

ANSI/AISC 358-16
An American National Standard

Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications

May 12, 2016

Supersedes ANSI/AISC 358-10, ANSI/AISC 358s1-11, ANSI/AISC 358s2-14
and all previous versions

Approved by the Connection Prequalification Review Panel



AMERICAN INSTITUTE OF STEEL CONSTRUCTION
130 East Randolph Street, Suite 2000, Chicago, Illinois 60601
www.aisc.org

AISC © 2016

by

American Institute of Steel Construction

*All rights reserved. This book or any part thereof
must not be reproduced in any form without the
written permission of the publisher.*

The AISC logo is a registered trademark of AISC.

The information presented in this publication has been prepared by a balanced committee following American National Standards Institute (ANSI) consensus procedures and recognized principles of design and construction. While it is believed to be accurate, this information should not be used or relied upon for any specific application without competent professional examination and verification of its accuracy, suitability and applicability by a licensed engineer or architect. The publication of this information is not a representation or warranty on the part of the American Institute of Steel Construction, its officers, agents, employees or committee members, or of any other person named herein, that this information is suitable for any general or particular use, or of freedom from infringement of any patent or patents. All representations or warranties, express or implied, other than as stated above, are specifically disclaimed. Anyone making use of the information presented in this publication assumes all liability arising from such use.

Caution must be exercised when relying upon standards and guidelines developed by other bodies and incorporated by reference herein since such material may be modified or amended from time to time subsequent to the printing of this edition. The American Institute of Steel Construction bears no responsibility for such material other than to refer to it and incorporate it by reference at the time of the initial publication of this edition.

Printed in the United States of America

PREFACE

(This Preface is not part of ANSI/AISC 358-16, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*, but is included for informational purposes only.)

ANSI/AISC 358-16, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*, was developed using a consensus process in concert with the *Specification for Structural Steel Buildings* (ANSI/AISC 360-16) and *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341-16). ANSI/AISC 358-16 is incorporated by reference into the *Seismic Provisions*.

This edition includes minor editorial updates and clarifications from previous versions as well as two additional prequalified moment connections: the Simpson Strong Frame moment connection (Chapter 12) and the double-tee moment connection (Chapter 13).

A nonmandatory Commentary has been prepared to provide background for the provisions of the Standard, and the user is encouraged to consult it. Additionally, nonmandatory User Notes are interspersed throughout the Standard to provide concise and practical guidance in the application of the provisions.

The reader is cautioned that professional judgment must be exercised when data or recommendations in this Standard are applied, as described more fully in the disclaimer notice preceding the Preface.

This standard was approved by the Connection Prequalification Review Panel:

Ronald O. Hamburger, Chairman	Kevin Moore
Scott F. Armbrust, Vice-Chairman	Thomas M. Murray
John Abruzzo	Charles W. Roeder
Joel A. Chandler	Paul A. Rouis
Michael L. Cochran	Thomas A. Sabol
Theodore L. Droessler	Robert E. Shaw, Jr.
Michael D. Engelhardt	James A. Swanson
Gary Glenn	Chia-Ming Uang
Robert Hazleton	Behnam Yousefi
Gregory H. Lynch	Keith A. Grubb, Secretary
Brett R. Manning	Margaret A. Matthew, Secretary

The CPRP gratefully acknowledges the following individuals for their contributions to this document:

Leigh Arber	Steven E. Pryor
Charles J. Carter	C. Mark Saunders
Cynthia J. Duncan	Matthew A. Troemner
James O. Malley	
Duane K. Miller	

TABLE OF CONTENTS

SYMBOLS	xiv
GLOSSARYxxiii
STANDARD	
CHAPTER 1. GENERAL	1
1.1. Scope	1
1.2. References	1
1.3. General	3
CHAPTER 2. DESIGN REQUIREMENTS	4
2.1. Special and Intermediate Moment Frame Connection Types	4
2.2. Connection Stiffness	4
2.3. Members	4
1. Rolled Wide-Flange Members	4
2. Built-up Members	4
2a. Built-up Beams	4
2b. Built-up Columns	5
3. Hollow Structural Sections (HSS)	7
2.4. Connection Design Parameters	7
1. Resistance Factors	7
2. Plastic Hinge Location	7
3. Probable Maximum Moment at Plastic Hinge	7
4. Continuity Plates	8
2.5. Panel Zones	8
2.6. Protected Zone	8
CHAPTER 3. WELDING REQUIREMENTS	10
3.1. Filler Metals	10
3.2. Welding Procedures	10
3.3. Backing at Beam-to-Column and Continuity Plate-to-Column Joints. . . .	10
1. Steel Backing at Continuity Plates	10
2. Steel Backing at Beam Bottom Flange	10
3. Steel Backing at Beam Top Flange	10
4. Prohibited Welds at Steel Backing	11
5. Nonfusible Backing at Beam Flange-to-Column Joints	11
3.4. Weld Tabs	11
3.5. Tack Welds	11
3.6. Continuity Plates	12
3.7. Quality Control and Quality Assurance	12

CHAPTER 4. BOLTING REQUIREMENTS	13
4.1. Fastener Assemblies	13
4.2. Installation Requirements	13
4.3. Quality Control and Quality Assurance	13
CHAPTER 5. REDUCED BEAM SECTION (RBS) MOMENT CONNECTION ..	14
5.1. General	14
5.2. Systems	14
5.3. Prequalification Limits	14
1. Beam Limitations	14
2. Column Limitations	15
5.4. Column-Beam Relationship Limitations	16
5.5. Beam Flange-to-Column Flange Weld Limitations	16
5.6. Beam Web-to-Column Flange Connection Limitations	16
5.7. Fabrication of Flange Cuts	17
5.8. Design Procedure	18
CHAPTER 6. BOLTED UNSTIFFENED AND STIFFENED EXTENDED END-PLATE MOMENT CONNECTIONS	21
6.1. General	21
6.2. Systems	21
6.3. Prequalification Limits	22
1. Beam Limitations	22
2. Column Limitations	22
6.4. Column-Beam Relationship Limitations	24
6.5. Continuity Plates	24
6.6. Bolts	24
6.7. Connection Detailing	24
1. Gage	24
2. Pitch and Row Spacing	24
3. End-Plate Width	26
4. End-Plate Stiffener	26
5. Finger Shims	27
6. Welding Details	27
6.8. Design Procedure	29
1. End-Plate and Bolt Design	29
2. Column-Side Design	34
CHAPTER 7. BOLTED FLANGE PLATE (BFP) MOMENT CONNECTION	39
7.1. General	39
7.2. Systems	39
7.3. Prequalification Limits	40
1. Beam Limitations	40
2. Column Limitations	40

7.4.	Column-Beam Relationship Limitations	41
7.5.	Connection Detailing	41
1.	Plate Material Specifications	41
2.	Beam Flange Plate Welds	41
3.	Single-Plate Shear Connection Welds	41
4.	Bolt Requirements	41
5.	Flange Plate Shims	42
7.6.	Design Procedure	42
CHAPTER 8. WELDED UNREINFORCED FLANGE-WELDED WEB (WUF-W)		
MOMENT CONNECTION.		
8.1.	General	46
8.2.	Systems	46
8.3.	Prequalification Limits	46
1.	Beam Limitations	46
2.	Column Limitations	47
8.4.	Column-Beam Relationship Limitations	48
8.5.	Beam Flange-to-Column Flange Welds	48
8.6.	Beam Web-to-Column Connection Limitations	48
8.7.	Design Procedure	49
CHAPTER 9. KAISER BOLTED BRACKET (KBB)		
MOMENT CONNECTION.		
9.1.	General	52
9.2.	Systems	53
9.3.	Prequalification Limits	53
1.	Beam Limitations	53
2.	Column Limitations	54
3.	Bracket Limitations	55
9.4.	Column-Beam Relationship Limitations	55
9.5.	Bracket-to-Column Flange Connection Limitations	55
9.6.	Bracket-to-Beam Flange Connection Limitations	56
9.7.	Beam Web-to-Column Connection Limitations	56
9.8.	Connection Detailing	57
9.9.	Design Procedure	57
CHAPTER 10. CONXTECH CONXL MOMENT CONNECTION		
10.1.	General	64
10.2.	Systems	67
10.3.	Prequalification Limits	68
1.	Beam Limitations	68
2.	Column Limitations	68
3.	Collar Limitations	69

10.4.	Collar Connection Limitations	70
10.5.	Beam Web-to-Collar Connection Limitations	72
10.6.	Beam Flange-to-Collar Flange Welding Limitations	72
10.7.	Column-Beam Relationship Limitations	72
10.8.	Design Procedure	73
10.9.	Part Drawings	78
CHAPTER 11. SIDEPLATE MOMENT CONNECTION		88
11.1.	General	88
11.2.	Systems	93
11.3.	Prequalification Limits	93
1.	Beam Limitations	93
2.	Column Limitations	95
3.	Connection Limitations	96
11.4.	Column-Beam Relationship Limitations	96
11.5.	Connection Welding Limitations	100
11.6.	Connection Detailing	102
1.	Plates	102
2.	Welds	103
11.7.	Design Procedure	105
CHAPTER 12. SIMPSON STRONG-TIE STRONG FRAME MOMENT CONNECTION		110
12.1.	General	110
12.2.	Systems	110
12.3.	Prequalification Limits	110
1.	Beam Limitations	110
2.	Column Limitations	112
3.	Bolting Limitations	112
12.4.	Column-Beam Relationship Limitations	113
12.5.	Continuity Plates	113
12.6.	Yield-Link Flange-to-Stem Weld Limitations	113
12.7.	Fabrication of Yield-Link Cuts	113
12.8.	Connection Detailing	115
1.	Beam Coping	115
2.	Yield-Links	115
3.	Shear Plate Connection Bolts	115
4.	Shear-Plate Shear Connection Welds	115
5.	Bolt Hole Requirements	115
6.	Buckling Restraint Assembly	115
7.	Shims	118
12.9.	Design Procedure	118

CHAPTER 13. DOUBLE-TEE MOMENT CONNECTION	127
13.1. General	127
13.2. Systems	129
13.3. Prequalification Limits	131
1. Beam Limitations	131
2. Column Limitations	131
13.4. Column-Beam Relationship Limitations	132
13.5. Connection Detailing	132
1. T-Stub Material Specifications	132
2. Continuity Plates	132
3. Single-Plate Shear Connection Welds	132
4. Bolts	132
5. T-Stub Shims	133
13.6. Design Procedure	133
APPENDIX A. CASTING REQUIREMENTS	145
A1. Cast Steel Grade	145
A2. Quality Control (QC)	145
1. Inspection and Nondestructive Testing Personnel	145
2. First Article Inspection (FAI) of Castings	145
3. Visual Inspection of Castings	145
4. Nondestructive Testing (NDT) of Castings	145
4a. Procedures	145
4b. Required NDT	146
5. Weld Repair Procedures	146
6. Tensile Requirements	146
7. Charpy V-Notch (CVN) Requirements	146
8. Casting Identification	147
A3. Manufacturer Documents	147
1. Submittal to Patent Holder	147
2. Submittal to Engineer of Record and Authority Having Jurisdiction	147
APPENDIX B. FORGING REQUIREMENTS	148
B1. Forged Steel Grade	148
B2. Bar Stock	148
B3. Forging Temperature	148
B4. Heat Treatment	148
B5. Finish	148
B6. Quality Assurance	148
B7. Documentation	148

COMMENTARY

INTRODUCTION	151
CHAPTER 1. GENERAL	152
1.1. Scope	152
1.2. References	153
1.3. General.....	153
CHAPTER 2. DESIGN REQUIREMENTS	154
2.1. Special and Intermediate Moment Frame Connection Types	154
2.3. Members.....	155
2. Built-up Members	155
2b. Built-up Columns	156
2.4. Connection Design Parameters	157
1. Resistance Factors	157
2. Plastic Hinge Location	157
3. Probable Maximum Moment at Plastic Hinge	157
4. Continuity Plates	157
CHAPTER 3. WELDING REQUIREMENTS	160
3.3. Backing at Beam-to-Column and Continuity Plate-to-Column Joints....	160
1. Steel Backing at Continuity Plates	160
2. Steel Backing at Beam Bottom Flange	160
3. Steel Backing at Beam Top Flange	161
4. Prohibited Welds at Steel Backing	161
5. Nonfusible Backing at Beam Flange-to-Column Joints	161
3.4. Weld Tabs.....	161
3.5. Tack Welds	162
3.6. Continuity Plates	162
3.7. Quality Control and Quality Assurance.....	163
CHAPTER 4. BOLTING REQUIREMENTS	164
4.1. Fastener Assemblies.....	164
4.2. Installation Requirements.....	164
4.3. Quality Control and Quality Assurance.....	164
CHAPTER 5. REDUCED BEAM SECTION (RBS) MOMENT CONNECTION ..	165
5.1 General.....	165
5.2 Systems	166
5.3. Prequalification Limits.....	166
1. Beam Limitations	166
2. Column Limitations.....	167
5.4. Column-Beam Relationship Limitations	169
5.5. Beam Flange-to-Column Flange Weld Limitations	169

5.6.	Beam Web-to-Column Flange Connection Limitations	169
5.7.	Fabrication of Flange Cuts	170
5.8.	Design Procedure	170
CHAPTER 6. BOLTED UNSTIFFENED AND STIFFENED EXTENDED		
END-PLATE MOMENT CONNECTIONS		
6.1.	General	174
6.2.	Systems	174
6.3.	Prequalification Limits	175
1.	Beam Limitations	175
2.	Column Limitations	175
6.4.	Column-Beam Relationship Limitations	175
6.5.	Continuity Plates	175
6.6.	Bolts	176
6.7.	Connection Detailing	176
6.8.	Design Procedure	177
CHAPTER 7. BOLTED FLANGE PLATE (BFP) MOMENT CONNECTION . . .		
7.1.	General	178
7.2.	Systems	180
7.3.	Prequalification Limits	180
1.	Beam Limitations	180
2.	Column Limitations	181
7.4.	Column-Beam Relationship Limitations	181
7.5.	Connection Detailing	182
7.6.	Design Procedure	183
CHAPTER 8. WELDED UNREINFORCED FLANGE-WELDED WEB (WUF-W)		
MOMENT CONNECTION		
8.1.	General	185
8.3.	Prequalification Limits	187
8.4.	Column-Beam Relationship Limitations	187
8.5.	Beam Flange-to-Column Flange Welds	188
8.6.	Beam Web-to-Column Connection Limitations	188
8.7.	Design Procedure	189
CHAPTER 9. KAISER BOLTED BRACKET (KBB) MOMENT		
CONNECTION		
9.1.	General	191
9.2.	Systems	191
9.3.	Prequalification Limits	192
1.	Beam Limitations	192
2.	Column Limitations	192
3.	Bracket Limitations	193

9.4.	Column-Beam Relationship Limitations	193
9.5.	Bracket-to-Column Flange Connection Limitations	193
9.6.	Bracket-to-Beam Flange Connection Limitations	194
9.7.	Beam Web-to-Column Connection Limitations	195
9.8.	Connection Detailing	196
9.9.	Design Procedure	196
CHAPTER 10. CONXTECH CONXL MOMENT CONNECTION		199
10.1.	General	199
10.2.	Systems	201
10.3.	Prequalification Limits	201
1.	Beam Limitations	201
2.	Column Limitations	202
3.	Collar Limitations	203
10.4.	Collar Connection Limitations	203
10.5.	Beam Web-to-Collar Connection Limitations	203
10.6.	Beam Flange-to-Collar Flange Welding Limitations	203
10.7.	Column-Beam Relationship Limitations	203
10.8.	Design Procedure	204
CHAPTER 11. SIDEPLATE MOMENT CONNECTION		206
11.1.	General	206
11.2.	Systems	208
11.3.	Prequalification Limits	209
1.	Beam Limitations	209
2.	Column Limitations	210
3.	Connection Limitations	212
11.4.	Column-Beam Relationship Limitations	212
11.5.	Connection Welding Limitations	213
11.6.	Connection Detailing	214
11.7.	Design Procedure	214
CHAPTER 12. SIMPSON STRONG-TIE STRONG FRAME MOMENT CONNECTION		223
12.1.	General	223
12.3.	Prequalification Limits	226
1.	Beam Limitations	226
2.	Column Limitations	226
3.	Bolting Limitations	226
12.4.	Column-Beam Relationship Limitations	227
12.5.	Continuity Plates	227
12.6.	Yield-Link Flange-to-Stem Weld Limitations	227
12.7.	Fabrication of Yield-Link Cuts	228

12.8.	Connection Detailing	228
12.9.	Design Procedure	228
CHAPTER 13.	DOUBLE-TEE MOMENT CONNECTION	232
13.1.	General	232
13.2.	Systems	233
13.3.	Prequalification Limits	234
13.5.	Connection Detailing	234
1.	T-Stub Material Specifications	234
2.	Continuity Plates	234
4.	Bolts	234
5.	T-Stub Shims	235
13.6.	Design Procedure	235
APPENDIX A.	CASTING REQUIREMENTS	242
A1.	Cast Steel Grade	242
A2.	Quality Control (QC)	242
2.	First Article Inspection (FAI) of Castings	242
3.	Visual Inspection of Castings	242
4.	Nondestructive Testing (NDT) of Castings	242
6.	Tensile Requirements	243
A3.	Manufacturer Documents	244
APPENDIX B.	FORGING REQUIREMENTS	245
REFERENCES	246

SYMBOLS

This Standard uses the following symbols in addition to the terms defined in the *Specification for Structural Steel Buildings* (ANSI/AISC 360-16) and the *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341-16). Some definitions in the following list have been simplified in the interest of brevity. In all cases, the definitions given in the body of the Standard govern. Symbols without text definitions, used in only one location and defined at that location, are omitted in some cases. The section or table number on the right refers to where the symbol is first used.

Symbol	Definition	Section
A_c	Contact areas between continuity plate and column flanges that have attached beam flanges, in. ² (mm ²)	6.5
A_c	Area of concrete in column, in. ² (mm ²)	10.8
A_s	Area of steel in column, in. ² (mm ²)	10.8
A_{tb}	Gross area of a tension bolt measured through its shank, in. ² (mm ²)	13.6
A_{vb}	Gross area of a shear bolt measured through its shank, in. ² (mm ²)	13.6
A_{y-link}	Yield area of reduced Yield-Link section, in. ² (mm ²)	12.9
A'_{y-link}	Estimated required Yield-Link yield area, in. ² (mm ²)	12.9
A_{\perp}	Perpendicular amplified seismic drag or chord forces transferred through the SidePlate connection, resulting from applicable building code, kips (N)	11.7
A_{\parallel}	In-plane factored lateral drag or chord axial forces transferred along the frame beam through the SidePlate connection, resulting from load case 1.0 E_Q per applicable building code, kips (N)	11.7
C_{pr}	Factor to account for peak connection strength, including strain hardening, local restraint, additional reinforcement, and other connection conditions.	2.4.3
C_t	Factor used in Equation 6.8-17	6.8.2
E	Modulus of elasticity of steel = 29,000 ksi (200 000 MPa).	13.6
F_{EXX}	Filler metal classification strength, ksi (MPa)	9.9
F_f	Maximum force in the T-stub and beam flange, kips (N)	13.6
F_{fu}	Factored beam flange force, kips (N)	6.8.1
F_{nt}	Nominal tensile strength of bolt from the AISC <i>Specification</i> , ksi (MPa)	6.8.1
F_{nt}	Nominal tensile stress of bolt from the AISC <i>Specification</i> , ksi (MPa)	13.6
F_{nv}	Nominal shear strength of bolt from the AISC <i>Specification</i> , ksi (MPa)	6.8.1
F_{nv}	Nominal shear stress of bolt from the AISC <i>Specification</i> , ksi (MPa)	13.6
F_{pr}	Probable maximum force in the T-stub and beam flange, kips (N)	13.6
F_{pr}	Force in the flange plate due to M_f , kips (N)	7.6
F_{su}	Required stiffener strength, kips (N)	6.8.2
F_u	Specified minimum tensile strength of yielding element, ksi (MPa)	2.4.3
F_{ub}	Specified minimum tensile strength of beam material, ksi (MPa).	7.6
F_{ub}	Specified minimum tensile stress of beam, ksi (MPa)	13.6

F_{uf}	Specified minimum tensile strength of flange material, ksi (MPa)	9.9
F_{up}	Specified minimum tensile strength of end-plate material, ksi (MPa)	6.8.1
F_{up}	Specified minimum tensile strength of plate material, ksi (MPa)	7.6
F_{ut}	Specified minimum tensile stress of T-stub, ksi (MPa)	13.6
F_{u-link}	Specified minimum tensile strength of Yield-Link stem material, ksi (MPa) . .	12.9
F_w	Nominal weld design strength per the AISC <i>Specification</i> , ksi (MPa)	9.9
F_y	Specified minimum yield stress of the yielding element, ksi (MPa)	2.4.3
F_{yb}	Specified minimum yield stress of beam material, ksi (MPa)	6.8.1
F_{yb}	Specified minimum yield stress of the beam, ksi (MPa)	11.4(3)
F_{yc}	Specified minimum yield stress of column flange material, ksi (MPa)	6.8.2
F_{yc}	Specified minimum yield stress of column web material, ksi (MPa)	6.8.2
F_{yc}	Specified minimum yield stress of the column, ksi (MPa)	11.4(3)
F_{yf}	Specified minimum yield stress of flange material, ksi (MPa)	9.9
F_{yp}	Specified minimum yield stress of end-plate material, ksi (MPa)	6.8.1
F_{ys}	Specified minimum yield stress of stiffener material, ksi (MPa)	6.8.1
F_{yt}	Specified minimum yield stress of the T-stub, ksi (MPa)	13.6
F_{y-link}	Specified minimum yield stress of Yield-Link stem material, ksi (MPa)	12.9
H_h	Distance along column height from $1/4$ of column depth above the top edge of lower-story side plates to $1/4$ of column depth below bottom edge of upper-story side plates, in. (mm)	11.4(3)
H_l	Height of story below node, in. (mm)	10.8
H_u	Height of story above node, in. (mm)	10.8
I_{beam}	Moment of inertia of the beam in plane of bending, in. ⁴ (mm ⁴)	Figure 11.16
I_{beam}	Strong-axis moment of inertia of the beam, in. ⁴ (mm ⁴)	13.6
I_{ft}	Moment of inertia of the T-flange per pair of tension bolts, in. ⁴ (mm ⁴)	13.6
I_{total}	Approximation of moment of inertia due to beam hinge location and side plate stiffness, in. ⁴ (mm ⁴)	Figure 11.16
K_1	Elastic axial stiffness contribution due to bending stiffness in Yield-Link flange, kip/in. (N/mm)	12.9
K_2	Elastic axial stiffness contribution due to nonyielding section of Yield-Link, kip/in. (N/mm)	12.9
K_3	Elastic axial stiffness contribution due to yielding section of Yield-Link, kip/in. (N/mm)	12.9
K_{comp}	Initial stiffness of a T-stub in compression, kip/in. (N/mm)	13.6
K_{eff}	Effective elastic axial stiffness of Yield-Link, kip/in. (N/mm)	12.9
K_{flange}	Initial stiffness of a T-flange, kip/in. (N/mm)	13.6
K_i	Initial stiffness of the connection, kip-in./rad (N-mm/rad)	13.6
K_{slip}	Initial stiffness of the slip mechanism between a T-stem and beam flange, kip/in. (N/mm)	13.6
K_{stem}	Initial stiffness of a T-stem, kip/in. (N/mm)	13.6
K_{ten}	Initial stiffness of a T-stub in tension, kip/in. (N/mm)	13.6
L	Distance between column centerlines, in. (mm)	11.3(5)
$L_{bm-side}$	Length of nonreduced Yield-Link at beam side, in. (mm)	Fig. 12.2
L_c	Clear distance, in direction of force, between edge of the hole and edge of the adjacent hole or edge of material, in. (mm)	6.8.1

$L_{col-side}$	Length of nonreduced Yield-Link at column side, in. (mm)	Fig. 12.2
L_{crit}	Length of critical shear plane through cover plate as shown in Figure C-11.6, in. (mm).	Commentary 11.7
L_{bb}	Length of bracket, in. (mm).	Table 9.1
L_h	Horizontal edge distance for bolts in Yield-Link flange to column flange connection, in. (mm)	Fig. 12.2
L_{ehb}	Horizontal end distance of the beam measured from the end of the beam to the centerline of the first row of shear bolts or to the centerline of the web bolts, in. (mm)	13.6
L_v	Vertical edge distance for bolts in Yield-Link flange to column flange connection, in. (mm)	Fig. 12.2
L_h	Distance between plastic hinge locations, in. (mm)	5.8
L_o	Theoretical length of the connected beam measured between the working points of the adjacent columns, in. (mm).	13.6
L_{sp}	Length of shear connection, in. (mm)	13.6
L_{st}	Length of end plate stiffener, in. (mm)	6.7.4
L_{vb}	Length of the shear bolt pattern in the T-stems and beam flanges, in. (mm).	13.6
L_{y-link}	Length of reduced Yield-Link section, in. (mm)	Fig. 12.2
M_{bolts}	Moment at collar bolts, kip-in. (N-mm)	10.8
M_{cant}	Factored gravity moments from cantilever beams that are not in the plane of the moment frame but are connected to the exterior face of the side plates, resulting from code-applicable load combinations, kip-in. (N-mm).	11.7
M_{group}	Maximum probable moment demand at any connection element, kip-in. (N-mm).	11.7
M_{cf}	Column flange flexural strength, kip-in. (N-mm)	6.8.2
M_f	Probable maximum moment at face of column, kip-in. (N-mm)	5.8
M_f	Moment developed at face of column, kip-in. (N-mm)	13.6
M_{np}	Moment without prying action in bolts, kip-in. (N-mm).	Table 6.2
M_{pe}	Plastic moment of beam based on expected yield stress, kip-in. (N-mm).	5.8
M_{pr}	Probable maximum moment at plastic hinge, kip-in. (N-mm)	2.4.3
M_{iw}	Additional moment due to shear amplification from center of reduced beam section to centerline of column, kip-in. (N-mm)	5.4(2)
M_{pcl}^*	Plastic moment nominal strength of the column below the node, about the axis under consideration, considering simultaneous axial loading and loading about the transverse axis, kip-in. (N-mm)	10.8
M_{pcu}^*	Plastic moment nominal strength of the column above the node, about the axis under consideration considering simultaneous axial loading and loading about the transverse axis, kip-in (N-mm)	10.8
M_{u-sp}	Moment in shear plate at the column face, kip-in. (N-mm)	12.9
M_v	Additional moment due to the beam shear acting on a lever arm extending from the assumed point of plastic hinging to the centerline of the column, kip-in. (N-mm).	10.8
$M_{ye-link}$	Moment at expected Yield-Link yield, kip-in. (N-mm).	12.9
N	Thickness of beam flange plus two times the reinforcing fillet weld size, in. (mm)	6.8.2

P	Axial load acting on the column at the section under consideration in accordance with the applicable load combination specified by the building code, but not considering amplified seismic load, kips (N)	10.8
P_{R-weld}	Required strength of Yield-Link stem to Yield-Link flange weld, kips (N)	12.9
P_{R-link}	Probable maximum tensile strength of Yield-Link, kips (N).	12.9
P_{slip}	Expected slip load of the shear bolts between the beam flange and T-stem, kips (N)	13.6
P_t	Minimum specified tensile strength of bolt, kips (N)	Table 6.2
P_{u-sp}	Required axial strength of beam web-to-column flange connection, kips (N)	12.9
$P_{ye-link}$	Expected yield strength of the Yield-Link, kips (N)	12.9
P'_{y-link}	Estimated required Yield-Link yield force, kips (N)	12.9
R_{pt}	Minimum bolt pretension, kips (N)	10.8
R_n	Required force for continuity plate design, kips (N)	6.8.1
R_n	Nominal strength	7.6
R_n^{pz}	Nominal panel zone shear strength, kips (N).	10.8
R_t	Ratio of expected tensile strength to specified minimum tensile strength for flange material	9.9
R_u	Ultimate strength of fillet weld, kips (N).	Commentary 11.4
R_n^{pz}	Required panel zone shear strength, kips (N)	10.8
R_y	Ratio of expected yield stress to specified minimum yield stress, F_y ,	2.4.3
S_1	Distance from face of column to nearest row of bolts, in. (mm)	7.6
S_1	Distance from the face of the column to the first row of shear bolts, in. (mm)	13.6
S_h	Distance from face of column to plastic hinge, in. (mm)	2.3.2a
T	Tension force per bolt, kips/bolt (N/bolt)	13.6
T_1	Nominal tension strength per bolt of the T-flange corresponding to a plastic mechanism in the T-flange, kips/bolt (N/bolt)	13.6
T_2	Nominal tension strength per bolt of the T-flange corresponding to a mixed-mode failure of the T-flange, kips/bolt (N/bolt)	13.6
T_3	Nominal tension strength per bolt of the T-flange corresponding to bolt fracture without T-flange yielding, kips/bolt (N/bolt)	13.6
T_{req}	Required T-stub force per tension bolt, kips/bolt (N/bolt)	13.6
V_{bolts}	Probable maximum shear at collar bolts, kips (N).	10.8
V_{cant}	Factored gravity shear forces from cantilever beams that are not in the plane of the moment frame but are connected to the exterior face of the side plates, resulting from code-applicable load combinations, kips (N)	11.7
V_{cf}	Probable maximum shear at face of collar flange, kips (N)	10.8
V_{col}	Column shear, kips (N)	10.8
V_f	Probable maximum shear at face of column, kips (N)	10.8
$V_{gravity}$	Beam shear force resulting from $1.2D + f_1L + 0.2S$, kips (N)	5.8
V_h	Beam shear force at plastic hinge location, kips (N)	7.6
V_{RBS}	Larger of the two values of shear force at center of reduced beam section at each end of beam, kips (N)	5.4(2)

V'_{RBS}	Smaller of the two values of shear force at center of reduced beam section at each end of beam, kips (N) Commentary 5.8
V_u	Required shear strength of beam and beam web-to-column connection, kips (N). 5.8
V_1, V_2	Factored gravity shear forces from gravity beams that are not in the plane of the moment frame but are connected to the exterior surfaces of the side plate, resulting from the load combination of $1.2D + f_1L + 0.2S$ (where f_1 is the load factor determined by the applicable building code for live loads, but not less than 0.5), kips (N) 11.7
W_T	Width of the T-stub measured parallel to the column flange width, in. (mm) . 13.6
W_{Whit}	Whitmore width of the stem of the T-stub, in. (mm) 13.6
Y_c	Column flange yield line mechanism parameter, in. (mm) 6.8.2
Y_C	Yield-line parameter used to determine column-flange strength. 13.6
Y_m	Simplified column flange yield-line mechanism parameter 9.9
Y_p	End-plate yield line mechanism parameter, in. (mm) Table 6.2
Z_b	Nominal plastic section modulus of beam, in. ³ (mm ³) 11.4
Z_c	Plastic section modulus of the column about either axis, in. ³ (mm ³) 10.8
Z_e	Effective plastic modulus of section (or connection) at location of a plastic hinge, in. ³ (mm ³) 2.4.3
Z_{ec}	Equivalent plastic section modulus of column at a distance of $\frac{1}{4}$ the column depth from top and bottom edge of side plates, projected to beam centerline, in. ³ (mm ³) 11.4(3)
Z_{RBS}	Plastic section modulus at the center of reduced beam section, in. ³ (mm ³) 5.8
Z_x	Plastic section modulus about x -axis, in. ³ (mm ³). 5.8
Z_x	Plastic section modulus about the x -axis of the gross section of the beam at the location of the plastic hinge, in. ³ (mm ³). 13.6
Z_{xb}	Plastic modulus of beam about x -axis, in. ³ (mm ³). 11.7
Z_{xc}	Plastic modulus of column about x -axis, in. ³ (mm ³) 11.7
$Z_{x,net}$	Plastic section modulus of the net section of the beam at the location of the plastic hinge, in. ³ (mm ³) 13.6
a	Horizontal distance from face of column flange to start of a reduced beam section cut, in. (mm). 5.4(2)
a	Distance from outside face of the collar to reduced beam section cut, in. (mm) 10.8
a	Horizontal distance from centerline of bolt holes in shear plate to face of column, in. (mm) 12.4(2)
a	Distance between bolt line and outside edge of T-flange, in. (mm) 13.6
a'	Distance between inside edge of bolt line and outside edge of T-flange, in. (mm) 13.6
a_c	Horizontal distance from inside tension bolts and edge of column flange, in. (mm) 13.6
b	Width of compression element as defined in the AISC <i>Specification</i> , in. (mm) 2.3.2b
b	Length of reduced beam section cut, in. (mm) 5.4(2)

b	Vertical distance from centerline of bolt holes in Yield-Link flange to face of Yield-Link stem, in. (mm)	12.9
b	Distance between effective T-stem and bolt line in the T-flange, in. (mm) . . .	13.6
b'	Distance between effective T-stem and inside edge of bolt line in T-flange, in. (mm)	13.6
b_{bb}	Width of bracket, in. (mm)	Table 9.1
$b_{bm-side}$	Width of nonreduced Yield-Link at beam side, in. (mm)	Fig. 12.2
b_c	Horizontal distance from column web to inside tension bolts, in. (mm)	13.6
b_{cf}	Width of column flange, in. (mm)	9.9
$b_{col-side}$	Width of nonreduced Yield-Link at column side, in. (mm)	Fig. 12.2
b_{fc}	Flange width of the column, in. (mm)	13.6
b_{flange}	Width of Yield-Link flange at column side, in. (mm)	Fig. 12.2
b_{fp}	Width of flange plate, in. (mm)	7.6
b_{ft}	Flange width of the T-stub, in. (mm)	13.6
b_p	Width of end plate, in. (mm)	Table 6.1
b_{yield}	Width of reduced Yield-Link section, in. (mm)	Fig. 12.2
c	Depth of cut at center of reduced beam section, in. (mm)	5.8
d	Overall depth of beam, in. (mm)	5.3.1
d_b	Diameter of column flange bolts, in. (mm)	9.9
d_b	Depth of the beam, in. (mm)	13.6
d_{b-brp}	Diameter of bolt connecting buckling restraint plate to beam flange, in. (mm)	Figure 12.3
$d_{b-flange}$	Diameter of bolt connecting Yield-Link flange to column flange, in. (mm) . . .	12.9
d_{b-sp}	Diameter of bolts in shear plate, in. (mm)	12.9
d_{b-stem}	Diameter of bolts connecting Yield-Link stem to beam flange, in. (mm)	12.9
$d_{b,req}$	Required bolt diameter, in. (mm)	6.8.1
d_c	Depth of column, in. (mm)	5.4(2)
d_{c1}, d_{c2}	Depth of column on each side of a bay in a moment frame, in. (mm)	11.3(5)
d_e	Column bolt edge distance, in. (mm)	Table 9.2
d_{eff}	Effective depth of beam, calculated as the centroidal distance between bolt groups in the upper and lower brackets, in. (mm)	9.9
d_{leg}^{CC}	Effective depth of collar corner assembly leg, in. (mm)	10.8
d_{pl}	Depth of vertical shear element, in. (mm)	Commentary 11.7
d_{tb}	Diameter of the tension bolts between the T-flange and the column flange, in. (mm)	13.6
d_{tht}	Diameter or width of the holes in the T-flange for the tension bolts, in. (mm)	13.6
d_{vb}	Diameter of the shear bolts between the T-stem and the beam flange, in. (mm)	13.6
d_{vht}	Diameter of the holes in the T-stem for the shear bolts, in. (mm)	13.6
f'_c	Specified compressive strength of concrete fill, ksi (MPa)	10.8
f_1	Load factor determined by the applicable building code for live loads but not less than 0.5	5.8
g	Horizontal distance (gage) between fastener lines, in. (mm)	Table 6.1
g	Column bolt gage, in. (mm)	Table 9.1

g_{flange}	Vertical distance between rows of bolts in connection of Yield-Link flange to column flange, in. (mm)	Fig. 12.2
g_{ic}	Gage of interior tension bolts in the column flange, in. (mm)	13.6
g_{stem}	Horizontal distance between rows of bolts in connection of Yield-Link stem to beam flange, in. (mm)	Fig. 12.2
g_{tb}	Gage of the tension bolts in the T-stub, in. (mm)	13.6
g_{vb}	Gage of the shear bolts in the T-stub, in. (mm)	13.6
h_1	Distance from the centerline of a compression flange to the tension-side inner bolt rows in four-bolt extended and four-bolt stiffened extended end-plate moment connections, in. (mm)	Table 6.2
h_{bb}	Height of bracket, in. (mm)	Table 9.1
h_{flange}	Height of Yield-Link flange, in. (mm)	Fig. 12.2
h_i	Distance from centerline of compression flange to the centerline of the i th tension bolt row, in. (mm)	6.8.1
h_o	Distance from centerline of compression flange to the tension-side outer bolt row in four-bolt extended and four-bolt stiffened extended end-plate moment connections, in. (mm)	Table 6.2
h_p	Height of plate, in. (mm)	8.6(2)
h_{st}	Height of stiffener, in. (mm)	6.7.4
k_1	Distance from web centerline to flange toe of fillet, in. (mm)	3.6
k_c	Distance from outer face of a column flange to web toe of fillet (design value) or fillet weld, in. (mm)	6.8.2
k_{det}	Largest value of k_1 used in production, in. (mm)	3.6
l	Bracket overlap distance, in. (mm)	9.9
l_{pl}	Effective length of horizontal shear plate, in. (mm)	Commentary 11.7
l_w	Length of available fillet weld, in. (mm)	9.9
l_w^{CC}	Total length of available fillet weld at collar corner assembly, in. (mm)	10.8
l_w^{CWX}	Total length of available fillet weld at collar web extension, in. (mm)	10.8
n	Number of bolts	7.6
n_b	Number of bolts at compression flange	6.8.1
n_{bb}	Number of beam bolts	Table 9.3
n_{bolt}	Number of bolts in Yield-Link stem-to-beam flange connection	12.9
$n_{bolt-sp}$	Total number of bolts in shear plate.	12.9
n_{cb}	Number of column bolts	Table 9.1
n_{cf}	Number of collar bolts per collar flange	10.8
n_i	Number of inner bolts	6.8.1
n_o	Number of outer bolts	6.8.1
n_{rows}	Number of rows of bolts in Yield-Link stem	12.9
n_{tb}	Number of tension bolts connecting the T-flange to the column flange	13.6
n_{vb}	Number of shear bolts connecting the T-stem to the beam flange.	13.6
p	Perpendicular tributary length per bolt, in. (mm)	9.9
p	Width of the T-stub tributary to a pair of tension bolts, in./bolt (mm/bolt) . . .	13.6
p_b	Vertical distance between inner and outer row of bolts in eight-bolt stiffened extended end-plate moment connection, in. (mm)	Table 6.1
p_b	Column bolt pitch, in. (mm)	Table 9.2

P_{fi}	Vertical distance from inside of a beam tension flange to nearest inside bolt row, in. (mm)	Table 6.1
P_{fo}	Vertical distance from outside of a beam tension flange to nearest outside bolt row, in. (mm)	Table 6.1
P_s	Vertical distance from continuity plate to horizontal row of tension bolts, in. (mm)	13.6
P_{si}	Distance from inside face of continuity plate to nearest inside bolt row, in. (mm)	6.7.2
P_{so}	Distance from outside face of continuity plate to nearest outside bolt row, in. (mm)	6.7.2
r_h	Radius of horizontal bracket, in. (mm)	Table 9.2
r_{nt}	Nominal tensile strength of a tension bolt, kips/bolt (N/bolt)	13.6
r_{nv}	Nominal shear strength of a shear bolt, kips/bolt (N/bolt)	13.6
r_t	Required tension force per bolt in Yield-Link flange to column flange connections, kips/bolt (N/bolt)	12.9
r_{ut}	Required collar bolt tension strength, kips (N)	10.8
r_v	Radius of bracket stiffener, in. (mm)	Table 9.2
s	Distance from centerline of most inside or most outside tension bolt row to the edge of a yield line pattern, in. (mm)	Table 6.2
s	Spacing of bolt rows in a bolted flange plate moment connection, in. (mm)	7.6
s	Vertical distance defining potential yield-line pattern in column flange, in. (mm)	13.6
s_b	Distance from center of last row of bolts to beam-side end of Yield-Link, in. (mm)	Fig. 12.2
s_c	Distance from the reduced section of the Yield-Link to the center of the first row of bolts, in. (mm)	Fig. 12.2
s_{flange}	Spacing between bolts for Yield-Link flange-to-column-flange connection, in. (mm)	Fig. 12.2
s_{stem}	Spacing between rows of bolts for Yield-Link stem-to-beam-flange connection, in. (mm)	Fig. 12.2
s_{vb}	Spacing of the shear bolts in the T-stub, in. (mm)	13.6
s_{vert}	Vertical distance from center of the top (or bottom) shear plate bolt to center of center shear plate bolt, in. (mm)	12.9
s_{bolts}	Distance from center of plastic hinge to the centroid of the collar bolts, in. (mm)	10.8
s_f	Distance from center of plastic hinge to face of column, in. (mm)	10.8
s_h	Distance from center of plastic hinge to center of column, in. (mm)	10.8
t_{bf}	Thickness of beam flange, in. (mm)	5.8
t_{bw}	Thickness of beam web, in. (mm)	6.8.1
t_{col}	Wall thickness of HSS or built-up box column, in. (mm)	10.8
t_{collar}	Distance from face of the column to outside face of the collar, in. (mm)	10.8
t_{cp}	Thickness of continuity plates, in. (mm)	13.6
t_{cp}	Thickness of cover plates, in. (mm)	Commentary 11.7
t_{cw}	Thickness of column web, in. (mm)	6.8.2
t_f^{CC}	Fillet weld size required to join collar corner assembly to column, in. (mm)	10.8

t_f^{CWX}	Fillet weld size required to join each side of beam web to collar web extension, in. (mm)	10.8
t_{fb}	Flange thickness of the beam, in. (mm)	13.6
t_{fc}	Flange thickness of the column, in. (mm)	13.6
t_{flange}	Thickness of Yield-Link flange, in. (mm)	Fig. 12.2
t_{ft}	Flange thickness of the T-stub, in. (mm)	13.6
$t_{ft,crit}$	Flange thickness of the T-stub above which prying is negligible, in. (mm) . . .	13.6
t_{leg}^{CC}	Effective thickness of collar corner assembly leg, in. (mm)	10.8
t_p	Thickness of end-plate, in. (mm)	Table 6.1
t_s	Thickness of stiffener, in. (mm)	6.8.1
t_{st}	Stem thickness of the T-stub, in. (mm)	13.6
$t_{st,eff}$	Effective stem thickness of the T-stub used for prying calculations (see Figure 13.6 and Equation 13.6-51), in. (mm)	13.6
t_{stem}	Thickness of Yield-Link stem, in. (mm)	Fig. 12.2
w	Minimum size of fillet weld, in. (mm).	Table 9.2
w	Uniform beam gravity load, kips per linear ft (N per linear mm)	Commentary 5.8
w_u	Distributed load on beam, kip/ft (N/mm), using the load combination $1.2D + f_1L + 0.2S$	10.8
x	Distance from plastic hinge location to centroid of connection element, in. (mm)	11.7
$\Delta_{0.04}$	Axial deformation in Yield-Link at a connection rotation of 0.04 rad.	12.9
$\Delta_{0.07}$	Axial deformation in Yield-Link at a connection rotation of 0.07 rad.	12.9
Δ_{slip}	Expected deformation at the onset of slip, 0.0076 in. (0.19 mm)	13.6
Δ_y	Axial deformation in Yield-Link at expected yield, in. (mm)	12.9
θ_y	Connection rotation at expected yield of Yield-Link, rad	12.9
α	Adjustment factor for predicting the expected slip load of the connection. . .	13.6
β_a	Adjustment factor to account for shear deformation in the T-flange outside of the tension bolts.	13.6
β_b	Adjustment factor to account for shear deformation in the T-flange between the tension bolts.	13.6
δ	Factor accounting for net area of T-stub flange	13.6
ϕ_d	Resistance factor for ductile limit states	2.4.1
ϕ_n	Resistance factor for nonductile limit states	2.4.1

GLOSSARY

This Standard uses the following terms in addition to the terms defined in the *Specification for Structural Steel Buildings* (ANSI/AISC 360-16) and the *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341-16).

Air carbon arc cutting. Process of cutting steel by the heat from an electric arc applied simultaneously with an air jet.

Backing. Piece of metal or other material, placed at the weld root to facilitate placement of the root pass.

Backgouge. Process of removing by grinding or air carbon arc cutting all or a portion of the root pass of a complete-joint-penetration groove weld, from the reverse side of a joint from which a root was originally placed.

Cascaded weld ends. Method of terminating a weld in which subsequent weld beads are stopped short of the previous bead, producing a cascade effect.

Concrete structural slab. Reinforced concrete slab or concrete fill on steel deck with a total thickness of 3 in. (75 mm) or greater and a concrete compressive strength in excess of 2,000 psi (14 MPa).

Full-length beam erection method. A method of erecting a SidePlate steel frame that employs a full-length beam assembly consisting of the beam with shop-installed cover plates (if required) and vertical shear elements (except for HSS beams) that are fillet-welded near the ends of the beam. In the field, the full-length beams are lifted up in between pre-installed side plates and are joined to the plates with fillet welds.

Horizontal shear plate (HSP). Plates that transfer a portion of the moment in the side plates to the web of a wide-flange column in a SidePlate moment connection.

Link-beam erection method. A method of erecting a SidePlate steel frame that utilizes column tree assemblies with shop-installed beam stubs, which are then connected in the field to a link beam using complete-joint-penetration (CJP) groove welds.

Nonfusible backing. Backing material that will not fuse with the base metals during the welding process.

Plastic hinge location. Location in a column-beam assembly where inelastic energy dissipation is assumed to occur through the development of plastic flexural straining.

Probable maximum moment at the plastic hinge. Expected moment developed at a plastic hinge location along a member, considering the probable (mean) value of the material strength for the specified steel and effects of strain hardening.

Reinforcing fillet. Fillet weld applied to a groove welded T-joint to obtain a contour to reduce stress concentrations associated with joint geometry.

Root. Portion of a multi-pass weld deposited in the first pass of welding.

Thermal cutting. Group of cutting processes that severs or removes metal by localized melting, burning or vaporizing of the workpiece.

Vertical shear elements (VSE). Structural elements that transfer shear from a wide-flange beam web to the outboard edge of the side plates in a SidePlate moment connection.

Weld tab. Piece of metal affixed to the end of a welded joint to facilitate the initiation and termination of weld passes outside the structural joint.

CHAPTER 1

GENERAL

1.1. SCOPE

This Standard specifies design, detailing, fabrication and quality criteria for connections that are prequalified in accordance with the AISC *Seismic Provisions for Structural Steel Buildings* (herein referred to as the AISC *Seismic Provisions*) for use with special moment frames (SMF) and intermediate moment frames (IMF). The connections contained in this Standard are prequalified to meet the requirements in the AISC *Seismic Provisions* only when designed and constructed in accordance with the requirements of this Standard. Nothing in this Standard shall preclude the use of connection types contained herein outside the indicated limitations, nor the use of other connection types, when satisfactory evidence of qualification in accordance with the AISC *Seismic Provisions* is presented to the authority having jurisdiction.

1.2. REFERENCES

The following publications form a part of this Standard to the extent that they are referenced and applicable:

American Institute of Steel Construction (AISC)

ANSI/AISC 341-16 *Seismic Provisions for Structural Steel Buildings* (herein referred to as the AISC *Seismic Provisions*)

ANSI/AISC 360-16 *Specification for Structural Steel Buildings* (herein referred to as the AISC *Specification*)

AISC *Steel Construction Manual*, 14th Ed.

American Society of Mechanical Engineers (ASME)

ASME B46.1-09 *Surface Texture, Surface Roughness, Waviness, and Lay*

American Society for Nondestructive Testing (ASNT)

ASNT-TC-1a-2011 *Personnel Qualification and Certification in Nondestructive Testing*

ASTM International (ASTM)

A36/A36M-14 *Standard Specification for Carbon Structural Steel*

A354-11 *Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners*

A370-15 *Standard Test Methods and Definitions for Mechanical Testing of Steel Products*

A488/A488M-16 *Standard Practice for Steel Castings, Welding, Qualifications of Procedures and Personnel*

- A490-14a *Standard Specification for Heat-Treated Steel Structural Bolts, Alloy Steel, Heat Treated, 150 ksi Minimum Tensile Strength*
- A572/A572M-15 *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*
- A574-13 *Standard Specification for Alloy Steel Socket Head Cap Screws*
- A609/A609M-12 *Standard Practice for Castings, Carbon, Low-Alloy, and Martensitic Stainless Steel, Ultrasonic Examination Thereof*
- A668/A668M-15 *Standard Specification for Steel Forgings, Carbon and Alloy, for General Industrial Use*
- A781/A781M-14b *Standard Specification for Castings, Steel and Alloy, Common Requirements, for General Industrial Use*
- A788/A788M-15 *Standard Specification for Steel Forgings, General Requirements*
- A802/A802M-95(2015) *Standard Practice for Steel Castings, Surface Acceptance Standards, Visual Examination*
- A903/A903M-99(2012)e1 *Standard Specification for Steel Castings, Surface Acceptance Standards, Magnetic Particle and Liquid Penetrant Inspection*
- A913/A913M-15 *Standard Specification for High-Strength Low-Alloy Steel Shapes of Structural Quality, Produced by Quenching and Self-Tempering Process (QST)*
- A958/A958M-15 *Standard Specification for Steel Castings, Carbon and Alloy, with Tensile Requirements, Chemical Requirements Similar to Standard Wrought Grades*
- A992/A992M-11(2015) *Standard Specification for Structural Steel Shapes*
- B19-15 *Standard Specification for Cartridge Brass Sheet, Strip, Plate, Bar, and Disks*
- B36/B36M-13 *Standard Specification for Brass Plate, Sheet, Strip, and Rolled Bar*
- E186-15 *Standard Reference Radiographs for Heavy Walled [2 to 4½ in. (50.8 to 114 mm)] Steel Castings*
- E446-15 *Standard Reference Radiographs for Steel Castings Up to 2 in. (50.8 mm) in Thickness*
- E709-15 *Standard Guide for Magnetic Particle Examination*
- F1852-14 *Standard Specification for “Twist Off” Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*
- F3125/F3125M-15a *Specification for High Strength Structural Bolts, Steel and Alloy Steel, Heat Treated, 120 ksi (830 MPa) and 150 ksi (1040 MPa) Minimum Tensile Strength, Inch and Metric Dimensions*

American Welding Society (AWS)

- AWS C4.1:2010 *Criteria for Describing Oxygen-Cut Surfaces*
- AWS D1.1/D1.1M-2015 *Structural Welding Code—Steel*
- AWS D1.8/D1.8M-2016 *Structural Welding Code—Seismic Supplement*

Manufacturers Standardization Society (MSS)

- MSS SP-55-2011 *Quality Standard for Steel Castings for Valves, Flanges and Fittings and Other Piping Components—Visual Method for Evaluation of Surface Irregularities*

Research Council on Structural Connections (RCSC)

Specification for Structural Joints using High-Strength Bolts, 2014 (herein referred to as the *RCSC Specification*)

1.3. GENERAL

All design, materials and workmanship shall conform to the requirements of the AISC *Seismic Provisions* and this Standard. The connections contained in this Standard shall be designed according to the load and resistance factor design (LRFD) provisions. Connections designed according to this Standard are permitted to be used in structures designed according to the LRFD or allowable strength design (ASD) provisions of the AISC *Seismic Provisions*.

CHAPTER 2

DESIGN REQUIREMENTS

2.1. SPECIAL AND INTERMEDIATE MOMENT FRAME CONNECTION TYPES

The connection types listed in Table 2.1 are prequalified for use in connecting beams to column flanges in special moment frames (SMF) and intermediate moment frames (IMF) within the limitations specified in this Standard.

2.2. CONNECTION STIFFNESS

All connections contained in this Standard shall be considered fully restrained (Type FR) for the purpose of seismic analysis.

Exception: For the Simpson Strong-Tie Strong Frame connection, a partially restrained (Type PR) connection, the seismic analysis must include the force-deformation characteristics of the specific connection per Section 12.9.

2.3. MEMBERS

The connections contained in this Standard are prequalified in accordance with the requirements of the AISC *Seismic Provisions* when used to connect members meeting the limitations of Sections 2.3.1, 2.3.2 or 2.3.3, as applicable.

1. Rolled Wide-Flange Members

Rolled wide-flange members shall conform to the cross-section profile limitations applicable to the specific connection in this Standard.

2. Built-up Members

Built-up members having a doubly symmetric, I-shaped cross section shall meet the following requirements:

- (1) Flanges and webs shall have width, depth and thickness profiles similar to rolled wide-flange sections meeting the profile limitations for wide-flange sections applicable to the specific connection in this Standard.
- (2) Webs shall be continuously connected to flanges in accordance with the requirements of Sections 2.3.2a or 2.3.2b, as applicable.

2a. Built-up Beams

The web and flanges shall be connected using complete-joint-penetration (CJP) groove welds with a pair of reinforcing fillet welds within a zone extending from the beam end to a distance not less than one beam depth beyond the plastic hinge

TABLE 2.1.
Prequalified Moment Connections

Connection Type	Chapter	Systems
Reduced beam section (RBS)	5	SMF, IMF
Bolted unstiffened extended end plate (BUEEP)	6	SMF, IMF
Bolted stiffened extended end plate (BSEEP)	6	SMF, IMF
Bolted flange plate (BFP)	7	SMF, IMF
Welded unreinforced flange-welded web (WUF-W)	8	SMF, IMF
Kaiser bolted bracket (KBB)	9	SMF, IMF
ConXtech ConXL moment connection (ConXL)	10	SMF, IMF
SidePlate moment connection (SidePlate)	11	SMF, IMF
Simpson Strong-Tie Strong Frame moment connection	12	SMF, IMF
Double-tee moment connection	13	SMF, IMF

location, S_h , unless specifically indicated in this Standard. The minimum size of these fillet welds shall be the lesser of $\frac{5}{16}$ in. (8 mm) and the thickness of the beam web.

Exception: This provision shall not apply where individual connection prequalifications specify other requirements.

2b. Built-up Columns

Built-up columns shall conform to the provisions of subsections (1) through (4), as applicable. Built-up columns shall satisfy the requirements of the AISC *Specification*, except as modified in this section. Transfer of all internal forces and stresses between elements of the built-up column shall be through welds.

(1) I-Shaped Columns

The elements of built-up I-shaped columns shall conform to the requirements of the AISC *Seismic Provisions*.

Within a zone extending from 12 in. (300 mm) above the upper beam flange to 12 in. (300 mm) below the lower beam flange, unless specifically indicated in this Standard, the column webs and flanges shall be connected using CJP groove welds with a pair of reinforcing fillet welds. The minimum size of the fillet welds shall be the lesser of $\frac{5}{16}$ in. (8 mm) and the thickness of the column web.

Exception: For SidePlate moment connections, each column flange may be connected to the column web using a pair of continuous fillet welds. The required shear strength of the fillet welds, ϕR_n , shall equal the shear developed at the column flange-to-web connection where the shear force in the column is the smaller of

- (a) The nominal shear strength of the column per AISC *Specification* Equation G2-1.

- (b) The maximum shear force that can be developed in the column when plastic hinge(s) form in the connected beam(s).

(2) Boxed Wide-Flange Columns

The wide-flange shape of a boxed wide-flange column shall conform to the requirements of the AISC *Seismic Provisions*.

The width-to-thickness ratio, b/t , of plates used as flanges shall not exceed $0.6\sqrt{E/F_y}$, where b shall be taken as not less than the clear distance between plates.

The width-to-thickness ratio, h/t_w , of plates used only as webs shall conform to the requirements of the AISC *Seismic Provisions*.

Within a zone extending from 12 in. (300 mm) above the upper beam flange to 12 in. (300 mm) below the lower beam flange, flange and web plates of boxed wide-flange columns shall be joined by CJP groove welds. Outside this zone, plate elements shall be continuously connected by fillet or groove welds.

(3) Built-up Box Columns

The width-to-thickness ratio, b/t , of plates used as flanges shall not exceed $0.6\sqrt{E/F_y}$, where b shall be taken as not less than the clear distance between web plates.

The width-to-thickness ratio, h/t_w , of plates used only as webs shall conform to the requirements of the AISC *Seismic Provisions*.

Within a zone extending from 12 in. (300 mm) above the upper beam flange to 12 in. (300 mm) below the lower beam flange, flange and web plates of box columns shall be joined by CJP groove welds. Outside this zone, box column web and flange plates shall be continuously connected by fillet welds or groove welds.

Exception: For ConXL moment connections, partial-joint-penetration (PJP) groove welds conforming to the requirements of Section 10.3.2 shall be permitted within the zone extending from 12 in. (300 mm) above the upper beam flange to 12 in. (300 mm) below the lower beam flange.

(4) Flanged Cruciform Columns

The elements of flanged cruciform columns, whether fabricated from rolled shapes or built up from plates, shall meet the requirements of the AISC *Seismic Provisions*.

Within a zone extending from 12 in. (300 mm) above the upper beam flange to 12 in. (300 mm) below the lower beam flange, the web of the tee-shaped sections shall be welded to the web of the continuous I-shaped section with CJP groove welds with a pair of reinforcing fillet welds. The minimum size of fillet

welds shall be the lesser of $\frac{5}{16}$ in. (8 mm) or the thickness of the column web. Continuity plates shall conform to the requirements for wide-flange columns.

Exception: For SidePlate moment connections, the web of the tee-shaped section(s) may be welded to the web of the continuous I-shaped section with a pair of continuous fillet welds. The required strength of the fillet welds, ϕR_n , shall equal the shear developed at the column web to tee-shaped section connection where the shear force in the column is the smaller of

- (a) The shear strength of the column section per AISC *Specification* Equation G2-1.
- (b) The maximum shear that can be developed in the column when plastic hinge(s) form in the connected beam(s).

3. Hollow Structural Sections (HSS)

The width-to-thickness ratio, h/t_w , of HSS members shall conform to the requirements of the AISC *Seismic Provisions* and shall conform to additional cross-section profile limitations applicable to the individual connection as specified in the applicable chapter.

User Note: Only the ConXL and SidePlate connections allow the use of HSS sections.

