$ ½-in.<br>$b >$ ½-in. | <br>T or Rev. | <br>C<br>See Note | <br>20,21 |
| Fillet Welds | Weld metal of continuous or intermittent longitudinal or transverse fillet welds | S | F [c] | 15,17,18<br>20,21 |
| Plug or Slot Welds | Base metal at plug or slot welds | T or Rev. | E | 27 |
| | Shear on plug or slot welds | S | F | 27 |
| Mechanically Fastened Connections | Base metal at gross section of high-strength bolted slip-critical connections, except axially loaded joints which induce out-of-plane bending in connected material | T or Rev. | B | 8 |
| | Base metal at net section of other mechanically fastened joints | T or Rev. | D | 8,9 |
| | Base metal at net section of fully tensioned high-strength, bolted-bearing connections | T or Rev. | B | 8,9 |
| Eyebar or Pin Plates | Base metal at net section of eyebar head or pin plate | T or Rev. | E | 28,29 |
| Attachments | Base metal at details attached by full-penetration groove welds subject to longitudinal and/or transverse loading when the detail embodies a transition radius $R$ with the weld termination ground smooth and for transverse loading, the weld soundness established by radiographic or ultrasonic inspection in accordance with 9.25.2 or 9.25.3 of AWS D1.1 | | | |

# TABLE A-K3.2 (cont'd)
# Type and Location of Material

| General Condition | Situation | Kind of Stress [a] | Stress Category (see Table A-K3.3) | Illustrative Example Nos. (see Fig. A-K3.1) [b] |
|---|---|---|---|---|
| Attachments (Continued) | Longitudinal loading | | | |
| | $R >$ 24 in. | T or Rev. | B | 14 |
| | 24 in. $> R >$ 6 in. | T or Rev. | C | 14 |
| | 6 in. $> R >$ 2 in. | T or Rev. | D | 14 |
| | 2 in. $> R$ | T or Rev. | E | 14 |
| | Detail base metal for transverse loading: equal thickness and reinforcement removed | | | |
| | $R >$ 24 in. | T or Rev. | B | 14 |
| | 24 in. $> R >$ 6 in. | T or Rev. | C | 14 |
| | 6 in. $> R >$ 2 in. | T or Rev. | D | 14 |
| | 2 in. $> R$ | T or Rev. | E | 14,15 |
| | Detail base metal for transverse loading: equal thickness and reinforcement not removed | | | |
| | $R >$ 24 in. | T or Rev. | C | 14 |
| | 24 in. $> R >$ 6 in. | T or Rev. | C | 14 |
| | 6 in. $> R >$ 2 in. | T or Rev. | D | 14 |
| | 2 in. $> R$ | T or Rev. | E | 14,15 |
| | Detail base metal for transverse loading: unequal thickness and reinforcement removed | | | |
| | $R >$ 2 in. | T or Rev. | D | 14 |
| | 2 in. $> R$ | T or Rev. | E | 14,15 |
| | Detail base metal for transverse loading: Unequal thickness and reinforcement not removed | | | |
| | All $R$ | T or Rev. | E | 14,15 |
| | Detail base metal for transverse loading | | | |
| | $R >$ 6 in. | T or Rev. | C | 19 |
| | 6 in. $> R >$ 2 in. | T or Rev. | D | 19 |
| | 2 in. $> R$ | T or Rev. | E | 19 |
| | Base metal at detail attached by full-penetration groove welds subject to longitudinal loading | | | |
| | $2 < a < 12b$ or 4 in. | T or Rev. | D | 15 |
| | $a > 12b$ or 4 in. when $b \leq 1$ in. | T or Rev. | E | 15 |
| | $a > 12b$ or 4 in. when $b > 1$ in. | T or Rev. | E′ | 15 |

# TABLE A-K3.2 (cont'd)
# Type and Location of Material

| General Condition | Situation | Kind of Stress [a] | Stress Category (see Table A-K3.3) | Illustrative Example Nos. (see Fig. A-K3.1) [b] |
|---|---|---|---|---|
| Attachments (Continued) | Base metal at detail attached by fillet welds or partial-penetration groove welds subject to longitudinal loading | | | |
| | $a < 2$ in. | T or Rev. | C | 15,23,24 25,26 |
| | 2 in. $< a < 12b$ or 4 in. | T or Rev. | D | 15,23,24,26 |
| | $a > 12b$ or 4 in. when $b \leq 1$ in. | T or Rev. | E | 15,23,24,26 |
| | $a > 12b$ or 4 in. when $b > 1$ in. | T or Rev. | E′ | 15,23,24,26 |
| | Base metal attached by fillet welds or partial-penetration groove welds subjected to longitudinal loading when the weld termination embodies a transition radius with the weld termination ground smooth | | | |
| | $R > 2$ in.<br>$R \leq 2$ in. | T or Rev.<br>T or Rev. | D<br>E | 19<br>19 |
| | Fillet-welded attachments where the weld termination embodies a transition radius, weld termination ground smooth, and main material subject to longitudinal loading<br><br>Detail base metal for transverse loading: | | | |
| | $R > 2$ in.<br>$R < 2$ in. | T or Rev.<br>T or Rev. | D<br>E | 19<br>19 |
| | Base metal at stud-type shear connector attached by fillet weld or automatic end weld | T or Rev. | C | 22 |
| | Shear stress on nominal area of stud-type shear connectors | S | F | |

# TABLE A-K 3.3
## Design Stress Range, ksi

| **Category (From Table A-K3.2)** | **Loading Condition 1** | **Loading Condition 2** | **Loading Condition 3** | **Loading Condition 4** |
|---|---|---|---|---|
| A | 63 | 37 | 24 | 24 |
| B | 49 | 29 | 18 | 16 |
| B′ | 39 | 23 | 15 | 12 |
| C | 35 | 21 | 13 | 10 [a] |
| D | 28 | 16 | 10 | 7 |
| E | 22 | 13 | 8 | 4.5 |
| E′ | 16 | 9.2 | 5.8 | 2.6 |
| F | 15 | 12 | 9 | 8 |
| [a] Flexural stress range of 12 ksi permitted at toe of stiffener welds or flanges. | | | | |

# TABLE A-K3.4
## Design Strength of A325 or A490 Bolts Subject to Tension

| Number of cycles | Design strength |
|---|---|
| Not more than 20,000 | As specified in Section J3 |
| From 20,000 to 500,000 | 0.30 $A_bF_u$ [a] |
| More than 500,000 | 0.25 $A_bF_u$ [a] |
| [a] At service loads. | |

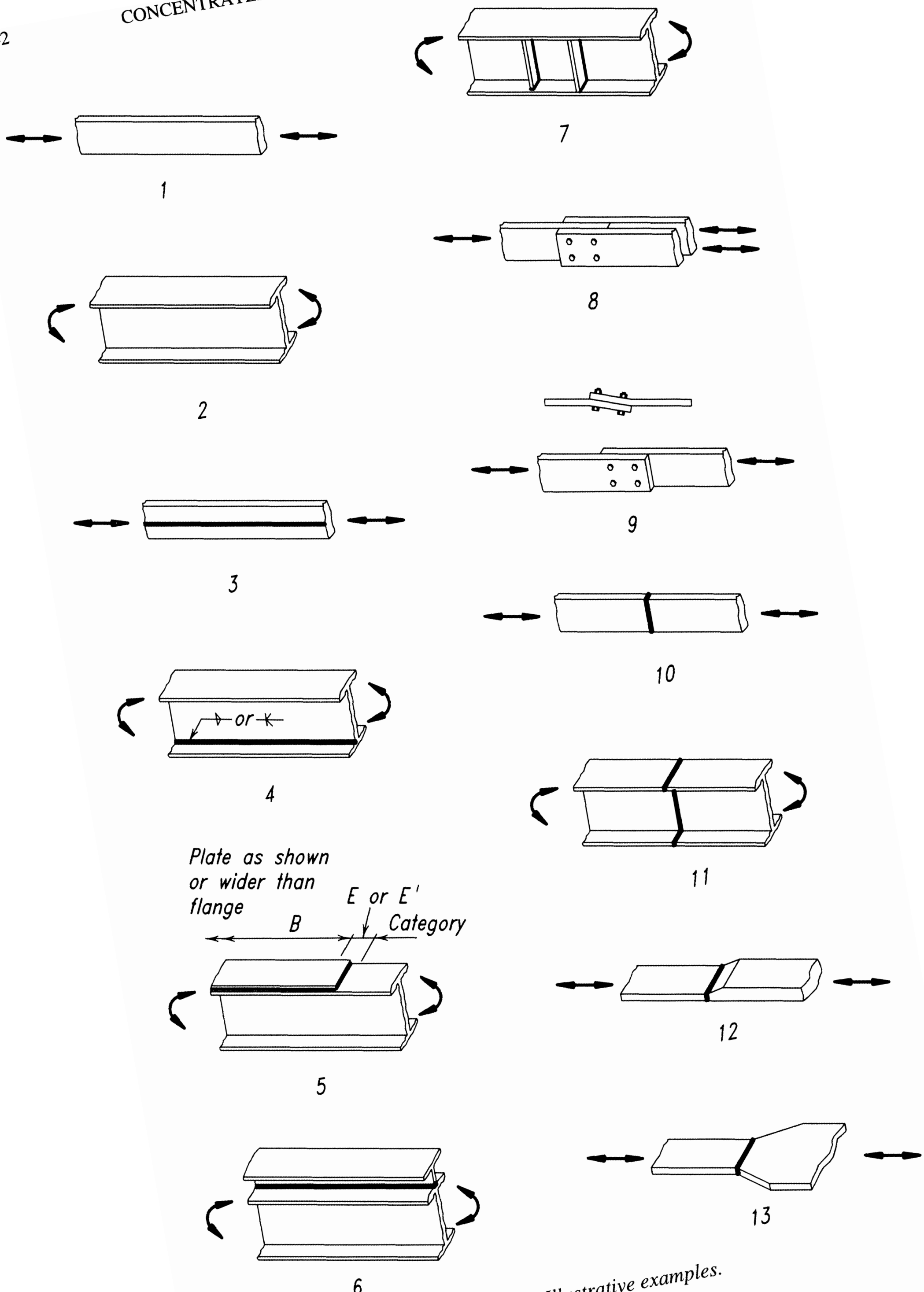



Fig. A-K3.1. *Illustrative examples.*

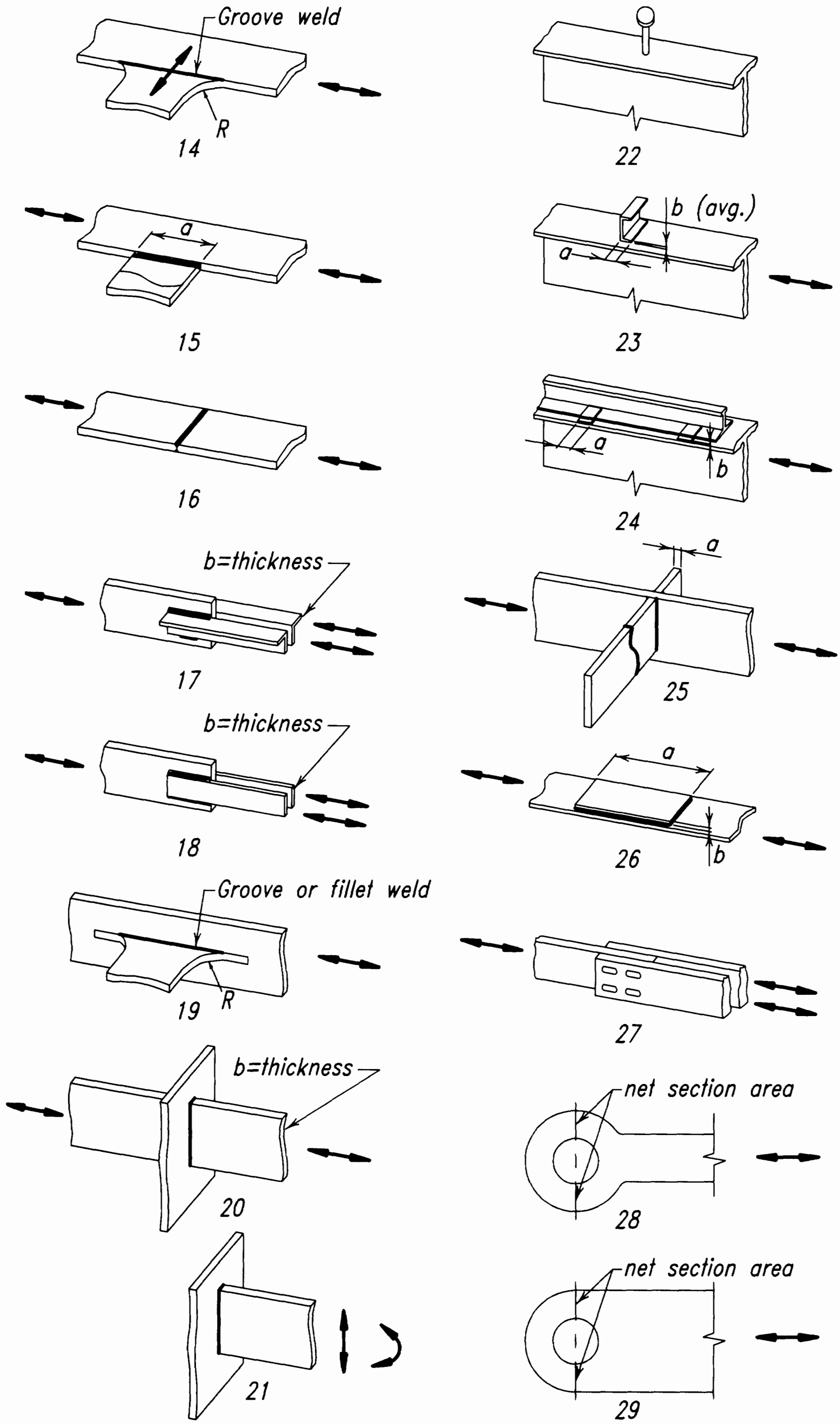



*Fig. A-K3.1. Illustrative examples (cont.).*

# NUMERICAL VALUES

## TABLE 1
## Design Strength as a Function of $F_y$

| $F_y$ (ksi) | Design Stress (ksi) | | |
|---|---|---|---|
| | $0.54F_y$ [a] | $0.85F_y$ [b] | $0.90F_y$ [c] |
| 33 | 17.8 | 28.1 | 29.7 |
| 35 | 18.9 | 29.8 | 31.5 |
| 36 | 19.4 | 30.6 | 32.4 |
| 42 | 21.6 | 34.0 | 36.0 |
| 42 | 22.7 | 35.7 | 37.8 |
| 45 | 24.3 | 38.3 | 40.5 |
| 46 | 24.8 | 39.1 | 41.4 |
| 40 | 27.0 | 42.5 | 45.0 |
| 55 | 29.7 | 46.8 | 49.5 |
| 60 | 32.4 | 51.0 | 54.0 |
| 65 | 35.1 | 55.3 | 58.5 |
| 70 | 37.8 | 59.5 | 63.0 |
| 90 | 48.6 | 76.5 | 81.0 |
| 100 | 54.0 | 85.0 | 90.0 |

[a] See Section F2, Equations F2-1
[b] See Section E2, Equation E2-1
[c] See Section D1, Equation D1-1

# TABLE 2
# Design Strength as a Function of $F_y$

| Item | ASTM Desig-nation | $F_y$ (ksi) | $F_u$ (ksi) | Design Strength (ksi): Connection Part of Designated Steel: Tension 0.75 × $F_u$ [a] | Design Strength (ksi): Connection Part of Designated Steel: Bearing 0.75 × 2.4$F_u$ [b] | Design Strength (ksi): Bolt of Threaded Part of Designated Steel: Tension 0.75 × 0.75$F_u$ [c] | Design Strength (ksi): Bolt of Threaded Part of Designated Steel: Shear 0.75 × 0.40$F_u$ [d] | Design Strength (ksi): Bolt of Threaded Part of Designated Steel: Shear 0.75 × 0.50$F_u$ [e] |
|---|---|---|---|---|---|---|---|---|
| Shapes, Plates, Bars, Sheet and Tubing or Threaded parts | A36 | 36 | 58-80 | 43.5 | 104 | 32.6 | 17.4 | 21.8 |
| | A53 | 35 | 60 | 45.0 | 108 | — | — | — |
| | [A242 / A588] | 50 | 70 | 52.5 | 126 | 39.4 | 21.0 | 26.3 |
| | | 42 | 63 | 47.3 | 113 | 35.4 | 18.9 | 23.2 |
| | | 40 | 60 | 45.0 | 108 | 33.8 | 18.0 | 22.5 |
| | A500 | 33/39 [f] | 45 | 33.8 | 81 | — | — | — |
| | | 42/46 [f] | 58 | 43.5 | 104 | — | — | — |
| | | 46/50 [f] | 62 | 46.5 | 112 | — | — | — |
| | A501 | 36 | 58 | 43.5 | 104 | — | — | — |
| | A529 | 42 | 60-85 | 45.0 | 108 | 33.8 | 18.0 | 22.5 |
| | A570 | 40 | 55 | 41.3 | 99 | — | — | — |
| | | 42 | 58 | 43.5 | 104 | — | — | — |
| | A572 | 42 | 60 | 45.0 | 108 | 33.8 | 18.0 | 22.5 |
| | | 50 | 65 | 48.8 | 117 | 36.6 | 19.5 | 24.4 |
| | | 60 | 75 | 56.3 | 135 | 42.2 | 22.5 | 28.1 |
| | | 65 | 80 | 60.0 | 144 | 45.0 | 24.0 | 30.0 |
| | A514 | 100 | 110-130 | 82.5 | 198 | 61.9 | 33.0 | 41.3 |
| | | 90 | 100-130 | 75.0 | 180 | 56.3 | 30.0 | 37.5 |
| | A606 | 45 | 65 | 48.8 | 117 | — | — | — |
| | | 50 | 70 | 52.5 | 126 | — | — | — |
| | A607 | 45 | 60 | 45.0 | 108 | — | — | — |
| | | 50 | 65 | 48.8 | 117 | — | — | — |
| | | 55 | 70 | 52.5 | 126 | — | — | — |
| | | 60 | 75 | 56.3 | 135 | — | — | — |
| | | 65 | 80 | 60.0 | 144 | — | — | — |
| | | 70 | 85 | 63.8 | 153 | — | — | — |
| | A618 | 50 | 70 | 52.5 | 126 | — | — | — |
| | | 50 | 65 | 48.8 | 117 | — | — | — |

# TABLE 2 (cont'd)
# Design Strength as a Function of $F_u$

| Item | ASTM Designation | $F_y$ (ksi) | $F_u$ (ksi) | Design Strength (ksi): Connection Part of Designated Steel | | Design Strength (ksi): Bolt of Threaded Part of Designated Steel | | |
|---|---|---|---|---|---|---|---|---|
| | | | | Tension 0.75 × $F_u$ [a] | Bearing 0.75 × $2.4F_u$ [b] | Tension 0.75 × $0.75F_u$ [c] | Shear 0.75 × $0.40F_u$ [d] | Shear 0.75 × $0.50F_u$ [e] |
| Bolts | A449 | 92 | 120 | — | — | 67.5 | 36.0 | 45.0 |
| | | 81 | 105 | — | — | 59.1 | 31.5 | 39.4 |
| | | 58 | 90 | — | — | 50.6 | 27.0 | 33.8 |

[a] On effective net area, see Sections D1, J5.2.
[b] Produced by fastener in shear, see Section J3.10. Note that smaller maximum design bearing stresses, as a function of hole type spacing, are given.
[c] On nominal body area, see Table J3.2.
[d] Threads not excluded from shear plane, see Table J3.2.
[e] Threads excluded from shear plane, see Table J3.2.
[f] Smaller value for circular shapes, larger for square or rectangular shapes.
Note: For dimensional and size limitations, see the appropriate ASTM Specification.

# TABLE 3-36
## Design Stress for Compression Members of 36 ksi Specified Yield Stress Steel, $\phi_c = 0.85$[a]

| $\frac{Kl}{r}$ | $\phi_c F_{cr}$ ksi | $\frac{Kl}{r}$ | $\phi_c F_{cr}$ ksi | $\frac{Kl}{r}$ | $\phi_c F_{cr}$ ksi | $\frac{Kl}{r}$ | $\phi_c F_{cr}$ ksi | $\frac{Kl}{r}$ | $\phi_c F_{cr}$ ksi |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 30.60 | 41 | 28.01 | 81 | 21.66 | 121 | 14.16 | 161 | 8.23 |
| 2 | 30.59 | 42 | 27.89 | 82 | 21.48 | 122 | 13.98 | 162 | 8.13 |
| 3 | 30.59 | 43 | 27.76 | 83 | 21.29 | 123 | 13.80 | 163 | 8.03 |
| 4 | 30.57 | 44 | 27.64 | 84 | 21.11 | 124 | 13.62 | 164 | 7.93 |
| 5 | 30.56 | 45 | 27.51 | 85 | 20.92 | 125 | 13.44 | 165 | 7.84 |
| 6 | 30.54 | 46 | 27.37 | 86 | 20.73 | 126 | 13.27 | 166 | 7.74 |
| 7 | 30.52 | 47 | 27.24 | 87 | 20.54 | 127 | 13.09 | 167 | 7.65 |
| 8 | 30.50 | 48 | 27.11 | 88 | 20.36 | 128 | 12.92 | 168 | 7.56 |
| 9 | 30.47 | 49 | 26.97 | 89 | 20.17 | 129 | 12.74 | 169 | 7.47 |
| 10 | 30.44 | 50 | 26.83 | 90 | 19.98 | 130 | 12.57 | 170 | 7.38 |
| 11 | 30.41 | 51 | 26.68 | 91 | 19.79 | 131 | 12.40 | 171 | 7.30 |
| 12 | 30.37 | 52 | 26.54 | 92 | 19.60 | 132 | 12.23 | 172 | 7.21 |
| 13 | 30.33 | 53 | 26.39 | 93 | 19.41 | 133 | 12.06 | 173 | 7.13 |
| 14 | 30.29 | 54 | 26.25 | 94 | 19.22 | 134 | 11.88 | 174 | 7.05 |
| 15 | 30.24 | 55 | 26.10 | 95 | 19.03 | 135 | 11.71 | 175 | 6.97 |
| 16 | 30.19 | 56 | 25.94 | 96 | 18.84 | 136 | 11.54 | 176 | 6.89 |
| 17 | 30.14 | 57 | 25.79 | 97 | 18.65 | 137 | 11.37 | 177 | 6.81 |
| 18 | 30.08 | 58 | 25.63 | 98 | 18.46 | 138 | 11.20 | 178 | 6.73 |
| 19 | 30.02 | 59 | 25.48 | 99 | 18.27 | 139 | 11.04 | 179 | 6.66 |
| 20 | 29.96 | 60 | 25.32 | 100 | 18.08 | 140 | 10.89 | 180 | 6.59 |
| 21 | 29.90 | 61 | 25.16 | 101 | 17.89 | 141 | 10.73 | 181 | 6.51 |
| 22 | 29.83 | 62 | 24.99 | 102 | 17.70 | 142 | 10.58 | 182 | 6.44 |
| 23 | 29.76 | 63 | 24.83 | 103 | 17.51 | 143 | 10.43 | 183 | 6.37 |
| 24 | 26.69 | 64 | 24.67 | 104 | 17.32 | 144 | 10.29 | 184 | 6.30 |
| 25 | 29.61 | 65 | 24.50 | 105 | 17.13 | 145 | 10.15 | 185 | 6.23 |
| 26 | 29.53 | 66 | 24.33 | 106 | 16.94 | 146 | 10.01 | 186 | 6.17 |
| 27 | 29.45 | 67 | 24.16 | 107 | 16.75 | 147 | 9.87 | 187 | 6.10 |
| 28 | 29.36 | 68 | 23.99 | 108 | 16.56 | 148 | 9.74 | 188 | 6.04 |
| 29 | 29.28 | 69 | 23.82 | 109 | 16.37 | 149 | 9.61 | 189 | 5.97 |
| 30 | 29.18 | 70 | 23.64 | 110 | 16.19 | 150 | 9.48 | 190 | 5.91 |
| 31 | 29.09 | 71 | 23.47 | 111 | 16.00 | 151 | 9.36 | 191 | 5.85 |
| 32 | 28.99 | 72 | 23.29 | 112 | 15.81 | 152 | 9.23 | 192 | 5.79 |
| 33 | 28.90 | 73 | 23.12 | 113 | 15.63 | 153 | 9.11 | 193 | 5.73 |
| 34 | 28.79 | 74 | 22.94 | 114 | 15.44 | 154 | 9.00 | 194 | 5.67 |
| 35 | 28.69 | 75 | 22.76 | 115 | 15.26 | 155 | 8.88 | 195 | 5.61 |
| 36 | 28.58 | 76 | 22.58 | 116 | 15.07 | 156 | 8.77 | 196 | 5.55 |
| 37 | 28.47 | 77 | 22.40 | 117 | 14.89 | 157 | 8.66 | 197 | 5.50 |
| 38 | 28.36 | 78 | 22.22 | 118 | 14.70 | 158 | 8.55 | 198 | 5.44 |
| 39 | 28.25 | 79 | 22.03 | 119 | 14.52 | 159 | 8.44 | 199 | 5.39 |
| 40 | 28.13 | 80 | 21.85 | 120 | 14.34 | 160 | 8.33 | 200 | 5.33 |

[a] When element width-to-thickness ration exceeds $\lambda_r$, see Appendix B5.3.

# TABLE 3-50
## Design Stress for Compression Members of 50 ksi Specified Yield Stress Steel, $\phi_c = 0.85$[a]

| $\frac{Kl}{r}$ | $\phi_c F_{cr}$ ksi | $\frac{Kl}{r}$ | $\phi_c F_{cr}$ ksi | $\frac{Kl}{r}$ | $\phi_c F_{cr}$ ksi | $\frac{Kl}{r}$ | $\phi_c F_{cr}$ ksi | $\frac{Kl}{r}$ | $\phi_c F_{cr}$ ksi |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 42.50 | 41 | 37.59 | 81 | 26.31 | 121 | 14.57 | 161 | 8.23 |
| 2 | 42.49 | 42 | 37.36 | 82 | 26.00 | 122 | 14.33 | 162 | 8.13 |
| 3 | 42.47 | 43 | 37.13 | 83 | 25.68 | 123 | 14.10 | 163 | 8.03 |
| 4 | 42.45 | 44 | 36.89 | 84 | 25.37 | 124 | 13.88 | 164 | 7.93 |
| 5 | 42.42 | 45 | 36.65 | 85 | 25.06 | 125 | 13.66 | 165 | 7.84 |
| 6 | 42.39 | 46 | 36.41 | 86 | 24.75 | 126 | 13.44 | 166 | 7.74 |
| 7 | 42.35 | 47 | 36.16 | 87 | 24.44 | 127 | 13.23 | 167 | 7.65 |
| 8 | 42.30 | 48 | 35.91 | 88 | 24.13 | 128 | 13.02 | 168 | 7.56 |
| 9 | 42.25 | 49 | 35.66 | 89 | 23.82 | 129 | 12.82 | 169 | 7.47 |
| 10 | 42.19 | 50 | 35.40 | 90 | 23.51 | 130 | 12.62 | 170 | 7.38 |
| 11 | 42.13 | 51 | 35.14 | 91 | 23.20 | 131 | 12.43 | 171 | 7.30 |
| 12 | 42.05 | 52 | 34.88 | 92 | 22.89 | 132 | 12.25 | 172 | 7.21 |
| 13 | 41.98 | 53 | 34.61 | 93 | 22.58 | 133 | 12.06 | 173 | 7.13 |
| 14 | 41.90 | 54 | 34.34 | 94 | 22.28 | 134 | 11.88 | 174 | 7.05 |
| 15 | 41.81 | 55 | 34.07 | 95 | 21.97 | 135 | 11.71 | 175 | 6.97 |
| 16 | 41.71 | 56 | 33.79 | 96 | 21.67 | 136 | 11.54 | 176 | 6.89 |
| 17 | 41.61 | 57 | 33.51 | 97 | 21.36 | 137 | 11.37 | 177 | 6.81 |
| 18 | 41.51 | 58 | 33.23 | 98 | 21.06 | 138 | 11.20 | 178 | 6.73 |
| 19 | 41.39 | 59 | 32.95 | 99 | 20.76 | 139 | 11.04 | 179 | 6.66 |
| 20 | 41.28 | 60 | 32.67 | 100 | 20.46 | 140 | 10.89 | 180 | 6.59 |
| 21 | 41.15 | 61 | 32.38 | 101 | 20.16 | 141 | 10.73 | 181 | 6.51 |
| 22 | 41.02 | 62 | 32.09 | 102 | 19.86 | 142 | 10.58 | 182 | 6.44 |
| 23 | 40.89 | 63 | 31.80 | 103 | 19.57 | 143 | 10.43 | 183 | 6.37 |
| 24 | 40.75 | 64 | 31.50 | 104 | 19.28 | 144 | 10.29 | 184 | 6.30 |
| 25 | 40.60 | 65 | 31.21 | 105 | 18.98 | 145 | 10.15 | 185 | 6.23 |
| 26 | 40.45 | 66 | 30.91 | 106 | 18.69 | 146 | 10.01 | 186 | 6.17 |
| 27 | 40.29 | 67 | 30.61 | 107 | 18.40 | 147 | 9.87 | 187 | 6.10 |
| 28 | 40.13 | 68 | 30.31 | 108 | 18.12 | 148 | 9.74 | 188 | 6.04 |
| 29 | 39.97 | 69 | 30.01 | 109 | 17.83 | 149 | 9.61 | 189 | 5.97 |
| 30 | 39.79 | 70 | 29.70 | 110 | 17.55 | 150 | 9.48 | 190 | 5.91 |
| 31 | 39.62 | 71 | 29.40 | 111 | 17.27 | 151 | 9.36 | 191 | 5.85 |
| 32 | 39.43 | 72 | 20.09 | 112 | 16.99 | 152 | 9.23 | 192 | 5.79 |
| 33 | 39.25 | 73 | 28.79 | 113 | 16.71 | 153 | 9.11 | 193 | 5.73 |
| 34 | 39.06 | 74 | 28.48 | 114 | 16.42 | 154 | 9.00 | 194 | 5.67 |
| 35 | 38.86 | 75 | 28.17 | 115 | 16.13 | 155 | 8.88 | 195 | 5.61 |
| 36 | 38.66 | 76 | 27.86 | 116 | 15.86 | 156 | 8.77 | 196 | 5.55 |
| 37 | 38.45 | 77 | 27.55 | 117 | 15.59 | 157 | 8.66 | 197 | 5.50 |
| 38 | 38.24 | 78 | 27.24 | 118 | 15.32 | 158 | 8.55 | 198 | 5.44 |
| 39 | 38.03 | 79 | 26.93 | 119 | 15.07 | 159 | 8.44 | 199 | 5.39 |
| 40 | 37.81 | 80 | 26.62 | 120 | 14.82 | 160 | 8.33 | 200 | 5.33 |

[a] When element width-to-thickness ratio exceeds $\lambda_r$, see Appendix B5.3.

# TABLE 4
## Values of $\phi_c F_{cr} / F_y$, $\phi_c = 0.85$ for Determining Design Stress for Compression Members for Steel of Any Yield Stress[a]

| $\lambda_c$ | $\phi_c F_{cr} / F_y$ | $\lambda_c$ | $\phi_c F_{cr} / F_y$ | $\lambda_c$ | $\phi_c F_{cr} / F_y$ | $\lambda_c$ | $\phi_c F_{cr} / F_y$ |
|---|---|---|---|---|---|---|---|
| 0.02 | 0.850 | 0.82 | 0.641 | 1.62 | 0.284 | 2.42 | 0.127 |
| 0.04 | 0.849 | 0.84 | 0.632 | 1.64 | 0.277 | 2.44 | 0.125 |
| 0.06 | 0.849 | 0.86 | 0.623 | 1.66 | 0.271 | 2.46 | 0.123 |
| 0.08 | 0.848 | 0.88 | 0.614 | 1.68 | 0.264 | 2.48 | 0.121 |
| 0.10 | 0.846 | 0.90 | 0.605 | 1.70 | 0.258 | 2.50 | 0.119 |
| 0.12 | 0.845 | 0.92 | 0.596 | 1.72 | 0.252 | 2.52 | 0.117 |
| 0.14 | 0.843 | 0.94 | 0.587 | 1.74 | 0.246 | 2.54 | 0.116 |
| 0.16 | 0.841 | 0.96 | 0.578 | 1.76 | 0.241 | 2.56 | 0.114 |
| 0.18 | 0.839 | 0.98 | 0.568 | 1.78 | 0.235 | 2.58 | 0.112 |
| 0.20 | 0.836 | 1.00 | 0.559 | 1.80 | 0.230 | 2.60 | 0.110 |
| 0.22 | 0.833 | 1.02 | 0.550 | 1.82 | 0.225 | 2.62 | 0.109 |
| 0.24 | 0.830 | 1.04 | 0.540 | 1.84 | 0.220 | 2.64 | 0.107 |
| 0.26 | 0.826 | 1.06 | 0.531 | 1.86 | 0.215 | 2.66 | 0.105 |
| 0.28 | 0.823 | 1.08 | 0.521 | 1.88 | 0.211 | 2.68 | 0.104 |
| 0.30 | 0.819 | 1.10 | 0.512 | 1.90 | 0.206 | 2.70 | 0.102 |
| 0.32 | 0.814 | 1.12 | 0.503 | 1.92 | 0.202 | 2.72 | 0.101 |
| 0.34 | 0.810 | 1.14 | 0.493 | 1.94 | 0.198 | 2.74 | 0.099 |
| 0.36 | 0.805 | 1.16 | 0.484 | 1.96 | 0.194 | 2.76 | 0.098 |
| 0.38 | 0.800 | 1.18 | 0.474 | 1.98 | 0.190 | 2.78 | 0.096 |
| 0.40 | 0.795 | 1.20 | 0.465 | 2.00 | 0.186 | 2.80 | 0.095 |
| 0.42 | 0.789 | 1.22 | 0.456 | 2.02 | 0.183 | 2.82 | 0.094 |
| 0.44 | 0.784 | 1.24 | 0.446 | 2.04 | 0.179 | 2.84 | 0.092 |
| 0.46 | 0.778 | 1.26 | 0.437 | 2.06 | 0.176 | 2.86 | 0.091 |
| 0.48 | 0.772 | 1.28 | 0.428 | 2.08 | 0.172 | 2.88 | 0.090 |
| 0.50 | 0.765 | 1.30 | 0.419 | 2.10 | 0.169 | 2.90 | 0.089 |
| 0.52 | 0.759 | 1.32 | 0.410 | 2.12 | 0.166 | 2.92 | 0.087 |
| 0.54 | 0.752 | 1.34 | 0.401 | 2.14 | 0.163 | 2.94 | 0.086 |
| 0.56 | 0.745 | 1.36 | 0.392 | 2.16 | 0.160 | 2.96 | 0.085 |
| 0.58 | 0.738 | 1.38 | 0.383 | 2.18 | 0.157 | 2.98 | 0.084 |
| 0.60 | 0.731 | 1.40 | 0.374 | 2.20 | 0.154 | 3.00 | 0.083 |
| 0.62 | 0.724 | 1.42 | 0.365 | 2.22 | 0.151 | 3.02 | 0.082 |
| 0.64 | 0.716 | 1.44 | 0.357 | 2.24 | 0.149 | 3.04 | 0.081 |
| 0.66 | 0.708 | 1.46 | 0.348 | 2.26 | 0.146 | 3.06 | 0.080 |
| 0.68 | 0.700 | 1.48 | 0.339 | 2.28 | 0.143 | 3.08 | 0.079 |
| 0.70 | 0.692 | 1.50 | 0.331 | 2.30 | 0.141 | 3.10 | 0.078 |
| 0.72 | 0.684 | 1.52 | 0.323 | 2.32 | 0.138 | 3.12 | 0.077 |
| 0.74 | 0.676 | 1.54 | 0.314 | 2.34 | 0.136 | 3.14 | 0.076 |
| 0.76 | 0.667 | 1.56 | 0.306 | 2.36 | 0.134 | 3.16 | 0.075 |
| 0.78 | 0.659 | 1.58 | 0.299 | 2.38 | 0.132 | 3.18 | 0.074 |
| 0.80 | 0.650 | 1.60 | 0.291 | 2.40 | 0.129 | 3.20 | 0.073 |

[a] When element width-to-thickness ratios exceed $\lambda_r$, see Appendix B5.3.
Values of $\lambda_c > 2.24$ exceed $Kl/r$ of 200 for $F_y = 36$
Values of $\lambda_c > 2.64$ exceed $Kl/r$ of 200 for $F_y = 50$

# TABLE 5
## Values of $Kl / r$ for $F_y$ = 36 and 50 ksi

| $\lambda_c$ | $Kl/r$ | | $\lambda_c$ | $Kl/r$ | |
|---|---|---|---|---|---|
| | $F_y$ = 36 | $F_y$ = 50 | | $F_y$ = 36 | $F_y$ = 50 |
| 0.02 | 1.8 | 1.5 | 0.82 | 73.1 | 62.0 |
| 0.04 | 3.6 | 3.0 | 0.84 | 74.9 | 63.6 |
| 0.06 | 4.3 | 4.5 | 0.86 | 76.7 | 65.1 |
| 0.08 | 7.1 | 6.1 | 0.88 | 78.5 | 66.6 |
| 0.10 | 8.9 | 7.6 | 0.90 | 80.2 | 68.1 |
| 0.12 | 10.7 | 9.1 | 0.92 | 82.0 | 69.6 |
| 0.14 | 12.5 | 10.6 | 0.94 | 83.8 | 71.1 |
| 0.16 | 14.3 | 12.1 | 0.96 | 85.6 | 72.6 |
| 0.18 | 16.0 | 13.6 | 0.98 | 87.4 | 74.1 |
| 0.20 | 17.8 | 15.1 | 1.00 | 89.2 | 75.7 |
| 0.22 | 19.6 | 16.6 | 1.02 | 90.9 | 77.2 |
| 0.24 | 21.4 | 18.2 | 1.04 | 92.7 | 78.7 |
| 0.26 | 23.2 | 19.7 | 1.06 | 94.5 | 80.2 |
| 0.28 | 25.0 | 21.2 | 1.08 | 96.3 | 81.7 |
| 0.30 | 26.7 | 22.7 | 1.10 | 98.1 | 83.2 |
| 0.32 | 28.5 | 24.2 | 1.12 | 99.9 | 84.7 |
| 0.34 | 30.3 | 25.7 | 1.14 | 101.6 | 86.3 |
| 0.36 | 32.1 | 27.2 | 1.16 | 103.4 | 87.8 |
| 0.38 | 33.9 | 28.8 | 1.18 | 105.2 | 89.3 |
| 0.40 | 35.7 | 30.3 | 1.20 | 107.0 | 90.8 |
| 0.42 | 37.4 | 31.8 | 1.22 | 108.8 | 92.3 |
| 0.44 | 39.2 | 33.3 | 1.24 | 110.6 | 93.8 |
| 0.46 | 41.0 | 34.8 | 1.26 | 112.3 | 95.3 |
| 0.48 | 42.8 | 36.3 | 1.28 | 114.1 | 96.8 |
| 0.50 | 44.6 | 37.8 | 1.30 | 115.9 | 98.4 |
| 0.52 | 46.4 | 39.3 | 1.32 | 117.7 | 99.9 |
| 0.54 | 48.1 | 40.9 | 1.34 | 119.5 | 101.4 |
| 0.56 | 49.9 | 42.4 | 1.36 | 121.3 | 102.9 |
| 0.58 | 51.7 | 43.9 | 1.38 | 123.0 | 104.4 |
| 0.60 | 53.5 | 45.4 | 1.40 | 124.8 | 105.9 |
| 0.62 | 55.3 | 46.9 | 1.42 | 126.6 | 107.4 |
| 0.64 | 57.1 | 48.4 | 1.44 | 128.4 | 108.9 |
| 0.66 | 58.8 | 49.9 | 1.46 | 130.2 | 110.5 |
| 0.68 | 60.6 | 51.4 | 1.48 | 132.0 | 112.0 |
| 0.70 | 62.4 | 53.0 | 1.50 | 133.7 | 113.5 |
| 0.72 | 64.2 | 54.5 | 1.52 | 135.5 | 115.0 |
| 0.74 | 66.0 | 56.0 | 1.54 | 137.3 | 116.5 |
| 0.76 | 67.8 | 57.5 | 1.56 | 139.1 | 118.0 |
| 0.78 | 69.5 | 59.0 | 1.58 | 140.9 | 119.5 |
| 0.80 | 71.3 | 60.5 | 1.60 | 142.7 | 121.1 |

# TABLE 5 (cont'd)
## Values of $Kl/r$ for $F_y$ = 36 and 50 ksi

| $\lambda_c$ | $Kl/r$ | | $\lambda_c$ | $Kl/r$ |
|---|---|---|---|---|
| | $F_y = 36$ | $F_y = 50$ | | $F_y = 50$ |
| 1.62 | 144.4 | 122.6 | 2.42 | 183.1 |
| 1.64 | 146.2 | 124.1 | 2.44 | 184.6 |
| 1.66 | 148.0 | 125.6 | 2.46 | 186.1 |
| 1.68 | 149.8 | 127.1 | 2.48 | 187.6 |
| 1.70 | 151.6 | 128.6 | 2.50 | 189.1 |
| 1.72 | 153.4 | 130.1 | 2.52 | 190.7 |
| 1.74 | 155.1 | 131.6 | 2.54 | 192.2 |
| 1.76 | 156.9 | 133.2 | 2.56 | 193.7 |
| 1.78 | 158.7 | 134.7 | 2.58 | 195.2 |
| 1.80 | 160.5 | 136.2 | 2.60 | 196.7 |
| 1.82 | 162.3 | 137.7 | 2.62 | 198.2 |
| 1.84 | 164.1 | 139.2 | 2.64 | 199.7 |
| 1.86 | 165.8 | 140.7 | | |
| 1.88 | 167.6 | 142.2 | | |
| 1.90 | 169.4 | 143.8 | | |
| 1.92 | 171.2 | 145.3 | | |
| 1.94 | 173.0 | 146.8 | | |
| 1.96 | 174.8 | 148.3 | | |
| 1.98 | 176.5 | 149.8 | | |
| 2.00 | 178.3 | 151.3 | | |
| 2.02 | 180.1 | 152.8 | | |
| 2.04 | 181.9 | 154.3 | | |
| 2.06 | 183.7 | 155.9 | | |
| 2.08 | 185.5 | 157.4 | | |
| 2.10 | 187.2 | 158.9 | | |
| 2.12 | 189.0 | 160.4 | | |
| 2.14 | 190.8 | 161.9 | | |
| 2.16 | 192.6 | 163.4 | | |
| 2.18 | 194.4 | 164.9 | | |
| 2.20 | 196.2 | 166.5 | | |
| 2.22 | 197.9 | 168.0 | | |
| 2.24 | 199.7 | 169.5 | | |
| 2.26 | | 171.0 | | |
| 2.28 | | 172.5 | | |
| 2.30 | | 174.0 | | |
| 2.32 | | 175.5 | | |
| 2.34 | | 177.0 | | |
| 2.36 | | 178.6 | | |
| 2.38 | | 180.1 | | |
| 2.40 | | 181.6 | | |

Heavy line indicates $Kl/r$ of 200.