2.4. CONNECTION DESIGN PARAMETERS

1. Resistance Factors

Where available strengths are calculated in accordance with the AISC *Specification*, the resistance factors specified therein shall apply. When available strengths are calculated in accordance with this Standard, the resistance factors ϕ_d and ϕ_n shall be used as specified in the applicable section of this Standard. The values of ϕ_d and ϕ_n shall be taken as follows:

- (a) For ductile limit states

$$\phi_d = 1.00$$
- (b) For nonductile limit states

$$\phi_n = 0.90$$

2. Plastic Hinge Location

The distance of the plastic hinge from the face of the column, S_h , shall be taken in accordance with the requirements for the individual connection as specified herein.

3. Probable Maximum Moment at Plastic Hinge

The probable maximum moment at the plastic hinge shall be:

$$M_{pr} = C_{pr} R_y F_y Z_e \quad (2.4-1)$$

where

R_y = ratio of the expected yield stress to the specified minimum yield stress, F_y , as specified in the AISC *Seismic Provisions*

Z_e = effective plastic section modulus of section (or connection) at location of the plastic hinge, in.³ (mm³)

C_{pr} = factor to account for peak connection strength, including strain hardening, local restraint, additional reinforcement and other connection conditions. Unless otherwise specifically indicated in this Standard, the value of C_{pr} shall be:

$$C_{pr} = \frac{F_y + F_u}{2F_y} \leq 1.2 \quad (2.4-2)$$

where

F_u = specified minimum tensile strength of yielding element, ksi (MPa)

F_y = specified minimum yield stress of yielding element, ksi (MPa)

4. Continuity Plates

Beam flange continuity plates shall be provided in accordance with the AISC *Seismic Provisions*.

Exceptions:

1. For bolted end-plate connections, continuity plates shall be provided in accordance with Section 6.5.
2. For the Kaiser bolted bracket connection, the provisions of Chapter 9 shall apply. When continuity plates are required by Chapter 9, thickness and detailing shall be in accordance with the AISC *Seismic Provisions*.
3. For the SidePlate connection, beam flange continuity plates are not required. Horizontal shear plates as defined in Chapter 11 may be required.
4. For the Simpson Strong-Tie Strong Frame connection, continuity plates shall be provided in accordance with Section 12.9.

2.5. PANEL ZONES

Panel zones shall conform to the requirements of the AISC *Seismic Provisions*.

Exception: For the SidePlate moment connection, the contribution of the side plates to the overall panel zone strength shall be considered as described in Section 11.4(2).

2.6. PROTECTED ZONE

The protected zone shall be as defined for each prequalified connection. Unless otherwise specifically indicated in this Standard, the protected zone of the beam shall be

defined as the area from the face of the column flange to one-half of the beam depth beyond the plastic hinge. The protected zone shall meet the requirements of the AISC *Seismic Provisions*, except as indicated in this Standard. Bolt holes in beam webs, when detailed in accordance with the individual connection provisions of this Standard, shall be permitted.

CHAPTER 3

WELDING REQUIREMENTS

3.1. FILLER METALS

Filler metals shall conform to the requirements of the AISC *Seismic Provisions*.

3.2. WELDING PROCEDURES

Welding procedures shall be in accordance with the AISC *Seismic Provisions*.

3.3. BACKING AT BEAM-TO-COLUMN AND CONTINUITY PLATE-TO-COLUMN JOINTS

1. Steel Backing at Continuity Plates

Steel backing used at continuity plate-to-column welds need not be removed. At column flanges, steel backing left in place shall be attached to the column flange using a continuous $\frac{5}{16}$ -in. (8-mm) fillet weld on the edge below the CJP groove weld.

When backing is removed, the root pass shall be backgouged to sound weld metal and backwelded with a reinforcing fillet. The reinforcing fillet shall be continuous with a minimum size of $\frac{5}{16}$ in. (8 mm).

2. Steel Backing at Beam Bottom Flange

Where steel backing is used with CJP groove welds between the bottom beam flange and the column, the backing shall be removed.

Following the removal of steel backing, the root pass shall be backgouged to sound weld metal and backwelded with a reinforcing fillet. The size of the reinforcing fillet leg adjacent to the column flange shall be a minimum of $\frac{5}{16}$ in. (8 mm), and the reinforcing fillet leg adjacent to the beam flange shall be such that the fillet toe is located on the beam flange base metal.

Exception: If the base metal and weld root are ground smooth after removal of the backing, the reinforcing fillet adjacent to the beam flange need not extend to base metal.

3. Steel Backing at Beam Top Flange

Where steel backing is used with CJP groove welds between the top beam flange and the column, and the steel backing is not removed, the steel backing shall be attached to the column by a continuous $\frac{5}{16}$ -in. (8-mm) fillet weld on the edge below the CJP groove weld.

4. Prohibited Welds at Steel Backing

Backing at beam flange-to-column flange joints shall not be welded to the underside of the beam flange, nor shall tack welds be permitted at this location. If fillet welds or tack welds are placed between the backing and the beam flange in error, they shall be repaired as follows:

- (1) The weld shall be removed such that the fillet weld or tack weld no longer attaches the backing to the beam flange.
- (2) The surface of the beam flange shall be ground flush and shall be free of defects.
- (3) Any gouges or notches shall be repaired. Repair welding shall be done with E7018 SMAW electrodes or other filler metals meeting the requirements of Section 3.1 for demand critical welds. A special welding procedure specification (WPS) is required for this repair. Following welding, the repair weld shall be ground smooth.

5. Nonfusible Backing at Beam Flange-to-Column Joints

Where nonfusible backing is used with CJP groove welds between the beam flanges and the column, the backing shall be removed and the root backgouged to sound weld metal and backwelded with a reinforcing fillet. The size of the reinforcing fillet leg adjacent to the column shall be a minimum of $\frac{5}{16}$ in. (8 mm), and the reinforcing fillet leg adjacent to the beam flange shall be such that the fillet toe is located on the beam flange base metal.

Exception: If the base metal and weld root are ground smooth after removal of the backing, the reinforcing fillet adjacent to the beam flange need not extend to base metal.

3.4. WELD TABS

Where used, weld tabs shall be removed to within $\frac{1}{8}$ in. (3 mm) of the base metal surface and the end of the weld finished, except at continuity plates where removal to within $\frac{1}{4}$ in. (6 mm) of the plate edge shall be permitted. Removal shall be by air carbon arc cutting (CAC-A), grinding, chipping, or thermal cutting. The process shall be controlled to minimize errant gouging. The edges where weld tabs have been removed shall be finished to a surface roughness of 500 μ -in. (13 microns) or better. The contour of the weld end shall provide a smooth transition to adjacent surfaces, free of notches, gouges, and sharp corners. Weld defects greater than $\frac{1}{16}$ in. (2 mm) deep shall be excavated and repaired by welding in accordance with an applicable WPS. Other weld defects shall be removed by grinding, faired to a slope not greater than 1:5.

3.5. TACK WELDS

In the protected zone, tack welds attaching backing and weld tabs shall be placed where they will be incorporated into a final weld.

3.6. CONTINUITY PLATES

Along the web, the corner clip shall be detailed so that the clip extends a distance of at least $1\frac{1}{2}$ in. (38 mm) beyond the published k_{det} dimension for the rolled shape. Along the flange, the plate shall be clipped to avoid interference with the fillet radius of the rolled shape and shall be detailed so that the clip does not exceed a distance of $\frac{1}{2}$ in. (13 mm) beyond the published k_1 dimension. The clip shall be detailed to facilitate suitable weld terminations for both the flange weld and the web weld. When a curved corner clip is used, it shall have a minimum radius of $\frac{1}{2}$ in. (13 mm).

At the end of the weld adjacent to the column web/flange juncture, weld tabs for continuity plates shall not be used, except when permitted by the engineer of record. Unless specified to be removed by the engineer of record, weld tabs shall not be removed when used in this location.

Where continuity plate welds are made without weld tabs near the column fillet radius, weld layers shall be permitted to be transitioned at an angle of 0° to 45° measured from the vertical plane. The effective length of the weld shall be defined as that portion of the weld having full size. Nondestructive testing (NDT) shall not be required on the tapered or transition portion of the weld not having full size.

3.7. QUALITY CONTROL AND QUALITY ASSURANCE

Quality control and quality assurance shall be in accordance with the AISC *Seismic Provisions*.

CHAPTER 4

BOLTING REQUIREMENTS

4.1. FASTENER ASSEMBLIES

Bolts shall be pretensioned high-strength bolts conforming to ASTM F3125 Grades A325, A325M, A490, A490M, F1852 or F2280, unless other fasteners are permitted by a specific connection.

4.2. INSTALLATION REQUIREMENTS

Installation requirements shall be in accordance with AISC *Seismic Provisions* and the RCSC *Specification*, except as otherwise specifically indicated in this Standard.

4.3. QUALITY CONTROL AND QUALITY ASSURANCE

Quality control and quality assurance shall be in accordance with the AISC *Seismic Provisions*.

CHAPTER 5

REDUCED BEAM SECTION (RBS) MOMENT CONNECTION

5.1. GENERAL

In a reduced beam section (RBS) moment connection (Figure 5.1), portions of the beam flanges are selectively trimmed in the region adjacent to the beam-to-column connection. Yielding and hinge formation are intended to occur primarily within the reduced section of the beam.

5.2. SYSTEMS

RBS connections are prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems within the limits of these provisions.

5.3. PREQUALIFICATION LIMITS

1. Beam Limitations

Beams shall satisfy the following limitations:

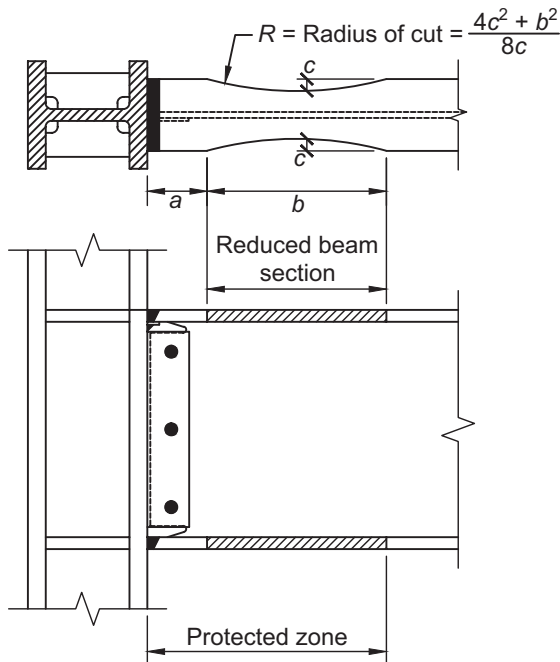


Fig. 5.1. Reduced beam section connection.

- (1) Beams shall be rolled wide-flange or built-up I-shaped members conforming to the requirements of Section 2.3.
- (2) Beam depth shall be limited to a maximum of W36 (W920) for rolled shapes. Depth of built-up sections shall not exceed the depth permitted for rolled wide-flange shapes.
- (3) Beam weight shall be limited to a maximum of 302 lb/ft (447 kg/m).
- (4) Beam flange thickness shall be limited to a maximum of 1¾ in. (44 mm).
- (5) The clear span-to-depth ratio of the beam shall be limited as follows:
 - (a) For SMF systems, 7 or greater.
 - (b) For IMF systems, 5 or greater.
- (6) Width-to-thickness ratios for the flanges and web of the beam shall conform to the requirements of the AISC *Seismic Provisions*.

When determining the width-to-thickness ratio of the flange, the value of b_f shall not be taken as less than the flange width at the ends of the center two-thirds of the reduced section provided that gravity loads do not shift the location of the plastic hinge a significant distance from the center of the reduced beam section.

- (7) Lateral bracing of beams shall be provided in conformance with the AISC *Seismic Provisions*. Supplemental lateral bracing shall be provided near the reduced section in conformance with the AISC *Seismic Provisions* for lateral bracing provided adjacent to the plastic hinges.

When supplemental lateral bracing is provided, its attachment to the beam shall be located no greater than $d/2$ beyond the end of the reduced beam section farthest from the face of the column, where d is the depth of the beam. No attachment of lateral bracing shall be made to the beam in the protected zone.

Exception: For both systems, where the beam supports a concrete structural slab that is connected between the protected zones with welded shear connectors spaced a maximum of 12 in. (300 mm) on center, supplemental top and bottom flange bracing at the reduced section is not required.

- (8) The protected zone shall consist of the portion of beam between the face of the column and the end of the reduced beam section cut farthest from the face of the column.

2. Column Limitations

Columns shall satisfy the following limitations:

- (1) Columns shall be any of the rolled shapes or built-up sections permitted in Section 2.3.
- (2) The beam shall be connected to the flange of the column.
- (3) Rolled shape column depth shall be limited to W36 (W920) maximum. The depth of built-up wide-flange columns shall not exceed that for rolled shapes.

Flanged cruciform columns shall not have a width or depth greater than the depth allowed for rolled shapes. Built-up box-columns shall not have a width or depth exceeding 24 in. (610 mm). Boxed wide-flange columns shall not have a width or depth exceeding 24 in. (610 mm) if participating in orthogonal moment frames.

- (4) There is no limit on the weight per foot of columns.
- (5) There are no additional requirements for flange thickness.
- (6) Width-to-thickness ratios for the flanges and web of columns shall conform to the requirements of the AISC *Seismic Provisions*.
- (7) Lateral bracing of columns shall conform to the requirements of the AISC *Seismic Provisions*.

5.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

Beam-to-column connections shall satisfy the following limitations:

- (1) Panel zones shall conform to the requirements of the AISC *Seismic Provisions*.
- (2) Column-beam moment ratios shall be limited as follows:
 - (a) For SMF systems, the column-beam moment ratio shall conform to the requirements of the AISC *Seismic Provisions*. The value of $\sum M_{pb}^*$ shall be taken equal to $\sum(M_{pr} + M_{uv})$, where M_{pr} is computed according to Equation 5.8-5, and where M_{uv} is the additional moment due to shear amplification from the center of the reduced beam section to the centerline of the column. M_{uv} can be computed as $V_{RBS}(a + b/2 + d_c/2)$, where V_{RBS} is the shear at the center of the reduced beam section computed per Step 4 of Section 5.8, a and b are the dimensions shown in Figure 5.1, and d_c is the depth of the column.
 - (b) For IMF systems, the column-beam moment ratio shall conform to the requirements of the AISC *Seismic Provisions*.

5.5. BEAM FLANGE-TO-COLUMN FLANGE WELD LIMITATIONS

Beam flange-to-column flange connections shall satisfy the following limitations:

- (1) Beam flanges shall be connected to column flanges using complete-joint-penetration (CJP) groove welds. Beam flange welds shall conform to the requirements for demand critical welds in the AISC *Seismic Provisions*.
- (2) Weld access hole geometry shall conform to the requirements of the AISC *Specification*.

5.6. BEAM WEB-TO-COLUMN FLANGE CONNECTION LIMITATIONS

Beam web to column flange connections shall satisfy the following limitations:

- (1) The required shear strength of the beam web connection shall be determined according to Equation 5.8-9.

- (2) Web connection details shall be limited as follows:
- (a) For SMF systems, the beam web shall be connected to the column flange using a CJP groove weld extending between weld access holes. The single plate shear connection shall extend between the weld access holes as shown in Figure 5.1. The single-plate shear connection shall be permitted to be used as backing for the CJP groove weld. The thickness of the plate shall be at least $\frac{3}{8}$ in. (10 mm). Weld tabs are not required at the ends of the CJP groove weld at the beam web. Bolt holes in the beam web for the purpose of erection are permitted.
 - (b) For IMF systems, the beam web shall be connected to the column flange as required for SMF systems.

Exception: For IMF, it is permitted to connect the beam web to the column flange using a bolted single-plate shear connection. The bolted single-plate shear connection shall be designed as a slip-critical connection, with the design slip resistance per bolt determined according to the AISC *Specification*. For seismic loading, the nominal bearing strength at bolt holes shall not be taken greater than the value given by Equation J3-6a of the AISC *Specification*. The design shear strength of the single-plate shear connection shall be determined based on shear yielding of the gross section and on shear rupture of the net section. The plate shall be welded to the column flange with a CJP groove weld or with fillet welds on both sides of the plate. The minimum size of the fillet weld on each side of the plate shall be 75% of the thickness of the plate. Standard holes shall be provided in the beam web and in the plate, except that short-slotted holes (with the slot parallel to the beam flanges) may be used in either the beam web or in the plate, but not in both. Bolts are permitted to be pretensioned either before or after welding.

5.7. FABRICATION OF FLANGE CUTS

The reduced beam section shall be made using thermal cutting to produce a smooth curve. The maximum surface roughness of the thermally cut surface shall be 500 μ -in. (13 microns) in accordance with ANSI B46.1, as measured using AWS C4.1 Sample 4 or a similar visual comparator. All transitions between the reduced beam section and the unmodified beam flange shall be rounded in the direction of the flange length to minimize notch effects due to abrupt transitions. Corners between the reduced section surface and the top and bottom of the flanges shall be ground to remove sharp edges, but a minimum chamfer or radius is not required.

Thermal cutting tolerances shall be plus or minus $\frac{1}{4}$ in. (6 mm) from the theoretical cut line. The beam effective flange width at any section shall have a tolerance of plus or minus $\frac{3}{8}$ in. (10 mm).

Gouges and notches that occur in the thermally cut RBS surface may be repaired by grinding if not more than $\frac{1}{4}$ in. (6 mm) deep. The gouged or notched area shall be faired in by grinding so that a smooth transition exists, and the total length of the area

ground for the transition shall be no less than five times the depth of the removed gouge on each side of the gouge. If a sharp notch exists, the area shall be inspected by magnetic particle testing (MT) after grinding to ensure that the entire depth of notch has been removed. Grinding that increases the depth of the RBS cut more than ¼ in. (6 mm) beyond the specified depth of cut is not permitted.

Gouges and notches that exceed ¼ in. (6 mm) in depth, but not exceeding ½ in. (13 mm) in depth, and those notches and gouges where repair by grinding would increase the effective depth of the RBS cut beyond tolerance may be repaired by welding. The notch or gouge shall be removed and ground to provide a smooth root radius of not less than ¼ in. (6 mm) in preparation for welding. The repair area shall be preheated to a minimum temperature of 150°F (66°C) or the value specified in AWS D1.1/D1.1M, whichever is greater, measured at the location of the weld repair.

Notches and gouges exceeding ½ in. (13 mm) in depth shall be repaired only with a method approved by the engineer of record.

5.8. DESIGN PROCEDURE

Step 1. Choose trial values for the beam sections, column sections and RBS dimensions a , b and c (Figure 5.1) subject to the limits:

$$0.5b_{bf} \leq a \leq 0.75b_{bf} \quad (5.8-1)$$

$$0.65d \leq b \leq 0.85d \quad (5.8-2)$$

$$0.1b_{bf} \leq c \leq 0.25b_{bf} \quad (5.8-3)$$

where

a = horizontal distance from face of column flange to start of an RBS cut, in. (mm)

b = length of RBS cut, in. (mm)

b_{bf} = width of beam flange, in. (mm)

c = depth of cut at center of reduced beam section, in. (mm)

d = depth of beam, in. (mm)

Confirm that the beams and columns are adequate for all load combinations specified by the applicable building code, including the reduced section of the beam, and that the design story drift for the frame complies with applicable limits specified by the applicable building code. Calculation of elastic drift shall consider the effect of the reduced beam section. In lieu of more detailed calculations, effective elastic drifts may be calculated by multiplying elastic drifts based on gross beam sections by 1.1 for flange reductions up to 50% of the beam flange width. Linear interpolation may be used for lesser values of beam width reduction.

Step 2. Compute the plastic section modulus at the center of the reduced beam section:

$$Z_{RBS} = Z_x - 2ct_{bf}(d - t_{bf}) \quad (5.8-4)$$

where

Z_{RBS} = plastic section modulus at center of reduced beam section, in.³ (mm³)

Z_x = plastic section modulus about x -axis, for full beam cross section, in.³ (mm³)

t_{bf} = thickness of beam flange, in. (mm)

Step 3. Compute the probable maximum moment, M_{pr} , at the center of the reduced beam section:

$$M_{pr} = C_{pr} R_y F_y Z_{RBS} \quad (5.8-5)$$

Step 4. Compute the shear force at the center of the reduced beam sections at each end of the beam.

The shear force at the center of the reduced beam sections shall be determined from a free-body diagram of the portion of the beam between the centers of the reduced beam sections. This calculation shall assume the moment at the center of each reduced beam section is M_{pr} and shall include gravity loads acting on the beam based on the load combination $1.2D + f_1 L + 0.2S$, where f_1 is the load factor determined by the applicable building code for live loads, but not less than 0.5.

User Note: The load combination of $1.2D + f_1 L + 0.2S$ is in conformance with ASCE/SEI 7-16. When using the International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.2 for S (snow) when the roof configuration is such that it does not shed snow off of the structure.

Step 5. Compute the probable maximum moment at the face of the column.

The moment at the face of the column shall be computed from a free-body diagram of the segment of the beam between the center of the reduced beam section and the face of the column, as illustrated in Figure 5.2.

Based on this free-body diagram, the moment at the face of the column is computed as follows:

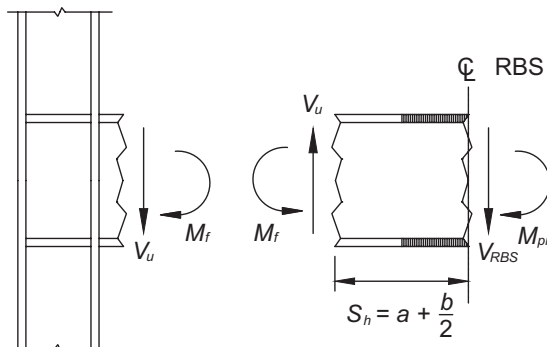


Fig. 5.2. Free-body diagram between center of RBS and face of column.

$$M_f = M_{pr} + V_{RBS}S_h \quad (5.8-6)$$

where

M_f = probable maximum moment at face of column, kip-in. (N-mm)

S_h = distance from face of column to plastic hinge, in. (mm)

= $a + b/2$, in. (mm)

V_{RBS} = larger of the two values of shear force at center of the reduced beam section at each end of beam, kips (N)

Step 6. Compute M_{pe} , the plastic moment of the beam based on the expected yield stress:

$$M_{pe} = R_y F_y Z_x \quad (5.8-7)$$

Step 7. Check the flexural strength of the beam at the face of the column:

$$M_f \leq \phi_d M_{pe} \quad (5.8-8)$$

If Equation 5.8-8 is not satisfied, adjust the values of c , a and b , or adjust the section size, and repeat Steps 2 through 7.

Step 8. Determine the required shear strength, V_u , of beam and beam web-to-column connection from:

$$V_u = \frac{2M_{pr}}{L_h} + V_{gravity} \quad (5.8-9)$$

where

L_h = distance between plastic hinge locations, in. (mm)

$V_{gravity}$ = beam shear force resulting from $1.2D + f_1L + 0.2S$ (where f_1 is the load factor determined by the applicable building code for live loads, but not less than 0.5), kips (N)

V_u = required shear strength of beam and beam web-to-column connection, kips (N)

Check design shear strength of beam according to Chapter G of the AISC *Specification*.

User Note: The load combination of $1.2D + f_1L + 0.2S$ is in conformance with ASCE/SEI 7-16. When using the International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.2 for S (snow) when the roof configuration is such that it does not shed snow off of the structure.

Step 9. Design the beam web-to-column connection according to Section 5.6.

Step 10. Check continuity plate requirements according to Chapter 2.

Step 11. Check column-beam relationship limitations according to Section 5.4.

CHAPTER 6

BOLTED UNSTIFFENED AND STIFFENED EXTENDED END-PLATE MOMENT CONNECTIONS

6.1. GENERAL

Bolted end-plate connections are made by welding the beam to an end-plate and bolting the end-plate to a column flange. The three end-plate configurations shown in Figure 6.1 are covered in this section and are prequalified under the *AISC Seismic Provisions* within the limitations of this Standard.

The behavior of this type of connection can be controlled by a number of different limit states including flexural yielding of the beam section, flexural yielding of the end-plates, yielding of the column panel zone, tension rupture of the end-plate bolts, shear rupture of the end-plate bolts, or rupture of various welded joints. The design criteria provide sufficient strength in the elements of the connections to ensure that the inelastic deformation of the connection is achieved by beam yielding.

6.2. SYSTEMS

Extended end-plate moment connections are prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems.

Exception: Extended end-plate moment connections with concrete structural slabs are prequalified only if:

- (1) In addition to the limitations of Section 6.3, the nominal beam depth is not less than 24 in. (600 mm);
- (2) There are no shear connectors within 1.5 times the beam depth from the face of the connected column flange; and

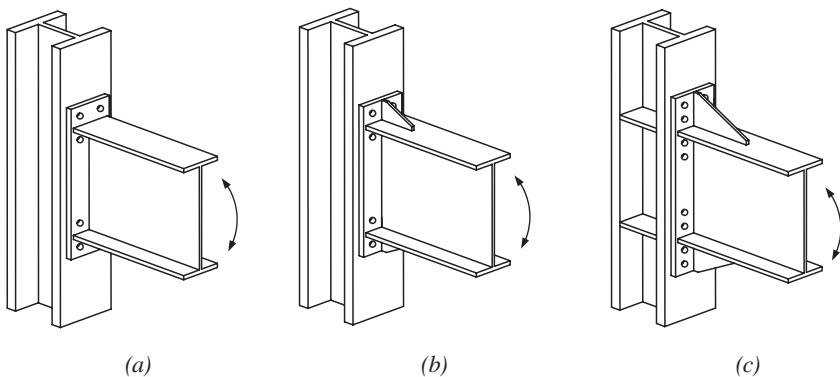


Fig. 6.1. Extended end-plate configurations: (a) four-bolt unstiffened, 4E; (b) four-bolt stiffened, 4ES; (c) eight-bolt stiffened, 8ES.

- (3) The concrete structural slab is kept at least 1 in. (25 mm) from both sides of both column flanges. It is permitted to place compressible material in the gap between the column flanges and the concrete structural slab.

6.3. PREQUALIFICATION LIMITS

Table 6.1 is a summary of the range of parameters that have been satisfactorily tested. All connection elements shall be within the ranges shown.

1. Beam Limitations

Beams shall satisfy the following limitations:

- (1) Beams shall be rolled wide-flange or built-up I-shaped members conforming to the requirements of Section 2.3. At moment-connected ends of welded built-up sections, within at least the depth of beam or three times the width of flange, whichever is less, the beam web and flanges shall be connected using either a complete-joint-penetration (CJP) groove weld or a pair of fillet welds each having a size 75% of the beam web thickness but not less than ¼ in. (6 mm). For the remainder of the beam, the weld size shall not be less than that required to accomplish shear transfer from the web to the flanges.
- (2) Beam depth, d , shall be limited to values shown in Table 6.1.
- (3) There is no limit on the weight per foot of beams.
- (4) Beam flange thickness shall be limited to the values shown in Table 6.1.
- (5) The clear span-to-depth ratio of the beam shall be limited as follows:
 - (a) For SMF systems, 7 or greater.
 - (b) For IMF systems, 5 or greater.
- (6) Width-to-thickness ratios for the flanges and web of the beam shall conform to the requirements of the AISC *Seismic Provisions*.
- (7) Lateral bracing of beams shall be provided in accordance with the AISC *Seismic Provisions*.
- (8) The protected zone shall be determined as follows:
 - (a) For unstiffened extended end-plate connections: the portion of beam between the face of the column and a distance equal to the depth of the beam or three times the width of the beam flange from the face of the column, whichever is less.
 - (b) For stiffened extended end-plate connections: the portion of beam between the face of the column and a distance equal to the location of the end of the stiffener plus one-half the depth of the beam or three times the width of the beam flange, whichever is less.

2. Column Limitations

Columns shall satisfy the following limitations:

TABLE 6.1
Parametric Limitations on Prequalification

Parameter	Four-Bolt Unstiffened (4E)		Four-Bolt Stiffened (4ES)		Eight-Bolt Stiffened (8ES)	
	Maximum in. (mm)	Minimum in. (mm)	Maximum in. (mm)	Minimum in. (mm)	Maximum in. (mm)	Minimum in. (mm)
t_{bf}	$\frac{3}{4}$ (19)	$\frac{3}{8}$ (10)	$\frac{3}{4}$ (19)	$\frac{3}{8}$ (10)	1 (25)	$\frac{9}{16}$ (14)
b_{bf}	9 $\frac{1}{4}$ (235)	6 (152)	9 (229)	6 (152)	12 $\frac{1}{4}$ (311)	7 $\frac{1}{2}$ (190)
d	55 (1400)	13 $\frac{3}{4}$ (349)	24 (610)	13 $\frac{3}{4}$ (349)	36 (914)	18 (457)
t_p	2 $\frac{1}{4}$ (57)	$\frac{1}{2}$ (13)	1 $\frac{1}{2}$ (38)	$\frac{1}{2}$ (13)	2 $\frac{1}{2}$ (64)	$\frac{3}{4}$ (19)
b_p	10 $\frac{3}{4}$ (273)	7 (178)	10 $\frac{3}{4}$ (273)	7 (178)	15 (381)	9 (229)
g	6 (152)	4 (102)	6 (152)	3 $\frac{1}{4}$ (83)	6 (152)	5 (127)
p_{fi}, p_{fo}	4 $\frac{1}{2}$ (114)	1 $\frac{1}{2}$ (38)	5 $\frac{1}{2}$ (140)	1 $\frac{3}{4}$ (44)	2 (51)	1 $\frac{5}{8}$ (41)
p_b	—	—	—	—	3 $\frac{3}{4}$ (95)	3 $\frac{1}{2}$ (89)

b_{bf} = width of beam flange, in. (mm)
 b_p = width of end-plate, in. (mm)
 d = depth of connecting beam, in. (mm)
 g = horizontal distance between bolts, in. (mm)
 p_b = vertical distance between the inner and outer row of bolts in an 8ES connection, in. (mm)
 p_{fi} = vertical distance from the inside of a beam tension flange to the nearest inside bolt row, in. (mm)
 p_{fo} = vertical distance from the outside of a beam tension flange to the nearest outside bolt row, in. (mm)
 t_{bf} = thickness of beam flange, in. (mm)
 t_p = thickness of end-plate, in. (mm)

- (1) Columns shall be any of the rolled shapes or built-up sections permitted in Section 2.3.
- (2) The end-plate shall be connected to the flange of the column.
- (3) Rolled shape column depth shall be limited to W36 (W920) maximum. The depth of built-up wide-flange columns shall not exceed that for rolled shapes. Flanged cruciform columns shall not have a width or depth greater than the depth allowed for rolled shapes.
- (4) There is no limit on the weight per foot of columns.
- (5) There are no additional requirements for flange thickness.
- (6) Width-to-thickness ratios for the flanges and web of the column shall conform to the requirements of the AISC *Seismic Provisions*.
- (7) Lateral bracing of columns shall conform to the requirements of the AISC *Seismic Provisions*.

6.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

Beam-to-column connections shall satisfy the following limitations:

- (1) Panel zones shall conform to the requirements of the AISC *Seismic Provisions*.
- (2) Column-beam moment ratios shall conform to the requirements of the AISC *Seismic Provisions*.

6.5. CONTINUITY PLATES

Continuity plates shall satisfy the following limitations:

- (1) The need for continuity plates shall be determined in accordance with Section 6.8.
- (2) When provided, continuity plates shall conform to the requirements of Section 6.8.
- (3) Continuity plates shall be attached to columns by welds in accordance with the AISC *Seismic Provisions*.

Exception: Continuity plates less than or equal to $\frac{3}{8}$ in. (10 mm) shall be permitted to be welded to column flanges using double-sided fillet welds. The required strength of the fillet welds shall not be less than $F_y A_c$, where A_c is defined as the contact areas between the continuity plate and the column flanges that have attached beam flanges and F_y is defined as the specified minimum yield stress of the continuity plate.

6.6. BOLTS

Bolts shall conform to the requirements of Chapter 4.

6.7. CONNECTION DETAILING

1. Gage

The gage, g , is as defined in Figures 6.2 through 6.4. The maximum gage dimension is limited to the width of the connected beam flange.

2. Pitch and Row Spacing

The minimum pitch distance is the bolt diameter plus $\frac{1}{2}$ in. (13 mm) for bolts up to 1 in. (25 mm) diameter, and the bolt diameter plus $\frac{3}{4}$ in. (19 mm) for larger diameter bolts. The pitch distances, p_{fi} and p_{fo} , are the distances from the face of the beam flange to the centerline of the nearer bolt row, as shown in Figures 6.2 through 6.4. The pitch distances, p_{si} and p_{so} , are the distances from the face of the continuity plate to the centerline of the nearer bolt row, as shown in Figures 6.2 through 6.4.

The spacing, p_b , is the distance between the inner and outer row of bolts in an 8ES end-plate moment connection and is shown in Figure 6.4. The spacing of the bolt rows shall be at least $2\frac{2}{3}$ times the bolt diameter.

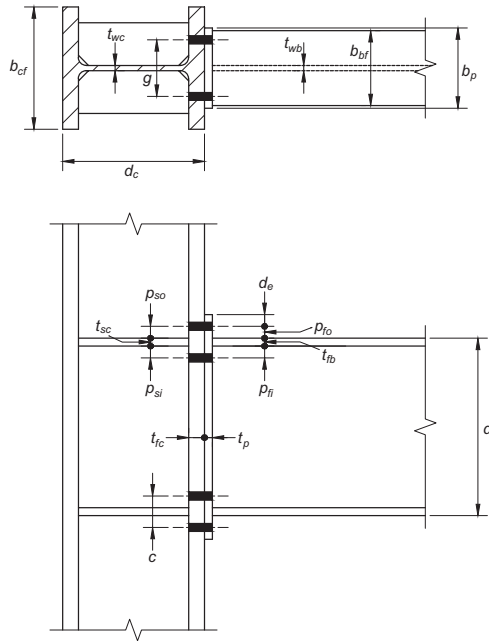


Fig. 6.2. Four-bolt unstiffened extended end-plate (4E) geometry.

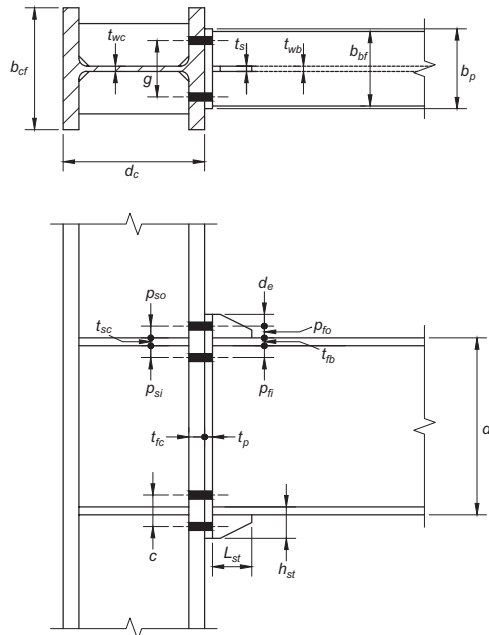


Fig. 6.3. Four-bolt stiffened extended end-plate (4ES) geometry.

User Note: A distance of three times the bolt diameter is preferred. The distance must be sufficient to provide clearance for any welds in the region.

3. End-Plate Width

The width of the end-plate shall be greater than or equal to the connected beam flange width. The effective end-plate width shall not be taken greater than the connected beam flange plus 1 in. (25 mm).

4. End-Plate Stiffener

The two extended stiffened end-plate connections, Figures 6.1(b) and (c), require a stiffener welded between the connected beam flange and the end-plate. The minimum stiffener length, L_{st} , shall be:

$$L_{st} = \frac{h_{st}}{\tan 30^\circ} \quad (6.9-1)$$

where h_{st} is the height of the stiffener, equal to the height of the end-plate from the outside face of the beam flange to the end of the end-plate as shown in Figure 6.5.

The stiffener plates shall be terminated at the beam flange and at the end of the end-plate with landings not less than 1 in. (25 mm) long. The stiffener shall be clipped

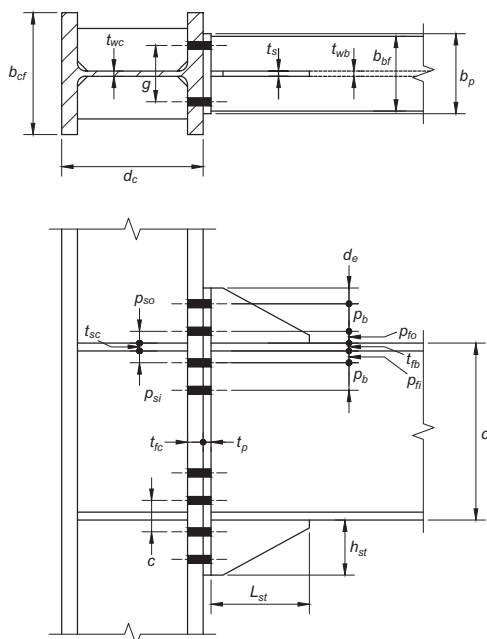


Fig. 6.4. Eight-bolt stiffened extended end-plate (8ES) geometry.

where it meets the beam flange and end-plate to provide clearance between the stiffener and the beam flange weld.

When the beam and end-plate stiffeners have the same material strengths, the thickness of the stiffeners shall be greater than or equal to the beam web thickness. If the beam and end-plate stiffener have different material strengths, the thickness of the stiffener shall not be less than the ratio of the beam-to-stiffener plate material yield stresses times the beam web thickness.

5. Finger Shims

The use of finger shims (illustrated in Figure 6.6) at the top and/or bottom of the connection and on either or both sides is permitted, subject to the limitations of the RCSC *Specification*.

6. Welding Details

Welding of the beam to the end-plate shall conform to the following limitations:

- (1) Weld access holes shall not be used.
- (2) The beam flange to end-plate joint shall be made using a CJP groove weld without backing. The CJP groove weld shall be made such that the root of the weld

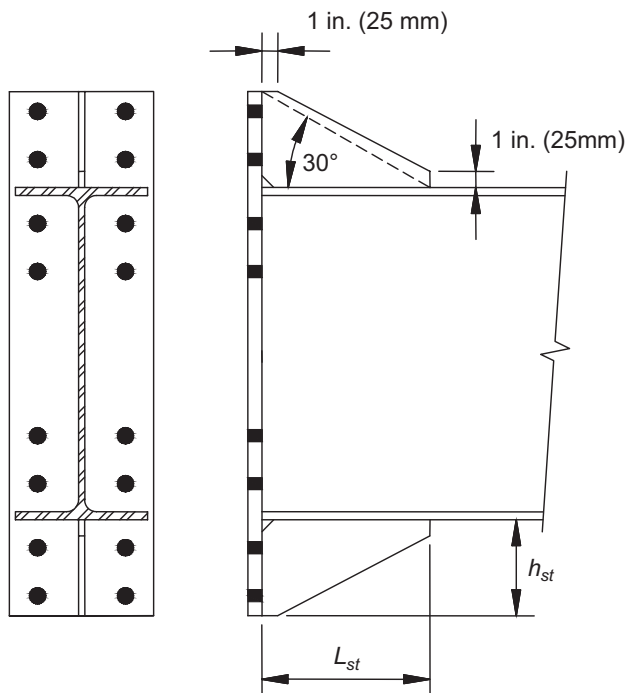


Fig. 6.5. End-plate stiffener layout and geometry for 8ES. Geometry for 4ES similar.

is on the beam web side of the flange. The inside face of the flange shall have a $\frac{5}{16}$ -in. (8-mm) fillet weld. These welds shall be demand critical.

- (3) The beam web to end-plate joint shall be made using either fillet welds or CJP groove welds. When used, the fillet welds shall be sized to develop the full strength of the beam web in tension from the inside face of the flange to 6 in. (150 mm) beyond the bolt row farthest from the beam flange.
- (4) Backgouging of the root is not required in the flange directly above and below the beam web for a length equal to $1.5k_1$. A full-depth PJP groove weld shall be permitted at this location.
- (5) When used, all end-plate-to-stiffener joints shall be made using CJP groove welds.

Exception: When the stiffener is $\frac{3}{8}$ in. (10 mm) thick or less, it is permitted to use fillet welds that develop the strength of the stiffener.

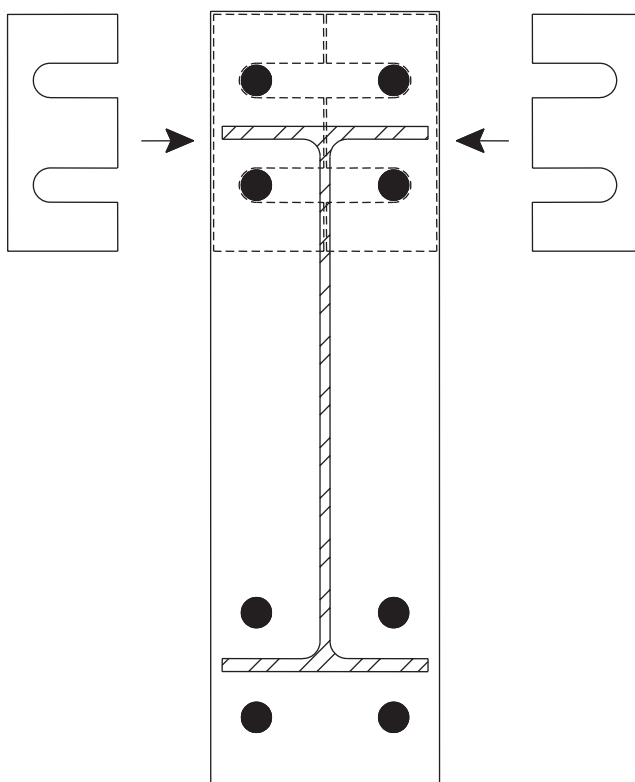


Fig. 6.6. Typical use of finger shims.

6.8. DESIGN PROCEDURE

Connection geometry is shown in Figures 6.2, 6.3 and 6.4 for the 4E, 4ES and 8ES connections, respectively.

1. End-Plate and Bolt Design

Step 1. Determine the sizes of the connected members (beams and column) and compute the moment at the face of the column, M_f .

$$M_f = M_{pr} + V_u S_h \quad (6.8-1)$$

where

L_h = distance between plastic hinge locations, in. (mm)

L_{st} = length of stiffener, as shown in Figure 6.5, in. (mm)

M_{pr} = probable maximum moment at plastic hinge, kip-in. (N-mm), given by Equation 2.4-1

S_h = distance from face of column to plastic hinge, in. (mm)

= the lesser of $d/2$ or $3b_{bf}$ for an unstiffened connection (4E)

= $L_{st} + t_p$ for a stiffened connection (4ES, 8ES)

$V_{gravity}$ = beam shear force resulting from $1.2D + f_1L + 0.2S$ (where f_1 is a load factor determined by the applicable building code for live loads, but not less than 0.5), kips (N)

V_u = shear force at end of beam, kips (N)

$$= \frac{2M_{pr}}{L_h} + V_{gravity} \quad (6.8-2)$$

b_{bf} = width of beam flange, in. (mm)

d = depth of connecting beam, in. (mm)

t_p = thickness of end-plate, in. (mm)

User Note: The load combination of $1.2D + f_1L + 0.2S$ is in conformance with ASCE/SEI 7-16. When using the International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.2 for S (snow) when the roof configuration is such that it does not shed snow off of the structure.

Step 2. Select one of the three end-plate moment connection configurations and establish preliminary values for the connection geometry (g , p_{fi} , p_{fo} , p_b , g , h_i , etc.) and bolt grade.

Step 3. Determine the required bolt diameter, $d_{b, req'd}$, using one of the following expressions.

For four-bolt connections (4E, 4ES):

$$d_{b, req} = \sqrt{\frac{2M_f}{\pi\phi_n F_{nt} (h_o + h_1)}} \quad (6.8-3)$$

For eight-bolt connections (8ES):

$$d_{b,req} = \sqrt{\frac{2M_f}{\pi\phi_n F_{nt}(h_1 + h_2 + h_3 + h_4)}} \quad (6.8-4)$$

where

F_{nt} = nominal tensile strength of bolt from the AISC *Specification*, ksi (MPa)

h_i = distance from centerline of the beam compression flange to the centerline of i th tension bolt row

h_o = distance from centerline of compression flange to tension-side outer bolt row, in. (mm)

$\phi_n = 0.90$

Step 4. Select a trial bolt diameter, d_b , not less than that required in Section 6.8.1 Step 3.

Step 5. Determine the required end-plate thickness, $t_{p,req'd}$.

$$t_{p,req} = \sqrt{\frac{1.11 M_f}{\phi_d F_{yp} Y_p}} \quad (6.8-5)$$

where

F_{yp} = specified minimum yield stress of end-plate material, ksi (MPa)

Y_p = end-plate yield line mechanism parameter from Tables 6.2, 6.3 or 6.4, in. (mm)

$\phi_d = 1.00$

Step 6. Select an end-plate thickness, t_p , not less than the required value.

Step 7. Calculate F_{fu} , the factored beam flange force.

$$F_{fu} = \frac{M_f}{d - t_{bf}} \quad (6.8-6)$$

where

d = depth of beam, in. (mm)

t_{bf} = thickness of beam flange, in. (mm)

Step 8. Check shear yielding of the extended portion of the four-bolt extended unstiffened end-plate (4E):

$$F_{fu}/2 \leq \phi_d R_n = \phi_d (0.6) F_{yp} b_p t_p \quad (6.8-7)$$

where b_p is the width of the end-plate, in. (mm), to be taken as not greater than the width of the beam flange plus 1 in. (25 mm).

If Equation 6.8-7 is not satisfied, increase the end-plate thickness or increase the yield stress of the end-plate material.

TABLE 6.2
Summary of Four-Bolt Extended Unstiffened
End-Plate Yield Line Mechanism Parameter

End-Plate Geometry and Yield Line Pattern	Bolt Force Model
End-Plate	$Y_p = \frac{b_p}{2} \left[h_1 \left(\frac{1}{p_{fi}} + \frac{1}{s} \right) + h_0 \left(\frac{1}{p_{io}} \right) - \frac{1}{2} \right] + \frac{2}{g} [h_1(p_{fi} + s)]$ $s = \frac{1}{2} \sqrt{b_p g} \quad \text{Note: If } p_{fi} > s, \text{ use } p_{fi} = s.$

Step 9. Check shear rupture of the extended portion of the end-plate in the four-bolt extended unstiffened end-plate (4E):

$$F_{fu}/2 \leq \phi_n R_n = \phi_n (0.6) F_{up} A_n \quad (6.8-8)$$

where

A_n = net area of end-plate

= $t_p [b_p - 2(d_b + 1/8)]$ when standard holes are used, in.²

= $t_p [b_p - 2(d_b + 3)]$ when standard holes are used, mm²

F_{up} = specified minimum tensile stress of end-plate, ksi (MPa)

d_b = bolt diameter, in. (mm)

If Equation 6.8-8 is not satisfied, increase the end-plate thickness or increase the yield stress of the end-plate material.

Step 10. If using either the four-bolt extended stiffened end-plate (4ES) or the eight-bolt extended stiffened end-plate (8ES) connection, select the end-plate stiffener thickness and design the stiffener-to-beam flange and stiffener-to-end-plate welds.

$$t_s \geq t_{bw} \left(\frac{F_{yb}}{F_{ys}} \right) \quad (6.8-9)$$

TABLE 6.3
Summary of Four-Bolt Extended Stiffened
End-Plate Yield Line Mechanism Parameter

End-Plate Geometry and Yield Line Pattern		Bolt Force Model
Case 1 ($d_e \leq s$)	Case 2 ($d_e > s$)	
Case 1 ($d_e \leq s$)	$Y_p = \frac{b_p}{2} \left[h_1 \left(\frac{1}{\rho_{fi}} + \frac{1}{s} \right) + h_0 \left(\frac{1}{\rho_{to}} + \frac{1}{2s} \right) \right] + \frac{2}{g} \left[h_1 (\rho_{fi} + s) + h_0 (d_e + \rho_{to}) \right]$	
Case 2 ($d_e > s$)	$Y_p = \frac{b_p}{2} \left[h_1 \left(\frac{1}{\rho_{fi}} + \frac{1}{s} \right) + h_0 \left(\frac{1}{s} + \frac{1}{\rho_{to}} \right) \right] + \frac{2}{g} \left[h_1 (\rho_{fi} + s) + h_0 (s + \rho_{to}) \right]$	
$s = \frac{1}{2} \sqrt{b_p g} \quad \text{Note: If } \rho_{fi} > s, \text{ use } \rho_{fi} = s.$		

where

F_{yb} = specified minimum yield stress of beam material, ksi (MPa)

F_{ys} = specified minimum yield stress of stiffener material, ksi (MPa)

t_{bw} = thickness of beam web, in. (mm)

t_s = end-plate stiffener thickness, in. (mm)

The stiffener geometry shall conform to the requirements of Section 6.7.4. In addition, to prevent local buckling of the stiffener plate, the following width-to-thickness criterion shall be satisfied:

$$\frac{h_{st}}{t_s} \leq 0.56 \sqrt{\frac{E}{F_{ys}}} \quad (6.8-10)$$

where h_{st} is the height of the stiffener, in. (mm), equal to the height of the end-plate from the outside face of the beam flange to the end of the end-plate.

TABLE 6.4
Summary of Eight-Bolt Extended Stiffened
End-Plate Yield Line Mechanism Parameter

End-Plate Geometry and Yield Line Pattern		Bolt Force Model
Case 1 ($d_e \leq s$)	Case 2 ($d_e > s$)	
<p>Case 1 ($d_e \leq s$)</p>	$Y_p = \frac{b_p}{2} \left[h_1 \left(\frac{1}{2d_e} \right) + h_2 \left(\frac{1}{p_{fo}} \right) + h_3 \left(\frac{1}{p_{fi}} \right) + h_4 \left(\frac{1}{s} \right) \right]$ $+ \frac{2}{g} \left[h_1 \left(d_e + \frac{3p_b}{4} \right) + h_2 \left(p_{fo} + \frac{p_b}{4} \right) + h_3 \left(p_{fi} + \frac{3p_b}{4} \right) + h_4 \left(s + \frac{p_b}{4} \right) \right] + g$	
<p>Case 2 ($d_e > s$)</p>	$Y_p = \frac{b_p}{2} \left[h_1 \left(\frac{1}{s} \right) + h_2 \left(\frac{1}{p_{fo}} \right) + h_3 \left(\frac{1}{p_{fi}} \right) + h_4 \left(\frac{1}{s} \right) \right]$ $+ \frac{2}{g} \left[h_1 \left(s + \frac{p_b}{4} \right) + h_2 \left(p_{fo} + \frac{3p_b}{4} \right) + h_3 \left(p_{fi} + \frac{p_b}{4} \right) + h_4 \left(s + \frac{3p_b}{4} \right) \right] + g$	
$s = \frac{1}{2} \sqrt{b_p g} \quad \text{Note: If } p_{fi} > s, \text{ use } p_{fi} = s.$		

The stiffener-to-beam-flange and stiffener-to-end-plate welds shall be designed to develop the stiffener plate in shear at the beam flange and in tension at the end-plate. Either fillet or CJP groove welds are suitable for the weld of the stiffener plate to the beam flange. CJP groove welds shall be used for the stiffener-to-end-plate weld. If the end-plate is $\frac{3}{8}$ in. (10 mm) thick or less, double-sided fillet welds are permitted.

Step 11. The bolt shear rupture strength of the connection is provided by the bolts at one (compression) flange; thus

$$V_u \leq \phi_n R_n = \phi_n (n_b) F_{nv} A_b \quad (6.8-11)$$

where

- A_b = nominal gross area of bolt, in.² (mm²)
- F_{nv} = nominal shear strength of bolt from the AISC *Specification*, ksi (MPa)
- V_u = shear force at end of the beam, kips (N), given by Equation 6.8-2
- n_b = number of bolts at compression flange
 - = 4 for 4E and 4ES connections
 - = 8 for 8ES connections

Step 12. Check bolt-bearing/tear-out failure of the end-plate and column flange:

$$V_u \leq \phi_n R_n = \phi_n (n_i) r_{ni} + \phi_n (n_o) r_{no} \quad (6.8-12)$$

where

- F_u = specified minimum tensile strength of end-plate or column flange material, ksi (MPa)
- L_c = clear distance, in direction of force, between edge of the hole and edge of the adjacent hole or edge of material, in. (mm)
- d_b = diameter of bolt, in. (mm)
- n_i = number of inner bolts
 - = 2 for 4E and 4ES connections
 - = 4 for 8ES connections
- n_o = number of outer bolts
 - = 2 for 4E and 4ES connections
 - = 4 for 8ES connections
- r_{ni} = $1.2 L_c t F_u < 2.4 d_b t F_u$ for each inner bolt
- r_{no} = $1.2 L_c t F_u < 2.4 d_b t F_u$ for each outer bolt
- t = end-plate or column flange thickness, in. (mm)

Step 13. Design the flange-to-end-plate and web-to-end-plate welds using the requirements of Section 6.7.6.

2. Column-Side Design

Step 1. Check the column flange for flexural yielding:

$$t_{cf} \geq \sqrt{\frac{1.11 M_f}{\phi_d F_{yc} Y_c}} \quad (6.8-13)$$

where

- F_{yc} = specified minimum yield stress of column flange material, ksi (MPa)
- Y_c = unstiffened column flange yield line mechanism parameter from Table 6.5 or Table 6.6, in. (mm)
- t_{cf} = column flange thickness, in. (mm)

If Equation 6.8-13 is not satisfied, increase the column size or add continuity plates.

If continuity plates are added, check Equation 6.8-13 using Y_c for the stiffened column flange from Tables 6.5 and 6.6.

Step 2. If continuity plates are required for column flange flexural yielding, determine the required stiffener force.

TABLE 6.5
Summary of Four-Bolt Extended Column Flange
Yield Line Mechanism Parameter

Unstiffened Column Flange Geometry and Yield Line Pattern	Stiffened Column Flange Geometry and Yield Line Pattern
Unstiffened Column Flange	$Y_c = \frac{b_{cf}}{2} \left[h_1 \left(\frac{1}{s} \right) + h_0 \left(\frac{1}{s} \right) \right] + \frac{2}{g} \left[h_1 \left(s + \frac{3c}{4} \right) + h_0 \left(s + \frac{c}{4} \right) + \frac{c^2}{2} \right] + \frac{g}{2}$ $s = \frac{1}{2} \sqrt{b_{cf} g}$
Stiffened Column Flange	$Y_c = \frac{b_{cf}}{2} \left[h_1 \left(\frac{1}{s} + \frac{1}{p_{si}} \right) + h_0 \left(\frac{1}{s} + \frac{1}{p_{so}} \right) \right] + \frac{2}{g} \left[h_1 (s + p_{si}) + h_0 (s + p_{so}) \right]$ $s = \frac{1}{2} \sqrt{b_{cf} g} \quad \text{Note: If } p_{si} > s, \text{ use } p_{si} = s.$

The column flange flexural design strength is

$$\phi_d M_{cf} = \phi_d F_{yc} Y_c t_{cf}^2 \quad (6.8-14)$$

where Y_c is the unstiffened column yield line mechanism parameter from Table 6.5 or Table 6.6, in. (mm). Therefore, the equivalent column flange design force is

$$\phi_d R_n = \frac{\phi_d M_{cf}}{(d - t_{bf})} \quad (6.8-15)$$

Using $\phi_d R_n$, the required force for continuity plate design is determined in Section 6.8.2 Step 6.

TABLE 6.6
Summary of Eight-Bolt Extended Column Flange
Yield Line Mechanism Parameter

Unstiffened Column Flange Geometry and Yield Line Pattern	Stiffened Column Flange Geometry and Yield Line Pattern
<p>Unstiffened Column Flange</p>	$Y_c = \frac{b_{cf}}{2} \left[h_1 \left(\frac{1}{s} \right) + h_4 \left(\frac{1}{s} \right) \right]$ $+ \frac{2}{g} \left[h_1 \left(p_b + \frac{c}{2} + s \right) + h_2 \left(\frac{p_b}{2} + \frac{c}{4} \right) + h_3 \left(\frac{p_b}{2} + \frac{c}{2} \right) + h_4 (s) \right] + \frac{g}{2}$ $s = \frac{1}{2} \sqrt{b_{cf} g}$
<p>Stiffened Column Flange</p>	$Y_c = \frac{b_{cf}}{2} \left[h_1 \left(\frac{1}{s} \right) + h_2 \left(\frac{1}{p_{so}} \right) + h_3 \left(\frac{1}{p_{si}} \right) + h_4 \left(\frac{1}{s} \right) \right]$ $+ \frac{2}{g} \left[h_1 \left(s + \frac{p_b}{4} \right) + h_2 \left(p_{so} + \frac{3p_b}{4} \right) + h_3 \left(p_{si} + \frac{p_b}{4} \right) + h_4 \left(s + \frac{3p_b}{4} \right) + p_b^2 \right] + g$ $s = \frac{1}{2} \sqrt{b_{cf} g} \text{ Note: If } p_{si} > s, \text{ use } p_{si} = s.$

Step 3. Check the local column web yielding strength of the unstiffened column web at the beam flanges.

Strength requirement:

$$F_{ju} \leq \phi_d R_n \quad (6.8-16)$$

$$R_n = C_t(6k_c + t_{bf} + 2t_p)F_{yc}t_{cw} \quad (6.8-17)$$

where

C_t = 0.5 if the distance from the column top to the top face of the beam flange is less than the depth of the column

= 1.0 otherwise

F_{yc} = specified minimum yield stress of column web material, ksi (MPa)

k_c = distance from outer face of column flange to web toe of fillet (design value) or fillet weld, in. (mm)

t_{cw} = column web thickness, in. (mm)

If the strength requirement of Equation 6.8-16 is not satisfied, column web continuity plates are required.

Step 4. Check the unstiffened column web buckling strength at the beam compression flange.

Strength requirement:

$$F_{ju} \leq \phi R_n \quad (6.8-18)$$

where $\phi = 0.75$.

(a) When F_{ju} is applied at a distance greater than or equal to $d_c/2$ from the end of the column

$$R_n = \frac{24t_{cw}^3 \sqrt{EF_{yc}}}{h} \quad (6.8-19)$$

(b) When F_{ju} is applied at a distance less than $d_c/2$ from the end of the column

$$R_n = \frac{12t_{cw}^3 \sqrt{EF_{yc}}}{h} \quad (6.8-20)$$

where h is the clear distance between flanges less the fillet or corner radius for rolled shapes; clear distance between flanges when welds are used for built-up shapes, in. (mm)

If the strength requirement of Equation 6.8-18 is not satisfied, then column web continuity plates are required.

Step 5. Check the unstiffened column web crippling strength at the beam compression flange.

Strength requirement:

$$F_{fu} \leq \phi R_n \quad (6.8-21)$$

where $\phi = 0.75$.

- (a) When F_{fu} is applied at a distance greater than or equal to $d_c/2$ from the end of the column

$$R_n = 0.80 t_{cw}^2 \left[1 + 3 \left(\frac{N}{d_c} \right) \left(\frac{t_{cw}}{t_{cf}} \right)^{1.5} \right] \sqrt{\frac{EF_{yc}t_{cf}}{t_{cw}}} \quad (6.8-22)$$

- (b) When F_{fu} is applied at a distance less than $d_c/2$ from the end of the column

- (i) for $N/d_c \leq 0.2$,

$$R_n = 0.40 t_{cw}^2 \left[1 + 3 \left(\frac{N}{d_c} \right) \left(\frac{t_{cw}}{t_{cf}} \right)^{1.5} \right] \sqrt{\frac{EF_{yc}t_{cf}}{t_{cw}}} \quad (6.8-23)$$

- (ii) for $N/d_c > 0.2$,

$$R_n = 0.40 t_{cw}^2 \left[1 + \left(\frac{4N}{d_c} - 0.2 \right) \left(\frac{t_{cw}}{t_{cf}} \right)^{1.5} \right] \sqrt{\frac{EF_{yc}t_{cf}}{t_{cw}}} \quad (6.8-24)$$

where

$N = b_f + 2w + 2t_p$, in. (mm)

d_c = overall depth of column, in. (mm)

t_p = end-plate thickness, in. (mm)

w = leg size of fillet weld or groove weld reinforcement, if used, in. (mm)

If the strength requirement of Equation 6.8-21 is not satisfied, then column web continuity plates are required.

Step 6. If stiffener plates are required for any of the column side limit states, the required strength is

$$F_{su} = F_{fu} - \min(\phi R_n) \quad (6.8-25)$$

where $\min(\phi R_n)$ is the minimum design strength value from Section 6.8.2 Step 2 (column flange bending), Step 3 (column web yielding), Step 4 (column web buckling), and Step 5 (column web crippling).

The design of the continuity plates shall also conform to Chapter E of the AISC *Seismic Provisions*, and the welds shall be designed in accordance with Section 6.5(3).

Step 7. Check the panel zone in accordance with Section 6.4(1).

CHAPTER 7

BOLTED FLANGE PLATE (BFP) MOMENT CONNECTION

7.1. GENERAL

Bolted flange plate (BFP) moment connections utilize plates welded to column flanges and bolted to beam flanges. The top and bottom plates must be identical. Flange plates are welded to the column flange using CJP groove welds and beam flange connections are made with high-strength bolts. The beam web is connected to the column flange using a bolted shear tab with bolts in short-slotted holes. Details for this connection type are shown in Figure 7.1. Initial yielding and plastic hinge formation are intended to occur in the beam in the region near the end of the flange plates.

7.2. SYSTEMS

Bolted flange plate connections are prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems within the limitations of these provisions.

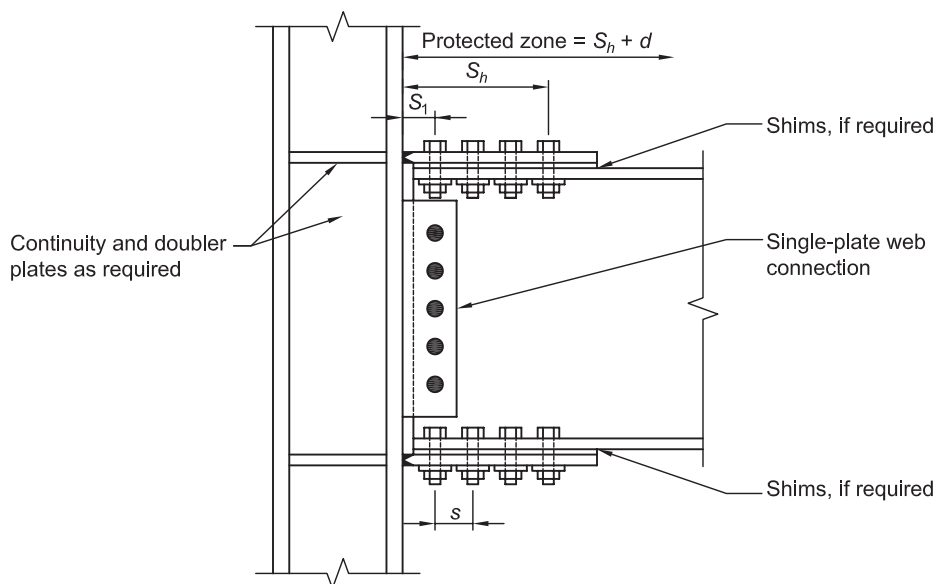


Fig. 7.1. Bolted flange plate moment connection.

Exception: Bolted flange plate connections in SMF systems with concrete structural slabs are only prequalified if the concrete structural slab is kept at least 1 in. (25 mm) from both sides of both column flanges. It is permissible to place compressible material in the gap between the column flanges and the concrete structural slab.

7.3. PREQUALIFICATION LIMITS

1. Beam Limitations

Beams shall satisfy the following limitations:

- (1) Beams shall be rolled wide-flange or built-up I-shaped members conforming to the requirements in Section 2.3.
- (2) Beam depth shall be limited to a maximum of W36 (W920) for rolled shapes. Depth of built-up sections shall not exceed the depth permitted for rolled wide-flange shapes.
- (3) Beam weight shall be limited to a maximum of 150 lb/ft (223 kg/m).
- (4) Beam flange thickness shall be limited to a maximum of 1 in. (25 mm).
- (5) The clear span-to-depth ratio of the beam shall be limited as follows:
 - (a) For SMF systems, 9 or greater.
 - (b) For IMF systems, 7 or greater.
- (6) Width-to-thickness ratios for the flanges and web of the beam shall conform to the requirements of the AISC *Seismic Provisions*.
- (7) Lateral bracing of beams shall be provided as follows:

Lateral bracing of beams shall conform to the requirements of the AISC *Seismic Provisions*. To satisfy the requirements of Chapter E of the AISC *Seismic Provisions* for lateral bracing at plastic hinges, supplemental lateral bracing shall be provided at both the top and bottom beam flanges, and shall be located a distance of d to $1.5d$ from the bolt farthest from the face of the column. No attachment of lateral bracing shall be made within the protected zone.

Exception: For both SMF and IMF systems, where the beam supports a concrete structural slab that is connected along the beam span between protected zones with welded shear connectors spaced at a maximum of 12 in. (300 mm) on center, supplemental top and bottom flange bracing at plastic hinges is not required.

- (8) The protected zone consists of the flange plates and the portion of the beam between the face of the column and a distance equal to one beam depth, d , beyond the bolt farthest from the face of the column.

2. Column Limitations

Columns shall satisfy the following limitations:

- (1) Columns shall be any of the rolled shapes or built-up sections permitted in Section 2.3.

- (2) The beam shall be connected to the flange of the column.
- (3) Rolled shape column depth shall be limited to W36 (W920) maximum when a concrete structural slab is provided. In the absence of a concrete structural slab, the rolled shape column depth is limited to W14 (W360) maximum. Flanged cruciform columns shall not have a width or depth greater than the depth allowed for rolled shapes. Built-up box columns shall not have a width or depth exceeding 24 in. (600 mm). Boxed wide-flange columns shall not have a width or depth exceeding 24 in. (600 mm) if participating in orthogonal moment frames.
- (4) There is no limit on weight per foot of columns.
- (5) There are no additional requirements for flange thickness.
- (6) Width-to-thickness ratios for the flanges and web of columns shall conform to the requirements of the AISC *Seismic Provisions*.
- (7) Lateral bracing of columns shall conform to the requirements of the AISC *Seismic Provisions*.

7.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

Beam-to-column connections shall satisfy the following limitations:

- (1) Panel zones shall conform to the requirements of the AISC *Seismic Provisions*.
- (2) Column-beam moment ratios shall conform to the requirements of the AISC *Seismic Provisions*.

7.5. CONNECTION DETAILING

1. Plate Material Specifications

All connection plates shall conform to one of the following specifications: ASTM A36/A36M or A572/A572M Grade 50 (345).

2. Beam Flange Plate Welds

Flange plates shall be connected to the column flange using CJP groove welds and shall be considered demand critical. Backing, if used, shall be removed. The root pass shall be backgouged to sound weld metal and back welded.

3. Single-Plate Shear Connection Welds

The single-plate shear connection shall be welded to the column flange. The single-plate to column-flange connection shall consist of CJP groove welds, two-sided PJP groove welds, or two-sided fillet welds.

4. Bolt Requirements

Bolts shall be arranged symmetrically about the axes of the beam and shall be limited to two bolts per row in the flange plate connections. The length of the bolt group shall

not exceed the depth of the beam. Standard holes shall be used in beam flanges. Holes in flange plates shall be standard or oversized holes. Bolt holes in beam flanges and in flange plates shall be made by drilling or by sub-punching and reaming. Punched holes are not permitted.

User Note: Although standard holes are permitted in the flange plate, their use will likely result in field modifications to accommodate erection tolerances.

Bolts in the flange plates shall be ASTM F3125 Grade A490, Grade A490M or Grade F2280 assemblies. Threads shall be excluded from the shear plane. Bolt diameter is limited to 1½ in. (28 mm) maximum.

5. Flange Plate Shims

Shims with a maximum overall thickness of ¼ in. (6 mm) may be used between the flange plate and beam flange as shown in Figure 7.1. Shims, if required, may be finger shims or may be made with drilled or punched holes.

7.6. DESIGN PROCEDURE

Step 1. Compute the probable maximum moment at the plastic hinge, M_{pr} , in accordance with Section 2.4.3.

Step 2. Compute the maximum bolt diameter to prevent beam flange tensile rupture.

For standard holes with two bolts per row:

$$d_b \leq \frac{b_f}{2} \left(1 - \frac{R_y F_y}{R_t F_u} \right) - \frac{1}{8} \text{ in.} \quad (7.6-2)$$

$$d_b \leq \frac{b_f}{2} \left(1 - \frac{R_y F_y}{R_t F_u} \right) - 3 \text{ mm} \quad (7.6-2M)$$

Select a bolt diameter. Check that the edge distance for the beam flange holes satisfies the AISC *Specification* requirements.

Step 3. Assume a flange plate thickness, t_p . Estimate the width of the flange plate, b_{fp} , considering bolt gage, bolt edge distance requirements, and the beam flange width. Determine the controlling nominal shear strength per bolt considering bolt shear and bolt bearing:

$$r_n = \min \begin{cases} 1.0 F_{nv} A_b \\ 2.4 F_{ub} d_b t_f \\ 2.4 F_{up} d_b t_p \end{cases} \quad (7.6-3)$$

where

A_b = nominal unthreaded body area of bolt, in.² (mm²)

F_{nv} = nominal shear strength of bolt from the AISC *Specification*, ksi (MPa)

F_{ub} = specified minimum tensile strength of beam material, ksi (MPa)

F_{up} = specified minimum tensile strength of plate material, ksi (MPa)

d_b = nominal bolt diameter, in. (mm)

t_f = beam flange thickness, in. (mm)

t_p = flange plate thickness, in. (mm)

Step 4. Select a trial number of bolts.

User Note: The following equation may be used to estimate the trial number of bolts.

$$n \geq \frac{1.25M_{pr}}{\phi_n r_n (d + t_p)} \quad (7.6-4)$$

where

n = number of bolts rounded to next higher even number increment

d = beam depth, in. (mm)

Step 5. Determine the beam plastic hinge location, S_h , as dimensioned from the face of the column.

$$S_h = S_1 + s \left(\frac{n}{2} - 1 \right) \quad (7.6-5)$$

where

S_1 = distance from face of column to nearest row of bolts, in. (mm)

n = number of bolts

s = spacing of bolt rows, in. (mm)

The bolt spacing between rows, s , and the edge distance shall be sufficiently large to ensure that l_c , as defined in the AISC *Specification*, is greater than or equal to $2d_b$.

Step 6. Compute the shear force at the beam plastic hinge location at each end of the beam.

The shear force at the hinge location, V_h , shall be determined from a free-body diagram of the portion of the beam between the plastic hinge locations. This calculation shall assume the moment at the plastic hinge location is M_{pr} and shall include gravity loads acting on the beam based on the load combination of $1.2D + f_1L + 0.2S$, where D is the dead load; f_1 is the load factor determined by the applicable building code for live loads, but not less than 0.5; L is the live load; and S is the snow load.

User Note: The load combination of $1.2D + f_1L + 0.2S$ is in conformance with ASCE/SEI 7-16. When using the International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.2 for S (snow) when the roof configuration is such that it does not shed snow off of the structure.

Step 7. Calculate the moment expected at the face of the column flange.

$$M_f = M_{pr} + V_h S_h \quad (7.6-6)$$

where V_h is the larger of the two values of shear force at the beam hinge location at each end of the beam, kips (N).

Equation 7.6-6 neglects the gravity load on the portion of the beam between the plastic hinge and the face of the column. The gravity load on this small portion of the beam is permitted to be included.

Step 8. Compute F_{pr} , the force in the flange plate due to M_f .

$$F_{pr} = \frac{M_f}{(d + t_p)} \quad (7.6-7)$$

where

d = depth of beam, in. (mm)

t_p = thickness of flange plate, in. (mm)

Step 9. Confirm that the number of bolts selected in Step 4 is adequate.

$$n \geq \frac{F_{pr}}{\phi_n r_n} \quad (7.6-8)$$

Step 10. Check that the thickness of the flange plate assumed in Step 3 is adequate:

$$t_p \geq \frac{F_{pr}}{\phi_d F_y b_{fp}} \quad (7.6-9)$$

where

F_y = specified minimum yield stress of flange plate, ksi (MPa)

b_{fp} = width of flange plate, in. (mm)

Step 11. Check the flange plate for the limit state of tensile rupture.

$$F_{pr} \leq \phi_n R_n \quad (7.6-10)$$

where R_n is defined in the tensile rupture provisions of Chapter J of the AISC *Specification*.

Step 12. Check the beam flange for the limit state of block shear rupture.

$$F_{pr} \leq \phi_n R_n \quad (7.6-11)$$

where R_n is as defined in the block shear rupture provisions of Chapter J of the AISC *Specification*.

Step 13. Check the flange plate for the limit states of compression buckling.

$$F_{pr} \leq \phi_n R_n \quad (7.6-12)$$

where R_n is defined in the compression buckling provisions of Section J4.4 of the AISC *Specification*.

User Note: When checking compression buckling of the flange plate, the effective length, KL , may be taken as $0.65S_1$.

Some iteration from Steps 3 through 13 may be required to determine an acceptable flange plate size.

Step 14. Determine the required shear strength, V_u , of the beam and the beam-web-column connection from:

$$V_u = \frac{2M_{pr}}{L_h} + V_{gravity} \quad (7.6-13)$$

where

L_h = distance between plastic hinge locations, in. (mm)

$V_{gravity}$ = beam shear force resulting from $1.2D + f_1L + 0.2S$ (where f_1 is a load factor determined by the applicable building code for live loads, but not less than 0.5), kips (N)

User Note: The load combination of $1.2D + f_1L + 0.2S$ is in conformance with ASCE/SEI 7-16. When using the International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.2 for S (snow) when the roof configuration is such that it does not shed snow off of the structure.

Check design shear strength of beam according to the AISC *Specification*.

Step 15. Design a single-plate shear connection for the required shear strength, V_u , calculated in Step 14 and located at the face of the column, meeting the requirements of the AISC *Specification*.

Step 16. Check the continuity plate requirements according to Chapter 2.

Step 17. Check the column panel zone according to Section 7.4.

The required shear strength of the panel zone shall be determined from the summation of the moments at the column faces as determined by projecting moments equal to $R_yF_yZ_e$ at the plastic hinge points to the column faces. For d , add twice the thickness of the flange plate to the beam depth.

CHAPTER 8

WELDED UNREINFORCED FLANGE-WELDED WEB (WUF-W) MOMENT CONNECTION

8.1. GENERAL

In the welded unreinforced flange-welded web (WUF-W) moment connection, inelastic rotation is developed primarily by yielding of the beam in the region adjacent to the face of the column. Connection rupture is controlled through special detailing requirements associated with the welds joining the beam flanges to the column flange, the welds joining the beam web to the column flange, and the shape and finish of the weld access holes. An overall view of the connection is shown in Figure 8.1.

8.2. SYSTEMS

WUF-W moment connections are prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems within the limits of these provisions.

8.3. PREQUALIFICATION LIMITS

1. Beam Limitations

Beams shall satisfy the following limitations:

- (1) Beams shall be rolled wide-flange or built-up I-shaped members conforming to the requirements of Section 2.3.

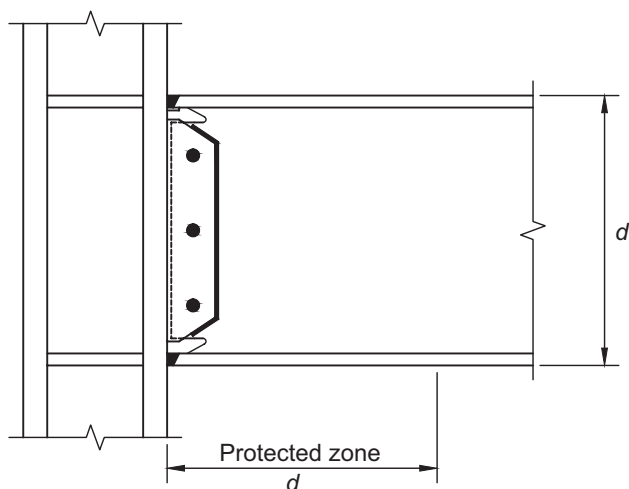


Fig. 8.1. WUF-W moment connection.

- (2) Beam depth is limited to a maximum of W36 (W920) for rolled shapes. Depth of built-up sections shall not exceed the depth permitted for rolled wide-flange shapes.
- (3) Beam weight is limited to a maximum of 150 lb/ft (224 kg/m).
- (4) Beam flange thickness is limited to a maximum of 1 in. (25 mm).
- (5) The clear span-to-depth ratio of the beam is limited as follows:
 - (a) For SMF systems, 7 or greater.
 - (b) For IMF systems, 5 or greater.
- (6) Width-to-thickness ratios for the flanges and web of the beam shall conform to the requirements of the AISC *Seismic Provisions*.
- (7) Lateral bracing of beams shall be provided as follows:

Lateral bracing of beams shall conform to the requirements of the AISC *Seismic Provisions*. To satisfy the requirements of the AISC *Seismic Provisions* for lateral bracing at plastic hinges, supplemental lateral bracing shall be provided at both the top and bottom beam flanges, and shall be located at a distance of d to $1.5d$ from the face of the column. No attachment of lateral bracing shall be made to the beam in the region extending from the face of the column to a distance d from the face of the column.

Exception: For both SMF and IMF systems, where the beam supports a concrete structural slab that is connected along the beam span between protected zones with welded shear connectors spaced at a maximum of 12 in. (300 mm) on center, supplemental top and bottom flange bracing at plastic hinges is not required.

- (8) The protected zone consists of the portion of beam between the face of the column and a distance one beam depth, d , from the face of the column.

2. Column Limitations

Columns shall satisfy the following limitations:

- (1) Columns shall be any of the rolled shapes or built-up sections permitted in Section 2.3.
- (2) The beam shall be connected to the flange of the column.
- (3) Rolled shape column depth shall be limited to a maximum of W36 (W920). The depth of built-up wide-flange columns shall not exceed that for rolled shapes. Flanged cruciform columns shall not have a width or depth greater than the depth allowed for rolled shapes. Built-up box columns shall not have a width or depth exceeding 24 in. (600 mm). Boxed wide-flange columns shall not have a width or depth exceeding 24 in. (600 mm) if participating in orthogonal moment frames.
- (4) There is no limit on the weight per foot of columns.
- (5) There are no additional requirements for flange thickness.

- (6) Width-to-thickness ratios for the flanges and web of columns shall conform to the requirements of the AISC *Seismic Provisions*.
- (7) Lateral bracing of columns shall conform to the requirements of the AISC *Seismic Provisions*.

8.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

Beam-to-column connections shall satisfy the following limitations:

- (1) Panel zones shall conform to the requirements of the AISC *Seismic Provisions*.
- (2) Column-beam moment ratios shall be limited as follows:
 - (a) For SMF systems, the column-beam moment ratio shall conform to the requirements of the AISC *Seismic Provisions*. The value of $\sum M_{pb}^*$ shall be taken equal to $\sum(M_{pr} + M_{uv})$, where M_{pr} is computed according to Step 1 in Section 8.7, and M_{uv} is the additional moment due to shear amplification from the plastic hinge to the centerline of the column. M_{uv} is permitted to be computed as $V_h(d_c/2)$, where V_h is the shear at the plastic hinge computed per Step 3 of Section 8.7, and d_c is the depth of the column.
 - (b) For IMF systems, the column-beam moment ratio shall conform to the requirements of the AISC *Seismic Provisions*.

8.5. BEAM FLANGE-TO-COLUMN FLANGE WELDS

Beam flange-to-column flange connections shall satisfy the following limitations:

- (1) Beam flanges shall be connected to column flanges using complete-joint-penetration (CJP) groove welds. Beam flange welds shall conform to the requirements for demand critical welds in the AISC *Seismic Provisions*.
- (2) Weld access hole geometry shall conform to the requirements of AWS D1.8/D1.8M Section 6.11.1.2. Weld access hole quality requirements shall conform to the requirements of AWS D1.8.

8.6. BEAM WEB-TO-COLUMN CONNECTION LIMITATIONS

The overall details of the beam web-to-column flange connection are shown in Figure 8.2. Single-plate shear connections shall conform to the requirements shown in Figure 8.2. Beam web-to-column flange connections shall satisfy the following limitations:

- (1) A single-plate shear connection shall be provided with a thickness equal at least to that of the beam web. The height of the single plate shall allow a 1/4-in. (6-mm) minimum and 1/2-in. (12-mm) maximum overlap with the weld access hole at the top and bottom as shown in Figure 8.3. The width shall extend 2 in. (50 mm) minimum beyond the end of the weld access hole.
- (2) The single-plate shear connection shall be welded to the column flange. The design shear strength of the welds shall be at least $h_p t_p (0.6R_y F_{yp})$, where h_p is the length of the plate, as shown in Figure 8.2, and t_p is the thickness of the plate.

- (3) The single-plate shear connection shall be connected to the beam web with fillet welds, as shown in Figures 8.2 and 8.3. The size of the fillet weld shall equal the thickness of the single plate minus $\frac{1}{16}$ in. (2 mm). The fillet welds shall extend along the sloped top and bottom portions of the single plate, and along the vertical single plate length, as shown in Figures 8.2 and 8.3. The fillet welds on the sloped top and bottom portions of the single plate shall be terminated at least $\frac{1}{2}$ in. (12 mm) but not more than 1 in. (25 mm) from the edge of the weld access hole, as shown in Figure 8.3.
- (4) Erection bolts in standard holes or horizontal short slots are permitted as needed.
- (5) A CJP groove weld shall be provided between the beam web and the column flange. This weld shall be provided over the full length of the web between weld access holes, and shall conform to the requirements for demand critical welds in the AISC *Seismic Provisions* and AWS D1.8/D1.8M. Weld tabs are not required. Weld tabs, if used, must be removed after welding in accordance with the requirements of Section 3.4. When weld tabs are not used, the use of cascaded weld ends within the weld groove shall be permitted at a maximum angle of 45° . Nondestructive testing (NDT) of cascaded weld ends need not be performed.

8.7. DESIGN PROCEDURE

Step 1. Compute the probable maximum moment at the plastic hinge, M_{pr} , in accordance with Section 2.4.3. The value of Z_e shall be taken as equal to Z_x of the beam section and the value of C_{pr} shall be taken as equal to 1.4.

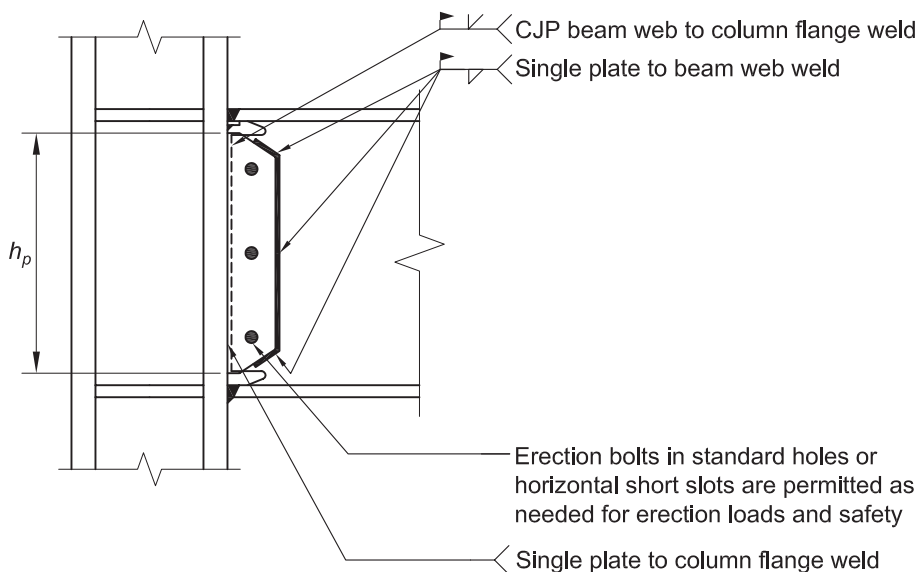


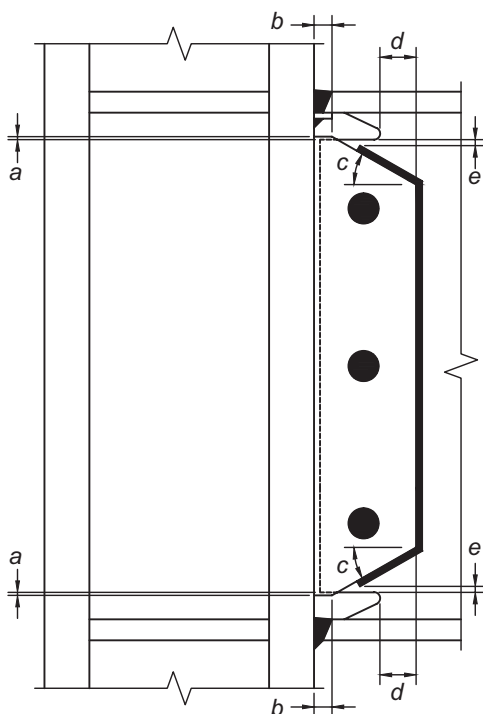
Fig. 8.2. General details of beam web-to-column flange connection.

User Note: The C_{pr} value of 1.4 for WUF-W moment connections is based on experimental data that shows a high degree of strain hardening.

Step 2. The plastic hinge location shall be taken to be at the face of the column; that is, $S_h = 0$.

Step 3. Compute the shear force, V_h , at the plastic hinge location at each end of the beam.

The shear force at the plastic hinge locations shall be determined from a free-body diagram of the portion of the beam between the plastic hinges. This calculation shall



Notes

- $a = \frac{1}{4}$ in. (6 mm) minimum, $\frac{1}{2}$ in. (12 mm) maximum
- $b = 1$ in. (25 mm) minimum
- $c = 30^\circ (\pm 10^\circ)$
- $d = 2$ in. (50 mm) minimum
- $e = \frac{1}{2}$ in. (12 mm) minimum distance, 1 in. (25 mm) maximum distance from end of fillet weld to edge of access hole

Fig. 8.3. Details at top and bottom of single-plate shear connection.

assume the moment at each plastic hinge is M_{pr} and shall include gravity loads acting on the beam between the hinges based on the load combination $1.2D + f_1L + 0.2S$.

User Note: The load combination of $1.2D + f_1L + 0.2S$ is in conformance with ASCE/SEI 7-16. When using the International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.2 for S (snow) when the roof configuration is such that it does not shed snow off of the structure.

Step 4. Check column-beam relationship limitations per Section 8.4. For SMF, the required shear strength of the panel zone, per the AISC *Seismic Provisions*, shall be determined from the summation of the probable maximum moments at the face of the column. The probable maximum moment at the face of the column shall be taken as M_{pr} , computed per Step 1. Provide doubler plates as necessary.

Step 5. Check beam design shear strength:

The required shear strength, V_u , of the beam shall be taken equal to the larger of the two values of V_h computed at each end of the beam in Step 3.

Step 6. Check column continuity plate requirements per Section 2.4.4. Provide continuity plates as necessary.

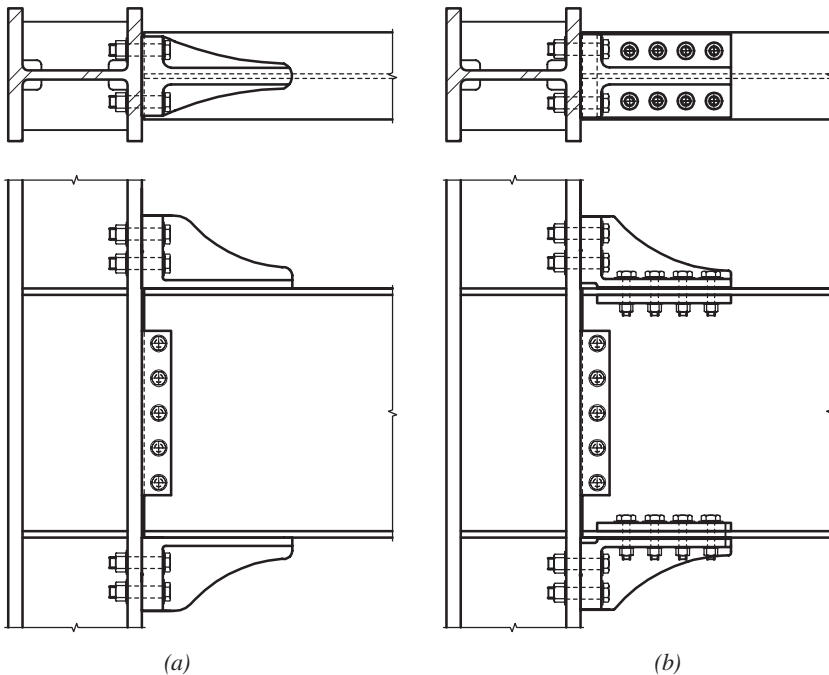
CHAPTER 9

KAISER BOLTED BRACKET (KBB) MOMENT CONNECTION

The user's attention is called to the fact that compliance with this chapter of the standard requires use of an invention covered by patent rights. By publication of this standard, no position is taken with respect to the validity of any claim(s) or of any patent rights in connection therewith. The patent holder has filed a statement of willingness to grant a license under these rights on reasonable and nondiscriminatory terms and conditions to applicants desiring to obtain such a license. The statement may be obtained from the standards developer.

9.1. GENERAL

In a Kaiser bolted bracket (KBB) moment connection, a cast high-strength steel bracket is fastened to each beam flange and bolted to the column flange as shown in Figure 9.1. The bracket attachment to the beam flange is permitted to be either



*Fig. 9.1. Kaiser bolted bracket connection.
(a) W-series connection; (b) B-series connection.*

welded (Figure 9.1a) or bolted (Figure 9.1b). When welded to the beam flange, the five W-series bracket configurations available are shown in Figure 9.2. When bolted to the beam flange, the two B-series bracket configurations available are shown in Figure 9.3. The bracket configuration is proportioned to develop the probable maximum moment strength of the connected beam. Yielding and plastic hinge formation are intended to occur primarily in the beam at the end of the bracket away from the column face.

9.2. SYSTEMS

KBB connections are prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems within the limits of these provisions.

Exception: KBB SMF systems with concrete structural slabs are prequalified only if the concrete structural slab is kept at least 1 in. (25 mm) from both sides of both column flanges and the vertical flange of the bracket. It is permitted to place compressible material in the gap in this location.

9.3. PREQUALIFICATION LIMITS

1. Beam Limitations

Beams shall satisfy the following limitations:

- (1) Beams shall be rolled wide-flange or built-up I-shaped members conforming to the requirements of Section 2.3.

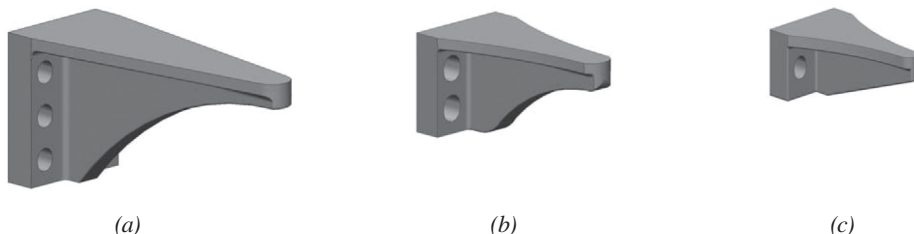


Fig. 9.2. Kaiser bolted bracket W-series configurations: (a) six column bolts, W1.0; (b) four column bolts, W2.0 and W2.1; and (c) two column bolts, W3.0 and W3.1.

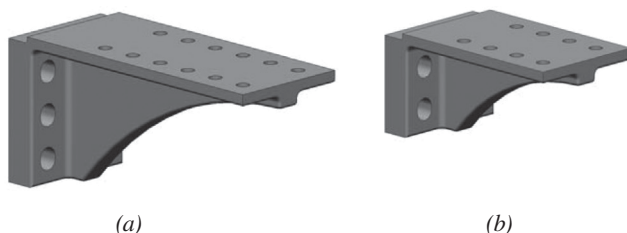


Fig. 9.3. Kaiser bolted bracket B-series configurations: (a) six column bolts, B1.0, and (b) four column bolts, B2.1.

- (2) Beam depth is limited to a maximum of W33 (W840) for rolled shapes. Depth of built-up sections shall not exceed the depth permitted for rolled wide-flange shapes.
- (3) Beam weight is limited to a maximum of 130 lb/ft (195 kg/m).
- (4) Beam flange thickness is limited to a maximum of 1 in. (25 mm).
- (5) Beam flange width shall be at least 6 in. (150 mm) for W-series brackets and at least 10 in. (250 mm) for B-series brackets.
- (6) The clear span-to-depth ratio of the beam shall be limited to 9 or greater for both SMF and IMF systems.
- (7) Width-to-thickness ratios for the flanges and web of the beam shall conform to the requirements of the AISC *Seismic Provisions*.
- (8) Lateral bracing of beams shall be provided as follows:
 - (a) For SMF systems, in conformance with the AISC *Seismic Provisions*. Supplemental lateral bracing shall be provided at the expected plastic hinge in conformance with the AISC *Seismic Provisions*.

When supplemental lateral bracing is provided, attachment of supplemental lateral bracing to the beam shall be located at a distance d to $1.5d$ from the end of the bracket farthest from the face of the column, where d is the depth of the beam. No attachment of lateral bracing shall be made to the beam in the region extending from the face of the column to a distance d beyond the end of the bracket.

- (b) For IMF systems, in conformance with the AISC *Seismic Provisions*.

Exception: For both systems, where the beam supports a concrete structural slab that is connected between the protected zones with welded shear connectors spaced at maximum of 12 in. (300 mm) on center, supplemental top and bottom flange bracing at the expected hinge is not required.

- (9) The protected zone consists of the portion of beam between the face of the column and one beam depth, d , beyond the end of the bracket farthest from the face of the column.

2. Column Limitations

The columns shall satisfy the following limitations:

- (1) Columns shall be any of the rolled shapes or built-up sections permitted in Section 2.3.
- (2) The beam shall be connected to the flange of the column.
- (3) The column flange width shall be at least 12 in. (300 mm).
- (4) Rolled shape column depth shall be limited to W36 (W920) maximum when a concrete structural slab is provided. In the absence of a concrete structural slab, rolled shape column depth is limited to W14 (W360) maximum. The

depth of built-up wide-flange columns shall not exceed that for rolled shapes. Flanged cruciform columns shall not have a width or depth greater than the depth allowed for rolled shapes. Built-up box columns shall not have a width or depth exceeding 16 in. (400 mm). Boxed wide-flange columns shall not have a width or depth exceeding 16 in. (400 mm) if participating in orthogonal moment frames.

- (5) There is no limit on the weight per foot of columns.
- (6) There are no additional requirements for flange thickness.
- (7) Width-to-thickness ratios for the flanges and web of columns shall conform to the requirements of the AISC *Seismic Provisions*.
- (8) Lateral bracing of the columns shall conform to the requirements of the AISC *Seismic Provisions*.

3. Bracket Limitations

The high strength cast-steel brackets shall satisfy the following limitations:

- (1) Bracket castings shall conform to the requirements of Appendix A.
- (2) Bracket configuration and proportions shall conform to Section 9.8.
- (3) Holes in the bracket for the column bolts shall be vertical short-slotted holes. Holes for the beam bolts shall be standard holes.
- (4) Material thickness, edge distance and end distance shall have a tolerance of $\pm 1/16$ in. (2 mm). Hole location shall have a tolerance of $\pm 1/16$ in. (2 mm). The overall dimensions of the bracket shall have a tolerance of $\pm 1/8$ in. (3 mm).

9.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

Beam-to-column connections shall satisfy the following limitations:

- (1) Panel zones shall conform to the requirements in the AISC *Seismic Provisions*.
- (2) Column-beam moment ratios shall conform to the requirements of the AISC *Seismic Provisions*.

9.5. BRACKET-TO-COLUMN FLANGE CONNECTION LIMITATIONS

Bracket-to-column flange connections shall satisfy the following limitations:

- (1) Column flange fasteners shall be pretensioned ASTM F3125 Grades A490, A490M, A354 Grade BD bolts, or A354 Grade BD threaded rods and shall conform to the requirements of Chapter 4.
- (2) Column flange bolt holes shall be $1/8$ in. (3 mm) larger than the nominal bolt diameter. Bolt holes shall be drilled or subpunched and reamed. Punched holes are not permitted.
- (3) The use of finger shims on either or both sides at the top and/or bottom of the bracket connection is permitted, subject to the limitations of the RCSC *Specification*.

- (4) When bolted to a box column, a steel washer plate shall be inserted between the box column and the bracket on both faces of the column. The washer plate shall be ASTM A572/A572M Grade 50 (345) or better and shall be designed to transfer the bolt forces to the outside edges of the column. Where required, the vertical plate depth may extend beyond the contact surface area by up to 4 in. (102 mm). The plate thickness shall not exceed 3 in. (75 mm). The fasteners shall pass through the interior of the box column and be anchored on the opposite face. The opposite face shall also have a steel washer plate.
- (5) When connecting to the orthogonal face of a box column concurrent with a connection on the primary column face, a 1¾-in. (44-mm) steel spacer plate shall be inserted between the beam flanges and the brackets of the orthogonal connection. The spacer plate shall be made of any of the structural steel materials included in the AISC *Specification* and shall be the approximate width and length matching that of the bracket contact surface area.

9.6. BRACKET-TO-BEAM FLANGE CONNECTION LIMITATIONS

Bracket-to-beam-flange connections shall satisfy the following limitations:

- (1) When welded to the beam flange, the bracket shall be connected using fillet welds. Bracket welds shall conform to the requirements for demand critical welds in the AISC *Seismic Provisions* and AWS D1.8/D1.8M, and to the requirements of AWS D1.1/D1.1M. The weld procedure specification (WPS) for the fillet weld joining the bracket to the beam flange shall be qualified with the casting material. Welds shall not be started or stopped within 2 in. (50 mm) of the bracket tip and shall be continuous around the tip.
- (2) When bolted to the beam flange, fasteners shall be pretensioned ASTM F3125 Grade A490 or Grade A490M bolts with threads excluded from the shear plane and shall conform to the requirements of Chapter 4.
- (3) Beam flange bolt holes shall be 1⅝ in. (29 mm) and shall be drilled using the bracket as a template. Punched holes are not permitted.
- (4) When bolted to the beam flange, a ⅛-in. (3-mm) -thick brass washer plate with an approximate width and length matching that of the bracket contact surface area shall be placed between the beam flange and the bracket. The brass shall be a half-hard tempered ASTM B19 or B36/B36M sheet.
- (5) When bolted to the beam flange, a 1-in. (25-mm) -thick by 4-in. (100-mm) -wide ASTM A572/A572M Grade 50 (345) plate washer shall be used on the opposite side of the connected beam flange.

9.7. BEAM WEB-TO-COLUMN CONNECTION LIMITATIONS

Beam web-to-column flange connections shall satisfy the following limitations:

- (1) The required shear strength of the beam web connection shall be determined according to Section 9.9.

- (2) The single-plate shear connection shall be connected to the column flange using a two-sided fillet weld, two-sided PJP groove weld, or CJP groove weld.

9.8. CONNECTION DETAILING

If welded to the beam flange, Figure 9.4 shows the connection detailing for the W-series bracket configurations. If bolted to the beam flange, Figure 9.5 shows the connection detailing for the B-series bracket configurations. Table 9.1 summarizes the KBB proportions and column bolt parameters. Table 9.2 summarizes the design proportions for the W-series bracket configuration. Table 9.3 summarizes the design proportions for the B-series bracket configurations.

9.9. DESIGN PROCEDURE

Step 1. Select beam and column elements which satisfy the limits of Section 9.3.

Step 2. Compute the probable maximum moment, M_{pr} , at the location of the plastic hinge according to Section 2.4.3.

Step 3. Select a trial bracket from Table 9.1.

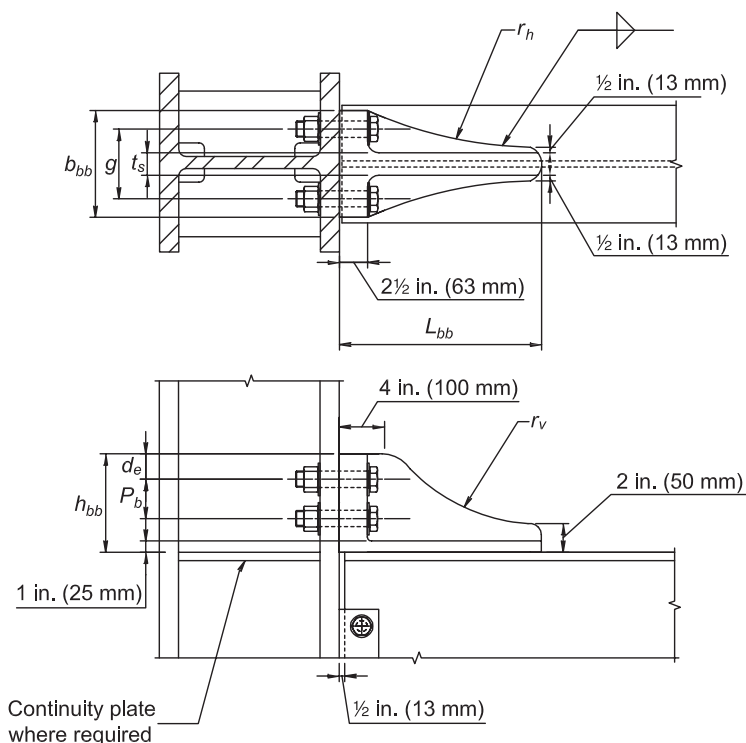


Fig. 9.4. W-series connection detailing.

Step 4. Compute the shear force at the beam hinge location at each end of the beam. The shear force at the hinge location, V_h , shall be determined from a free-body diagram of the portion of the beam between the hinge locations. This calculation shall assume the moment at the hinge location is M_{pr} and shall include gravity loads acting on the beam based on the load combination $1.2D + f_1L + 0.2S$, kips (N), where f_1 is the load factor determined by the applicable building code for live loads, but not less than 0.5.

User Note: The load combination of $1.2D + f_1L + 0.2S$ is in conformance with ASCE/SEI 7-16. When using the International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.2 for S (snow) when the roof configuration is such that it does not shed snow off of the structure.

Step 5. Compute the probable maximum moment at the face of the column:

$$M_f = M_{pr} + V_h S_h \quad (9.9-1)$$

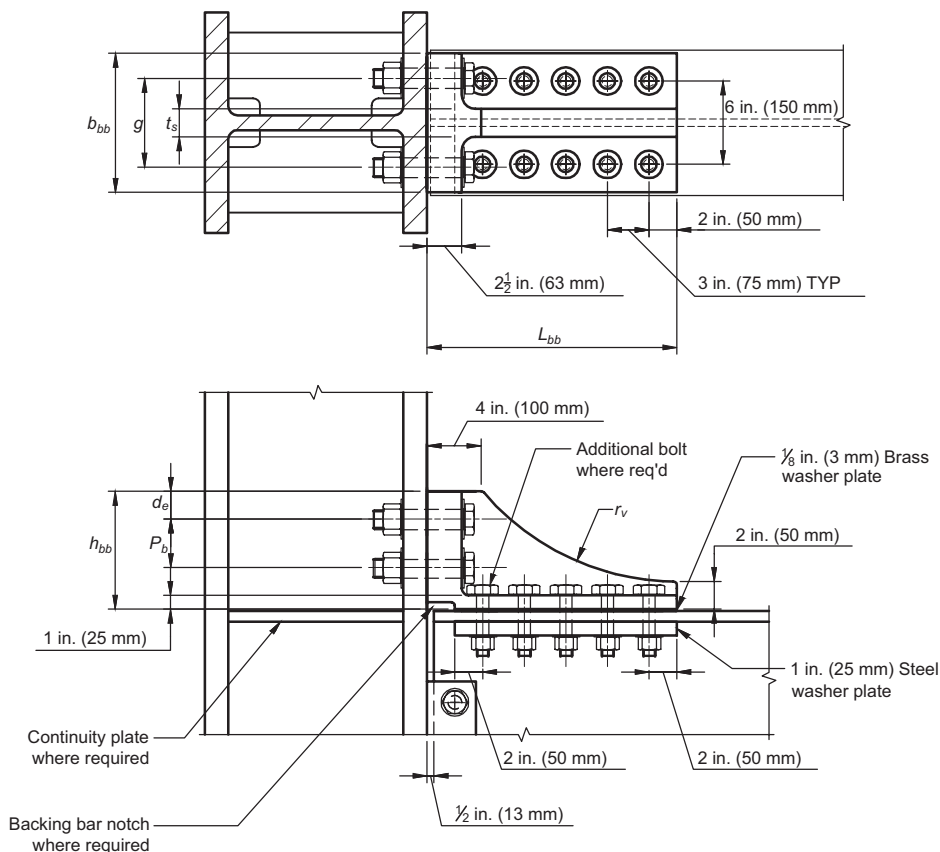


Fig. 9.5. B-series connection detailing.

TABLE 9.1
Kaiser Bolted Bracket Proportions

Bracket Designation	Bracket Length, L_{bb} in. (mm)	Bracket Height, h_{bb} in. (mm)	Bracket Width, b_{bb} in. (mm)	Number of Column Bolts, n_{cb}	Column Bolt Gage, g in. (mm)	Column Bolt Diameter in. (mm)
W3.0	16 (400)	5½ (140)	9 (229)	2	5½ (140)	1¾ (35)
W3.1	16 (400)	5½ (140)	9 (229)	2	5½ (140)	1½ (38)
W2.0	16 (400)	8¾ (222)	9½ (241)	4	6 (152)	1¾ (35)
W2.1	18 (450)	8¾ (222)	9½ (241)	4	6½ (165)	1½ (38)
W1.0	25½ (648)	12 (305)	9½ (241)	6	6½ (165)	1½ (38)
B2.1	18 (450)	8¾ (222)	10 (250)	4	6½ (165)	1½ (38)
B1.0	25½ (648)	12 (305)	10 (250)	6	6½ (165)	1½ (38)

TABLE 9.2
W-Series Bracket Design Proportions

Bracket Designation	Column Bolt Edge Distance, d_e in. (mm)	Column Bolt Pitch, p_b in. (mm)	Bracket Stiffener Thickness, t_s in. (mm)	Bracket Stiffener Radius, r_v in. (mm)	Bracket Horizontal Radius, r_h in. (mm)	Minimum Fillet Weld Size, w in. (mm)
W3.0	2½ (64)	n.a.	1 (25)	n.a.	28 (711)	½ (13)
W3.1	2½ (64)	n.a.	1 (25)	n.a.	28 (711)	⅝ (16)
W2.0	2¼ (57)	3½ (88)	2 (50)	12 (300)	28 (711)	¾ (19)
W2.1	2¼ (57)	3½ (88)	2 (50)	16 (400)	38 (965)	⅞ (22)
W1.0	2 (50)	3½ (88)	2 (50)	28 (711)	n.a.	⅞ (22)

TABLE 9.3.
B-Series Bracket Design Proportions

Bracket Designation	Column Bolt Edge Distance, d_e in. (mm)	Column Bolt Pitch, p_b in. (mm)	Bracket Stiffener Thickness, t_s in. (mm)	Bracket Stiffener Radius, r_v in. (mm)	Number of Beam Bolts, n_{bb}	Beam Bolt Diameter in. (mm)
B2.1	2 (50)	3½ (88)	2 (50)	16 (400)	8 or 10	1½ (28)
B1.0	2 (50)	3½ (88)	2 (50)	28 (711)	12	1½ (28)

where

M_f = probable maximum moment at face of the column, kip-in. (N-mm)

S_h = distance from face of the column to plastic hinge, in. (mm)

= L_{bb} per Table 9.1, in. (mm)

V_h = larger of the two values of shear force at beam hinge location at each end of beam, kips (N)

Equation 9.9-1 neglects the gravity load on the portion of the beam between the plastic hinge and the face of the column. If desired, the gravity load on this small portion of the beam is permitted to be included.

Step 6. The following relationship shall be satisfied for the bracket column bolt tensile strength:

$$r_{ut} \leq \phi_n F_{nt} A_b \quad (9.9-2)$$

where

A_b = bolt nominal cross-sectional area, in.² (mm²)

F_{nt} = nominal tensile strength of bolt from the AISC *Specification*, ksi (MPa)

d_{eff} = effective beam depth, calculated as the centroidal distance between bolt groups in the upper and lower brackets, in. (mm)

n_{cb} = number of column bolts per Table 9.1

$$r_{ut} = \frac{M_f}{d_{eff} n_{cb}} \quad (9.9-3)$$

Step 7. Determine the minimum column flange width to prevent flange tensile rupture:

$$b_{cf} \geq \frac{2[d_b + \frac{1}{8} \text{ in.}]}{\left(1 - \frac{R_y F_{yf}}{R_t F_{uf}}\right)} \quad (9.9-4)$$

$$b_{cf} \geq \frac{2[d_b + 3 \text{ mm}]}{\left(1 - \frac{R_y F_{yf}}{R_t F_{uf}}\right)} \quad (9.9-4M)$$

where

F_{yf} = specified minimum yield stress of flange material, ksi (MPa)

F_{uf} = specified minimum tensile strength of flange material, ksi (MPa)

R_y = ratio of expected yield stress to specified minimum yield stress for flange material

R_t = ratio of expected tensile strength to specified minimum tensile strength for flange material

b_{cf} = width of column flange, in. (mm)

d_b = diameter of column flange bolts, in. (mm)

Step 8. Check the minimum column flange thickness to eliminate prying action:

$$t_{cf} \geq \sqrt{\frac{4.44r_{ut}b'}{\phi_d p F_y}} \quad (9.9-5)$$

where

$$b' = 0.5(g - k_1 - 0.5t_{cw} - d_b) \quad (9.9-6)$$

g = column bolt gauge, in. (mm)

k_1 = column web centerline distance to flange toe of fillet, in. (mm)

p = perpendicular tributary length per bolt, in. (mm)

= 3.5 in. (88 mm) for W1.0 and B1.0

= 5.0 in. (125 mm) for all other brackets

t_{cw} = column web thickness, in. (mm)

If the selected column flange thickness is less than that required to eliminate prying action, select a column with a satisfactory flange thickness or include the bolt prying force in Equation 9.9-2 per Part 9 of the AISC *Steel Construction Manual*.

Step 9. The column flange thickness shall satisfy the following requirement to eliminate continuity plates:

$$t_{cf} \geq \sqrt{\frac{M_f}{\phi_d F_{yf} d_{eff} Y_m}} \quad (9.9-7)$$

where

Y_m = simplified column flange yield line mechanism parameter

= 5.9 for W3.0 and W3.1

= 6.5 for W2.0, W2.1 and B2.1

= 7.5 for W1.0 and B1.0

t_{cf} = minimum column flange thickness required to eliminate continuity plates, in. (mm)

Step 10. Continuity Plate Requirements

For W14 and shallower columns, continuity plates are not required if Equation 9.9-7 is satisfied. For column sections deeper than W14, continuity plates shall be provided.

Step 11. If the bracket is welded to the beam flange proceed to Step 14; otherwise, determine the minimum beam flange width to prevent beam flange tensile rupture:

$$b_{bf} \geq \frac{2[d_b + \frac{1}{32} \text{ in.}]}{\left(1 - \frac{R_y F_{yf}}{R_t F_{uf}}\right)} \quad (9.9-8)$$

$$b_{bf} \geq \frac{2[d_b + 1 \text{ mm}]}{\left(1 - \frac{R_y F_{yf}}{R_t F_{uf}}\right)} \quad (9.9-8M)$$

where

b_{bf} = width of beam flange, in. (mm)

d_b = diameter of beam flange bolts, in. (mm)

Step 12. The following relationship shall be satisfied for the beam bolt shear strength:

$$\frac{M_f}{\phi_n F_{nv} A_b d_{eff} n_{bb}} < 1.0 \quad (9.9-9)$$

where

F_{nv} = nominal shear strength of bolt from the AISC *Specification*, ksi (MPa)

n_{bb} = number of beam bolts per Table 9.3

Step 13. Check the beam flange for block shear per the following:

$$\frac{M_f}{d_{eff}} \leq \phi_n R_n \quad (9.9-10)$$

where R_n is as defined in the block shear provisions of Chapter J of the AISC *Specification*.

Step 14. If the bracket is bolted to the beam flange, proceed to Step 15. Otherwise, the following relationship shall be satisfied for the fillet weld attachment of the bracket to the beam flange:

$$\frac{M_f}{\phi_n F_w d_{eff} l_w (0.707w)} < 1.0 \quad (9.9-11)$$

where

F_w = nominal weld design strength per the AISC *Specification*
= $0.60F_{EXX}$

F_{EXX} = filler metal classification strength, ksi (MPa)

l_w = length of available fillet weld, in. (mm)

$$= 2(L_{bb} - 2.5 \text{ in.} - l) \quad (9.9-12)$$

$$= 2(L_{bb} - 64 \text{ mm} - l) \quad (9.9-12M)$$

where

L_{bb} = bracket length per Table 9.3, in. (mm)

l = bracket overlap distance, in. (mm)

= 0 in. (0 mm) if $b_{bf} \geq b_{bb}$

= 5 in. (125 mm) if $b_{bf} < b_{bb}$

w = minimum fillet weld size per Table 9.2, in. (mm)

Step 15. Determine the required shear strength, V_u , of the beam and beam web-to-column connection from:

$$V_u = \frac{2M_{pr}}{L_h} + V_{gravity} \quad (9.9-13)$$

where

L_h = distance between plastic hinge locations, in. (mm)

$V_{gravity}$ = beam shear force resulting from $1.2D + f_1L + 0.2S$ (where f_1 is a load factor determined by the applicable building code for live loads, but not less than 0.5), kips (N)

User Note: The load combination of $1.2D + f_1L + 0.2S$ is in conformance with ASCE/SEI 7-16. When using the International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.2 for S (snow) when the roof configuration is such that it does not shed snow off of the structure.

Check design shear strength of beam according to Chapter G of the AISC *Specification*.

Step 16. Design the beam web-to-column connection according to Section 9.7.

Step 17. Check column panel zone according to Section 9.4. Substitute the effective depth, d_{eff} , of the beam and brackets for the beam depth, d .

Step 18. (Supplemental) If the column is a box configuration, determine the size of the steel washer plate between the column flange and the bracket such that:

$$Z_x \geq \frac{M_f(b_{cf} - t_{cw} - g)}{4\phi_d F_y d_{eff}} \quad (9.9-14)$$

where

F_y = specified minimum yield stress of washer material, ksi (MPa)

Z_x = plastic section modulus of washer plate, in.³ (mm³)

g = column bolt gage, in. (mm)

CHAPTER 10

CONXTECH CONXL MOMENT CONNECTION

The user's attention is called to the fact that compliance with this chapter of the standard requires use of an invention covered by patent rights. By publication of this standard, no position is taken with respect to the validity of any claim(s) or of any patent rights in connection therewith. The patent holder has filed a statement of willingness to grant a license under these rights on reasonable and nondiscriminatory terms and conditions to applicants desiring to obtain such a license, and the statement may be obtained from the standards developer.*

10.1. GENERAL

The ConXtech® ConXL™ moment connection permits full-strength, fully restrained connection of wide-flange beams to concrete-filled 16-in. (400-mm) square HSS or built-up box columns using a high-strength, field-bolted collar assembly. Beams are shop-welded to forged flange and web fittings (collar flange assembly) and are field-bolted together through either forged or cast steel column fittings (collar corner assembly) that are shop welded to the columns. Beams may include reduced beam section (RBS) cutouts if necessary to meet strong-column/weak-beam criteria. ConXL connections may be used to provide moment connections to columns in orthogonal frames. All moment beams connecting to a ConXL node (intersection of moment beams and column) must be of the same nominal depth.

Figure 10.1 shows the connection geometry and major connection components. Each ConXL collar assembly is made up of either forged or cast collar corners and forged collar flanges. At each ConXL node, there are four collar corner assemblies (Figure 10.2), one at each corner of the square built-up or HSS column. Each ConXL node also contains four collar flange assemblies (Figure 10.3), one for each face of the square column. Each collar flange assembly can contain the end of a moment beam that is shop-welded to the collar flange assembly. The combination of collar corner assemblies, collar flange assemblies, and square concrete-filled column create the ConXL node.