# TABLE 6
# Slenderness Ratios of Elements as a Function of $F_y$
# From Table B5.1

| Ratio | $F_y$ (ksi) | | | | | |
|---|---|---|---|---|---|---|
| | **36** | **42** | **46** | **50** | **60** | **65** |
| $65/\sqrt{F_y}$ | 10.8 | 10.0 | 9.6 | 9.2 | 8.4 | 8.1 |
| $76/\sqrt{F_y}$ | 12.7 | 11.7 | 11.2 | 10.7 | 9.8 | 9.4 |
| $95/\sqrt{F_y}$ | 15.8 | 14.7 | 14.0 | 13.4 | 12.3 | 11.8 |
| $127/\sqrt{F_y}$ | 21.2 | 19.6 | 18.7 | 18.0 | 16.4 | 15.8 |
| $141/\sqrt{F_y - 10}$ | 27.7 | 24.9 | 23.5 | 22.3 | 19.9 | 19.0 |
| $190/\sqrt{F_y}$ | 31.7 | 29.3 | 28.0 | 26.9 | 24.5 | 23.6 |
| $238/\sqrt{F_y}$ | 39.7 | 36.7 | 35.1 | 33.7 | 30.7 | 29.5 |
| $253/\sqrt{F_y}$ | 42.2 | 39.0 | 37.3 | 35.8 | 32.7 | 31.4 |
| $317/\sqrt{F_y}$ | 52.8 | 48.9 | 46.7 | 44.8 | 40.9 | 39.3 |
| $640/\sqrt{F_y}$ | 107.0 | 98.8 | 94.4 | 90.5 | 82.6 | 79.4 |
| $970/\sqrt{F_y}$ | 162.0 | 150.0 | 143.0 | 137.0 | 125.0 | 120.0 |
| $1{,}300/F_y$ | 36.1 | 31.0 | 28.3 | 26.0 | 21.7 | 20.0 |
| $2{,}070/F_y$ | 57.5 | 49.3 | 45.0 | 41.4 | 34.5 | 31.8 |
| $3{,}300/F_y$ | 91.7 | 78.6 | 71.7 | 66.0 | 55.0 | 50.8 |
| $8{,}970/F_y$ | 249.0 | 214.0 | 195.0 | 179.0 | 150.0 | 138.0 |

# TABLE 7
# Values of $C_m$
# for Use in Section C1

| $\frac{M_1}{M_2}$ | $C_m$ | $\frac{M_1}{M_2}$ | $C_m$ | $\frac{M_1}{M_2}$ | $C_m$ |
|---|---|---|---|---|---|
| -1.00 | 1.00 | -0.45 | 0.78 | 0.10 | 0.56 |
| -0.95 | 0.98 | -0.40 | 0.76 | 0.15 | 0.54 |
| -0.90 | 0.96 | -0.35 | 0.74 | 0.20 | 0.52 |
| -0.85 | 0.94 | -0.30 | 0.72 | 0.25 | 0.50 |
| -0.80 | 0.92 | -0.25 | 0.70 | 0.30 | 0.48 |
| -0.75 | 0.90 | -0.20 | 0.68 | 0.35 | 0.46 |
| -0.70 | 0.88 | -0.15 | 0.66 | 0.40 | 0.44 |
| -0.65 | 0.86 | -0.10 | 0.64 | 0.45 | 0.42 |
| -0.60 | 0.84 | -0.05 | 0.62 | 0.50 | 0.40 |
| | | | | 0.60 | 0.36 |
| -0.55 | 0.82 | 0 | 0.60 | 0.80 | 0.28 |
| -0.50 | 0.80 | 0.05 | 0.58 | 1.00 | 0.20 |

Note 1: $C_m = 0.6 - 0.4(M_1 / M_2)$.
Note 2: $M_1 / M_2$ is positive for reverse curvature and negative for single curvature. $|M_1| \leq |M_2|$

# TABLE 8

## Values of $P_e / A_g$ for Use in Section C1 for Steel of Any Yield Stress

| $\frac{Kl}{r}$ | $P_e / A_g$ (ksi) | $\frac{Kl}{r}$ | $P_e / A_g$ (ksi) | $\frac{Kl}{r}$ | $P_e / A_g$ (ksi) | $\frac{Kl}{r}$ | $P_e / A_g$ (ksi) | $\frac{Kl}{r}$ | $P_e / A_g$ (ksi) | $\frac{Kl}{r}$ | $P_e / A_g$ (ksi) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 21 | 649.02 | 51 | 110.04 | 81 | 43.62 | 111 | 23.23 | 141 | 14.40 | 171 | 9.79 |
| 22 | 591.36 | 52 | 105.85 | 82 | 42.57 | 112 | 22.82 | 142 | 14.19 | 172 | 9.67 |
| 23 | 541.06 | 53 | 101.89 | 83 | 41.55 | 113 | 22.42 | 143 | 14.00 | 173 | 9.56 |
| 24 | 496.91 | 54 | 98.15 | 84 | 40.56 | 114 | 22.02 | 144 | 13.80 | 174 | 9.45 |
| 25 | 457.95 | 55 | 94.62 | 85 | 39.62 | 115 | 21.64 | 145 | 13.61 | 175 | 9.35 |
| 26 | 423.40 | 56 | 91.27 | 86 | 38.70 | 116 | 21.27 | 146 | 13.43 | 176 | 9.24 |
| 27 | 392.62 | 57 | 88.09 | 87 | 37.81 | 117 | 20.91 | 147 | 13.25 | 177 | 9.14 |
| 28 | 365.07 | 58 | 85.08 | 88 | 36.96 | 118 | 20.56 | 148 | 13.07 | 178 | 9.03 |
| 29 | 340.33 | 59 | 82.22 | 89 | 36.13 | 119 | 20.21 | 149 | 12.89 | 179 | 8.93 |
| 30 | 318.02 | 60 | 79.51 | 90 | 35.34 | 120 | 19.88 | 150 | 12.72 | 180 | 8.83 |
| 31 | 297.83 | 61 | 76.92 | 91 | 34.56 | 121 | 19.55 | 151 | 12.55 | 181 | 8.74 |
| 32 | 279.51 | 62 | 74.46 | 92 | 33.82 | 122 | 19.23 | 152 | 12.39 | 182 | 8.64 |
| 33 | 262.83 | 63 | 72.11 | 93 | 33.09 | 123 | 18.92 | 153 | 12.23 | 183 | 8.55 |
| 34 | 247.59 | 64 | 69.88 | 94 | 32.39 | 124 | 18.61 | 154 | 12.07 | 184 | 8.45 |
| 35 | 233.65 | 65 | 67.74 | 95 | 31.71 | 125 | 18.32 | 155 | 11.91 | 185 | 8.36 |
| 36 | 220.85 | 66 | 65.71 | 96 | 31.06 | 126 | 18.03 | 156 | 11.76 | 186 | 8.27 |
| 37 | 209.07 | 67 | 63.76 | 97 | 30.42 | 127 | 17.75 | 157 | 11.61 | 187 | 8.18 |
| 38 | 198.21 | 68 | 61.90 | 98 | 29.80 | 128 | 17.47 | 158 | 11.47 | 188 | 8.10 |
| 39 | 188.18 | 69 | 60.12 | 99 | 29.20 | 129 | 17.20 | 159 | 11.32 | 189 | 8.01 |
| 40 | 178.89 | 70 | 58.41 | 100 | 28.62 | 130 | 16.94 | 160 | 11.18 | 190 | 7.93 |
| 41 | 170.27 | 71 | 56.78 | 101 | 28.06 | 131 | 16.68 | 161 | 11.04 | 191 | 7.85 |
| 42 | 162.26 | 72 | 55.21 | 102 | 27.51 | 132 | 16.43 | 162 | 10.91 | 192 | 7.76 |
| 43 | 154.80 | 73 | 53.71 | 103 | 26.98 | 133 | 16.18 | 163 | 10.77 | 193 | 7.68 |
| 44 | 147.84 | 74 | 52.57 | 104 | 26.46 | 134 | 15.94 | 164 | 10.64 | 194 | 7.60 |
| 45 | 141.34 | 75 | 50.88 | 105 | 25.96 | 135 | 15.70 | 165 | 10.51 | 195 | 7.53 |
| 46 | 135.26 | 76 | 49.55 | 106 | 25.47 | 136 | 15.47 | 166 | 10.39 | 196 | 7.45 |
| 47 | 129.57 | 77 | 48.27 | 107 | 25.00 | 137 | 15.25 | 167 | 10.26 | 197 | 7.38 |
| 48 | 124.23 | 78 | 47.04 | 108 | 24.54 | 138 | 15.03 | 168 | 10.14 | 198 | 7.30 |
| 49 | 119.21 | 79 | 45.86 | 109 | 24.09 | 139 | 14.81 | 169 | 10.02 | 199 | 7.23 |
| 50 | 114.49 | 80 | 44.72 | 110 | 23.65 | 140 | 14.60 | 170 | 9.90 | 200 | 7.16 |

Note: $P_e / A_g = \dfrac{\pi^2 E}{(Kl / r)^2}$, use for both $P_{e1}$ and $P_{e2}$.

# TABLE 9-36
## $\dfrac{\phi_v V_n}{A_w}$ (ksi) for Plate Girders by Appendix F2 for 36 ksi Yield Stress Steel, Tension Field Action Not Included

| $\dfrac{h}{t_w}$ | Aspect ratio *a* / *h*: Stiffener Spacing to Web Depth | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | **0.5** | **0.6** | **0.7** | **0.8** | **0.9** | **1.0** | **1.2** | **1.4** | **1.6** | **1.8** | **2.0** | **2.5** | **3.0** | **Over 3.0** |
| 60 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 |
| 70 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 |
| 80 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 18.9 | 18.2 | 17.9 | 16.9 |
| 90 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 18.5 | 17.8 | 17.2 | 16.8 | 16.2 | 15.9 | 14.7 |
| 100 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.2 | 17.6 | 16.6 | 16.0 | 15.5 | 14.9 | 13.8 | 13.2 | 11.9 |
| 110 | 19.4 | 19.4 | 19.4 | 19.4 | 18.4 | 17.4 | 16.0 | 14.8 | 13.7 | 12.8 | 12.3 | 11.4 | 10.9 | 9.8 |
| 120 | 19.4 | 19.4 | 19.4 | 18.1 | 16.9 | 16.0 | 14.0 | 12.5 | 11.5 | 10.8 | 10.3 | 9.6 | 9.2 | 8.3 |
| 130 | 19.4 | 19.4 | 18.2 | 16.7 | 15.6 | 14.1 | 11.9 | 10.6 | 9.8 | 9.2 | 8.8 | 8.2 | 7.8 | 7.0 |
| 140 | 19.4 | 18.8 | 16.9 | 15.5 | 13.5 | 12.1 | 10.3 | 9.2 | 8.4 | 7.9 | 7.6 | 7.0 | 6.7 | 6.1 |
| 150 | 19.4 | 17.6 | 15.7 | 13.5 | 11.8 | 10.6 | 8.9 | 8.0 | 7.3 | 6.9 | 6.6 | 6.1 | 5.9 | 5.3 |
| 160 | 18.9 | 16.5 | 14.1 | 11.9 | 10.4 | 9.3 | 7.9 | 7.0 | 6.5 | 6.1 | 5.8 | 5.4 | | 4.6 |
| 170 | 17.8 | 15.5 | 12.5 | 10.5 | 9.2 | 8.2 | 7.0 | 6.2 | 5.7 | 5.4 | 5.1 | | | 4.1 |
| 180 | 16.8 | 13.9 | 11.1 | 9.4 | 8.2 | 7.3 | 6.2 | 5.5 | 5.1 | 4.8 | 4.6 | | | 3.7 |
| 200 | 14.9 | 11.2 | 9.0 | 7.6 | 6.6 | 5.9 | 5.0 | 4.5 | 4.1 | | | | | 3.0 |
| 220 | 12.3 | 9.3 | 7.5 | 6.3 | 5.5 | 4.9 | 4.2 | | | | | | | 2.5 |
| 240 | 10.3 | 7.8 | 6.3 | 5.3 | 4.6 | 4.1 | | | | | | | | 2.1 |
| 260 | 8.8 | 6.6 | 5.3 | 4.5 | 3.9 | 3.5 | | | | | | | | 1.8 |
| 280 | 7.6 | 5.7 | 4.6 | 3.9 | | | | | | | | | | |
| 300 | 6.6 | 5.0 | 4.0 | | | | | | | | | | | |
| 320 | 5.8 | 4.4 | | | | | | | | | | | | |

# TABLE 9-50
# $\frac{\phi_v V_n}{A_w}$ (ksi) for Plate Girders by Appendix F2 for 50 ksi Yield Stress Steel, Tension Field Action Not Included

| $\frac{h}{t_w}$ | Aspect ratio *a* / *h*: Stiffener Spacing to Web Depth | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | **0.5** | **0.6** | **0.7** | **0.8** | **0.9** | **1.0** | **1.2** | **1.4** | **1.6** | **1.8** | **2.0** | **2.5** | **3.0** | **Over 3.0** |
| 60 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 26.6 |
| 70 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 26.9 | 26.1 | 25.5 | 24.6 | 24.0 | 22.8 |
| 80 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 26.0 | 24.5 | 23.5 | 22.8 | 22.3 | 21.5 | 20.6 | 18.6 |
| 90 | 27.0 | 27.0 | 27.0 | 27.0 | 26.5 | 25.1 | 23.1 | 21.8 | 20.4 | 19.2 | 18.3 | 17.0 | 16.3 | 14.7 |
| 100 | 27.0 | 27.0 | 27.0 | 25.6 | 23.9 | 22.6 | 20.1 | 17.9 | 16.5 | 15.5 | 14.9 | 13.8 | 13.2 | 11.9 |
| 110 | 27.0 | 27.0 | 25.3 | 23.2 | 21.7 | 19.6 | 16.6 | 14.8 | 13.7 | 12.8 | 12.3 | 11.4 | 10.9 | 9.8 |
| 120 | 27.0 | 25.9 | 23.2 | 21.1 | 18.4 | 16.5 | 14.0 | 12.5 | 11.5 | 10.8 | 10.3 | 9.6 | 9.2 | 8.3 |
| 130 | 27.0 | 23.9 | 21.4 | 18.0 | 15.7 | 14.1 | 11.9 | 10.6 | 9.8 | 9.2 | 8.8 | 8.2 | 7.8 | 7.0 |
| 140 | 25.5 | 22.2 | 18.4 | 15.5 | 13.5 | 12.1 | 10.3 | 9.2 | 8.4 | 7.9 | 7.6 | 7.0 | 6.7 | 6.1 |
| 150 | 23.8 | 19.9 | 16.1 | 13.5 | 11.8 | 10.6 | 8.9 | 8.0 | 7.3 | 6.9 | 6.6 | 6.1 | 5.9 | 5.3 |
| 160 | 22.3 | 17.5 | 14.1 | 11.9 | 10.4 | 9.3 | 7.9 | 7.0 | 6.5 | 6.1 | 5.8 | 5.4 | | 4.6 |
| 170 | 20.6 | 15.5 | 12.5 | 10.5 | 9.2 | 8.2 | 7.0 | 6.2 | 5.7 | 5.4 | 5.1 | | | 4.1 |
| 180 | 18.3 | 13.9 | 11.1 | 9.4 | 8.2 | 7.3 | 6.2 | 5.5 | 5.1 | 4.8 | 4.6 | | | 3.7 |
| 200 | 14.9 | 11.2 | 9.0 | 7.6 | 6.6 | 5.9 | 5.0 | 4.5 | 4.1 | | | | | 3.0 |
| 220 | 12.3 | 9.3 | 7.5 | 6.3 | 5.5 | 4.9 | 4.2 | | | | | | | 2.5 |
| 240 | 10.3 | 7.8 | 6.3 | 5.3 | 4.6 | 4.1 | | | | | | | | 2.1 |
| 260 | 8.8 | 6.6 | 5.3 | 4.5 | 3.9 | 3.5 | | | | | | | | 1.8 |
| 280 | 7.6 | 5.7 | 4.6 | 3.9 | | | | | | | | | | |

# TABLE 10-36
# $\frac{\phi_v V_n}{A_w}$ (ksi) for Plate Girders by Appendix G for 36 ksi Yield Stress Steel, Tension Field Action Included[b]

(Italic values indicate gross area, as percent of $(h \times t_w)$ required for pairs of intermediate stiffeners of 36 ksi yield stress steel with $V_u / \phi V_n = 1.0$) [a]

| $\frac{h}{t_w}$ | **Aspect ratio *a* / *h*: Stiffener Spacing to Web Depth** | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | **0.5** | **0.6** | **0.7** | **0.8** | **0.9** | **1.0** | **1.2** | **1.4** | **1.6** | **1.8** | **2.0** | **2.5** | **3.0** | **Over 3.0 [c]** |
| 60 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 |
| 70 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 |
| 80 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.1 | 18.6 | 18.3 | 16.9 |
| 90 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.0 | 18.5 | 18.2 | 17.8 | 17.3 | 16.8 | 14.7 |
| 100 | 19.4 | 19.4 | 19.4 | 19.4 | 19.4 | 19.3 | 18.6 | 18.1 | 17.6 | 17.2 | 16.6 | 15.6 | 14.9 | 11.9 |
| 110 | 19.4 | 19.4 | 19.4 | 19.4 | 19.1 | 18.7 | 17.9 | 17.2 | 16.3 | 15.6 | 15.1 | 14.0 | 13.3 | 9.8 |
| 120 | 19.4 | 19.4 | 19.4 | 19.0 | 18.5 | 18.1 | 17.0 | 16.0 | 15.1 | 14.4 | 13.9 | 12.8 | 12.0 | 8.3 |
| 130 | 19.4 | 19.4 | 19.1 | 18.6 | 18.1 | 17.4 | 16.1 | 15.1 | 14.2 | 13.5 | 12.9 | 11.8 | 11.0 | 7.0 |
| 140 | 19.4 | 19.3 | 18.7 | 18.2 | 17.4 | 16.6 | 15.4 | 14.4 | 13.5 | 12.8 | 12.2 | 11.0 | 10.2 | 6.1 |
| 150 | 19.4 | 19.0 | 18.4 | 17.5 | 16.7 | 16.0 | 14.8 | 13.8 | 12.9 | 12.2 | 11.6 | 10.4 | 9.6 | 5.3 |
| 160 | 19.3 | 18.7 | 17.9 | 17.0 | 16.2 | 15.5 | 14.3 | 13.3 | 12.4 | 11.7 | 11.1 | 9.9 | | 4.6 |
| 170 | 19.1 | 18.4 | 17.4 | 16.6 | 15.8 | 15.1 | 13.9 | 12.9 | 12.0 | 11.3<br>*0.3* | 10.7<br>*0.4* | | | 4.1 |
| 180 | 18.9 | 18.0 | 17.1 | 16.2 | 15.5 | 14.8 | 13.6<br>*0.2* | 12.6<br>*0.7* | 11.7<br>*1.1* | 11.0<br>*1.3* | 10.4<br>*1.5* | | | 3.7 |
| 200 | 18.4 | 17.3 | 16.4 | 15.6<br>*0.1* | 14.9<br>*0.9* | 14.2<br>*1.4* | 13.1<br>*2.1* | 12.0<br>*2.5* | 11.2<br>*2.8* | | | | | 3.0 |
| 220 | 17.8 | 16.9 | 16.0<br>*1.1* | 15.2<br>*2.0* | 14.5<br>*2.6* | 13.8<br>*3.0* | 12.7<br>*3.6* | | | | | | | 2.5 |
| 240 | 17.4 | 16.5<br>*1.5* | 15.7<br>*2.7* | 14.9<br>*3.4* | 14.2<br>*3.9* | 13.5<br>*4.3* | | | | | | | | 2.1 |
| 260 | 17.1<br>*1.3* | 16.2<br>*3.0* | 15.4<br>*4.0* | 14.6<br>*4.6* | 14.0<br>*5.0* | 13.3<br>*5.4* | | | | | | | | 1.8 |
| 280 | 16.8<br>*2.7* | 16.0<br>*4.2* | 15.2<br>*5.0* | 14.4<br>*5.6* | | | | | | | | | | |

# TABLE 10-36 (cont'd)
# $\frac{\phi_v V_n}{A_w}$ (ksi) for Plate Girders by Appendix G for 36 ksi Yield Stress Steel, Tension Field Action Included[b]

(Italic values indicate gross area, as percent of ($h \times t_w$) required for pairs of intermediate stiffeners of 36 ksi yield stress steel with $V_u / \phi V_n = 1.0$)[a]

| $\frac{h}{t_w}$ | Aspect ratio *a* / *h*: Stiffener Spacing to Web Depth | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | **0.5** | **0.6** | **0.7** | **0.8** | **0.9** | **1.0** | **1.2** | **1.4** | **1.6** | **1.8** | **2.0** | **2.5** | **3.0** | **Over 3.0 [c]** |
| 300 | 16.6 *3.9* | 15.8 *5.2* | 15.0 *5.9* | | | | | | | | | | | |
| 320 | 16.4 *4.9* | 15.6 *6.0* | | | | | | | | | | | | |

[a] For area of single-angle and single-plate stiffeners, or when $V_u / \phi V_n < 1.0$, see Equation A-G4-1.
[b] For end-panels and all panels in hybrid and web-tapered plate girders, use Table 9-36.
[c] Same as for Table 9-36.
Note: Girders so proportioned that the computed shear is less than that given in right-hand column do not require intermediate stiffeners.

# TABLE 10-50
## $\dfrac{\phi_v V_n}{A_w}$ (ksi) for Plate Girders by Appendix G for 50 ksi Yield Stress Steel, Tension Field Action Included[b]

(Italic values indicate gross area, as percent of $(h \times t_w)$ required for pairs of intermediate stiffeners of 50 ksi yield stress steel with $V_u / \phi V_n = 1.0$)[a]

| | **Aspect ratio *a* / *h*: Stiffener Spacing to Web Depth** | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $\dfrac{h}{t_w}$ | **0.5** | **0.6** | **0.7** | **0.8** | **0.9** | **1.0** | **1.2** | **1.4** | **1.6** | **1.8** | **2.0** | **2.5** | **3.0** | **Over 3.0 [c]** |
| 60 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 26.6 |
| 70 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 26.9 | 26.5 | 26.1 | 25.4 | 24.9 | 22.8 |
| 80 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 27.0 | 26.5 | 25.8 | 25.1 | 24.6 | 24.1 | 23.3 | 22.4 | 18.6 |
| 90 | 27.0 | 27.0 | 27.0 | 27.0 | 26.8 | 26.3 | 25.3 | 24.4 | 23.4 | 22.5 | 21.7 | 20.2 | 19.2 | 14.7 |
| 100 | 27.0 | 27.0 | 27.0 | 26.5 | 25.9 | 25.3 | 24.0 | 22.5 | 21.4 | 20.4 | 19.6 | 18.0 | 17.0 | 11.9 |
| 110 | 27.0 | 27.0 | 26.5 | 25.8 | 25.1 | 24.2 | 22.4 | 21.0 | 19.8 | 18.8 | 18.0 | 16.4 | 15.3 | 9.8 |
| 120 | 27.0 | 26.7 | 25.9 | 25.1 | 24.0 | 23.0 | 21.2 | 19.8 | 18.6 | 17.6 | 16.8 | 15.2 | 14.1 | 8.3 |
| 130 | 27.0 | 26.2 | 25.4 | 24.1 | 23.0 | 22.0 | 20.3 | 18.9 | 17.7 | 16.7 | 15.9 | 14.2 | 13.1 | 7.0 |
| 140 | 26.7 | 25.8 | 24.5 | 23.3 | 22.2 | 21.3 | 19.6 | 18.2 | 17.0 | 16.0 | 15.1 | 13.5 | 12.3 | 6.1 |
| 150 | 26.3 | 25.2 | 23.9 | 22.7 | 21.6 | 20.7 | 19.0 | 17.6 | 16.4 | 15.4 | 14.5 | 12.9 | 11.7 | 5.3 |
| 160 | 26.0 | 24.6 | 23.3 | 22.2 | 21.1 | 20.2 | 18.5 | 17.1 | 15.9<br>*0.2* | 14.9<br>*0.4* | 14.0<br>*0.5* | 12.4<br>*0.8* | | 4.6 |
| 170 | 25.6 | 24.1 | 22.8 | 21.7 | 20.7 | 19.8 | 18.1<br>*0.5* | 16.7<br>*1.0* | 15.2<br>*1.2* | 14.5<br>*1.4* | 13.6<br>*1.6* | | | 4.1 |
| 180 | 25.1 | 23.7 | 22.4 | 21.3 | 20.3<br>*0.4* | 19.4<br>*0.9* | 17.8<br>*1.5* | 16.4<br>*1.9* | 15.2<br>*2.2* | 14.2<br>*2.3* | 13.3<br>*2.5* | | | 3.7 |
| 200 | 24.3 | 23.0 | 21.8<br>*1.0* | 20.8<br>*1.8* | 19.8<br>*2.3* | 18.9<br>*2.7* | 17.3<br>*3.2* | 15.9<br>*3.5* | 14.7<br>*3.7* | | | | | 3.0 |
| 220 | 23.7 | 22.5<br>*1.7* | 21.4<br>*2.7* | 20.4<br>*3.3* | 19.4<br>*3.8* | 18.5<br>*4.1* | 16.9<br>*4.5* | | | | | | | 2.5 |
| 240 | 23.2<br>*1.8* | 22.1<br>*3.2* | 21.0<br>*4.0* | 20.0<br>*4.6* | 19.1<br>*4.9* | 18.2<br>*5.2* | | | | | | | | 2.1 |
| 260 | 23.0<br>*3.2* | 21.8<br>*4.4* | 20.8<br>*5.1* | 19.8<br>*5.6* | 18.8<br>*5.9* | 18.0<br>*6.1* | | | | | | | | |
| 280 | 22.7<br>*4.4* | 21.6<br>*5.4* | 20.6<br>*6.0* | 19.6<br>*6.4* | | | | | | | | | | |

[a] For area of single-angle and single-plate stiffeners, or when $V_u / \phi V_n < 1.0$, see Equation A-G4-1.
[b] For end-panels and all panels in hybrid and web-tapered plate girders, use Table 9-50.
[c] Same as for Table 9-50.
Note: Girders so proportioned that the computed shear is less than that given in right-hand column do not require intermediate stiffeners.

# TABLE 11
# Nominal Horizontal Shear Load for One Connector $Q_n$, kips[a] From Equations I5-1 and I5-2

| Connector [b] | Specified Compressive Strength of Concrete, $f_c'$, ksi [d] | | |
|---|---|---|---|
| | 3.0 | 3.5 | 4.0 |
| ½-in. dia. × 2-in. hooked or headed stud | 9.4 | 10.5 | 11.6 |
| ⅝-in. dia. × 2½-in. hooked or headed stud | 14.6 | 16.4 | 18.1 |
| ¾-in. dia. × 3-in. hooked or headed stud | 21.0 | 23.6 | 26.1 |
| ⅞-in. dia. × 3½-in. hooked or headed stud | 28.6 | 32.1 | 35.5 |
| Channel C3 × 4.1 | 10.2 $L_c$ [c] | 11.5 $L_c$ [c] | 12.7 $L_c$ [c] |
| Channel C4 × 5.4 | 11.1 $L_c$ [c] | 12.4 $L_c$ [c] | 13.8 $L_c$ [c] |
| Channel C5 × 6.7 | 11.9 $L_c$ [c] | 13.3 $L_c$ [c] | 14.7 $L_c$ [c] |

[a] Applicable only to concrete made with ASTM C33 aggregates.
[b] The nominal horizontal loads tabulated may also be used for studs longer than shown.
[c] $L_c$ = length of channel, inches.
[d] $F_u > 0.5(f_c'w)^{0.75}$, $w$ = 145 lbs./cu. ft.

# COMMENTARY on the Load and Resistance Factor Design Specification for Structural Steel Buildings

December 1, 1993

## INTRODUCTION

The Specification is intended to be complete for normal design usage.

The Commentary furnishes background information and references for the benefit of the engineer seeking further understanding of the derivations and limits of the specification.

The Specification and Commentary are intended for use by design professionals with demonstrated engineering competence.

# CHAPTER A

# GENERAL PROVISIONS

## A1. SCOPE

Load and Resistance Factor Design (LRFD) is an improved approach to the design of structural steel for buildings. It involves explicit consideration of limit states, multiple load factors, and resistance factors, and implicit probabilistic determination of reliability. The designation LRFD reflects the concept of factoring both loads and resistance. This type of factoring differs from the AISC allowable stress design (ASD) Specification (AISC, 1989), where only the resistance is divided by a factor of safety (to obtain allowable stress) and from the plastic design portion of that Specification, where only the loads are multiplied by a common load factor. The LRFD method was devised to offer the designer greater flexibility, more rationality, and possible overall economy.

The format of using resistance factors and multiple load factors is not new, as several such design codes are in effect [the ACI-318 Strength Design for Reinforced Concrete (ACI, 1989) and the AASHTO Load Factor Design for Bridges (AASHTO, 1989)]. Nor should the new LRFD method give designs radically different from the older methods, since it was tuned, or "calibrated," to typical representative designs of the earlier methods. The principal new ingredient is the use of a probabilistic mathematical model in the development of the load and resistance factors, which made it possible to give proper weight to the accuracy with which the various loads and resistances can be determined. Also, it provides a rational methodology for transference of test results into design provisions. A more rational design procedure leading to more uniform reliability is the practical result.

## A2. LIMITS OF APPLICABILITY

### 2. Types of Construction

The provisions for these types of construction have been revised to provide for a truer recognition of the actual degree of connection restraint in the structural design. All connections provide some restraint. Depending on the amount of restraint offered, connections are classified as either Type FR or PR. This classification renames the Type I connection of the AISC ASD Specification to Type FR and includes both Type II and Type III of that Specification under a new, more general classification of Type PR.

Just as in the allowable stress design (ASD) provisions, construction utilizing Type FR connections may be designed in LRFD using either elastic or plastic analysis provided the appropriate Specification provisions are satisfied.

For Type PR construction which uses the "simple framing" approach, the restraint of the connection is ignored, provided the given conditions are met. This is no change from the ASD provisions. Where there is evidence of the actual moment rotation capability of a given type of connection, the use of designs incorporating the connection restraint is permitted just as in ASD. The designer should, when incorporating connection restraint into the design, take into account the reduced connection stiffness on the stability of the structure and its effect on the magnitude of second order effects.

## A3. MATERIAL

### 1. Structural Steel

### 1a. ASTM Designations

The grades of structural steel approved for use under the LRFD Specification, covered by ASTM standard specifications, extend to a yield stress of 100 ksi. Some of these ASTM standards specify a minimum yield point, while others specify a minimum yield strength. The term "yield stress" is used in the Specification as a generic term to denote either the yield point or the yield strength.

It is important to be aware of limitations of availability that may exist for some combinations of strength and size. Not all structural section sizes are included in the various material specifications. For example, the 60 ksi yield strength steel in the A572 specification includes plate only up to 1¼-in. in thickness. Another limitation on availability is that even when a product is included in the specifications, it may be only infrequently produced by the mills. Specifying these products may result in procurement delays or require ordering large quantities directly from the producing mills. Consequently, it is prudent to check availability before completing the details of a design.

Properties in the direction of rolling are of principal interest in the design of steel structures. Hence, yield stress as determined by the standard tensile test is the principal mechanical property recognized in the selection of the steels approved for use under the Specification. It must be recognized that other mechanical and physical properties of rolled steel, such as anisotropy, ductility, notch toughness, formability, corrosion resistance, etc., may also be important to the satisfactory performance of a structure.

It is not possible to incorporate in the Commentary adequate information to impart full understanding of all factors which might merit consideration in the selection and specification of materials for unique or especially demanding applications. In such a situation the user of the Specification is advised to make use of reference material contained in the literature on the specific properties of concern and to specify supplementary material production or quality requirements as provided for in ASTM material specifications. One such case is the design of highly restrained welded connections (AISC, 1973). Rolled steel is anisotropic, especially insofar as ductility is concerned; therefore, weld contraction strains in the region of highly restrained welded connections may exceed the capabilities of the material if special attention is not given to material selection, details, workmanship, and inspection.

Another special situation is that of fracture control design for certain types of service conditions (AASHTO, 1989). The relatively warm temperatures of steel in buildings, the essentially static strain rates, the stress intensity, and the number of cycles of full design stress make the probability of fracture in building structures extremely remote. Good workmanship and good design details incorporating joint geometry that avoids severe stress concentrations are generally the most effective means of providing fracture-resistant construction. However, for especially demanding service conditions such as low temperatures with impact loading, the specification of steels with superior notch toughness may be warranted.

### 1c. Heavy Shapes

The web-to-flange intersection and the web center of heavy hot-rolled shapes as well as the interior portions of heavy plates may contain a coarser grain structure and/or lower toughness material than other areas of these products. This is probably caused by ingot segregation, as well as somewhat less deformation during hot rolling, higher finishing temperature, and a slower cooling rate after rolling for these heavy sections. This characteristic is not detrimental to suitability for service for compression members, or for non-welded members.

However, when heavy cross sections are joined by splices or connections using complete-joint penetration welds which extend through the coarser and/or lower notch-tough interior portions, tensile strains induced by weld shrinkage may result in cracking, for example a complete-joint penetration welded connection of a heavy cross section beam to any column section. When members of lesser thickness are joined by complete-joint penetration welds, which induce smaller weld shrinkage strains, to the finer grained and/or more notch-tough surface material of ASTM A6 Group 4 and 5 shapes and heavy built-up cross sections, the potential for cracking is significantly lower, for example a complete penetration groove welded connection of a non-heavy cross-section beam to a heavy cross-section column.