Figure 10.2 shows the collar corner assemblies. The collar corner assembly is made up of a collar corner top (CCT) piece; a collar corner bottom (CCB) piece; and for beam depths greater than 18 in. (460 mm), a collar corner middle (CCM) piece. The CCT, CCB and CCM are partial-joint-penetration (PJP) groove welded together to

* The connectors and structures illustrated are covered by one or more of the following U.S. and foreign patents: U.S. Pat. Nos.: 7,941,985; 6,837,016; 7,051,917; 7,021,020; Australia Pat. Nos.: 2001288615; 2004319371; Canada Pat. Nos.: 2,458,706; 2,564,195; China Pat. Nos.: ZL 01 8 23730.4; ZL 2004 8 0042862.5; Japan Pat. Nos.: 4165648; 4427080; Mexico Pat. Nos.: 262,499; 275284; Hong Kong Pat. No.: 1102268. Other U.S. and foreign patent protection pending.

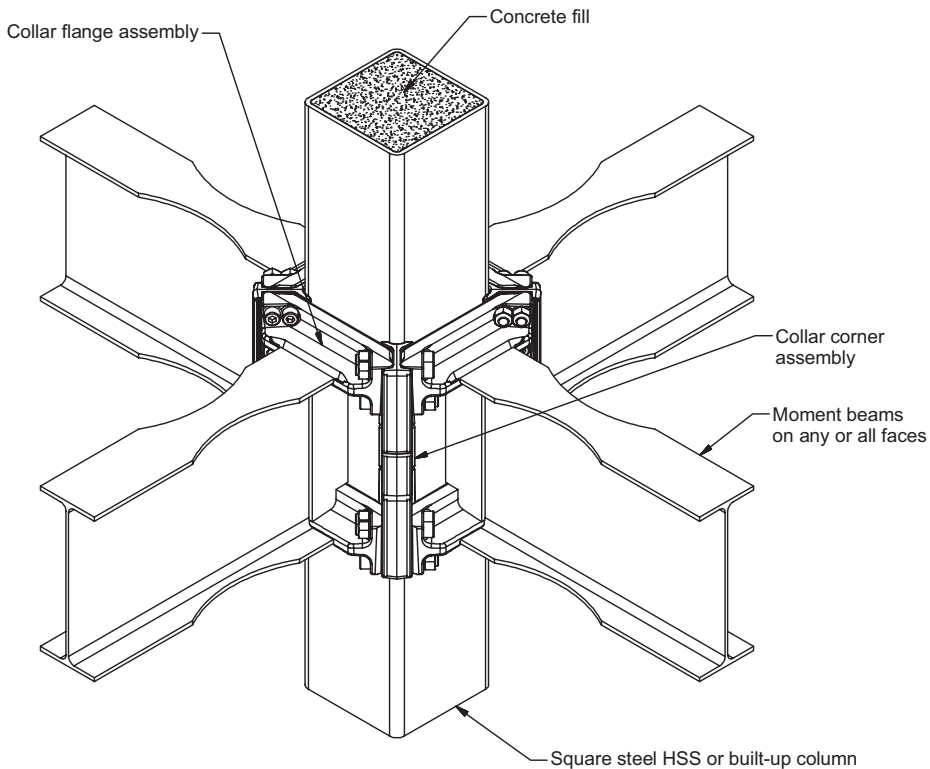


Fig. 10.1. Assembled ConXL moment connection.

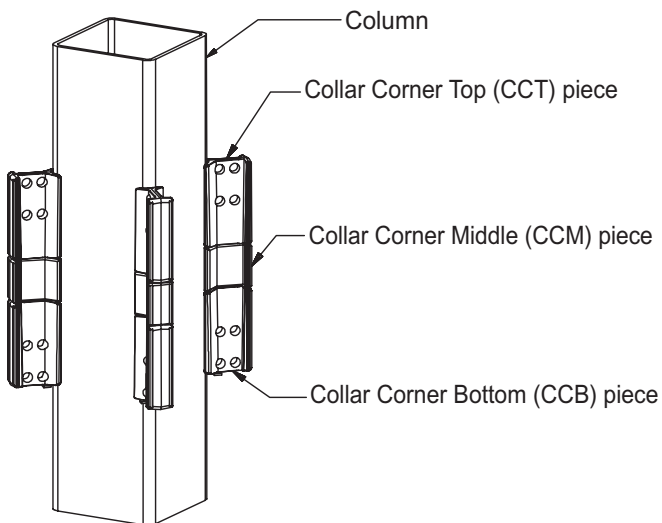


Fig. 10.2. Column with attached collar corner assemblies.

create the collar corner assembly; they are then shop fillet welded to the corners of the square column.

Figure 10.3 shows the collar flange assembly. Each collar flange assembly is made up of a collar flange top (CFT), collar flange bottom (CFB), and a collar web extension (CWX).

If a beam at the node requires a moment connection, the CFT (or CFB) is aligned with and shop-welded to the top (or bottom) flange of the beam.

Moment-connected beam webs are also shop-welded to the CWX. If a beam at the node does not require a moment connection, the size of the CWX remains unchanged, and a shear plate connection is shop-welded to the CWX to accommodate a non-moment-connected beam that does not need to match the nominal depth of the moment-connected beam(s).

If no beams exist on a node at a particular column face, the CFT and CFB are aligned at the nominal depth of the moment beam, and the CWX shall be permitted to be optionally omitted.

Section 10.9 contains drawings indicating the dimensions of individual pieces.

Columns are delivered to the job site with the collar corner assemblies shop-welded to the column at the proper floor framing locations. Beams are delivered to the job site with the collar flange assemblies shop-welded to the ends of the beams. During frame erection, the collar flange assemblies with or without beams are lowered into

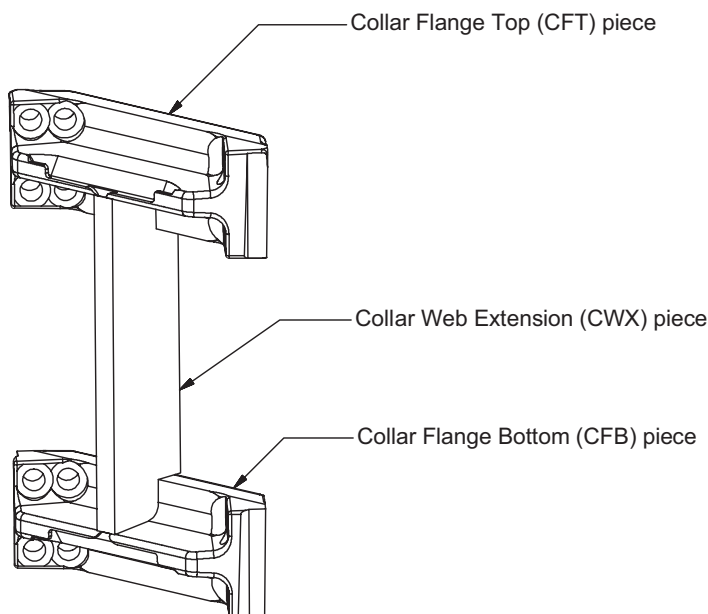


Fig. 10.3. Collar flange assembly.

the column collar corner assemblies. When all four faces of the column are filled with collar flanges, the collar bolts are inserted and pretensioned, effectively clamping and compressing the collar flange assemblies around the collar corner assemblies and square column.

Beam flange flexural forces in moment beams are transferred to the collar flange assemblies via CJP groove welds. Collar flanges transfer compressive beam flange forces to the collar corners through flexure of the collar flange and direct bearing onto the collar corners. The collar flange transfers beam flange tensile forces in flexure to the pretensioned collar bolts. The collar bolts transfer these forces in tension through the orthogonal collar flanges, which then transfer the forces through the rear collar bolts attached to the collar flange on the opposite face of the column. These combined forces are then transferred to the column walls through a combination of bearing and the fillet welds attaching the collar corners to the column. Finally, a portion of these forces are transferred to the concrete fill, which is in direct contact with the column walls.

The behavior of this connection is controlled by flexural hinging of the beams adjacent to the collar assembly. When RBS cutouts are provided, yielding and plastic hinge formation primarily occur within the reduced beam section.

10.2. SYSTEMS

The ConXL moment connection is prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems within the limits of these provisions. The ConXL moment connection is prequalified for use in planar moment-resisting frames or in orthogonal intersecting moment-resisting frames.

ConXL SMF systems with concrete structural slabs are prequalified only if a vertical flexible joint at least 1 in. (25 mm) thick is placed in the concrete slab around the collar assembly and column, similar to that shown in Figure 10.4.

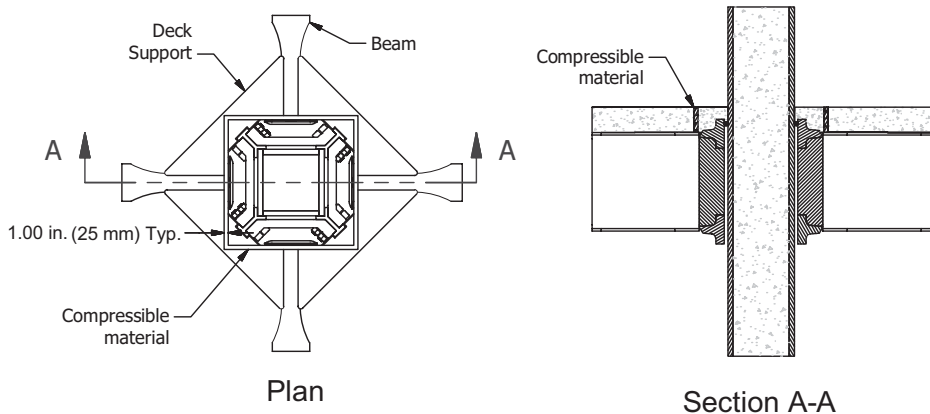


Fig.10.4. Use of compressible material to isolate structural slab from connection.

10.3. PREQUALIFICATION LIMITS

1. Beam Limitations

Beams shall satisfy the following limitations:

- (1) Beams shall be rolled wide-flange or built-up I-shaped members conforming to the requirements of Section 2.3.
- (2) Beam depths shall be limited to the following beam shapes or their built-up equivalents: W30 (W760), W27 (W690), W24 (W610), W21 (W530) and W18 (W460).
- (3) Beam flange thickness shall be limited to a maximum of 1 in. (25 mm).
- (4) Beam flange width shall be limited to a maximum of 12 in. (300 mm).
- (5) The clear span-to-depth ratio of the beam shall be limited as follows:
 - (a) For SMF systems, 7 or greater.
 - (b) For IMF systems, 5 or greater.
- (6) Width-to-thickness ratios for beam flanges and webs shall conform to the limits of the AISC *Seismic Provisions*. The value of b_f used to determine the width-to-thickness ratio of beams with RBS cutouts shall not be less than the flange width at the center two-thirds of the reduced section provided that gravity loads do not shift the location of the plastic hinge a significant distance from the center of the reduced beam section.
- (7) Lateral bracing of beams shall conform to the applicable limits of the AISC *Seismic Provisions*.

Exception: For SMF and IMF systems, where the beam supports a concrete structural slab that is connected between the protected zones with welded shear connectors spaced at a maximum of 12 in. (300 mm) on center, supplemental top and bottom flange bracing at the expected hinge is not required.

- (8) For RBS connections, the protected zone consists of the portion of the connection assembly and beam between the column face and the farthest end of the reduced beam section. For beams without reduced beam sections, the protected zone consists of the portion of the connection assembly and beam extending from the column face to a distance of d from the outside face of the collar flange.

2. Column Limitations

Columns shall satisfy the following limitations:

- (1) Columns shall be square 16 in. (400 mm) HSS sections or square 16 in. (400 mm) built-up box sections permitted in Section 2.3.
- (2) There is no limit on column weight per foot.
- (3) Column wall thickness shall not be less than $\frac{3}{8}$ in. (10 mm). Column wall thickness for HSS columns shall not be less than $\frac{3}{8}$ in. (10 mm) nominal.

- (4) Width-to-thickness ratios for columns shall conform to the applicable limits for filled composite columns in the AISC *Seismic Provisions*.
- (5) Lateral bracing of columns shall conform to the applicable limits in the AISC *Seismic Provisions*.
- (6) Columns shall be completely filled with structural concrete having unit weight not less than 110 lb/ft^3 (17 kN/m^3). Concrete shall have 28-day compressive strength not less than 3,000 psi (21 MPa).
- (7) Flanges and webs of built-up box columns shall be connected using partial-joint-penetration groove welds with a groove weld size not less than $\frac{3}{4}$ of the thickness of the connected plates in accordance with Figure 10.5.

3. Collar Limitations

Collars shall satisfy the following limitations:

- (1) Collar forgings shall conform to the requirements of Appendix B, Forging Requirements. Forged parts shall conform to the material requirements of ASTM A572/A572M Grade 50 (Grade 345).
- (2) Cast collar parts shall conform to the requirements of Appendix A, Casting Requirements. Cast parts shall conform to the requirements of ASTM A958/A958M Grade SC8620, class 80/50.
- (3) Collar configuration and proportions shall conform to Section 10.9, ConXL Part Drawings.
- (4) Collar flange bolt holes shall be $\frac{1}{8}$ in. (3 mm) larger than the nominal bolt diameter. Bolt holes shall be drilled.

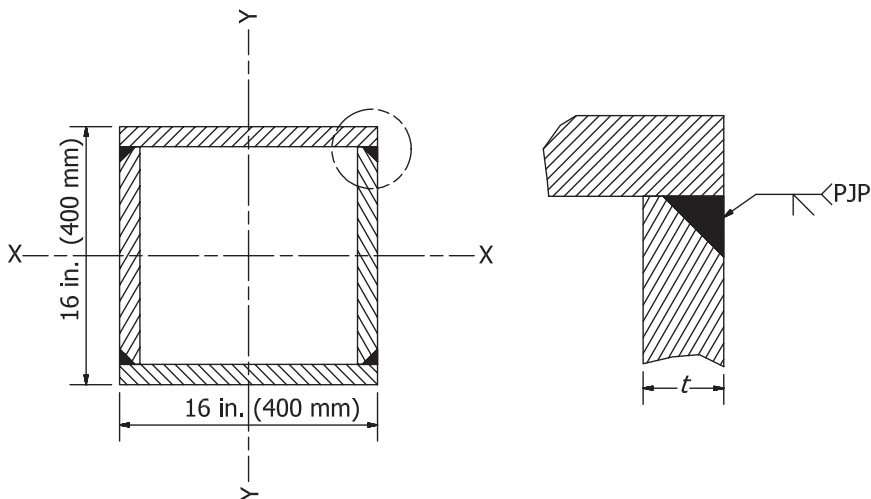


Fig. 10.5. Built-up box column flange-to-web connection detail.

- (5) Collar corner bolt holes shall be $\frac{1}{8}$ in. (3 mm) larger than the nominal bolt diameter. Bolt holes shall be drilled.
- (6) Material thickness, edge distance, end distance and overall dimension shall have a tolerance of $\pm \frac{1}{16}$ in. (2 mm).
- (7) Faying surfaces shall be machined and meet the requirements for Class A slip-critical surfaces as defined in the AISC *Specification*.

10.4. COLLAR CONNECTION LIMITATIONS

Collar connections shall satisfy the following limitations:

- (1) Collar bolts shall be pretensioned $1\frac{1}{4}$ -in. (31.8-mm) -diameter high-strength bolts conforming to ASTM A574 with threads excluded from the shear plane and shall conform to the requirements of Sections 4.2 and 4.3.
- (2) The collar bolts shall be pretensioned to the requirements for ASTM F3125 Grade A490 bolts in the RCSC *Specification*.
- (3) Welding of CCT, CCM and CCB pieces to form collar corner assemblies shall consist of partial-joint-penetration groove welds per Figure 10.6.

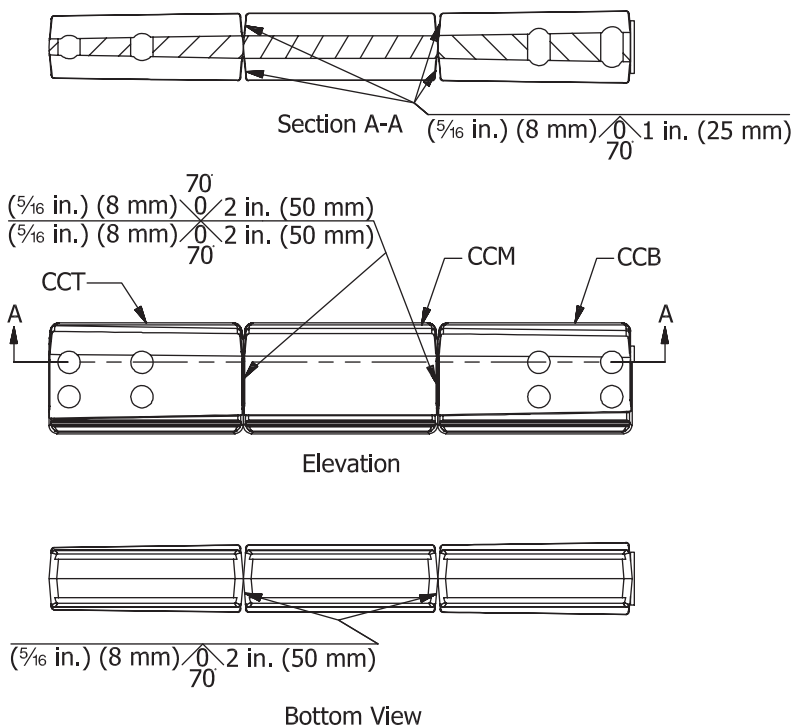


Fig. 10.6. Collar corner assembly welding.

- (4) Welding of collar corner assemblies to columns shall consist of flare bevel groove welds with $\frac{3}{8}$ -in. (10-mm) fillet reinforcing per Figure 10.7.
- (5) Collar flanges shall be welded to CWX pieces with $\frac{5}{16}$ -in. (8-mm) fillet welds, each side per Figure 10.8.

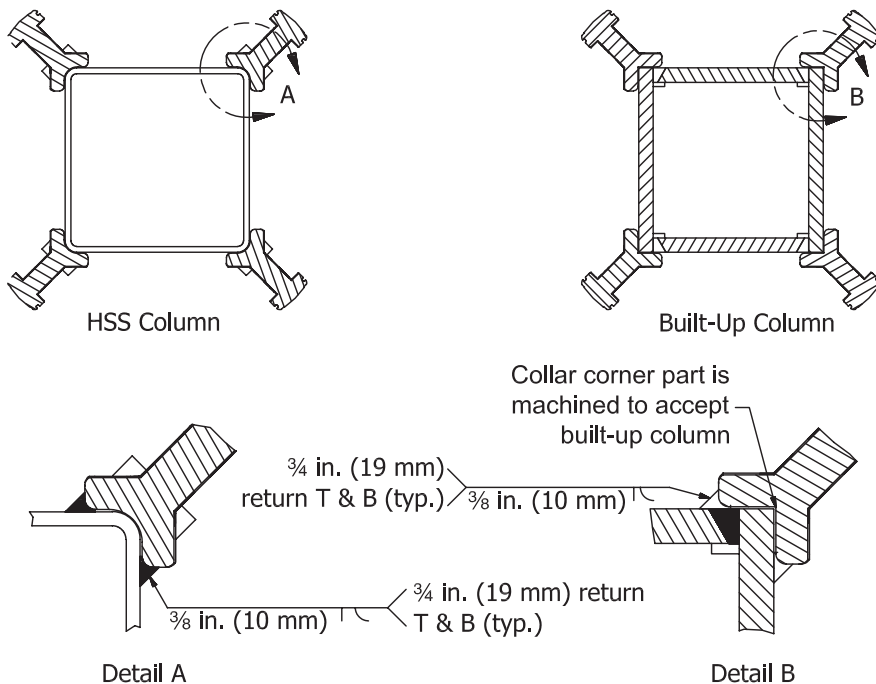


Fig. 10.7. Collar-corner-assembly-to-column weld, plan view.

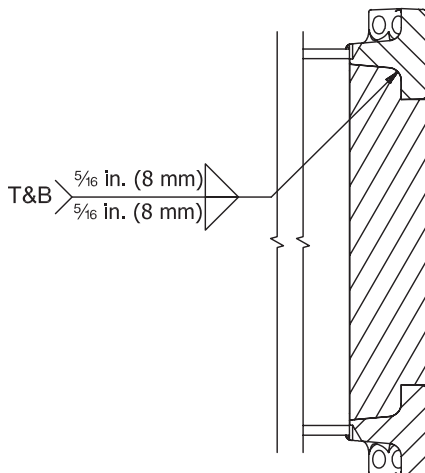


Fig. 10.8. Collar-web-extension-to-collar-flange welds, elevation.

- (6) Beams shall be welded to collar flange assemblies with complete-joint-penetration groove welds per Figure 10.9.

10.5. BEAM WEB-TO-COLLAR CONNECTION LIMITATIONS

Beam-web-to-collar connections shall satisfy the following limitations:

- (1) The required shear strength of the beam web connection shall be determined according to Section 10.8.
- (2) The beam web is welded to the collar web extension (CWX) with a two-sided fillet weld. The fillet welds shall be sized to develop the required shear strength of the connection.

10.6. BEAM FLANGE-TO-COLLAR FLANGE WELDING LIMITATIONS

Welding of the beam to the collar flange shall conform to the following limitations:

- (1) Weld access holes are not allowed. Welding access to top and bottom flanges shall be made available by rotating the beam to allow a CJP weld in the flat position (position 1G per AWS D1.1/D1.1M).
- (2) The beam-flange-to-collar-flange weld shall be made with a CJP groove weld within the weld prep area of the collar flange. Reinforcing $\frac{5}{16}$ -in. (8-mm) fillet welds shall be placed on the back side of the CJP groove welds. The CJP flange weld shall conform to the requirements for demand critical welds in the AISC *Seismic Provisions* and AWS D1.8/D1.8M and to the requirements of AWS D1.1/D1.1M.

10.7. COLUMN-BEAM RELATIONSHIP LIMITATIONS

Beam-to-column connections shall satisfy the following limitations:

- (1) Panel zones shall conform to the applicable requirements of the AISC *Seismic Provisions*.
- (2) Column-beam moment ratios shall be limited as follows:

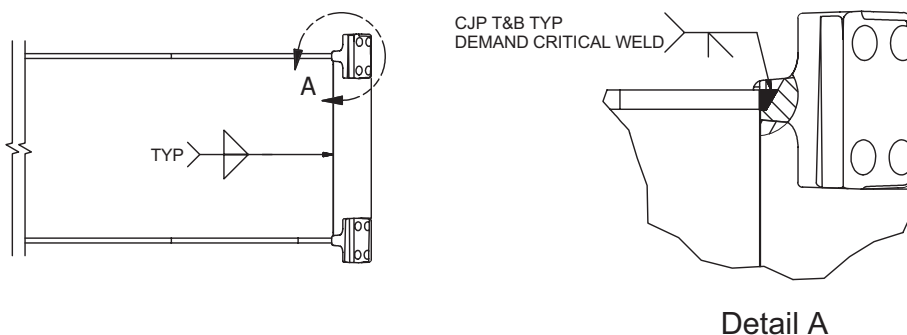


Fig. 10.9. Collar-flange-assembly-to-beam welds, elevation.

- (a) For SMF systems, the column-beam moment ratio about each principal axis shall conform to the requirements of the AISC *Seismic Provisions* considering simultaneous development of the expected plastic moments in the moment-connected beams framing into all sides of the ConXL node.
- (b) For IMF systems, the column-beam moment ratio shall conform to the requirements of the AISC *Seismic Provisions*.

10.8. DESIGN PROCEDURE

Step 1. Compute the probable maximum moment at the plastic hinge, M_{pr} , in accordance with Section 2.4.3.

$$M_{pr} = C_{pr} R_y F_y Z_e \quad (2.4-1)$$

where

$$C_{pr} = \frac{F_y + F_u}{2F_y} \leq 1.2 \quad (\text{for RBS beams}) \quad (2.4-2)$$

$$C_{pr} = 1.1 \quad (\text{for non-RBS beams})$$

F_u = specified minimum tensile strength of yielding element, ksi (MPa)

F_y = specified minimum yield stress of yielding element, ksi (MPa)

R_y = ratio of expected yield stress to specified minimum yield stress, F_y , as specified in the AISC *Seismic Provisions*

Z_e = effective plastic section modulus of the section at location of plastic hinge, in.³ (mm³)

For beams with an RBS cutout, the plastic hinge shall be assumed to occur at the center of the reduced section of beam flange. For beams without an RBS cutout, the plastic hinge shall be assumed to occur at a distance $d/2$ from the outside face of the collar (see Figure 10.10), where d is the beam depth.

Step 2. Compute the shear force, V_h , at the location of the plastic hinge at each end of the beam.

The shear force at each plastic hinge location shall be determined from a free-body diagram of the portion of the beam between the plastic hinge locations. This calculation shall assume that the moment at the center of the plastic hinge is M_{pr} and shall consider gravity loads acting on the beams between plastic hinges in accordance with the equation:

$$V_h = \frac{2M_{pr}}{L_h} + V_{gravity} \quad (10.8-1)$$

where

L_h = distance between plastic hinge locations, in. (mm)

$V_{gravity}$ = beam shear force resulting from $1.2D + f_1L + 0.2S$ (where f_1 is the load factor determined by the applicable building code from live loads, but not less than 0.5), kips (N).

User Note: The load combination of $1.2D + f_1L + 0.2S$ is in conformance with ASCE/SEI 7-16. When using the International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.2 for S (snow) when the roof configuration is such that it does not shed snow off of the structure.

When concentrated loads are present on the beam between the points of plastic hinging, they must be considered using standard considerations of statics when calculating the beam shear and using the same load combination.

Step 3. Confirm that columns are adequate to satisfy biaxial strong column-weak beam conditions. For the purpose of satisfying this requirement, it shall be permitted to take the yield strength of the column material as the specified F_y and to consider the full composite behavior of the column for axial load and flexural action.

User Note: The specified value of F_y need not be the minimum value associated with the grade of steel if project specifications require a higher minimum yield strength.

The value of $\sum M_{pb}^*$ about each axis shall be taken equal to $\sum(M_{pr} + M_v)$, where M_{pr} is computed according to Equation 2.4.3-1, and where M_v is the additional moment due to the beam shear acting on a lever arm extending from the assumed point of

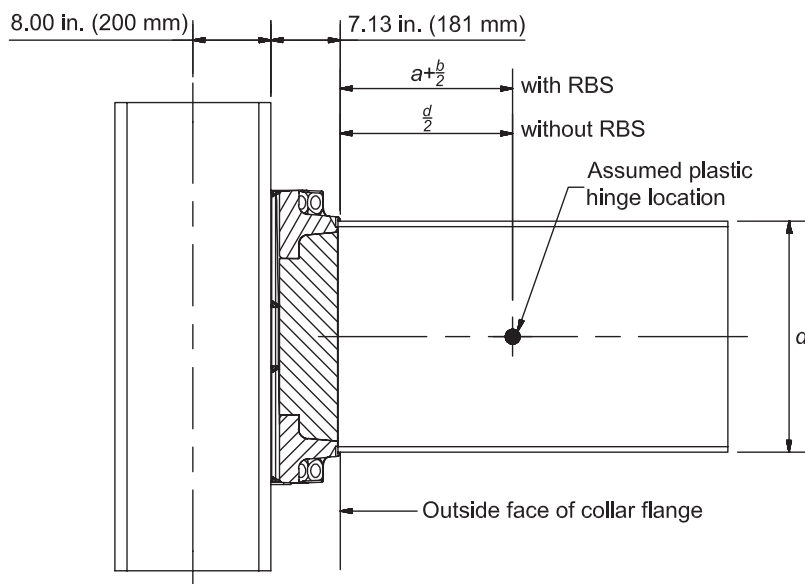


Fig. 10.10. Assumed plastic hinge location.

plastic hinging to the centerline of the column. M_v on each side of the column can be computed as the quantity $V_h s_h$, where V_h is the shear at the point of theoretical plastic hinging, computed in accordance with Equation 10.8-1 and s_h is the distance of the assumed point of plastic hinging to the column centerline.

For beams with reduced beam section (RBS) cutout, the distance s_h shall be taken as the distance from the center of the column to the center of the reduced section of beam flange. For beams without an RBS cutout, the distance s_h shall be taken as the distance from the center of the column to a point one-half the beam depth ($d/2$) from the outside face of collar (see Figure 10.10).

The value of $\sum M_{pc}^*$ about each axis shall be taken as:

$$\sum M_{pc}^* = M_{pcu}^* + M_{pcl}^* + \frac{\sum M_{pb}^*}{(H_u + H_l)} d \quad (10.8-2)$$

where

H_u = height of story above node, in. (mm)

H_l = height of story below node, in. (mm)

M_{pcu}^* = plastic moment nominal strength of column above node, about axis under consideration considering simultaneous axial loading and loading about transverse axis, kip-in. (N-mm)

M_{pcl}^* = plastic moment nominal strength of column below node, about axis under consideration considering simultaneous axial loading and loading about transverse axis, kip-in. (N-mm)

For sections with equal properties about both axes, it is permitted to take M_{pcu}^* and M_{pcl}^* as:

$$M_{pcu}^* = M_{pcl}^* = 0.67 Z_c F_y \left(1 - \frac{P_u}{A_s F_y + 0.85 A_c f'_c} \right) \quad (10.8-3)$$

where

A_c = area of concrete in column, in.² (mm²)

A_s = area of steel in column, in.² (mm²)

f'_c = specified compressive strength of concrete fill, ksi (MPa)

P_u = axial load acting on column at section under consideration in accordance with the applicable load combination specified by the building code, but not considering amplified seismic load, kips (N)

Z_c = plastic section modulus of the column about either axis, in.³ (mm³)

Step 4. Compute the moment at the collar bolts for each beam:

$$M_{bolts} = M_{pr} + V_h s_{bolts} \quad (10.8-4)$$

where

M_{bolts} = moment at collar bolts, kip-in. (N-mm)

s_{bolts} = distance from center of plastic hinge to centroid of collar bolts, in. (mm)

$$= \frac{t_{collar}}{2} + a + \frac{b}{2} \quad (\text{for RBS beams}) \quad (10.8-5)$$

$$= \frac{t_{collar}}{2} + \frac{d}{2} \quad (\text{for non-RBS beams}) \quad (10.8-6)$$

where

a = distance from outside face of collar to RBS cut, in. (mm)

b = length of RBS cut, in. (mm)

t_{collar} = distance from face of the column to outside face of collar, taken as 7 $\frac{1}{8}$ in. (181 mm) as illustrated in Figure 10.10

Step 5. Verify that the beam flange force does not exceed the available tensile strength of the bolts at the flange connection. The following relationship shall be satisfied for the collar bolts tensile strength:

$$\frac{r_{ut}}{\phi_d R_{pt}} = \frac{r_{ut}}{102} \leq 1.0 \quad (10.8-7)$$

$$\frac{r_{ut}}{\phi_d R_{pt}} = \frac{r_{ut}}{454,000} \leq 1.0 \quad (10.8-7M)$$

where

R_{pt} = minimum bolt pretension, kips (N)

n_{cf} = number of collar bolts per collar flange
= 8

r_{ut} = required collar bolt tensile strength, kips (N)

$$= \frac{M_{bolts}}{n_{cf} d \sin 45^\circ} = 0.177 \frac{M_{bolts}}{d} \quad (10.8-8)$$

Step 6: Compute V_{bolts} , the probable maximum shear at the collar bolts, equal to the shear at the plastic hinge, V_h , plus any additional gravity loads between the plastic hinge and center of the collar flange, using the load combination of Step 2. Confirm that V_{bolts} is less than the slip-critical, Class A bolt design strength in accordance with the AISC *Specification* and using a resistance factor, ϕ , of unity.

User Note: Note that for 1 $\frac{1}{4}$ -in. (31.8-mm)-diameter ASTM A574 bolts, the value of T_b is the same as for 1 $\frac{1}{4}$ -in. (31.8-mm)-diameter ASTM F3125 Grade A490 or Grade A490M bolts and has a value of 102 kips (454 kN).

Step 7: Compute V_{cf} , the probable maximum shear at the face of collar flange, equal to the shear at the plastic hinge, V_h , plus any additional gravity loads between the plastic hinge and the outside face of the collar flange using the load combination of Step 2.

Check the design shear strength of the beam according to the requirements of the AISC *Specification* against V_{cf} .

Step 8: Determine required size of the fillet weld connecting the beam web to the collar web extension (CWX) using the following relationship:

$$t_f^{CWX} \geq \frac{\sqrt{2}V_{cf}}{\phi_n F_w l_w^{CWX}} \quad (10.8-9)$$

where

F_w = nominal weld design strength per the AISC *Specification*
= $0.60F_{EXX}$, ksi (MPa)

l_w^{CWX} = total length of available fillet weld at CWX, in. (mm), taken as 54 in. (1370 mm) for W30 (W760) sections, 48 in. (1220 mm) for W27 (W690) sections, 42 in. (1070 mm) for W24 (W610) sections, 36 in. (914 mm) for W21 (W530) sections, and 30 in. (760 mm) for W18 (W460) sections

t_f^{CWX} = fillet weld size required to join each side of beam web to CWX, in. (mm)

Step 9: Compute V_f , the probable maximum shear at the face of column, equal to the shear at the plastic hinge, V_h , plus any additional gravity loads between the plastic hinge and the face of the column using the load combination of Step 2.

Determine size of fillet weld connecting collar corner assemblies to column using the following relationship:

$$t_f^{CC} \geq \frac{\sqrt{2}V_f}{\phi_n F_w l_w^{CC}} \quad (10.8-10)$$

where

l_w^{CC} = total length of available fillet weld at collar corner assembly, in. (mm), taken as 72 in. (1830 mm) for W30 (W760) sections, 66 in. (1680 mm) for W27 (W690) sections, 60 in. (1520 mm) for W24 (W610) sections, 54 in. (1370 mm) for W21 (W530) sections, and 48 in. (1220 mm) for W18 (W460) sections

t_f^{CC} = fillet weld size required to join collar corner assembly to column, in. (mm)

Step 10: Determine the required shear strength of the column panel zone, R_n^{pz} , using the following relationship:

$$R_n^{pz} = \frac{\sum(M_{pr} + V_h s_f)}{d} - V_{col} \quad (10.8-11)$$

where

$$H = \frac{H_u + H_l}{2} \quad (10.8-17)$$

V_{col} = column shear, kips (N)

$$= \frac{\sum(M_{pr} + V_h s_h)}{H} \quad (10.8-12)$$

d_c = depth of column, in. (mm)

$$s_f = \text{distance from center of plastic hinge to face of column, in. (mm)}$$

$$= t_{collar} + a + \frac{b}{2} \quad (\text{RBS beam}) \quad (10.8-13)$$

$$= t_{collar} + \frac{d}{2} \quad (\text{non-RBS beam}) \quad (10.8-14)$$

$$s_h = \text{distance from center of plastic hinge to center of column, in. (mm)}$$

$$= \frac{d_c}{2} + t_{collar} + a + \frac{b}{2} \quad (\text{RBS beam}) \quad (10.8-15)$$

$$= \frac{d_c}{2} + t_{collar} + \frac{d}{2} \quad (\text{non-RBS beam}) \quad (10.8-16)$$

Step 11: Determine the nominal design panel zone shear strength, ϕR_n^{Pz} , using the following relationship:

$$\phi R_n^{Pz} = \phi_d 0.6 F_y A_{pz} \quad (10.8-18)$$

where

$$A_{pz} = 2d_c t_{col} + 4 \left(d_{leg}^{CC} t_{leg}^{CC} \right) \quad (10.8-19)$$

d_{leg}^{CC} = effective depth of collar corner assembly leg, taken as 3½ in. (89 mm)

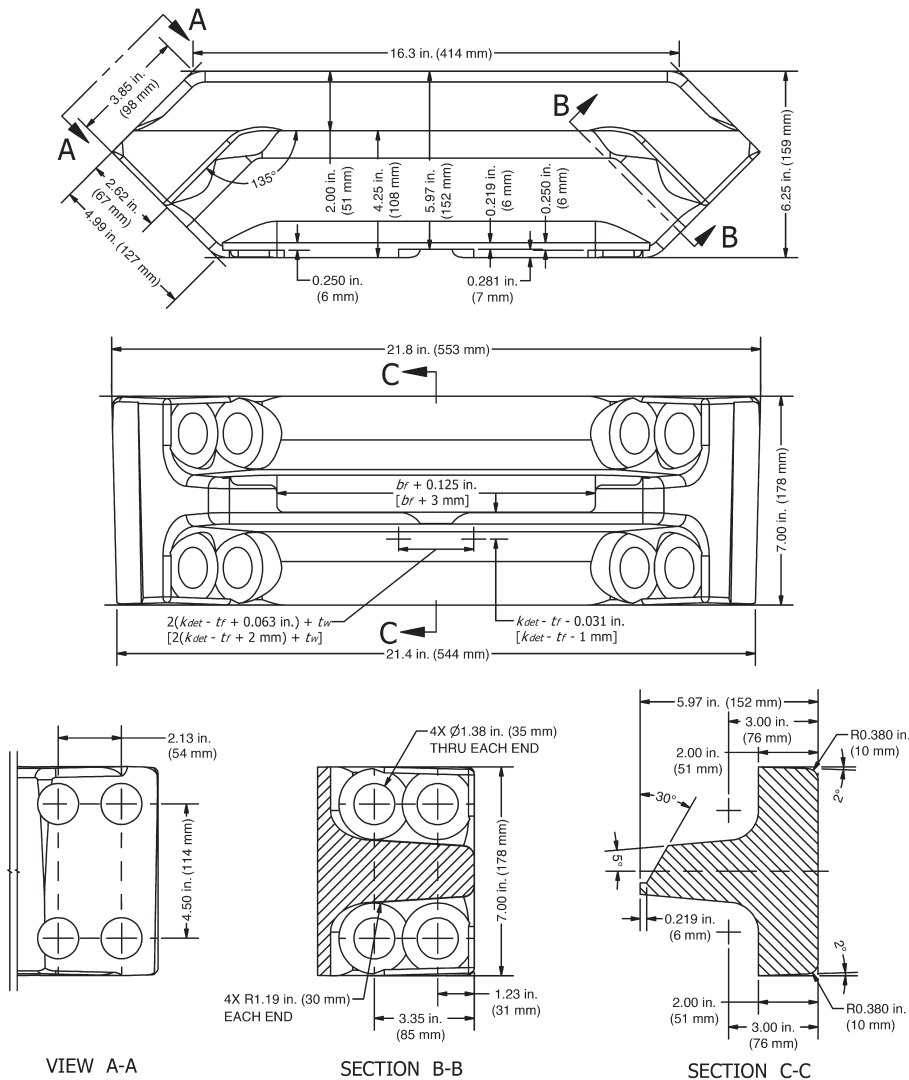
t_{col} = wall thickness of HSS or built-up box column, in. (mm)

t_{leg}^{CC} = effective thickness of collar corner assembly leg, taken as ½ in. (13 mm)

User Note: If the required strength exceeds the design strength, the designer may increase the column section and/or decrease the beam section strength, assuring that all other design criteria are met.

10.9. PART DRAWINGS

Figures 10.11 through 10.19 provide the dimensions of the various components of the ConXtech ConXL moment connection.

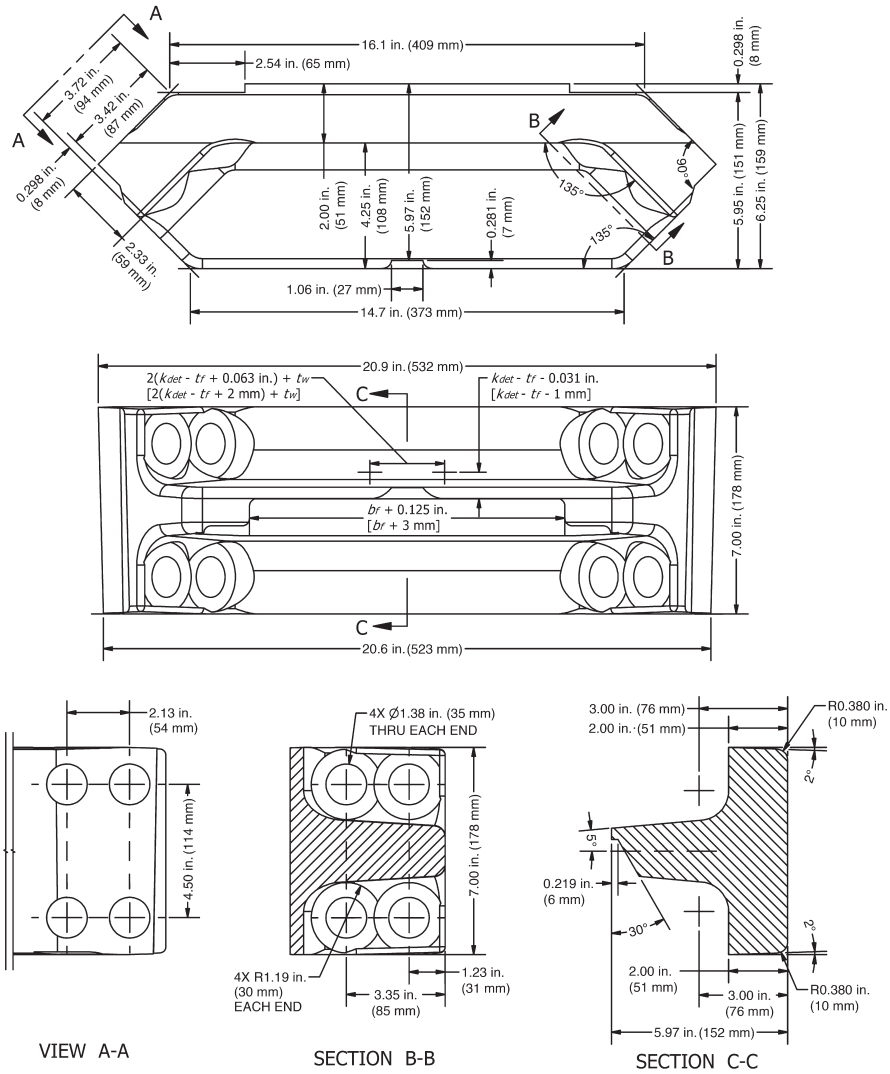


NOTES		
Item	Description	Tolerance Value
br	BEAM FLANGE WIDTH	n/a
tr	BEAM FLANGE THICKNESS	n/a
t_w	BEAM WEB THICKNESS	n/a
k_{det}	TOP-OF-STEEL TO WORKABLE GAGE	n/a
	FORGING TOLERANCE	+/- 0.3%
	DIE WEAR TOLERANCE	+ 0.5% / - 0.0
	MILLING TOLERANCE	+/- 0.020"

ConXL COLLAR FLANGE TOP

UNITS: IN / MM SIZE: A

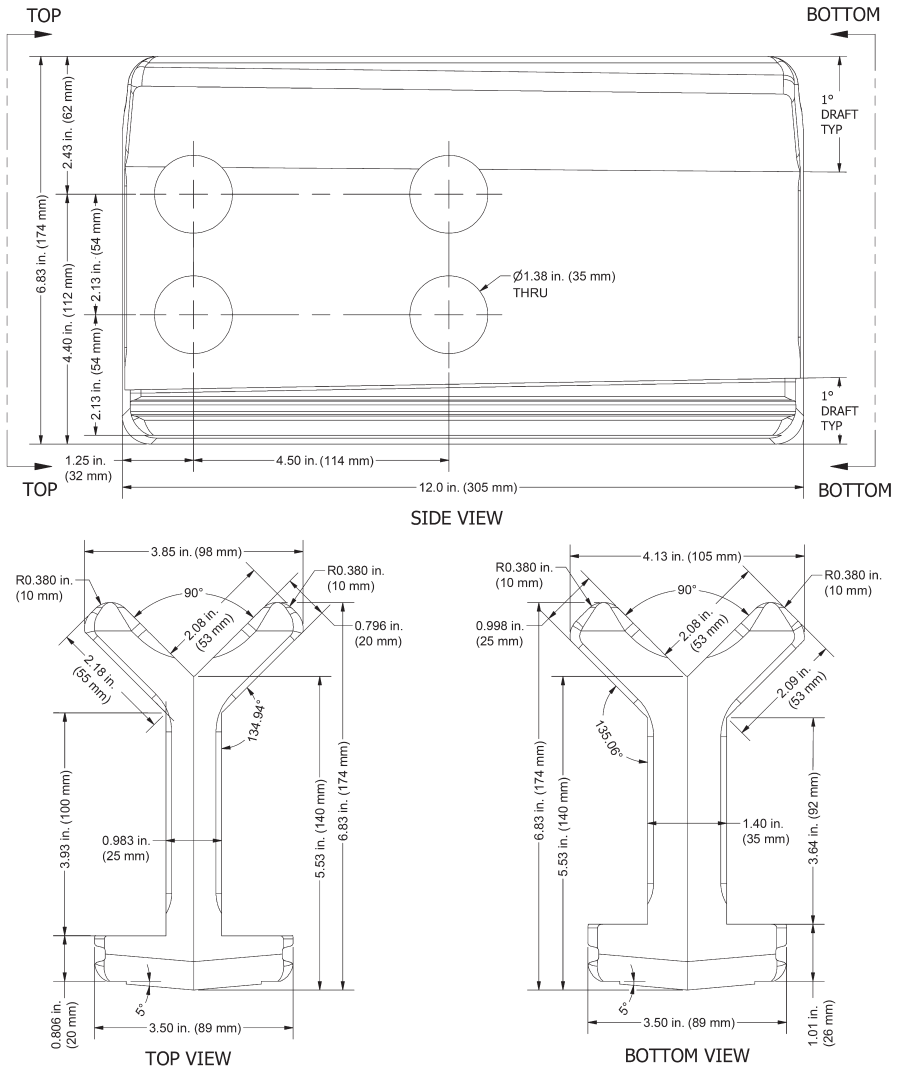
Fig. 10.11. Forged collar flange top (CFT).



NOTES		
Item	Description	Tolerance Value
br	BEAM FLANGE WIDTH	n/a
tr	BEAM FLANGE THICKNESS	n/a
t_w	BEAM WEB THICKNESS	n/a
k_{col}	TOP-OF-STEEL TO WORKABLE GAGE	n/a
	FORGING TOLERANCE	+/- 0.3%
	DIE WEAR TOLERANCE	+ 0.5% / - 0.0
	MILLING TOLERANCE	+/- 0.020"

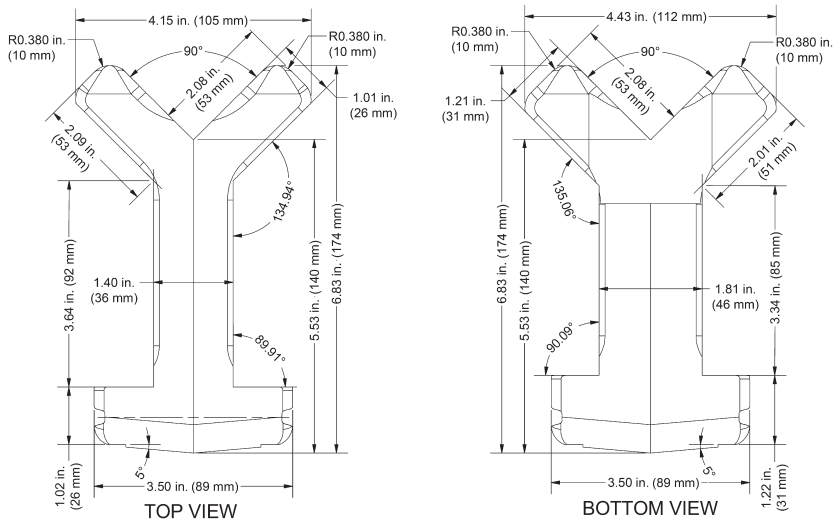
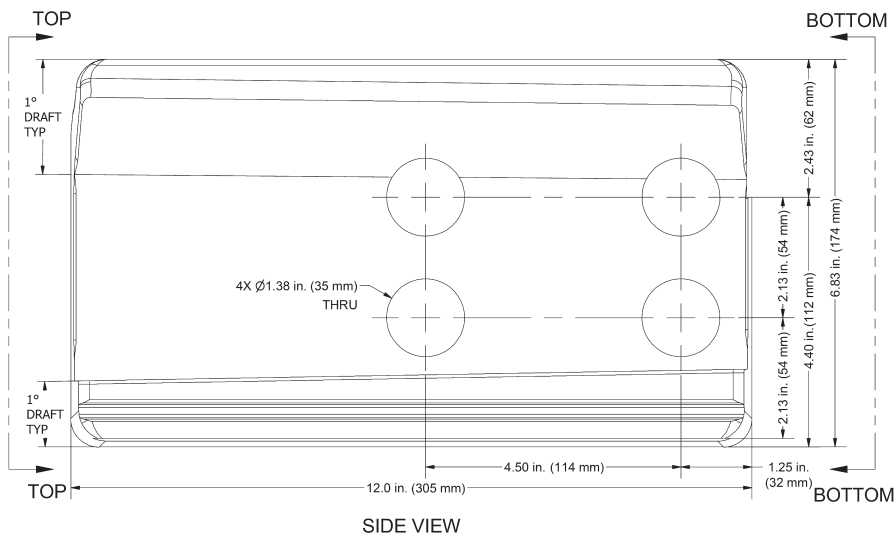
ConXL COLLAR FLANGE BOTTOM		
UNITS:	IN / MM	SIZE: A

Fig. 10.12. Forged collar flange bottom (CFB).



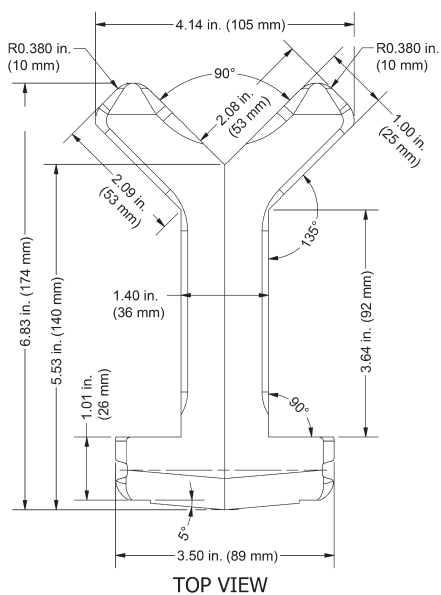
ConXL COLLAR CORNER TOP		NOTES	
		Description	Tolerance Value
UNITS: IN / MM <td>SIZE: A <td>FORGING TOLERANCE</td> <td>+/- 0.3%</td> </td>	SIZE: A <td>FORGING TOLERANCE</td> <td>+/- 0.3%</td>	FORGING TOLERANCE	+/- 0.3%
		DIE WEAR TOLERANCE	+ 0.5% / - 0.0
		MILLING TOLERANCE	+/- 0.020"

Fig. 10.13. Forged collar corner top (CCT).



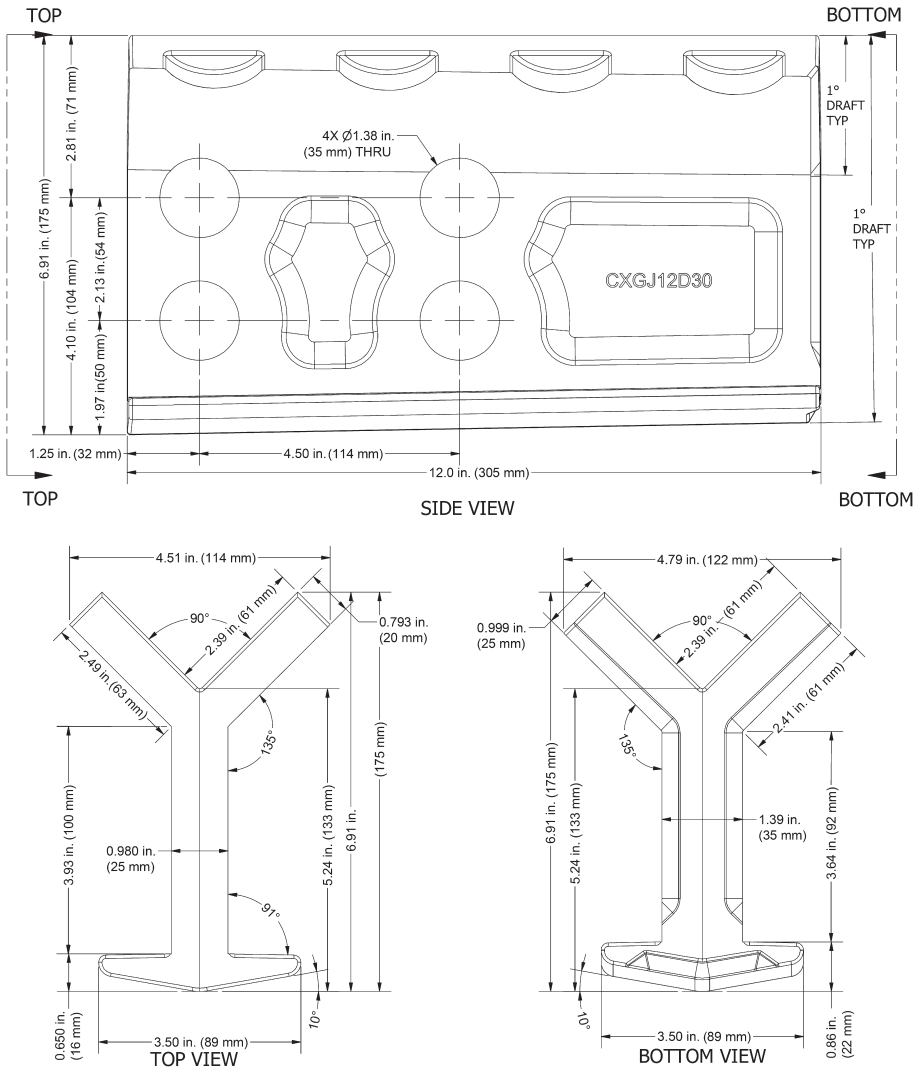
ConXL COLLAR CORNER BOTTOM			NOTES	
		Description	Tolerance Value	
		FORGING TOLERANCE	±0.3%	
		DIE WEAR TOLERANCE	+0.5% / -0.0	
		MILLING TOLERANCE	±0.020"	
UNITS: IN / MM	SIZE: A			

Fig. 10.14. Forged collar corner bottom (CCB).



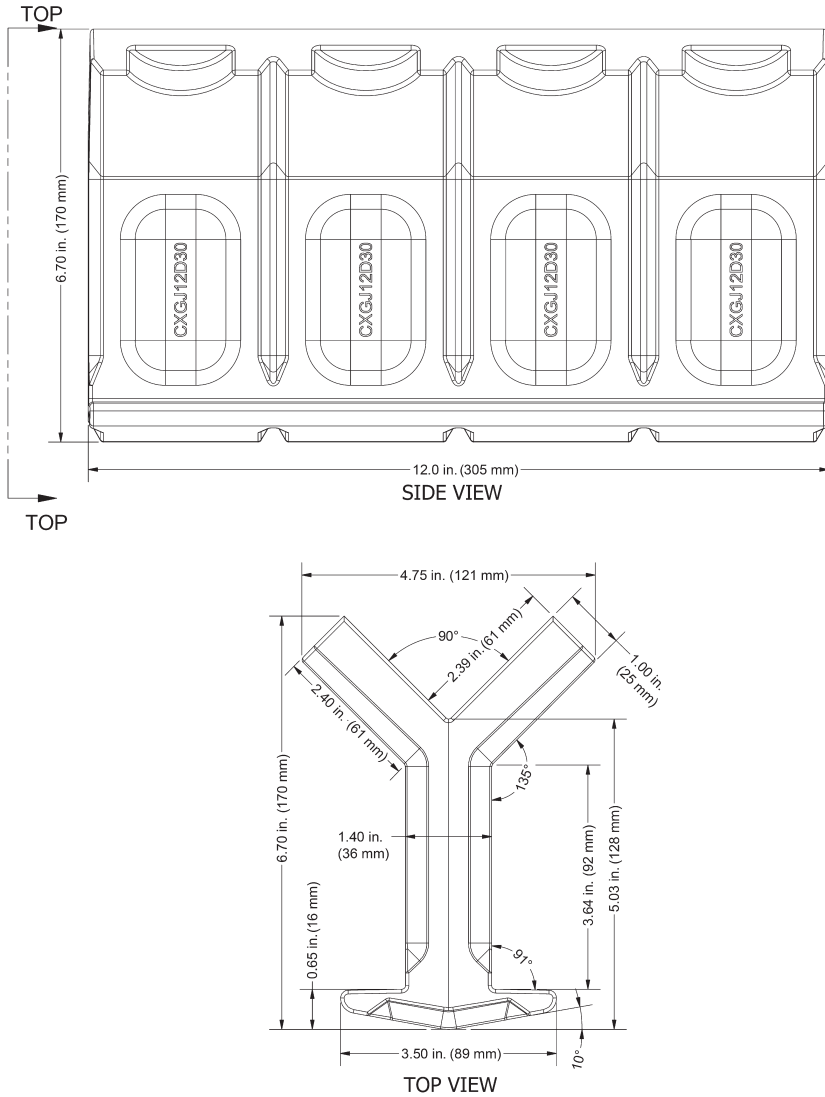
ConXL COLLAR CORNER MIDDLE			NOTES		
			Description	Tolerance Value	
UNITS:	IN	SIZE:	A	FORGING TOLERANCE	+/- 0.3%
				DIE WEAR TOLERANCE	+ 0.5% / - 0.0
				MILLING TOLERANCE	+/- 0.020"

Fig. 10.15. Forged collar corner middle (CCM).



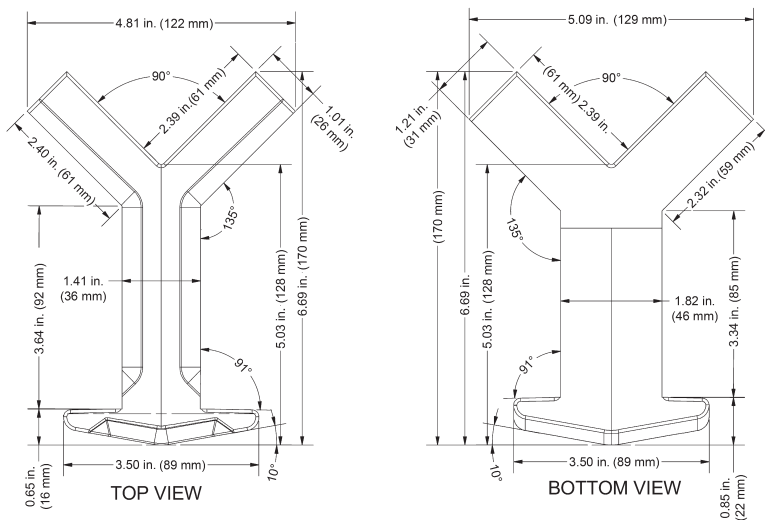
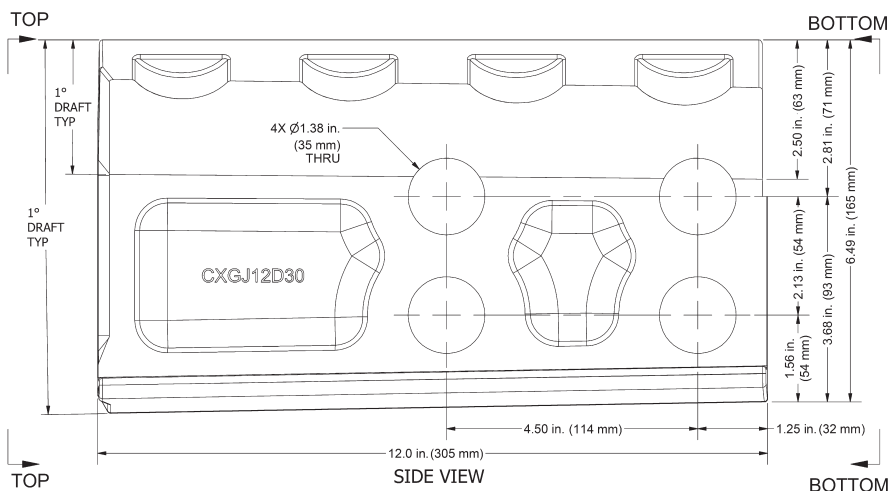
ConXL COLLAR CORNER TOP		NOTES	
		Description	Tolerance Value
UNITS: IN / MM	SIZE: A	FORGING TOLERANCE	+/- 0.3%
		DIE WEAR TOLERANCE	+ 0.5% / - 0.0
		MILLING TOLERANCE	+/- 0.020"

Fig. 10.16. Cast collar corner top (CCT).



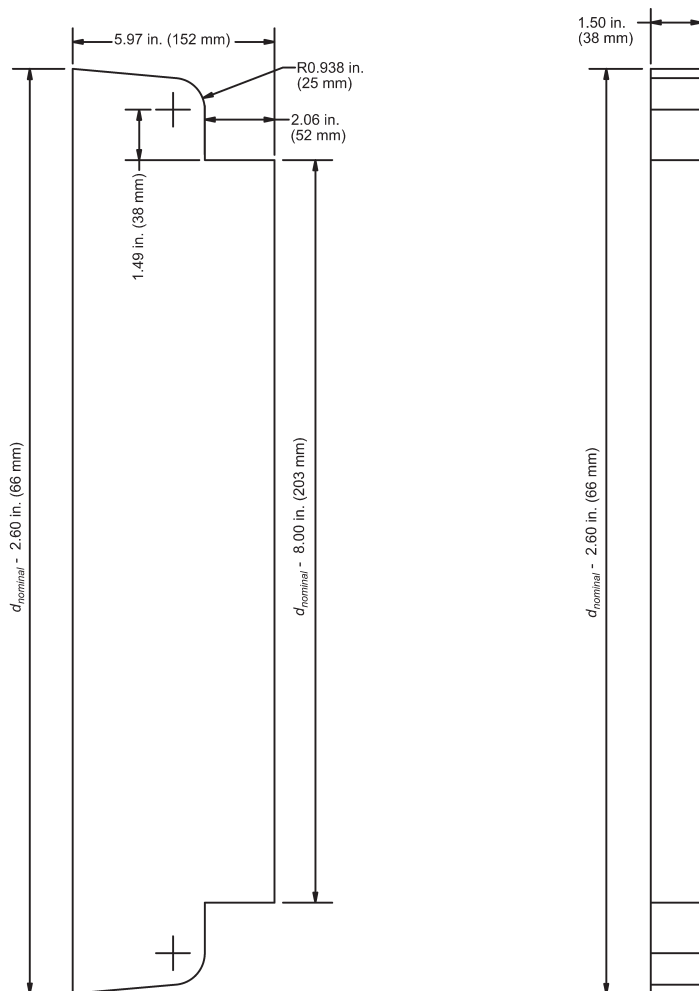
ConXL COLLAR CORNER MIDDLE			NOTES	
Description		Tolerance Value		
FORGING TOLERANCE		± 0.3%		
DIE WEAR TOLERANCE		+ 0.5% / - 0.0		
MILLING TOLERANCE		± 0.020"		
UNITS: IN / MM	SIZE: A			

Fig. 10.17. Cast collar corner middle (CCM).



ConXL COLLAR CORNER BOTTOM		NOTES	
		Description	Tolerance Value
UNITS:	IN / MM	FORGING TOLERANCE	+/- 0.3%
SIZE:	A	DIE WEAR TOLERANCE	+ 0.5% / - 0.0
		MILLING TOLERANCE	+/- 0.020"

Fig. 10.18. Cast collar corner bottom (CCB).



ConXL COLLAR WEB EXTENSION			NOTES		
			Item	Description	Tolerance Value
	d	BEAM FLANGE DEPTH		n/a	
		CUT TOLERANCE		+ 0.0 / - 0.030 in.	
	UNITS: IN	SIZE: A			

Fig. 10.19. Collar web extension (CWX).

CHAPTER 11

SIDEPLATE MOMENT CONNECTION

The user's attention is called to the fact that compliance with this chapter of the standard requires use of an invention covered by multiple U.S. and foreign patent rights. By publication of this standard, no position is taken with respect to the validity of any claim(s) or of any patent rights in connection therewith. The patent holder has filed a statement of willingness to grant a license under these rights on reasonable and nondiscriminatory terms and conditions to applicants desiring to obtain such a license, and the statement may be obtained from the standard's developer.*

11.1. GENERAL

The SidePlate[®] moment connection is a fully restrained connection of beams (comprising either rolled or built-up wide-flange sections or hollow structural sections) to columns (comprising either rolled or built-up wide-flange sections, built-up biaxial sections of wide-flange and/or tee section(s), or built-up box column sections) using fillet welds and interconnecting plates to connect the moment-resisting beam to its corresponding column as shown in Figure 11.1.

The connection system is typically constructed exclusively of fillet welds (except for flare bevel groove welds at rounded edges of HSS sections as applicable) for both shop fabrication and field erection. The connection features a physical separation, or gap, between the face of the column flange and the end of the beam. The connection of the beam to the column is accomplished with parallel full-depth side plates that sandwich and connect the beam(s) and the column together. Top and bottom beam flange cover plates (rectangular or U-shaped) are used at the end(s) of the beam, as applicable, which also serve to bridge any difference between flange widths of the beam(s) and of the column. Column horizontal shear plates and beam vertical shear elements (or shear plates as applicable) are attached to the column and beam webs, respectively.

Figure 11.2 shows the connection geometry and major connection components for uniaxial configurations. Figure 11.3 shows the connection geometry and major connection components for biaxial configurations, capable of connecting up to four beams to a column.

Moment frames that utilize the SidePlate connection system can be constructed using one of three methods. Most commonly, construction is with the SidePlate FRAME[®]

* The SidePlate[®] connection configurations and structures illustrated herein, including their described fabrication and erection methodologies, are protected by one or more of the following U.S. and foreign patents: U.S. Pat. Nos.: 5,660,017; 6,138,427; 6,516,583; 6,591,573; 7,178,296; 8,122,671; 8,122,672; 8,146,322; 8,176,706; 8,205,408; Mexico Pat. No.: 208,750; New Zealand Pat. No.: 300,351; British Pat. No.: 2497635; all held by MiTek Holdings LLC. Other U.S. and foreign patent protection are pending.

configuration that utilizes the full-length beam erection method, as shown in Figure 11.4a. This method employs a full-length beam assembly consisting of the beam with shop-installed cover plates (if required) and vertical shear elements (except for HSS beams), which are fillet-welded near the ends of the beam.

Column assemblies are typically delivered to the job site with the horizontal shear plates and side plates shop fillet welded to the column at the proper floor framing locations. Where built-up box columns are used, horizontal shear plates are not required, nor applicable.

During frame erection, the full-length beams are lifted up in between the side plates that are kept spread apart at the top edge of the side plates with a temporary shop-installed spreader [Figure 11.4(a)]. A few bolts connecting the beam's vertical shear plates (shear elements as applicable) to adjacent free ends of the side plates are initially inserted to provide temporary shoring of the full-length beam assembly, after which the temporary spreader is removed. The remaining erection bolts are then inserted, and all bolts are installed snug tight. These erection bolts also act as a clamp to effectively close any root gap that might have existed between the interior face of the side plates and the longitudinal edges of the top cover plate, while bringing the top face of the wider bottom cover plate into a snug fit with the bottom edges of the

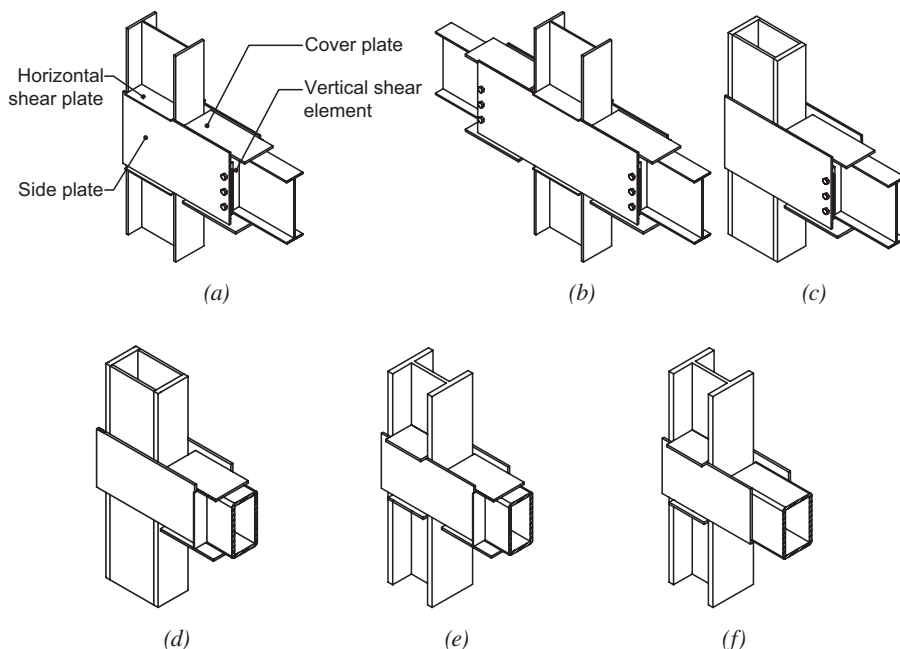


Fig. 11.1. Assembled SidePlate uniaxial configurations: (a) one-sided wide flange beam and column construction; (b) two-sided wide-flange beam and column construction; (c) wide-flange beam to built-up box column; (d) HSS beam without cover plates to wide-flange column; (e) HSS beam with cover plates to wide-flange column; and (f) HSS beam with cover plates to built-up box column.

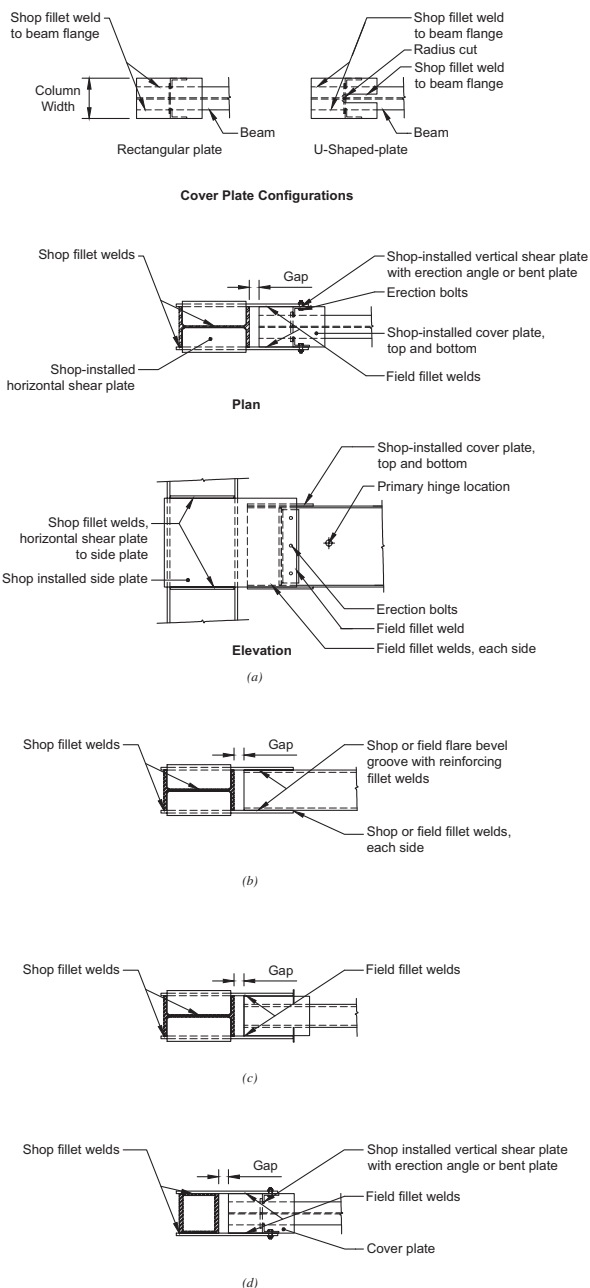


Fig. 11.2. SidePlate uniaxial configuration geometry and major components: (a) typical wide-flange beam to wide-flange column, detail, plan and elevation views; (b) HSS beam without cover plates to wide-flange column, plan view; (c) HSS beam with cover plates to wide-flange column, plan view; and (d) wide-flange beam to built-up box column, plan view.

side plates. To complete the field assembly, four horizontal fillet welds joining the side plates to the cover plates are then deposited in the horizontal welding position (position 2F per AWS D1.1/D1.1M), and, when applicable, two vertical single-pass field fillet welds joining the side plates to the vertical shear elements are deposited in the vertical welding position (position 3F per AWS D1.1/D1.1M).

Where the full-length beam erection method using the SidePlate FRAME configuration is not used, the original SidePlate configuration may be used. The original SidePlate configuration utilizes the link-beam erection method, which connects a link beam assembly to the beam stubs of two opposite column tree assemblies with field complete-joint-penetration (CJP) groove welds [Figures 11.4(b) and 11.4(c)]. In cases where moment frames can be shop prefabricated and shipped to the site in one piece, no field bolting or welding is required [Figure 11.4(d)]. As depicted in Figure 11.4, the full-length beam erection method can alternately be configured such that the width of bottom flange cover plate is equal to the width of the top cover plate (i.e., both cover plates fit within the separation of the side plates), in lieu of the bottom

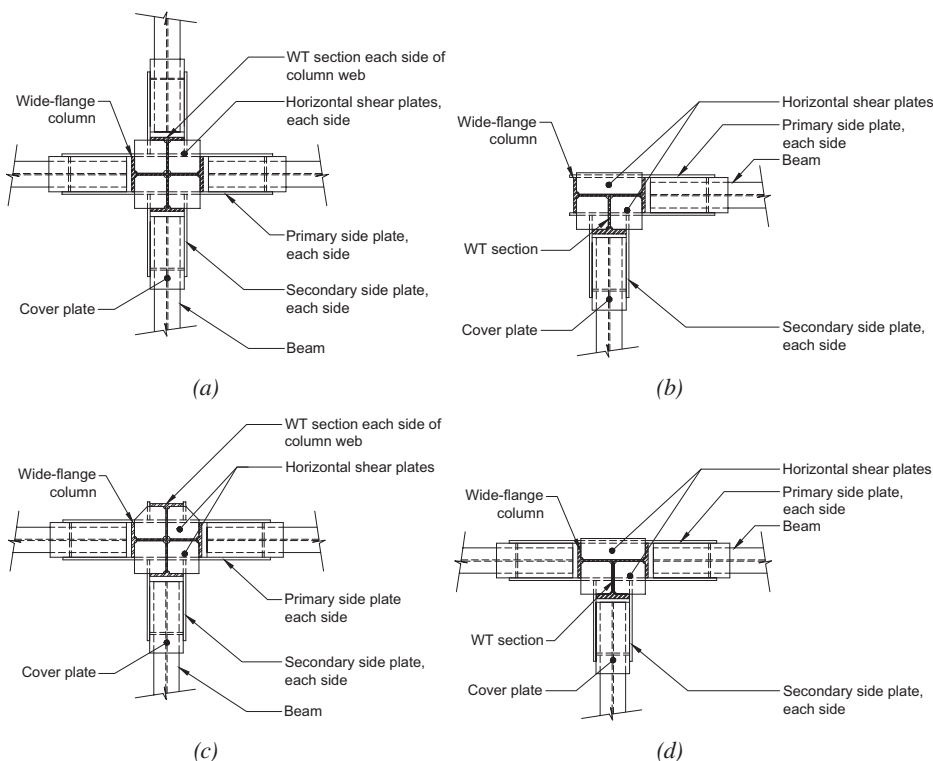


Fig. 11.3. SidePlate biaxial dual-strong axis configurations in plan view: (a) full four-sided wide-flange column configuration; (b) corner two-sided wide-flange column configuration with single WT; (c) tee three-sided wide-flange column configuration with double WT (primary); and (d) tee three-sided wide-flange column configuration with single WT.

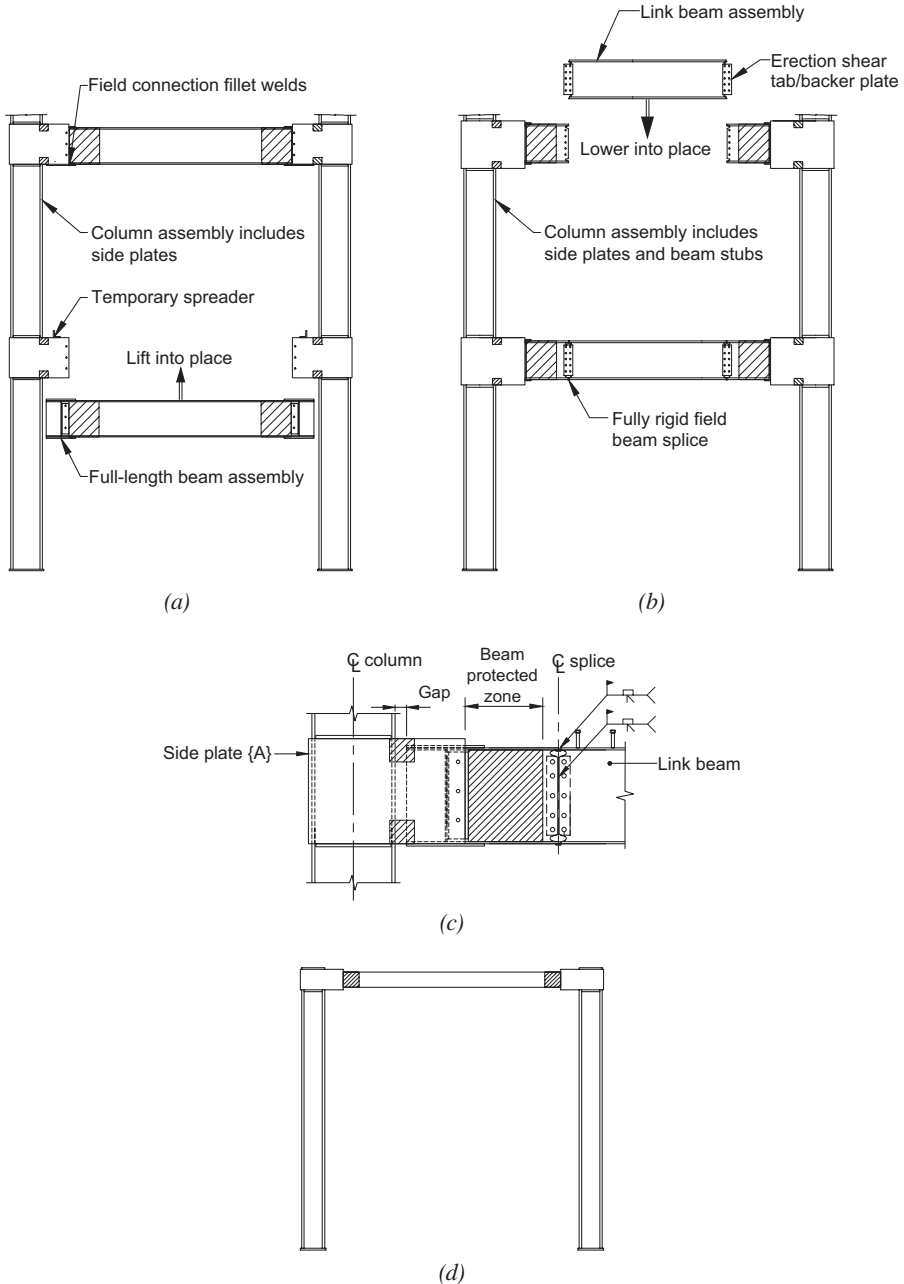


Fig. 11.4. SidePlate construction methods: (a) full-length beam erection method (SidePlate FRAME configuration); (b) link-beam erection method (original SidePlate configuration); (c) link beam-to-beam stub splice detail; and (d) all shop-prefabricated single-story moment frame (no field welding); multi-story frames are dependent on transportation capabilities.

cover plate being wider than the distance between side plates. Note that when this option is selected by the engineer, the two bottom fillet welds connecting the cover plates to the side plates will be deposited in the overhead welding position (position 4F per AWS D1.1/D1.1M).

The SidePlate moment connection is proportioned to develop the probable maximum moment capacity of the connected beam. Beam flexural, axial and shear forces are mainly transferred to the top and bottom rectangular cover plates via four shop horizontal fillet welds that connect the edges of the beam flange tips to the corresponding face of each cover plate (two welds for each beam flange). When the U-shaped cover plates are used, the same load transfer occurs via four shorter shop horizontal fillet welds that connect the edge of the beam flange tips to the corresponding face of each cover plate (two welds for each beam flange), as well as four shop horizontal fillet welds that connect the top face of the beam top flange and the bottom face of the bottom beam flange to the corresponding inside edge of each U-shaped cover plate (two welds for each beam flange face). These same forces are then transferred from the cover plates to the side plates via four field horizontal fillet welds that connect the cover plates to the side plates. The side plates transfer all of the forces from the beam (including that portion of shear in the beam that is transferred from the beam's web via vertical shear elements), across the physical gap to the column via shop fillet welding of the side plates to the column flange tips (a total of four shop fillet welds; two for each column flange), and to the horizontal shear plates (a total of four shop fillet welds; one for each horizontal shear plate). The horizontal shear plates are in turn shop fillet welded to the column web and under certain conditions, also to the inside face of column flanges.

Plastic hinge formation is intended to occur primarily in the beam beyond the end of the side plates away from the column face, with limited yielding occurring in some of the connection elements. The side plates, in particular, are designed with the expectation of developing moment capacity larger than the plastic moment capacity of the beam, and this results in yielding and strain hardening in the vicinity of the side plate protected zones.

11.2. SYSTEMS

The SidePlate moment connection is prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems within the limits of these provisions. The SidePlate moment connection is prequalified for use in planar moment-resisting frames and orthogonal intersecting moment-resisting frames (biaxial configurations, capable of connecting up to four beams at a column), as illustrated in Figure 11.3.

11.3. PREQUALIFICATION LIMITS

1. Beam Limitations

Beams shall satisfy the following limitations:

- (1) Beams shall be rolled wide-flange, hollow structural section (HSS), or built-up

I-shaped beams conforming to the requirements of Section 2.3. Beam flange thickness shall be limited to a maximum of 2.5 in. (63 mm).

- (2) Beam depths shall be limited to W40 (W1000) for rolled shapes. Depth of built-up sections shall not exceed the depth permitted for rolled wide-flange shapes.
- (3) Beam depths shall be limited as follows for HSS shapes:
 - (a) For SMF systems, HSS10 (HSS 254) or smaller.
 - (b) For IMF systems, HSS12 (HSS 304.8) or smaller.
- (4) Beam weight is limited to 302 lb/ft (449 kg/m).
- (5) The ratio of the hinge-to-hinge span of the beam, L_h , to beam depth, d , shall be limited as follows:
 - (a) For SMF systems, L_h/d is limited to:
 - 6 or greater with rectangular shaped cover plates.
 - 4.5 or greater with U-shaped cover plates.
 - (b) For IMF systems, L_h/d is limited to 3 or greater.

The hinge-to-hinge span of the beam, L_h , is defined as the distance between the locations of plastic hinge formation at each moment-connected end of that beam. The location of plastic hinge shall be taken as one-third of the beam depth, $d/3$, away from the end of the side plate extension, as shown in Figure 11.5. Thus,

$$L_h = L - \frac{1}{2}(d_{c1} + d_{c2}) - 2(0.33 + 0.77)d \quad (11.3-1)$$

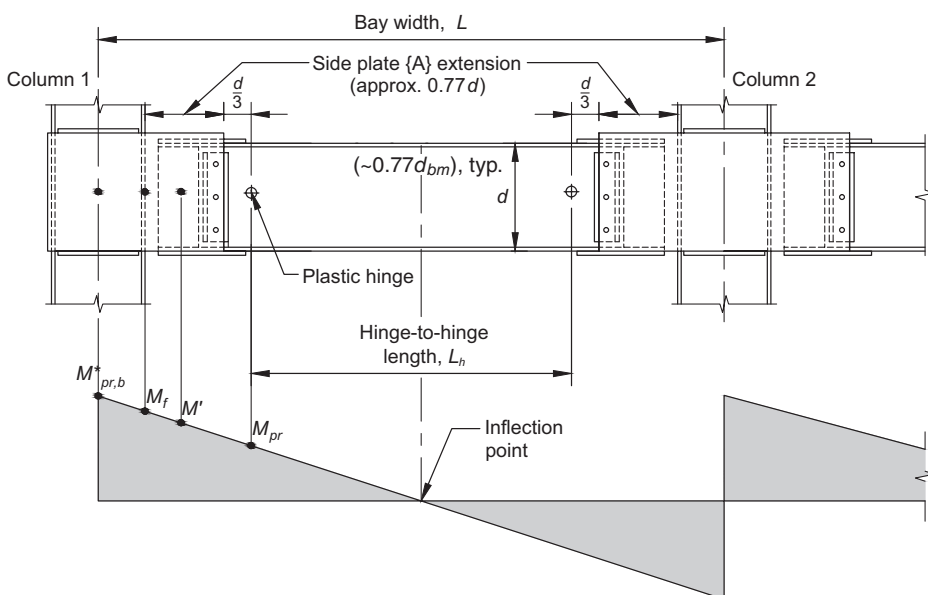


Fig. 11.5. Plastic hinge location and hinge-to-hinge length.

where

L = distance between column centerlines, in. (mm)

d_{c1}, d_{c2} = depth of column on each side of a bay in a moment frame, in. (mm)

User Note: The $0.33d$ constant represents the distance of the plastic hinge from the end of the side plate extension. The $0.77d$ constant represents the typical extension of the side plates from the face of column flange.

- (6) Width-to-thickness ratios for beam flanges and webs shall conform to the limits of the AISC *Seismic Provisions*.
- (7) Lateral bracing of wide-flange beams shall be provided in conformance with the AISC *Seismic Provisions*. Lateral bracing of HSS beams shall be provided in conformance with Appendix 1, Section 1.3.2c of the AISC *Specification*, taking $M'_1/M_2 = -1$ in AISC *Specification* Equation A-1-7. For either wide-flange or HSS beams, the segment of the beam connected to the side plates shall be considered to be braced. Supplemental top and bottom beam flange bracing at the expected hinge is not required.
- (8) The protected zone in the beam shall consist of the portion of the beam as shown in Figures 11.6 and 11.7.

2. Column Limitations

Columns shall satisfy the following limitations:

- (1) Columns shall be any of the rolled shapes, built-up I-shaped sections, flanged cruciform sections consisting of rolled shapes or built-up from plates or built-up box sections meeting the requirements of Section 2.3.
- (2) The beam shall be connected to the side plates that are connected to the flange tips of the column.
- (3) Rolled shape column depth shall be limited to W44 (W1100). The depth of built-up wide-flange columns shall not exceed that for rolled shapes. Flanged

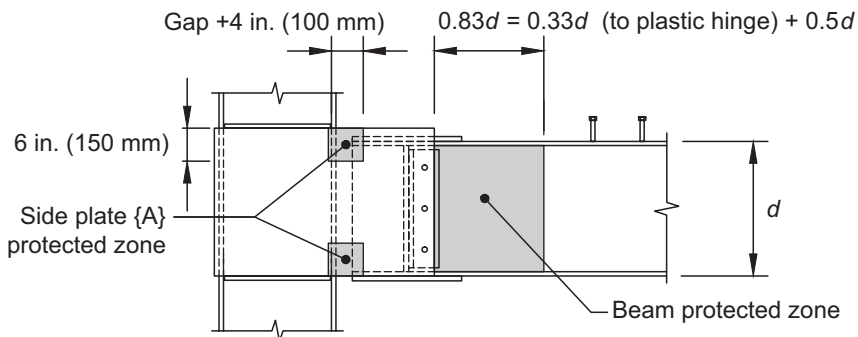


Fig. 11.6. Location of beam and side plate protected zones (one-sided connection shown).

cruciform columns shall not have a width or depth greater than the depth allowed for rolled shapes. Built-up box columns shall not have a width exceeding 24 in. (610 mm).

- (4) There is no limit on column weight per foot.
- (5) There are no additional requirements for column flange thickness.
- (6) Width-to-thickness ratios for the flanges and webs of columns shall conform to the requirements of the AISC *Seismic Provisions*.
- (7) Lateral bracing of columns shall conform to the requirements of the AISC *Seismic Provisions*.

3. Connection Limitations

The connection shall satisfy the following limitations:

- (1) All connection steel plates, which consist of side plates, cover plates, horizontal shear plates, and vertical shear elements, must be fabricated from structural steel that complies with ASTM A572/A572M Grade 50 (Grade 345).

Exception: The vertical shear element as defined in Section 11.6 may be fabricated using ASTM A36/A36M material.

- (2) The extension of the side plates beyond the face of the column shall be within the range of $0.65d$ to $1.0d$, where d is the nominal depth of the beam.
- (3) The protected zone in the side plates shall consist of a portion of each side plate that is 6 in. (150 mm) high by a length of the gap distance plus 4 in. (100 mm) long, centered at the gap region along the top and bottom edges of each side plate (Figures 11.6 and 11.7).

11.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

Beam-to-column connections shall satisfy the following limitations:

- (1) Beam flange width and thickness for rolled shapes shall satisfy the following equations for geometric compatibility (see Figure 11.8):

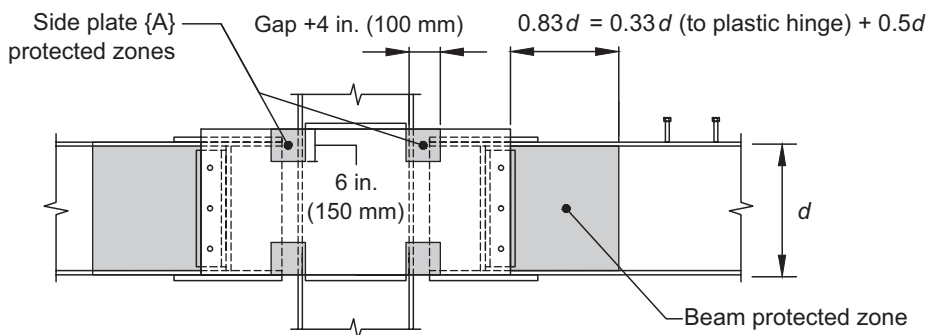


Fig. 11.7. Location of beam and side plate protected zones (two-sided connection shown).

$$b_{bf} + 1.1t_{bf} + \frac{1}{2} \text{ in.} \leq b_{cf} \quad (11.4-1)$$

$$b_{bf} + 1.1t_{bf} + 12 \text{ mm} \leq b_{cf} \quad (11.4-1M)$$

where

b_{bf} = width of beam flange, in. (mm)

b_{cf} = width of column flange, in. (mm)

t_{bf} = thickness of beam flange, in. (mm)

- (2) Panel zones shall conform to the applicable requirements of the AISC *Seismic Provisions*.

User Note: The column web panel zone strength shall be determined by Section J10.6a of the AISC *Specification*.

- (3) Column-beam moment ratios shall be limited as follows:
- (a) For SMF systems, the column-beam moment ratio shall conform to the requirements of the AISC *Seismic Provisions* as follows:
- (1) The value of $\sum M_{pb}^*$ shall be the sum of the projections of the expected flexural strengths of the beam(s) at the plastic hinge locations to the column centerline (Figure 11.9). The expected flexural strength of the beam shall be computed as:

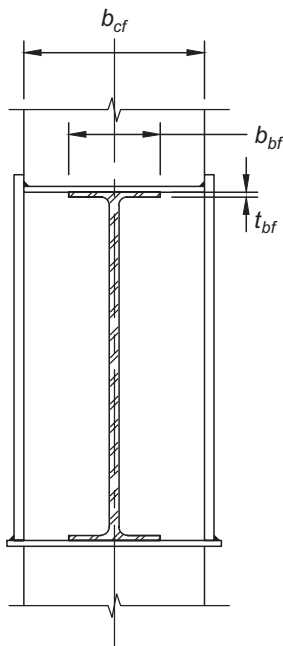


Fig. 11.8. Geometric compatibility.

$$\sum M_{pb}^* = \sum (1.1R_y F_{yb} Z_b + M_v) \quad (11.4-2)$$

where

F_{yb} = specified minimum yield stress of beam, ksi (MPa)

M_v = additional moment due to shear amplification from the center of the plastic hinge to the centerline of the column. M_v shall be computed as the quantity $V_h s_h$, where V_h is the shear at the point of theoretical plastic hinging, computed in accordance with Equation 11.4-3, and s_h is the distance of the assumed point of plastic hinging to the column centerline, which is equal to half the depth of the column plus the extension of the side plates beyond the face of column plus the distance from the end of the side plates to the plastic hinge, $d/3$.

$$V_h = \frac{2M_{pr}}{L_h} + V_{gravity} \quad (11.4-3)$$

where

L_h = distance between plastic hinge locations, in. (mm)

M_{pr} = probable maximum moment at plastic hinge, kip-in. (N-mm)

$V_{gravity}$ = beam shear force resulting from $1.2D + f_1 L + 0.2S$ (where f_1 is the load factor determined by the applicable building code for live loads, but not less than 0.5), kips (N)

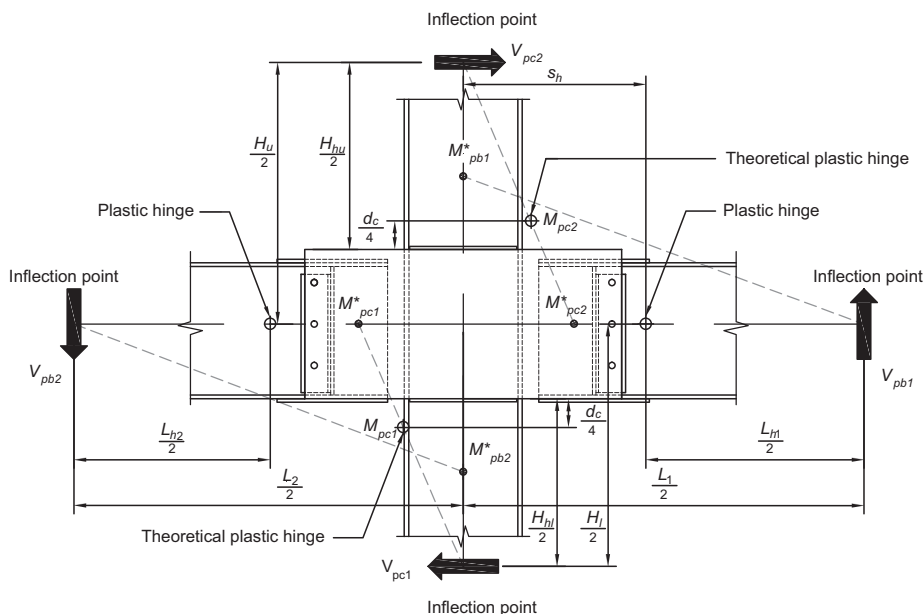


Fig. 11.9. Force and distance designations for computation of column-beam moment ratios.

- R_y = ratio of expected yield stress to specified minimum yield stress F_y as specified in the AISC *Seismic Provisions*
- Z_b = nominal plastic section modulus of beam, in.³ (mm³)

User Note: The load combination of $1.2D + f_1L + 0.2S$ is in conformance with ASCE/SEI 7-16. When using the International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.2 for S (snow) when the roof configuration is such that it does not shed snow off the structure.

- (2) The value of $\sum M_{pc}^*$ shall be the sum of the projections of the nominal flexural strengths (M_{pc}) of the column above and below the connection joint, at the location of theoretical hinge formation in the column (i.e., one quarter the column depth above and below the extreme fibers of the side plates), to the beam centerline, with a reduction for the axial force in the column (Figure 11.9). The nominal flexural strength of the column shall be computed as:

$$\sum M_{pc}^* = \sum Z_{ec} (F_{yc} - P_{uc}/A_g) \quad (11.4-4)$$

where

- F_{yc} = the minimum specified yield strength of the column at the connection, ksi (MPa)
- H = story height, in. (mm)
- H_h = distance along column height from $1/4$ of column depth above top edge of lower story side plates to $1/4$ of column depth below bottom edge of upper story side plates, in. (mm)
- P_{uc}/A_g = ratio of column axial compressive load, computed in accordance with load and resistance factor provisions, to gross area of the column, ksi (MPa)
- Z_c = plastic section modulus of column, in.³ (mm³)
- Z_{ec} = the equivalent plastic section modulus of column (Z_c) at a distance of $1/4$ column depth from top and bottom edge of side plates, projected to beam centerline, in.³ (mm³), and computed as:

$$Z_{ec} = \frac{Z_c (H/2)}{H_h/2} = \frac{Z_c H}{H_h} \quad (11.4-5)$$

- (b) For IMF systems, the column-beam moment ratio shall conform to the requirements of the AISC *Seismic Provisions*.

11.5. CONNECTION WELDING LIMITATIONS

Filler metals for the welding of beams, columns and plates in the SidePlate connection shall meet the requirements for seismic force-resisting system welds in the AISC *Seismic Provisions*.

User Note: Mechanical properties for filler metals for seismic force-resisting system welds are detailed in AWS D1.8/D1.8M as referenced in the AISC *Seismic Provisions*.

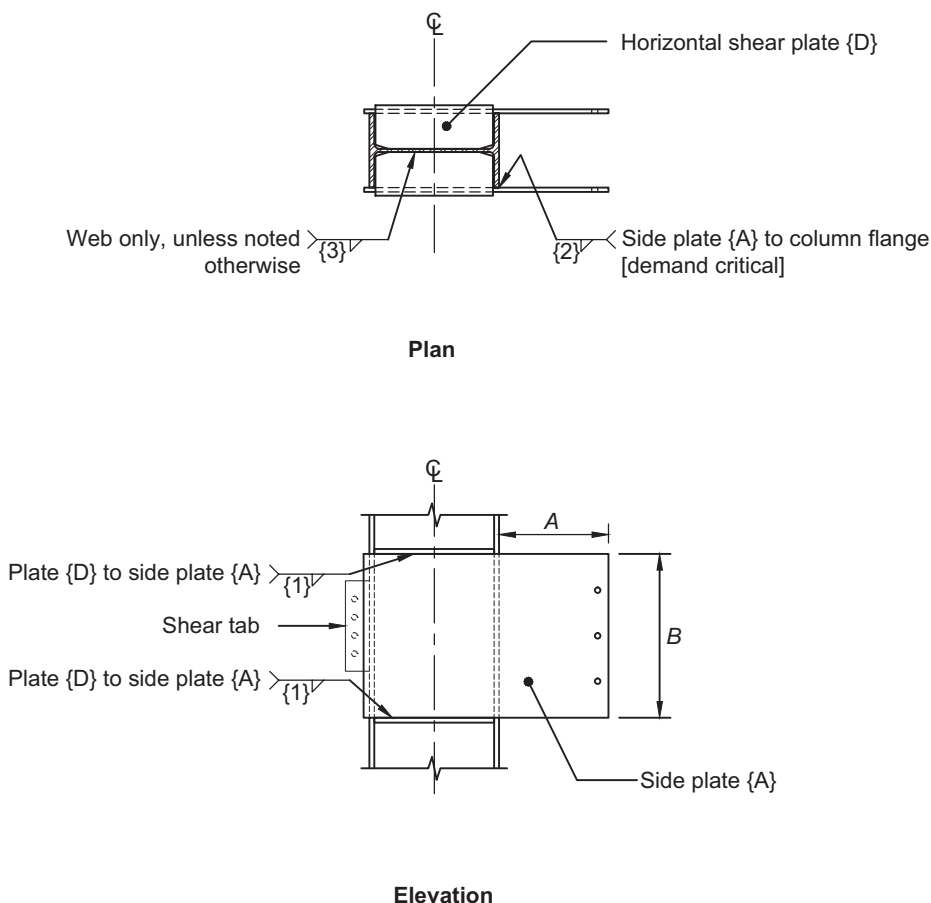
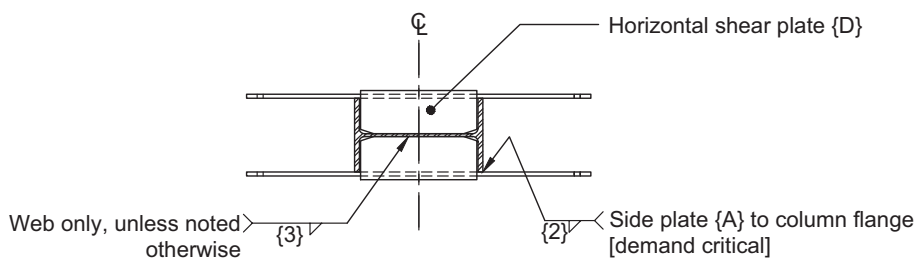


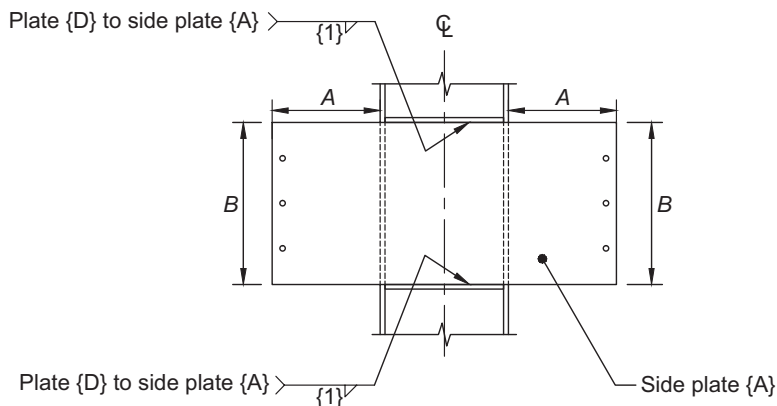
Fig. 11.10. One-sided SidePlate moment connection (A-type), column shop detail.

The following welds are considered demand critical welds:

- (1) Shop fillet weld {2} that connects the inside face of the side plates to the column (see plan views in Figures 11.10, 11.11 and 11.12) and for biaxial dual-strong axis configurations connects the outside face of the secondary side plates to the outside face of primary side plates (see Figure 11.3).
- (2) Shop fillet weld {5} that connects the edge of the beam flange to the beam flange cover plate (see Figure 11.13).
- (3) Shop fillet weld {5a} that connects the outside face of the beam flange to the beam flange U-shaped cover plate (see Figure 11.13).
- (4) Field fillet weld {7} that connects the beam flange cover plates to the side plates (see Figure 11.14), or connects the HSS flange to the side plates.



Plan view



Elevation

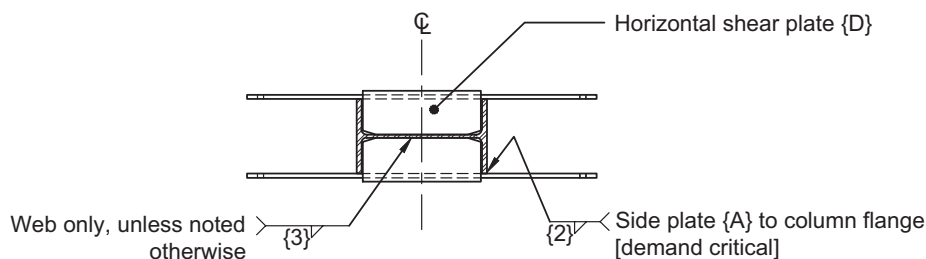
Fig. 11.11. Two-sided SidePlate moment connection (B-type), column shop detail.

11.6. CONNECTION DETAILING

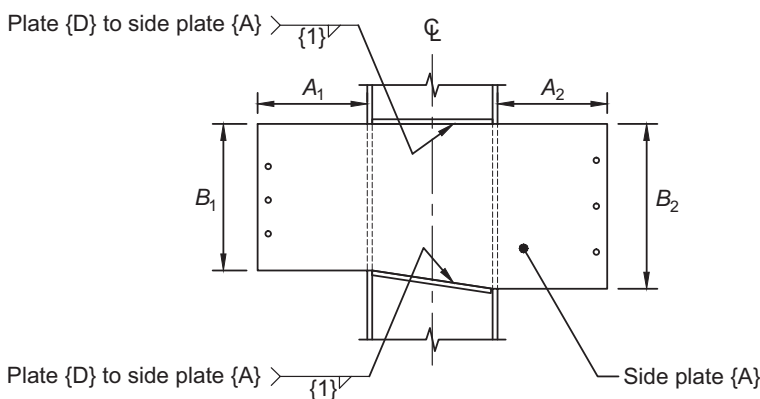
The following designations are used herein to identify plates and welds in the Side-Plate connection shown in Figures 11.10 through 11.15:

1. Plates

{A} Side plate, located in a vertical plane parallel to the web(s) of the beam, connecting frame beam to column.



Plan



Elevation

Fig. 11.12. Two-sided SidePlate moment connection (C-type), column shop detail.

- {B} Beam flange cover plate bridging between side plates {A}, as applicable.
- {C} Vertical shear plate.
- {D} Horizontal shear plate (HSP). This element transfers horizontal shear from the top and bottom edges of the side plates {A} to the web of a wide-flange column.
- {E} Erection angle. One of the possible vertical shear elements {F}.
- {F} Vertical shear elements (VSE). These elements, which may consist of angles and plates or bent plates, transfer shear from the beam web to the outboard edge of the side plates {A}.

2. Welds

- {1} Shop fillet weld connecting exterior edge of side plate {A} to the horizontal shear plate {D} or to the web of built-up box column.
- {2} Shop fillet weld connecting inside face of side plate {A} to the tip of the column flange, and for biaxial dual-strong axis configurations connects outside face of secondary side plates to outside face of primary side plates.
- {3} Shop fillet weld connecting horizontal shear plate {D} to wide-flange column web. Weld {3} is also used at the column flanges where required to resist orthogonal loads through the connection due to collectors, chords or cantilevers.

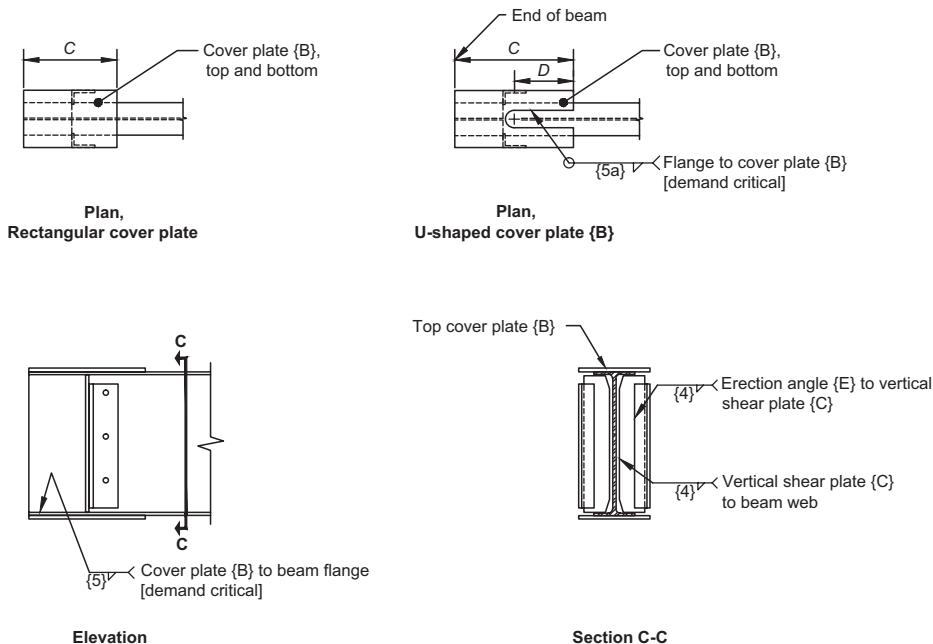
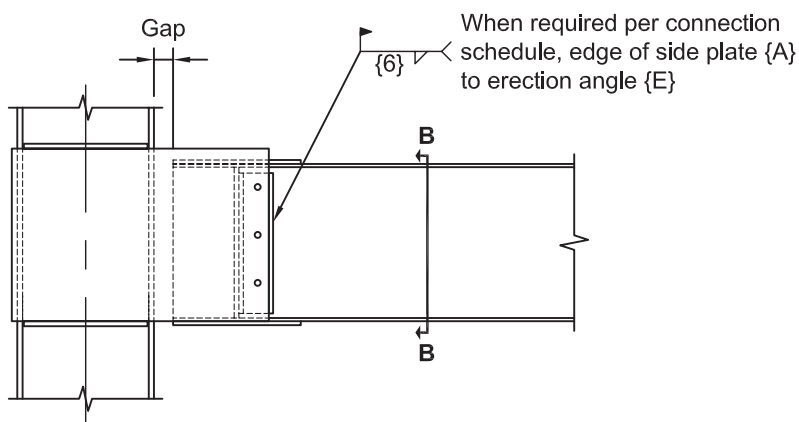
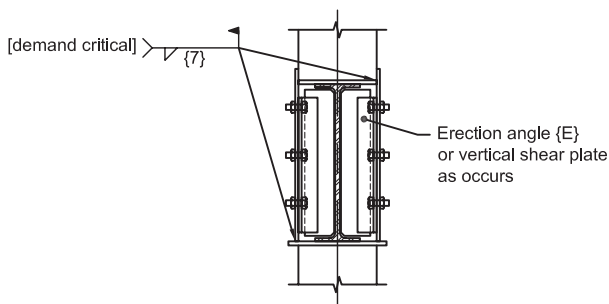


Fig. 11.13. Full-length beam shop detail.

- {4} Shop fillet weld connecting vertical shear elements {F} to the beam web, and where applicable, the vertical shear plate {C} to the erection angle {E}.
- {5} Shop fillet weld connecting beam flange tip to cover plate {B}.
- {5a} Shop fillet weld connecting outside face of beam flange to cover plate {B} U-shaped slot.
- {6} Field vertical fillet weld connecting vertical shear element (angle or bent plate) {F} to end of side plate {A}.
- {7} Field horizontal fillet weld connecting the cover plate {B} to the side plate {A}, or connects HSS flange to side plates.



Elevation



Section B-B

Fig. 11.14. Full-length beam-to-side plate field erection detail.

Figure 11.10 shows the connection detailing for a one-sided moment connection configuration in which one beam frames into a column (A-type). Figure 11.11 shows the connection detailing for a two-sided moment connection configuration in which the beams are identical (B-type). Figure 11.12 shows the connection detailing for a two-sided moment connection configuration in which the beams differ in depth (C-type). Figure 11.13 shows the full-length beam assembly shop detail. Figure 11.14 shows the full-length beam-to-side-plate field erection detail. If two beams frame into a column to form a corner, the connection detailing is referred to as a D-type (not shown). The connection detailing for a three-sided and four-sided moment connection configuration is referred to as an E-type and F-series, respectively (not shown). Figure 11.15 shows the link beam-to-beam stub splice detail used with the original SidePlate configuration.

11.7. DESIGN PROCEDURE

Step 1. Choose trial frame beam and column section combinations that satisfy geometric compatibility based on Equation 11.4-1 or 11.4-1M. For SMF systems, check that the section combinations satisfy the preliminary column-beam moment ratio given by:

$$\sum(F_{yc}Z_{xc}) > 1.7 \sum(F_{yb}Z_{xb}) \quad (11.7-1)$$

where

F_{yb} = specified minimum yield stress of beam, ksi (MPa)

F_{yc} = specified minimum yield stress of column, ksi (MPa)

Z_{xb} = plastic section modulus of beam, in.³ (mm³)

Z_{xc} = plastic section modulus of column, in.³ (mm³)

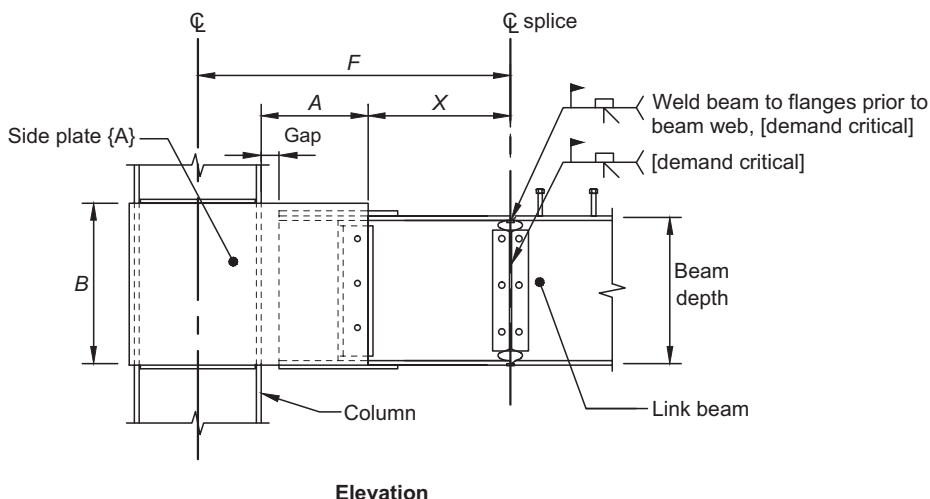


Fig. 11.15. Link-beam erection method detail.

Step 2. Approximate the effects on global frame performance of the increase in lateral stiffness and strength of the SidePlate moment connection, due to beam hinge location and side plate stiffening, in the mathematical elastic steel frame computer model by using 100% rigid offset in the panel zone and by increasing the moment of inertia, elastic section modulus, and plastic section modulus of the beam to approximately three times that of the beam, for a distance of approximately 77% of the beam depth beyond the column face (approximately equal to the extension of the side plate beyond the face of the column), illustrated in Figure 11.16.

SMF beams that have a combination of shallow depth and heavy weight (i.e., beams with a relatively large flange area such as those found in the widest flange series of a particular nominal beam depth) require that the extension of the side plate {A} be increased, up to the nominal depth of the beam, d .

User Note: This increase in extension of side plate {A} lengthens fillet weld {7}, thus limiting the extremes in the size of fillet weld {7}. Regardless of the extension of the side plate {A}, the plastic hinge occurs at a distance of $d/3$ from the end of the side plates.

Step 3. Confirm that the frame beams and columns satisfy all applicable building code requirements, including, but not limited to, stress checks and design story drift checks.

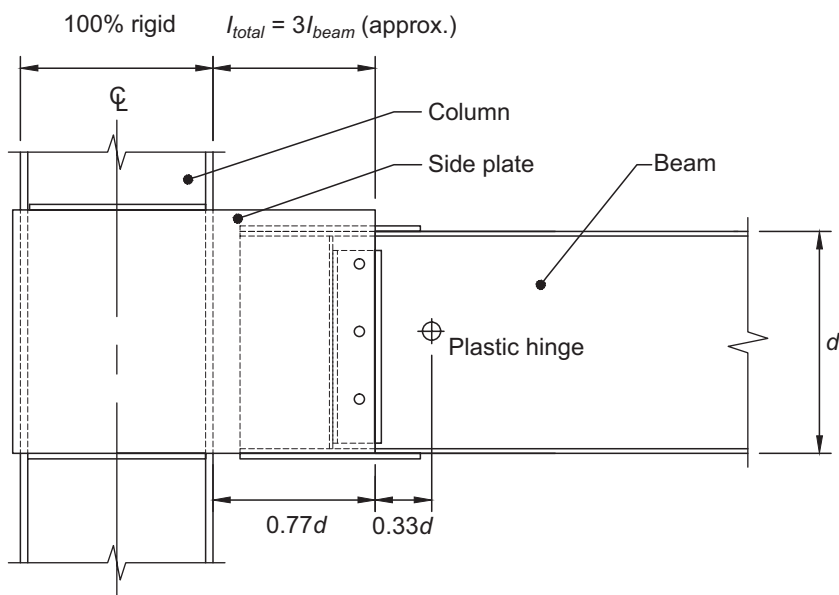


Fig. 11.16. Modeling of component stiffness for linear-elastic analysis.

Step 4. Confirm that the frame beam and column sizes comply with prequalification limitations per Section 11.3.

Step 5. Upon completion of the preliminary and/or final selection of lateral load resisting frame beam and column member sizes using SidePlate connection technology, the engineer of record submits a computer model to SidePlate Systems, Inc. In addition, the engineer of record shall submit the following additional information, as applicable:

$V_{gravity}$ = factored gravity shear in moment frame beam resulting from the load combination of $1.2D + f_1L + 0.2S$ (where f_1 is the load factor determined by the applicable building code for live loads, but not less than 0.5), kips (N)

User Note: The load combination of $1.2D + f_1L + 0.2S$ is in conformance with ASCE/SEI 7-16. When using the 2015 International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.2 for S (snow) when the roof configuration is such that it does not shed snow off of the structure.