For critical applications such as primary tension members, material should be specified to provide adequate toughness at service temperatures. Because of differences in the strain rate between the Charpy V-Notch (CVN) impact test and the strain rate experienced in actual structures, the CVN test is conducted at a temperature higher than the anticipated service temperature for the structure. The location of the CVN test is shown in Figure C-A3.1.

The toughness requirements of A3.1c are intended only to provide material of reasonable toughness for ordinary service application. For unusual applications and/or low temperature service, more restrictive requirements and/or toughness requirements for other section sizes and thicknesses may be appropriate.

To minimize the potential for fracture, the notch toughness requirements of A3.1c must be used in conjunction with good design and fabrication procedures. Specific requirements are given in Sections J1.5, J1.6, J2.3, and M2.2.

### 3. Bolts, Washers, and Nuts

The ASTM standard for A307 bolts covers two grades of fasteners. Either grade may be used under the LRFD Specification; however, it should be noted that Gr. B is intended for pipe flange bolting and Gr. A is the quality long in use for structural applications.

## 4. Anchor Bolts and Threaded Rods

Since there is a limit on the maximum available length of A325 and A490, the use of these bolts for anchor bolts with design lengths longer than the maximum available lengths has presented problems in the past. The inclusion of A687 material in this Specification allows the use of higher strength material for bolts longer than A325 and A490 bolts. The designer should be aware that pretensioning anchor bolts is not recommended due to relaxation and stress corrosion after pretensioning.

The designer should specify the appropriate thread and SAE fit for threaded rods used as load-carrying members.

## 5. Filler Metal and Flux for Welding

The filler metal specifications issued by the American Welding Society are general specifications which include filler metals suitable for building construction, as well as consumables that would not be suitable for building construction. For example, some electrodes covered by the specifications are specifically limited to single pass applications, while others are restricted to sheet metal applications. Many of the filler metals listed are "low hydrogen", that is, they deposit weld metal with low levels of diffusable hydrogen. Other materials are not. Filler metals listed under the various A5 specifications may or may not have required impact toughness, depending on the specific electrode classification. Notch toughness is generally not critical for weld metal used in building construction. However, on structures subject to dynamic loading, the engineer may require the filler metals used to deliver notch-tough weld deposits. Filler metals may be classified in either the as welded or post weld heat treated (stress relieved) condition. Since most structural applications will not involve stress relief, it is important to utilize filler materials that are classified in conditions similar to those experienced by the actual structure.

When specifying filler metal and/or flux by AWS designation, the applicable standard specifications should be carefully reviewed to assure a complete understanding of the designation reference. This is necessary because the AWS designation systems are not consistent. For example, in the case of electrodes for shielded metal arc welding (AWS A5.1), the first two or three digits indicate the nominal tensile strength classification, in ksi, of the weld metal and the final two digits indicate the type of coating; however, in the case of mild steel

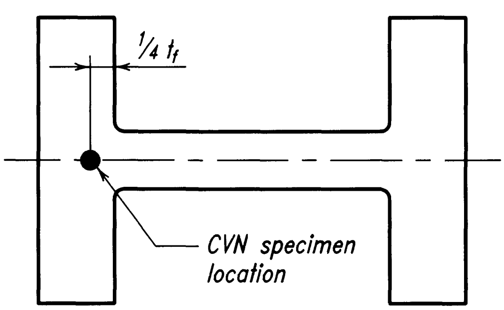



*Fig. C-A3.1. Location from which Charpy impact specimen shall be taken.*

electrodes for submerged arc welding (AWS A5.17), the first one or two digits times 10 indicate the nominal tensile strength classification, while the final digit or digits times 10 indicate the testing temperature in degrees F, for weld metal impact tests. In the case of low-alloy steel covered arc welding electrodes (AWS A5.5), certain portions of the designation indicate a requirement for stress relief, while others indicate no stress relief requirement.

Engineers do not, in general, specify the exact filler metal to be employed on a particular structure. Rather, the decision as to which welding process and which filler metal is to be utilized is usually left with the fabricator. To ensure that the proper filler metals are used, codes restrict the usage of certain filler materials, or impose qualification testing to prove the suitability of the specific electrode.

## A4. LOADS AND LOAD COMBINATIONS

### 1. Loads, Load Factors, and Load Combinations

The load factors and load combinations given in Section A4.1 were developed to be used with the recommended minimum loads given in ASCE 7 *Minimum Design Loads for Buildings and Other Structures* (ASCE, 1988). The load factors and load combinations are developed in Ellingwood et al. (1982). The target reliability indices $\beta$ underlying the load factors are approximately 3.0 for combinations with gravity loads only (dead, snow, and live loads), 2.5 for combinations with wind included, and 1.75 for combinations with earthquake loads. See Commentary A5.3 for definition of $\beta$.

The load factors and load combinations recognize that when several loads act in combination with the dead load (e.g., dead plus live plus wind), only one of these takes on its maximum lifetime value, while the other load is at its "arbitrary point-in-time value" (i.e., at a value which can be expected to be on the structure at any time). For example, under dead, live, and wind loads the following combinations are appropriate:

$$\gamma_D D + \gamma_L L \tag{C-A4-1}$$

$$\gamma_D D + \gamma_{L_a} L_a + \gamma_w W \tag{C-A4-2}$$

$$\gamma_D D + \gamma_L L + \gamma_{w_a} W_a \tag{C-A4-3}$$

where $\gamma$ is the appropriate load factor as designated by the subscript symbol. Subscript $a$ refers to an "arbitrary point-in-time" value.

The mean value of arbitrary point-in-time live load $L_a$ is on the order of 0.24 to 0.4 times the mean maximum lifetime live load $L$ for many occupancies, but its dispersion is far greater. The arbitrary point-in-time wind load $W_a$, acting in conjunction with the maximum lifetime live load, is the maximum daily wind. It turns out that $\gamma_{W_a} W_a$ is a negligible quantity so only two load combinations remain:

$$1.2D + 1.6L \tag{C-A4-4}$$

$$1.2D + 0.5L + 1.3W \tag{C-A4-5}$$

The load factor 0.5 assigned to $L$ in the second equation reflects the statistical properties of $L_a$, but to avoid having to calculate yet another load, it is reduced so it can be combined with the maximum lifetime wind load.

The nominal loads *D, L, W, E,* and *S* are the code loads or the loads given in ASCE 7. The new specified earthquake loads are based on post-elastic energy dissipation in the structure, and are higher than those traditionally specified for allowable stress design (NEHRP, 1992). The new edition of ASCE Standard 7 on structural loads expected to be released in 1993 has adopted the new seismic design recommendations, as has the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 1992). The load factors on *E* in Load Combinations A4-5 and A4-6 have been reduced from 1.5 to 1.0 to be consistent with the specification of earthquake force in these new documents. The reader is referred to the commentaries to these documents for an expanded discussion on seismic loads, load factors, and seismic design of steel buildings.

### 2. Impact

A mass of the total moving load (wheel load) is used as the basis for impact loads on crane runway girders, because maximum impact load results when cranes travel while supporting lifted loads.

The increase in load, in recognition of random impacts, is not required to be applied to supporting columns because the impact load effects (increase in eccentricities or increases in out-of-straightness) will not develop or will be negligible during the short duration of impact. For additional information on crane girder design criteria see AISE Technical Report No. 13.

## A5. DESIGN BASIS

### 1. Required Strength at Factored Loads

LRFD permits the use of both elastic and plastic structural analyses. LRFD provisions result in essentially the same methodology for, and end product of, plastic design as included in the AISC ASD Specification (AISC, 1989), except that the LRFD provisions tend to be more liberal, reflecting added experience and the results of further research. The 10 percent redistribution permitted is consistent with that in the AISC ASD Specification (AISC, 1989).

### 2. Limit States

A limit state is a condition which represents the limit of structural usefulness. Limit states may be dictated by functional requirements, such as maximum deflections or drift; they may be conceptual, such as plastic hinge or mechanism formation; or they may represent the actual collapse of the whole or part of the structure, such as fracture or instability. Design criteria ensure that a limit state is violated only with an acceptably small probability by selecting the load and resistance factors and nominal load and resistance values which will never be exceeded under the design assumptions.

Two kinds of limit states apply for structures: limit states of strength which define safety against the extreme loads during the intended life of the structure, and limit states of serviceability which define the functional requirements. The LRFD Specification, like other structural codes, focuses on the limit states of strength because of overriding considerations of public safety for the life, limb, and property of human beings. This does not mean that limit states of serviceability are not important to the designer, who must equally ensure functional performance and economy of design. However, these latter considerations permit more exercise of judgment on the part of designers. Minimum considerations of public safety, on the other hand, are not matters of individual judgment and, therefore, specifications dwell more on the limit states of strength than on the limit states of serviceability.

Limit states of strength vary from member to member, and several limit states may apply to a given member. The following limit states of strength are the most common: onset of yielding, formation of a plastic hinge, formation of a plastic mechanism, overall frame or member instability, lateral-torsional buckling, local buckling, tensile fracture, development of fatigue cracks, deflection instability, alternating plasticity, and excessive deformation.

The most common serviceability limit states include unacceptable elastic deflections and drift, unacceptable vibrations, and permanent deformations.

## 3. Design for Strength

The general format of the LRFD Specification is given by the formula:

$$\Sigma\gamma_i Q_i \leq \phi R_n \tag{C-A5-1}$$

where

$\Sigma$ = summation
$i$ = type of load, i.e., dead load, live load, wind, etc.
$Q_i$ = nominal load effect
$\gamma_i$ = load factor corresponding to $Q_i$
$\Sigma\gamma_i Q_i$ = required resistance
$R_n$ = nominal resistance
$\phi$ = resistance factor corresponding to $R_n$
$\phi R_n$ = design strength

The left side of Equation C-A5-1 represents the required resistance computed by structural analysis based upon assumed loads, and the right side of Equation C-A5-1 represents a limiting structural capacity provided by the selected members. In LRFD, the designer compares the effect of factored loads to the strength actually provided. The term design strength refers to the resistance or strength $\phi R_n$ that must be provided by the selected member. The load factors $\gamma$ and the resistance factors $\phi$ reflect the fact that loads, load effects (the computed forces and moments in the structural elements), and the resistances can be determined only to imperfect degrees of accuracy. The resistance factor $\phi$ is equal to or less than 1.0 because there is always a chance for the actual resistance to be less than the nominal value $R_n$ computed by the equations given in Chapters D through K. Similarly, the load factors $\gamma$ reflect the fact that the actual load effects may deviate from the nominal values of $Q_i$ computed from the specified nominal loads. These factors account for unavoidable inaccuracies in the theory, variations in the material properties and dimensions, and uncertainties in the determination of loads. They provide a margin of reliability to account for unexpected loads. They do not account for gross error or negligence.

The LRFD Specification is based on (1) probabilistic models of loads and resistance, (2) a calibration of the LRFD criteria to the 1978 edition of the AISC ASD Specification for selected members, and (3) the evaluation of the resulting criteria by judgment and past experience aided by comparative design office studies of representative structures.

The following is a brief probabilistic basis for LRFD (Ravindra and Galambos, 1978, and Ellingwood et al., 1982). The load effects $Q$ and the resistance factor $R$ are assumed to be statistically independent random variables. In Figure C-A5.1, frequency distributions for $Q$ and $R$ are portrayed as separate curves on a common plot for a hypothetical case. As long as the resistance $R$ is greater than (to the right of) the effects of the loads $Q$, a margin of safety for the particular limit state exists. However, because $Q$ and $R$ are random variables, there is some small probability that $R$ may be less than $Q$, ($R < Q$). This limit state probability is related to the degree of overlap of the frequency distributions in Figure C-A5.1, which depends on their relative positioning ($R_m$ *vs.* $Q_m$) and their dispersions.

An equivalent situation may be represented as in Figure C-A5.2. If the expression $R < Q$ is divided by $Q$ and the result expressed logarithmically, the result will be a single frequency distribution curve combining the uncertainties of both $R$ and $Q$. The probability of attaining a limit state ($R < Q$) is equal to the probability that $\ln(R / Q) < 0$ and is represented by the shaded area in the diagram.

The shaded area may be reduced and thus reliability increased in either of two ways: (1) by moving the mean of $\ln(R / Q)$ to the right, or (2) by reducing the spread of the curve for a given position of the mean relative to the origin. A convenient way of combining these two approaches is by defining the position of the mean using the standard deviation of $\ln(R / Q)$, as the unit of measure. Thus, the distance from the origin to the mean is measured as the number of standard deviations of the function $\ln(R / Q)$. As shown in Figure C-A5.2, this

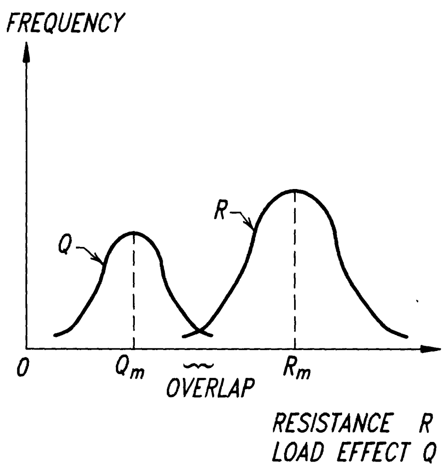



*Fig. C-A5.1. Frequency distribution of load effect Q and resistance R.*

is stated as $\beta$ times $\sigma_{\ln(R/Q)}$, the standard deviation of $\ln(R/Q)$. The factor $\beta$ therefore is called the "reliability index."

If the actual shape of the distribution of $\ln(R/Q)$ were known, and if an acceptable value of the probability of reaching the limit state could be agreed upon, one could establish a completely probability-based set of design criteria. Unfortunately, this much information frequently is not known. The distribution shape of each of the many variables (material, loads, etc.) has an influence on the shape of the distribution of $\ln(R/Q)$. Often only the means and the standard deviations of the many variables involved in the makeup of the resistance and the load effect can be estimated. However, this information is enough to build an approximate design criterion which is independent of the knowledge of the distribution, by stipulating the following design condition:

$$\beta\sigma_{\ln(R/Q)} \approx \beta\sqrt{V_R^2 + V_Q^2} \leq \ln(R_m / Q_m) \qquad \text{(C-A5-2)}$$

In this formula, the standard deviation has been replaced by the approximation $\sqrt{V_R^2 + V_Q^2}$, where $V_R = \sigma_R / R_m$ and $V_Q = \sigma_Q / Q_m$ ($\sigma_R$ and $\sigma_Q$ are the standard deviations, $R_m$ and $Q_m$ are the mean values, $V_R$ and $V_Q$ are the coefficients of variation, respectively, of the resistance $R$ and the load effect $Q$). For structural elements and the usual loadings $R_m$, $Q_m$, and the coefficients of variation, $V_R$ and $V_Q$, can be estimated, so a calculation of

$$\beta = \frac{\ln(R_m / Q_m)}{\sqrt{V_R^2 + V_Q^2}} \qquad \text{(C-A5-3)}$$

will give a comparative value of the measure of reliability of a structure or component.

The description of the determination of $\beta$ as given above is a simple way of defining the probabilistic method used in the development of LRFD. A more refined method, which can accommodate more complex design situations (such as the beam-column interaction equation) and include probabilistic distributions other than the lognormal distribution used to derive Equation C-A5-3, has been developed since the publication of Ravindra and Galambos (1978), and is fully described in Galambos, et al. (1982). This latter method has been used in the development of the recommended load factors (see Section A4). The two methods give essentially the same $\beta$ values for most steel structural members and connections.

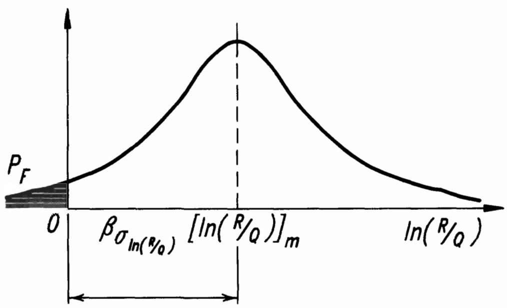



*Fig. C-A5.2. Definition of reliability index.*

Statistical properties (mean values and coefficients of variations) are presented for the basic material properties and for steel beams, columns, composite beams, plate girders, beam-columns, and connection elements in a series of eight articles in the September 1978 issue of the *Journal of the Structural Division of ASCE* (Vol. 104, ST9). The corresponding load statistics are given in Galambos, et al. (1982). Based on these statistics, the values of $\beta$ inherent in the 1978 edition of the AISC ASD Specification were evaluated under different load combinations (live/dead, wind/dead, etc.), and for various tributary areas for typical members (beams, columns, beam-columns, structural components, etc.). As might be expected, there was a considerable variation in the range of $\beta$ values. Examination of the many $\beta$ values associated with ASD revealed certain trends. For example, compact rolled beams (flexure) and tension members (yielding) had $\beta$ values that decreased from about 3.1 at $L/D = 0.50$ to 2.4 at $L/D = 4$. This decrease is a result of ASD applying the same factor to dead load, which is relatively predictable, and live load, which is more variable. For bolted or welded connections, $\beta$ was on the order of 4 to 5. Reliability indices for load combinations involving wind and earthquake loads tended to be lower. Based on a thorough assessment of implied reliabilities in existing acceptable design practice, common load factors for various structural materials (steel, reinforced concrete, etc.) were developed in Ellingwood et al. (1982).

One of the features of the probability-based method used in the development of LRFD is that the variations of $\beta$ values can be reduced by specifying several "target" $\beta$ values and selecting multiple load and resistance factors to meet these targets. The Committee on Specifications set the point at which LRFD is calibrated to ASD at $L/D = 3.0$ for braced compact beams in flexure and tension members at yield. The resistance factor, $\phi$, for these limit states is 0.90, and the implied $\beta$ is approximately 2.6 for members and 4.0 for connections; this larger $\beta$ value for connections reflects the fact that connections are expected to be stronger than the members that they connect. Limit states for other members are handled consistently.

Computer methods as well as charts are given in Ellingwood et al. (1982) for the use of specification writers to determine the resistance factors $\phi$. These factors can also be approximately determined by the following:

$$\phi = (R_m / R_n) \exp(-0.55\beta V_r) \qquad \text{(C-A5-4)*}$$

where

$R_m$ = mean resistance
$R_n$ = nominal resistance according to the equations in Chapters D through K
$V_r$ = coefficient of variation of the resistance

## 4. Design for Serviceability and Other Considerations

Nominally, serviceability should be checked at the unfactored loads. For combinations of gravity and wind or seismic loads some additional reduction factor may be warranted.

* Note that exp $(x)$ is identical to the more familiar $e^x$.

# CHAPTER B

# DESIGN REQUIREMENTS

## B2. NET AREA

Critical net area is based on net width and load transfer at a particular chain.

## B3. EFFECTIVE NET AREA FOR TENSION MEMBERS

Section B3 deals with the effect of shear lag, which is applicable to both welded and bolted tension members. The reduction coefficient $U$ is applied to the net area $A_n$ of bolted members and to the gross area $A_g$ of welded members. As the length of connection $l$ is increased, the shear lag effect is diminished. This concept is expressed empirically by Equation B3-3. Munse and Chesson (1963) have shown, using this expression to compute an effective net area, with few exceptions, the estimated strength of some 1,000 bolted and riveted connection test specimens correlated with observed test results within a scatterband of ±10 percent. Newer research (Easterling and Gonzales, 1993) provides further justification for current provisions.

For any given profile and connected elements, $\bar{x}$ is a fixed geometric property. It is illustrated as the distance from the connection plane, or face of the member, to the centroid of the member section resisting the connection force. See Figure C-B3.1. Length $l$ is dependent upon the number of fasteners or equivalent length of weld required to develop the given tensile force, and this in turn is dependent upon the mechanical properties of the member and the capacity of the fasteners or weld used. The length $l$ is illustrated as the distance, parallel to the line of force, between the first and last fasteners in a line for bolted connections. The number of bolts in a line, for the purpose of the determination of $l$, is determined by the line with the maximum number of bolts in the connection. For staggered bolts, the out-to-out dimension is used for $l$. See Figure C-B3.2. There is insufficient data to establish a value of $U$ if all lines have only one bolt, but it is probably conservative to use $A_e$ equal to the net area of the connected element. For welded connections, $l$ is the length of the member parallel to the line of force that is welded. For combinations of longitudinal and transverse welds (see Figure C-B3.3), $l$ is the length of longitudinal weld because the transverse weld has little or no effect on the shear lag problem, i.e., it does little to get the load into the unattached portions of the member.

Previous issues of this Specification have presented values for $U$ for bolted or riveted connections of W, M, and S shapes, tees cut from these shapes, and other shapes. These values are acceptable for use in lieu of calculated values from Equation B3-3 and are retained here for the convenience of designers.

For bolted or riveted connections the following values of $U$ may be used:

(a) W, M, or S shapes with flange widths not less than two-thirds the depth, and structural tees cut from these shapes, provided the connection is to the flanges and has no fewer than three fasteners per line in the direction of stress, $U = 0.90$

(b) W, M, or S shapes not meeting the conditions of subparagraph a, structural tees cut from these shapes, and all other shapes including built-up cross

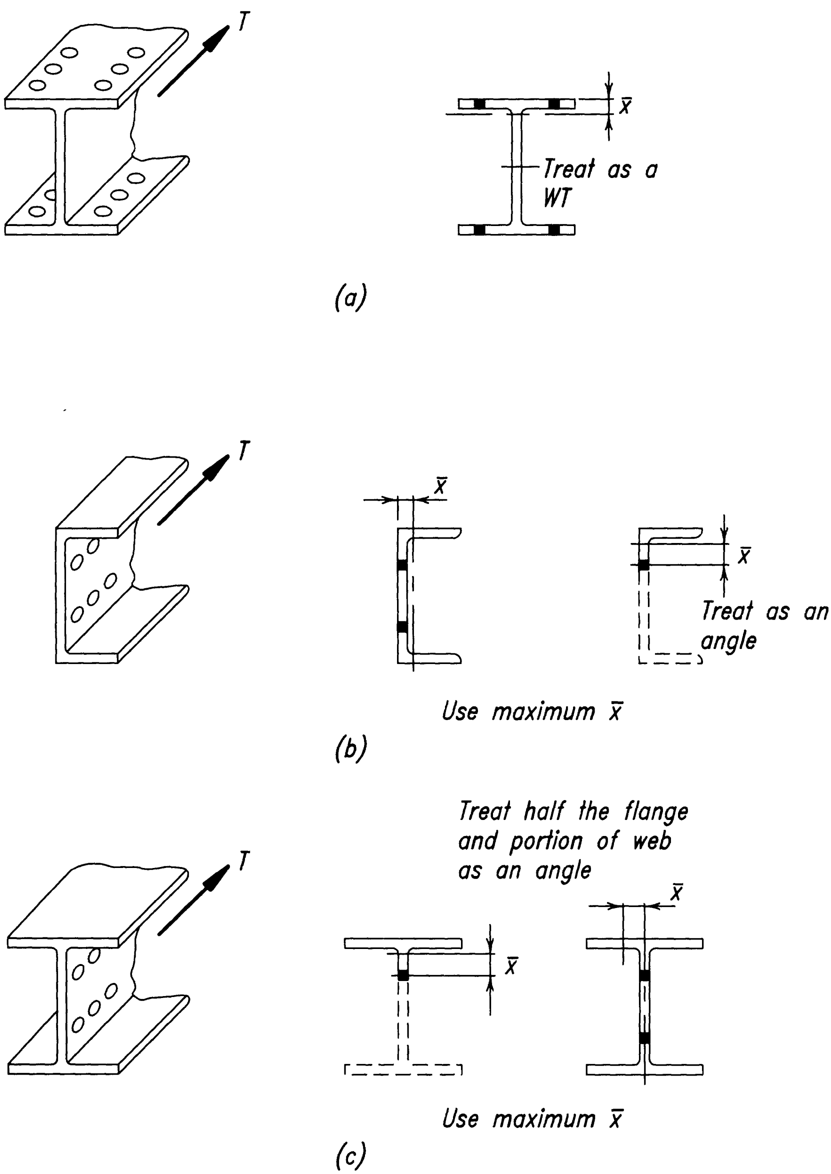



*Fig. C-B3.1. Determination of $\bar{x}$ for U.*

sections, provided the connection has no fewer than three fasteners per line in the direction of stress, $U = 0.85$

(c) All members having only two fasteners per line in the direction of stress, $U = 0.75$

When a tension load is transmitted by fillet welds to some but not all elements of a cross section, the weld strength will control.

## B5. LOCAL BUCKLING

For the purposes of this Specification, steel sections are divided into compact sections, noncompact sections, and sections with slender compression elements. Compact sections are capable of developing a *fully plastic* stress distribution and they possess a rotational capacity of approximately 3 before the onset of local buckling (Yura et al., 1978). Noncompact sections can develop the yield stress in compression elements before local buckling occurs, but will not resist inelastic local buckling at the strain levels required for a fully plastic stress distribution. Slender compression elements buckle elastically before the yield stress is achieved.

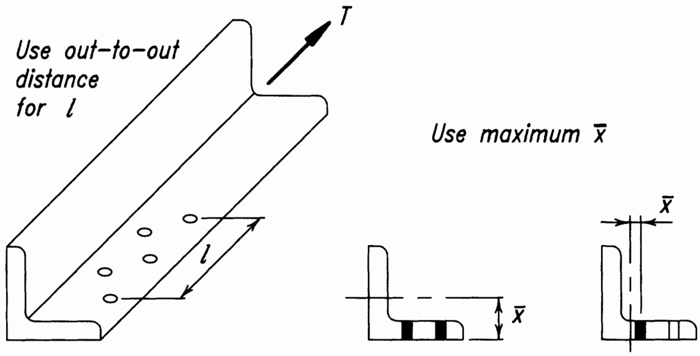



*Fig. C-B3.2. Staggered holes.*

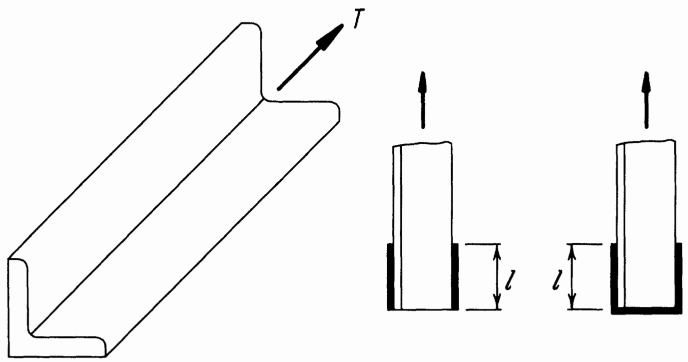



*Fig. C-B3.3. Longitudinal and transverse welds.*

# TABLE C-B5.1
## Limiting Width-Thickness Ratios for Compression Elements

| Description of Element | Width-Thickness Ratio | Limiting Width-thickness Ratios $\lambda_p$ | |
|---|---|---|---|
| | | Non-seismic | Seismic |
| Flanges of I-shaped sections (including hybrid sections) and channels in flexure [a] | $b/t$ | $65/\sqrt{F_y}$ | $52/\sqrt{F_y}$ |
| Webs in combined flexural and axial compression | $h/t_w$ | For $P_u/\phi_b P_y \leq 0.125$ | |
| | | $\frac{640}{\sqrt{F_y}}\left(1-\frac{2.75P_u}{\phi_b P_y}\right)$ | $\frac{520}{\sqrt{F_y}}\left(1-\frac{1.54P_u}{\phi_b P_y}\right)$ |
| | | For $P_u/\phi_b P_y > 0.125$ | |
| | | $\frac{191}{\sqrt{F_y}}\left(2.33-\frac{P_u}{\phi_b P_y}\right) \geq \frac{253}{\sqrt{F_y}}$ | |
| [a] For hybrid beams use $F_{yf}$ in place of $F_y$. | | | |

The dividing line between compact and noncompact sections is the limiting width-thickness ratio $\lambda_p$. For a section to be compact, all of its compression elements must have width-thickness ratios smaller than the limiting $\lambda_p$.

A greater inelastic rotation capacity than provided by the limiting values $\lambda_p$ given in Table C-B5.1 may be required for some structures in areas of high seismicity. It has been suggested that in order to develop a ductility of from 3 to 5 in a structural member, ductility factors for elements would have to lie in the range of 5 to 15. Thus, in this case it is prudent to provide for an inelastic rotation of 7 to 9 times the elastic rotation (Chopra and Newmark, 1980). In order to provide for this rotation capacity, the limits $\lambda_p$ for local flange and web buckling would be as shown in Table C-B5.1 (Galambos, 1976).

More information on seismic design is contained in the AISC *Seismic Provisions for Structural Steel Buildings.*

Another limiting width-thickness ratio is $\lambda_r$, representing the distinction between noncompact sections and sections with slender compression elements. As long as the width-thickness ratio of a compression element does not exceed the limiting value $\lambda_r$, local elastic buckling will not govern its strength. However, for those cases where the width-thickness ratios exceed $\lambda_r$, elastic buckling strength must be considered. A design procedure for such slender-element compression sections, based on elastic buckling of plates, is given in Appendix B5.3. The effective width Equation A-B5-12 applies strictly to stiffened elements under uniform compression. It does not apply to cases where the compression element is under stress gradient. A method of dealing with the stress gradient in a compression element is provided in Section B2 of the AISI *Design Specifications for Cold-Formed Steel Structural Members,* 1986 and Addendum, 1989. An exception is plate girders with slender webs. Such plate girders are capable of developing postbuckling strength in excess of the elastic buckling load. A design procedure for plate girders including tension field action is given in Appendix G.

The values of the limiting ratios $\lambda_p$ and $\lambda_r$ specified in Table B5.1 are similar to those in AISC (1989) and Table 2.3.3.3 of Galambos (1976), except that: (1) $\lambda_p = 65 / \sqrt{F_y}$, limited in Galambos (1976) to indeterminate beams when moments are determined by elastic analysis and to determinate beams, was adopted for all conditions on the basis of Yura et al. (1978); and (2) $\lambda_p = 1{,}300 / F_y$ for circular hollow sections was obtained from Sherman (1976).

The high shape factor for circular hollow sections makes it impractical to use the same slenderness limits to define the regions of behavior for different types of loading. In Table B5.1, the values of $\lambda_p$ for a compact shape that can achieve the plastic moment, and $\lambda_r$ for bending, are based on an analysis of test data from several projects involving the bending of pipes in a region of constant moment (Sherman and Tanavde, 1984 and Galambos, 1988). The same analysis produced the equation for the inelastic moment capacity in Table A-F1.1 in Appendix F1. However, a more restrictive value of $\lambda_p$ is required to prevent inelastic local buckling from limiting the plastic hinge rotation capacity needed to develop a mechanism in a circular hollow beam section (Sherman, 1976).

The values of $\lambda_r$ for axial compression and for bending are both based on test data. The former value has been used in building specifications since 1968 (Winter, 1970). Appendices B5 and F1 also limit the diameter-to-thickness ratio for any circular section to $13{,}000 / F_y$. Beyond this, the local buckling strength decreases rapidly, making it impractical to use these sections in building construction.

Following the SSRC recommendations (Galambos, 1988) and the approach used for other shapes with slender compression elements, a $Q$ factor is used for circular sections to account for interaction between local and column buckling. The $Q$ factor is the ratio between the local buckling stress and the yield stress. The local buckling stress for the circular section is taken from the inelastic AISI criteria (Winter, 1970) and is based on tests conducted on fabricated and manufactured cylinders. Subsequent tests on fabricated cylinders (Galambos, 1988) confirm that this equation is conservative.

The definitions of the width and thickness of compression elements agree with the 1978 AISC ASD Specification with minor modifications. Their applicability extends to sections formed by bending and to unsymmetrical and hybrid sections.

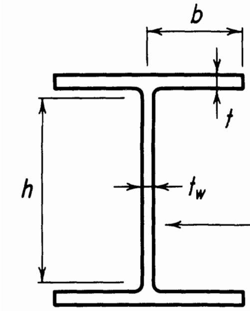
For built-up I-shaped sections under axial compression, modifications have been made to the flange local buckling criterion to include web-flange interaction. The $k_c$ in the $\lambda_r$ limit, in Equations A-B5-7 and A-B5-8 and the elastic buckling Equation A-B5-8 are the same that are used for flexural members. Theory indicates that the web-flange interaction in axial compression is at least as severe as in flexure. Rolled shapes are excluded from this criteria because there are no standard sections with proportions where the interaction would occur. In built-up sections where the interaction causes a reduction in the flange local buckling strength, it is likely that the web is also a thin stiffened element.

The $k_c$ factor accounts for the interaction of flange and web local buckling demonstrated in experiments conducted by Johnson (1985). The maximum limit of 0.763 corresponds to $F_{cr} = 20{,}000 / \lambda^2$ which was used as the local buckling strength in earlier editions of both the ASD and LRFD Specifications. An $h / t_w = 27.5$ is required to reach $k_c = 0.763$. Fully fixed restraint for an unstiffened compression element corresponds to $k_c = 1.3$ while zero restraint gives $k_c = 0.42$. Because of web-flange interactions it is possible to get $k_c < 0.42$ from the new $k_c$ formula. If $h / t_w > 970 / \sqrt{F_y}$ use $h / t_w = 970 / \sqrt{F_y}$ in the $k_c$ equation, which corresponds to the 0.35 limit.

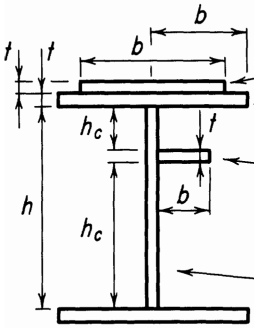
Illustrations of some of the requirements of Table B5.1 are shown in Figure C-B5.1.

## B7. LIMITING SLENDERNESS RATIOS

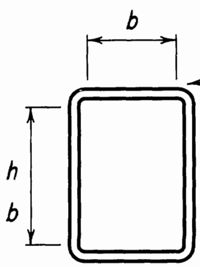
Chapters D and E provide reliable criteria for resistance of axially loaded members based on theory and confirmed by test for all significant parameters including slenderness. The advisory upper limits on slenderness contained in Section B7 are based on professional judgment and practical considerations of economics, ease of handling, and care required to minimize inadvertent damage during fabrication, transport, and erection. Out-of-straightness within reasonable tolerances does not affect the strength of tension members, and the effect of out-of-straightness within specified tolerances on the strength of compression members is accounted for in formulas for resistance. Applied tension tends to reduce, whereas compression tends to amplify, out-of-straightness. Therefore, more liberal criteria are suggested for tension members, including those subject to small compressive forces resulting from transient loads such as earthquake and wind. For members with slenderness ratios greater than 200, these compressive forces correspond to stresses less than 2.6 ksi.



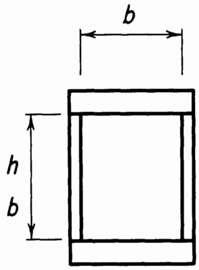

| BENDING | AXIAL COMPRESSION |
|---|---|
| $\lambda_p = \frac{65}{\sqrt{F_y}}$, $\lambda_r = \frac{141}{\sqrt{F_y - 10}}$ | $\lambda_r = \frac{95}{\sqrt{F_y}}$ |
| $\lambda_p = \frac{640}{\sqrt{F_y}}$, $\lambda_r = \frac{970}{\sqrt{F_y}}$ | $\lambda_r = \frac{253}{\sqrt{F_y}}$ |




| BENDING | AXIAL COMPRESSION |
|---|---|
| (perforated $\lambda_r = \frac{317}{\sqrt{F_y}}$ | $\lambda_r = \frac{317}{\sqrt{F_y}}$ ) |
| $\lambda_p = \frac{190}{\sqrt{F_y}}$, $\lambda_r = \frac{238}{\sqrt{F_y}}$ | $\lambda_r = \frac{238}{\sqrt{F_y}}$ |
| $\lambda_p = \frac{65}{\sqrt{F_y}}$, $\lambda_r = \frac{162}{\sqrt{(F_{yw} - 16.5)/k_c}}$ | $\lambda_r = \frac{109}{\sqrt{F_y/k_c}}$ |
| $\lambda_r = \frac{95}{\sqrt{F_y}}$ | $\lambda_r = \frac{109}{\sqrt{F_y/k_c}}$ |
| $\lambda_p = \frac{640}{\sqrt{F_y}}$, $\lambda_r = \frac{970}{\sqrt{F_y}}$ | $\lambda_r = \frac{253}{\sqrt{F_y}}$ |




| BENDING | AXIAL COMPRESSION |
|---|---|
| $\lambda_p = \frac{190}{\sqrt{F_y}}$, $\lambda_r = \frac{238}{\sqrt{F_y}}$ | $\lambda_r = \frac{238}{\sqrt{F_y}}$ |
| $\lambda_p = \frac{640}{\sqrt{F_y}}$, $\lambda_r = \frac{970}{\sqrt{F_u}}$ | $\lambda_r = \frac{238}{\sqrt{F_y}}$ |




| BENDING | AXIAL COMPRESSION |
|---|---|
| $\lambda_r = \frac{253}{\sqrt{F_y}}$ | $\lambda_r = \frac{253}{\sqrt{F_y}}$ |
| $\lambda_r = \frac{970}{\sqrt{F_y}}$ | $\lambda_r = \frac{253}{\sqrt{F_y}}$ |

*Fig. C-B5.1. Selected examples of Table B5.1 requirements.*

# CHAPTER C

# FRAMES AND OTHER STRUCTURES

## C1. SECOND ORDER EFFECTS

While resistance to wind and seismic loading can be provided in certain buildings by means of shear walls, which also provide for overall frame stability at factored gravity loading, other building frames must provide this resistance by frame action. This resistance can be achieved in several ways, e.g., by a system of bracing, by a moment-resisting frame, or by any combination of lateral force-resisting elements.

For frames under combined gravity and lateral loads, drift (horizontal deflection caused by applied loads) occurs at the start of loading. At a given value of the applied loads, the frame has a definite amount of drift $\Delta$. In unbraced frames, additional secondary bending moments, known as the $P\Delta$ moments, may be developed in the columns and beams of the lateral load-resisting systems in each story. $P$ is the total gravity load above the story and $\Delta$ is the story drift. As the applied load increases, the $P\Delta$ moments also increase. Therefore, the $P\Delta$ effect must often be accounted for in frame design. Similarly, in braced frames, increases in axial forces occur in the members of the bracing systems; however, such effects are usually less significant. The designer should consider these effects for all types of frames and determine if they are significant. Since $P\Delta$ effects can cause frame drifts to be larger than those calculated by ignoring them, they should also be included in the service load drift analysis when they are significant.