- (a) Factored gravity shear loads, V_1 and/or V_2 , from gravity beams that are not in the plane of the moment frame, but connect to the exterior face of the side plate(s) where

V_1, V_2 = beam shear force resulting from the load combination of $1.2D + f_1L + 0.2S$ (where f_1 is the load factor determined by the applicable building code for live loads, but not less than 0.5), kips (N)

- (b) Factored gravity loads, M_{cant} and V_{cant} , from cantilever gravity beams that are not in the plane of the moment frame, but connect to the exterior face of the side plate(s) where

M_{cant} = cantilever beam moment resulting from code applicable load combinations, kip-in. (N-mm)

V_{cant} = cantilever beam shear force resulting from code applicable load combinations, kips (N)

User Note: Code applicable load combinations may need to include the following when looking at cantilever beams: $1.2D + f_1L + 0.2S$ and $(1.2 + 0.2S_{DS})D + \rho Q_E + f_1L + 0.2S$, which are in conformance with ASCE/SEI 7-16. When using the 2015 International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.2 for S (snow) when the roof configuration is such that it does not shed snow off of the structure.

- (c) Perpendicular amplified seismic lateral drag or chord axial forces, A_{\perp} , transferred through the SidePlate connection.

A_{\perp} = amplified seismic drag or chord force resulting from the applicable building code, kips (N)

User Note: Where linear-elastic analysis is used to determine perpendicular collector or chord forces used to design the SidePlate connection, such forces should include the applicable load combinations specified by the building code, including considering the amplified seismic load (Ω_o). Where nonlinear analysis or capacity design is used, collector or chord forces determined from the analysis are used directly, without consideration of additional amplified seismic load.

- (d) In-plane factored lateral drag or chord axial forces, A_{\parallel} , transferred along the frame beam through the SidePlate connection.

A_{\parallel} = amplified seismic drag or chord force resulting from applicable building code, kips (N)

Step 6. Upon completion of the mathematical model review and after additional information has been supplied by the engineer of record, SidePlate engineers provide project-specific connection designs. Strength demands used for the design of critical load transfer elements (plates, welds and column) throughout the SidePlate beam-to-column connection and the column are determined by superimposing maximum probable moment, M_{pr} , at the known beam hinge location, then amplifying the moment demand to each critical design section, based on the span geometry, as shown in Figure 11.5, and including additional moment due to gravity loads. For each of the design elements of the connection, the moment demand is computed per Equation 11.7-2 and the associated shear demand is computed as:

$$M_{group} = M_{pr} + V_u x \quad (11.7-2)$$

where

C_{pr} = connection-specific factor to account for peak connection strength, including strain hardening, local restraint, additional reinforcement, and other connection conditions. The equation used in the calculation of the C_{pr} is provided by SidePlate as part of the connection design.

User Note: In practice, the value of C_{pr} for SidePlate connections as determined from testing and nonlinear analysis ranges from 1.15 to 1.35.

F_y = specified minimum yield stress of yielding element, ksi (MPa)

L_h = distance between plastic hinge locations, in. (mm)

M_{group} = maximum probable moment demand at any connection element, kip-in. (N-mm)

M_{pr} = maximum probable moment at plastic hinge per Section 2.4.3, kip-in. (N-mm), computed as:

$$M_{pr} = C_{pr} R_y F_y Z_x \quad (11.7-3)$$

- R_y = ratio of expected yield stress to specified minimum yield stress, F_y
- $V_{gravity}$ = gravity beam shear resulting from $1.2D + f_1L + 0.2S$ (where f_1 is the load factor determined by the applicable building code for live loads, but not less than 0.5), kips (N)
- V_u = maximum shear demand from probable maximum moment and factored gravity loads, kips (N), computed as:

$$V_u = \frac{2M_{pr}}{L_h} + V_{gravity} \quad (11.7-4)$$

- Z_x = plastic section modulus of beam about x -axis, in.³ (mm³)
- x = distance from plastic hinge location to centroid of connection element, in. (mm)

Step 7. SidePlate designs all connection elements per the proprietary connection design procedures contained in SidePlate FRAME Connection Design Software (version 5.2, revised January 2013). The version is clearly indicated on each page of calculations. The final design includes structural notes and details for the connections.

User Note: The procedure uses an ultimate strength design approach to size plates and welds, incorporating strength, plasticity and fracture limits. For welds, an ultimate strength analysis incorporating the instantaneous center of rotation may be used as described in the AISC *Steel Construction Manual*. Refer to the Commentary for an in-depth discussion of the process.

In addition to the column web panel zone strength requirements, the column web shear strength shall be sufficient to resist the shear loads transferred at the top and bottom of the side plates. The design shear strength of the column web shall be determined in accordance with AISC *Specification* Section G2.1.

Step 8. Engineer of record reviews SidePlate calculations and drawings to ensure that all project specific connection designs have been appropriately designed and detailed based on information provided in Step 5.

CHAPTER 12

SIMPSON STRONG-TIE STRONG FRAME MOMENT CONNECTION

The user's attention is called to the fact that compliance with this chapter of the standard requires use of an invention covered by patent rights. By publication of this standard, no position is taken with respect to the validity of any claim(s) or of any patent rights in connection therewith. The patent holder has filed a statement of willingness to grant a license under these rights on reasonable and nondiscriminatory terms and conditions to applicants desiring to obtain such a license. The statement may be obtained from the standard's developer.*

12.1. GENERAL

The Simpson Strong-Tie® Strong Frame® moment connection is a partially restrained (Type PR) connection that uses a modified shear plate connection (single-plate shear connection) for shear transfer and a modified T-stub connection (the Yield-Link™ structural fuse) for moment transfer, as shown in Figure 12.1. The shear plate utilizes a three-bolt connection wherein the upper and lower bolt holes in the shear plate are horizontal slots and the center bolt hole is a standard hole. Matching holes in the beam web are all standard holes. This prevents moment transfer through the shear plate connection. While all shear plate bolts participate in shear resistance, the center bolt is designed to also resist the axial force in the beam at the connection. The modified T-stub connections, which bolt to both the beam flange and column flange, are configured as yielding links and contain a reduced yielding area in the stem of the link that is prevented from buckling in compression via a separate buckling restraint plate. The connection is based on a capacity-based design approach, wherein connection response remains elastic under factored load combinations, and seismic inelastic rotation demand is confined predominantly within the connection with little, if any, inelastic behavior expected from the members.

12.2. SYSTEMS

The Simpson Strong-Tie connection is prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems within the limits of these provisions.

12.3. PREQUALIFICATION LIMITS

1. Beam Limitations

Beams shall satisfy the following limitations:

* The proprietary design of the Yield-Link structural fuse and its use in moment-resisting connections is protected under U.S. Pat. Nos.: 8,375,652; 8,001,734; 8,763,310; Japan Pat. No.: 5398980; and China Pat. No.: ZL200710301531.4. Other U.S and foreign patent protection are pending.

- (1) Beams shall be rolled wide-flange or welded built-up I-shaped members
- (2) Beam depth is limited to a maximum of W16 (W410) for rolled shapes. Beam depth for built-up members shall not exceed the maximum depth of the permitted W16 (W410) shapes.
- (3) There are no limits on the beam web width-to-thickness ratio beyond those listed in the AISC *Specification*. The beam flange width-to-thickness ratio shall not exceed λ_r per Table B4.1b of the AISC *Specification*, and flange thickness shall not be less than 0.40 in. (10 mm).
- (4) Lateral bracing of beams and joints: there are no requirements for stability bracing of beams or joints beyond those in the AISC *Specification*.
- (5) The protected zone shall consist of the Yield-Links, the shear plate, and the portions of the beam in contact with the Yield-Links and shear plate.

User Note: Limits on beam weight and span-to-depth ratio are not required for the SST moment connection because plastic hinging in the connection occurs solely within the Yield-Links. Span-to-depth ratio is typically limited to control moment gradient and beam shear, both of which are limited by the shear plate connection within the design procedure.

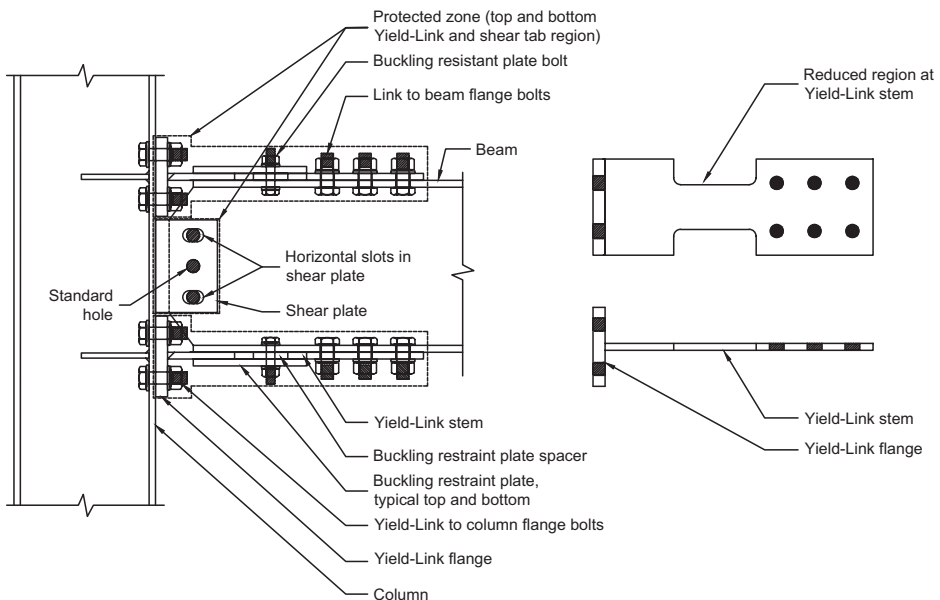


Fig. 12.1. Simpson Strong-Tie Strong Frame moment connection.

2. Column Limitations

Columns shall satisfy the following limitations:

- (1) Columns shall be any of the rolled or built-up I-shaped members permitted in Section 2.3.
- (2) The beam shall be connected to the flange of the column.
- (3) Column depth is limited to a maximum of W18 (W460) for rolled shapes. Column depth for built-up members shall not exceed the maximum depth permitted for W18 (W460) shapes.
- (4) There is no limit on the weight per foot of columns.
- (5) There are no additional requirements for flange thickness.
- (6) Column width-to-thickness ratios shall comply with the following:
 - (a) Where column-to-foundation connections are designed to restrain column end rotation, column width-to-thickness ratios shall comply with AISC 341 Table D1.1 for highly ductile members within the first story.
 - (b) At other locations and for other conditions, column width-to-thickness ratios shall comply with the AISC *Specification*.
- (7) Lateral bracing of columns shall be provided in accordance with the AISC *Seismic Provisions*.

Exception: When columns are designed in accordance with Section 12.9 and maximum nominal flexural strength, M_n , outside the panel zone is limited such that $M_n \leq F_y S_x$, it is permitted that bracing be provided at the level of the top flange of the beam only.

3. Bolting Limitations

Bolts shall conform to the requirements of Chapter 4.

Exceptions:

- (1) The following connections shall be made with ASTM F3125 Grade A325 or A325M bolts installed either as snug-tight or pretensioned, except as noted. It shall be permitted to use ASTM F1852 bolts for pretensioned applications.
 - (a) Yield-Link flange-to-column flange bolts
 - (b) Buckling restraint plate bolts installed snug tight
 - (c) Shear-plate bolts
- (2) The Yield-Link stem-to-beam flange bolts shall be pretensioned ASTM F3125 Grade A325, A325M, A490, A490M, F1852 or F2280 bolt assemblies. Faying surface preparation between the Yield-Link stem and beam flange shall not be required, but faying surfaces shall not be painted.

12.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

Beam connection-to-column connections shall satisfy the following limitations:

- (1) Panel zones shall conform to the requirements of the AISC *Specification*.
- (2) Column-beam connection moment ratios shall be limited as follows:
 - (a) For SMF systems, the column-beam connection moment ratio shall conform to the requirements of the AISC *Seismic Provisions*. The value of ΣM_{pb}^* shall be taken equal to $\Sigma (M_{pr} + M_{iw})$, where M_{pr} is computed according to Equation 12.9-16, and where M_{iw} is the additional moment due to shear amplification from the center bolt in the shear plate to the centerline of the column. M_{iw} is computed as $V_u(a + d_c/2)$, where V_u is the shear at the shear-plate connection computed per Step 12 of Section 12.9, a is the distance from the centerline of the shear-plate bolts to the face of the column as shown in Figure 12.3c, and d_c is the depth of the column.
 - (b) For IMF systems, the column-beam moment ratio shall conform to the requirements of the AISC *Seismic Provisions*.

12.5. CONTINUITY PLATES

Continuity plates shall satisfy the following limitations:

- (1) The need for continuity plates shall be determined in accordance with Section 12.9.
- (2) Where required, design of continuity plates shall be in accordance with the AISC *Specification*.
- (3) Continuity plates may be welded to the column flange and column web with fillet welds.

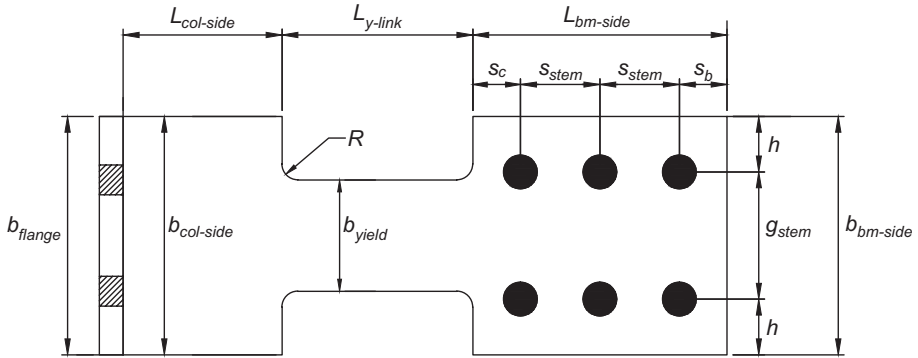
12.6. YIELD-LINK FLANGE-TO-STEM WELD LIMITATIONS

Yield-Link flange-to-stem connections may be CJP groove welds or double-sided fillet welds.

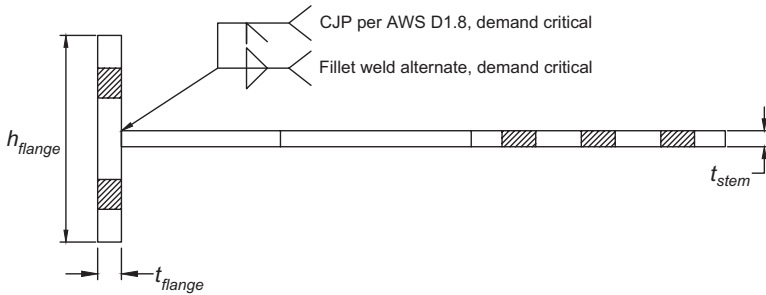
- (1) CJP groove welds shall conform to the requirements of demand critical welds in the AISC *Seismic Provisions*.
- (2) Double-sided fillet welds shall be designed to develop the tensile strength of the unreduced Yield-Link stem at the column side, $b_{col-side}$, and shall be demand critical.

12.7. FABRICATION OF YIELD-LINK CUTS

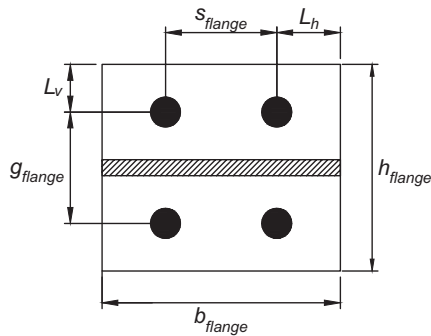
The reduced section of the Yield-Link shall be cut using the following methods: laser, plasma, or water-jet method. Maximum roughness of the cut surface shall be 250 μ -in. (6.5 microns) in accordance with ASME B46.1. All transitions between the reduced section of the Yield-Link, and the nonreduced sections of the Yield-Link shall utilize a smooth radius, R , as shown in Figure 12.2a, where $R = \frac{1}{2}$ in. (12 mm).



(a) Yield-Link plan view



(b) Yield-Link elevation view



(c) Yield-Link flange view

Fig. 12.2. Yield-Link geometries.

Cutting tolerance at the reduced section shall be plus or minus $\frac{1}{16}$ in. (2 mm) from the theoretical cut line.

12.8. CONNECTION DETAILING

1. Beam Coping

Beams shall be coped in accordance with Figure 12.3(a).

2. Yield-Links

Yield-Links shall conform to the requirements of Figures 12.2 and 12.3, and shall be welded from ASTM A572 Grade 50 material or cut from rolled sections conforming to the ASTM A992 or ASTM A913 Grade 50 specification. Yield-Link stem thickness shall be 0.50 in. (13 mm), with a thickness tolerance of plus 0.03 in. (0.8 mm) and minus 0.01 in. (0.25 mm). Yield-Link flange edge distances, L_v , and L_{h_s} , shall conform to AISC *Specification* Tables J3.4 or J3.4M.

3. Shear Plate Connection Bolts

The shear-plate connection shall include a single column of three bolts as shown in Figure 12.1. Top and bottom holes in the shear-plate shall be slotted per Section 12.8.5(b).

4. Shear-Plate Shear Connection Welds

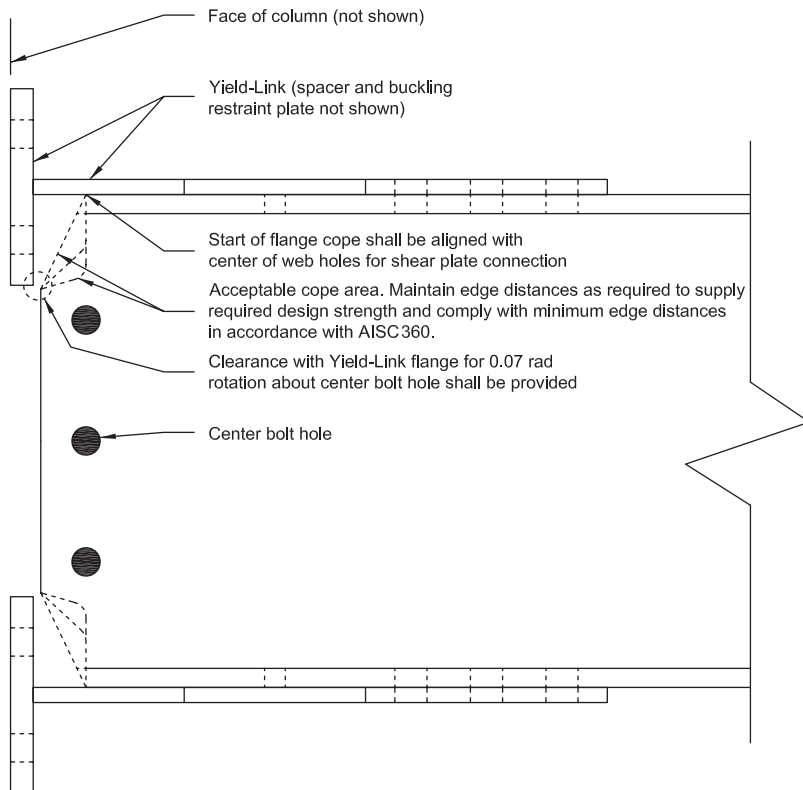
The single-shear plate connection shall be welded to the column flange. The single shear plate to column flange weld may consist of double-sided fillet welds, PJP welds, or CJP welds, sized in accordance with Section 12.9, Step 15.4.

5. Bolt Hole Requirements

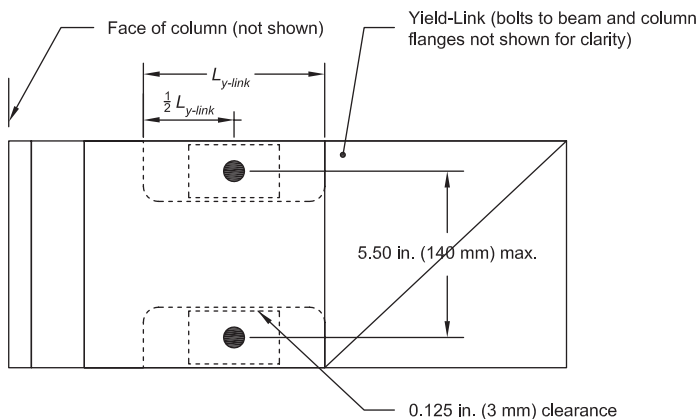
- (a) Standard bolt holes shall be provided in the beam flanges and beam webs. Oversized holes or vertical slots are permitted in the column flanges.
- (b) The top and bottom holes in the shear plate shall be slots to accommodate a connection rotation of at least 0.07 rad. The center hole in the shear plate shall be a standard hole.
- (c) Bolt holes in the Yield-Link stem and beam flange shall be drilled, sub-punched and reamed, laser cut, plasma cut, or cut by water-jet. Bolt hole surface roughness shall be per AISC *Specification* requirements.

6. Buckling Restraint Assembly

The buckling restraint assembly consists of the buckling restraint plate, the buckling restraint spacer plate, and the buckling restraint bolts and shall conform to the requirements of Figure 12.3. The buckling restraint plate shall be 0.875 in. (22 mm) thick, with a specified minimum yield stress $F_y \geq 50$ ksi (345 MPa) and shall extend to the end of the cut region on the Yield-Link plate. The buckling restraint spacer plate shall have the same thickness at the Yield-Link stem, with a specified minimum yield

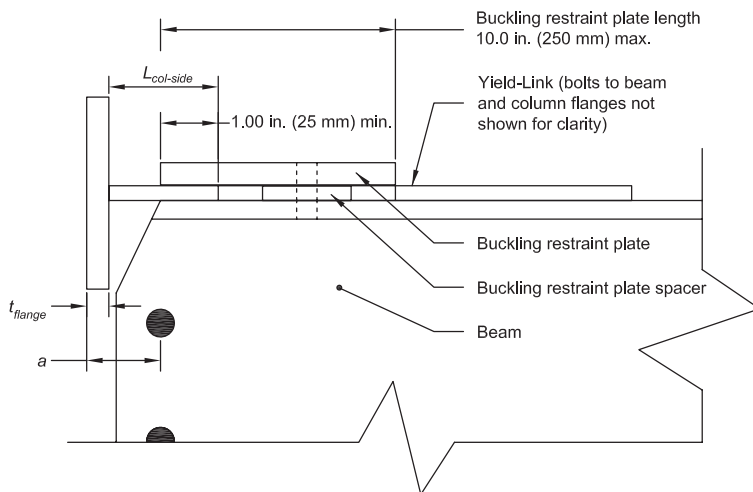


(a) Beam coping

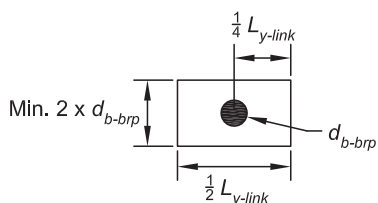


(b) Buckling restraint spacer plate placement

Fig. 12.3. Connection detailing.



(c) Buckling restraint plate and Yield-Link $L_{col-side}$ limitations



(d) Buckling restraint spacer plate dimensions.

Fig. 12.3. Connection detailing.

stress $F_y \geq 36$ ksi (250 MPa). Buckling restraint bolts shall have a minimum diameter of 0.625 in. (16 mm) and a maximum diameter of 0.75 in. (20 mm).

7. Shims

The use of finger shims at the Yield-Link flange-to-column flange is permitted, subject to the limitations of the RCSC *Specification*.

12.9. DESIGN PROCEDURE

Step 1. Choose trial values for the beam sections and column sections subject to the prequalification limits of Section 12.3 assuming fully restrained beam-to-column connections and all load combinations specified by the applicable building code. Estimate the design story drift for compliance with the applicable limits specified by the applicable building code as 1.2 times larger than the value calculated assuming fully restrained connections.

Step 2. Check the strength and deflection of the beam assuming the beam is simply supported between shear-plate connections. Check beam strength for the applicable vertical load combinations of the applicable building code. Check that the deflection of the beam under dead and live loads is less than $L_h/360$, where L_h is the length of the beam between the center of the shear plate bolts at each end of the beam.

User Note: The deflection check serves to estimate beam stiffness needed to limit member end rotations. Other values may be acceptable.

Step 3. Estimate the required Yield-Link yield strength from Step 1.

$$P'_{y-link} = M_u / (\phi_b d) \quad (12.9-1)$$

$$A'_{y-link} = P'_{y-link} / F_{y-link} \quad (12.9-2)$$

where

A'_{y-link} = estimated required Yield-Link yield area, in.² (mm²)

F_{y-link} = specified minimum yield stress of Yield-Link stem material, ksi (MPa)

M_u = moment demand from elastic analysis assuming fully restrained connections, kip-in. (N-mm)

P'_{y-link} = estimated required Yield-Link yield force, kips (N)

d = depth of beam, in. (mm)

ϕ_b = 0.90

Step 4. Determine the nonreduced width and length of the Yield-Link at column side. See Figure 12.2(a).

Step 4.1. Determine nonreduced Yield-Link stem widths, $b_{col-side}$ and $b_{bm-side}$.

User Note: Try setting $b_{col-side}$ and $b_{bm-side}$ equal to the minimum beam flange width and column flange width, respectively.

Step 4.2. Nonreduced Yield-Link stem length at column side, $L_{col-side}$, shall have maximum length equal to 5 in. (127 mm) and a minimum length equal to $a - t_{flange} + 1$ in. ($a - t_{flange} + 25$ mm). See Figure 12.3(c).

Step 5. Determine the width of the yielding section of the Yield-Link stem, b_{yield} , where the thickness of the Yield-Link stem, t_{stem} , shall be taken as $\frac{1}{2}$ in. (13 mm).

$$b_{yield,req'd} \geq A'_{y-link} / t_{stem} \quad (12.9-3)$$

The value of $b_{yield,req'd}$ shall not exceed the least of $0.5b_{col-side}$, $0.5b_{bm-side}$, or $3\frac{1}{2}$ in. (88 mm).

Step 6. Determine the minimum yielding length of the Yield-Link stem, L_{y-link} , such that the axial strain in the straight portion of the Yield-Link is less than or equal to 0.085 in./in. at 0.05 rad of connection rotation.

$$L_{y-link} = \frac{0.05}{0.085} \left(\frac{d + t_{stem}}{2} \right) + 2R \quad (12.9-4)$$

where R , the radius between the reduced width and the nonreduced width at the beam and column sides, is taken as $\frac{1}{2}$ in. (13 mm).

Step 7. Compute the expected yield strength and probable maximum tensile strength of the Yield-Link.

$$P_{ye-link} = A_{y-link} R_y F_{y-link} \quad (12.9-5)$$

$$P_{r-link} = A_{y-link} R_t F_{u-link} \quad (12.9-6)$$

where

A_{y-link} = area of reduced Yield-Link section (b_{yield})(t_{stem}), in.² (mm²)

F_{u-link} = specified minimum tensile strength of Yield-Link stem material, ksi (MPa)

R_t = ratio of expected tensile strength to specified minimum tensile strength, F_u , as related to overstrength in material yield stress, R_y ; taken as 1.2 for Yield-Link stem material

R_y = ratio of the expected yield stress to specified minimum yield stress, F_y ; taken as 1.1 for Yield-Link stem material

Step 8. Determine the nonreduced width, $b_{bm-side}$, and length, $L_{bm-side}$, at beam side of the Yield-Link using P_{r-link} from Step 7.

Step 8.1. Design bolts for shear transfer between the Yield-Link stem and the beam flange per the AISC *Specification* and determine bolt diameter, d_{b-stem} .

Step 8.2. Determine the nonreduced width of the Yield-Link stem on the beam side, $b_{bm-side}$.

User Note: Try setting $b_{bm-side}$ equal to $b_{col-side}$ from Step 4.1.

Step 8.3. Determine the nonreduced length of the Yield-Link stem at beam side, $L_{bm-side}$.

$$L_{bm-side} = s_c + [(n_{rows} - 1) \times s_{stem}] + s_b \quad (12.9-7)$$

where

n_{rows} = number of rows of bolts from Step 8.1.

s_b = distance from center of last row of bolts to beam-side end of Yield-Link stem, from Table J3.4 of the AISC *Specification*, in. (mm)

s_c = distance from reduced section of Yield-Link to center of first row of bolts, equal to $1.5d_{b-stem}$, in. (mm)

s_{stem} = spacing between rows of bolts for Yield-Link stem to beam flange connection, minimum $2\frac{2}{3}d_{b-stem}$, in. (mm)

Step 8.4. Check the Yield-Link stem at the beam side for tensile yielding, tensile rupture, block shear rupture, and bolt bearing (where deformation at the hole is a design consideration) per the AISC *Specification*. Check the beam flange for bolt bearing (where deformation at the bolt hole is a design consideration) and block shear rupture per the AISC *Specification*.

Step 9. Design the Yield-Link flange-to-column flange connection using P_{r-link} from Step 7.

Step 9.1. Design bolts for tension force transfer between the Yield-Link flange and the column flange per the AISC *Specification* and determine the diameter of the flange bolts, $d_{b-flange}$. The required tension force per bolt in the Yield-Link flange to column flange connection, r_t , is equal to $P_{r-link}/4$.

Step 9.2. Determine the thickness of the Yield-Link flange, t_{flange} , required to prevent prying action.

$$t_{flange} = \sqrt{\frac{4r_t b'}{p\phi_d F_u}} \quad (12.9-8)$$

$$b' = (b - d_{b-flange} / 2) \quad (12.9-9)$$

where

b = vertical distance from centerline of bolts in Yield-Link flange to face of Yield-Link stem, in. (mm)

$d_{b-flange}$ = diameter of bolt connecting Yield-Link flange and column flange, in. (mm)

p = minimum of $b_{flange}/2$ or s_{flange} , in. (mm)

Step 9.3. Check the thickness of the Yield-Link flange, t_{flange} , for shear yielding and shear rupture per the AISC *Specification*.

Step 9.4. Design the stem-to-flange weld of the Yield-Link as either a CJP weld or a double-sided fillet weld that will develop the tensile strength of the Yield-Link at the column side, P_{r-weld} :

$$P_{r-weld} = b_{col-side} t_{stem} R_t F_u - link \quad (12.9-10)$$

Step 10. Select the buckling restraint plate (BRP) per Section 12.8.6.

Step 11. Verify the elastic frame drift and connection moment demand by accounting for actual connection stiffness.

Step 11.1. Model the connection using a pair of nonlinear axial links or a nonlinear rotational spring at each connection, determined from the following properties:

$$K_1 = \text{elastic axial stiffness contribution due to bending stiffness in Yield-Link flange, kip/in. (N/mm)}$$

$$= (0.75)(192)E \left(\frac{w_{col-side} t_{flange}^3}{12} \right) \frac{1}{g_{flange}^3} \quad (12.9-11)$$

$$K_2 = \text{elastic axial stiffness contribution due to nonyielding section of Yield-Link, kip/in. (N/mm)}$$

$$= \frac{t_{stem} b_{col-side} E}{L_{col-side} + s_c + l_v} \quad (12.9-12)$$

where

$l_v = 0$ when four or fewer bolts are used at Yield-Link-to-beam connection
 $= s_{stem}/2$ when more than four bolts are used at Yield-Link-to-beam connection

$$K_3 = \text{elastic axial stiffness contribution due to yielding section of Yield-Link, kip/in. (N/mm)}$$

$$= \frac{t_{stem} b_{yield} E}{L_{y-link}} \quad (12.9-13)$$

$$K_{eff} = \text{effective elastic axial stiffness of Yield-Link, kip/in. (N/mm)}$$

$$= \frac{K_1 K_2 K_3}{(K_1 K_2 + K_2 K_3 + K_1 K_3)} \quad (12.9-14)$$

$$M_{pr} = \text{probable maximum moment capacity of Yield-Link pair, kip-in. (N-mm)}$$

$$= P_{r-link} (d + t_{stem}) \quad (12.9-16)$$

$$M_{ye-link} = \text{expected yield moment of Yield-Link pair, kip-in. (N-mm)}$$

$$= P_{ye-link} (d + t_{stem}) \quad (12.9-15)$$

$$n_{bolt} = \text{number of bolts in Yield-Link stem-to-beam-flange connection}$$

$$\Delta_{0.04} = \text{axial deformation in Yield-Link at a connection rotation of 0.04 rad}$$

$$= \frac{0.04(d + t_{stem})}{2} \quad (12.9-17)$$

$$\Delta_{0.07} = \text{axial deformation in Yield-Link at a connection rotation of 0.07 rad}$$

$$= \frac{0.07(d + t_{stem})}{2} \quad (12.9-18)$$

$$\begin{aligned}\Delta_y &= \text{axial deformation in Yield-Link at expected yield, in. (mm)} \\ &= \frac{P_{ye-link}}{K_{eff}}\end{aligned}\quad (12.9-19)$$

$$\begin{aligned}\theta_y &= \text{connection rotation at expected yield of Yield-Link, rad} \\ &= \frac{\Delta_y}{0.5(d + t_{stem})}\end{aligned}\quad (12.9-20)$$

All other terms were previously defined or shown in Figure 12.2. Refer to Figure 12.4(a) for a plot of Yield-Link axial force versus Yield-Link axial deformation. Refer to Figure 12.4(b) for the moment versus rotation relationship required for the analysis and modeling of the Simpson Strong-Tie moment connection.

Step 11.2. Considering the applicable drift limit and all applicable load combinations specified by the applicable building code, but not including the amplified seismic load, verify that:

- The connection moment demand, M_u , is less than or equal to the connection design moment capacity, ϕM_n , taking ϕ as 0.90 and M_n as $M_{ye-link}/R_y$.
- The drift complies with applicable limits.

Adjust connection stiffness and/or number of connections as needed to comply.

Step 12. Determine the required shear strength, V_u , of the beam and beam web-to-column flange connection using:

$$V_u = \frac{2M_{pr}}{L_h} + V_{gravity} \quad (12.9-21)$$

where

L_h = horizontal distance between centerlines of the bolts in shear plate at each end of beam, in. (mm)

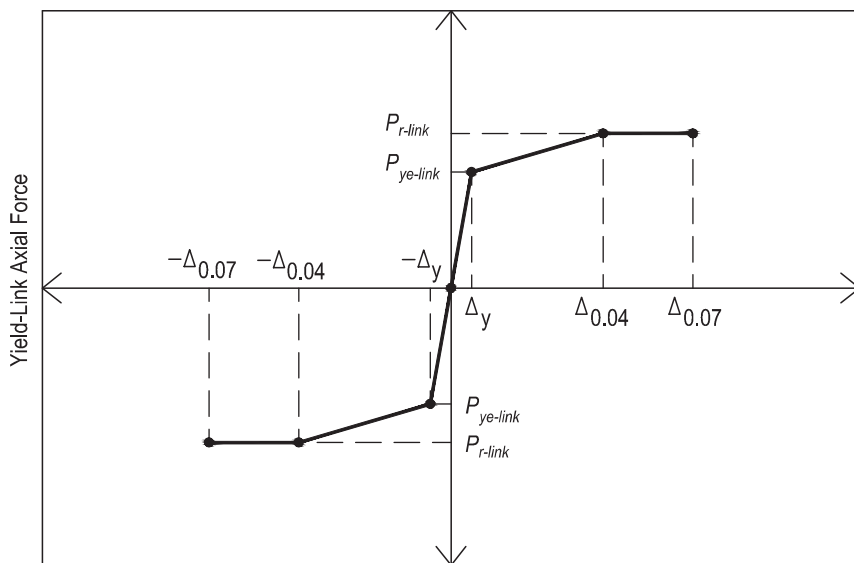
$V_{gravity}$ = shear force in the beam, kips (N), resulting from $1.2D + f_1L + 0.2S$ (where f_1 is the load factor determined by the applicable building code for live loads, but not less than 0.5). The shear force at the shear plate connection shall be determined from a free-body diagram of the portion of the beam between the shear plate connections.

User Note: The load combination of $1.2D + f_1L + 0.2S$ is in conformance with ASCE/SEI 7-16. When using the International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.2 for S (snow) when the roof configuration is such that it does not shed snow off the structure.

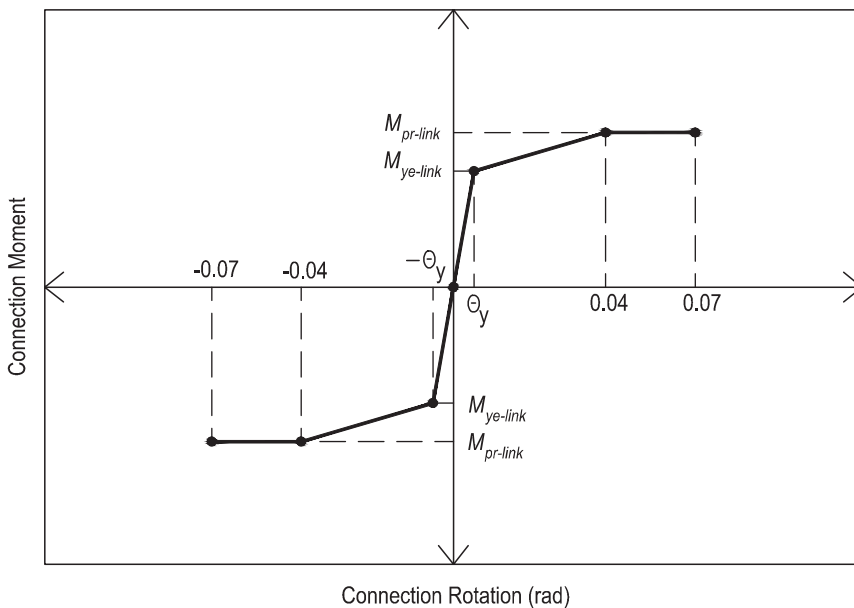
Step 13. Verify the beam and column sizes selected in Step 1.

Step 13.1. Beams shall satisfy the AISC *Specification* considering:

- Gravity load from all applicable load combinations.



(a) Yield-Link axial force vs. Yield-Link axial deformation.



(b) Connection moment vs. rotation.

Fig. 12.4. Simpson Strong-Tie moment connection modeling parameters.

- (b) Axial force due to seismic effects determined as the minimum of the maximum the system can deliver or as determined from the amplified seismic load.
- (c) The application of M_{pr} at each end of the beam as required.

Step 13.2. Column strength shall satisfy the AISC *Specification* considering loads from all applicable load combinations in the applicable building code, where the seismic effects are determined from the minimum of either the maximum the system can deliver or the amplified seismic loads. According to Section 12.3 2.(7), if column bracing is only provided at the level of the top flange of the beam, in addition to the requirements of the AISC *Specification*, the maximum design flexural strength of the column outside the panel zone, $\phi_b M_n$, shall be taken as $\phi_b M_n \leq \phi_b F_y S_x$, where $\phi_b = 0.90$.

Step 14. Check the column-beam relationship limitations according to Section 12.4.

Step 15. Design the beam web-to-column flange connection for the following required strengths:

$$M_{u-sp} = \text{moment in shear plate at column face, kip-in. (N-mm)} \\ = V_u a$$

P_{u-sp} = required axial strength of the connection shall be taken as the minimum of the following:

- (1) The maximum axial force the system can deliver.
- (2) The axial force calculated using the load combinations of the applicable building code, including the amplified seismic load.

$$V_u = V_u \text{ from Step 12.}$$

a = horizontal distance from centerline of the bolt holes in shear plate to face of the column, in. (mm). See Figure 12.3(c).

Step 15.1.

- (a) Calculate the maximum shear plate bolt shear by sizing the shear plate center bolt to take all axial load from the beam and a portion of the vertical loads, V_{u-bolt} , kips (N).

$$V_{u-bolt} = \sqrt{P_{u-sp}^2 + \left(\frac{V_u}{n_{bolt-sp}} \right)^2} \quad (12.9-22)$$

where $n_{bolt-sp}$ is 3, the total number of bolts in the shear plate.

- (b) Select a bolt diameter, d_{b-sp} that satisfies the AISC *Specification*.

Step 15.2. Determine the shear-plate geometry required to accommodate a connection rotation of ± 0.07 rad.

$$L_{slot} = d_{b-sp} + \frac{1}{8} \text{ in.} + 0.14s_{vert} \quad (12.9-23)$$

$$L_{slot} = d_{b-sp} + 3 \text{ mm} + 0.14s_{vert} \quad (12.9-23M)$$

where

d_{b-sp} = diameter of bolts in shear plate, in (mm)

s_{vert} = vertical distance from center of top (or bottom) shear plate bolt to center of center shear-plate bolt, in. (mm)

Step 15.3. Check the shear plate for tension and shear yielding, tension and shear rupture, block shear, combined tension and bending yielding at the column face, and bolt bearing, where deformation at the bolt hole is a design consideration, per the AISC *Specification*.

Step 15.4. Size the weld at the shear plate-to-column flange joint to develop the plate in shear, tension and bending. For double fillet welds, the minimum leg size shall be $\frac{5}{8}t_p$.

Step 15.5. Check the beam web for tension and shear yielding, tension and shear rupture, block shear, and bolt bearing, where deformation at the bolt hole is a design consideration, per the AISC *Specification*.

Step 15.6. Detail the beam flange and web cope such that the flange begins at a point aligned with the centerline of the shear-plate bolts. Check entering and tightening clearances as appropriate. See Figure 12.3(a).

User Note: Checking the beam web for flexure at the cope is not required since the flange copes do not extend beyond the centerline of the bolts in the beam shear-plate connection.

Step 16. Check the column panel zone shear strength per the AISC *Specification*. The required shear strength shall be determined from the summation of the probable maximum axial strengths of the Yield-Link. Doubler plates shall be used as required.

Step 17. Check the column web for the concentrated force(s) of P_{r-link} , according to the AISC *Specification*.

Step 18. Check the minimum column flange thickness for flexural yielding.

$$t_{cf_{min}} = \sqrt{\frac{1.11M_{pr}}{\phi_d F_{yc} Y_c}} \quad (12.9-24)$$

where

F_{yc} = specified minimum yield strength of column flange material, ksi (MPa)

Y_c = column flange yield line mechanism parameter from Table 6.5 or 6.3.

For connections away from column ends, Table 6.5 shall be used. For connections at column ends, Table 6.3 shall be used. An unstiffened column flange connection at the end of a column may be used where a rational analysis demonstrates that the unstiffened column flange design moment strength, as controlled by flexural yielding of the column flange, meets or exceeds the connection moment demand, $M_{pr-link}$.

Step 19. If a continuity plate or stiffener plate is required for any of the column limit states in Steps 17 and 18, the required strength, F_{su} , is

$$F_{su} = P_{r-link} - \text{minimum } (\phi R_n) \quad (12.9-25)$$

where

ϕR_n = design strengths from Step 17, kips (N)

Step 19.1. Design the continuity plate or stiffener plate per the AISC *Specification*.

Step 19.2. Design the stiffener-to-column web weld and the stiffener to-column flange weld per the AISC *Specification*.

The continuity plate or stiffener shall conform to Section J10.8 of the AISC *Specification* and shall have a minimum thickness of $\frac{1}{4}$ in. (6 mm).

CHAPTER 13

DOUBLE-TEE MOMENT CONNECTIONS

13.1. GENERAL

Double-tee connections utilize T-stub components that are bolted to both the column flange and the beam flanges using high-strength bolts. Either four bolts or eight bolts attach the T-stub components to the column flanges. The top and bottom T-stubs shall be identical. T-stubs shall be cut from rolled sections. The beam web is connected to the column with a bolted single-plate shear connection. A detail for this connection is shown in Figure 13.1. Yielding and hinge formation are intended to occur in the beam near the ends of the stems of the T-stubs. Figures 13.2 through 13.6 provide details regarding the dimensioning notation used in this chapter.

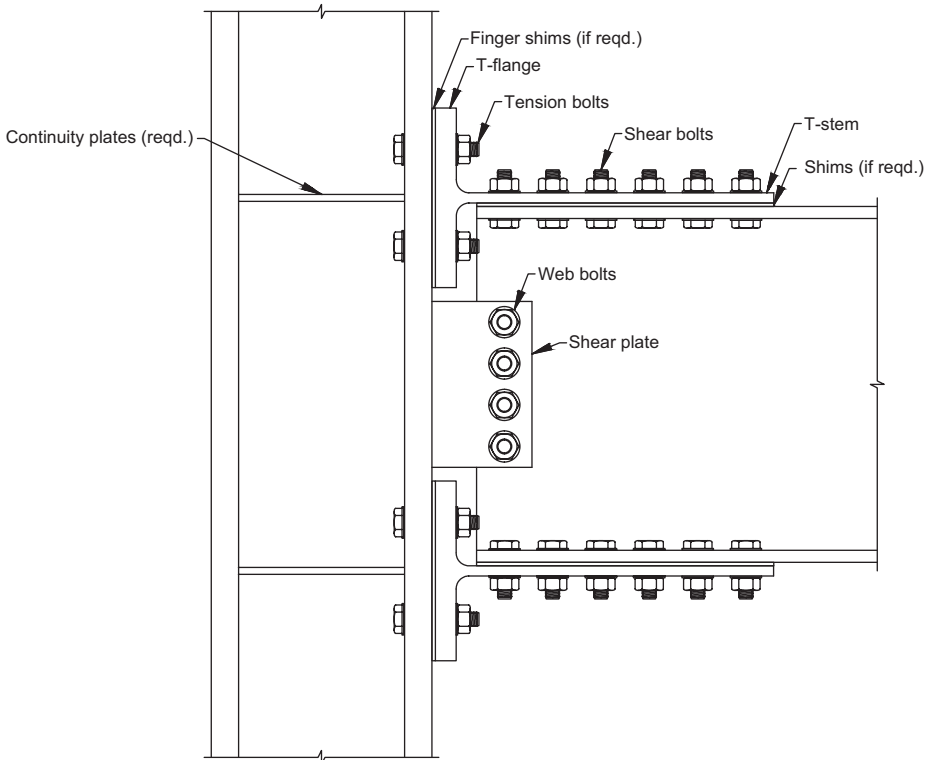


Fig. 13.1. Typical double-tee connection.

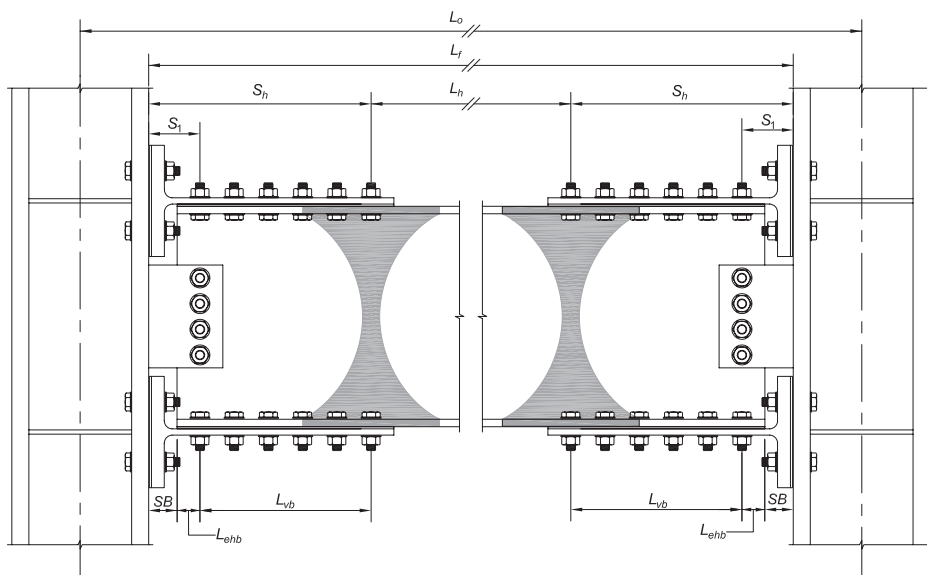


Fig. 13.2. Beam dimensions for double-tee connections. Shaded regions represent plastic hinges.

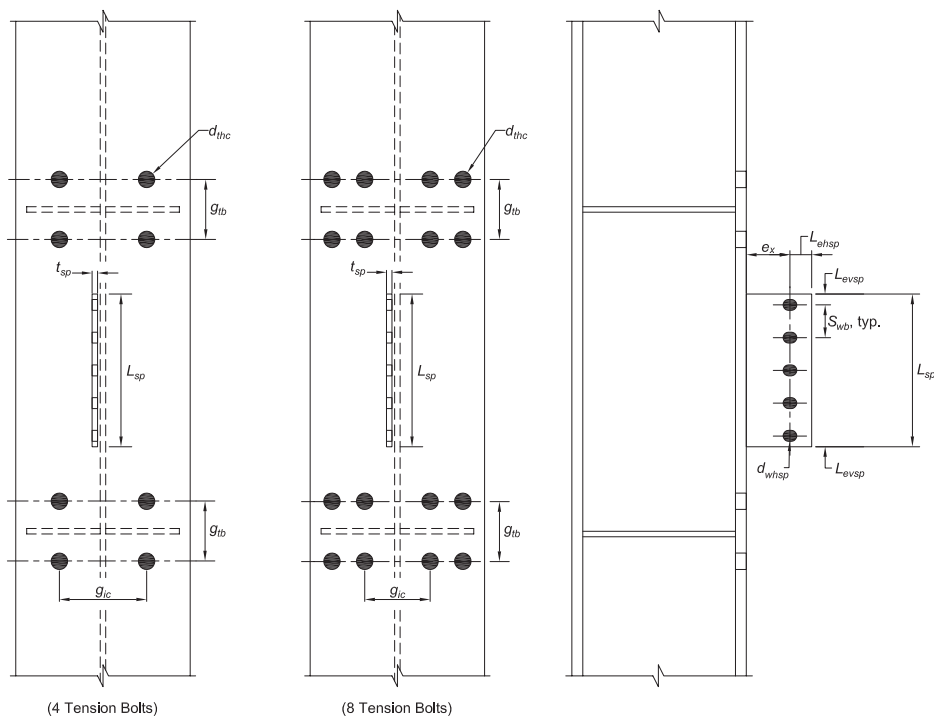


Fig. 13.3. Column and shear plate dimensions for double-tee connections.

13.2. SYSTEMS

Double-tee connections are prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems within the limitation of these provisions.

Exception: Double-tee connections in SMF systems with concrete structural slabs are prequalified only if:

1. There are no welded steel headed stud anchors attached to the beam flange between the face of the column and a location one beam depth beyond the shear bolts farthest from the face of the column; and
2. The concrete slab is kept at least 1 in. (25 mm) from both sides of both column flanges and the T-stub flange. It is permitted to place compressible material in the gap between the face of the T-stub and the concrete slab.

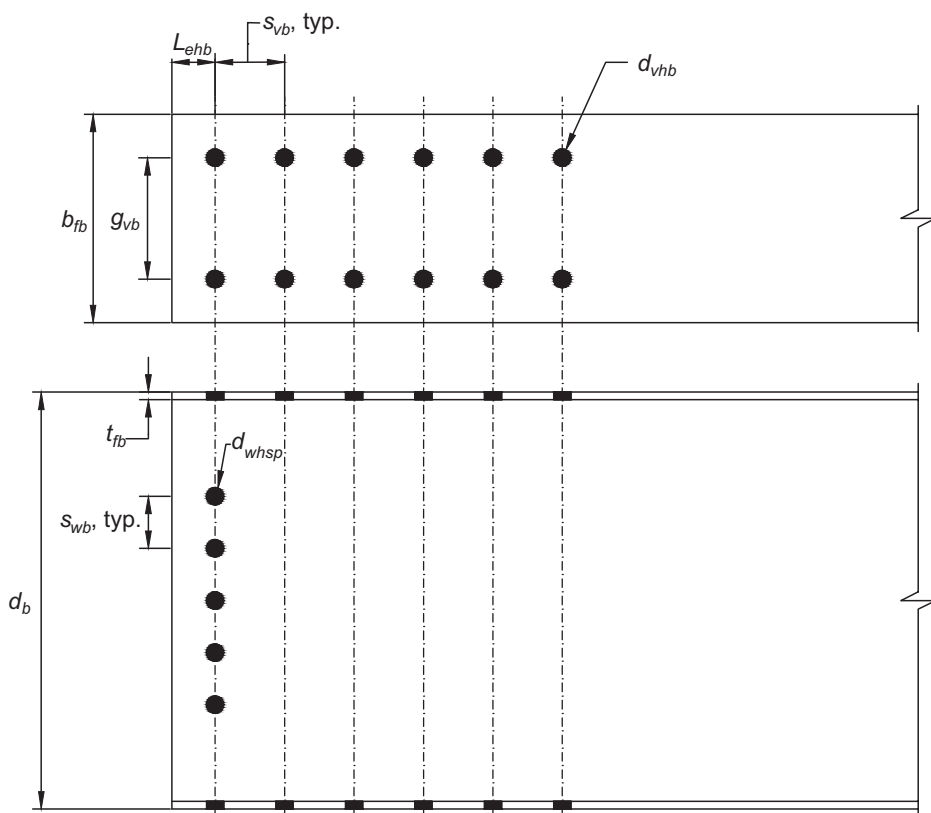


Fig. 13.4. Additional beam dimensions for double-tee connections.

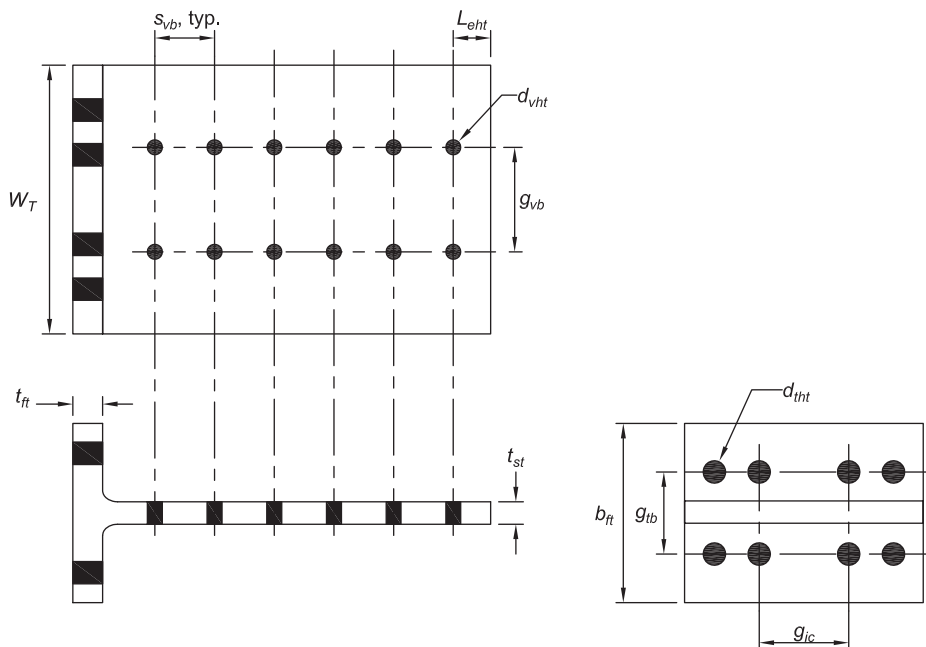


Fig. 13.5. T-stub dimensions for double-tee connections.

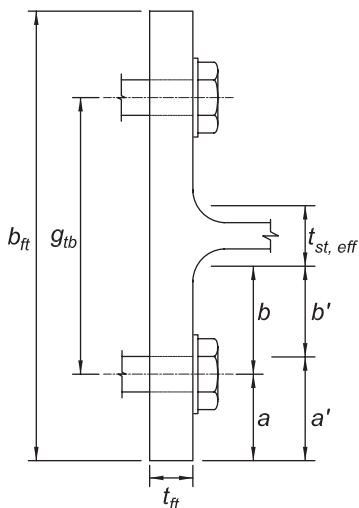


Fig. 13.6. T-stub flange dimensions for double-tee connections.

User Note: Note that connections designed for use in SMF and IMF systems must be designed as fully restrained (FR) connections. It is possible to design double-tee connections that qualify as partially restrained (PR), even when they satisfy all of the strength requirements stipulated within this specification. As a result, care must be taken during design to ensure that the connections resulting from this chapter have not only adequate strength, but that they also have adequate stiffness.

13.3. PREQUALIFICATION LIMITS

1. Beam Limitations

Beams shall satisfy the following limitations:

- (1) Beams shall be rolled wide-flange or welded built-up I-shaped members conforming to the requirements in Section 2.3.
- (2) Beam depth, d_b , is limited to a maximum of W24 (W610) for rolled shapes. The depth of built-up members shall not exceed the depth permitted for rolled wide-flange shapes.
- (3) Beam weight is limited to a maximum of 55 lb/ft (82 kg/m).
- (4) Beam flange thickness is limited to a maximum of $\frac{5}{8}$ in. (15 mm).
- (5) The clear span-to-depth ratio of the beam shall be limited to 9 or greater for both SMF and IMF systems.
- (6) Width-to-thickness ratios for the flanges and web of the beam shall conform to the limits of the AISC *Seismic Provisions*.
- (7) Lateral bracing of beams shall be in conformance with the AISC *Seismic Provisions* for SMF or IMF systems, as applicable. To satisfy the requirements for lateral bracing at plastic hinges, lateral bracing shall be provided at a location on the beam that is between d_b and $1.5d_b$ beyond the bolt farthest from the face of the column. No attachment of lateral bracing shall be made to the beam within the protected zone.

Exception: For both SMF and IMF systems, where the beam supports a concrete structural slab that is connected between the protected zones with welded steel headed stud anchors, spaced a maximum of 12 in (300 mm) on center, supplemental lateral bracing at plastic hinges is not required.

- (8) The protected zone consists of the T-stubs and the portion of the beam between the face of the column and one beam depth, d , beyond the bolt farthest from the face of the column.

2. Column Limitations

Columns shall satisfy the following limitations:

- (1) The beam shall be connected to the flange of the column.

- (2) Columns shall be any of the rolled shapes, welded built-up I-shapes, or flanged cruciform columns permitted in Section 2.3.
- (3) Rolled shape column depth shall be limited to W36 (W920) maximum when a concrete structural slab is provided. In the absence of a concrete structural slab, rolled shape column depth is limited to W14 (W360) maximum. The depth of built-up I-shaped columns shall not exceed that for rolled shapes. Flanged cruciform columns shall not have a width or depth greater than the depth allowed for rolled shapes.
- (4) Width-to-thickness ratios for the flanges and web of the column shall conform to the applicable limits of the AISC *Seismic Provisions*.
- (5) Lateral bracing of columns shall conform to the applicable limits of the AISC *Seismic Provisions*.

13.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

Beam-to-column connections shall satisfy the following limitations:

- (1) Panel zones shall conform to the applicable requirements of the AISC *Seismic Provisions*.
- (2) The column-beam moment ratio shall conform to the applicable requirements of the AISC *Seismic Provisions*.

13.5. CONNECTION DETAILING

1. T-Stub Material Specifications

T-stubs shall be cut from rolled sections and shall conform to either ASTM A992/A992M or ASTM A913/A913M Grade 50 (345).

2. Continuity Plates

Continuity plates shall be provided at the column with a thickness not less than the thickness of the beam flange. Continuity plates shall extend to the edge of the column flange, less $\frac{1}{4}$ in. (6 mm). The continuity plate welds shall be provided in accordance with the AISC *Seismic Provisions*.

3. Single-Plate Shear Connection Welds

The single-plate shear connection shall be welded to the column flange. The single-plate to column-flange connection shall consist of CJP groove welds, two-sided PJP groove welds, or two-sided fillet welds.

4. Bolts

Bolts shall satisfy the following requirements:

- (1) Bolts shall be arranged symmetrically about the axes of the members.

- (2) Shear bolts in the T-stem-to-beam-flange connection shall be limited to two bolts per row. Tension bolts in the T-flange-to-column-flange connection shall be arranged in two horizontal rows of either two or four bolts.
- (3) Types of holes:
 - (a) Standard holes shall be used in the beam flange and column flange.
 - (b) Standard or short-slotted holes (with slots aligned parallel to the axis of the beam) shall be used in either the beam web or the shear plate.
 - (c) Standard or oversized holes shall be used in the T-stem.
 - (d) Standard, oversized or short-slotted holes (with slots aligned parallel to the axis of the column) shall be used in the T-flange.
- (4) Bolt holes in the T-stubs and beam flanges shall be drilled or sub-punched and reamed. Bolt holes in the shear tab and the beam web may be drilled, sub-punched and reamed, punched or thermally cut.
- (5) The ratio of tension-bolt gage to T-flange thickness, g_{tb}/t_{fb} , shall be no larger than 7.0.
- (6) All bolts shall be installed as pretensioned high-strength bolts.
- (7) Faying surfaces of the beam flange and T-stem shall satisfy the requirements for slip-critical connections in accordance with AISC *Specification* Section J3.8. Faying surfaces shall have a Class A slip coefficient or higher.

User Note: The use of oversized holes in the T-stem with pretensioned bolts that are not designed as slip critical is permitted, consistent with Section D2.2 of the AISC *Seismic Provisions*.

5. T-Stub Shims

- (1) Steel shims with a maximum thickness of $\frac{1}{4}$ in. (6 mm) may be used between the stems of the tees and the flanges of the beam at either or both locations, subject to the limitations of the RCSC *Specification*.
- (2) The use of finger shims between the flanges of the tees and the flange of the column is permitted at either or both locations, subject to the limitations of the RCSC *Specification*.

13.6. DESIGN PROCEDURE

Step 1. Compute the probable maximum moment at the plastic hinge.

$$M_{pr} = C_{pr} R_y F_{yb} Z_x \quad (13.6-1)$$

where

C_{pr} = factor to account for peak strength as defined in Section 2.4.3

F_{yb} = specified minimum yield stress of the beam, ksi (MPa)

R_y = ratio of expected yield stress to the specified minimum yield stress

Z_x = plastic section modulus about the x -axis of the gross section of the beam at the location of the plastic hinge, in.³ (mm³)

Step 2. Determine the shear bolt diameter. To preclude a net section fracture of the beam flange, the net section of the beam section shall satisfy the following:

$$Z_{x,net}R_tF_{ub} \geq Z_x R_yF_{yb} \quad (13.6-2)$$

where

F_{ub} = specified minimum tensile strength of beam, ksi (MPa)

R_t = ratio of expected tensile strength to the specified minimum tensile strength

$Z_{x,net}$ = plastic section modulus of the net section of the beam at the location of the plastic hinge, in.³ (mm³)

User Note: $Z_{x,net}$ of the beam may be computed by accounting for only the holes in the tension flange or, more simply, $Z_{x,net}$ of the beam may be computed by accounting for the holes in both flanges. Note that if the former approach is employed, the plastic neutral axis will not be at the mid-depth of the beam, which complicates the calculations somewhat. If the latter approach is employed, the calculations are a bit simpler and the requirement of Equation 13.6-2 may be met with a maximum shear bolt diameter that is determined by:

$$d_{vb} \leq \left(\frac{Z_x}{2t_{fb}(d_b - t_{fb})} \right) \left(1 - \frac{R_yF_{yb}}{R_tF_{ub}} \right) - \frac{1}{8} \text{ in.} \quad (13.6-3)$$

$$d_{vb} \leq \left(\frac{Z_x}{2t_{fb}(d_b - t_{fb})} \right) \left(1 - \frac{R_yF_{yb}}{R_tF_{ub}} \right) - 3 \text{ mm} \quad (13.6-3M)$$

where

d_b = depth of the beam, in. (mm)

d_{vb} = diameter of the shear bolts between the T-stem and the beam flange, in. (mm)

t_{fb} = flange thickness of the beam, in. (mm)

Step 3. Determine the design shear strength per shear bolt based on the limit states of shear fracture and material bearing as follows:

$$\phi r_{nv} = \min \begin{cases} \text{bolt shear} & \phi_n F_{nv} A_{vb} \\ \text{beam flange bearing} & \phi_d 2.4 d_{vb} t_{fb} F_{ub} \\ \text{T-stem bearing} & \phi_d 2.4 d_{vb} t_{st} F_{ut} \end{cases} \quad (13.6-4)$$

where

A_{vb} = gross area of a shear bolt measured through its shank, in.² (mm²)

F_{nv} = nominal shear stress of a bolt from the AISC *Specification*, ksi (MPa)

d_{vb} = diameter of the shear bolts between the T-stem and the beam flange, in. (mm)

r_{nv} = nominal shear strength of a shear bolt, kips/bolt (N/bolt)
 t_{fb} = flange thickness of the beam, in. (mm)
 t_{st} = stem thickness of the T-stub, in. (mm)
 $\phi_d = 1.00$
 $\phi_n = 0.90$

Step 4. Estimate the number of shear bolts, n_{vb} , required in each beam flange as follows:

$$n_{vb} \geq \frac{1.25M_{pr}}{d_b \phi r_{nv}} \quad (13.6-5)$$

where n_{vb} is an even integer.

Step 5. Determine the location of the plastic hinge in the beam. The plastic hinge is assumed to form at the shear bolts farthest from the face of the column. The distance from the face of the column to the plastic hinge, S_h , based on the estimated number of shear bolts, the horizontal end distance, and bolt spacing is:

$$S_h = S_1 + L_{vb} \quad (13.6-6)$$

where

$$L_{vb} = s_{vb} \left(\frac{n_{vb}}{2} - 1 \right), \text{ in. (mm)} \quad (13.6-7)$$

S_1 = distance between the face of the column and the first row of shear bolts, in. (mm)

s_{vb} = spacing of the shear bolts, in. (mm)

Step 6. Calculate the shear force at the beam plastic hinge location at each end of the beam. The shear force at the hinge location, V_h , shall be determined from a free-body diagram of the portion of the beam between the plastic hinge locations. This calculation shall assume the moment at the plastic hinge location is M_{pr} and shall include gravity loads acting on the beam based on the load combination $1.2D + f_1L + 0.2S$, where f_1 is the load factor determined by the applicable building code for live loads, but not less than 0.5.

User Note: The load combination $1.2D + f_1L + 0.2S$ is in conformance with ASCE/SEI 7-16. When using the 2015 International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.2 when the roof configuration is such that it does not shed snow off of the structure.

Step 7. Compute the expected beam moment at the face of the column. The moment developed at the face of the column, M_f , shall be determined as:

$$M_f = M_{pr} + V_h S_h \quad (13.6-10)$$

where

V_h = Beam shear force at plastic hinge location, kips (N)

Step 8. Compute the probable force in the T-stub, F_{pr} , due to M_f .

$$F_{pr} = \frac{M_f}{1.05d_b} \quad (13.6-11)$$

where $1.05d_b$ is used to estimate the sum of the depth of the beam and the thickness of the T-stem.

Step 9. Determine the size of the T-stem required. The stem thickness shall be determined based on the limit states of gross section yielding and net section fracture (checked in this step) and compression due to flexural buckling (checked in Step 16).

In sizing the T-stem, the Whitmore width, W_{Whit} , shall be estimated as:

$$W_{Whit} = 2L_{vb} \tan 30^\circ + g_{vb} \quad (13.6-12)$$

where

g_{vb} = Gage of shear bolts in the T-stub, in. (mm)

The minimum stem thickness, t_{st} , based on yielding of the T-stem is:

$$t_{st} \geq \frac{F_{pr}}{\min(W_T, W_{Whit}) \phi_d F_{yt}} \quad (13.6-13)$$

where

F_{yt} = specified minimum yield stress of the T-stub, ksi (MPa)

W_T = width of the T-stub measured parallel to the column flange width, in. (mm)

The minimum stem thickness, t_{st} , based on fracture of the T-stem is:

$$t_{st} \geq \frac{F_{pr}}{\phi_n F_{ut} [\min(W_T, W_{Whit}) - 2(d_{vht} + 1/16 \text{ in.})]} \quad (13.6-14)$$

$$t_{st} \geq \frac{F_{pr}}{\phi_n F_{ut} [\min(W_T, W_{Whit}) - 2(d_{vht} + 2 \text{ mm})]} \quad (13.6-14M)$$

where

F_{ut} = specified minimum tensile stress of T-stub, ksi (MPa)

d_{vht} = diameter of the holes in the T-stem for the shear bolts, in. (mm)

To ensure that compression buckling of the T-stem will not control, select the thickness of the T-stem such that:

$$t_{st} \geq \frac{S_1 - t_{ft}}{9.60} \quad (13.6-15)$$

where

S_1 = distance from the face of the column to the first row of shear bolts, in. (mm)

t_{ft} = flange thickness of the T-stub, in. (mm)

Step 10. Determine the size of the bolts connecting the T-stub to the column flange. The minimum diameter of the tension bolts, d_{tb} , shall be determined as:

$$d_{tb} \geq \sqrt{\frac{4F_{pr}}{n_{tb}\phi_n\pi F_{nt}}} \quad (13.6-16)$$

where

F_{nt} = nominal tensile stress of bolt from the AISC *Specification*, ksi (MPa)

n_{tb} = number of tension bolts connecting the T-flange to the column flange

Step 11. Determine the preliminary configuration of the T-flange. The flange width of the T-stub, b_{ft} , shall be computed as:

$$b_{ft} \geq g_{tb} + 2a \quad (13.6-17)$$

where

$$a = 1.5d_{tb} \leq 1.25b \quad (13.6-18)$$

b = distance between effective T-stem and bolt line in the T-flange, in. (mm)

g_{tb} = gage of the tension bolts in the T-stub, in. (mm)

User Note: The limit of $a \leq 1.25b$ in Equation 13.6-18 is a computation limit only and is not a physical limitation on the dimension a . The dimension b may be estimated as $0.40g_{tb}$ for preliminary estimation of prying forces.

The design strength of a single tension bolt, ϕr_{nt} , shall be computed as:

$$\phi r_{nt} = \phi_n A_{tb} F_{nt} \quad (13.6-19)$$

where

A_{tb} = gross area of a tension bolt measured through its shank, in.² (mm²)

The required T-flange strength, T_{req} , in units of kips per tension bolt (N per tension bolt) shall be computed as:

$$T_{req} = F_{pr} / n_{tb} \quad (13.6-20)$$

The minimum flange thickness, t_{ft} , based on a mixed-mode failure, which will typically govern, shall be computed as:

$$t_{ft} \geq 2 \sqrt{\frac{T_{req}(a' + b') - \phi r_{nt} a'}{\phi_d F_{yt} p}} \quad (13.6-21)$$

where

$$a' = a + \frac{1}{2}d_{tb} \quad (13.6-23)$$

$$b' = b - \frac{1}{2}d_{tb} \quad (13.6-24)$$

$$p = \frac{2W_T}{n_{tb}} \quad (13.6-22)$$

In certain situations, the term under the radical in Equation 13.6-21 can be negative, resulting in an erroneous required flange thickness. An alternate formulation of Equation 13.6-21 that can be applied in these cases is:

$$t_{ft} \geq 2 \sqrt{\frac{\phi r_{mt} a' b'}{\phi_d F_{yt} P [a' + \delta(a' + b')]}} \quad (13.6-25)$$

where

$$\delta = \left(1 - \frac{d_{ht}}{P}\right) \quad (13.6-26)$$

User Note: In all cases, the flange thickness required to eliminate prying action can be computed as:

$$t_{ft,crit} = \sqrt{\frac{4\phi r_{mt} b'}{\phi_d F_{yt} P}} \quad (13.6-27)$$

Step 12. Select a W-shape from which the T-stubs will be cut. A W-shape from which the T-stubs will be cut shall be selected based on:

1. The minimum depth required to accommodate the setback and horizontal end distance of the beam, S_1 , and the length of the shear-bolt group, L_{vb} , found in Step 5.
2. The minimum web thickness, t_{st} , found in Step 9.
3. The minimum flange width, b_{ft} , and flange thickness, t_{ft} , found in Step 11.

Step 13. Check the connection rotational stiffness to ensure that the connection is classified as fully restrained. The following shall be satisfied:

$$K_i \geq \frac{18EI_{beam}}{L_o} \quad (13.6-28)$$

where

E = modulus of elasticity of steel = 29,000 ksi (200,000 MPa)

I_{beam} = strong axis moment of inertia of the beam, in.⁴ (mm⁴)

L_o = theoretical length of the connected beam measured between the working points of the adjacent columns, in. (mm)

$$K_i = \frac{d_b^2 K_{ten} K_{comp}}{K_{ten} + K_{comp}} \quad (13.6-29)$$

$$K_{ten} = \left(\frac{1}{K_{flange}} + \frac{1}{K_{stem}} + \frac{1}{K_{slip}} \right)^{-1} \quad (13.6-30)$$

$$K_{comp} = \left(\frac{1}{K_{stem}} + \frac{1}{K_{slip}} \right)^{-1} \quad (13.6-31)$$

$$K_{flange} = \frac{12n_{tb}EI_{ft}(a'\beta_a + 3b'\beta_b)}{b'^3\beta_b(4a'\beta_a + 3b'\beta_b)}a \quad (13.6-32)$$

$$K_{stem} = \frac{t_{st}E(W_T - b_{fb})^2}{L_{stem} \left[(W_T - b_{fb}) + b_{fb} \ln \left(\frac{b_{fb}}{W_T} \right) \right]} \quad (13.6-33)$$

where

L_{stem} = length of stem, in. (mm)

$$K_{slip} = \frac{P_{slip}}{\Delta_{slip}} \quad (13.6-34)$$

$$P_{slip} = n_{vb}\alpha(0.70F_{nt}A_{vb})\mu \quad (13.6-35)$$

where

α = 1.00 for ASTM F3125 Grades A325, A325M and F1852 bolts
 = 0.88 for ASTM F3125 Grades A490, A490M and F2280 bolts

$$I_{ft} = \frac{Pt_{ft}^3}{12} \quad (13.6-36)$$

$$\beta_a = 1 + \frac{12EI_{ft}}{Gpt_{ft}a'^2} \quad (13.6-37)$$

$$\beta_b = 1 + \frac{12EI_{ft}}{Gpt_{ft}b'^2} \quad (13.6-38)$$

$$\Delta_{slip} = 0.0076 \text{ in. (0.19 mm)} \quad (13.6-39)$$

Step 14. Compute the maximum force in the T-stub due to M_f . Using the actual T-stem thickness, the actual flange force that is to be carried by the T-stubs, F_f , shall be computed as:

$$F_f = \frac{M_f}{d_b + t_{st}} \quad (13.6-40)$$

Step 15. Back-check the strength of the shear bolts with the actual flange force. Use ϕr_{nv} from Step 3 to confirm that the number of shear bolts, n_{vb} , estimated in Step 4 is adequate to resist the actual flange force, F_f .

$$\phi R_n \geq F_f \quad (13.6-41)$$

Step 16. Back-check the strength of the T-stem using the maximum beam flange force. Back-check that the gross section yielding, net section fracture, and flexural buckling strengths of the T-stem are adequate to resist the maximum flange force, F_f .

$$\phi R_n \geq F_f$$

For stem gross section yielding

$$\phi R_n = \phi_d F_{yt} \min(W_T, W_{Whit}) t_{st} \quad (13.6-42)$$

For stem net section fracture

$$\phi R_n = \phi_n F_{ut} \left[\min(W_T, W_{Whit}) - 2(d_{vht} + 1/16 \text{ in.}) \right] t_{st} \quad (13.6-43)$$

$$\phi R_n = \phi_n F_{ut} \left[\min(W_T, W_{Whit}) - 2(d_{vht} + 2 \text{ mm}) \right] t_{st} \quad (13.6-43M)$$

For stem flexural buckling

$$\frac{KL}{r} = \frac{(0.75)(S_1 - t_{ft})}{\sqrt{\frac{W_T t_{st}^3}{12 W_T t_{st}}}} = 2.60 \left(\frac{S_1 - t_{ft}}{t_{st}} \right) \quad (13.6-44)$$

If $KL/r \leq 25$ then

$$\phi R_n = \phi_d F_{yt} \min(W_T, W_{Whit}) t_{st} \quad (13.6-45)$$

If $KL/r > 25$ then ϕR_n is determined using the provisions in Section E3 of the AISC *Specification* using KL/r as determined previously and taking ϕ equal to ϕ_n .

Step 17. Back-check the flange strength of the T-stub. The flange strength of the T-stub shall be computed as:

$$\phi R_n = n_{tb} \phi T \quad (13.6-46)$$

where ϕT is the minimum of ϕT_1 (plastic flange mechanism), ϕT_2 (mixed-mode failure), and ϕT_3 (tension bolt fracture with no prying), as computed in the following.

For the limit state of a plastic mechanism in the tension flange, the design strength per tension bolt shall be calculated as:

$$\phi T_1 = \frac{(1 + \delta)}{4b'} p \phi_d F_{yt} t_{ft}^2 \quad (13.6-47)$$

For the limit state of tension flange yielding followed by fracture of the bolts (a mixed-mode failure), the design strength per tension bolt shall be calculated as:

$$\phi T_2 = \frac{\phi r_m a'}{a' + b'} + \frac{p \phi_d F_{yt} t_{ft}^2}{4(a' + b')} \quad (13.6-48)$$

For the limit state of bolt fracture without yielding of the tension flange, the design strength per tension bolt shall be calculated as:

$$\phi T_3 = \phi r_{nt} \quad (13.6-49)$$

where

$$a' = a + \frac{1}{2} d_{tb} \quad (13.6-50)$$

$$a = \left(\frac{1}{2} \right) (b_{ft} - g_{tb}) \leq 1.25 b \quad (13.6-51)$$

$$b' = b - \frac{1}{2} d_{tb} \quad (13.6-52)$$

$$b = \left(\frac{1}{2}\right)(g_{tb} - t_{st,eff}) \quad (13.6-53)$$

$$p = \frac{2W_T}{n_{tb}} \quad (13.6-22)$$

$$t_{st,eff} = k_1 + \frac{t_{st}}{2} \quad (13.6-54)$$

$$\delta = \left(1 - \frac{d_{tht}}{p}\right) \quad (13.6-26)$$

where

d_{tht} = diameter or width of holes in T-flange for the tension bolts, in. (mm)

Step 18. Check the bearing and tear-out strength of the beam flange and T-stem. Bearing and tearout of the shear bolts shall be checked in a manner consistent with Chapter J of the AISC *Specification*. For these calculations, bearing and tearout are considered to be ductile failure modes.

Step 19. Check block shear of the beam flange and the T-stem. Block shear of the T-stem and beam flange shall be checked in a manner consistent with Chapter J of the AISC *Specification*. For the purpose of this design, the block shear failure shall be considered a ductile failure mode and ϕ_d shall be used.

The alternate block shear mechanism illustrated in Figure 13.7 need not be checked.

Step 20. Determine the configuration of the shear connection to the web.

User Note: Because of the large setback required, the shear connection will most likely need to be designed as an extended shear tab. Most importantly, the length of the shear connection, L_{sc} , should be determined so as to fit between the flanges of the T-stubs allowing ample clearance.

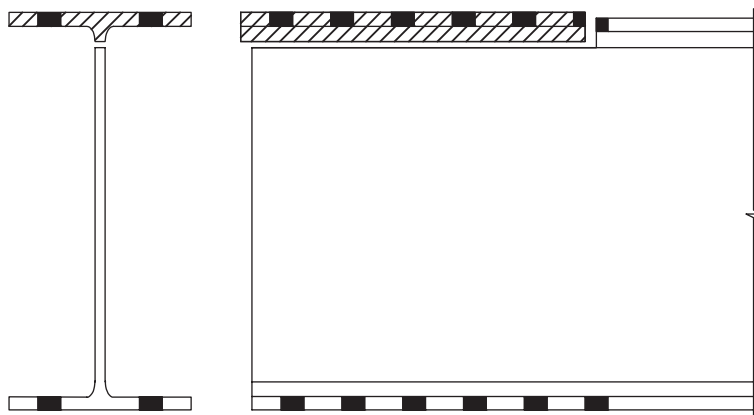


Fig. 13.7. Alternate block shear mechanism.

Step 21. Check the column flange for flexural yielding (see Figure 13.8):

The column flange flexural design strength is

$$\phi R_n = \phi_d F_{yc} Y_C t_{fc} \quad (13.6-55)$$

where

F_{yc} = specified minimum yield stress of column flange material, ksi (MPa)

$$Y_C = \left(\frac{2}{b} \right) \left(s + p_s + \frac{a_c b_c + b_c^2}{s} + \frac{a_c b_c + b_c^2}{p_s} \right) \quad (13.6-56)$$

$$a_c = \frac{b_{fc} - g_{ic}}{2} \quad (13.6-57)$$

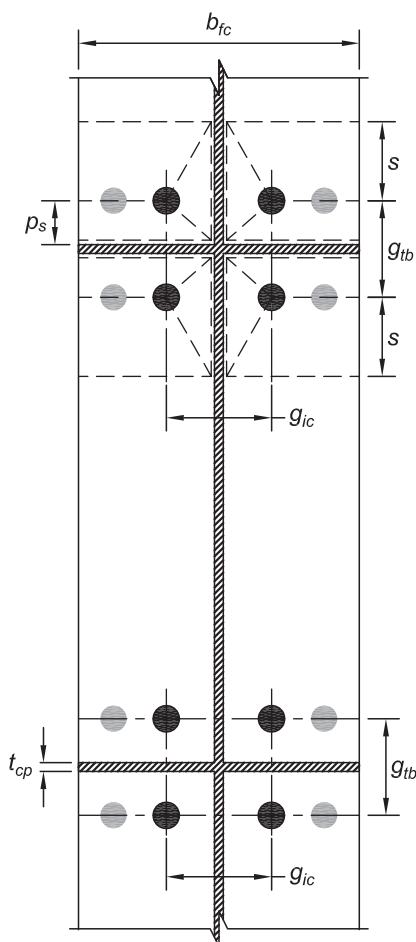


Fig. 13.8. Dimension for yield-line analysis of column flange.

$$b_c = \frac{g_{ic}}{2} \quad (13.6-58)$$

b_{fc} = flange width of the column, in. (mm)

g_{ic} = gage of interior tension bolts in the column flange, in. (mm)

g_{tb} = gage of tension bolts in T-stub, in. (mm)

$$p_s = \frac{g_{tb} - t_{cp}}{2} \leq s \quad (13.6-59)$$

$$s = \frac{\sqrt{b_{fc} g_{ic}}}{2} \quad (13.6-60)$$

t_{cp} = thickness of continuity plates, in. (mm)

Alternatively, the column flange thickness shall satisfy the following:

$$t_{fc} \geq \sqrt{\frac{1.11F_f}{\phi_d F_{yc} Y_C}} \quad (13.6-61)$$

User Note: The presence of continuity plates stiffening the column flanges precludes the need to check prying forces resulting from column-flange deformations.

Step 22. Check the column web strength for web yielding, web crippling, and panel-zone shear failures in accordance with the AISC *Seismic Provisions*.

Step 23. Detail continuity plates and, if required, detail doubler plates in accordance with the AISC *Seismic Provisions*.

APPENDIX A

CASTING REQUIREMENTS

A1. CAST STEEL GRADE

Cast steel grade shall be in accordance with ASTM A958/A958M Grade SC8620 class 80/50.

A2. QUALITY CONTROL (QC)

1. Inspection and Nondestructive Testing Personnel

Visual inspection and nondestructive testing shall be conducted by the manufacturer in accordance with a written practice by qualified inspectors. The procedure and qualification of inspectors is the responsibility of the manufacturer. Qualification of inspectors shall be in accordance with ASNT-TC-1a or an equivalent standard. The written practice shall include provisions specifically intended to evaluate defects found in cast steel products. Qualification shall demonstrate familiarity with inspection and acceptance criteria used in evaluation of cast steel products.

2. First Article Inspection (FAI) of Castings

The first article is defined as the first production casting made from a permanently mounted and rigged pattern. FAI shall be performed on the first casting produced from each pattern. The first article casting dimensions shall be measured and recorded. FAI includes visual inspection in accordance with Section A2.3, nondestructive testing in accordance with Section A2.4, tensile testing in accordance with Section A2.6, and Charpy V-notch testing in accordance with Section A2.7.

3. Visual Inspection of Castings

Visual inspection of all casting surfaces shall be performed to confirm compliance with ASTM A802/A802M and MSS SP-55 with a surface acceptance Level I.

4. Nondestructive Testing (NDT) of Castings

4a. Procedures

Radiographic testing (RT) shall be performed by quality assurance (QA) according to the procedures prescribed in ASTM E446 and ASTM E186 with an acceptance Level III or better.

Ultrasonic testing (UT) shall be performed by QA according to the procedures prescribed by ASTM A609/A609M Procedure A with an acceptance Level 3, or better.

Magnetic particle testing (MT) shall be performed by QA according to the procedures prescribed by ASTM E709 with an acceptance Level V, or better, in accordance with ASTM A903/A903M.

4b. Required NDT

(1) First Article

RT and MT shall be performed on the first article casting.

(2) Production Castings

UT shall be performed on 100% of the castings.

MT shall be performed on 50% of the castings.

(3) Reduction of Percentage of UT

The UT rate is permitted to be reduced if approved by the engineer of record and the authority having jurisdiction. The UT rate may be reduced to 25%, provided the number of castings not conforming to Section A2.4a is demonstrated to be 5% or less. A sampling of at least 40 castings shall be made for such reduction evaluation. This reduction is not permitted for castings with weld repairs.

(4) Reduction of Percentage of MT

The MT rate is permitted to be reduced if approved by the engineer of record and the authority having jurisdiction. The MT rate may be reduced to 10%, provided the number of castings not conforming to Section A2.4a is demonstrated to be 5% or less. A sampling of at least 20 castings shall be made for such reduction evaluation. This reduction is not permitted for castings with weld repairs.

5. Weld Repair Procedures

Castings with discontinuities that exceed the requirements of Section A2.4a shall be weld repaired. Weld repair of castings shall be performed in accordance with ASTM A488/A488M. The same testing method that discovered the discontinuities shall be repeated on repaired castings to confirm the removal of all discontinuities that exceed the requirements of Section A2.4a.

6. Tensile Requirements

Tensile tests shall be performed for each heat in accordance with ASTM A370 and ASTM 781/A781M.

7. Charpy V-Notch (CVN) Requirements

CVN testing shall be performed in accordance with ASTM A370 and ASTM 781/A781M. Three notched specimens shall be tested with the first heat, and with each subsequent 20th ton (18,100 kg) of finished material. The specimens shall have a minimum CVN toughness of 20 ft-lb (27 J) at 70°F (21°C).

8. Casting Identification

The castings shall be clearly marked with the pattern number and a unique serial number for each individual casting providing traceability to heat and production records.

A3. MANUFACTURER DOCUMENTS

1. Submittal to Patent Holder

The following documents shall be submitted to the patent holder, prior to the initiation of production as applicable:

- (1) Material chemical composition report
- (2) First article inspection report

2. Submittal to Engineer of Record and Authority Having Jurisdiction

The following documents shall be submitted to the engineer of record and the authority having jurisdiction, prior to, or with shipment as applicable:

- (1) Production inspection and NDT reports
- (2) Tensile and CVN test reports
- (3) Weld repair reports
- (4) Letter of approval by the patent holder of the manufacturer's FAI report

APPENDIX B

FORGING REQUIREMENTS

B1. FORGED STEEL GRADE

Raw material shall conform to the requirements of ASTM A572/A572M, Grade 50 (345). The forging process shall conform to the requirements of ASTM A788 and ASTM A668. Mechanical properties shall conform to the requirements of Table B1.1.

B2. BAR STOCK

Bar stock shall be cut to billets appropriate to the part being forged. All billets shall be marked with the heat number.

B3. FORGING TEMPERATURE

Billets shall be forged at a minimum temperature of 2,150°F (1,180°C) and a maximum temperature of 2,250°F (1230°C).

B4. HEAT TREATMENT

Immediately following impression forging, the part being forged shall be normalized for one hour at 1,650°F (899°C) then air cooled.

B5. FINISH

Finished forgings shall have shot blast finish, clean of mill scale.

B6. QUALITY ASSURANCE

One sample of bar stock from each heat shall be cut to a length of 6 in. (150 mm) and forged to a 5-in. by 2-in.-thick bar (125 mm by 50 mm). Samples shall be marked with longitudinal and transverse directions. Chemistry and physical properties in accordance with Table B1.1 shall be verified to ASTM A572/A572M Grade 50 (345) for both the longitudinal and transverse directions on each sample.

Magnetic particle testing shall be conducted on the initial 12 pieces from each run to verify tooling and forging procedures. Cracks shall not be permitted. If cracks are found, the tooling or forging procedure shall be modified, and an additional 12 initial pieces shall be tested. This process shall be repeated until 12 crack-free samples are obtained prior to production.

B7. DOCUMENTATION

Laboratory test data documenting chemistry, strength, elongation, reduction of area, and Charpy requirements for the samples tested in accordance with Section B6 shall be submitted.

TABLE B1.1
Required Mechanical Properties

Yield strength	50 ksi (345 MPa) minimum
Tensile strength	65 ksi (450 MPa) minimum
Elongation in 2 in. (50 mm)	22% minimum
Reduction of area	38% minimum
Charpy V-notch toughness	20 ft-lb at 70°F (27 J at 21°C)

Inspection reports documenting satisfactory performance of magnetic particle tests per Section B6 shall be submitted.

Certification of conformance with the requirements of this Appendix shall be submitted to the purchaser.

COMMENTARY

on Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications

May 12, 2016

This Commentary is not part of ANSI/AISC 358-16, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*. It is included for informational purposes only.

INTRODUCTION

The Standard is intended to be complete for normal design usage.

The Commentary furnishes background information and references for the benefit of the design professional seeking further understanding of the basis, derivations and limits of the Standard.

The Standard and Commentary are intended for use by design professionals with demonstrated engineering competence.

CHAPTER 1

GENERAL

1.1. SCOPE

Design of special moment frames (SMF) and intermediate moment frames (IMF) in accordance with the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2016a), hereafter referred to as the AISC *Seismic Provisions*, and applicable building codes includes an implicit expectation that they will experience substantial inelastic deformations when subjected to design-level earthquake ground shaking, generally concentrated at the moment-resisting beam-to-column connections. In the 1994 Northridge earthquake, a number of steel moment frame buildings were found to have experienced brittle fractures that initiated at the welded beam flange-to-column flange joints of moment connections. These brittle fractures were unexpected and were quite different from the anticipated behavior of ductile beam flexural yielding in plastic hinge zones. Where they occurred, these brittle fractures prevented the formation of ductile plastic hinge zones and resulted in frame behavior substantially different from that upon which the design requirements for these systems were based.

Following this discovery, the Federal Emergency Management Agency (FEMA) provided funding to a coalition of universities and professional associations, known as the SAC Joint Venture. Over a period of six years, the SAC Joint Venture, with participation from AISC, AISI, AWS, and other industry groups, conducted extensive research into the causes of the damage that had occurred in the Northridge earthquake and effective means of reducing the possibility of such damage in future earthquakes.

Numerous issues were identified in the SAC studies as contributing causes of these brittle fractures. This Standard specifically addresses the following four causes that were identified in the SAC study:

- (1) Connection geometries that resulted in large stress concentrations in regions of high triaxiality and limited ability to yield;
- (2) Use of weld filler metals with low inherent notch toughness and limited ductility;
- (3) High variability in the yield strengths of beams and columns, resulting in unanticipated zones of weakness in connection assemblies; and
- (4) Welding practice and workmanship that fell outside the acceptable parameters of the AWS D1.1/D1.1M, *Structural Welding Code*, at that time.

A more complete listing of the causes of damage sustained in the Northridge earthquake may be found in a series of publications (FEMA 350, FEMA 351, FEMA 352, FEMA 353, FEMA 355C, and FEMA 355D) published in 2000 by the SAC Joint Venture that presented recommendations for design and construction of moment resisting frames designed to experience substantial inelastic deformation during design ground

shaking. These recommendations included changes to material specifications for base metals and welding filler metals, improved quality assurance procedures during construction, and the use of connection geometries that had been demonstrated by testing and analysis to be capable of resisting appropriate levels of inelastic deformation without fracture. Most of these recommendations have been incorporated into the AISC *Seismic Provisions* as well as into AWS D1.8/D1.8M, *Structural Welding Code—Seismic Supplement* (AWS, 2016).

Following the SAC Joint Venture recommendations, the AISC *Seismic Provisions* require that moment connections used in special or intermediate steel moment frames be demonstrated by testing to be capable of providing the necessary ductility. Two means of demonstration are acceptable. One means consists of project-specific testing in which a limited number of full-scale specimens, representing the connections to be used in a structure, are constructed and tested in accordance with a protocol prescribed in Chapter K of the AISC *Seismic Provisions*. Recognizing that it is costly and time consuming to perform such tests, the AISC *Seismic Provisions* also provide for prequalification of connections consisting of a rigorous program of testing, analytical evaluation and review by an independent body, the Connection Prequalification Review Panel (CPRP). Connections contained in this Standard have met the criteria for prequalification when applied to framing that complies with the limitations contained herein and when designed and detailed in accordance with this Standard.

1.2. REFERENCES

References for this Standard are listed at the end of the Commentary.

1.3. GENERAL

Connections prequalified under this Standard are intended to withstand inelastic deformation primarily through controlled yielding in specific behavioral modes. To obtain connections that will behave in the indicated manner, proper determination of the strength of the connection in various limit states is necessary. The strength formulations contained in the LRFD method are consistent with this approach.

CHAPTER 2

DESIGN REQUIREMENTS

2.1. SPECIAL AND INTERMEDIATE MOMENT FRAME CONNECTION TYPES

Limitations included in this Standard for various prequalified connections include specification of permissible materials for base metals, mechanical properties for weld filler metals, member shape and profile, connection geometry, detailing, and workmanship. These limitations are based on conditions, demonstrated by testing and analytical evaluation, for which reliable connection behavior can be attained. It is possible that these connections can provide reliable behavior outside these limitations; however, this has not been demonstrated. When any condition of base metal, mechanical properties, weld filler metals, member shape and profile, connection geometry, detailing, or workmanship falls outside the limitations specified herein, project-specific qualification testing should be performed to demonstrate the acceptability of connection behavior under these conditions.

Limited testing of connections of wide-flange beams to the webs of I-shaped columns had been conducted prior to the Northridge earthquake by Tsai and Popov (1986, 1988). This testing demonstrated that these “minor-axis” connections were incapable of developing reliable inelastic behavior even at a time when major axis connections were thought capable of developing acceptable behavior. No significant testing of such minor axis connections following the Northridge earthquake has been conducted. Consequently, such connections are not currently prequalified under this Standard.

Similarly, although there has been only limited testing of connections in assemblies subjected to biaxial bending of the column, the judgment of the CPRP was that as long as columns are designed to remain essentially elastic and inelastic behavior is concentrated within the beams, it would be possible to obtain acceptable behavior of beam-column connection assemblies subjected to biaxial loading. Flanged cruciform section columns, built-up box columns, and boxed wide-flange columns are permitted to be used in assemblies subjected to biaxial loading for those connection types where inelastic behavior is concentrated in the beam, rather than in the column. It should be noted that the strong column-weak beam criteria contained in the AISC *Seismic Provisions* are valid only for planar frames. When both axes of a column participate in a moment frame, columns should be evaluated for the ability to remain essentially elastic while beams framing to both column axes undergo flexural hinging.

Nearly all moment frame connection tests have been performed on single- or double-sided beam-column subassemblies with the beam perpendicular to the vertical axis of the column (i.e., a level beam) and coplanar with the strong axis of the column (i.e.,

unskewed). The reality of building construction is that sloping beams occur in most structures, such as at the roof. Occasionally, skewed configurations are required to accommodate architectural requirements.

This Standard does not contain provisions that explicitly address sloped or skewed moment frame beams because of the wide variety of connections that have obtained prequalification and the lack of systematic physical testing for each connection. Professional judgment, therefore, is required to determine whether a proposed frame geometry is appropriately covered by the prequalification limits in this Standard.

One factor to consider when evaluating the impact of frame beam slope is the absolute angle at which the beam slopes. For example, even so-called “flat” roofs slope at approximately $\frac{1}{4}$ in./ft, which equates to an angle of 1.2° . Testing of frame beams using reduced beam sections sloping at 28° (Ball et al., 2010) indicates that at this angle the performance of the connection can be adversely impacted depending on how it is configured. The former angle likely is not significant, while the latter angle appears to be, at least for the reduced beam section. It is reasonable to assume that different connections will likely have different thresholds above which the slope becomes significant because of characteristics that govern connection behavior.

For example, connection geometry and limit states that govern connection behavior will likely influence the threshold above which frame beam slope becomes significant. Connection performance of sloped beams is expected to be influenced by components of differing length or changes in geometry (e.g., the relative distance from the face of column to the first bolt or the length of the flange plate). These variations may impose different levels of load on components at the top flange versus those on the bottom flange due to changes in relative stiffness. The relative onset of limit states such as local flange buckling, prying and the like may be influenced by the angle of slope and is expected to vary from connection type to connection type.

Limited analytical studies have been performed relative to the impact of beam skew on connection performance. Prinz and Richards (2016) report that frame beam skew angles between 10° and 20° in reduced beam section connections appear to cause limited increases in column torsional demand and limited additional flange tip yielding. They also report that skew angles of 10° reduce low cycle fatigue capacity in the reduced section region by less than one cycle. Thus, skew angles of less than 5° to 10° might be considered acceptable in reduced beam sections connections. Similar analytical studies have not been conducted for other connection geometries.

2.3. MEMBERS

2. Built-up Members

The behavior of built-up I-shaped members has been extensively tested in bolted end-plate connections and has been demonstrated to be capable of developing the necessary inelastic deformations. These members have not generally been tested in other prequalified connections; however, the conditions of inelastic deformation

imposed on the built-up shapes in these other connection types are similar to those tested for the bolted end-plate connections.

2b. Built-up Columns

Four built-up column cross-section shapes are covered by this Standard. These are illustrated in Figure C-2.1 and include:

- (1) I-shaped welded columns that resemble standard rolled wide-flange shapes in cross-section shape and profile.
- (2) Cruciform W-shape columns, fabricated by splitting a wide-flange section in half and welding the webs on either side of the web of an unsplit wide-flange section at its mid-depth to form a cruciform shape, each outstanding leg of which terminates in a rectangular flange.
- (3) Box columns, fabricated by welding four plates together to form a closed box-shaped cross section.
- (4) Boxed W-shape columns constructed by adding side plates to the sides of an I-shaped cross section.

The preponderance of connection tests reviewed as the basis for prequalifications contained in this Standard consisted of rolled wide-flange beams connected to the flanges of rolled wide-flange columns. A limited number of tests of connections of wide-flange beams to built-up box section columns were also reviewed (Anderson and Linderman, 1991; Kim et al., 2008).

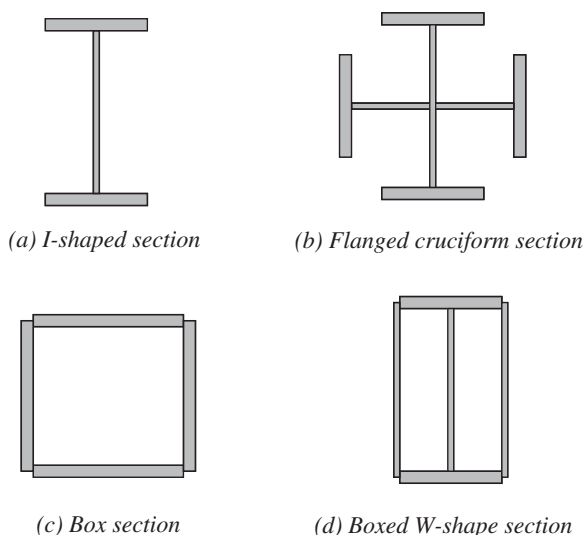


Fig. C-2.1. Column shapes. Plate preparation and welds are not shown.

The flanged cruciform column and boxed wide-flange columns have not specifically been tested. However, it was the judgment of the CPRP that as long as such column sections met the limitations for I-shaped sections and box-shaped sections, respectively, and connection assemblies are designed to ensure that most inelastic behavior occurred within the beam as opposed to the column, the behavior of assemblies employing these sections would be acceptable. Therefore, prequalification has been extended to these cross sections for connection types where the predominant inelastic behavior is in the beam rather than the column.

2.4. CONNECTION DESIGN PARAMETERS

1. Resistance Factors

A significant factor considered in the formulation of resistance factors is the occurrence of various limit states. Limit states that are considered brittle (nonductile) and subject to sudden catastrophic failure are typically assigned lower resistance factors than those that exhibit yielding (ductile) failure. Because, for the prequalified connections, design demand is determined based on conservative estimates of the material strength of weak elements of the connection assembly, and because materials, workmanship and quality assurance are more rigorously controlled than for other structural elements, resistance factors have been set somewhat higher than those traditionally used. It is believed that these resistance factors, when used in combination with the design, fabrication, erection, and quality-assurance requirements contained in the Standard, will provide reliable service in the prequalified connections.

2. Plastic Hinge Location

This Standard specifies the presumed location of the plastic hinge for each prequalified connection type. In reality, inelastic deformation of connection assemblies is generally distributed to some extent throughout the connection assembly. The plastic hinge locations specified herein are based on observed behavior during connection assembly tests and indicate the locations of most anticipated inelastic deformation in connection assemblies conforming to the particular prequalified type.

3. Probable Maximum Moment at Plastic Hinge

The probable plastic moment at the plastic hinge is intended to be a conservative estimate of the maximum moment likely to be developed by the connection under cyclic inelastic response. It includes consideration of likely material overstrength and strain hardening.

4. Continuity Plates

Beam flange continuity plates serve several purposes in moment connections. They help to distribute beam flange forces to the column web, they stiffen the column web to prevent local crippling under the concentrated beam-flange forces, and they minimize stress concentrations that can occur in the joint between the beam flange and column due to nonuniform stiffness of the attached elements.

Almost all connection assembly testing has been conducted on specimens that include a significant length (typically one-half story height) of column above and below the beam or beams framing into the column. Thus, the condition that typically exists in a structure's top story, where the column terminates at the level of the beam top flange, has not specifically been tested to demonstrate acceptable detailing. A cap plate detail similar to that illustrated in Figure C-2.2 is believed to be capable of providing reliable performance when connection elements do not extend above the beam top flange. In some connections, e.g., extended end-plate and Kaiser bolted bracket connections, portions of the connection assembly extend above the column top flange. In such cases, the column should be extended to a sufficient height above the beam flange to accommodate attachment and landing of those connection elements. The stiffener plates should be placed in the column web, opposite the beam top flange, as is done at intermediate framing levels.

The attachment of continuity plates to column webs is designed to be capable of transmitting the maximum shear forces that can be delivered to the continuity plate. This may be limited by the beam-flange force, the shear strength of the continuity plate itself, or the welded joint between the continuity plate and column flange.

The AISC *Seismic Provisions* require that continuity plates be attached to column flanges with CJP groove welds so the strength of the beam flange can be properly developed into the continuity plate. For single-sided connections in which a

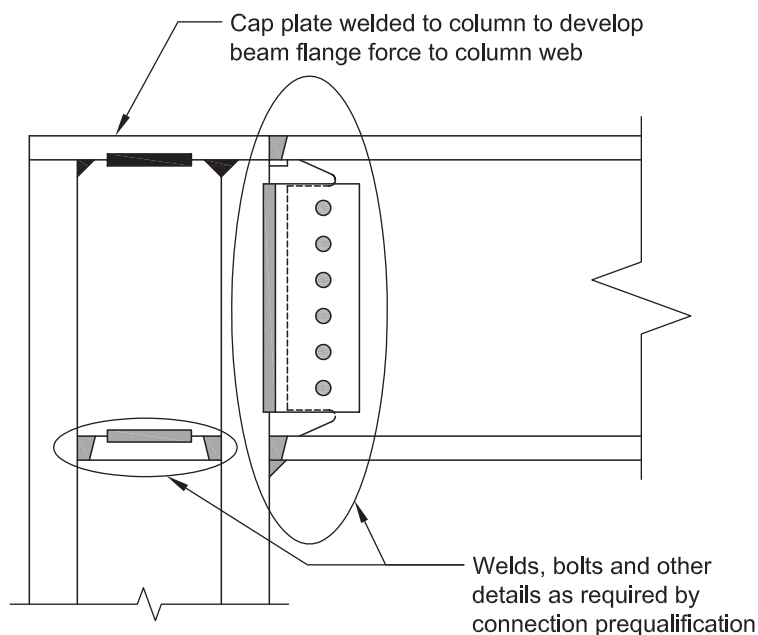


Fig. C-2.2. Example cap plate detail at column top for RBS connection.

moment-connected beam attaches to only one of the column flanges, it is generally not necessary to use CJP groove welds to attach the continuity plate to the column flange that does not have a beam attached. In such cases, acceptable performance can often be obtained by attaching the continuity plate to the column with a pair of minimum-size fillet welds.

When beams are moment connected to the side plates of boxed wide-flange column sections, continuity plates or cap plates should always be provided opposite the beam flanges, as is required for box section columns.

CHAPTER 3

WELDING REQUIREMENTS

3.3. BACKING AT BEAM-TO-COLUMN AND CONTINUITY PLATE-TO-COLUMN JOINTS

At the root of groove welds between beam flanges or continuity plates and column flanges, the inherent lack of a fusion plane between the left-in-place steel backing and the column flange creates a stress concentration and notch effect, even when the weld has uniform and sound fusion at the root. Further, when ultrasonic testing is performed, this left-in-place backing may mask significant flaws that may exist at the weld root. These flaws may create a more severe notch condition than that caused by the backing itself (Chi et al., 1997).

1. Steel Backing at Continuity Plates

The stress and strain level at the groove weld between a continuity plate and column flange is considerably different than that at the beam flange-to-column flange connection; therefore, it is not necessary to remove the backing. The addition of the fillet weld beneath the backing makes the inherent notch at the interface an internal notch, rather than an external notch, reducing the notch effect. When backing is removed, the required reinforcing fillet weld reduces the stress concentration at the right-angle intersection of the continuity plate and the column flange.

2. Steel Backing at Beam Bottom Flange

The removal of backing, whether fusible or nonfusible, followed by backgouging to sound weld metal, is required so that potential root defects within the welded joint are detected and eliminated, and the stress concentration at the weld root is eliminated.

The influence of left-in-place steel backing is more severe on the bottom flange, as compared to the top flange, because at the bottom flange, the stress concentration from the backing occurs at the point of maximum applied and secondary tensile stresses in the groove weld, at the weld root, and at the outer fiber of the beam flange.

A reinforcing fillet weld with a $\frac{5}{16}$ -in. (8-mm) leg on the column flange helps to reduce the stress concentration at the right-angle intersection of the beam flange and column flange and is placed at the location of maximum stress. The fillet weld's horizontal leg may need to be larger than $\frac{5}{16}$ in. (8 mm) to completely cover the weld root area, eliminating the potential for multiple weld toes at the root that serve as small stress concentrations and potential fracture initiation points. When grinding the weld root and base metal area, previously deposited weld toe regions and their associated

fracture initiation sites are removed, and the horizontal leg of the fillet weld need not be extended to the base metal.

3. Steel Backing at Beam Top Flange

Because of differences in the stress and strain conditions at the top and bottom flange connections, the stress/strain concentration and notch effect created by the backing/column interface at the top flange is at a lower level, compared to that at the bottom flange. Therefore, backing removal is not required. The addition of the reinforcing fillet weld makes the inherent notch at the interface an internal notch, rather than an external notch, further reducing the effect. Because backing removal, backgouging and backwelding would be performed through an access hole beneath the top flange, these operations should be avoided whenever possible.

4. Prohibited Welds at Steel Backing

Tack welds for beam flange-to-column connections should be made within the weld groove. Tack welds or fillet welds to the underside of the beam at the backing would direct stress into the backing itself, increasing the notch effect at the backing/column flange interface. In addition, the weld toe of the tack weld or fillet weld on the beam flange would act as a stress concentration and a potential fracture initiation site.

Proper removal of these welds is necessary to remove the stress concentration and potential fracture initiation site. Any repair of gouges and notches by filling with weld metal must be made using filler metals with the required notch toughness.

5. Nonfusible Backing at Beam Flange-to-Column Joints

After nonfusible backing is removed, backgouging to sound metal removes potential root flaws within the welded joint. A reinforcing fillet weld with a $\frac{5}{16}$ -in. (8-mm) leg on the column flange helps reduce the stress concentration at the right-angle intersection of the beam flange and column flange.

The fillet weld's horizontal leg may need to be larger than $\frac{5}{16}$ in. (8 mm) to completely cover the weld root area, eliminating the potential for small stress concentrations and potential fracture initiation points. When grinding the weld root and base metal area, previously deposited weld toe regions and their associated fracture initiation sites are removed; therefore, the horizontal leg of the fillet weld need not be extended to base metal.

3.4. WELD TABS

Weld tabs are used to provide a location for initiation and termination of welds outside the final weld location, improving the quality of the final weld. The removal of weld tabs is performed to remove the weld discontinuities and defects that may be present at these start and stop locations. Because weld tabs are located at the ends of welds, any remaining weld defects at the weld-end removal areas may act as external notches and fracture initiation sites and are, therefore, removed. A smooth transition is needed between base metal and weld to minimize stress concentrations.

3.5. TACK WELDS

Tack welds outside weld joints may create unintended load paths and may create stress concentrations that become crack initiation sites when highly strained. By placing tack welds within the joint, the potential for surface notches and hard heat affected zones (HAZs) is minimized. When placed within the joint, the HAZ of a tack weld is tempered by the subsequent passes for the final weld.

Tack welds for beam flange-to-column connections are preferably made in the weld groove. Tack welds of backing to the underside of beam flanges would be unacceptable, and any tack welds between weld backing and beam flanges are to be removed in accordance with Section 3.3.4. Steel backing may be welded to the column under the beam flange, where a reinforcing fillet is typically placed.

When tack welds for the attachment of weld tabs are placed within the weld joint, they become part of the final weld.

3.6. CONTINUITY PLATES

The rotary straightening process used by steel rolling mills to straighten rolled sections cold works the webs of these shapes in and near the k -area. This cold working can result in an increase in hardness, yield strength, ultimate tensile strength, and yield-to-tensile ratio and a decrease in notch toughness. In some instances, Charpy V-notch toughness has been recorded to be less than 2 ft-lb at 70°F [3 J at 20°C] (Barsom and Korvink, 1998). These changes do not negatively influence the in-service behavior of uncracked shapes. However, the potential for post-fabrication k -area base metal cracking exists in highly restrained joints at the weld terminations for column continuity plates, web doublers, and thermal cut coped beams.

When the minimum clip dimensions are used along the member web, the available continuity plate length must be considered in the design and detailing of the welds to the web. For fillet welds, the fillet weld should be held back one to two weld sizes from each clip. For groove welds, weld tabs should not be used in the k -area because they could cause base metal fracture from the combination of weld shrinkage, the stress concentration/notch effect at the weld end, and the low notch-toughness web material.

When the maximum clip dimensions are used along the member flange, the width—hence, the capacity—of the continuity plate is not reduced substantially. Care must be used in making quality weld terminations near the member radius, as the use of common weld tabs is difficult. If used, their removal in this region may damage the base metal, necessitating difficult repairs. The use of cascaded ends within the weld groove may be used within the dimensional limits stated. Because of the incomplete filling of the groove, the unusual configuration of the weld, and the relatively low level of demand placed upon the weld at this location, nondestructive testing of cascaded weld ends in groove welds at this location is not required.

3.7. QUALITY CONTROL AND QUALITY ASSURANCE

Chapter J of the AISC *Seismic Provisions* specifies the minimum requirements for a quality assurance plan for the seismic force-resisting system. It may be appropriate to supplement the Chapter J provisions with additional requirements for a particular project based on the qualifications of the contractor(s) involved and their demonstrated ability to produce quality work. Contract documents are to define the quality control (QC) and quality assurance (QA) requirements for the project.

QC includes those tasks to be performed by the contractor to ensure that their materials and workmanship meet the project's quality requirements. Routine welding QC items include personnel control, material control, preheat measurement, monitoring of welding procedures, and visual inspection.

QA includes those tasks to be performed by an agency or firm other than the contractor. QA includes monitoring of the performance of the contractor in implementing the contractor's QC program, ensuring that designated QC functions are performed properly by the contractor on a routine basis. QA may also include specific inspection tasks that are included in the contractor's QC plan, and may include nondestructive testing of completed joints.

CHAPTER 4

BOLTING REQUIREMENTS

4.1. FASTENER ASSEMBLIES

ASTM F3125 Grade F1852 twist-off type tension-control fastener assemblies are appropriate equivalents for ASTM F3125 Grade A325 or A325M bolts. ASTM F3125 Grade F2280 twist-off type tension control fastener assemblies are appropriate substitutes for ASTM F3125 Grade A490 or A490M bolts. Such assemblies are commonly produced and used and are addressed by the RCSC *Specification for Structural Joints Using High-Strength Bolts* (RCSC, 2014).

4.2. INSTALLATION REQUIREMENTS

Section D2 of the AISC *Seismic Provisions* designates all bolted joints to be pretensioned joints, with the additional requirement that the joint's faying surfaces meet Class A conditions for slip-critical joints. Some connection types designate the bolted joint to be designed as slip-critical, and others waive the faying surface requirements of the AISC *Seismic Provisions*.

4.3. QUALITY CONTROL AND QUALITY ASSURANCE

See Commentary Section 3.7.

CHAPTER 5

REDUCED BEAM SECTION (RBS) MOMENT CONNECTION

5.1. GENERAL

In a reduced beam section (RBS) moment connection, portions of the beam flanges are selectively trimmed in the region adjacent to the beam-to-column connection. In an RBS connection, yielding and hinge formation are intended to occur primarily within the reduced section of the beam and thereby limit the moment and inelastic deformation demands developed at the face of the column.

A large number of RBS connections have been tested under a variety of conditions by different investigators at institutions throughout the world. A listing of relevant research is presented in the references at the end of this document. Review of available test data indicates that RBS specimens, when designed and constructed according to the limits and procedures presented herein, have developed interstory drift angles of at least 0.04 rad under cyclic loading on a consistent basis. Tests on RBS connections show that yielding is generally concentrated within the reduced section of the beam and may extend, to a limited extent, to the face of the column. Peak strength of specimens is usually achieved at an interstory drift angle of approximately 0.02 to 0.03 rad. Specimen strength then gradually reduces due to local and lateral-torsional buckling of the beam. Ultimate failure typically occurs at interstory drift angles of approximately 0.05 to 0.07 rad, by low cycle fatigue fracture at local flange buckles within the RBS.

RBS connections have been tested using single-cantilever type specimens (one beam attached to a column), and double-sided specimens (specimen consisting of a single column with beams attached to both flanges). Tests have been conducted primarily on bare-steel specimens, although some testing is also reported on specimens with composite slabs. Tests with composite slabs have shown that the presence of the slab provides a beneficial effect by helping to maintain the stability of the beam at larger interstory drift angles.

Most RBS test specimens were tested pseudo-statically, using a loading protocol in which applied displacements are progressively increased, such as the loading protocol specified in ATC-24 (ATC, 1992) and the loading protocol developed in the FEMA/SAC program and adopted in Chapter K of the AISC *Seismic Provisions*. Two specimens were tested using a loading protocol intended to represent near-source ground motions that contain a large pulse. Several specimens were also tested dynamically. Radius-cut RBS specimens have performed well under all of these loading conditions. See Commentary Section 5.7 for a discussion of other shapes of RBS cuts.

5.2. SYSTEMS

Review of the research literature presented in the reference section at the end of this document and summarized in Commentary Section 5.1 indicates that the radius-cut RBS connection meets the prequalification requirements in Section K1 of the AISC *Seismic Provisions* for special and intermediate moment frames.

5.3. PREQUALIFICATION LIMITS

1. Beam Limitations

A wide range of beam sizes has been tested with the radius-cut RBS. The smallest beam size reported in the literature was a Canadian W530×82, roughly equivalent to a W21×50. The heaviest beam reported was a W36×300 (W920×446) (FEMA, 2000e), which is no longer produced. Although the AISC *Seismic Provisions* permit limited increases in beam depth and weight compared to the maximum sections tested, the prequalification limits for maximum beam depth and weight were established based on the test data for a W36×300 (W920×446). It was the judgment of the CPRP that for the purposes of establishing initial prequalification limits, adherence to the maximum tested specimen would be appropriately conservative. There is no evidence that modest deviations from the maximum tested specimen would result in significantly different performance, and the limit on maximum flange thickness is approximately 4% thicker than the 1.68-in. (43-mm) flange in a W36×300 (W920×446).