In unbraced frames designed by plastic analysis, the limit of $0.75\phi_c P_y$ on column axial loads has also been retained to help ensure stability.

The designer may use second-order elastic analysis to compute the maximum factored forces and moments in a member. These represent the required strength. Alternatively, for structures designed on the basis of elastic analysis, the designer may use first order analysis and the amplification factors $B_1$ and $B_2$.

In the general case, a member may have first order moments not associated with sidesway which are multiplied by $B_1$, and first order moments produced by forces causing sidesway which are multiplied by $B_2$.

The factor $B_2$ applies only to moments caused by forces producing sidesway and is calculated for an entire story. In building frames designed to limit $\Delta_{oh} / L$ to a predetermined value, the factor $B_2$ may be found in advance of designing individual members.

Drift limits may also be set for design of various categories of buildings so that the effect of secondary bending can be insignificant (Kanchanalai and Lu, 1979;

ATC, 1978). It is conservative to use the $B_2$ factor with the sum of the sway and the no-sway moments, i.e., with $M_{lt} + M_{nt}$.

The two kinds of first order moment $M_{nt}$ and $M_{lt}$ may both occur in sidesway frames from gravity loads. $M_{nt}$ is defined as a moment developed in a member with frame sidesway prevented. If a significant restraining force is necessary to prevent sidesway of an unsymmetrical structure (or an unsymmetrically loaded symmetrical structure), the moments induced by releasing the restraining force will be $M_{lt}$ moments, to be multiplied by $B_2$. In most reasonably symmetric frames, this effect will be small. If such a moment $B_2M_{lt}$ is added algebraically to the $B_1M_{nt}$ moment developed with sidesway prevented, a fairly accurate value of $M_u$ will result. End moments produced in sidesway frames by lateral loads from wind or earthquake will always be $M_{lt}$ moments to be multiplied by $B_2$.

When first order end moments in members subjected to axial compression are magnified by $B_1$ and $B_2$ factors, equilibrium requires that they be balanced by moments in connected members (Figure C-C1.1). This can generally be accomplished satisfactorily by distributing the difference between the magnified moment and the first order moment to any other moment-resisting members attached to the compressed member (or members) in proportion to the relative stiffness of the uncompressed members. Minor imbalances may be neglected in the judgment of the engineer. However, complex conditions, such as occur when there is significant magnification in several members meeting at a joint, may require a second order elastic analysis. Connections shall also be designed to resist the magnified end moments.

For compression members in braced frames, $B_1$ is determined from $C_m$ values which are similar to the values in the AISC ASD Specification. A significant

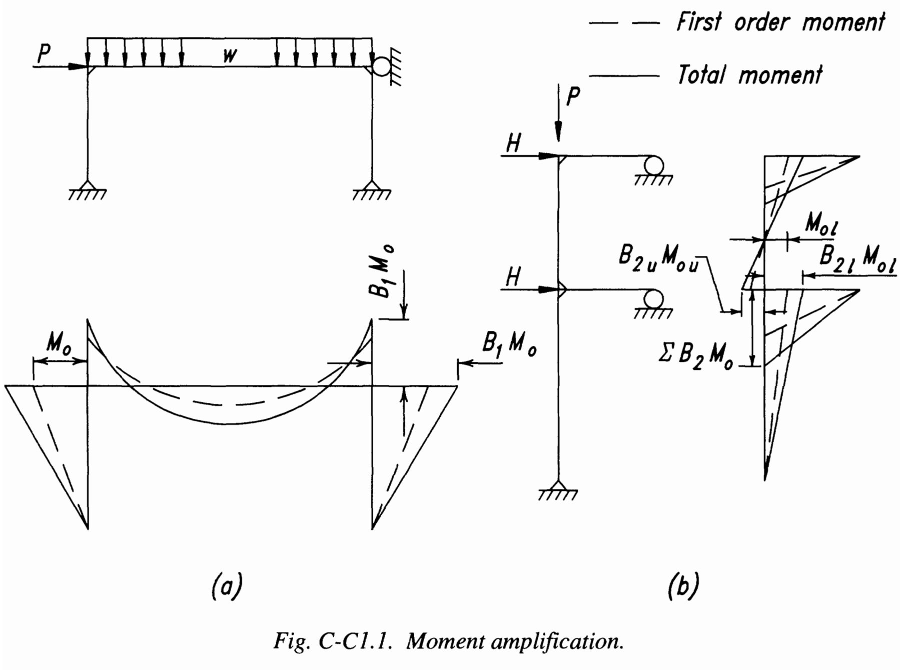



*Fig. C-C1.1. Moment amplification.*

difference, however, is that $B_1$ is never less than 1. When $C_m = 1$ for a compression member loaded between its supports, the factors of $^8\!/_9$ and $^1\!/_2$ make the new equations more liberal than Equation H1-1 of the AISC ASD Specification. For $C_m \leq 1$ (for members with unequal end moments), the new equations will be slightly more conservative than the AISC ASD Specification for a very slender member with low $C_m$. For the entire range of $l / r$ and $C_m$, the equations compare very closely to exact inelastic solutions of braced members.

The center-to-center member length is usually used in the structural analysis. In braced and unbraced frames, $P_n$ is governed by the maximum slenderness ratio regardless of the plane of bending. However, $P_{e1}$ and $P_{e2}$ are always calculated using the slenderness ratio in the plane of bending. Thus, when flexure is about the strong axis only, two different values of slenderness ratio may be involved in solving a given problem.

When second order analysis is used, it must account for the interaction of the factored load effects, that is, combinations of factored loads must be used in analysis. Superposition of forces obtained from separate analyses is not adequate.

When bending occurs about both the x and the y axes, the required flexural strength calculated about each axis is adjusted by the value of $C_m$ and $P_{e1}$ or $P_{e2}$ corresponding to the distribution of moment and the slenderness ratio in its plane of bending, and is then taken as a fraction of the design bending strength, $\phi_b M_n$, about that axis, with due regard to the unbraced length of the compression flange where this is a factor.

Equations C1-2 and C1-3 approximate the maximum second order moments in compression members with no relative joint translation and no transverse loads between the ends of the member. This approximation is compared to an exact solution (Ketter, 1961) in Figure C-C1.2. For single curvature, Equation C1-3 is slightly unconservative, for a zero end moment it is almost exact, and for double curvature it is conservative. The 1978 AISC ASD Specification imposed the limit $C_m \geq 0.4$ which corresponds to a $M_1 / M_2$ ratio of 0.5. However, Figure C-C1.2 shows that if, for example, $M_1 / M_2 = 0.8$, the $C_m = 0.28$ is already very conservative, so the limit has been removed. The limit was originally adopted from Austin (1961), which was intended to apply to lateral-torsional buckling, not second-order in-plane bending strength. The AISC Specifications, both in the 1989 ASD and LRFD, use a modification factor $C_b$ as given in Equation F1-3 for lateral-torsional buckling. $C_b$ is approximately the inverse of $C_m$ as presented in Austin (1961) with a 0.4 limit. In Zandonini (1985) it was pointed out that Equation C1-3 could be used for in-plane second order moments if the 0.4 limit was eliminated. Unfortunately, Austin (1961) was misinterpreted and a lateral-torsional buckling solution was used for an in-plane second-order analysis. This oversight has now been corrected.

For beam columns with transverse loadings, the second-order moment can be approximated by using the following equation

$$C_m = 1 + \psi P_u / P_{e1}$$

For simply supported members

where

$$\psi = \frac{\pi^2 \delta_0 EI}{M_0 L^2} - 1$$

$\delta_0$ = maximum deflection due to transverse loading, in.

$M_0$ = maximum factored design moment between supports due to transverse loading, kip-in.

For restrained ends some limiting cases (Iwankiw, 1984) are given in Table C-C1.1 together with two cases of simply supported beam-columns. These values of $C_m$ are always used with the maximum moment in the member. For the restrained-end cases, the values of $B_1$ will be most accurate if values of $K < 1.0$ corresponding to the end boundary conditions are used in calculating $P_{e1}$. In lieu of using the equations above, $C_m = 1.0$ can be used conservatively for transversely loaded members with unrestrained ends and 0.85 for restrained ends.

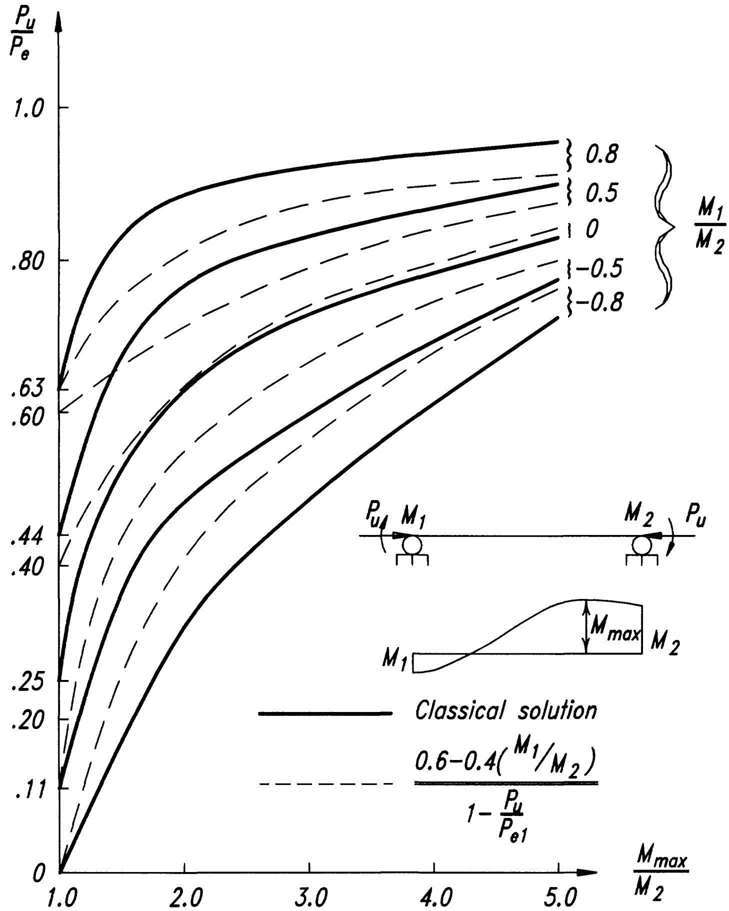
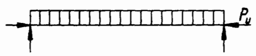
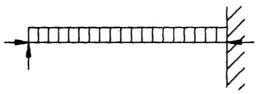
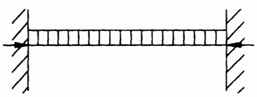
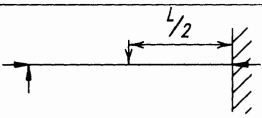



*Fig. C-C1.2. Second-order moments for braced beam-column.*

## TABLE C-C1.1
## Amplification Factors $\psi$ and $C_m$

| Case | $\psi$ | $C_m$ |
|---|---|---|
|  | 0 | 1.0 |
|  | –0.4 | $1 - 0.4 \dfrac{P_u}{P_{e1}}$ |
|  | –0.4 | $1 - 0.4 \dfrac{P_u}{P_{e1}}$ |
|  | –0.2 | $1 - 0.2 \dfrac{P_u}{P_{e1}}$ |
|  | –0.3 | $1 - 0.3 \dfrac{P_u}{P_{e1}}$ |
|  | –0.2 | $1 - 0.2 \dfrac{P_u}{P_{e1}}$ |

If, as in the case of a derrick boom, a beam-column is subject to transverse (gravity) load and a calculable amount of end moment, the value $\delta_0$ should include the deflection between supports produced by this moment.

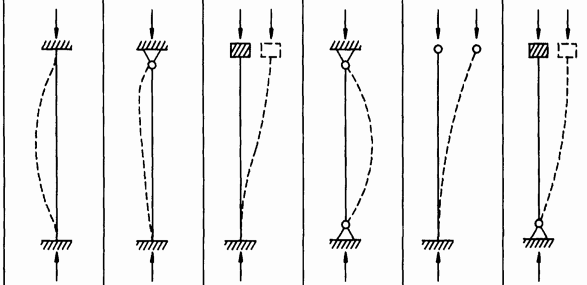
Stiffness reduction adjustment due to column inelasticity is permitted.

## C2. FRAME STABILITY

The stability of structures must be considered from the standpoint of the structure as a whole, including not only the compression members, but also the beams, bracing system, and connections. The stability of individual elements must also be provided. Considerable attention has been given in the technical literature to this subject, and various methods of analysis are available to assure stability. The SSRC *Guide to Stability Design Criteria for Metal Structures* (Galambos, 1988) devotes several chapters to the stability of different types of members considered as individual elements, and then considers the effects of individual elements on the stability of the structure as a whole.

The effective length concept is one method of estimating the interaction effects of the total frame on a compression element being considered. This concept uses

## TABLE C-C2.1
## *K* Values for Columns

| | (a) | (b) | (c) | (d) | (e) | (f) |
|---|---|---|---|---|---|---|
| Buckled shape of column is shown by dashed line. |  | | | | | |
| Theoretical *K* value | 0.5 | 0.7 | 1.0 | 1.0 | 2.0 | 2.0 |
| Recommended design value when ideal conditions are approximated | 0.65 | 0.80 | 1.2 | 1.0 | 2.10 | 2.0 |
| End condition code |  *Rotation fixed and translation fixed*<br>*Rotation free and translation fixed*<br>*Rotation fixed and translation free*<br>*Rotation free and translation free* | | | | | |

*K* factors to equate the strength of a framed compression element of length *L* to an equivalent pin-ended member of length *KL* subject to axial load only. Other rational methods are available for evaluating the stability of frames subject to gravity and side loading and individual compression members subject to axial load and moments. However, the effective-length concept is the only tool currently available for handling several cases which occur in practically all structures, and it is an essential part of many analysis procedures. Although the concept is completely valid for ideal structures, its practical implementation involves several assumptions of idealized conditions which will be mentioned later.

Two conditions, opposite in their effect upon column strength under axial loading, must be considered. If enough axial load is applied to the columns in an unbraced frame dependent entirely on its own bending stiffness for resistance to lateral deflection of the tops of the columns with respect to their bases (see Figure C-C2.1), the effective length of these columns will exceed the actual length. On the other hand, if the same frame were braced to resist such lateral movement, the effective length would be less than the actual length, due to the restraint (resistance to joint translation) provided by the bracing or other lateral support. The ratio *K*, effective column length to actual unbraced length, may be greater or less than 1.0.

The theoretical *K* values for six idealized conditions in which joint rotation and translation are either fully realized or nonexistent are tabulated in Table C-C2.1.

Also shown are suggested design values recommended by the Structural Stability Research Council (formerly the Column Research Council) for use when these conditions are approximated in actual design. In general, these suggested values are slightly higher than their theoretical equivalents, since joint fixity is seldom fully realized.

If the column base in Case f of Table C-C2.1 were truly pinned, $K$ would actually exceed 2.0 for a frame such as that pictured in Figure C-C2.1, because the flexibility of the horizontal member would prevent realization of full fixity at the top of the column. On the other hand, it has been shown (Galambos, 1960) that the restraining influence of foundations, even where these footings are designed only for vertical load, can be very substantial in the case of flat-ended column base details with ordinary anchorage. For this condition, a design $K$ value of 1.5 would generally be conservative in Case f.

While in some cases masonry walls provide enough lateral support for building frames to control lateral deflection, light curtain wall construction and wide column spacing can create a situation where only the bending stiffness of the frame provides this support. In this case the effective length factor $K$ for an unbraced length of column $L$ is dependent upon the bending stiffness provided by the other in-plane members entering the joint at each end of the unbraced segment. If the combined stiffness provided by the beams is sufficiently small, relative to that of the unbraced column segments, $KL$ could exceed two or more story heights (Bleich, 1952).

Several rational methods are available to estimate the effective length of the columns in an unbraced frame with sufficient accuracy. These range from simple interpolation between the idealized cases shown in Table C-C2.1 to very complex analytical procedures. Once a trial selection of framing members has been made, the use of the alignment chart in Figure C-C2.2 affords a fairly rapid

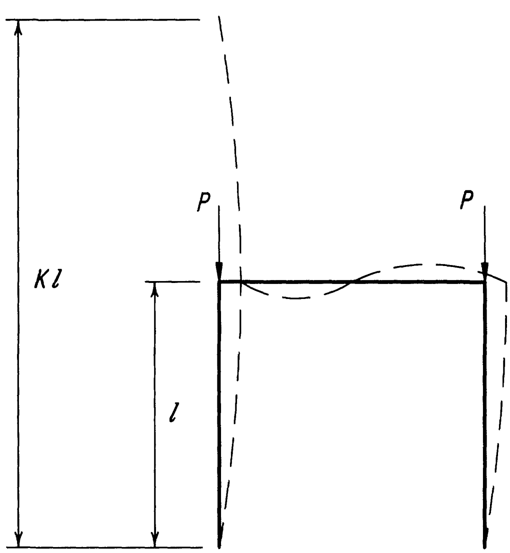



*Fig. C-C2.1. Column effective length.*

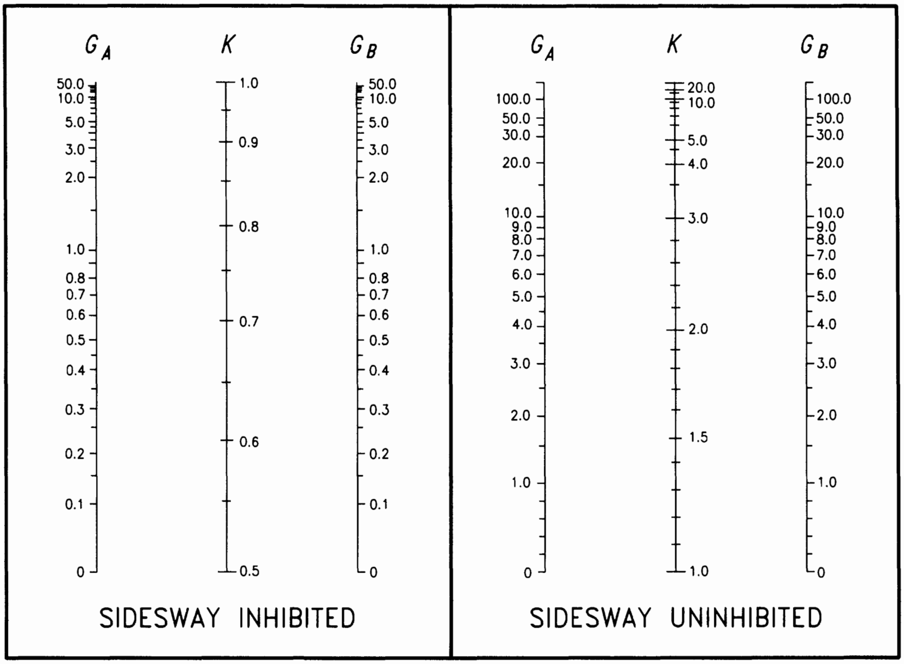
method for determining adequate $K$ values. However, it should be noted that this alignment chart is based upon assumptions of idealized conditions which seldom exist in real structures (Galambos, 1988). These assumptions are as follows:

(1) Behavior is purely elastic.

(2) All members have constant cross section.

(3) All joints are rigid.

(4) For braced frames, rotations at opposite ends of beams are equal in magnitude, producing single-curvature bending.




*The subscripts A and B refer to the joints at the two ends of the column section being considered. G is defined as*

$$G = \frac{\Sigma(I_c / L_c)}{\Sigma(I_g / L_g)}$$

*in which $\Sigma$ indicates a summation of all members rigidly connected to that joint and lying on the plane in which buckling of the column is being considered. $I_c$ is the moment of inertia and $L_c$ the unsupported length of a column section, and $I_g$ is the moment of inertia and $L_g$ the unsupported length of a girder or other restraining member. $I_c$ and $I_g$ are taken about axes perpendicular to the plane of buckling being considered.*

*For column ends supported by but not rigidly connected to a footing or foundation, G is theoretically infinity, but, unless actually designed as a true friction-free pin, may be taken as "10" for practical designs. If the column end is rigidly attached to a properly designed footing, G may be taken as 1.0. Smaller values may be used if justified by analysis.*

*Fig. C-C2.2. Alignment chart for effective length of columns in continuous frames.*

(5) For unbraced frames, rotations at opposite ends of the restraining beams are equal in magnitude, producing reverse-curvature bending.

(6) The stiffness parameters $L\sqrt{P/EI}$ of all columns are equal.

(7) Joint restraint is distributed to the column above and below the joint in proportion to $I/L$ of the two columns.

(8) All columns buckle simultaneously.

(9) No significant axial compression force exists in the girders.

Where the actual conditions differ from these assumptions, unrealistic designs may result. There are design procedures available which may be used in the calculation of $G$ for use in Figure C-C2.2 to give results that better reflect the conditions in real structures (Yura, 1971; Disque, 1973; Bjorhovde, 1984; Davison et al., 1988).

Leaning columns (sized for gravity loads only, based on an assumed $K$ of 1.0) may be used in unbraced frames provided that the destabilizing effects due to their lack of lateral stiffness from simple connections to the frame ($K = \infty$) is included in the design of the moment frame columns. A stabilizing column in one direction may be a leaning column in the transverse direction if it is rigidly connected in only one plane. LeMessurier (1977) presented an overall discussion of this problem and recommended a general solution for unbraced frames. In lieu of this and more exact analyses, the following design approximations are suggested.

When unbraced moment-resisting frames are the only source of lateral rigidity for a given direction of a story, the upper bound of sidesway stiffness in that direction, measured, in shear force per radian of drift, is $\Sigma P_L = \Sigma HL/\Delta_{oh}$. This force may be found from a first-order lateral load analysis, without gravity loads, where $\Sigma H$ is the total story shear. (The calculation of $B_2$ using interstory drift as in LRFD Equation C1-4 also uses the term $\Sigma HL/\Delta_{oh}$). Since most of the moment-resisting columns in the frame will directly support axial loads, the bending stiffness of the columns will be reduced, lowering the sidesway stiffness $\Sigma P_L$.

An estimate of the reduced sidesway stiffness of the frame may be found by calculating $P_{e2}$ for each moment-resisting column, in the direction under consideration, by using the nomograph for sidesway $K$ based on local boundary conditions measured by $G_A$ and $G_B$. $G$ is normally assumed as:

$$G = \frac{\Sigma \dfrac{I_c}{L_c}}{\Sigma \dfrac{I_g}{L_g}}$$

This definition of $G$ is based on the assumption that girders restraining columns have equal moments (same clockwise direction) at each end determined by an analysis for lateral loads only. When this assumption is violated, a significant overestimate of $\Sigma P_{e2}$ may occur. Accurate $G$ values may be found from an examination of girder end moments from such an analysis. The correct $L_g$ should be taken as $L_g' = L_g\left[2 - \frac{M_F}{M_N}\right]$ where $M_F$ is the moment at the far end of the girder under consideration and $M_N$ is the moment at the near end. When $\frac{M_F}{M_N} > 2$, $L_g'$ becomes negative which, although real, will result in negative values of $G$. Negative values of $G$ are beyond the scope of the nomograph but are valid for use in Equation C-C2-2.

The reduced total stiffness of the whole story, when each rigidly connected column is loaded with its maximum load $P_{e2}$, is $\Sigma P_{e2}$. $\Sigma P_{e2}$ calculated in this way will always satisfy:

$$.82\Sigma P_L < \Sigma P_{e2} < \Sigma P_L$$

Many common framing arrangements include within a story loaded columns designed, in a particular direction, with $K = 1$. Such columns, often called leaning columns, receive lateral stability from the stiffness of columns with rigid moment-resisting connections. The required axial compressive strength of such leaning columns is called $P_{uo}$ where the subscript implies no shear resistance to lateral loads. The ratio of the loads on all leaning columns in a story to the total of all loads on the story is:

$$\frac{\Sigma P_{uo}}{\Sigma P_u} = R_L$$

The ratio of all story loads to the loads on columns providing sidesway is:

$$N = \frac{1}{1 - R_L} = \frac{\Sigma P_u}{\Sigma P_u - \Sigma P_{uo}}$$

If the story stiffness $\Sigma P_{e2}$ is calculated from the nomograph $K$ values, the net stiffness available to stabilize the rigid column is:

$$\Sigma P_{e2}(1 - R_L) = \frac{\Sigma P_{e2}}{N}$$

If there is no redistribution of $\Sigma P_{e2} / N$ among the rigid columns, the modified capacity of an individual column is, conservatively:

$$P_{e2}' = \frac{P_{e2}}{N}$$

It follows that a modified $K_i'$ including leaning effects is:

$$K_i' = \sqrt{N} \times K_i \tag{C-C2-1}$$

A more exact value of $K'$ to account for loss of stiffness to leaning columns can be found from an iterative solution of:

$$6\left[\frac{\frac{\pi}{K_i'}}{\tan\frac{\pi}{K'}} - R_L\right][G_A + G_B] - \left(\frac{\pi}{K'}\right)^2 G_A G_B + 36\left[1 - R_L\left(\frac{\tan\frac{\pi}{2K'}}{\frac{\pi}{2K'}}\right)\right] = 0 \tag{C-C2-2}$$

When $R_L = 0$, this equation reduces to the equation solved by the sidesway uninhibited nomograph.

The 1993 LRFD Specification no longer limits $K$ to unity in sidesway frames and redistribution of stiffness between members of a frame may be advantageous. There are several ways of doing this. Based on the assumption that $\Sigma P_{e2}$ is constant, regardless of loading distribution, an adjusted distribution of stiffness to the $i$th column of a story is:

$$P_{ei}' = \frac{P_{ui}}{\Sigma P_u}[\Sigma P_{e2}] = \left(\frac{\pi}{K_i'}\right)^2 \frac{EI_i}{L^2} \qquad \text{(C-C2-3a)}$$

except

$$P_{ei}' < 1.6 P_{ei} \qquad \text{(C-C2-3b)}$$

or in terms of $K$ directly with $E$ and $L^2$ constant,

$$K_i' = \sqrt{\frac{\Sigma P_u}{P_{ui}} \times \frac{I_i}{\Sigma\left(\frac{I_i}{K_i^2}\right)}} \qquad \text{(C-C2-4a)}$$

except

$$K_i' \geq \sqrt{\frac{5}{8}}\, K_i \qquad \text{(C-C2-4b)}$$

where

$K_i'$ = effective length factor with story stability effect for $i$th rigid column
$I_i$ = moment of inertia in plane of bending for $i$th rigid column
$K_i$ = effective length for $i$th rigid-column factor based on alignment chart for unbraced frame
$P_{ui}$ = required axial compressive strength for $i$th rigid column
$\Sigma P_u$ = required axial compressive strength of all columns in a story

These expressions include consideration of leaning effects but, in addition, allow concentration of lateral stiffness on relatively weak columns. To limit the error involved with the assumption that $\Sigma P_{e2}$ is constant and to avoid the possibility of failure of a weak column in the sidesway prevented mode, the modified $P_{ei}'$ for a member should not exceed 1.6 times the $P_{ei}$ for the member included in the sum $\Sigma P_{e2}$.

An alternate formulation which is simple to use but may give lower design values than the expressions above when leaning effects are minimal is:

$$P_{ei}' = \frac{P_{ui}}{\Sigma P_u}[\Sigma P_L][.85 + .15 R_L] = \left(\frac{\pi}{K_i'}\right)^2 \frac{EI_i}{L^2} \qquad \text{(C-C2-5a)}$$

except

$$P_{ei}' \leq 1.7P_{Li} \tag{C-C2-5b}$$

where $P_{Li} = \dfrac{H_i L}{\Delta_{oh}}$ and $H_i$ is the shear in the $i$th column included in $\Sigma H$.

The limits set by Equations C-C2-3b, C-C2-4b, and C-C2-5b have been chosen to avoid unconservative error exceeding five percent in extreme cases.

Although $K_i'$ may be found, it is an unnecessary step since the key parameter $\lambda_c^2 = AF_y / P_e'$ is the only one required by Chapter E to find the design capacity $\phi_c P_n$. Design values may be found directly from:

$$\phi_c P_n = \phi_c\, .658^{\left[\frac{AF_y}{P_e'}\right]} AF_y \text{ when } P_e' > \frac{4}{9} AF_y \tag{C-C2-6a}$$

$$\phi_c P_n = \phi_c\, .877 P_e' \text{ when } P_e' \leq \frac{4}{9} AF_y \tag{C-C2-6b}$$

Because frames that use partially restrained (PR) connections violate the condition that all joints are rigid, special attention should be paid to calculation of the proper $G$ value (Barakat and Chen, 1991).

If roof decks or floor slabs, anchored to shear walls or vertical plane bracing systems, are counted upon to provide lateral support for individual columns in a building frame, due consideration must be given to their stiffness when functioning as horizontal diaphragms (Winter, 1958).

Translation of the joints in the plane of a truss is inhibited and, due to end restraint, the effective length of compression members might be assumed to be less than the distance between panel points. However, it is usual practice to take $K$ as equal to 1.0 (Galambos, 1988); if all members of the truss reached their ultimate load capacity simultaneously, the restraints at the ends of the compression members would be greatly reduced.

# CHAPTER D

# TENSION MEMBERS

## D1. DESIGN TENSILE STRENGTH

Due to strain hardening, a ductile steel bar loaded in axial tension can resist, without fracture, a force greater than the product of its gross area and its coupon yield stress. However, excessive elongation of a tension member due to uncontrolled yielding of its gross area not only marks the limit of its usefulness, but can precipitate failure of the structural system of which it is a part. On the other hand, depending upon the reduction of area and other mechanical properties of the steel, the member can fail by fracture of the net area at a load smaller than required to yield the gross area. Hence, general yielding of the gross area and fracture of the net area both constitute failure limit states. The relative values of $\phi_t$ given for yielding and fracture reflect the same basic difference in factor of safety as between design of members and design of connections in the AISC ASD Specification.

The length of the member in the net area is negligible relative to the total length of the member. As a result, the strain hardening condition is quickly reached and yielding of the net area at fastener holes does not constitute a limit state of practical significance.

## D2. BUILT-UP MEMBERS

The slenderness ratio $L / r$ of tension members other than rods, tubes, or straps should preferably not exceed the limiting value of 300. This slenderness limit recommended for tension members is not essential to the structural integrity of such members; it merely assures a degree of stiffness such that undesirable lateral movement ("slapping" or vibration) will be unlikely.

See Section B7 and Commentary Section E4.

## D3. PIN-CONNECTED MEMBERS AND EYEBARS

Forged eyebars have generally been replaced by pin-connected plates or eyebars thermally cut from plates. Provisions for the proportioning of eyebars contained in the LRFD Specification are based upon standards evolved from long experience with forged eyebars. Through extensive destructive testing, eyebars have been found to provide balanced designs when they are thermally cut instead of forged. The somewhat more conservative rules for pin-connected members of nonuniform cross section and those not having enlarged "circular" heads are likewise based on the results of experimental research (Johnston, 1939).

Somewhat stockier proportions are provided for eyebars and pin-connected members fabricated from steel having a yield stress greater than 70 ksi, in order to eliminate any possibility of their "dishing" under the higher design stress.

# CHAPTER E

# COLUMNS AND OTHER COMPRESSION MEMBERS

## E1. EFFECTIVE LENGTH AND SLENDERNESS LIMITATIONS

### 1. Effective Length

The Commentary on Section C2 regarding frame stability and effective length factors applies here. Further analytic methods, formulas, charts, and references for the determination of effective length are provided in Chapter 15 of the SSRC Guide (Galambos, 1988).

### 2. Design by Plastic Analysis

The limitation on $\lambda_c$ is essentially the same as that for $l / r$ in Chapter N of the 1989 AISC Specification—Allowable Stress Design and Plastic Design.

## E2. DESIGN COMPRESSIVE STRENGTH FOR FLEXURAL-TORSIONAL BUCKLING*

Equations E2-2 and E2-3 are based on a reasonable conversion of research data into design equations. Conversion of the allowable stress design (ASD) equations which was based on the CRC—Column Research Council—curve (Galambos, 1988) was found to be cumbersome for two reasons. The first was the nature of the ASD variable safety factor. Secondly, the difference in philosophical origins of the two design procedures requires an assumption of a live load-to-dead load ratio ($L / D$).

Since all $L / D$ ratios could not be considered, a value of approximately 1.1 at $\lambda$ equal to 1.0 was used to calibrate the exponential equation for columns with the lower range of $\lambda$ against the appropriate ASD provision. The coefficient with the Euler equation was obtained by equating the ASD and LRFD expressions at $\lambda$ of 1.5.

Equations E2-2 and E2-3 are essentially the same curve as column-strength curve 2P of the Structural Stability Research Council which is based on an initial out-of-straightness curve of $l / 1{,}500$ (Bjorhovde, 1972 and 1988; Galambos, 1988; Tide, 1985).

It should be noted that this set of column equations has a range of reliability ($\beta$) values. At low- and high-column slenderness, $\beta$ values exceeding 3.0 and 3.3 respectively are obtained compared to $\beta$ of 2.60 at $L / D$ of 1.1. This is considered satisfactory, since the limits of out-of-straightness combined with residual stress have not been clearly established. Furthermore, there has been

* For tapered members see Commentary Appendix F3.

no history of unacceptable behavior of columns designed using the ASD procedure. This includes cases with $L/D$ ratios greater than 1.1.

Equations E2-2 and E2-3 can be restated in terms of the more familiar slenderness ratio $Kl/r$. First, Equation E2-2 is expressed in exponential form,

$$F_{cr} = [\exp(-0.419\lambda_c^2)]F_y \qquad \text{(C-E2-1)}$$

Note that $\exp(x)$ is identical to $e^x$. Substitution of $\lambda_c$ according to definition of $\lambda_c$ in Section E2 gives,

For $\dfrac{Kl}{r} \leq 4.71\sqrt{\dfrac{E}{F_y}}$

$$F_{cr} = \left\{\exp\left[-0.0424\frac{F_y}{E}\left(\frac{Kl}{r}\right)^2\right]\right\}F_y \qquad \text{(C-E2-2)}$$

For $\dfrac{Kl}{r} > 4.71\sqrt{\dfrac{E}{F_y}}$

$$F_{cr} = \frac{0.877\pi^2 E}{\left(\dfrac{Kl}{r}\right)^2} \qquad \text{(C-E2-3)}$$

## E3. DESIGN COMPRESSIVE STRENGTH FOR FLEXURAL-TORSIONAL BUCKLING

Torsional buckling of symmetric shapes and flexural-torsional buckling of unsymmetric shapes are failure modes usually not considered in the design of hot-rolled columns. They generally do not govern, or the critical load differs very little from the weak axis planar buckling load. Such buckling loads may, however, control the capacity of symmetric columns made from relatively thin plate elements and unsymmetric columns. Design equations for determining the strength of such columns are given in Appendix E3.

Tees that conform to the limits in Table C-E3.1 need not be checked for flexural-torsional buckling.

A simpler and more accurate design strength for the special case of tees and double-angles is based on Galambos (1991) wherein the y-axis of symmetry flexural-buckling strength component is determined directly from the column formulas.

The separate AISC *Specification for Load and Resistance Factor Design of Single-Angle Members* contains detailed provisions not only for the limit state of compression, but also for tension, shear, flexure, and combined forces.

## TABLE C-E3.1
## Limiting Proportions for Tees

| Shape | Ratio of Full Flange Width to Profile Depth | Ratio of Flange Thickness to Web or Stem Thickness |
|---|---|---|
| Built-up tees | ≥ 0.50 | ≥ 1.25 |
| Rolled tees | ≥ 0.50 | ≥ 1.10 |

## E4. BUILT-UP MEMBERS

Requirements for detailing and design of built-up members, which cannot be stated in terms of calculated stress, are based upon judgment and experience.

The longitudinal spacing of connectors connecting components of built-up compression members must be such that the slenderness ratio $l / r$ of individual shapes does not exceed three-fourths of the slenderness ratio of the entire member. Additional requirements are imposed for built-up members consisting of angles. However, these minimum requirements do not necessarily ensure that the effective slenderness ratio of the built-up member is equal to that for the built-up member acting as a single unit. Section E4 gives formulas for modified slenderness ratios that are based on research and take into account the effect of shear deformation in the connectors (Zandonini, 1985). Equation E4-1 for snug-tight intermediate connectors is emperically based on test results (Zandonini, 1985). The new Equation E4-2 is derived from theory and verified by test data. In both cases the end connection must be welded or slip-critical bolted (Aslani and Goel, 1991). The connectors must be designed to resist the shear forces which develop in the buckled member. The shear stresses are highest where the slope of the buckled shape is maximum (Bleich, 1952).

Maximum fastener spacing less than that required for strength may be needed to ensure a close fit over the entire faying surface of components in continuous contact. Specific requirements are given for weathering steel members exposed to atmospheric corrosion (Brockenbrough, 1983).

The provisions governing the proportioning of perforated cover plates are based upon extensive experimental research (Stang and Jaffe, 1948).

# CHAPTER F

# BEAMS AND OTHER FLEXURAL MEMBERS

## F1. DESIGN FOR FLEXURE

### 1. Yielding

The bending strength of a laterally braced compact section is the plastic moment $M_p$. If the shape has a large shape factor (ratio of plastic moment to the moment corresponding to the onset of yielding at the extreme fiber), significant inelastic deformation may occur at service load if the section is permitted to reach $M_p$ at factored load. The limit of $1.5M_y$ at factored load will control the amount of inelastic deformation for sections with shape factors greater than 1.5. This provision is not intended to limit the plastic moment of a hybrid section with a web yield stress lower than the flange yield stress. Yielding in the web does not result in significant inelastic deformations. In hybrid sections, $M_y = F_{yf}S$.

Lateral-torsional buckling cannot occur if the moment of inertia about the bending axis is equal to or less than the moment of inertia out of plane. Thus, for shapes bent about the minor axis and shapes with $I_x = I_y$, such as square or circular shapes, the limit state of lateral-torsional buckling is not applicable and yielding controls if the section is compact.

### 2. Lateral-Torsional Buckling

### 2a. Doubly Symmetric Shapes and Channels with $L_b \leq L_r$

The basic relationship between nominal moment $M_n$ and unbraced length $L_b$ is shown in Figure C-F1.1 for a compact section with $C_b = 1.0$. There are four principal zones defined on the basic curve by $L_{pd}$, $L_p$, and $L_r$. Equation F1-4 defines the maximum unbraced length $L_p$ to reach $M_p$ with uniform moment. Elastic lateral-torsional buckling will occur when the unbraced length is greater than $L_r$ given by Equation F1-6. Equation F1-2 defines the inelastic lateral-torsional buckling as a straight line between the defined limits $L_p$ and $L_r$. Buckling strength in the elastic region $L_b > L_r$ is given by Equation F1-14 for I-shaped members.

For other moment diagrams, the lateral buckling strength is obtained by multiplying the basic strength by $C_b$ as shown in Figure C-F1.1. The maximum $M_n$, however, is limited to $M_p$. Note that $L_p$ given by Equation F1-4 is merely a definition which has physical meaning when $C_b = 1.0$. For $C_b$ greater than 1.0, larger unbraced lengths are permitted to reach $M_p$ as shown by the curve for $C_b > 1.0$. For design, this length could be calculated by setting Equation F1-2 equal to $M_p$ and solving this equation for $L_b$ using the desired $C_b$ value.

The equation

$$C_b = 1.75 + 1.05(M_1 / M_2) + 0.3(M_1 / M_2)^2 \leq 2.3 \qquad \text{(C-F1-1)}$$

has been used since 1961 to adjust the flexural-torsional buckling equation for variations in the moment diagram within the unbraced length. This equation is applicable only to moment diagrams that are straight lines between braced points. The equation provides a lower bound fit to the solutions developed by Salvadori (1956) which are shown in Figure C-F1.2. Another equation

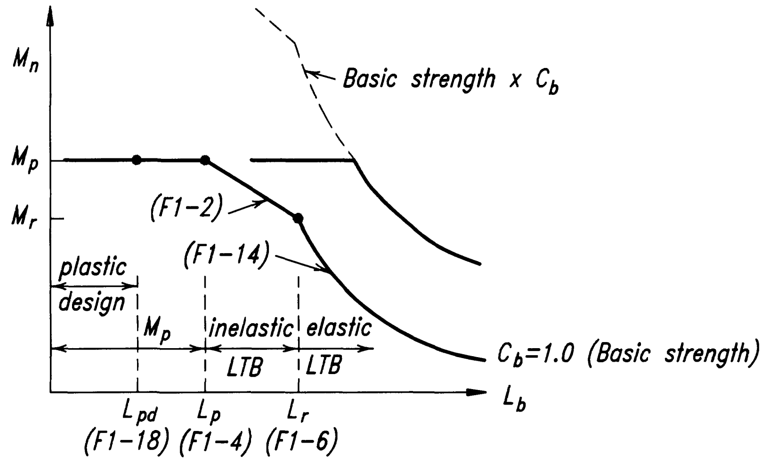



*Fig. C-F1.1. Nominal moment as a function of unbraced length and moment gradient.*

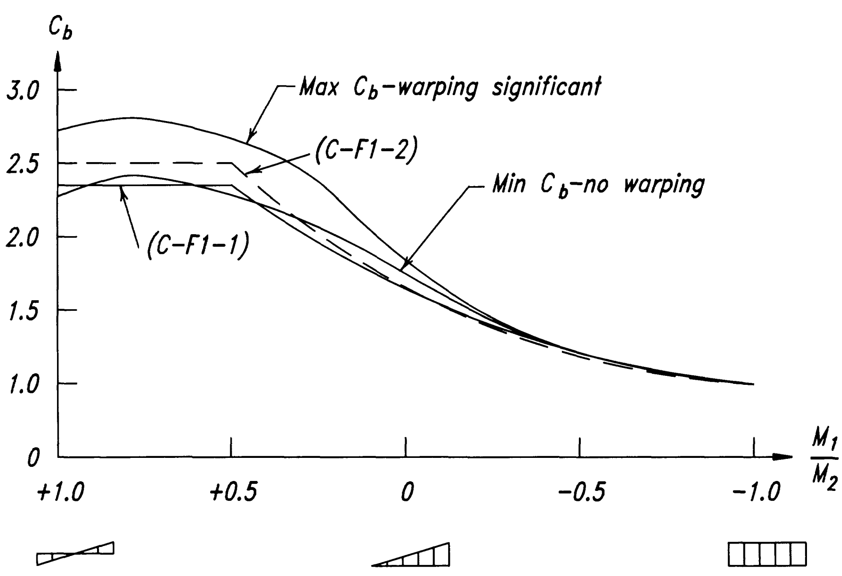



*Fig. C-F1.2. Moment modifier $C_b$ for beams.*

$$C_b = \frac{1}{0.6 - 0.4\dfrac{M_1}{M_2}} \leq 2.5 \tag{C-F1-2}$$

fits the average value theoretical solutions when the beams are bent in reverse curvature and also provides a reasonable fit to the theory. If the maximum moment within the unbraced segment is equal to or larger than the end moment, $C_b = 1.0$ is used.

The equations above can be easily misinterpreted and misapplied to moment diagrams that are not straight within the unbraced segment. Kirby and Nethercot (1979) presented an equation which applies to various shapes of moment diagrams within the unbraced segment. Their equation has been adjusted slightly to the following

$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_C} \tag{C-F1-3}$$

This equation gives more accurate solutions for fixed-end beams, and the adjusted equation reduces exactly to Equation C-F1-2 for a straight line moment diagram in single curvature. The new $C_b$ equation is shown in Figure C-F1.3 for straight line moment diagrams. Other moment diagrams along with exact theoretical solutions in the SSRC Guide (Galambos, 1988) show good comparison with the new equation. The absolute value of the three interior quarter-point moments plus the maximum moment, regardless of its location are used in the equation. The maximum moment in the unbraced segment is always used for comparison with the resistance. The length between braces, not the distance to inflection points, and $C_b$ is used in the resistance equation.

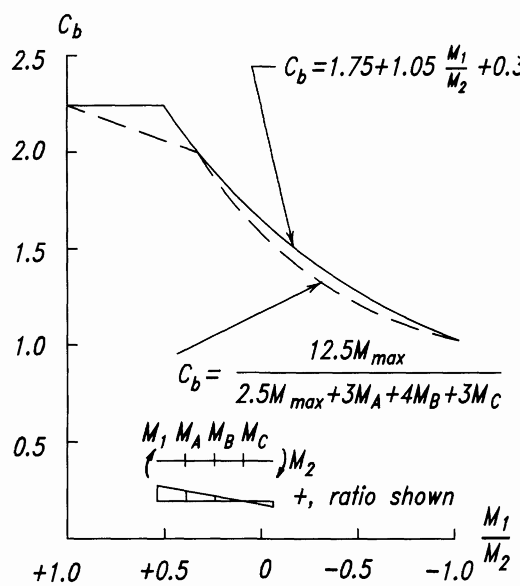



*Fig. C-F1.3. $C_b$ for a straight line moment diagram—prismatic beam.*

It is still satisfactory to use the former $C_b$ factor, Equation C-F1-1, for straight line moment diagrams within the unbraced length.

The elastic strength of hybrid beams is identical to homogeneous beams. The strength advantage of hybrid sections becomes evident only in the inelastic and plastic slenderness ranges.

## 2b. Doubly Symmetric Shapes and Channels with $L_b > L_r$

The equation given in the Specification assumes that the loading is applied along the beam centroidal axis. If the load is placed on the top flange and the flange is not braced, there is a tipping effect that reduces the critical moment; conversely, if the load is suspended from the bottom flange and is not braced, there is a stabilizing effect which increases the critical moment (Galambos, 1988). For unbraced top flange loading, the reduced critical moment may be conservatively approximated by setting the warping buckling factor $X_2$ to zero.

An effective length factor of unity is implied in these critical moment equations to represent a worst case pinned-pinned unbraced segment. Including consideration of any end restraint of the adjacent segments on the critical segment can increase its buckling capacity. The effects of beam continuity on lateral-torsional buckling have been studied and a simple and conservative design method, based on the analogy of end-restrained nonsway columns with an effective length factor less than one, has been proposed (Galambos, 1988).

## 2c. Tees and Double-Angles

The lateral-torsional buckling strength (LTS) of singly symmetric tee beams is given by a fairly complex formula (Galambos, 1988). Equation F1-15 is a simplified formulation based on Kitipornchai and Trahair (1980). See also Ellifritt, et al., 1992.

The $C_b$ used for I-shaped beams is unconservative for tee beams with the stem in compression. For such cases $C_b = 1.0$ is appropriate. When beams are bent in reverse curvature, the portion with the stem in compression may control the LTB resistance even though the moments may be small relative to other portions of the unbraced length with $C_b \approx 1.0$. This is because the LTB strength of a tee with the stem in compression may be only about one-fourth of the capacity for the stem in tension. Since the buckling strength is sensitive to the moment diagram, $C_b$ has been conservatively taken as 1.0. In cases where the stem is in tension, connection details should be designed to minimize any end restraining moments which might cause the stem to be in compression.

## 2d. Unbraced Length for Plastic Analysis

In the AISC ASD Specification, Chapter N, the unbraced length of a beam that permits the attainment of plastic moments, and ensures sufficient rotation capacity to redistribute moments, is given by two formulas which depend on the moment ratio at the ends of the unbraced length. One length is permitted for $M_1 / M_2 < -0.5$ (almost uniform moment), and a substantially larger length for $M_1 / M_2 > -0.5$. These two equations are replaced by Equation F1-18 to provide a continuous function between unbraced length and end moment ratio so there is no abrupt change for a slight change in moment ratio near $-0.5$. At $M_1 / M_2 = -0.5$ (uniform moment) the maximum unbraced length is almost the same as that in the AISC ASD Specification. There is a substantial increase in unbraced length for positive moment ratios (reverse curvature) because the yielding is confined to zones close to the brace points (Yura, et al., 1978).

Equation F1-19 is an equation in similar form for solid rectangular bars and symmetric box beams. Equations F1-18 and F1-19 assume that the moment diagram within the unbraced length next to plastic hinge locations is reasonably linear. For nonlinear diagrams between braces, judgment should be used in choosing a representative ratio.

Equations F1-18 and F1-19 were developed to provide rotation capacities of at least 3.0, which are sufficient for most applications (Yura, et al., 1978). When inelastic rotations of 7 to 9 are deemed appropriate in areas of high seismicity, as discussed in Commentary Section B5, Equation F1-18 would become:

$$L_{pd} = \frac{2500 r_y}{F_y} \qquad \text{(C-F1-3)}$$

## F2. DESIGN FOR SHEAR

For unstiffened webs $k_v = 5.0$,

therefore $187\sqrt{k_v / F_{yw}} = 418 / \sqrt{F_{yw}}$, and $234\sqrt{k_v / F_{yw}} = 523 / \sqrt{F_{yw}}$.

For webs with $h / t_w \leq 187\sqrt{k_v / F_{yw}}$, the nominal shear strength $V_n$ is based on shear yielding of the web, Equation F2-1 and Equation A-F2-1. This $h / t_w$ limit was determined by setting the critical stress causing shear buckling $F_{cr}$ equal to the yield stress of the web $F_{yw}$ in Equation 35 of Cooper et al. (1978) and Timoshenko and Gere (1961). When $h / t_w > 187\sqrt{k_v / F_{yw}}$, the web shear strength is based on buckling. Basler (1961) suggested taking the proportional limit as 80 percent of the yield stress of the web. This corresponds to $h / t_w = (187/0.8)(\sqrt{k_v / F_{yw}})$. Thus, when $h / t_w > 234(\sqrt{k_v / F_{yw}})$, the web strength is determined from the elastic buckling stress given by Equation 6 of Cooper et al., (1978) and Timoshenko and Gere (1961):

$$F_{cr} = \frac{\pi^2 E k_v}{12(1 - \nu^2)(h / t_w)^2} \qquad \text{(C-F2-1)}$$

The nominal shear strength, given by Equation F2-3 and A-F2-3, was obtained by multiplying $F_{cr}$ by the web area and using $E = 29{,}000$ ksi and $\nu = 0.3$. A straight line transition, Equation F2-2 and AF2-2, is used between the limits $187(\sqrt{k_v / F_{yw}})$ and $234(\sqrt{k_v / F_{yw}})$.

The shear strength of flexural members follows the approach used in the AISC ASD Specification, except for two simplifications. First, the expression for the plate buckling coefficient $k_v$ has been simplified; it corresponds to that given by AASHTO *Standard Specification for Highway Bridges* (1989). The earlier expression for $k_v$ was a curve fit to the exact expression; the new expression is just as accurate. Second, the alternate method (tension field action) for web shear strength is placed in Appendix G because it was desired that only one method appear in the main body of the Specification with alternate methods given in the Appendix. When designing plate girders, thicker unstiffened webs will frequently be less costly than lighter stiffened web designs because of the additional fabrication. If a stiffened girder design has economic advantages, the tension field method in Appendix G will require fewer stiffeners.

The equations in this section were established assuming monotonically increasing loads. If a flexural member is subjected to load reversals causing cyclic yielding over large portions of a web, such as may occur during a major earthquake, special design considerations may apply (Popov, 1980).

## F4. BEAMS AND GIRDERS WITH WEB OPENINGS

Web openings in structural floor members may be necessary to accommodate various mechanical, electrical, and other systems. Strength limit states, including local buckling of the compression flange, web, and tee-shaped compression zone above or below the opening, lateral buckling and moment-shear interaction, or serviceability may control the design of a flexural member with web openings. The location, size, and number of openings are important and empirical limits for them have been identified. One general procedure for assessing these effects and the design of any needed reinforcement for both steel and composite beams is given in Darwin (1990) and in ASCE (1992, 1992a).

# CHAPTER H

# MEMBERS UNDER COMBINED FORCES AND TORSION

## H1. SYMMETRIC MEMBERS SUBJECT TO BENDING AND AXIAL FORCE

Equations H1-1a and H1-1b are simplifications and clarifications of similar equations used in the AISC ASD Specification since 1961. Previously, both equations had to be checked. In the new formulation the applicable equation is governed by the value of the first term, $P_u / \phi P_n$. For bending about one axis only, the equations have the form shown in Figure C-H1.1.

The first term $P_u / \phi P_n$ has the same significance as the axial load term $f_a / F_a$ in Equations H1-1 of the AISC ASD Specification. This means that for members in compression $P_n$ must be based on the largest effective slenderness ratio $Kl / r$. In the development of Equations H1-1a and H1-1b, a number of alternative formulations were compared to the exact inelastic solutions of 82 sidesway cases reported in Kanchanalai (1977). In particular, the possibility of using $Kl / r$ as the actual column length ($K$ = 1) in determining $P_n$, combined with an elastic second order moment $M_u$, was studied. In those cases where the true $P_n$ based on $Kl / r$, with $K$ = 1.0, was in the inelastic range, the errors proved to be unacceptably large without the additional check that $P_u \leq \phi_c P_n$, $P_n$ being based on effective length. Although deviations from exact solutions were reduced, they still remained high.

In summary, it is not possible to formulate a safe general interaction equation

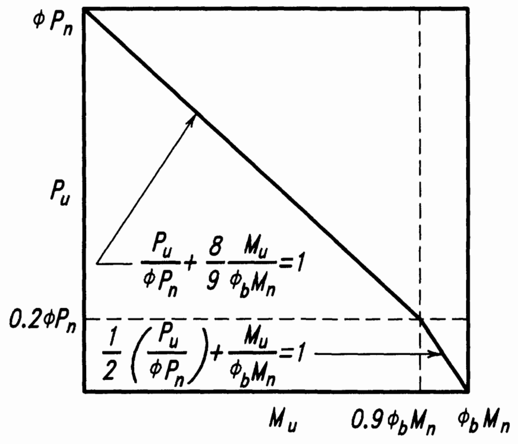



*Fig. C-H1.1. Beam-column interaction equations.*

for compression without considering effective length directly (or indirectly by a second equation). Therefore, the requirement that the nominal compressive strength $P_n$ be based on the effective length $KL$ in the general equation is continued in the LRFD Specification as it has been in the AISC ASD Specification since 1961. It is not intended that these provisions be applicable to limit nonlinear secondary flexure that might be encountered in large amplitude earthquake stability design (ATC, 1978).

The defined term $M_u$ is the maximum moment in a member. In the calculation of this moment, inclusion of beneficial second order effects of tension is optional. But consideration of detrimental second order effects of axial compression and translation of gravity loads is required. Provisions for calculation of these effects are given in Chapter C.

The interaction equations in Appendix H3 have been recommended for biaxially loaded H and wide flange shapes in Galambos (1988) and Springfield (1975). These equations which can be used only in braced frames represent a considerable liberalization over the provisions given in Section H1; it is, therefore, also necessary to check yielding under service loads, using the appropriate load and resistance factors for the serviceability limit state in Equation H1-1a or H1-1b with $M_{ux} = S_x F_y$ and $M_{uy} = S_y F_y$. Appendix H3 also provides interaction equations for rectangular box-shaped beam-columns. These equations are taken from Zhou and Chen (1985).

## H2. UNSYMMETRIC MEMBERS AND MEMBERS UNDER TORSION AND COMBINED TORSION, FLEXURE, SHEAR, AND/OR AXIAL FORCE

This section deals with types of cross sections and loadings not covered in Section H1, especially where torsion is a consideration. For such cases it is recommended to perform an elastic analysis based on the theoretical numerical methods available from the literature for the determination of the maximum normal and shear stresses, or for the elastic buckling stresses. In the buckling calculations an equivalent slenderness parameter is determined for use in Equation E2-2 or E2-3, as follows:

$$\lambda_e = \sqrt{F_y / F_e}$$

where $F_e$ is the elastic buckling stress determined from a stability analysis. This procedure is similar to that of Appendix E3.

For the analysis of members with open sections under torsion refer to AISC (1983).

# CHAPTER I

# COMPOSITE MEMBERS

## I1. DESIGN ASSUMPTIONS

*Force Determination.* Loads applied to an unshored beam before the concrete has hardened are resisted by the steel section alone, and only loads applied after the concrete has hardened are considered as resisted by the composite section. It is usually assumed for design purposes that concrete has hardened when it attains 75 percent of its design strength. In beams properly shored during construction, all loads may be assumed as resisted by the composite cross section. Loads applied to a continuous composite beam with shear connectors throughout its length, after the slab is cracked in the negative moment region, are resisted in that region by the steel section and by properly anchored longitudinal slab reinforcement.

For purposes of plastic analysis all loads are considered resisted by the composite cross section, since a fully plastic strength is reached only after considerable yielding at the locations of plastic hinges.

*Elastic Analysis.* The use of constant stiffness in elastic analyses of continuous beams is analogous to the practice in reinforced concrete design.

*Plastic Analysis.* For composite beams with shear connectors, plastic analysis may be used only when the steel section in the positive moment region has a compact web, i.e., $h / t_w \leq 640\sqrt{F_{yf}}$ , and when the steel section in the negative moment region is compact, as required for steel beams alone. No compactness limitations are placed on encased beams, but plastic analysis is permitted only if the direct contribution of concrete to the strength of sections is neglected; the concrete is relied upon only to prevent buckling.

*Plastic Stress Distribution for Positive Moment.* Plastic stress distributions are described in Commentary Section I3, and a discussion of the composite participation of slab reinforcement is presented.

*Plastic Stress Distribution for Negative Moment.* Plastic stress distributions are described in Commentary Section I3.

*Elastic Stress Distribution.* The strain distribution at any cross section of a composite beam is related to slip between the structural steel and concrete elements. Prior to slip, strain in both steel and concrete is proportional to the distance from the neutral axis for the elastic transformed section. After slip, the strain distribution is discontinuous, with a jump at the top of the steel shape. The strains in steel and concrete are proportional to distances from separate neutral axes, one for steel and the other for concrete.

*Fully Composite Beam.* Either tensile yield strength of the steel section or the compressive stress of the concrete slab governs the maximum flexural strength of a fully composite beam subjected to a positive moment. The tensile yield strength of the longitudinal reinforcing bars in the slab governs the maximum flexural strength of a fully composite beam subjected to a negative moment. When shear connectors are provided in sufficient numbers to fully develop this maximum flexural strength, any slip that occurs prior to yielding is minor and has negligible influence both on stresses and stiffness.

*Partially Composite Beam.* The effects of slip on elastic properties of a partially composite beam can be significant and should be accounted for in calculations of deflections and stresses at service loads. Approximate elastic properties of partially composite beams are given in Commentary Section I3. For simplified design methods, see Hansell, et al. (1978).

*Concrete-Encased Beam.* When the dimensions of a concrete slab supported on steel beams are such that the slab can effectively serve as the flange of a composite T-beam, and the concrete and steel are adequately tied together so as to act as a unit, the beam can be proportioned on the assumption of composite action.

Two cases are recognized: fully encased steel beams, which depend upon natural bond for interaction with the concrete, and those with mechanical anchorage to the slab (shear connectors), which do not have to be encased.

## I2. COMPRESSION MEMBERS

### 1. Limitations

(a) The lower limit of four percent on the cross-sectional area of structural steel differentiates between composite and reinforced concrete columns. If the area is less than four percent, a column with a structural steel core should be designed as a reinforced concrete column.

(b) The specified minimum quantity of transverse and longitudinal reinforcement in the encasement should be adequate to prevent severe spalling of the surface concrete during fires.

(c) Very little of the supporting test data involved concrete strengths in excess of 6 ksi, even though the cylinder strength for one group of four columns was 9.6 ksi. Normal weight concrete is believed to have been used in all tests. Thus, the upper limit of concrete strength is specified as 8 ksi for normal weight concrete. A lower limit of 3 ksi is specified for normal weight concrete and 4 ksi for lightweight concrete to encourage the use of good quality, yet readily available, grades of structural concrete.

(d) Encased steel shapes and longitudinal reinforcing bars are restrained from buckling as long as the concrete remains sound. A limit strain of 0.0018, at which unconfined concrete remains unspalled and stable, serves analytically to define a failure condition for composite cross sections under uniform axial strain. The limit strain of 0.0018 corresponds approximately to 55 ksi.

(e) The specified minimum wall thicknesses are identical to those in the 1989 ACI Building Code (1989). The purpose of this provision is to prevent buckling of the steel pipe or tubing before yielding.

## 2. Design Strength

The procedure adopted for the design of axially loaded composite columns is described in detail in Galambos and Chapuis (1980). It is based on the equation for the strength of a short column derived in Galambos and Chapuis (1980), and the same reductions for slenderness as those specified for steel columns in Section E2. The design follows the same path as the design of steel columns, except that the yield stress of structural steel, the modulus of elasticity of steel, and the radius of gyration of the steel section, are modified to account for the effect of concrete and longitudinal reinforcing bars. A detailed explanation of the origin of these modifications may be found in SSRC Task Group 20 (1979). Galambos and Chapuis (1980) includes comparisons of the design procedure with 48 tests of axially loaded stub columns, 96 tests of concrete-filled pipes or tubing, and 26 tests of concrete-encased steel shapes. The mean ratio of the test failure loads to the predicted strengths was 1.18 for all 170 tests, and the corresponding coefficient of variation was 0.19.

## 3. Columns with Multiple Steel Shapes

This limitation is based on Australian research reported in Bridge and Roderick (1978), which demonstrated that after hardening of concrete the composite column will respond to loading as a unit even without lacing, tie plates, or batten plates connecting the individual steel sections.

## 4. Load Transfer

To avoid overstressing either the structural steel section or the concrete at connections, a transfer of load to concrete by direct bearing is required.

When a supporting concrete area is wider on all sides than the loaded area, the maximum design strength of concrete is specified by ACI (1989) as $1.7\phi_B f_c' A_B$ where $\phi_B = 0.7$ is the strength reduction factor in bearing on concrete and $A_B$ is the loaded area. Because the AISC LRFD Specification is based on the lower ASCE 7 load factors (ASCE, 1988), $\phi_B = 0.60$ in the AISC LRFD Specification. The portion of the design load of an axially loaded column $\phi P_n$ resisted by the concrete may be expressed as $(c_2 f_c' A_c / A_s F_{my})\phi_B P_n$.

Accordingly,

$$A_B \geq \frac{\phi_B \, c_2 \, A_c \, P_n}{\phi_B 1.7 A_s F_{my}} = \frac{c_2 \, A_c \, P_n}{1.7 A_s F_{my}} \qquad \text{(C-I2-1)}$$

# I3. FLEXURAL MEMBERS

## 1. Effective Width

LRFD provisions for effective width omit any limit based on slab thickness, in accordance with both theoretical and experimental studies, as well as current composite beam codes in other countries (ASCE, 1979). The same effective width rules apply to composite beams with a slab on either one side or both sides of the beam. To simplify design, effective width is based on the full span, center-to-center of supports, for both simple and continuous beams.

## 2. Strength of Beams with Shear Connectors

This section applies to simple and continuous composite beams with shear connectors, constructed with or without temporary shores.

*Positive Flexural Design Strength.* Flexural strength of a composite beam in the positive moment region may be limited by the plastic strength of the steel section, the concrete slab, or shear connectors. In addition, web buckling may limit flexural strength if the web is slender and a significantly large portion of the web is in compression.

According to Table B5.1, local web buckling does not reduce the plastic strength of a bare steel beam if the beam depth-to-web thickness ratio is not larger than $640/\sqrt{F_y}$. In the absence of web buckling research on composite beams, the same ratio is conservatively applied to composite beams. Furthermore, for more slender webs, the LRFD Specification conservatively adopts first yield as the flexural strength limit. In this case, stresses on the steel section from permanent loads applied to unshored beams before the concrete has hardened must be superimposed on stresses on the composite section from loads applied to the beams after hardening of concrete. In this superposition, all permanent loads should be multiplied by the dead load factor and all live loads should be multiplied by the live load factor. For shored beams, all loads may be assumed as resisted by the composite section.

When first yield is the flexural strength limit, the elastic transformed section is used to calculate stresses on the composite section. The modular ratio $n = E/E_c$ used to determine the transformed section depends on the specified unit weight and strength of concrete. Note that this procedure for compact beams differs from the requirements of Section I2 of the 1989 AISC ASD Specification.

*Plastic Stress Distribution for Positive Moment.* When flexural strength is determined from the plastic stress distribution shown in Figure C-I3.1, compression force $C$ in the concrete slab is the smallest of:

$$C = A_{sw}F_{yw} + 2A_{sf}F_{yf} \qquad \text{(C-I3-1)}$$

$$C = 0.85f_c'A_c \qquad \text{(C-I3-2)}$$

$$C = \Sigma Q_n \qquad \text{(C-I3-3)}$$

For a non-hybrid steel section, Equation C-I3-1 becomes $C = A_sF_y$

where

$f_c'$ = specified compressive strength of concrete, ksi
$A_c$ = area of concrete slab within effective width, in.$^2$
$A_s$ = area of steel cross section, in.$^2$
$A_{sw}$ = area of steel web, in.$^2$
$A_{sf}$ = area of steel flange, in.$^2$
$F_y$ = minimum specified yield stress of steel, ksi
$F_{yw}$ = minimum specified yield stress of web steel, ksi
$F_{yf}$ = minimum specified yield stress of flange steel, ksi
$\Sigma Q_n$ = sum of nominal strengths of shear connectors between the point of

maximum positive moment and the point of zero moment to either side, kips

Longitudinal slab reinforcement makes a negligible contribution to the compression force, except when Equation C-I3-2 governs. In this case, the area of longitudinal reinforcement within the effective width of the concrete slab times the yield stress of the reinforcement may be added in determining $C$.

The depth of the compression block is

$$a = \frac{C}{0.85 f_c' b} \tag{C-I3-4}$$

where

$b$ = effective width of concrete slab, in.

A fully composite beam corresponds to the case of $C$ governed by the yield strength of the steel beam or the compressive strength of the concrete slab, as in Equation C-I3-1 or C-I3-2. The number and strength of shear connectors govern $C$ for a partially composite beam as in Equation C-I3-3.

The plastic stress distribution may have the plastic neutral axis (PNA) in the web, in the top flange of the steel section or in the slab, depending on the value of $C$.

The nominal plastic moment resistance of a composite section in positive bending is given by the following equation and Figure C-I3.1:

$$M_n = C(d_1 + d_2) + P_y(d_3 - d_2) \tag{C-I3-5}$$

where

$P_y$ = tensile strength of the steel section; for a non-hybrid steel section $P_y = A_s F_y$, kips
$d_1$ = distance from the centroid of the compression force $C$ in concrete to the top of the steel section, in.
$d_2$ = distance from the centroid of the compression force in the steel section to the top of the steel section, in. For the case of no compression in the steel section $d_2 = 0$.

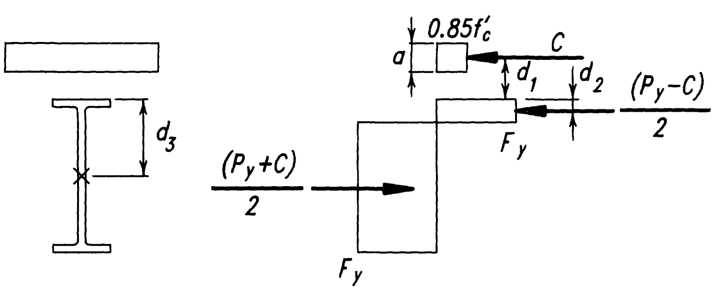



*Fig. C-I3.1. Plastic stress distribution for positive moment in composite beams.*

$d_3$ = distance from $P_y$ to the top of the steel section, in.

Equation C-I3-5 is generally applicable including both non-hybrid and hybrid steel sections symmetrical about one or two axes.

*Approximate Elastic Properties of Partially Composite Beams.* Elastic calculations for stress and deflection of partially composite beams should include the effects of slip.

The effective moment of inertia $I_{eff}$ for a partially composite beam is approximated by

$$I_{eff} = I_s + \sqrt{(\Sigma Q_n / C_f)}(I_{tr} - I_s) \tag{C-I3-6}$$

where

$I_s$ = moment of inertia for the structural steel section, in.$^4$
$I_{tr}$ = moment of inertia for the fully composite uncracked transformed section, in.$^4$
$\Sigma Q_n$ = strength of shear connectors between the point of maximum positive moment and the point of zero moment to either side, kips
$C_f$ = compression force in concrete slab for fully composite beam; smaller of Equations C-I3-1 and C-I3-2, kips

The effective section modulus $S_{eff}$, referred to the tension flange of the steel section for a partially composite beam, is approximated by

$$S_{eff} = S_s + \sqrt{(\Sigma Q_n / C_f)}(S_{tr} - S_s) \tag{C-I3-7}$$

where

$S_s$ = section modulus for the structural steel section, referred to the tension flange, in.$^3$
$S_{tr}$ = section modulus for the fully composite uncracked transformed section, referred to the tension flange of the steel section, in.$^3$

Equations C-I3-6 and C-I3-7 should not be used for ratios $\Sigma Q_n / C_f$ less than 0.25. This restriction is to prevent excessive slip, as well as substantial loss in beam stiffness. Studies indicate that Equations C-I3-6 and C-I3-7 adequately reflect the reduction in beam stiffness and strength, respectively, when fewer connectors are used than required for full composite action (Grant et al., 1977).

*Negative Flexural Design Strength.* The flexural strength in the negative moment region is the strength of the steel beam alone or the plastic strength of the composite section made up of the longitudinal slab reinforcement and the steel section.

*Plastic Stress Distribution for Negative Moment.* When an adequately braced compact steel section and adequately developed longitudinal reinforcing bars act compositely in the negative moment region, the nominal flexural strength is determined from the plastic stress distributions as shown in Figure C-I3.2. The tensile force $T$ in the reinforcing bars is the smaller of:

$$T = A_r F_{yr} \tag{C-I3-8}$$

$$T = \Sigma Q_n \quad \text{(C-I3-9)}$$

where

$A_r$ = area of properly developed slab reinforcement parallel to the steel beam and within the effective width of the slab, in.$^2$

$F_{yr}$ = specified yield stress of the slab reinforcement, ksi

$\Sigma Q_n$ = sum of the nominal strengths of shear connectors between the point of maximum negative moment and the point of zero moment to either side, kips

A third theoretical limit on $T$ is the product of the area and yield stress of the steel section. However, this limit is redundant in view of practical limitations on slab reinforcement.

The nominal plastic moment resistance of a composite section in negative bending is given by the following equation:

$$M_n = T(d_1 + d_2) + P_{yc}(d_3 - d_2) \quad \text{(C-I3-10)}$$

where

$P_{yc}$ = the compressive strength of the steel section; for a non-hybrid section $P_{yc} = A_s F_y$ , kips

$d_1$ = distance from the centroid of the longitudinal slab reinforcement to the top of the steel section, in.

$d_2$ = distance from the centroid of the tension force in the steel section to the top of the steel section, in.

$d_3$ = distance from $P_{yc}$ to the top of the steel section, in.

*Transverse Reinforcement for the Slab.* Where experience has shown that longitudinal cracking detrimental to serviceability is likely to occur, the slab should be reinforced in the direction transverse to the supporting steel section. It is recommended that the area of such reinforcement should be at least 0.002 times the concrete area in the longitudinal direction of the beam and should be uniformly distributed.

## 3. Strength of Concrete-Encased Beams

Tests of concrete-encased beams demonstrated that (1) the encasement drastically reduces the possibility of lateral-torsional instability and prevents local buckling of the encased steel, (2) the restrictions imposed on the encasement

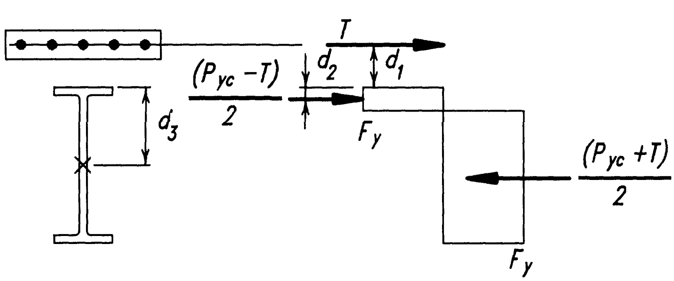



*Fig. C-I3.2. Plastic stress distribution for negative moment.*

practically prevent bond failure prior to first yielding of the steel section, and (3) bond failure does not necessarily limit the moment capacity of an encased steel beam (ASCE, 1979). Accordingly, the LRFD Specification permits two alternate design methods: one based on the first yield in the tension flange of the composite section and the other based on the plastic moment capacity of the steel beam alone. No limitations are placed on the slenderness of either the composite beam or the elements of the steel section, since the encasement effectively inhibits both local and lateral buckling.

In the method based on first yield, stresses on the steel section from permanent loads applied to unshored beams before the concrete has hardened must be superimposed on stresses on the composite section from loads applied to the beams after hardening of the concrete. In this superposition, all permanent loads should be multiplied by the dead load factor and all live loads should be multiplied by the live load factor. For shored beams, all loads may be assumed as resisted by the composite section. Complete interaction (no slip) between the concrete and steel is assumed.

The contribution of concrete to the strength of the composite section is ordinarily larger in positive moment regions than in negative moment regions. Accordingly, design based on the composite section is more advantageous in the regions of positive moments.

## 4. Strength During Construction

When temporary shores are not used during construction, the steel beam alone must resist all loads applied before the concrete has hardened enough to provide composite action. Unshored beam deflection caused by wet concrete tends to increase slab thickness and dead load. For longer spans this may lead to instability analogous to roof ponding. An excessive increase of slab thickness may be avoided by beam camber.

When forms are not attached to the top flange, lateral bracing of the steel beam during construction may not be continuous and the unbraced length may control flexural strength, as defined in Section F1.

The LRFD Specification does not include special requirements for a margin against yield during construction. According to Section F1, maximum factored moment during construction is $0.90F_yZ$ where $F_yZ$ is the plastic moment ($0.90F_yZ \approx 0.90 \times 1.1F_yS$). This is equivalent to approximately the yield moment, $F_yS$. Hence, required flexural strength during construction prevents moment in excess of the yield moment.

Load factors for construction loads should be determined for individual projects according to local conditions, with the factors listed in Section A4 as a guide. Once the concrete has hardened, slab weight becomes a permanent dead load and the dead load factor applies to any load combinations.

## 5. Formed Steel Deck

Figure C-I3.3 is a graphic presentation of the terminology used in Section I3.5.

When studs are used on beams with formed steel deck, they may be welded directly through the deck or through prepunched or cut-in-place holes in the deck. The usual procedure is to install studs by welding directly through the deck; however, when the deck thickness is greater than 16 gage for single thickness, or 18 gage for each sheet of double thickness, or when the total thickness of galvanized coating is greater than 1.25 ounces/sq. ft, special precautions and procedures recommended by the stud manufacturer should be followed.

The design rules for composite construction with formed steel deck are based upon a study (Grant, et al., 1977) of the then available test results. The limiting parameters listed in Section I3.5 were established to keep composite construction with formed steel deck within the available research data.

Seventeen full size composite beams with concrete slab on formed steel deck

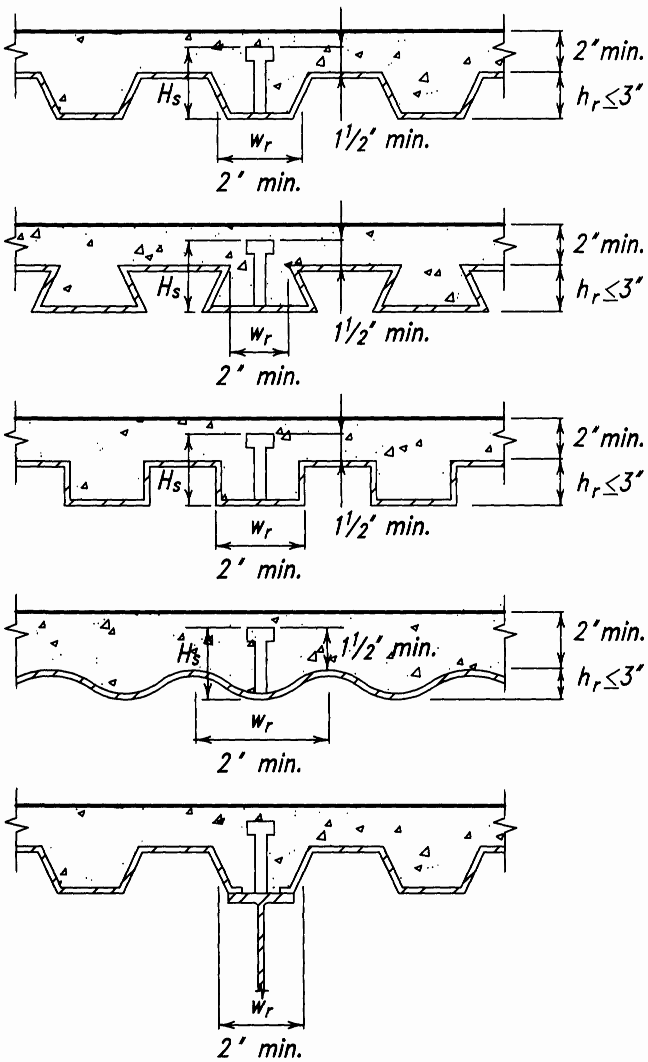



*Fig. C-I3.3. Steel deck limits.*

were tested at Lehigh University and the results supplemented by the results of 58 tests performed elsewhere. The range of stud and steel deck dimensions encompassed by the 75 tests were limited to:

(1) Stud dimensions: ¾-in. dia. × 3.00 to 7.00 in.

(2) Rib width: 1.94 in. to 7.25 in.

(3) Rib height: 0.88 in. to 3.00 in.

(4) Ratio $w_r / h_r$: 1.30 to 3.33

(5) Ratio $H_s / h_r$: 1.50 to 3.41

(6) Number of studs in any one rib: 1, 2, or 3

The strength of stud connectors installed in the ribs of concrete slabs on formed steel deck with the ribs oriented perpendicular to the steel beam is reasonably estimated by the strength of stud connectors in flat soffit composite slabs multiplied by values computed from Equation I3-1.

For the case where ribs run parallel to the beam, limited testing (Grant et al., 1977) has shown that shear connection is not significantly affected by the ribs. However, for narrow ribs, where the ratio $w_r / h_r$ is less than 1.5, a shear stud reduction factor, Equation I3-2, has been employed in view of lack of test data.

The Lehigh study (Grant et al., 1977) also indicated that Equation C-I3-7 for effective section modulus and Equation C-I3-6 for effective moment of inertia were valid for composite construction with formed steel deck.

Based on the Lehigh test data (Grant, et al., 1977), the maximum spacing of steel deck anchorage to resist uplift was increased from 16 to 18 inches in order to accommodate current production profiles.

When metal deck includes units for carrying electrical wiring, crossover headers are commonly installed over the cellular deck perpendicular to the ribs. They create trenches which completely or partially replace sections of the concrete slab above the deck. These trenches, running parallel to or transverse to a composite beam, may reduce the effectiveness of the concrete flange. Without special provisions to replace the concrete displaced by the trench, the trench should be considered as a complete structural discontinuity in the concrete flange.

When trenches are parallel to the composite beam, the effective flange width should be determined from the known position of the trench.

Trenches oriented transverse to composite beams should, if possible, be located in areas of low bending moment and the full required number of studs should be placed between the trench and the point of maximum positive moment. Where the trench cannot be located in an area of low moment, the beam should be designed as non-composite.

## 6. Design Shear Strength

A conservative approach to vertical shear provisions for composite beams is adopted by assigning all shear to the steel section web. This neglects any concrete slab contribution and serves to simplify the design.

## I4. COMBINED COMPRESSION AND FLEXURE

The procedure adopted for the design of beam-columns is described and supported by comparisons with test data in Galambos and Chapuis (1980). The basic approach is identical to that specified for steel columns in Section H1.

The nominal axial strength of a beam-column is obtained from Section I2.2, while the nominal flexural strength is determined from the plastic stress distribution on the composite section. An approximate formula for this plastic moment resistance of a composite column is given in Galambos and Chapuis (1980).

$$M_n = M_p = ZF_y + \tfrac{1}{3}(h_2 - 2c_r)\,A_rF_{yr} + \left(\frac{h_2}{2} - \frac{A_wF_y}{1.7f_c'h_1}\right)A_wF_y \qquad \text{(C-I4-1)}$$

where

$A_w$ = web area of encased steel shape; for concrete-filled tubes, $A_w = 0$, in.$^2$
$Z$ = plastic section modulus of the steel section, in.$^3$
$c_r$ = average of distance from compression face to longitudinal reinforcement in that face and distance from tension face to longitudinal reinforcement in that face, in.
$h_1$ = width of composite cross section perpendicular to the plane of bending, in.
$h_2$ = width of composite cross section parallel to the plane of bending, in.

The supporting comparisons with beam-column tests included 48 concrete-filled pipes or tubing and 44 concrete-encased steel shapes (Galambos and Chapuis, 1980). The overall mean test-to-prediction ratio was 1.23 and the coefficient of variation 0.21.

The last paragraph in Section I4 provides a transition from beam-columns to beams. It involves bond between the steel section and concrete. Section I3 for beams requires either shear connectors or full, properly reinforced encasement of the steel section. Furthermore, even with full encasement, it is assumed that bond is capable of developing only the moment at first yielding in the steel of the composite section. No test data are available on the loss of bond in composite beam-columns. However, consideration of tensile cracking of concrete suggests $P_u / \phi_c P_n = 0.3$ as a conservative limit. It is assumed that when $P_u / \phi_c P_n$ is less than 0.3, the nominal flexural strength is reduced below that indicated by plastic stress distribution on the composite cross section unless the transfer of shear from the concrete to the steel is provided for by shear connectors.

## I5. SHEAR CONNECTORS

### 1. Materials

Tests (Ollgaard et al., 1971) have shown that fully composite beams with concrete meeting the requirements of Part 3, Chapter 4, "Concrete Quality," of ACI (1989), made with ASTM C33 or rotary-kiln produced C330 aggregates, develop full flexural capacity.

## 2. Horizontal Shear Force

Composite beams in which the longitudinal spacing of shear connectors was varied according to the intensity of statical shear, and duplicate beams in which the connectors were uniformly spaced, exhibited the same ultimate strength and the same amount of deflection at normal working loads. Only a slight deformation in the concrete and the more heavily stressed connectors is needed to redistribute the horizontal shear to other less heavily stressed connectors. The important consideration is that the total number of connectors be sufficient to develop the shear $V_h$ on either side of the point of maximum moment. The provisions of the LRFD Specification are based upon this concept of composite action.

In computing the design flexural strength at points of maximum negative bending, reinforcement parallel to the steel beam within the effective width of the slab may be included, provided such reinforcement is properly anchored beyond the region of negative moment. However, enough shear connectors are required to transfer, from the slab to the steel beam, the ultimate tensile force in the reinforcement.

## 3. Strength of Stud Shear Connectors

Studies have defined stud shear connector strength in terms of normal weight and lightweight aggregate concretes as a function of both concrete modulus of elasticity and concrete strength as given by Equation I5-1.

Equation I5-1, obtained from Ollgaard, et al. (1971), corresponds to Tables I4.1 and I4.2 in Section I4 of the 1989 AISC ASD Specification. Note that an upper bound on stud shear strength is the product of the cross-sectional area of the stud times its ultimate tensile strength.

The LRFD Specification does not specify a resistance factor for shear connector strength. The resistance factor for the flexural strength of a composite beam accounts for all sources of variability, including those associated with the shear connectors.

## 4. Strength of Channel Shear Connectors

Equation I5-2 is a modified form of the formula for the strength of channel connectors developed by Slutter and Driscoll (1965). The modification has extended its use to lightweight concrete.

## 6. Shear Connector Placement and Spacing

Uniform spacing of shear connectors is permitted except in the presence of heavy concentrated loads.

When stud shear connectors are installed on beams with formed steel deck, concrete cover at the sides of studs adjacent to sides of steel ribs is not critical. Tests have shown that studs installed as close as is permitted to accomplish welding of studs does not reduce the composite beam capacity.

Studs not located directly over the web of a beam tend to tear out of a thin flange before attaining full shear-resisting capacity. To guard against this contingency, the size of a stud not located over the beam web is limited to 2½ times the flange thickness (Goble, 1968).

The minimum spacing of connectors along the length of the beam, in both flat soffit concrete slabs and in formed steel deck with ribs parallel to the beam, is six diameters; this spacing reflects development of shear planes in the concrete slab (Ollgaard et al., 1971). Since most test data are based on the minimum transverse spacing of four diameters, this transverse spacing was set as the minimum permitted. If the steel beam flange is narrow, this spacing requirement may be achieved by staggering the studs with a minimum transverse spacing of three diameters between the staggered row of studs. The reduction in connector capacity in the ribs of formed steel decks is provided by the factor $0.85 / \sqrt{N_r}$, which accounts for the reduced capacity of multiple connectors, including the effect of spacing. When deck ribs are parallel to the beam and the design requires more studs than can be placed in the rib, the deck may be split so that adequate spacing is available for stud installation. Figure C-I5.1 shows possible connector arrangements.

## I6. SPECIAL CASES

Tests are required for construction that falls outside the limits given in the Specification. Different types of shear connectors may require different spacing and other detailing than stud and channel connectors.

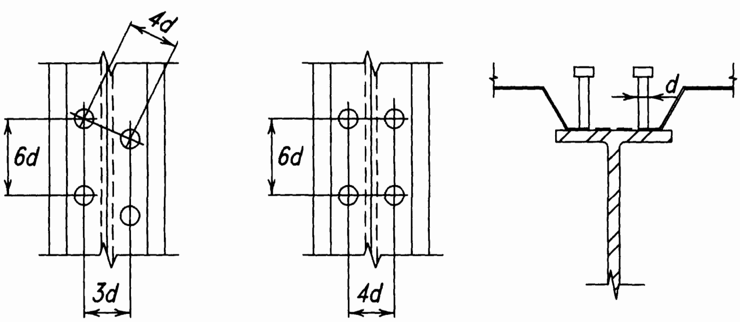



*Fig. C-I5.1. Shear connector arrangements.*

# CHAPTER J

# CONNECTIONS, JOINTS, AND FASTENERS

## J1. GENERAL PROVISIONS

### 5. Splices in Heavy Sections

Solidified but still-hot weld metal contracts significantly as it cools to ambient temperature. Shrinkage of large welds between elements which are not free to move to accommodate the shrinkage, causes strains in the material adjacent to the weld that can exceed the yield point strain. In thick material, the weld shrinkage is restrained in the thickness direction as well as in the width and length directions, causing triaxial stresses to develop that may inhibit the ability of ductile steel to deform in a ductile manner. Under these conditions, the possibility of brittle fracture increases.

When splicing ASTM A6 Group 4 and 5 rolled sections or heavy welded built-up members, the potentially harmful weld shrinkage strains can be avoided by using bolted splices or fillet-welded lap splices or splices that combine a welded and bolted detail (see Figure C-J1.1). Details and techniques that perform well for materials of modest thickness usually must be changed or supplemented by more demanding requirements when welding thick material. Also, the provisions of the *Structural Welding Code,* AWS D1.1, are minimum requirements that apply to most structural welding situations; however, when designing and fabricating welded splices of ASTM A6 Group 4 and 5 shapes and similar built-up cross sections, special consideration must be given to all aspects of the welded splice detail.

- Notch-toughness requirements should be specified for tension members. See Commentary A3.
- Generously sized weld access holes, Figure C-J1.2, are required to provide increased relief from concentrated weld shrinkage strains, to avoid close juncture of welds in orthogonal directions, and to provide adequate clearance for the exercise of high quality workmanship in hole preparation, welding, and ease of inspection.
- Preheating for thermal cutting is required to minimize the formation of a hard surface layer.
- Grinding to bright metal and inspection using magnetic particle or dye-penetrant methods is required to remove the hard surface layer and to assure smooth transitions free of notches or cracks.

In addition to tension splices of truss chord members and tension flanges of flexural members, other joints fabricated of heavy sections subject to tension should be given special consideration during design and fabrication.

## 8. Placement of Welds and Bolts

Slight eccentricities between the gravity axis of single and double angle members and the center of gravity of connecting rivets or bolts have long been ignored as having negligible effect on the static strength of such members. Tests (Gibson and Wake, 1942) have shown that similar practice is warranted in the case of welded members in statically loaded structures.

However, the fatigue life of eccentrically loaded welded angles has been shown to be very short (Kloppel and Seeger, 1964). Notches at the roots of fillet welds are harmful when alternating tensile stresses are normal to the axis of the weld, as could occur due to bending when axial cyclic loading is applied to angles with end welds not balanced about the neutral axis. Accordingly, balanced welds are indicated when such members are subjected to cyclic loading (see Figure C-J1.3).

## 9. Bolts in Combination with Welds

Welds will not share the load equally with mechanical fasteners in bearing-type connections. Before ultimate loading occurs, the fastener will slip and the weld will carry an indeterminately larger share of the load.

Accordingly, the sharing of load between welds and A307 bolts or high-strength bolts in a bearing-type connection is not recommended. For similar reasons, A307 bolts and rivets should not be assumed to share loads in a single group of fasteners.

For high-strength bolts in slip-critical connections to share the load with welds it is advisable to fully tension the bolts before the weld is made. If the weld is placed first, angular distortion from the heat of the weld might prevent the faying action required for development of the slip-critical force. When the bolts are fully tensioned before the weld is made, the slip-critical bolts and the weld may be assumed to share the load on a common shear plane (Kulak, et al., 1987). The heat of welding near bolts will not alter the mechanical properties of the bolts.

In making alterations to existing structures, it is assumed that whatever slip is

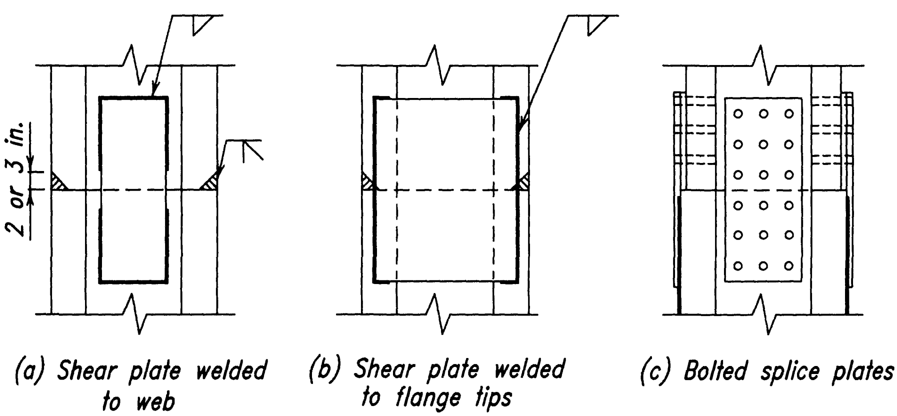
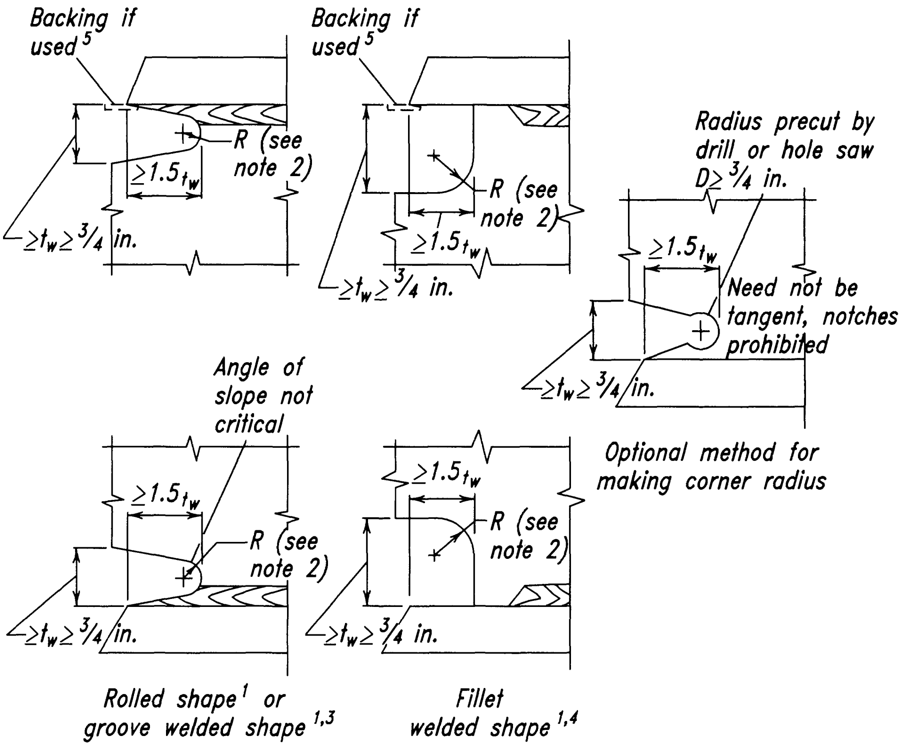



*Fig. C-J1.1. Alternative splices that minimize weld restraint tensile stresses.*

likely to occur in high-strength bolted bearing-type connections or riveted connections will have already taken place. Hence, in such cases the use of welding to resist all stresses, other than those produced by existing dead load present at the time of making the alteration, is permitted.

It should be noted that combinations of fasteners as defined herein does not refer to connections such as shear plates for beam-to-column connections which are welded to the column and bolted to the beam flange or web (Kulak, et al., 1987) and other comparable connections.




*Notes:*

1. *For ASTM A6 Group 4 and 5 shapes and welded built-up shapes with plate thickness more than 2 in., preheat to 150°F prior to thermal cutting, grind and inspect thermally cut edges of access hole using magnetic particle or dye penetration methods prior to making web and flange splice groove welds.*
2. *Radius shall provide smooth notch-free transition;* $R \geq 3/8$*-in. (typical 1/2-in.)*
3. *Access opening made after welding web to flange.*
4. *Access opening made before welding web to flange.*
5. *These are typical details for joints welded from one side against steel backing. Alternative joint designs should be considered.*

*Fig. C-J1.2. Weld access hole and beam cope geometry.*

### 10. High-Strength Bolts in Combination with Rivets

When high-strength bolts are used in combination with rivets, the ductility of the rivets permits the direct addition of the strengths of both fastener types.

## J2. WELDS

### 1. Groove Welds

The engineer preparing contract design drawings cannot specify the depth of groove without knowing the welding process and the position of welding. Accordingly, only the effective throat for partial joint-penetration groove welds should be specified on design drawings, allowing the fabricator to produce this effective throat with his own choice of welding process and position.

The weld reinforcement is not used in determining the effective throat thickness of a groove weld (see Table J2.1).

### 2. Fillet Welds

### 2a. Effective Area

The effective throat of a fillet weld is based upon the root of the joint and the face of the diagrammatic weld, hence this definition gives no credit for weld penetration or reinforcement at the weld face. If the fillet weld is made by the submerged arc welding process, some credit for penetration is made. If the leg size of the resulting fillet weld exceeds ⅜-in., then 0.11 in. is added to the theoretical throat. This increased weld throat is allowed because the submerged arc process produces deep penetration of welds of consistent quality. However, it is necessary to run a short length of fillet weld to be assured that this increased penetration is obtained. In practice, this is usually done initially by cross-sectioning the runoff plates of the joint. Once this is done, no further testing is required, as long as the welding procedure is not changed.

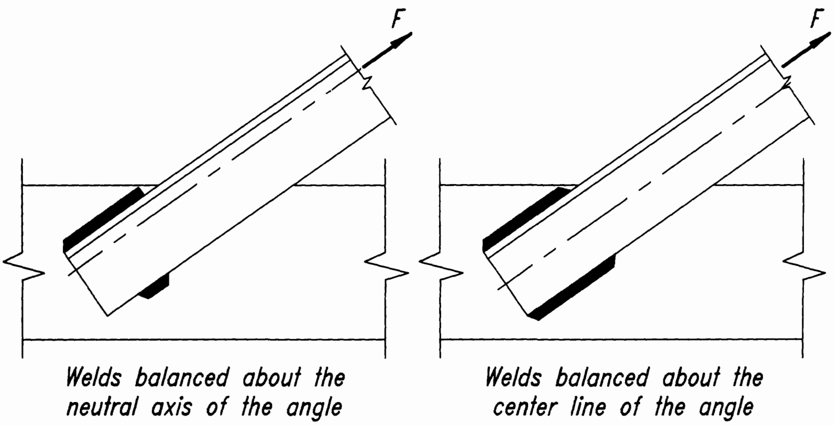



*Figure C-J1.3*

## 2b. Limitations

Table J2.4 provides a minimum size of fillet weld for a given thickness of the thicker part joined.

The requirements are not based upon strength considerations, but upon the quench effect of thick material on small welds. Very rapid cooling of weld metal may result in a loss of ductility. Further, the restraint to weld-metal shrinkage provided by thick material may result in weld cracking. Because a 5⁄16-in. fillet weld is the largest that can be deposited in a single pass by SMAW process, 5⁄16-in. applies to all material 3⁄4-in. and greater in thickness, but minimum preheat and interpass temperature are required by AWS D1.1.* Both the design engineer and the shop welder must be governed by the requirements.

Table J2.3 gives the minimum effective throat of a partial joint-penetration groove weld. Notice that Table J2.3 for partial joint-penetration groove welds goes up to a plate thickness of over 6 in. and a minimum weld throat of 5⁄8-in., whereas, for fillet welds Table J2.4 goes up to a plate thickness of over 3⁄4-in. and a minimum leg size of fillet weld of only 5⁄16-in. The additional thickness for partial-penetration welds is to provide for reasonable proportionality between weld and material thickness.

For plates of 1⁄4-in. or more in thickness, it is necessary that the inspector be able to identify the edge of the plate to position the weld gage. This is assured if the weld is kept back at least 1⁄16-in. from the edge, as shown in Figure C-J2.1.

Where longitudinal fillet welds are used alone in a connection (see Figure C-J2.2), Section J2.2b requires the length of each weld to be at least equal to the width of the connecting material because of shear lag (Fisher, et al., 1978).

By providing a minimum lap of five times the thickness of the thinner part of a lap joint, the resulting rotation of the joint when pulled will not be excessive, as shown in Figure C-J2.3. Fillet welded lap joints under tension tend to open and

* See Table J2.4.

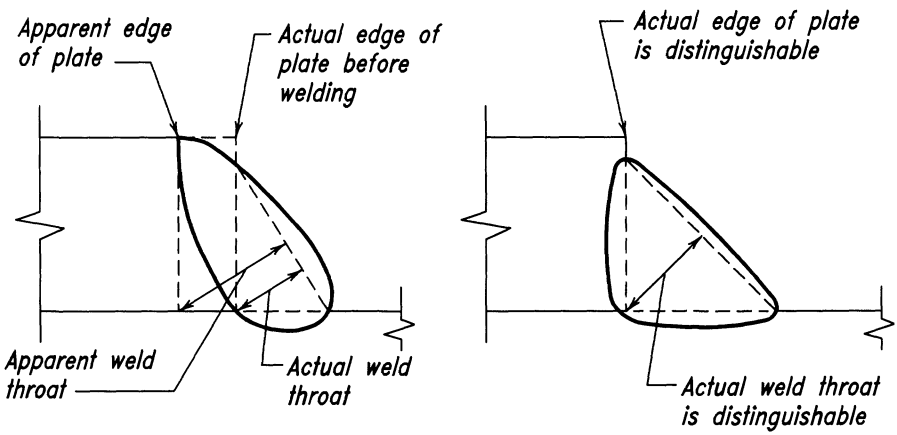



*Fig. C-J2.1. Identification of plate edge.*

apply a tearing action at the root of the weld as shown in Figure C-J2.4b, unless restrained by a force *F* as shown in Figure C-J2.4a.

End returns are not essential for developing the capacity of fillet welded connections and have a negligible effect on their strength. Their use has been encouraged to insure that the weld size is maintained over the length of the weld, to enhance the fatigue resistance of cyclically loaded flexible end connections, and to increase the plastic deformation capability of such connections.

The weld capacity database on which the specifications were developed had no end returns. This includes the study by Higgins and Preece (1968), seat angle tests by Lyse and Schreiner (1935), the seat and top angle tests by Lyse and Gibson (1937), beam webs welded directly to column or girder by fillet welds by Johnston and Deits (1941), and the eccentrically loaded welded connections reported by Butler, Pal, and Kulak (1972). Hence, the current design-resistance values and joint-capacity models do not require end returns, when the required weld size is provided. Johnston and Green (1940) noted that movement consistent with the design assumption of no end restraint (i.e., joint flexibility) was

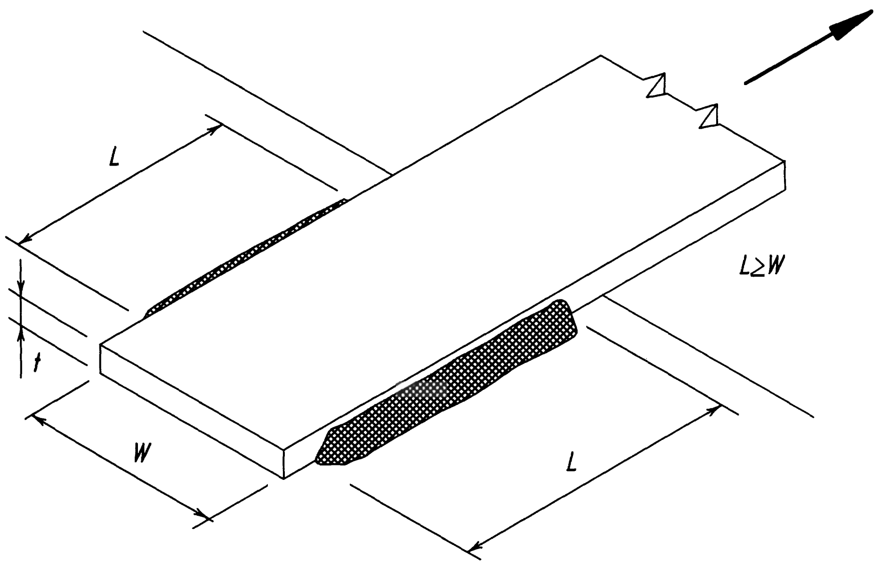



*Fig. C-J2.2. Longitudinal fillet welds.*

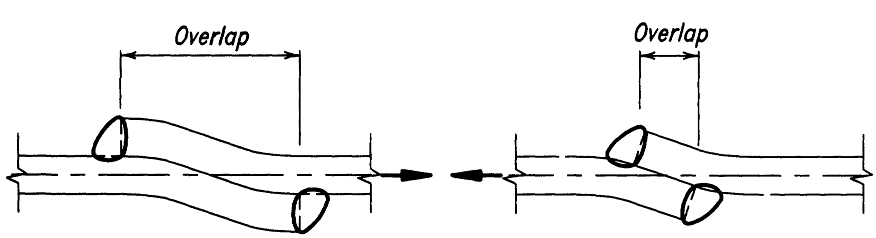



*Fig. C-J2.3. Minimum lap.*

enhanced without end returns. They also verified that greater plastic deformation of the connection was achieved when end returns existed, although the strength was not significantly different.

There are numerous welded joints where it is not possible to provide end returns and where it is also possible to provide the desired weld size. These joints as well as the seat angle and the web angle connections cited earlier do not require end returns when the weld size is adequate and fatigue is not a design consideration.

## 4. Design Strength

The strength of welds is governed by the strength of either the base material or the deposited weld metal. Table J2.5 contains the resistance factors and nominal weld strengths, as well as a number of limitations.

It should be noted that in Table J2.5 the nominal strength of fillet welds is determined from the effective throat area, whereas the strength of the connected parts is governed by their respective thicknesses. Figure C-J2.5 illustrates the shear planes for fillet welds and base material:

(a) Plane 1-1, in which the resistance is governed by the shear strength for material A.

(b) Plane 2-2, in which the resistance is governed by the shear strength of the weld metal.

(c) Plane 3-3, in which the resistance is governed by the shear strength of the material B.

The resistance of the welded joint is the lowest of the resistance calculated in each plane of shear transfer. Note that planes 1-1 and 3-3 are positioned away from the fusion areas between the weld and the base material. Tests have

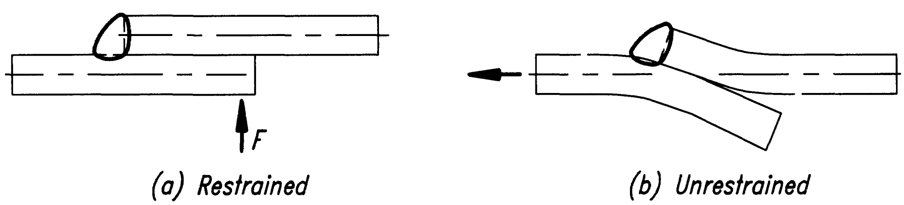



*Fig. C-J2.4. Restraint of lap joints.*

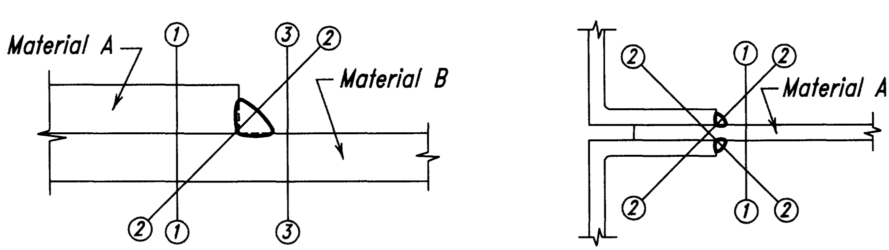



*Fig. C-J2.5. Shear planes for fillet welds loaded in longitudinal shear.*

demonstrated that the stress on this fusion area is not critical in determining the shear strength of fillet welds (Preece, 1968).

The shear planes for plug and partial penetration groove welds are shown in Figure C-J2.6 for the weld and base metal. Generally the base metal will govern the shear strength.

### 5. Combination of Welds

This method of adding weld strengths does not apply to a welded joint using a partial-penetration single bevel groove weld with a superimposed fillet weld. In this case, the effective throat of the combined joint must be determined and the design strength based upon this throat area.

### 7. Mixed Weld Metal

Problems can occur when incompatible weld metals are used in combination and notch-tough composite weld metal is required. For instance, tack welds deposited using a self-shielded process with aluminum deoxidizers in the electrodes and subsequently covered by SAW weld passes can result in composite weld metal with low notch-toughness, despite the fact that each process by itself could provide notch-tough weld metal.

## J3. BOLTS AND THREADED PARTS

### 1. High-Strength Bolts

In general, the use of high-strength bolts is required to conform to the provisions of the *Load and Resistance Factor Design Specification for Structural Joints Using ASTM A325 or A490 Bolts* (RCSC, 1988) as approved by the Research Council on Structural Connections.

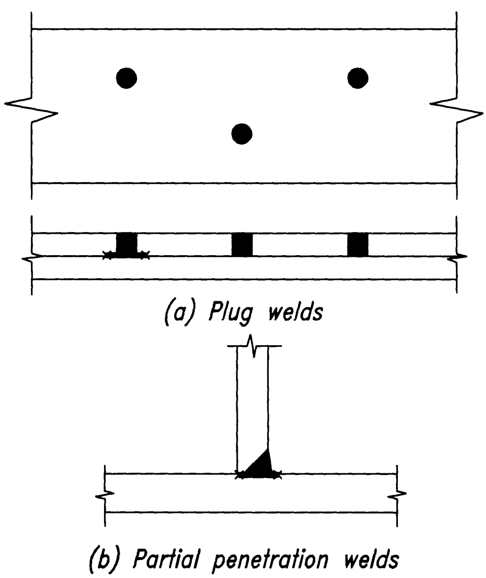



*Fig. C-J2.6. Shear planes for plug and partial-penetration welds.*

Occasionally the need arises for the use of high-strength bolts of diameters and lengths in excess of those available for A325 and A490 bolts, as for example, anchor bolts for fastening machine bases. For this situation Section A3.3 permits the use of A449 bolts and A354 threaded rods.

## 2. Size and Use of Holes

To provide some latitude for adjustment in plumbing up a frame during erection, three types of enlarged holes are permitted, subject to the approval of the designer. The nominal maximum sizes of these holes are given in Table J3.3.

The use of these enlarged holes is restricted to connections assembled with bolts and is subject to the provisions of Sections J3.3 and J3.4.

## 3. Minimum Spacing

The *maximum* factored strength $R_n$ at a bolt or rivet hole in bearing requires that the distance between the centerline of the first fastener and the edge of a plate toward which the force is directed should not be less than $1\frac{1}{2}d$, where $d$ is the fastener diameter (Kulak et al., 1987). By similar reasoning the distance measured in the line of force, from the centerline of any fastener to the nearest edge of an adjacent hole, should not be less than $3d$, to ensure maximum design strength in bearing. Plotting of numerous test results indicates that the critical bearing strength is directly proportional to the above defined distances up to a maximum value of $3d$, above which no additional bearing strength is achieved (Kulak et al., 1987). Table J3.7 lists the increments that must be added to adjust the spacing upward to compensate for an increase in hole dimension parallel to the line of force. Section J3.10 gives the bearing strength criteria as a function of spacing.

## 4. Minimum Edge Distance

Critical bearing stress is a function of the material tensile strength, the spacing of fasteners, and the distance from the edge of the part to the center line of the nearest fastener. Tests have shown (Kulak et al., 1987) that a linear relationship exists between the ratio of critical bearing stress to tensile strength (of the connected material) and the ratio of fastener spacing (in the line of force) to fastener diameter. The following equation affords a good lower bound to published test data for single-fastener connections with standard holes, and is conservative for adequately spaced multi-fastener connections:

$$\frac{F_{pcr}}{F_u} = \frac{l_e}{d} \tag{C-J3-1}$$

where

$F_{pcr}$ = critical bearing stress, ksi
$F_u$ = tensile strength of the connected material, ksi
$l_e$ = distance, along a line of transmitted force, from the center of a fastener to the nearest edge of an adjacent fastener or to the free edge of a connected part (in the direction of stress), in.
$d$ = diameter of fastener, in.

The provisions of Section J3.3 are concerned with $l_e$ as hole spacing, whereas Section J3.4 is concerned with $l_e$ as edge distance in the direction of stress.

Section J3.10 establishes a maximum bearing strength. Spacing and/or edge distance may be increased to provide for a required bearing strength, or bearing force may be reduced to satisfy a spacing and/or edge distance limitation.

It has long been known that the critical bearing stress of a single fastener connection is more dependent upon a given edge distance than multi-fastener connections (Jones, 1940). For this reason, longer edge distances (in the direction of force) are required for connections with one fastener in the line of transmitted force than required for those having two or more.

The recommended minimum distance transverse to the direction of load is primarily a workmanship tolerance. It has little, if any, effect on the strength of the member.

## 5. Maximum Spacing and Edge Distance

Limiting the edge distance to not more than 12 times the thickness of an outside connected part, but not more than six inches, is intended to provide for the exclusion of moisture in the event of paint failure, thus preventing corrosion between the parts which might accumulate and force these parts to separate. More restrictive limitations are required for connected parts of unpainted weathering steel exposed to atmospheric corrosion.

## 6. Design Tension or Shear Strength

Tension loading of fasteners is usually accompanied by some bending due to the deformation of the connected parts. Hence, the resistance factor $\phi$, by which $R_n$ is multiplied to obtain the design tensile strength of fasteners, is relatively low. The nominal tensile strength values in Table J3.2 were obtained from the equation

$$R_n = 0.75 A_b F_u \tag{C-J3-2}$$

While the equation was developed for bolted connections (Kulak et al., 1987), it was also conservatively applied to threaded parts and to rivets. The nominal strength of A307 bolts was discounted by 5 ksi.

In connections consisting of only a few fasteners, the effects of strain on the shear in bearing fasteners is negligible (Kulak et al., 1987; Fisher et al., 1978). In longer joints, the differential strain produces an uneven distribution between fasteners (those near the end taking a disproportionate part of the total load), so that the maximum strength per fastener is reduced. The AISC ASD Specification permits connections up to 50 in. in length without a reduction in maximum shear stress. With this in mind the resistance factor $\phi$ for shear in bearing-type connections has been selected to accommodate the same range of connections.

The values of nominal shear strength in Table J3.2 were obtained from the equation

$$R_n / m A_b = 0.50 F_u \tag{C-J3-3}$$

when threads are excluded from the shear planes and

$$R_n / m A_b = 0.40 F_u \tag{C-J3-4}$$

when threads are not excluded from the shear plane, where $m$ is the number of shear planes (Kulak et al., 1987). While developed for bolted connections, the equations were also conservatively applied to threaded parts and rivets. The value given for A307 bolts was obtained from Equation C-J3-4 but is specified for all cases regardless of the position of threads. For A325 bolts, no distinction is made between small and large diameters, even though the minimum tensile strength $F_u$ is lower for bolts with diameters in excess of one inch. It was felt that such a refinement of design was not justified, particularly in view of the low resistance factor $\phi$, increasing ratio of tensile area to gross area and other compensating factors.

## 7. Combined Tension and Shear in Bearing-Type Connections

Tests have shown that the strength of bearing fasteners subject to combined shear and tension resulting from externally applied forces can be closely defined by an ellipse (Kulak et al., 1987). Such a curve can be replaced, with only minor deviations, by three straight lines as shown in Figure C-J3.1. This latter representation offers the advantage that no modification of either type stress is required in the presence of fairly large magnitudes of other types. This linear representation was adopted for Table J3.5, giving a limiting tensile stress $F_t$ as a function of the shearing stress $f_v$ for bearing-type connections.

## 8. High-Strength Bolts in Slip-Critical Connections

Connections classified as slip-critical include those cases where slip could theoretically exceed an amount deemed by the Engineer of Record to affect the suitability for service of the structure by excessive distortion or reduction in strength or stability, even though the nominal strength of the connection may be adequate. Also included are those cases where slip of any magnitude must be prevented, for example, joints subject to fatigue, connectors between elements of built-up members at their ends (Sections D2 and E4), and bolts in combination with welds (Section J1.9).

The onset of slipping in a high-strength bolted, slip-critical connection is not an

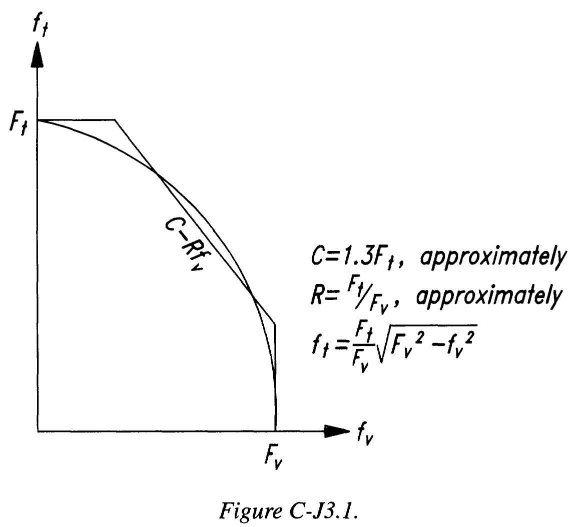



*Figure C-J3.1.*

indication that maximum capacity of the connection has been reached. Its occurrence may be only a serviceability limit state. In the case of bolts in holes with only small clearance, such as standard holes and slotted holes loaded transverse to the axis of the slot in practical connections, the freedom to slip generally does not exist because one or more bolts are in bearing even before load is applied due to normal fabrication tolerances and erection procedures. Further, the consequences of slip, if it can occur at all, are trivial except for a few situations as noted above.

Slip of slip-critical connections is likely to occur at approximately 1.4 to 1.5 times the service loads. For standard holes, oversized holes, and short slotted holes the connection can be designed either at service loads (Section J3.8a) or at factored loads (Appendix J3.8b). The nominal loads and $\phi$ factors have been adjusted accordingly. The number of connectors will be essentially the same for the two procedures because they have been calibrated to give similar results. Slight differences will occur because of variation in the ratio of live load to dead load.

In connections containing long slots that are parallel to the direction of the applied load, slip of the connection prior to attainment of the factored load might be large enough to alter the usual assumption of analysis that the undeformed structure can be used to obtain the internal forces. To guard against this occurring, the design slip resistance is further reduced by 0.85 when designing at service load (Section J3.8a) and by setting $\phi$ to 0.60 in conjunction with factored loads (Appendix J3.8b).

While the possibility of a slip-critical connection slipping into bearing under anticipated service conditions is small, such connections must comply with the provisions of Section J3.10 in order to prevent connection failure at the maximum load condition.

## 10. Bearing Strength at Bolt Holes

The recommended bearing stress on pins is not the same as for bolts as explained in Section J8.

Bearing values are not provided as a protection to the fastener, because it needs no such protection. Therefore, the same bearing value applies to joints assembled by bolts, regardless of fastener shear strength or the presence or absence of threads in the bearing area.

Tests (Frank and Yura, 1981) have demonstrated that hole elongation greater than 0.25 in. will begin to develop as the bearing stress is increased beyond the values given in Equations J3-1a and J3-1d, especially if it is combined with high tensile stress on the net section, even though rupture does not occur. Equations J3-1b and J3-1c consider the effect of hole ovalization (deformation greater than 0.25 in.) whenever the upper design limit ($3.0dtF_u$) is deemed acceptable. These latter equations also establish the design limit for a single bolt, or two or more bolts, whenever the bolt arrangement results in each bolt singly in line with the direction of the applied force. Because two separate limit states are considered (deformation and strength) with both limit states equated to a bearing stress ($2.4F_u$ or $2.0F_u$ and $3.0F_u$, respectively) conflicting design strengths may result, either acceptable, when intermediate edge distance and bolt spacing values are considered.

### 11. Long Grips

Provisions requiring a decrease in calculated stress for A307 bolts having long grips (by arbitrarily increasing the required number in proportion to the grip length) are not required for high-strength bolts. Tests (Bendigo et al., 1963) have demonstrated that the ultimate shearing strength of high-strength bolts having a grip of eight or nine diameters is no less than that of similar bolts with much shorter grips.

## J4. DESIGN RUPTURE STRENGTH

Tests (Birkemoe and Gilmor, 1978) on coped beams indicated that a tearing failure mode (rupture) can occur along the perimeter of the bolt holes as shown in Figure C-J4.1. This block shear mode combines tensile strength on one plane and shear strength on a perpendicular plane. The failure path is defined by the center lines of the bolt holes. The block shear failure mode is not limited to the coped ends of beams. Other examples are shown in Figure CJ4.1 and C-J4.2.

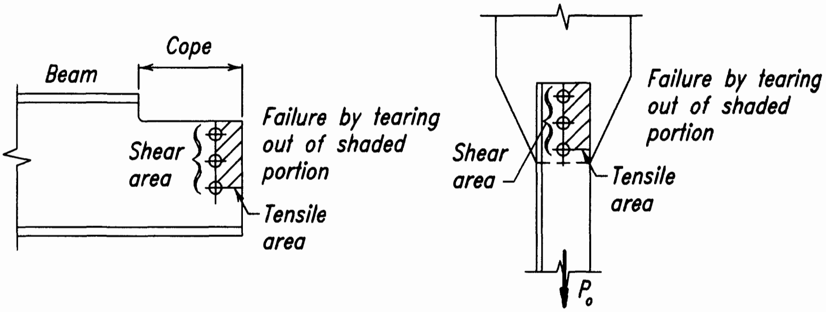



*Fig. C-J4.1. Failure surface for block shear rupture limit state.*

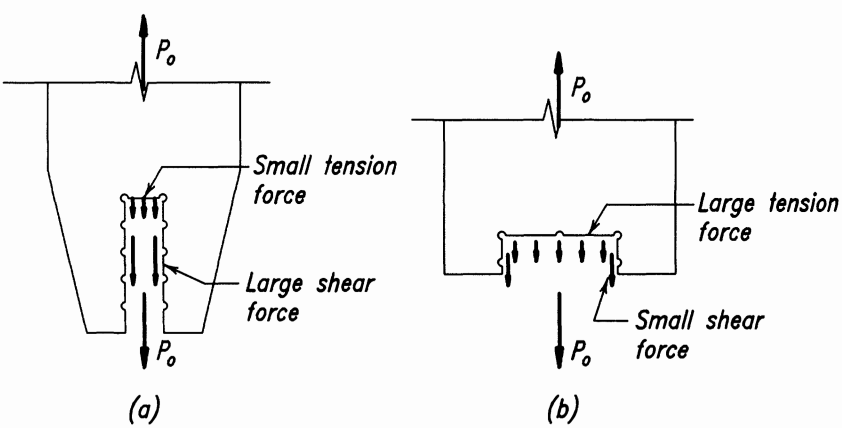



*Fig. C-J4.2 Block shear rupture in tension.*

The block shear failure mode should also be checked around the periphery of welded connections. Welded connection block shear is determined using $\phi = 0.75$ in conjunction with the area of both the fracture and yielding planes (Yura, 1988).

The LRFD Specification has adopted a conservative model to predict block shear strength. Test results suggest that it is reasonable to add the yield strength on one plane to the rupture strength of the perpendicular plane (Ricles and Yura, 1983 and Hardash and Bjorhovde, 1985). Therefore, two possible block shear strengths can be calculated; rupture strength $F_u$ on the net tensile section along with shear yielding $0.6F_y$ on the gross section on the shear plane(s), or rupture $0.6F_u$ on the net shear area(s) combined with yielding $F_y$ on the gross tensile area. This is the basis of Equations J4-3 and J4-4.

These equations are consistent with the philosophy in Chapter D for tension members, where gross area is used for the limit state of yielding and net area is used for rupture. The controlling equation is the one that produces the *larger* rupture force. This can be explained by the two extreme examples given in Figure C-J4.2. In Case a, the total force is resisted primarily by shear, so shear rupture, not shear yielding, should control the block shear tearing mode; therefore, use Equation J4-4. For Case b, block shear cannot occur until the tension area ruptures as given by Equation J4-3. If Equation J4-4 (shear rupture on the small area and yielding on the large tension area) is checked for Case b, a smaller $P_o$ will result. In fact, as the shear area gets smaller and approaches zero, the use of Equation J4-4 for Case b would give a block shear strength based totally on *yielding* of the gross tensile area. Block shear is a rupture or tearing phenomenon not a yielding limit state. Therefore, the proper equation to use is the one with the larger rupture term.

## J5. CONNECTING ELEMENTS

### 2. Design Strength of Connecting Elements in Tension

Tests have shown that yield will occur on the gross section area before the tensile capacity of the net section is reached, if the ratio $A_n / A_g \leq 0.85$ (Kulak et al., 1987). Since the length of connecting elements is small compared to the member length, inelastic deformation of the gross section is limited. Hence, the effective net area $A_n$ of the connecting element is limited to $0.85A_g$ in recognition of the limited inelastic deformation and to provide a reserve capacity.

## J6. FILLERS

The practice of securing fillers by means of additional fasteners, so that they are, in effect, an integral part of a shear-connected component, is not required where a connection is designed to be a slip-critical connection using high-strength bolts. In such connections, the resistance to slip between filler and either connected part is comparable to that which would exist between the connected parts if no fill were present.

Filler plates may be used in lap joints of welded connections that splice parts of different thickness, or where there may be an offset in the joint.

## J8. BEARING STRENGTH

The LRFD Specification provisions for bearing on milled surfaces, Section J8, follow the same philosophy of earlier AISC ASD Specifications. In general, the design is governed by a deformation limit state at service loads resulting in stresses nominally at $\frac{9}{10}$ of yield. Adequate safety is provided by post-yield strength as deformation increases. Tests on pin connections (Johnston, 1939) and on rockers (Wilson, 1934) have confirmed this behavior.

As used throughout the LRFD Specification, the terms "milled surface," "milled," and "milling" are intended to include surfaces which have been accurately sawed or finished to a true plane by any suitable means.

## J9. COLUMN BASES AND BEARING ON CONCRETE

The equations for resistance of concrete in bearing are the same as ACI 318-89 except that AISC equations use $\phi = 0.60$ while ACI uses $\phi = 0.70$, since ACI specifies larger load factors than the ASCE load factors specified by AISC.

## J10. ANCHOR BOLTS AND EMBEDMENTS

ACI 318 and 349 Appendix B and the PCI Handbook include recommended procedures for the design of anchor bolts and embedments.

# CHAPTER K

# CONCENTRATED FORCES, PONDING, AND FATIGUE

## K1. FLANGES AND WEBS WITH CONCENTRATED FORCES

### 1. Design Basis

The LRFD Specification separates flange and web strength requirements into distinct categories representing different limit state criteria, i.e., local flange bending (Section K1.2), local web yielding (Section K1.3), web crippling (Section K1.4), sidesway web buckling (Section K1.5), compression buckling of the web (Section K1.6), and panel zone web shear (Section K1.7).

These criteria are applied to two distinct types of concentrated forces which act on member flanges. *Single concentrated forces* may be tensile, such as those delivered by tension hangers, or compressive, such as those delivered by bearing plates at beam interior positions, reactions at beam ends, and other *bearing* connections. *Double concentratedforces,* one tensile and one compressive, form a couple on the same side of the loaded member, such as that delivered to column flanges through welded and bolted *moment* connections.

### 2. Local Flange Bending

Where a tensile force is applied through a plate welded across a flange, that flange must be sufficiently rigid to prevent deformation of the flange and the corresponding high-stress concentration in the weld in line with the web.

The effective column flange length for local flange bending is $12t_f$ (Graham, et al., 1959). Thus, it is assumed that yield lines form in the flange at $6t_f$ in each direction from the point of the applied concentrated force. To develop the fixed edge consistent with the assumptions of this model, an additional $4t_f$ and therefore a total of $10t_f$, is required for the full flange-bending strength given by Equation K1-1. In the absence of applicable research, a 50 percent reduction has been introduced for cases wherein the applied concentrated force is less than $10t_f$ from the member end.

This criterion given by Equation K1-1 was originally developed for *moment* connections, but it also applies to *single concentrated forces* such as tension hangers consisting of a plate welded to the bottom flange of a beam and transverse to the beam web.

### 3. Local Web Yielding

The web strength criteria have been established to limit the stress in the web of a member into which a force is being transmitted. It should matter little whether the member receiving the force is a beam or a column; however, Galambos (1976) and AISC (1978), references upon which the LRFD Specification is based, did make such a distinction. For beams, a 2:1 stress gradient through the flange was used, whereas the gradient through column flanges was 2½:1. In Section K1.3, the 2½:1 gradient is used for both cases.

This criterion applies to both *bearing* and *moment* connections.

## 4. Web Crippling

The expression for resistance to web crippling at a concentrated force is a departure from previous specifications (IABSE, 1968; Bergfelt, 1971; Hoglund, 1971; and Elgaaly, 1983). Equations K1-4 and K1-5 are based on research by Roberts (1981). The increase in Equation K1-5b for $N/d > 0.2$ was developed after additional testing (Elgaaly, 1991) to better represent the effect of longer bearing lengths at ends of members. All tests were conducted on bare steel beams without the expected beneficial contributions of any connection or floor attachments. Thus, the resulting criteria are considered conservative for such applications.

These equations were developed for *bearing* connections, but are also generally applicable to *moment* connections. However, for the rolled shapes listed in Part 1 of the LRFD Manual with $F_y$ not greater than 50 ksi, the web crippling criterion will never control the design in a *moment* connection except for a W12×50 or W10×33 column.

The web crippling phenomenon has been observed to occur in the web adjacent to the load flange. For this reason, a half-depth stiffener (or stiffeners) or a half-depth doubler plate is expected to eliminate this limit state.

## 5. Sidesway Web Buckling

The sidesway web buckling criterion was developed after observing several unexpected failures in tested beams (Summers and Yura, 1982). In those tests the compression flanges were braced at the concentrated load, the web was squeezed into compression, and the tension flange buckled (see Figure C-K1.1).

Sidesway web buckling will not occur in the following cases. For flanges restrained against rotation:

$$\frac{h/t_w}{l/b_f} > 2.3 \qquad \text{(C-K1-1)}$$

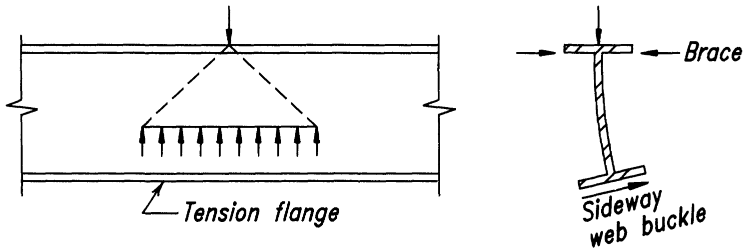



*Fig. C-K1.1. Sidesway web buckling.*

For flanges *not* restrained against rotation:

$$\frac{h/t_w}{l/b_f} > 1.7 \qquad \text{(C-K-1-2)}$$

where $l$ is as shown in Figure C-K1.2.

Sidesway web buckling can also be prevented by the proper design of lateral bracing or stiffeners at the load point. It is suggested that local bracing at both flanges be designed for one percent of the concentrated force applied at that point. Stiffeners must extend from the load point through at least one-half the beam or girder depth. In addition, the pair of stiffeners should be designed to carry the full load. If flange rotation is permitted at the loaded flange, neither stiffeners nor doubler plateswill be effective.

In the 1st Edition LRFD Manual, the sidesway web buckling equations were based on the assumption that $h/t_f = 40$, a convenient assumption which is generally true for economy beams. This assumption has been removed so that the equations will be applicable to all sections.

These equations were developed only for *bearing* connections and do not apply to *moment* connections.

## 6. Compression Buckling of the Web

When compressive forces are applied to both flanges of a member at the same

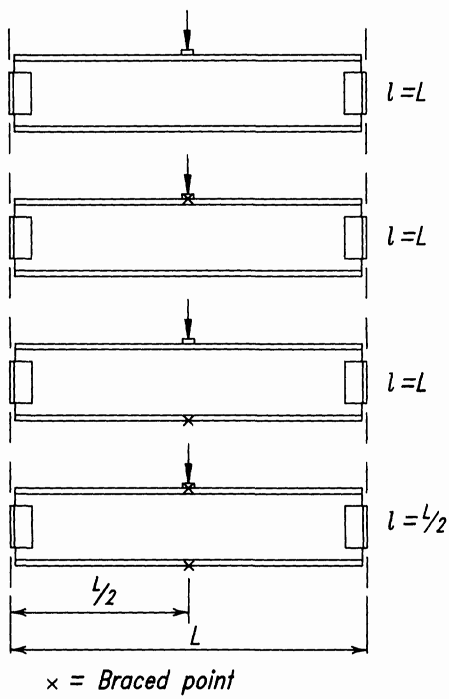



*Fig. C-K1.2. Unbraced flange length.*

location, as by *moment* connections at both flanges of a column, the member web must have its slenderness ratio limited to avoid the possibility of buckling. This is done in the LRFD Specification with Equation K1-8, which is a modified form of a similar equation used in the ASD Specification. This equation is applicable to a pair of *moment* connections, and to other pairs of compressive forces applied at both flanges of a member, for which $N/d$ is small (<1). When $N/d$ is not small, the member web should be designed as a compression member in accordance with Chapter E.

Equation K1-8 is predicated on an interior member loading condition. In the absence of applicable research, a 50 percent reduction has been introduced for cases wherein the compressive forces are close to the member end.

Equation K1-8 has also traditionally been applied when there is a *moment* connection to only one flange of the column and compressive force is applied to only one flange. Its use in this case is conservative.

## 7. Panel Zone Web Shear

The column web shear stresses may be high within the boundaries of the rigid connection of two or more members whose webs lie in a common plane. Such webs should be reinforced when the calculated factored force $\Sigma F$ along plane A-A in Figure C-K1.3 exceeds the column web design strength $\phi R_v$, where

$$\Sigma F_u = \frac{M_{u1}}{d_{m1}} + \frac{M_{u2}}{d_{m2}} - V_u \qquad \text{(C-K1-3)}$$

and

$M_{u1} = M_{u1L} + M_{u1G}$ = the sum of the moments due to the factored lateral load $M_{u1L}$ and the moments due to factored gravity load $M_{u1G}$ on the leeward side of the connection, kip-in.

$M_{u2} = M_{u2L} - M_{u2G}$ = the difference between the moments due to the factored lateral load $M_{u2L}$ and the moments due to factored gravity load $M_{u2G}$ on the windward side of the connection, kip-in.

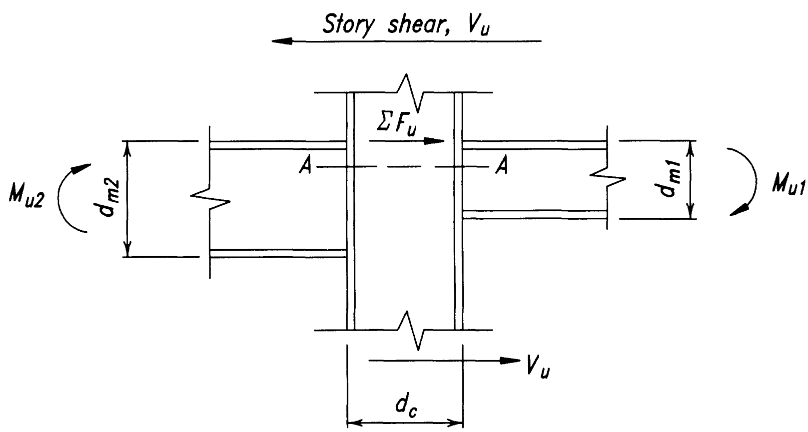



*Fig. C-K1.3. Forces in panel zone.*

$d_{m1}, d_{m2}$ = distance between flange forces in a moment connection, in.

Conservatively, 0.95 times the beam depth has been used for $d_m$ in the past.

If $\Sigma F_u \leq \phi R_v$, no reinforcement is necessary, i.e., $t_{req} \leq t_w$, where $t_w$ is the column web thickness.

Consistent with elastic first order analysis, Equations K1-9 and K1-10 limit panel-zone behavior to the elastic range. While such connection panels possess large reserve capacity beyond initial general shear yielding, the corresponding inelastic joint deformations may adversely affect the strength and stability of the frame or story (Fielding and Huang, 1971 and Fielding and Chen, 1973). Panel-zone shear yielding affects the overall frame stiffness and, therefore, the ultimate-strength second-order effects may be significant. The shear/axial interaction expression of Equation K1-10, as shown in Figure C-K1.4, is chosen to ensure elastic panel behavior.

If adequate connection ductility is provided and the frame analysis considers the inelastic panel-zone deformations, then the additional inelastic shear strength is recognized in Equations K1-11 and K1-12 by the factor

$$\left(1 + \frac{3b_{cf}\,t_{cf}^2}{d_b\,d_c\,t_w}\right)$$

This inelastic shear strength has been most often utilized for design of frames in high seismic zones and should be used when the panel zone is to be designed to match the strength of the members from which it is formed.

The shear/axial interaction expression incorporated in Equation K1-12 (see Figure C-K1.5) is similar to that contained in the previous issue of this specification and recognizes the observed fact that when the panel-zone web has completely yielded in shear, the axial column load is carried in the flanges.

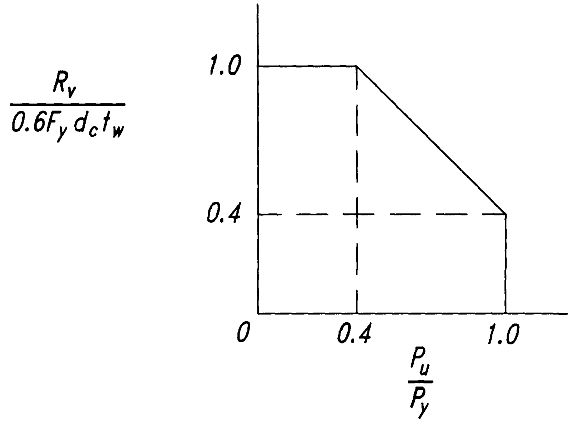



*Fig. C-K1.4. Interaction of shear and axial force—elastic.*

## K2. PONDING

As used in the LRFD Specification, *ponding* refers to the retention of water due solely to the deflection of flat roof framing. The amount of this water is dependent upon the flexibility of the framing. Lacking sufficient framing stiffness, its accumulated weight can result in collapse of the roof if a strength evaluation is not made (ASCE, 1990).

Representing the deflected shape of the primary and critical secondary member as a half-sine wave, the weight and distribution of the ponded water can be estimated and, from this, the contribution that the deflection each of these members makes to the total ponding deflection can be expressed (Marino, 1966):

For the primary member:

$$\Delta_w = \frac{[\alpha_p \Delta_o\ 1 + 0.25\pi\alpha_s + 0.25\pi\rho(1 + \alpha_s)]}{1 - 0.25\pi\alpha_p\alpha_s}$$

For the secondary member:

$$\delta_w = \frac{\left[\alpha_s\delta_o\ 1 + \frac{\pi^3}{32}\alpha_p + \frac{\pi^2}{8\rho}(1 + \alpha_p) + 0.185\alpha_s\alpha_p\right]}{1 - 0.25\pi\alpha_p\alpha_s}$$

In these expressions $\Delta_o$ and $\delta_o$ are, respectively, the primary and secondary beam deflections due to loading present at the initiation of ponding, $\alpha_p = C_p / (1 - C_p)$, $\alpha_s = C_s / (1 - C_s)$, and $\rho = \delta_o / \Delta_o = C_s / C_p$.

Using the above expressions for $\Delta_w$ and $\delta_w$, the ratios $\Delta_w / \Delta_o$ and $\delta_w / \delta_o$ can be computed for any given combination of primary and secondary beam framing using, respectively, the computed value of parameters $C_p$ and $C_s$ defined in the LRFD Specification.

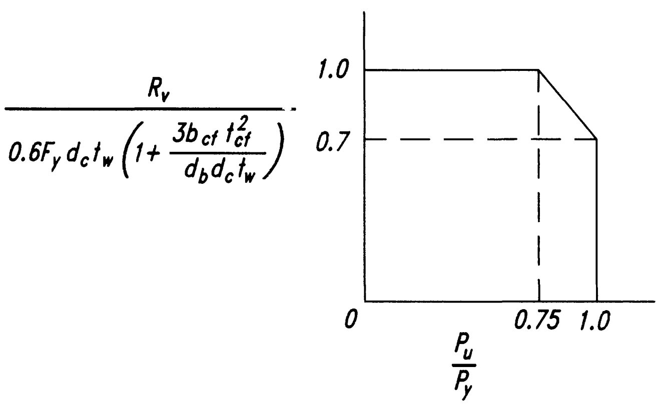



*Fig. C-K1.5. Interaction of shear and axial force—inelastic.*

Even on the basis of unlimited elastic behavior, it is seen that the ponding deflections would become infinitely large unless

$$\left(\frac{C_p}{1-C_p}\right)\left(\frac{C_s}{1-C_s}\right) < \frac{4}{\pi}$$

Since elastic behavior is not unlimited, the effective bending strength available in each member to resist the stress caused by ponding action is restricted to the difference between the yield stress of the member and the stress $f_o$ produced by the total load supported by it before consideration of ponding is included.

Note that elastic deflection is directly proportional to stress. The admissible amount of ponding in either the primary or critical (midspan) secondary member, in terms of the applicable ratio $\Delta_w / \Delta_o$ and $\delta_w / \delta_o$, can be represented as $(F_y - f_o)/f_o$. Substituting this expression for $\Delta_w / \Delta_o$ and $\delta_w / \delta_o$, and combining with the foregoing expressions for $\Delta_w$ and $\delta_w$, the relationship between critical values for $C_p$ and $C_s$ and the available elastic bending strength to resist ponding is obtained. The curves presented in Figures A-K2.1 and A-K2.2 are based upon this relationship. They constitute a design aid for use when a more exact determination of required flat roof framing stiffness is needed than given by the LRFD Specification provision that $C_p + 0.9C_s \leq 0.25$.

Given any combination of primary and secondary framing, the stress index is computed as

$$U_p = \left(\frac{F_y - f_o}{f_o}\right)_p \quad \text{for the primary member}$$

$$U_s = \left(\frac{F_y - f_o}{f_o}\right)_s \quad \text{for the secondary member}$$

where $f_o$, in each case, is the computed bending stress, ksi, in the member due to the supported loading, neglecting ponding effect. Depending upon geographic location, this loading should include such amount of snow as might also be present, although ponding failures have occurred more frequently during torrential summer rains when the rate of precipitation exceeded the rate of drainage runoff and the resulting hydraulic gradient over large roof areas caused substantial accumulation of water some distance from the eaves.

Given the size, spacing, and span of a tentatively selected combination of primary and secondary beams, for example, one may enter Figure A-K2.1 at the level of the computed stress index $U_p$, determined for the primary beam; move horizontally to the computed $C_s$ value of the secondary beams; then move downward to the abscissa scale. The combined stiffness of the primary and secondary framing is sufficient to prevent ponding if the flexibility constant read from this latter scale is more than the value of $C_p$ computed for the given primary member; if not, a stiffer primary or secondary beam, or combination of both, is required.

If the roof framing consists of a series of equally-spaced wall-bearing beams, they would be considered as secondary members, supported on an infinitely stiff primary member. For this case, one would use Figure A-K2.2. The limiting value of $C_s$ would be determined by the intercept of a horizontal line representing the $U_s$ value and the curve for $C_p = 0$.

The ponding deflection contributed by a metal deck is usually such a small part of the total ponding deflection of a roof panel that it is sufficient merely to limit its moment of inertia (in.$^4$ per foot of width normal to its span) to 0.000025 times the fourth power of its span length, as provided in the LRFD Specification. However, the stability against ponding of a roof consisting of a metal roof deck of relatively slender depth-span ratio, spanning between beams supported directly on columns, may need to be checked. This can be done using Figure A-K2.1 or A-K2.2 with the following computed values:

$U_p$ = stress index for the supporting beam
$U_s$ = stress index for the roof deck
$C_p$ = flexibility constant for the supporting beams
$C_s$ = flexibility constant for one foot width of the roof deck ($S$ = 1.0)

Since the shear rigidity of the web system is less than that of a solid plate, the moment of inertia of steel joists and trusses should be taken as somewhat less than that of their chords.

# CHAPTER L

# SERVICEABILITY DESIGN CONSIDERATIONS

Serviceability criteria are formulated to prevent disruptions of the functional use and damage to the structure during its normal everyday use. While malfunctions may not result in the collapse of a structure or in loss of life or injury, they can seriously impair the usefulness of the structure and lead to costly repairs. Neglect of serviceability may result in unacceptably flexible structures.

There are essentially three types of structural behavior which may impair serviceability:

(1) Excessive local damage (local yielding, buckling, slip, or cracking) that may require excessive maintenance or lead to corrosion.

(2) Excessive deflection or rotation that may affect the appearance, function, or drainage of the structure, or may cause damage to nonstructural components and their attachments.

(3) Excessive vibrations induced by wind or transient live loads which affect the comfort of occupants of the structure or the operation of mechanical equipment.

In allowable stress design, the AISC Specification accounts for possible local damage with factors of safety included in the allowable stresses, while deflection and vibration are controlled, directly or indirectly, by limiting deflections and span-depth ratios. In the past, these rules have led to satisfactory performance of structures, with perhaps the exception of large open floor areas without partitions. In LRFD the serviceability checks should consider the appropriate loads, the response of the structure, and the reaction of the occupants to the structural response.

Examples of loads that may require consideration of serviceability include permanent live loads, wind, and earthquake; effects of human activities such as walking, dancing, etc.; temperature fluctuations; and vibrations induced by traffic near the building or by the operation of mechanical equipment within the building.

Serviceability checks are concerned with adequate performance under the appropriate load conditions. Elastic behavior can usually be assumed. However, some structural elements may have to be examined with respect to their long-term behavior under load.

It is difficult to specify limiting values of structural performance based on serviceability considerations because these depend to a great extent on the type of structure, its intended use, and subjective physiological reaction. For example, acceptable structural motion in a hospital clearly would be much less than in an ordinary industrial building. It should be noted that humans perceive levels of structural motion that are far less than motions that would cause any structural damage. Serviceability limits must be determined through careful consideration by the designer and client.

## L1. CAMBER

The engineer should consider camber when deflections at the appropriate load level present a serviceability problem.

## L2. EXPANSION AND CONTRACTION

As in the case of deflections, the satisfactory control of expansion cannot be reduced to a few simple rules, but must depend largely upon the good judgment of qualified engineers.

The problem is more serious in buildings with masonry walls than with prefabricated units. Complete separation of the framing, at widely spaced expansion joints, is generally more satisfactory than more frequently located devices dependent upon the sliding of parts in bearing, and usually less expensive than rocker or roller expansion bearings.

Creep and shrinkage of concrete and yielding of steel are among the causes, other than temperature, for dimensional changes.

## L3. DEFLECTIONS, VIBRATION, AND DRIFT

### 1. Deflections

Excessive transverse deflections or lateral drift may lead to permanent damage to building elements, separation of cladding, or loss of weathertightness, damaging transfer of load to non-load-supporting elements, disruption of operation of building service systems, objectionable changes in appearance of portions of the buildings, and discomfort of occupants.

The LRFD Specification does not provide specific limiting deflections for individual members or structural assemblies. Such limits would depend on the function of the structure (ASCE, 1979; CSA, 1989; Ad Hoc Committee, 1986). Provisions that limit deflections to a percentage of span may not be adequate for certain long-span floor systems; a limit on maximum deflection that is independent of span length may also be necessary to minimize the possibility of damage to adjoining or connecting nonstructural elements.

### 2. Floor Vibration

The increasing use of high-strength materials and efficient structural schemes leads to longer spans and more flexible floor systems. Even though the use of a deflection limit related to span length generally precluded vibration problems in the past, some floor systems may require explicit consideration of the dynamic, as well as the static, characteristics of the floor system.

The dynamic response of structures or structural assemblies may be difficult to analyze because of difficulties in defining the actual mass, stiffness, and damping characteristics. Moreover, different load sources cause varying responses. For example, a steel beam-concrete slab floor system may respond to live loading as a non-composite system, but to transient excitation from human activity as an orthotropic composite plate. Nonstructural partitions, cladding, and built-in furniture significantly increase the stiffness and damping of the structure and frequently eliminate potential vibration problems. The damping can also depend on the amplitude of excitation.

The general objective in minimizing problems associated with excessive structural motion is to limit accelerations, velocities, and displacements to levels that would not be disturbing to the building occupants. Generally, occupants of a building find sustained vibrations more objectionable than transient vibrations.

The levels of peak acceleration that people find annoying depend on frequency of response. Thresholds of annoyance for transient vibrations are somewhat higher and depend on the amount of damping in the floor system. These levels depend on the individual and the activity at the time of excitation (ASCE, 1979; ISO, 1974; CSA, 1989; Murray, 1991; and Ad Hoc Committee, 1986).

The most effective way to reduce effects of continuous vibrations is through vibration isolation devices. Care should be taken to avoid resonance, where the frequency of steady-state excitation is close to the fundamental frequency of the system. Transient vibrations are reduced most effectively by increasing the damping in the structural assembly. Mechanical equipment which can produce objectionable vibrations in any portion of a structure should be adequately isolated to reduce the transmission of such vibrations to critical elements of the structure.

### 3. Drift

The LRFD Specification does not provide specific limiting values for lateral drift. If a drift analysis is desired, the stiffening effect of non-load-supporting elements such as partitions and infilled walls may be included in the analysis of drift.

Some irrecoverable inelastic deformations may occur at given load levels in certain types of construction. The effect of such deformations may be negligible or serious, depending on the function of the structure, and should be considered by the designer on a case by case basis.

The deformation limits should apply to structural assemblies as a whole. Reasonable tolerance should also be provided for creep. Where load cycling occurs, consideration should be given to the possibility of increases in residual deformation that may lead to incremental failure.

## L5. CORROSION

Steel members may deteriorate in particular service environments. This deterioration may appear either in external corrosion, which would be visible upon inspection, or in undetected changes that would reduce its strength. The designer should recognize these problems by either factoring a specific amount of damage tolerance into the design or providing adequate protection systems (e.g., coatings, cathodic protection) and/or planned maintenance programs so that such problems do not occur.

# CHAPTER M

# FABRICATION, ERECTION, AND QUALITY CONTROL

## M2. FABRICATION

### 1. Cambering, Curving, and Straightening

The use of heat for straightening or cambering members is permitted for A514 and A852 steel, as it is for other steels. However, the maximum temperature permitted is 1,100°F compared to 1,200°F for other steels.

The cambering of flexural members, when required by the contract documents, may be accomplished in various ways. In the case of trusses and girders, the desired curvature can be built in during assembly of the component parts. Within limits, rolled beams can be cold-cambered at the producing mills.

The local application of heat has come into common use as a means of straightening or cambering beams and girders. The method depends upon an ultimate shortening of the heat-affected zones. A number of such zones, on the side of the member that would be subject to compression during cold-cambering or "gagging," are heated enough to be "upset" by the restraint provided by surrounding unheated areas. Shortening takes place upon cooling.

While the final curvature or camber can be controlled by these methods, it must be realized that some deviation, due to workmanship error and permanent change due to handling, is inevitable.

### 2. Thermal Cutting

Preferably thermal cutting shall be done by machine. The requirement for a positive preheat of 150°F minimum when thermal cutting beam copes and weld access holes in ASTM A6 Group 4 and 5 shapes, and in built-up shapes made of material more than two inches thick, tends to minimize the hard surface layer and the initiation of cracks.

### 5. Bolted Construction

In the past, it has been required to tighten to a specified tension all ASTM A325 and A490 bolts in both slip-critical and bearing-type connections. The requirement was changed in 1985 to permit most bearing-type connections to be tightened to a snug-tight condition.

In a snug-tight bearing connection, the bolts cannot be subjected to tension loads, slip can be permitted, and loosening or fatigue due to vibration or load fluctuations are not design considerations.

It is suggested that snug-tight bearing-type connections be used in applications where A307 bolts would be permitted.

This section provides rules for the use of oversized and slotted holes paralleling the provisions which have been in the RCSC Specification (RCSC, 1988) since 1972, extended to include A307 bolts which are outside the scope of the high-strength bolt specifications.

## M3. SHOP PAINTING

The surface condition of steel framing disclosed by the demolition of long-standing buildings has been found to be unchanged from the time of its erection, except at isolated spots where leakage may have occurred. Even in the presence of leakage, the shop coat is of minor influence (Bigos et al., 1954).

The LRFD Specification does not define the type of paint to be used when a shop coat is required. Conditions of exposure and individual preference with regard to finish paint are factors which bear on the selection of the proper primer. Hence, a single formulation would not suffice. For a comprehensive treatment of the subject, see SSPC (1989).

### 5. Surfaces Adjacent to Field Welds

The Specification allows for welding through surface materials, including appropriate shop coatings, that do not adversely affect weld quality nor create objectionable fumes.

## M4. ERECTION

### 4. Fit of Column Compression Joints and Base Plates

Tests at the University of California-Berkeley (Popov and Stephen, 1977) on spliced full-size columns with joints that had been intentionally milled out-of-square, relative to either strong or weak axis, demonstrated that the load-carrying capacity was the same as that for a similar unspliced column. In the tests, gaps of $\frac{1}{16}$-in. were not shimmed; gaps of $\frac{1}{4}$-in. were shimmed with non-tapered mild steel shims. Minimum size partial-penetration welds were used in all tests. No tests were performed on specimens with gaps greater than $\frac{1}{4}$-in.

### 5. Field Welding

The purpose of wire brushing shop paint, on surfaces adjacent to joints to be field welded, is to reduce the possibility of porosity and cracking and also to reduce any environmental hazard. Although there are limited tests which indicate that painted surfaces result in sound welds without wire brushing, other studies have resulted in excessive porosity and/or cracking when welding coated surfaces. Wire brushing to reduce the paint film thickness minimizes rejectable welds. Grinding or other procedures beyond wire brushing is not necessary.

# APPENDIX B

# DESIGN REQUIREMENTS

## B5. LOCAL BUCKLING

### 1. Classification of Steel Sections

The limiting width-thickenss $\lambda_p$ and $\lambda_r$ ratios for webs in pure flexure $(P_u / \phi_b P_y = 0)$ and with axial compression have been revised in terms of $(h / t)$ rather than $(h_c / t)$. The simplified formulation in Table B5.1 for $\lambda_r$ based on double symmetry with equal flanges $(h / h_c = 1)$ is unconservative when the compression flange is smaller than the tension flange, and conservative if the reverse is true. The more accurate limit is given in Appendix B5.1 as a function of $h_c$. Figure C-A-B5.1 illustrates the $\lambda_r$ variation for axial compression and flange asymmetry effects.

The ¾ minimum and 3⁄2 maximum restrictions on $h / h_c$ in Equations A-B5-1 and A-B5-2 approximately correspond to the 0.1 and 0.9 range of $I_{yc} / I_y$ for a member to be considered a singly symmetric I shape. Otherwise, when the flange areas differ by more than a factor of two, the member should be conservatively designed as a tee section.

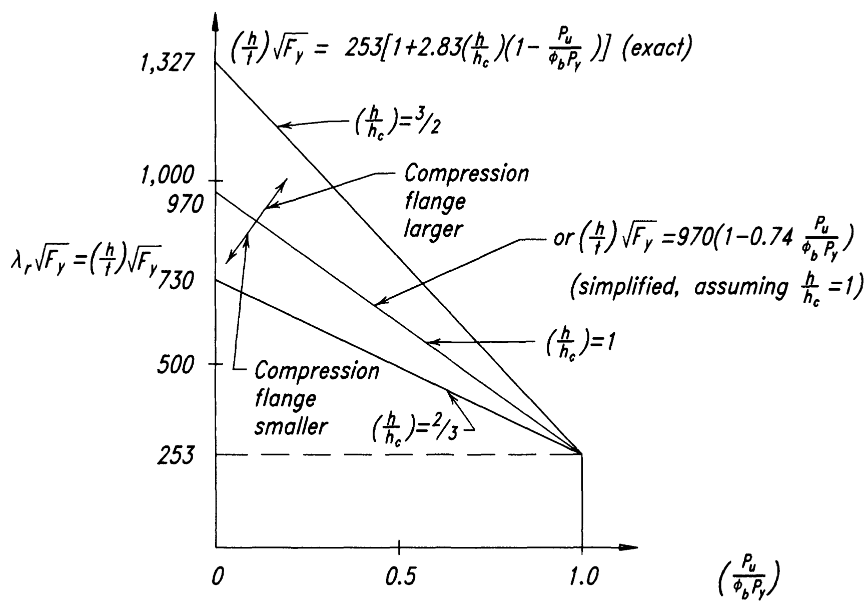



*Fig. C-A-B5.1. Local web buckling for I-shaped members.*

# APPENDIX E

# COLUMNS AND OTHER COMPRESSION MEMBERS

## E3. DESIGN COMPRESSIVE STRENGTH FOR FLEXURAL-TORSIONAL BUCKLING

The equations in Appendix E3 for determining the flexural-torsional elastic buckling loads of columns are derived in texts on structural stability (Timoshenko and Gere (1961), Bleich (1952), Galambos (1968), and Chen and Atsuta (1977), for example). Since these equations for flexural-torsional buckling apply only to elastic buckling, they must be modified for inelastic buckling when $F_{cr} > 0.5F_y$. This is accomplished through the use of the equivalent slenderness factor $\lambda_e = \sqrt{F_y / F_e}$.

# APPENDIX F

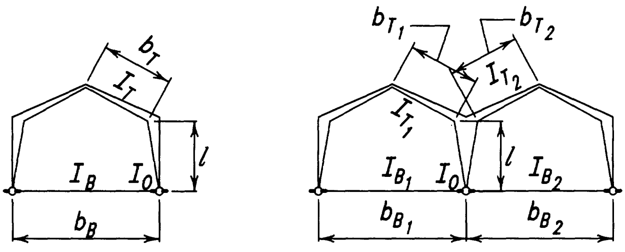
# BEAMS AND OTHER FLEXURAL MEMBERS

## F1. DESIGN FOR FLEXURE

Three limit states must be investigated to determine the moment capacity of flexural members: lateral-torsional buckling (LTB), local buckling of the compression flange (FLB), and local buckling of the web (WLB). These limit states depend, respectively, on the beam slenderness ratio $L_b / r_y$, the width-thickness ratio $b / t$ of the compression flange and the width-thickness ratio $h / t_w$ of the web. For convenience, all three measures of slenderness are denoted by $\lambda$.

Variations in $M_n$ with $L_b$ are shown in Figure C-A-F1.1. The discussion of plastic, inelastic, and elastic buckling in Commentary Section F1 with reference to lateral-torsional buckling applies here except for an important difference in the significance of $\lambda_p$ for lateral-torsional buckling and local buckling. Values of $\lambda_p$ for FLB and WLB produce a compact section with a rotation capacity of about three (after reaching $M_p$) before the onset of local buckling, and therefore meet the requirements for plastic analysis of load effects (Commentary Section B5). On the other hand, values of $\lambda_p$ for LTB do not allow plastic analysis because they do not provide rotation capacity beyond that needed to develop $M_p$. Instead $L_b \leq L_{pd}$ (Section F1.2d) must be satisfied.

Analyses to include restraint effects of adjoining elements are discussed in Galambos (1988). Analysis of the lateral stability of members with shapes not covered in this appendix must be performed according to the available literature (Galambos, 1988).




$$G_T = \frac{b_T I_O}{l I_T} \qquad G_T = \frac{I_O}{l} \sum \frac{b_T}{I_T}$$

$$G_B = \frac{b_B I_O}{l I_B} \qquad G_B = \frac{I_O}{l} \sum \frac{b_B}{I_B}$$

*Figure C-A-F1.1*

See the Commentary for Section B5 for the discussion of the equation regarding the bending capacity of circular sections.

## F3. WEB-TAPERED MEMBERS

### 1. General Requirements

The provision contained in Appendix F3 covers only those aspects of the design of tapered members that are unique to tapered members. For other criteria of design not specifically covered in Appendix F3, see the appropriate portions of this Specification and Commentary.

The design of wide-flange columns with a single web taper and constant flanges follows the same procedure as for uniform columns according to Section E2, except the column slenderness parameter $\lambda_c$ for major axis buckling is determined for a slenderness ratio $K_\gamma L / r_{ox}$, and for minor axis buckling for $KL / r_{oy}$, where $K_\gamma$ is an effective length factor for tapered members, $K$ is the effective length factor for prismatic members, and $r_{ox}$ and $r_{oy}$ are the radii of gyration about the x and the y axes, respectively, taken at the smaller end of the tapered member.

For stepped columns or columns with other than a single web taper, the elastic critical stress is determined by analysis or from data in reference texts or research reports (Chapters 11 and 13 in Timoshenko and Gere (1961) and Bleich (1952) and Kitipornchai and Trahair [1980], and then the same procedure of using $\lambda_{eff}$ is utilized in calculating the factored resistance.

This same approach is recommended for open section built-up columns (columns with perforated cover plates, lacing, and battens) where the elastic critical buckling stress determination must include a reduction for the effect of shear. Methods for calculating the elastic buckling strength of such columns are given in Chapter 12 of the SSRC Guide (Galambos, 1988) and in Timoshenko and Gere (1961) and Bleich (1952).

### 3. Design Compressive Strength

The approach in formulating $F_{a\gamma}$ of tapered columns is based on the concept that the critical stress for an axially loaded tapered column is equal to that of a prismatic column of different length, but of the same cross section as the smaller end of the tapered column. This has resulted in an equivalent effective length factor $K_\gamma$ for a tapered member subjected to axial compression (Lee et al., 1972). This factor, which is used to determine the value of $S$ in Equations A-F3-2 and $\lambda_c$ in Equation E2-3, can be determined accurately for a symmetrical rectangular rigid frame comprised of prismatic beams and tapered columns.

With modifying assumptions, such a frame can be used as a mathematical model to determine with sufficient accuracy the influence of the stiffness $\Sigma(I / b)_g$ of beams and rafters which afford restraint at the ends of a tapered column in other cases such as those shown in Figure C-A-F1.1. From Equations A-F3-2 and E2-3, the critical load $P_{cr}$ can be expressed as $\pi^2 EI_o / (K_\gamma l)^2$. The value of $K_\gamma$ can be obtained by interpolation, using the appropriate chart from Lee et al. (1972) and restraint modifiers $G_T$ and $G_B$. In each of these modifiers the tapered column, treated as a prismatic member having a moment of inertia $I_o$, computed at the smaller end, and its actual length $l$, is assigned the stiffness $I_o / l$, which is then divided by the stiffness of the restraining members at the end of the tapered column under consideration.

## 4. Design Flexural Strength

The development of the design bending stress for tapered beams follows closely with that for prismatic beams. The basic concept is to replace a tapered beam by an equivalent prismatic beam with a different length, but with a cross section identical to that of the smaller end of the tapered beam (Lee et al., 1972). This has led to the modified length factors $h_s$ and $h_w$ in Equations A-F3-6 and A-F3-7.

Equations A-F3-6 and A-F3-7 are based on total resistance to lateral buckling, using both St. Venant and warping resistance. The factor $B$ modifies the basic $F_{b\gamma}$ to members which are continuous past lateral supports. Categories a, b, and c of Appendix F3.4 usually apply; however, it is to be noted that they apply only when the axial force is small and adjacent unbraced segments are approximately equal in length. For a single member, or segments which do not fall into category a, b, c, or d, the recommended value of $B$ is unity. The value of $B$ should also be taken as unity when computing the value of $F_{b\gamma}$ to obtain $M_n$ to be used in Equations H1-1 and C1-1, since the effect of moment gradient is provided for by the factor $C_m$. The background material is given in WRC Bulletin No. 192 (Morrell and Lee, 1974).

# APPENDIX G

# PLATE GIRDERS

Appendix G is taken from AISI Bulletin 27 (Galambos, 1978). Comparable provisions are included in the AISC ASD Specification. The provisions are presented in an appendix as they are seldom used and produce designs which are often less economical than plate girders designed without tension-field action.

The web slenderness ratio $h / t_w = 970 / \sqrt{F_{yf}}$ that distinguishes plate girders from beams is written in terms of the flange yield stress, because for hybrid girders inelastic buckling of the web due to bending depends on the flange strain.

The equation for $R_e$ used in the 1986 LRFD Specification was the same as that used in the AASHTO *Standard Specification for Highway Bridges*. In this edition, the equation for $R_e$, used in the AISC ASD Specification since 1969, is used because its derivation is published (Gaylord and Gaylord, 1992 and ASCE-AASHTO, 1968) and it is more accurate than the AASHTO equation.

## G2. DESIGN FLEXURAL STRENGTH

In previous versions of the AISC Specification a coefficient of $0.0005a_r$ was used in $R_{PG}$ based on the work of Basler (1961). This value is valid for $a_r \leq 2$. In that same paper, Basler developed a more general coefficient, applicable to all ratios of $A_w / A_f$ which has now been adopted because application of the previous equation to sections with large $a_r$ values gives unreasonable results. An arbitrary limit of $a_r \leq 10$ is imposed so that the $R_{PG}$ expression is not applied to sections approaching a tee shape.

# APPENDIX H

# MEMBERS UNDER COMBINED FORCES AND TORSION

## H3. ALTERNATIVE INTERACTION EQUATIONS FOR MEMBERS UNDER COMBINED STRESS

In the case of members not subject to flexural buckling, i.e., $L_b < L_{pd}$, the use of somewhat more liberal interaction Equations A-H3-5 and A-H3-6 is acceptable as an alternative when the flexure is about one axis only.

The alternative interaction Equations A-H3-1 and A-H3-2 for biaxially loaded H and wide-flange column shapes were taken from Galambos (1988), Springfield (1975), and Tebedge and Chen (1974).

For I-shaped members with $b_f / d > 1.0$, use of Section H1 is recommended, because no additional research is available for this case.

# APPENDIX J

# CONNECTIONS, JOINTS, AND FASTENERS

## J2. WELDS

### 4. Design Strength

When weld groups are loaded in shear by an external load that does not act through the center of gravity of the group, the load is eccentric and will tend to cause a relative rotation and translation between the parts connected by the weld. The point about which rotation tends to take place is called the instantaneous center of rotation. Its location is dependent upon the load eccentricity, geometry of the weld group, and deformation of the weld at different angles of the resultant elemental force relative to the weld axis.

The individual resistance force of each unit weld element can be assumed to act on a line perpendicular to a ray passing through the instantaneous center and that element's location (see Figure C-A-J2.1).

The ultimate shear strength of weld groups can be obtained from the load deformation relationship of a single-unit weld element. This relationship was originally given by Butler (1972) for E60 electrodes. Curves for E70 electrodes used in the Appendix were obtained by Lesik (1990).

Unlike the load-deformation relationship for bolts, strength and deformation performance in welds are dependent on the angle $\theta$ that the resultant elemental force makes with the axis of the weld element (see Figure C-A-J2.1). The actual load deformation relationship for welds is given in Figure C-A-J2.2, taken from Kennedy and Lesik (1990). Conversion of the SI equation to foot-pound units results in the following weld strength equation for $R_n$:

$$R_n = 0.852(1.0 + 0.50 \sin^{1.5}\theta)F_{EXX}\, A_w$$

Because the maximum strength is limited to $0.60F_{EXX}$ for longitudinally loaded welds ($\theta = 0°$), the LRFD Specification provision provides, in the reduced equation coefficient, a reasonable margin for any variation in welding techniques and procedures. To eliminate possible computational difficulties, the maximum deformation in the weld elements is limited to $0.17D$. For design convenience, a simple elliptical formula is used for $f(p)$ to closely approximate the empirically derived polynomial in Lesik (1990).

The total resistance of all the weld elements combine to resist the eccentric ultimate load, and when the correct location of the instantaneous center has been selected, the three in-plane equations of statics ($\Sigma F_x$, $\Sigma F_y$, $\Sigma M$) will be satisfied. Numerical techniques, such as those given by Brandt (1982), have been developed to locate the instantaneous center of rotation subject to convergence tolerances.

Earlier editions of the AISC *Manual of Steel Construction* (AISC, 1980, 1986, 1989) took advantage of the inelastic redistribution of stresses that is inherent in the Appendix J2.4 procedure. However, in each of the utilized computational techniques the resulting coefficients were factored down so that the maximum

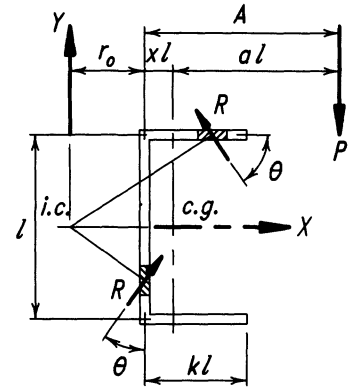



*Figure C-A-J2.1*

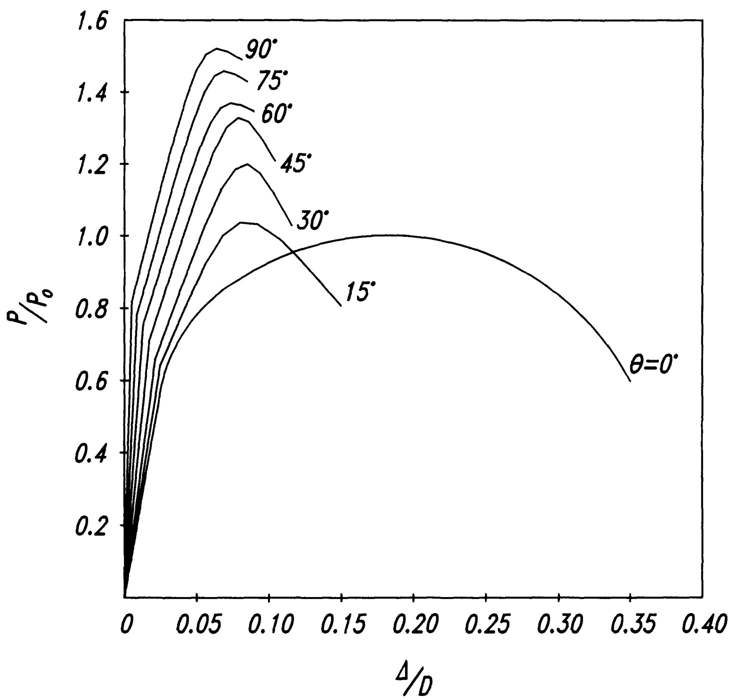



*Figure C-A-J2.2*

stress, at any point in the weld group, did not exceed the limiting value specified by either the Allowable Stress Design or LRFD Specifications, $0.3F_u$ or $0.6F_u$, respectively. As a result, the tabulated weld-capacity data shown in the appropriate referenced manual tables will be found to be conservative relative to the data obtained using the computational procedure presented in Appendix J2.4.

# APPENDIX K

# CONCENTRATED FORCES, PONDING, AND FATIGUE

## K3. FATIGUE

Because most members in building frames are not subject to a large enough number of cycles of full design stress application to require design for fatigue, the provisions covering such designs have been placed in Appendix K3.

When fatigue is a design consideration, its severity is most significantly affected by the number of load applications, the magnitude of the stress range, and the severity of the stress concentrations associated with the particular details. These factors are not encountered in normal building designs; however, when encountered and when fatigue is of concern, all provisions of Appendix K3 must be satisfied.

Members or connections subject to less than 20,000 cycles of loading will not involve a fatigue condition, except in the case of repeated loading involving large ranges of stress. For such conditions, the admissible range of stress can conservatively be taken as one and one-half times the applicable value given in Table A-K3.3 for "Loading Condition 1."

Fluctuation in stress which does not involve tensile stress does not cause crack propagation and is not considered to be a fatigue situation. On the other hand, in elements of members subject solely to calculated compression stress, fatigue cracks may initiate in regions of high tensile residual stress. In such situations, the cracks generally do not propagate beyond the region of the residual tensile stress, because the residual stress is relieved by the crack. For this reason stress ranges that are completely in compression are not included in the column headed by "Kind of Stress" in Table A-K3.2. This is also true of comparable tables of the current AASHTO and AREA specifications.

When fabrication details involving more than one category occur at the same location in a member, the stress range at that location must be limited to that of the most restrictive category. By locating notch-producing fabrication details in regions subject to a small range of stress, the need for a member larger than required by static loading will often be eliminated.

Extensive test programs (Fisher et al., 1970; and Fisher et al., 1974) using full size specimens, substantiated by theoretical stress analysis, have confirmed the following general conclusions:

(1) Stress range and notch severity are the dominant stress variables for welded details and beams.

(2) Other variables such as minimum stress, mean stress, and maximum stress are not significant for design purposes.

(3) Structural steels with yield points of 36 to 100 ksi do not exhibit significantly different fatigue strength for given welded details fabricated in the same manner.

Allowable stress ranges can be read directly from Table A-K3.3 for a particular category and loading condition. The values are based on extensive research (Keating and Fisher, 1985).

Provisions for bolts subjected to tension are given in Table A-K3.4. Tests have uncovered dramatic differences in fatigue life, not completely predictable from the various published equations for estimating the actual magnitude of prying force (Kulak et al., 1987). To limit the uncertainties regarding prying action on the fatigue behavior of these bolts, the tensile stresses given in Table J3.2 are approved for use under extended cyclic loading only if the prying force, included in the design tensile force, is small. When this cannot be assured, the design tensile stress is drastically reduced to cover any conceivable prying effect.

The use of other types of mechanical fasteners to resist applied cyclic loading in tension is not recommended. Lacking a high degree of assured pretension, the range of stress is generally too great to resist such loading for long.

However, all types of mechanical fasteners survive unharmed when subject to cyclic shear stresses sufficient to fracture the connected parts, which is provided for elsewhere in Appendix K3.

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# Glossary

*Alignment chart for columns.* A nomograph for determining the effective length factor $K$ for some types of columns

*Amplification factor.* A multiplier of the value of moment or deflection in the unbraced length of an axially loaded member to reflect the secondary values generated by the eccentricity of the applied axial load within the member

*Aspect ratio.* In any rectangular configuration, the ratio of the lengths of the sides

*Batten plate.* A plate element used to join two parallel components of a built-up column, girder, or strut rigidly connected to the parallel components and designed to transmit shear between them

*Beam.* A structural member whose primary function is to carry loads transverse to its longitudinal axis

*Beam-column.* A structural member whose primary function is to carry loads both transverse and parallel to its longitudinal axis

*Bent.* A plane framework of beam or truss members which support loads and the columns which support these members

*Biaxial bending.* Simultaneous bending of a member about two perpendicular axes

*Bifurcation.* The phenomenon whereby a perfectly straight member under compression may either assume a deflected position or may remain undeflected, or a beam under flexure may either deflect and twist out of plane or remain in its in-plane deflected position

*Braced frame.* A frame in which the resistance to lateral load or frame instability is primarily provided by a diagonal, a $K$ brace, or other auxiliary system of bracing

*Brittle fracture.* Abrupt cleavage with little or no prior ductile deformation

*Buckling load.* The load at which a perfectly straight member under compression assumes a deflected position

*Built-up member.* A member made of structural metal elements that are welded, bolted, or riveted together

*Cladding.* The exterior covering of the structural components of a building

*Cold-formed members.* Structural members formed from steel without the application of heat

*Column.* A structural member whose primary function is to carry loads parallel to its longitudinal axis

*Column curve.* A curve expressing the relationship between an axial column strength and slenderness ratio

*Combined mechanism.* A mechanism determined by plastic analysis procedure which combines elementary beam, panel, and joint mechanisms

*Compact section.* Compact sections are capable of developing a fully plastic stress distribution and possess rotation capacity of approximately three before the onset of local buckling

*Composite beam.* A steel beam structurally connected to a concrete slab so that the beam and slab respond to loads as a unit. See also *Concrete-encased beam*

*Concrete-encased beam.* A beam totally encased in concrete cast integrally with the slab

*Connection.* Combination of joints used to transmit forces between two or more members. Categorized by the type and amount of force transferred (moment, shear, end reaction). See also *Splices*

*Critical load.* The load at which bifurcation occurs as determined by a theoretical stability analysis

*Curvature.* The rotation per unit length due to bending

*Design documents.* See *Structural design documents*

*Design strength.* Resistance (force, moment, stress, as appropriate) provided by element or connection; the product of the nominal strength and the resistance factor

*Diagonal bracing.* Inclined structural members carrying primarily axial load employed to enable a structural frame to act as a truss to resist horizontal loads

*Diaphragm.* Floor slab, metal wall, or roof panel possessing a large in-plane shear stiffness and strength adequate to transmit horizontal forces to resisting systems

*Diaphragm action.* The in-plane action of a floor system (also roofs and walls) such that all columns framing into the floor from above and below are maintained in their same position relative to each other

*Double concentrated forces.* Two equal and opposite forces which form a couple on the same side of the loaded member

*Double curvature.* A bending condition in which end moments on a member cause the member to assume an S shape

*Drift.* Lateral deflection of a building

*Drift index.* The ratio of lateral deflection to the height of the building

*Ductility factor.* The ratio of the total deformation at maximum load to the elastic-limit deformation

*Effective length.* The equivalent length *KL* used in compression formulas and determined by a bifurcation analysis

*Effective length factor K.* The ratio between the effective length and the unbraced length of the member measured between the centers of gravity of the bracing members

*Effective moment of inertia.* The moment of inertia of the cross section of a member that remains elastic when partial plastification of the cross section takes place, usually under the combination of residual stress and applied stress. Also, the moment of inertia based on effective widths of elements that buckle locally. Also, the moment of inertia used in the design of partially composite members

*Effective stiffness.* The stiffness of a member computed using the effective moment of inertia of its cross section

*Effective width.* The reduced width of a plate or slab which, with an assumed uniform stress distribution, produces the same effect on the behavior of a structural member as the actual plate width with its nonuniform stress distribution

*Elastic analysis.* Determination of load effects (force, moment, stress, as appropriate) on members and connections based on the assumption that material deformation disappears on removal of the force that produced it

*Elastic-perfectly plastic.* A material which has an idealized stress-strain curve that varies linearly from the point of zero strain and zero stress up to the yield point of the material, and then increases in strain at the value of the yield stress without any further increases in stress

*Embedment.* A steel component cast in a concrete structure which is used to transmit externally applied loads to the concrete structure by means of bearing, shear, bond, friction, or any combination thereof. The embedment may be fabricated of structural-steel plates, shapes, bars, bolts, pipe, studs, concrete reinforcing bars, shear connectors, or any combination thereof

*Encased steel structure.* A steel-framed structure in which all of the individual frame members are completely encased in cast-in-place concrete

*Euler formula.* The mathematical relationship expressing the value of the Euler load in terms of the modulus of elasticity, the moment of inertia of the cross section, and the length of a column

*Euler load.* The critical load of a perfectly straight, centrally loaded pin-ended column

*Eyebar.* A particular type of pin-connected tension member of uniform thickness with forged or flame cut head of greater width than the body proportioned to provide approximately equal strength in the head and body

*Factored load.* The product of the nominal load and a load factor

*Fastener.* Generic term for welds, bolts, rivets, or other connecting device

*Fatigue.* A fracture phenomenon resulting from a fluctuating stress cycle

*First-order analysis.* Analysis based on first-order deformations in which equilibrium conditions are formulated on the undeformed structure

*Flame-cut plate.* A plate in which the longitudinal edges have been prepared by oxygen cutting from a larger plate

*Flat width.* For a rectangular tube, the nominal width minus twice the outside corner radius. In absence of knowledge of the corner radius, the flat width may be taken as the total section width minus three times the thickness

*Flexible connection.* A connection permitting a portion, but not all, of the simple beam rotation of a member end

*Floor system.* The system of structural components separating the stories of a building

*Force.* Resultant of distribution of stress over a prescribed area. A reaction that develops in a member as a result of load (formerly called total stress or stress). Generic term signifying axial loads, bending moment, torques, and shears

*Fracture toughness.* Measurement of the ability to absorb energy without fracture. Generally determined by impact loading of specimens containing a notch having a prescribed geometry

*Frame buckling.* A condition under which bifurcation may occur in a frame

*Frame instability.* A condition under which a frame deforms with increasing lateral deflection under a system of increasing applied monotonic loads until a maximum value of the load called the stability limit is reached, after which the frame will continue to deflect without further increase in load

*Fully composite beam.* A composite beam with sufficient shear connectors to develop the full flexural strength of the composite section

*High-cycle fatigue.* Failure resulting from more than 20,000 applications of cyclic stress

*Hybrid beam.* A fabricated steel beam composed of flanges with a greater yield strength than that of the web. Whenever the maximum flange stress is less than or equal to the web yield stress the girder is considered homogeneous

*Hysteresis loop.* A plot of force versus displacement of a structure or member subjected to reversed, repeated load into the inelastic range, in which the path followed during release and removal of load is different from the path for the addition of load over the same range of displacement

*Inclusions.* Nonmetallic material entrapped in otherwise sound metal

*Incomplete fusion.* Lack of union by melting of filler and base metal over entire prescribed area

*Inelastic action.* Material deformation that does not disappear on removal of the force that produced it

*Instability.* A condition reached in the loading of an element or structure in which continued deformation results in a decrease of load-resisting capacity

*Joint.* Area where two or more ends, surfaces, or edges are attached. Categorized by type of fastener or weld used and method of force transfer

*K bracing.* A system of struts used in a braced frame in which the pattern of the struts resembles the letter K, either normal or on its side

*Lamellar tearing.* Separation in highly restrained base metal caused by through-thickness strains induced by shrinkage of adjacent weld metal

*Lateral bracing member.* A member utilized individually or as a component of a lateral bracing system to prevent buckling of members or elements and/or to resist lateral loads

*Lateral (or lateral-torsional) buckling.* Buckling of a member involving lateral deflection and twist

*Leaning column.* Gravity-loaded column where connections to the frame (simple connections) do not provide resistance to lateral loads

*Limit state.* A condition in which a structure or component becomes unfit for service and is judged either to be no longer useful for its intended function (*serviceability limit state*) or to be unsafe (*strength limit state*)

*Limit states.* Limits of structural usefulness, such as brittle fracture, plastic collapse, excessive deformation, durability, fatigue, instability, and serviceability

*Load factor.* A factor that accounts for unavoidable deviations of the actual load from the nominal value and for uncertainties in the analysis that transforms the load into a load effect

*Loads.* Forces or other actions that arise on structural systems from the weight of all permanent construction, occupants and their possessions, environmental effects, differential settlement, and restrained dimensional changes. *Permanent* loads are those loads in which variations in time are rare or of small magnitude. All other loads are *variable* loads. See *Nominal loads*

*LRFD (Load and Resistance Factor Design).* A method of proportioning structural components (members, connectors, connecting elements, and assemblages) such that no applicable limit state is exceeded when the structure is subjected to all appropriate load combinations

*Local buckling.* The buckling of a compression element which may precipitate the failure of the whole member

*Low-cycle fatigue.* Fracture resulting from a relatively high-stress range resulting in a relatively small number of cycles to failure

*Lower bound load.* A load computed on the basis of an assumed equilibrium moment diagram in which the moments are not greater than $M_p$ that is less than or at best equal to the true ultimate load

*Mechanism.* An articulated system able to deform without an increase in load, used in the special sense that the linkage may include real hinges or plastic hinges, or both

*Mechanism method.* A method of plastic analysis in which equilibrium between external forces and internal plastic hinges is calculated on the basis of an assumed mechanism. The failure load so determined is an upper bound

*Nominal loads.* The magnitudes of the loads specified by the applicable code

*Nominal strength.* The capacity of a structure or component to resist the effects of loads, as determined by computations using specified material strengths and dimensions and formulas derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions

*Noncompact section.* Noncompact sections can develop the yield stress in compression elements before local buckling occurs, but will not resist inelastic local buckling at strain levels required for a fully plastic stress distribution

*P-Delta effect.* Secondary effect of column axial loads and lateral deflection on the moments in members

*Panel zone.* The zone in a beam-to-column connection that transmits moment by a shear panel

*Partially composite beam.* A composite beam for which the shear strength of shear connectors governs the flexural strength

*Plane frame.* A structural system assumed for the purpose of analysis and design to be two-dimensional

*Plastic analysis.* Determination of load effects (force, moment, stress, as appropriate) on members and connections based on the assumption of rigid-plastic behavior, i.e., that equilibrium is satisfied throughout the structure and yield is not exceeded anywhere. Second order effects may need to be considered

*Plastic design section.* The cross section of a member which can maintain a full plastic moment through large rotations so that a mechanism can develop; the section suitable for plastic design

*Plastic hinge.* A yielded zone which forms in a structural member when the plastic moment is attained. The beam is assumed to rotate as if hinged, except that it is restrained by the plastic moment $M_p$

*Plastic-limit load.* The maximum load that is attained when a sufficient number of yield zones have formed to permit the structure to deform plastically without further increase in load. It is the largest load a structure will support, when perfect plasticity is assumed and when such factors as instability, second-order effects, strain hardening, and fracture are neglected

*Plastic mechanism.* See *Mechanism*

*Plastic modulus.* The section modules of resistance to bending of a completely yielded cross section. It is the combined static moment about the neutral axis of the cross-sectional areas above and below that axis

*Plastic moment.* The resisting moment of a fully yielded cross section

*Plastic strain.* The difference between total strain and elastic strain

*Plastic zone.* The yielded region of a member

*Plastification.* The process of successive yielding of fibers in the cross section of a member as bending moment is increased

*Plate girder.* A built-up structural beam

*Post-buckling strength.* The load that can be carried by an element, member, or frame after buckling

*Primary stress.* A primary stress is any normal stress or shear stress developed by an imposed loading which is necessary to satisfy the laws of equilibrium of external and internal forces, moments, and torques. A primary stress is not self-limiting.

*Redistribution of moment.* A process which results in the successive formation of plastic hinges so that less highly stressed portions of a structure may carry increased moments

*Required strength.* Load effect (force, moment, stress, as appropriate) acting on element or connection determined by structural analysis from the factored loads (using most appropriate critical load combinations)

*Residual stress.* The stresses that remain in an unloaded member after it has been formed into a finished product. (Examples of such stresses include, but are not limited to, those induced by cold bending, cooling after rolling, or welding.)

*Resistance.* The capacity of a structure or component to resist the effects of loads. It is determined by computations using specified material strengths, dimensions and formulas derived from accepted principles of structural mechanics, or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions. Resistance is a generic term that includes both strength and serviceability limit states

*Resistance factor.* A factor that accounts for unavoidable deviations of the actual strength from the nominal value and the manner and consequences of failure

*Rigid frame.* A structure in which connections maintain the angular relationship between beam and column members under load

*Root of the flange.* Location on the web of the corner radius termination point or the toe of the flange-to-web weld. Measured as the $k$ distance from the far side of the flange

*Rotation capacity.* The incremental angular rotation that a given shape can accept prior to local failure defined as $R = (\theta_u / \theta_p) - 1$ where $\theta_u$ is the overall rotation attained at the factored load state and $\theta_p$ is the idealized rotation corresponding to elastic theory applied to the case of $M = M_p$

*St. Venant torsion.* That portion of the torsion in a member that induces only shear stresses in the member

*Second-order analysis.* Analysis based on second-order deformations, in which equilibrium conditions are formulated on the deformed structure

*Service load.* Load expected to be supported by the structure under normal usage; often taken as the nominal load

*Serviceability limit state.* Limiting condition affecting the ability of a structure to preserve its appearance, maintainability, durability, or the comfort of its occupants or function of machinery under normal usage

*Shape factor.* The ratio of the plastic moment to the yield moment, or the ratio of the plastic modulus to the section modulus for a cross section

*Shear friction.* Friction between the embedment and the concrete that transmits shear loads. The relative displacement in the plane of the shear load is considered to be resisted by shear-friction anchors located perpendicular to the plane of the shear load

*Shear lugs.* Plates, welded studs, bolts, and other steel shapes that are embedded in the concrete and located transverse to the direction of the shear force and that transmit shear loads, introduced into the concrete by local bearing at the shear lug-concrete interface

*Shear wall.* A wall that in its own plane resists shear forces resulting from applied wind, earthquake, or other transverse loads or provides frame stability. Also called a structural wall

*Sidesway.* The lateral movement of a structure under the action of lateral loads, unsymmetrical vertical loads, or unsymmetrical properties of the structure

*Sidesway buckling.* The buckling mode of a multistory frame precipitated by the relative lateral displacements of joints, leading to failure by sidesway of the frame

*Simple plastic theory.* See *Plastic design*

*Single curvature.* A deformed shape of a member having one smooth continuous arc, as opposed to double curvature which contains a reversal

*Slender-element section.* The cross section of a member which will experience local buckling in the elastic range

*Slenderness ratio.* The ratio of the effective length of a column to the radius of gyration of the column, both with respect to the same axis of bending

*Slip-critical joint.* A bolted joint in which the slip resistance of the connection is required

*Space frame.* A three-dimensional structural framework (as contrasted to a plane frame)

*Splice.* The connection between two structural elements joined at their ends to form a single, longer element

*Stability-limit load.* Maximum (theoretical) load a structure can support when second-order instability effects are included

*Stepped column.* A column with changes from one cross section to another occurring at abrupt points within the length of the column

*Stiffener.* A member, usually an angle or plate, attached to a plate or web of a beam or girder to distribute load, to transfer shear, or to prevent buckling of the member to which it is attached

*Stiffness.* The resistance to deformation of a member or structure measured by the ratio of the applied force to the corresponding displacement

*Story drift.* The difference in horizontal deflection at the top and bottom of a story

*Strain hardening.* Phenomenon wherein ductile steel, after undergoing considerable deformation at or just above yield point, exhibits the capacity to resist substantially higher loading than that which caused initial yielding

*Strain-hardening strain.* For structural steels that have a flat (plastic) region in the stress-strain relationship, the value of the strain at the onset of strain hardening

*Strength design.* A method of proportioning structural members using load factors and resistance factors such that no applicable limit state is exceeded (also called load and resistance factor design)

*Strength limit state.* Limiting condition affecting the safety of the structure, in which the ultimate load-carrying capacity is reached

*Stress.* Force per unit area

*Stress concentration.* Localized stress considerably higher than average (even in uniformly loaded cross sections of uniform thickness) due to abrupt changes in geometry or localized loading

*Strong axis.* The major principal axis of a cross section

*Structural design documents.* Documents prepared by the designer (plans, design details, and job specifications)

*Structural system.* An assemblage of load-carrying components which are joined together to provide regular interaction or interdependence

*Stub column.* A short compression-test specimen, long enough for use in measuring the stress-strain relationship for the complete cross section, but short enough to avoid buckling as a column in the elastic and plastic ranges

*Subassemblage.* A truncated portion of a structural frame

*Supported frame.* A frame which depends upon adjacent braced or unbraced frames for resistance to lateral load or frame instability. (This transfer of load is frequently provided by the floor or roof system through diaphragm action or by horizontal cross bracing in the roof.)

*Tangent modulus.* At any given stress level, the slope of the stress-strain curve of a material in the inelastic range as determined by the compression test of a small specimen under controlled conditions

*Temporary structure.* A general term for anything that is built or constructed (usually to carry construction loads) that will eventually be removed before or after completion of construction and does not become part of the permanent structural system

*Tensile strength.* The maximum tensile stress that a material is capable of sustaining

*Tension field action.* The behavior of a plate girder panel under shear force in which diagonal tensile stresses develop in the web and compressive forces develop in the transverse stiffeners in a manner analogous to a Pratt truss

*Toe of the fillet.* Termination point of fillet weld or of rolled section fillet

*Torque-tension relationship.* Term applied to the wrench torque required to produce specified pre-tension in high-strength bolts

*Turn-of-nut method.* Procedure whereby the specified pre-tension in high-strength bolts is controlled by rotation of the wrench a predetermined amount after the nut has been tightened to a snug fit

*Unbraced frame.* A frame in which the resistance to lateral load is provided by the bending resistance of frame members and their connections

*Unbraced length.* The distance between braced points of a member, measured between the centers of gravity of the bracing members

*Undercut.* A notch resulting from the melting and removal of base metal at the edge of a weld

*Universal-mill plate.* A plate in which the longitudinal edges have been formed by a rolling process during manufacture. Often abbreviated as UM plate

*Upper bound load.* A load computed on the basis of an assumed mechanism which will always be at best equal to or greater than the true ultimate load

*Vertical bracing system.* A system of shear walls, braced frames, or both, extending through one or more floors of a building

*Von Mises yield criterion.* A theory which states that inelastic action at any point in a body under any combination of stresses begins only when the strain energy of distortion per unit volume absorbed at the point is equal to the strain energy of distortion absorbed per unit volume at any point in a simple tensile bar stressed to the elastic limit under a state of uniaxial stress. It is often called the maximum strain-energy-of-distortion theory. Accordingly, shear yield occurs at 0.58 times the yield strength

*Warping torsion.* That portion of the total resistance to torsion that is provided by resistance to warping of the cross section

*Weak axis.* The minor principal axis of a cross section

*Weathering steel.* A type of high-strength, low-alloy steel which can be used in normal environments (not marine) and outdoor exposures without protective paint covering. This steel develops a tight adherent rust at a decreasing rate with respect to time

*Web buckling.* The buckling of a web plate

*Web crippling.* The local failure of a web plate in the immediate vicinity of a concentrated load or reaction

*Working load.* Also called service load. The actual load assumed to be acting on the structure

*Yield moment.* In a member subjected to bending, the moment at which an outer fiber first attains the yield stress

*Yield plateau.* The portion of the stress-strain curve for uniaxial tension or compression in which the stress remains essentially constant during a period of substantially increased strain

*Yield point.* The first stress in a material at which an increase in strain occurs without an increase in stress, the yield point less than the maximum attainable stress

*Yield strength.* The stress at which a material exhibits a specified limiting deviation from the proportionality of stress to strain. Deviation expressed in terms of strain

*Yield stress.* Yield point, yield strength, or yield stress level as defined

*Yield-stress level.* The average stress during yielding in the plastic range, the stress determined in a tension test when the strain reaches 0.005 in. per in.