Beam depth and beam span-to-depth ratio are significant in the inelastic behavior of beam-to-column connections. For the same induced curvature, deep beams will experience greater strains than shallower beams. Similarly, beams with a shorter span-to-depth ratio will have a sharper moment gradient across the beam span, resulting in reduced length of the beam participating in plastic hinging and increased strains under inelastic rotation demands. Most of the beam-to-column assemblies that have been tested had configurations approximating beam spans of about 25 ft (7.6 m) and beam depths varying from W30 (W760) to W36 (W920) so that beam span-to-depth ratios were typically in the range of eight to ten (FEMA, 2000e). Given the degree to which most specimens significantly exceeded the minimum interstory drift demands, it was judged reasonable to set the minimum span-to-depth ratio at seven for SMF and five for IMF.

Local buckling requirements for members subjected to significant inelastic rotation are covered in the AISC *Seismic Provisions*. For the purposes of calculating the width-to-thickness ratio, it is permitted to take the flange width at the two-thirds point of the RBS cut. This provision recognizes that the plastic hinge of the beam forms within the length of the RBS cut, where the width of the flange is less than at the uncut section. This provision will result in a lower width-to-thickness ratio when taken at the RBS cut compared to that at the uncut section. Many of the RBS tests conducted as a part of the FEMA/SAC program used a W30×99 (W760×147) beam that does not quite satisfy the flange width-to-thickness ratio at the uncut section. Nevertheless, the tests were successful. For these reasons, it was judged reasonable to permit the calculation of the width-to-thickness ratio a reasonable distance into the RBS cut.

In developing this prequalification, the CPRP also reviewed lateral bracing requirements for beams with RBS connections. Some concerns were raised in the past that the presence of the RBS flange cuts might make the beam more prone to lateral-torsional buckling and that supplemental lateral bracing should be provided at the RBS. The issue of lateral bracing requirements for beams with RBS connections was subsequently investigated in both experimental and analytical studies (FEMA, 2000f; Yu et al., 2000). These studies indicated that for bare steel specimens (no composite slab), interstory drift angles of 0.04 rad can be achieved without a supplemental lateral brace at the RBS, as long as the normal lateral bracing required for beams in SMF systems is provided in accordance with Section D1.2b of the AISC *Seismic Provisions*.

Studies also indicated that although supplemental bracing is not required at the RBS to achieve 0.04-rad interstory drift angles, the addition of a supplemental brace can result in improved performance. Tests on RBS specimens with composite slabs indicated that the presence of the slab provided a sufficient stabilizing effect that a supplemental brace at the RBS is not likely to provide significantly improved performance (FEMA, 2000f; Engelhardt, 1999; Tremblay et al., 1997). Based on the available data, beams with RBS connections that support a concrete structural slab are not required to have a supplemental brace at the RBS.

In cases where a supplemental brace is provided, the brace should not be connected within the reduced section (protected zone). Welded or bolted brace attachments in this highly strained region of the beam may serve as fracture initiation sites. Consequently, if a supplemental brace is provided, it should be located at or just beyond the end of the RBS that is farthest from the face of the column.

The protected zone is defined as shown in Figure 5.1 and extends from the face of the column to the end of the RBS farthest from the column. This definition is based on test observations that indicate yielding typically does not extend past the far end of the RBS cut.

2. Column Limitations

Nearly all tests of RBS connections have been performed with the beam flange welded to the column flange (i.e., strong-axis connections). The limited amount of weak-axis testing has shown acceptable performance. In the absence of more tests, the CPRP recommended limiting prequalification to strong-axis connections only.

The majority of RBS specimens were constructed with W14 (W360) columns. However, a number of tests have also been conducted using deeper columns, including W18, W27 and W36 (W460, W690 and W920) columns. Testing of deep-column RBS specimens under the FEMA/SAC program indicated that stability problems may occur when RBS connections are used with deep columns (FEMA, 2000f). In FEMA 350 (FEMA, 2000b), RBS connections were only prequalified for W12 (W310) and W14 (W360) columns.

The specimens in the FEMA/SAC tests conducted showed a considerable amount of column twisting (Gilton et al., 2000). However, two of the three specimens tested achieved 0.04-rad rotation, albeit with considerable strength degradation. The third specimen just fell short of 0.04-rad rotation and failed by fracture of the column web near the k -area. Subsequent study attributed this fracture to column twisting.

Subsequent to the FEMA/SAC tests, an analytical study (Shen et al., 2002) concluded that boundary conditions used in these tests may not be representative of what would be found in an actual building. Consequently, the large-column twisting (and, presumably, resultant k -area column fracture) seen in the FEMA/SAC tests would not be expected in real buildings. The study also concluded that deep columns should not behave substantially different from W14 (W360) columns and that no special bracing is needed when a slab is present. This was followed by a more extensive analytical and large-scale experimental investigation on RBS connections with columns up to W36 (W920) in depth (Ricles et al., 2004). This investigation showed that good performance can be achieved with deep columns when a composite slab is present or when adequate lateral bracing is provided for the beam and/or column in the absence of a slab. Based on a review of this recent research, the prequalification of RBS connections is extended herein to include W36 (W920) columns.

The behavior of RBS connections with cruciform columns is expected to be similar to that of a rolled wide-flange column because the beam flange frames into the column flange, the principal panel zone is oriented parallel to that of the beam, and the web of the cut wide-flange column is to be welded with a CJP groove weld to the continuous web 1 ft above and below the depth of the frame girder. Given these similarities and the lack of evidence suggesting behavioral limit states different from those associated with rolled wide-flange shapes, cruciform column depths are limited to those imposed on wide-flange shapes.

Successful tests have also been conducted on RBS connections with built-up box columns. The largest box column for which test data were available was 24 in. by 24 in. (600 mm by 600 mm). Consequently, RBS connections have been prequalified for use with built-up box columns up to 24 in. (600 mm). Limits on the width-to-thickness ratios for the walls of built-up box columns are specified in Section 2.3.2b(3) and were chosen to reasonably match the box columns that have been tested.

The use of box columns participating in orthogonal moment frames—that is, with RBS connections provided on orthogonal beams—is also prequalified. Although no data were available for test specimens with orthogonal beams, this condition should provide ostensibly the same performance as single-plane connections, because the RBS does not rely on panel zone yielding for good performance and the column is expected to remain essentially elastic for the case of orthogonal connections.

Based on successful tests on wide-flange columns and on built-up box columns, boxed wide-flange columns would also be expected to provide acceptable performance. Consequently, RBS connections are prequalified for use with boxed wide-flange columns. When moment connections are made only to the flanges of the wide-flange portion of the boxed wide-flange, the column may be up to W36 (W920) in depth.

When the boxed wide-flange column participates in orthogonal moment frames, then neither the depth nor the width of the column is allowed to exceed 24 in. (600 mm), applying the same limits as for built-up boxes.

5.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

Column panel zone strength provided on RBS test specimens has varied over a wide range. This includes specimens with very strong panel zones (no yielding in the panel zone), specimens with very weak panel zones (essentially all yielding in the panel zone and no yielding in the beam), and specimens where yielding has been shared between the panel zone and the beam. Good performance has been achieved for all levels of panel zone strength (FEMA, 2000f), including panel zones that are substantially weaker than permitted in AISC *Seismic Provisions* Section E3.6e. However, there are concerns that very weak panel zones may promote fracture in the vicinity of the beam-flange groove welds due to “kinking” of the column flanges at the boundaries of the panel zone. Consequently, the minimum panel zone strength specified in Section E3.6e of the AISC *Seismic Provisions* is required for prequalified RBS connections.

5.5. BEAM FLANGE-TO-COLUMN FLANGE WELD LIMITATIONS

Complete-joint-penetration groove welds joining the beam flanges to the column flanges provided on the majority of RBS test specimens have been made by the self-shielded flux cored arc welding process (FCAW-S) using electrodes with a minimum specified Charpy V-notch toughness. Three different electrode designations have commonly been used in these tests: E71T-8, E70TG-K2 and E70T-6. Further, for most specimens, the bottom flange backing was removed and a reinforcing fillet added, top flange backing was fillet welded to the column, and weld tabs were removed at both the top and bottom flanges.

Test specimens have employed a range of weld access-hole geometries, and results suggest that connection performance is not highly sensitive to the weld access-hole geometry. Consequently, prequalified RBS connections do not require specific access-hole geometry. Weld access holes should satisfy the requirements of Section 6.11 of AWS D1.8/D1.8M (AWS, 2016). The alternative geometry for weld access holes specified in Section 6.11.1.2 of AWS D1.8/D1.8M is not required for RBS connections.

5.6. BEAM WEB-TO-COLUMN CONNECTION LIMITATIONS

Two types of web connection details have been used for radius-cut RBS test specimens: a welded and a bolted detail. In the welded detail, the beam web is welded directly to the column flange using a complete-joint-penetration (CJP) groove weld. For the bolted detail, pretensioned high-strength bolts are used. Specimens with both types of web connections have achieved at least 0.04-rad interstory drift angles, and consequently, both types of web connection details were permitted for RBS connections in FEMA 350 (2000b).

Previous test data (Engelhardt et al., 2000) indicate that beyond an interstory drift angle of 0.04 rad, specimens with bolted web connections show a higher incidence of fracture occurring near the beam-flange groove welds, as compared to specimens with welded web connections. Thus, while satisfactory performance is possible with a bolted web connection, previous test data indicate that a welded web is beneficial in reducing the vulnerability of RBS connections to fracture at the beam-flange groove welds.

Subsequent to the SAC/FEMA testing on RBS connections, a test program (Lee et al., 2004) was conducted that directly compared nominally identical RBS connections except for the web connection detail. The RBS specimens with welded web connections achieved a 0.04-rad interstory drift angle, whereas RBS specimens with bolted web connections failed to achieve 0.04 rad.

Thus, while past successful tests have been conducted on RBS connections with bolted web connections, recent data have provided contradictory evidence, suggesting bolted web connections may not be suitable for RBS connections when used for SMF applications. Until further data are available, a welded web connection is required for RBS connections prequalified for use in SMF. For IMF applications, bolted web connections are acceptable.

The beam web-to-plate CJP groove weld is intended to extend the full distance between the weld access holes to minimize the potential for crack-initiation at the ends of the welds—hence, the requirement for the plate to extend from one weld access hole to the other. All specimens were tested with the full-depth weld configuration.

5.7. FABRICATION OF FLANGE CUTS

Various shapes of flange cutouts are possible for RBS connections, including a constant cut, a tapered cut, and a radius cut. Experimental work has included successful tests on all of these types of RBS cuts. The radius cut avoids abrupt changes of cross section, reducing the chances of a premature fracture occurring within the reduced section. Further, the majority of tests reported in the literature used radius-cut RBS sections. Consequently, only the radius-cut RBS shape is prequalified.

An issue in the fabrication of RBS connections is the required surface finish and smoothness of the RBS flange cuts. No research data was found that specifically addressed this issue. Consequently, finish requirements for RBS cuts were chosen by the CPRP based on judgment and are consistent with those specified in FEMA 350 (2000b).

5.8. DESIGN PROCEDURE

Dimensions of the RBS cuts for the test specimens reported in the literature vary over a fairly small range. The distance from the face of the column to the start of the RBS radius cut (designated as a in Figure 5.1) ranged from 50 to 75% of the beam-flange width. The length of the cuts (designated as b in Figure 5.1) varied from approximately 75 to 85% of the beam depth. The amount of flange width removed at

the minimum section of the RBS varied from about 38 to 55%. Flange removal for prequalified RBS connections is limited to a maximum of 50% to avoid excessive loss of strength or stiffness.

The design procedure presented herein for prequalified RBS connections is similar to that presented in FEMA 350 (2000b). The overall basis for sizing the RBS radius cut in this design procedure is to limit the maximum beam moment that can develop at the face of the column to the actual plastic moment (based on expected yield stress) of the beam when the minimum section of the RBS is fully yielded and strain hardened. Test data indicate that connecting the beam at the face of the column in accordance with the requirements herein allows the connection to resist this level of moment while minimizing the chance of fracture at the beam-flange groove welds.

Step 4 of the design procedure requires computation of the shear force at the center of the RBS radius cut. This shear force is a function of the gravity load on the beam and the plastic moment capacity of the RBS. An example calculation is shown in Figure C-5.1 for the case of a beam with a uniformly distributed gravity load.

In **Step 5**, Equation 5.8-6 neglects the gravity load on the portion of the beam between the center of the reduced beam section and the face of the column. If desired, the

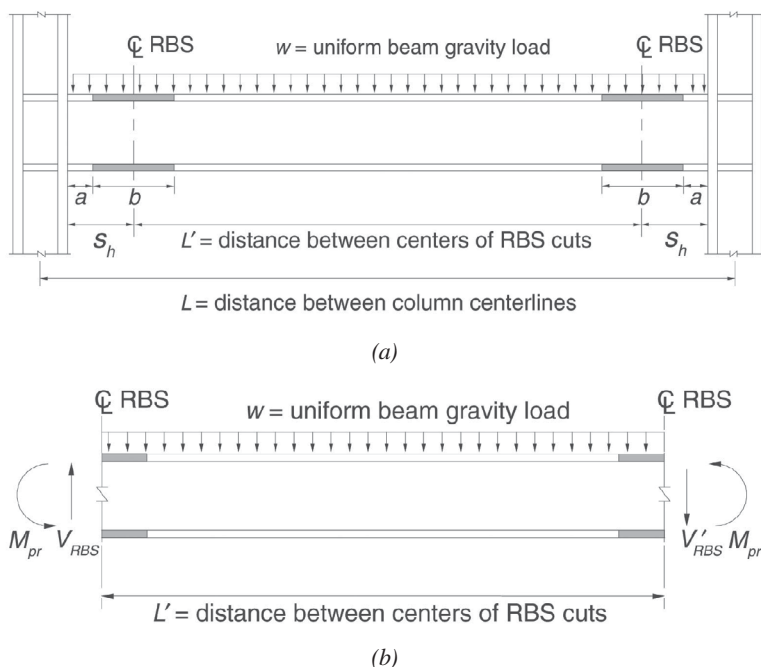


Fig. C-5.1. Example calculation of shear at center of RBS cuts: (a) beam with RBS cuts and uniform gravity load; (b) free-body diagram of beam between RBS cuts and calculation of shear at RBS.

gravity load on this small portion of the beam is permitted to be included in the free-body diagram shown in Figure 5.2 and in Equation 5.8-6.

For gravity load conditions other than a uniform load, the appropriate adjustment should be made to the free-body diagram in Figure C-5.1 and to Equations C-5.8-1 and C-5.8-2.

$$V_{RBS} = \frac{2M_{pr}}{L_h} + \frac{wL_h}{2} \quad (\text{C-5.8-1})$$

$$V'_{RBS} = \frac{2M_{pr}}{L_h} - \frac{wL_h}{2} \quad (\text{C-5.8-2})$$

Equations C-5.8-1 and C-5.8-2 assume that plastic hinges will form at the RBS at each end of the beam. If the gravity load on the beam is very large, the plastic hinge at one end of the beam may move toward the interior portion of the beam span. If this is the case, the free-body diagram in Figure C-5.1 should be modified to extend between the actual plastic hinge locations. To determine whether Equations C-5.8-1 and C-5.8-2 are valid, the moment diagram for the segment of the beam shown in Figure C-5.1(b)—that is, for the segment of the beam between the centers of the RBS cuts—is drawn. If the maximum moment occurs at the ends of the span, then Equations C-5.8-1 and C-5.8-2 are valid. If the maximum moment occurs within the span and exceeds M_{pe} of the beam (see Equation 5.8-7), then the modification described above will be needed.

Nearly all moment frame connection tests have been performed on single- or double-sided beam-column subassemblies with the beam perpendicular to the vertical axis of the column (i.e., a level beam). Nevertheless, sloping beams occur in most structures, such as at the roof. This Standard does not contain provisions that explicitly address sloped beams because of the lack of systematic physical testing. Professional judgment, therefore, is required to determine whether a proposed frame beam slope is appropriately covered by the prequalification limits in this Standard.

Shallow slopes are not thought to significantly reduce connection performance, but identifying a threshold above which this is no longer true has not been determined. Testing of frame beams using reduced beam sections sloping at 28 degrees (Ball et al., 2010) indicates that at this angle the performance of the connection is adversely impacted unless adjustments are made in the geometry of the reduced beam section cut. When the a dimension was equal at the top and bottom flanges (i.e., the centerline of the RBS is parallel to the vertical axis of the column), physical testing produced fracture of the beam flange CJP weld at the toe location in both specimens tested at lower than anticipated drift demands. Finite element analyses by Kim et al. (2010) showed that connections with the centerline of the RBS connection perpendicular to the beam flanges (i.e., using unequal a dimensions) reduces the demand on the beam flange weld at the toe location, see Figure C-5.2.

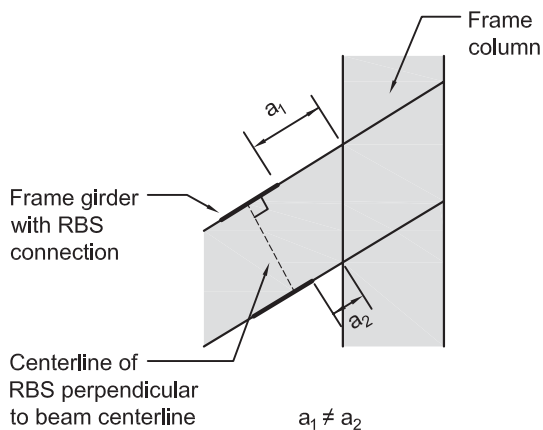


Fig. C-5.2. Analytical studies (Kim et al., 2010) suggest that sloped frame beam with RBS centerline perpendicular to beam flanges perform better than if RBS centerline is parallel to column centerline.

CHAPTER 6

BOLTED UNSTIFFENED AND STIFFENED EXTENDED END-PLATE MOMENT CONNECTIONS

6.1. GENERAL

The three extended end-plate moment configurations currently addressed in this chapter are the most commonly used end-plate connection configurations in steel moment frames. AISC Design Guide 4, *Extended End-Plate Moment Connections, Seismic and Wind Applications* (Murray and Sumner, 2003) provides background, design procedures, and complete design examples for the three configurations. The guide was developed before this Standard was written, and there are small differences between the design procedures in the guide and in Section 6.8. The primary differences are in the resistances factors. The Standard supersedes the design guide in all instances.

Prequalification test results for the three extended end-plate moment connections are found in FEMA (1997); Meng (1996); Meng and Murray (1997); Ryan and Murray (1999); Sumner et al. (2000a); Sumner et al. (2000b); Sumner and Murray (2001); and Sumner and Murray (2002). Results of similar testing but not used for prequalification are found in Adey et al. (1997); Adey et al. (1998); Adey et al. (2000); Castellani et al. (1998); Coons (1999); Ghobarah et al. (1990); Ghobarah et al. (1992); Johnstone and Walpole (1981); Korol et al. (1990); Popov and Tsai (1989); and Tsai and Popov (1990).

The intent of the design procedure in Section 6.8 is to provide an end-plate moment connection with sufficient strength to develop the strength of the connected flexural member. The connection does not provide any contribution to inelastic rotation. All inelastic deformation for an end-plate connection is achieved by beam yielding and/or column panel zone deformation.

The design procedure in Section 6.8 is based on Borgsmiller and Murray (1995) and is similar to the “thick plate” procedure in AISC Design Guide 16, *Flush and Extended Multiple-Row Moment End-Plate Connections* (Murray and Shoemaker, 2002). The procedure is basically the same as that in FEMA 350 (2000b), but with much clarification. Applicable provisions in FEMA 353 (2000d) are incorporated into the procedure as well.

6.2. SYSTEMS

The three extended end-plate moment connections in Figure 6.1 are prequalified for use in IMF and SMF systems, except in SMF systems where the beam is in direct contact with concrete structural slabs. The exception applies only when shear studs are used to attach the concrete slab to the connected beam and is because of the lack of test data to date. Prequalification testing has generally been performed with bare

steel specimens. Sumner and Murray (2002) performed one test in which a slab was present. In this test, headed studs were installed from near the end-plate moment connection to the end of the beam, and the concrete was in contact with the column flanges and web. The lower bolts failed prematurely by tension rupture because of the increase in the distance from the neutral axis due to the presence of the composite slab. In later testing, Murray repeated this test but placed a flexible material between the vertical face of the end plate and the slab to inhibit slab participation in transfer of load to the column. This specimen performed acceptably and resulted in provisions for using concrete structural slabs when such flexible material is placed between the slab and the plate.

6.3. PREQUALIFICATION LIMITS

The parametric limitations in Table 6.1 were determined from reported test data in the prequalification references. Only connections that are within these limits are prequalified.

For tapered members, the depth of the beam at the connection is used to determine the limiting span-to-depth ratio.

1. Beam Limitations

The beam size limitations in Table 6.1 are directly related to connection testing. Because many of the tested beam sections were built-up members, the limitations are in cross-section dimensions instead of rolled-beam designations. There is no evidence that modest deviations from these dimensions will result in significantly different performance.

Similar to RBS testing, most of the tested beam-column assemblies had configurations approximating beam span-to-depth ratios in the range of 8 to 10. However, it was judged reasonable to set the minimum span-to-depth ratio at 7 for SMF and 5 for IMF.

The protected zone requirements are based on test observations.

2. Column Limitations

Extended end-plate moment connections may be used only with rolled or built-up I-shaped sections and must be flange connected. There are no other specific column requirements for extended end-plate moment connections.

6.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

There are no specific column-to-beam relationship limitations for extended end-plate moment connections.

6.5. CONTINUITY PLATES

Continuity plate design must conform to the requirements of Section 2.4.4. The design procedure in Section 6.8 contains provisions specific to extended end-plate

moment connections, and the procedure is discussed generally in AISC Design Guide 13, *Wide-Flange Column Stiffening at Moment Connections* (Carter, 1999).

6.6. BOLTS

Prequalification tests have been conducted with both pretensioned ASTM F3125 Grade A325 and A490 bolts. Bolt length should be such that at least two complete threads are between the unthreaded portion of the shank and the face of the nut after the bolt is pretensioned. Slip-critical connection provisions are not required for end-plate moment connections.

6.7. CONNECTION DETAILING

Maximum gage—that is, the horizontal distance between outer bolt columns—is limited to the width of the beam flange to ensure a stiff load path. Monotonic tests have shown that the stiffness and strength of an end-plate moment connection are decreased when the bolt gage is wider than the beam flange.

Inner bolt pitch—the distance between the face of the beam flange and the first row of inside or outside bolts—must be sufficient to allow bolt tightening. The minimum pitch values specified have been found to be satisfactory. An increase in pitch distance can significantly increase the required end-plate thickness.

The end-plate can be wider than the beam flange, but the width used in design calculations is limited to the beam flange width plus 1 in. (25 mm). This limitation is based on the CPRP's assessment of unpublished results of monotonic tests of end-plate connections.

The requirements for the length of beam-flange-to-end-plate stiffeners are established to ensure a smooth load path. The 30° angle is the same as used for determining the Whitmore section width in other types of connections. The required 1-in. (25-mm) land is needed to ensure the quality of the vertical and horizontal weld terminations.

Tests have shown that the use of finger shims between the end-plate and the column flange do not affect the performance of the connection (Sumner et al., 2000a).

Design procedures are not available for connections of beams with composite action at an extended end-plate moment connection. Therefore, careful composite slab detailing is necessary to prevent composite action that may increase tension forces in the lower bolts. Welded steel stud anchors are not permitted within 1½ times the beam depth, and compressible material is required between the concrete slab and the column face (Sumner and Murray, 2002; Yang et al., 2003).

Cyclic testing has shown that use of weld access holes can cause premature fracture of the beam flange at extended end-plate moment connections (Meng and Murray, 1997). Short to long weld access holes were investigated with similar results. Therefore, weld access holes are not permitted for extended end-plate moment connections.

Strain gage measurements have shown that the web plate material in the vicinity of the inside tension bolts generally reaches the yield strain (Murray and Kukreti, 1988).

Consequently, it is required that the web-to-end-plate weld(s) in the vicinity of the inside bolts be sufficient to develop the strength of the beam web.

The beam-flange-to-end-plate and stiffener weld requirements equal or exceed the welding that was used to prequalify the three extended end-plate moment connections. Because weld access holes are not permitted, the beam-flange-to-end plate weld at the beam web is necessarily a partial-joint-penetration (PJP) groove weld. The prequalification testing has shown that these conditions are not detrimental to the performance of the connection.

6.8. DESIGN PROCEDURE

The design procedure in this section, with some modification, was used to design the prequalification test specimens. The procedure is very similar to that in AISC Design Guide 4 (Murray and Sumner, 2003), except that different resistance factors are used. Example calculations are found in the design guide. Column stiffening example calculations are found in AISC Design Guide 13 (Carter, 1999).

CHAPTER 7

BOLTED FLANGE PLATE (BFP) MOMENT CONNECTION

7.1. GENERAL

The bolted flange plate (BFP) connection is a field-bolted connection. The fundamental seismic behaviors expected with the BFP moment connection include:

- (1) Initial yielding of the beam at the last bolt away from the face of the column.
- (2) Slip of the flange plate bolts, which occurs at similar resistance levels to the initial yielding in the beam flange, but the slip does not contribute greatly to the total deformation capacity of the connection.
- (3) Secondary yielding in the column panel zone, which occurs as the expected moment capacity and strain hardening occur.
- (4) Limited yielding of the flange plate, which may occur at the maximum deformations.

This sequence of yielding has resulted in very large inelastic deformation capacity for the BFP moment connection, but the design procedure is somewhat more complex than some other prequalified connections.

The flange plates and web shear plate are shop-welded to the column flange and field-bolted to the beam flanges and web, respectively. ASTM F3125 Grade A490 or A490M bolts with threads excluded from the shear plane are used for the beam flange connections because the higher shear strength of the Grade A490 or A490M bolts reduces the number of bolts required and reduces the length of the flange plate. The shorter flange plates that are, therefore, possible reduce the seismic inelastic deformation demands on the connection and simplify the balance of the resistances required for different failure modes in the design procedure. Flange plate connections with ASTM F3125 Grade A325 or A325M bolts may be possible but will be more difficult to accomplish because of the reduced bolt strength, greater number of bolts, and longer flange plates required. As a result, the connection is not prequalified for use with Grade A325 or A325M bolts.

Prequalification of the BFP moment connection is based upon 20 BFP moment connection tests under cyclic inelastic deformation (FEMA, 2000e; Schneider and Teeraparbpong, 1999; Sato et al., 2008). Additional evidence supporting prequalification is derived from bolted T-stub connection tests (FEMA, 2000e; Swanson et al., 2000), because the BFP moment connection shares many yield mechanisms, failure modes, and connection behaviors with the bolted T-stub connection. The tests were performed under several deformation-controlled test protocols, but most use

variations of the ATC-24 (ATC, 1992) or the SAC steel protocol (Krawinkler et al., 2000), which are both very similar to the prequalification test protocol of Chapter K of the AISC *Seismic Provisions* (AISC, 2016a). The 20 BFP tests were performed on connections with beams ranging in depth from W8 (W200) to W36 (W920) sections, and the average total demonstrated ductility capacity exceeded 0.057 rad. Hence, the inelastic deformation capacity achieved with BFP moment connections is among the best achieved from seismic testing of moment frame connections. However, the design of the connection is relatively complex because numerous yield mechanisms and failure modes must be considered in the design process. Initial and primary yielding in the BFP moment connection is flexural yielding of the beam near the last row of bolts at the end of the flange plate. However, specimens with the greatest ductility achieve secondary yielding through shear yielding of the column panel zone and limited tensile yielding of the flange plate. Hence, a balanced design that achieves yielding from multiple yield mechanisms is encouraged.

Most past tests have been conducted on specimens with single-sided connections, and the force-deflection behavior is somewhat pinched as shown in Figure C-7.1. Because plastic hinging at the end of the flange plate is the controlling yield mechanism, the expected plastic moment at this location dominates the connection design. The pinching is caused by a combination of bolt slip and the sequence of yielding and strain hardening encountered in the connection. Experiments have shown that the expected peak moment capacity at the plastic hinge is typically on the order of 1.15 times the expected M_p of the beam, as defined in the AISC *Seismic Provisions*, and the expected moment at the face of the column is on the order of 1.3 to 1.5 times

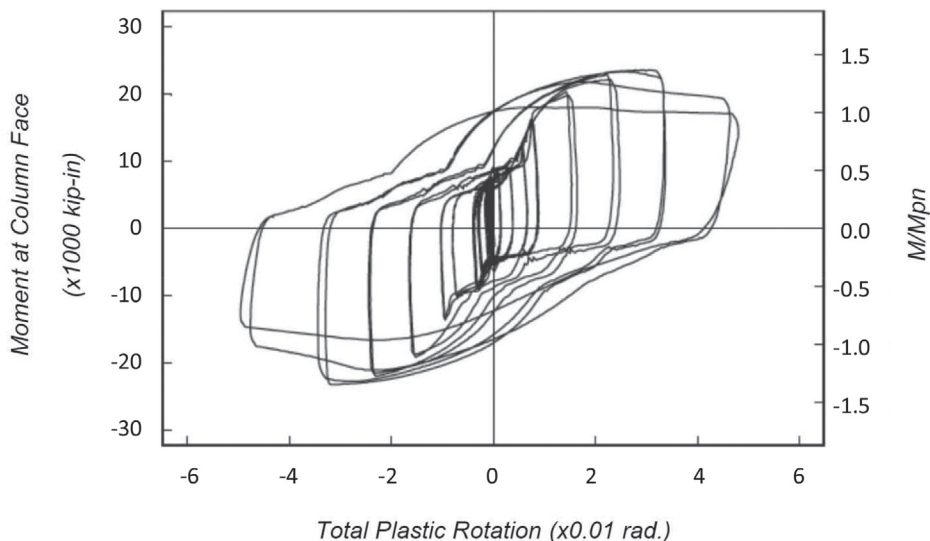


Fig. C-7.1. Moment at face of column versus total connection rotation for a BFP moment connection with a W30×108 (W760×161) beam and a W14×233 (W360×347) column.

the expected M_p of the beam, depending upon the span length, number of bolts, and length of the flange plate. The stiffness of this connection is usually slightly greater than 90% of that anticipated with a truly rigid, fully restrained (FR) connection. This reduced stiffness is expected to result in elastic deflection no more than 10% larger than computed with an FR connection, and so elastic calculations with rigid connections are considered to be adequate for most practical design purposes.

7.2. SYSTEMS

Review of the research literature shows that BFP moment connections meet the qualifications and requirements of both SMF and IMF frames. However, no test data are available for BFP moment connections with composite slabs, so the BFP moment connection is not prequalified with reinforced concrete structural slabs that contact the face of the columns. Reinforced concrete structural slabs that make contact with the column may:

- Significantly increase the moment at the face of the column.
- Cause significant increases of the force and strain demands in the bottom flange plate.
- Result in reduced inelastic deformation capacity of the connection.

Therefore, prequalification of the BFP moment connection is restricted to the case where the concrete structural slab has a minimum separation or isolation from the column. In general, isolation is achieved if steel stud anchors are not included in the protected zone and if the slab is separated from all surfaces of the column by an open gap or by use of compressible foam-like material.

7.3. PREQUALIFICATION LIMITS

1. Beam Limitations

The SMF prequalification limits largely reflect the range of past testing of the BFP moment connection. Limits for IMF connections somewhat exceed these limits because 18 of the past 20 tests used to prequalify the connection developed plastic rotations larger than those required to qualify as a SMF connection, and all 20 tests greatly exceed the rotation required to qualify as an IMF connection.

BFP moment connections have been tested with beams as large as the W36×150 (W920×223) while achieving the ductility required for qualification as an SMF. Consequently, the W36 (W920) beam depth, 150 lb/ft weight limit (223 kg/m mass limit), and 1 in. (25 mm) flange thickness limits are adopted in this provision. Past tests have shown adequate inelastic rotation capacity to qualify as an SMF in tests with span-to-depth ratios less than 5 and greater than 16, so lower bound span-to-depth ratio limits of 7 and 9 are conservatively adopted for the IMF and SMF applications, respectively. Inelastic deformation is expected for approximately one beam depth beyond the end of the flange plate, and limited yielding is expected in the flange plate. As a result, the protected zone extends from the column face to a distance equal to the depth of the beam beyond the bolt farthest from the face of the column.

Primary plastic hinging of the BFP moment connection occurs well away from the face of the column, and lateral-torsional deformation will occur as extensive yielding develops in the connection. As a result, lateral bracing of the beam is required at the end of the protected zone. The bracing is required within the interval between 1 and 1.5 beam depths beyond the flange bolts farthest from the face of the column. This permits some variation in the placement of the lateral support to allow economical use of transverse framing for lateral support where possible. As with other moment frame connections, supplemental lateral bracing at the column flange connection can typically be accommodated by the stiffness of the diaphragm and transverse framing.

As for other prequalified connections, the BFP moment connection requires compact flanges and webs as defined by the AISC *Seismic Provisions*, and built-up I-shaped beams conforming to Section 2.3 are permitted. It should be noted, however, that the BFP and most other prequalified connections do not have specific seismic test data to document the prequalification of built-up beam sections. This prequalification is provided because long experience shows that built-up steel sections provide flexural behavior similar to hot-rolled shapes with comparable materials and proportions.

2. Column Limitations

BFP moment connections have been tested with wide-flange columns up to W14×233 (W360×347) sections. The SMF prequalification limits largely reflect the range of past testing of the BFP moment connection. All 20 tests were completed with strong-axis bending of the column, and the prequalification of the BFP moment connections is limited to connections made to the column flange.

As with most other prequalified connections, the BFP moment connection has not been tested with columns deeper than W14 (W360) sections or with built-up column sections. It was the judgment of the CPRP that the BFP moment connection places similar or perhaps smaller demands on the column than other prequalified connections. The demands may be smaller because of the somewhat smaller strain-hardening moment increase achieved with the BFP moment connection as compared to the welded web-welded flange and other FR connections. The location of yielding of the BFP moment connection is somewhat analogous to the RBS connection, and therefore, prequalification limits for the column are comparable to those used for the RBS connection.

7.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

The BFP moment connection is expected to sustain primary yielding in the beam starting at the last flange plate bolt line away from the face of the column. Secondary yielding is expected in the column panel zone and very limited subsequent yielding is expected in the flange plate. Yielding in the column outside the connection panel zone is strongly discouraged. Therefore, the BFP moment connection employs a similar weak beam-strong column check and panel zone resistance check as used for other prequalified connections.

7.5. CONNECTION DETAILING

The BFP moment connection requires plate steel for the flange plate, shear plate, and possibly panel zone doubler plates. Past tests have been performed with plates fabricated both from ASTM A36/A36M and A572/A572M Grade 50 (Grade 345) steels. Therefore, the prequalification extends to both plate types. The designer should be aware of potential pitfalls with the material selection for the flange plate design. The flange plate must develop tensile yield strength over the gross section and ultimate tensile fracture resistance over the effective net section. A36/A36M steel has greater separation of the nominal yield stress and the minimum tensile strength, and this may simplify the satisfaction of these dual requirements. However, variation in expected yield stress is larger for A36/A36M steel, and design calculations may more accurately approximate actual flange plate performance with A572/A572M steel.

The flange plate welds are shop welds, and these welds are subject to potential secondary yielding caused by strain hardening at the primary yield location in the beam. As a result, the welds are required to be demand-critical complete-joint-penetration (CJP) groove welds. If backing is used, it must be removed, and the weld must be backgouged to sound material and backwelded to ensure that the weld can sustain yielding of the flange plate. Because the welds are shop welds, considerable latitude is possible in the selection of the weld process as long as the finished weld meets the demand critical weld requirements stipulated in the AISC *Seismic Provisions*. In the test specimens used to prequalify this connection, electroslag, gas shielded metal arc, and flux cored arc welding have been used.

The BFP moment connection places somewhat less severe demands on the web connection than most FR connections because of the somewhat greater flexibility of the bolted flange connection. As a result, the shear plate may be welded with CJP groove welds, partial-joint-penetration (PJP) groove welds, or fillet welds.

Bolts in the flange plate are limited to two rows of bolts, and the bolt holes must be made by drilling or sub-punching and reaming. These requirements reflect testing used to prequalify the BFP moment connection, but they also reflect practical limitations in the connection design. Net section rupture is a clear possibility in the beam flange and flange plates, and it is very difficult to meet the net section rupture criteria if more than two rows of bolts are employed.

A single row of bolts causes severe eccentricity in the connection and would lead to an excessively long connection. Punched bolt holes without reaming are not permitted because punching may induce surface roughness in the hole that may initiate cracking of the net section under high tensile stress. As noted earlier, the connection is prequalified only for A490 or A490M bolts with threads excluded from the shear plane. Bolt diameter is limited to a maximum of 1½ in. (28 mm) because larger bolts are seldom used and the 1½ in. (28 mm) diameter is the maximum used in past BFP tests. The bolt diameter must be selected to ensure that flange yielding over the gross area exceeds the net section capacity of the beam flange.

Oversized bolt holes were included in some past tests because the oversized holes permit easier alignment of the bolts and erection of the connection and resulted in good performance of the connection. Further, the beam must fit between two welded flange plates with full consideration of rolling and fabrication tolerances. As a result, shims may be used to simplify erection while ensuring a tight connection fit.

7.6. DESIGN PROCEDURE

The BFP moment connection is somewhat more complex than some other connections, because a larger number of yield locations and failure modes are encountered with this connection. **Step 1** of this procedure defines the maximum expected moment, M_{pr} , at the last bolt away from the face of the column in the flange plate. The beam flange must have greater net section fracture resistance than its yield resistance because tensile yield of the flange is a ductile mechanism and net section rupture is a brittle failure. **Step 2** establishes the maximum bolt diameter that can meet this balanced criterion. While this requirement is rational, it should be noted that net section rupture of the beam flange has not occurred in any past BFP tests, because the beam web clearly reduces any potential for flange rupture.

The shear strength of the flange bolts is the smallest strength permitted based on bolt shear with threads excluded from the shear plane, bolt bearing on the flange plate, bolt bearing on the beam flange, and block shear considerations. **Step 3** provides this evaluation. **Step 4** is an approximate evaluation of the number of bolts needed to develop the BFP moment connection. The moment for the bolts is larger than M_{pr} because the centroid of the bolt group is at a different location than the primary hinge location. However, this moment cannot be accurately determined until the geometry of the flange plate and bolt spacing are established. The 1.25 factor is used as an empirical increase in this moment to provide this initial estimate for the number of bolts required. The bolts are tightened to meet slip-critical criteria, but the connection is not slip-critical: The bolts are designed as bearing bolts.

Once the required number of bolts is established, bolt spacing and an initial estimate of the flange plate length can be established. This geometry is illustrated and summarized in Figure 7.1, and **Step 5** defines critical dimensions of this geometry for later design checks.

Step 6 is similar to other connection types in that the shear force at the plastic hinge is based upon the maximum shear achieved with maximum expected moments at the plastic hinges at both ends of the beam plus the shear associated with appropriate gravity loads on the beam.

Step 7 uses the geometry established in **Step 5** and the maximum shear force established in **Step 6** to determine the maximum expected moment at the face of the column flange, M_f . The maximum expected force in the flange plate, F_{pr} , is determined from M_f in **Step 8**.

The flange plate bolts cannot experience a tensile force larger than F_{pr} , so **Step 9** checks the actual number of bolts required in the connection. If this number is larger

or smaller than that estimated in **Step 4**, it may be necessary to change the number of bolts and repeat **Steps 5** through **9** until convergence is achieved.

Steps 10 and **11** check the flange plate width and thickness to ensure that tensile yield strength and tensile rupture strength, respectively, exceed the maximum expected tensile force in the flange. The net section rupture check of **Step 11** employs the nonductile resistance factor, while the flange yielding check of **Step 10** employs the ductile resistance factor; this check also allows limited yielding in the flange plate and ensures ductility of the connection. **Step 12** checks block shear of the bolt group in the flange plate, and **Step 13** checks the flange plate for buckling, when F_{pr} is in compression. Both block shear and buckling of the flange plate are treated as nonductile behaviors.

Step 14 is somewhat parallel to **Step 6** except that the beam shear force at the face of the column is established, and this shear force is then used to size and design the single shear-plate connection in **Step 15**.

Continuity plates and panel zone shear strength are checked in **Steps 16** and **17**, respectively. These checks are comparable to those used for other prequalified connections.

As previously noted, the BFP moment connection has provided quite large inelastic rotational capacity in past research. It has done this by attaining primary yielding in the beam at the end of the flange plate away from the column and through secondary yielding as shear yielding in the column panel zone and tensile yielding in the flange plate. Bolt slip occurs but does not contribute greatly to connection ductility. This rather complex design procedure attempts to achieve these goals by balancing the resistances for different yield mechanisms and failure modes in the connection and by employing somewhat greater conservatism for brittle behaviors than for ductile behaviors.

CHAPTER 8

WELDED UNREINFORCED FLANGE-WELDED WEB (WUF-W) MOMENT CONNECTION

8.1. GENERAL

The welded unreinforced flange-welded web (WUF-W) moment connection is an all-welded moment connection, wherein the beam flanges and the beam web are welded directly to the column flange. A number of welded moment connections that came into use after the 1994 Northridge earthquake, such as the reduced beam section and connections provided with beam flange reinforcement, were designed to move the plastic hinge away from the face of the column. In the case of the WUF-W moment connection, the plastic hinge is not moved away from the face of the column. Rather, the WUF-W moment connection employs design and detailing features that are intended to permit the connection to achieve SMF performance criteria without fracture. Key features of the WUF-W moment connection that are intended to control fracture are as follows:

- The beam flanges are welded to the column flange using CJP groove welds that meet the requirements of demand critical welds in the AISC *Seismic Provisions*, along with the requirements for treatment of backing and weld tabs and welding quality control and quality assurance requirements, as specified in Chapter 3.
- The beam web is welded directly to the column flange using a CJP groove weld that extends the full-depth of the web—that is, from weld access hole to weld access hole. This is supplemented by a single-plate connection, wherein a single plate is welded to the column flange and is then fillet welded to the beam web. Thus, the beam web is attached to the column flange with both a CJP groove weld and a welded single-plate connection. The single-plate connection adds stiffness to the beam web connection, drawing stress toward the web connection and away from the beam flange-to-column flange connections. The single plate also serves as backing for the CJP groove weld connecting the beam web to the column flange.
- Instead of using a conventional weld access hole detail as specified in AISC *Specification* Section J1.6 (AISC, 2016b), the WUF-W moment connection employs a special seismic weld access hole with requirements on size, shape and finish that reduce stress concentrations in the region around the access hole detailed in AWS D1.8/D1.8M (AWS, 2016).

Prequalification of the WUF-W moment connection is based on the results of two major research and testing programs. Both programs combined large-scale tests with extensive finite element studies. Both are briefly described herein.

The first research program on the WUF-W moment connection was conducted at Lehigh University as part of the SAC-FEMA program. Results are reported in several publications (Ricles et al., 2000, 2002). This test program formed the basis of prequalification of the WUF-W moment connection in FEMA 350 (FEMA, 2000e). As part of the Lehigh program, tests were conducted on both interior and exterior type specimens. The exterior specimens consisted of one beam attached to a column. The interior specimens consisted of a column with beams attached to both flanges. One of the interior specimens included a composite floor slab. All specimens used W36×150 (W920×223) beams. Three different column sizes were used: W14×311, W14×398 and W27×258 (W360×463, W360×592 and W690×384). All WUF-W moment connection specimens tested in the Lehigh program satisfied the rotation criteria for SMF connections (± 0.04 -rad total rotation). Most specimens significantly exceeded the qualification criteria. Considering that the interior type specimens included two WUF-W moment connections each, 12 successful WUF-W moment connections were tested in the Lehigh program. This research program included extensive finite element studies that supported the development of the special seismic weld access hole and the details of the web connection.

The second major research program on the WUF-W moment connection was conducted at the University of Minnesota. The purpose of this research program was to examine alternative doubler plate details, continuity plate requirements, and effects of a weak panel zone. All test specimens used the WUF-W moment connection. Results are reported in several publications (Lee et al., 2002, 2005a, 2005b). Six interior type specimens were tested in the Minnesota program. All specimens used W24×94 beams. Three column sizes were used: W14×283, W14×176 and W14×145. All specimens were designed with panel zones weaker than permitted by the AISC *Seismic Provisions*. Two of the test specimens, CR1 and CR4, were inadvertently welded with low-toughness weld metal. This resulted in premature weld failure in specimen CR4 (failure occurred at about 0.015-rad rotation). With the exception of CR4, all specimens achieved a total rotation of ± 0.04 rad, and sustained multiple cycles of loading at ± 0.04 rad prior to failure. All successful specimens exhibited substantial panel zone yielding, due to the weak panel zone design. This test program was also supported by extensive finite element studies.

Considering the WUF-W moment connection research programs at both Lehigh and the University of Minnesota, WUF-W moment connection specimens have shown excellent performance in tests. There is only one reported failed test, due to the inadvertent use of low-toughness weld metal for beam flange CJP groove welds (Minnesota specimen CR4). Of all of the WUF-W moment connection specimens that showed good performance (achieved rotations of at least ± 0.04 rad), approximately one-half had panel zones weaker than permitted by the AISC *Seismic Provisions*. The other half satisfied the panel zone strength criteria of the AISC *Seismic Provisions*. This suggests that the WUF-W moment connection performs well for both strong and weak panel zones; therefore, the connection is not highly sensitive to panel zone strength.

The protected zone for the WUF-W moment connection is defined as the portion of the beam extending from the face of the column to a distance d from the face of the column, where d is the depth of the beam. Tests on WUF-W moment connection specimens show that yielding in the beam is concentrated near the face of the column, but extends to some degree over a length of the beam approximately equal to its depth.

8.3. PREQUALIFICATION LIMITS

The WUF-W moment connection is prequalified for beams up to W36 (W920) in depth, up to 150 lb/ft in weight (223 kg/m mass limit) and up to a beam flange thickness of 1 in. (25 mm). This is based on the fact that a W36×150 (W920×223) is the deepest and heaviest beam tested with the WUF-W moment connection. The 1-in. (25-mm) flange thickness limitation represents a small extrapolation of the 0.94-in. (23.9-mm) flange thickness for the W36×150 (W920×223). Limits are also placed on span-to-depth ratio based on the span-to-depth ratios of the tested connections and based on judgment of the CPRP.

Beam lateral bracing requirements for the WUF-W moment connection are identical to those for the RBS moment connection. The effects of beam lateral bracing on cyclic loading performance have been investigated more extensively for the RBS moment connection than for the WUF-W moment connection. However, the available data for the WUF-W moment connection suggest that beams are less prone to lateral-torsional buckling than with the RBS moment connections. Consequently, it is believed that lateral bracing requirements established for the RBS moment connection are satisfactory, and perhaps somewhat conservative, for the WUF-W moment connection.

Column sections used in WUF-W moment connection test specimens were W14 (W360) and W27 (W690) sections. However, column limitations for the WUF-W moment connection are nearly the same as for the RBS moment connection, which includes wide-flange shapes up to W36 (W920) and box columns up to 24 in. by 24 in. (610 mm by 610 mm). A primary concern with deep columns in moment frames has been the potential for twisting and instability of the column driven by lateral-torsional buckling of the beam. Because beams with WUF-W moment connections are viewed as somewhat less prone to lateral-torsional buckling than beams with RBS moment connections, the column limitations established for the RBS moment connection were judged as appropriate for the WUF-W moment connection.

8.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

WUF-W moment connection test specimens have shown good performance with a range of panel zone shear strengths, ranging from very weak to very strong panel zones. Tests conducted at the University of Minnesota (Lee et al., 2005b) showed excellent performance on specimens with panel zones substantially weaker than required in the AISC *Seismic Provisions*. However, there are concerns that very weak panel zones may contribute to premature connection fracture under some circumstances, and it is

believed further research is needed before weak panel zone designs can be prequalified. Consequently, the minimum panel zone strength required in AISC *Seismic Provisions* Section E3.6e is required for prequalified WUF-W moment connections for SMF. For IMF systems, the AISC *Seismic Provisions* have no special panel zone strength requirements, beyond the AISC *Specification*. This may lead to designs in which inelastic action is concentrated within the panel zone. As described earlier, based on successful tests on WUF-W moment connection specimens with weak panel zones, this condition is not viewed as detrimental for IMF systems.

8.5. BEAM FLANGE-TO-COLUMN FLANGE WELDS

The welds must meet the requirements of demand critical welds in the AISC *Seismic Provisions*, as well as the detailing and quality control and quality assurance requirements specified in Chapter 3. These beam flange-to-column flange weld requirements reflect the practices used in the test specimens that form the basis for prequalification of the WUF-W moment connection and reflect what are believed to be best practices for beam flange groove welds for SMF and IMF applications.

A key feature of the WUF-W moment connection is the use of a special weld access hole. The special seismic weld access hole has specific requirements on the size, shape and finish of the access hole. This special access hole was developed in research on the WUF-W moment connection (Ricles et al., 2000, 2002) and is intended to reduce stress concentrations introduced by the presence of the weld access hole. The size, shape and finish requirements for the special access hole are specified in AWS D1.8/D1.8M Section 6.11.1.2 (AWS, 2016).

8.6. BEAM WEB-TO-COLUMN CONNECTION LIMITATIONS

The beam web is connected to the column flange with a full-depth (weld access hole-to-weld access hole) CJP groove weld, with a single plate serving as backing. The single plate is fillet welded to the beam web and also welded to the column flange. See Figure 8.2 for detail. The use of the CJP groove weld combined with the fillet-welded single plate is believed to increase the stiffness of the beam web connection. The stiffer beam web connection serves to draw stress away from the beam flanges and therefore reduces the demands on the beam flange groove welds.

Most of the details of the beam web-to-column connection are fully prescribed in Section 8.6; thus, few design calculations are needed for this connection. An exception to this is the connection of the single plate to the column. This connection must develop the shear strength of the single plate, as specified in Section 8.6(2). This can be accomplished by the use of CJP groove welds, PJP groove welds, fillets welds, or combinations of these welds. The choice of these welds is left to the discretion of the designer. In developing the connection between the single plate and the column flange, designers should consider the following issues:

- The use of a single-sided fillet weld between the single plate and the column flange should be avoided. If the single plate is inadvertently loaded or struck in

the out-of-plane direction during erection, the fillet weld may break and may lead to erection safety concerns.

- The end of the beam web must be set back from the face of the column flange a specified amount to accommodate the web CJP root opening dimensional requirements. Consequently, the single plate-to-column weld that is placed in the web CJP root opening must be small enough to fit in that specified root opening. For example, if the CJP groove weld is detailed with a 1/4-in. (6-mm) root opening, a fillet weld between the single plate and the column flange larger than 1/4 in. (6 mm) will cause the root of the CJP groove weld to exceed 1/4 in. (6 mm).
- Placement of the CJP groove weld connecting the beam web to the column flange will likely result in intermixing of weld metal, with the weld attaching the single plate to the column flange. Requirements for intermix of filler metals specified in AWS D1.8/D1.8M (AWS, 2016) should be followed in this case.

The CJP groove weld connecting the beam web to the column flange must meet the requirements of demand critical welds. Note that weld tabs are permitted, but not required, at the top and bottom ends of this weld. If weld tabs are used, they should be removed after welding according to the requirements of Section 3.4. If weld tabs are not used, the CJP groove weld should be terminated in a manner that minimizes notches and stress concentrations, such as with the use of cascaded ends.

The fillet weld connecting the beam web to the single plate should be terminated a small distance from the weld access hole, as shown in Figure 8.3. This is to avoid introducing notches at the edge of the weld access hole.

8.7. DESIGN PROCEDURE

For the WUF-W moment connection, many of the details of the connection of the beam to the column flange are fully prescribed in Sections 8.5 and 8.6. Consequently, the design procedure for the WUF-W moment connection largely involves typical checks for continuity plates, panel zone shear strength, column-beam moment ratio, and beam shear strength.

With the WUF-W moment connection, yielding of the beam (i.e., plastic hinge formation) occurs over the portion of the beam extending from the face of the column to a distance of approximately one beam depth beyond the face of the column. For purposes of the design procedure, the location of the plastic hinge is taken to be at the face of the column. That is, $S_h = 0$ for the WUF-W moment connection. It should be noted that the location of the plastic hinge for design calculation purposes is somewhat arbitrary, because the plastic hinge does not occur at a single point but, instead, occurs over some length of the beam. The use of $S_h = 0$ is selected to simplify the design calculations. The value of C_{pr} was calibrated so that when used with $S_h = 0$, the calculated moment at the column face reflects values measured in experiments. Note that the moment in the beam at the column face is the key parameter in checking panel zone strength, column-beam moment ratio, and beam shear strength.

The value of C_{pr} for the WUF-W moment connection is specified as 1.4, based on an evaluation of experimental data. Tests on WUF-W moment connections with strong panel zones (Ricles et al., 2000) showed maximum beam moments, measured at the face of the column, as high as $1.49M_p$, where M_p was based on measured values of F_y . The average maximum beam moment at the face of the column was $1.33M_p$. Consequently, strain hardening in the beam with a WUF-W moment connection is quite large. The value of C_{pr} of 1.4 was chosen to reflect this high degree of strain hardening. Combining the value of $C_{pr} = 1.4$ with $S_h = 0$ results in a moment at the face of the column, $M_f = M_{pr} = 1.4R_yF_yZ$, that reasonably reflects maximum column face moments measured in experiments.

CHAPTER 9

KAISER BOLTED BRACKET (KBB) MOMENT CONNECTION

9.1. GENERAL

The Kaiser bolted bracket (KBB) moment connection is designed to eliminate field welding and facilitate frame erection. Depending on fabrication preference, the brackets can be either fillet welded (W-series) or bolted (B-series) to the beam. The B-series can also be utilized to improve the strength of weak or damaged connections, although it is not prequalified for that purpose. Information on the cast steel and the process used to manufacture the brackets is provided in Appendix A.

The proprietary design of the brackets is protected under U.S. patent number 6,073,405 held by Steel Cast Connections LLC. Information on licensing rights can be found at <http://www.steelcastconnections.com>. The connection is not prequalified when brackets of an unlicensed design and/or manufacture are used.

Connection prequalification is based on 21 full-scale bolted bracket tests representing both new and repaired applications (Kasai and Bleiman, 1996; Gross et al., 1999; Newell and Uang, 2006; and Adan and Gibb, 2009). These tests were performed using beams ranging in depth from W16 to W36 (W410 to W920) and columns using W12, W14 and W27 (W310, W360 and W690) sections. Built-up box columns have also been tested. The test subassemblies have included both single cantilever and double-sided column configurations. Concrete slabs were not present in any tests. During testing, inelastic deformation was achieved primarily through the formation of a plastic hinge in the beam. Some secondary yielding was also achieved in the column panel zone. Peak strength typically occurred at an interstory drift angle between 0.025 and 0.045 rad. Specimen strength then gradually decreased with additional yielding and deformation. In the KBB testing reported by Adan and Gibb (2009), the average specimen maximum interstory drift angle exceeded 0.055 rad.

9.2. SYSTEMS

Review of the research literature and testing referenced in this document indicates that the KBB moment connection meets the prequalification requirement for special and intermediate moment frames.

The exception associated with concrete structural slab placement at the column and bracket flanges is based on testing conducted on the stiffened extended end-plate moment connection (Seek and Murray, 2008). While bolted bracket testing has been conducted primarily on bare-steel specimens, some limited testing has also been performed on specimens with a concrete structural slab. In these tests, the presence of the slab provided a beneficial effect by maintaining the stability of the beam at larger

interstory drift angles (Gross et al., 1999; Newell and Uang, 2006). However, in the absence of more comprehensive testing with a slab, the placement of the concrete is subject to the exception.

9.3. PREQUALIFICATION LIMITS

1. Beam Limitations

A wide range of beam sizes was tested with bolted brackets. The lightest beam size reported in the literature was a W16×40 (W410×60). The heaviest beam reported was a W36×210 (W920×313). In the W36×210 test, the specimen met the requirements, but subsequently experienced an unexpected nonductile failure of the bolts connecting the bracket to the column. The next heaviest beams reported to have met the requirements were W33×130 and W36×150 (W840×193 and W410×60). Based on the judgment of the CPRP, the maximum beam depth and weight was limited to match that of the W33×130 (W840×193). The maximum flange thickness was established to match a modest increase above that of the W36×150 (W410×60).

The limitation associated with minimum beam flange width is required to accommodate fillet weld attachment of the W-series bracket and to prevent beam flange tensile rupture when using the B-series bracket.

Bolted bracket connection test assemblies used configurations approximating beam spans between 24 and 30 ft (7.3 and 9.1 m). The beam span-to-depth ratios were in the range of 8 to 20. Given the degree to which most specimens significantly exceeded the requirement, it was judged reasonable to set the minimum span-to-depth ratio at 9 for both SMF and IMF systems.

As with other prequalified connections, beams supporting a concrete structural slab are not required to have a supplemental brace near the expected plastic hinge. If no floor slab is present, then a supplemental brace is required. The brace may not be located within the protected zone.

2. Column Limitations

Bolted bracket connection tests were performed with the brackets bolted to the column flange (i.e., strong-axis connections). In the absence of additional testing with brackets bolted to the column web (weak-axis connections), the prequalification is limited to column flange connections.

Test specimen wide-flange column sizes ranged from W12×65 to W27×281 (W310×97 to W690×418). Testing performed by Ricles et al. (2004) of deep-column RBS connections demonstrated that deep columns do not behave substantially different from W14 (W360) columns when a slab is present or when adequate lateral bracing is provided for the beam and/or column in the absence of a slab. Based on the similarity in performance to that of the RBS connection, the KBB is prequalified to include column sizes up to W36 (W920).

The behavior of a flanged cruciform column in KBB connections is expected to be similar to that of a rolled wide-flange. Therefore, flanged cruciform columns are prequalified, subject to the limitations imposed on rolled wide-flange shapes.

Two of the tests were successfully conducted using a built-up box column. In the first box column test, connections were made on two opposing column faces. Then, in the second test, a connection was made to the orthogonal face of the same column. These two tests were intended to prequalify a box column participating in orthogonal moment frames. The tested box column was 15 $\frac{7}{8}$ in. (390 mm) square (Adan and Gibb, 2009). Consequently, bolted bracket connections are prequalified for use with built-up box columns up to 16 in. (406 mm) square.

Based on both successful wide-flange and built-up box column testing, acceptable performance would also be expected for boxed wide-flange columns. Therefore, the use of boxed wide-flange columns is also prequalified. When moment connections are made only to the flanges of the wide-flange portion of the boxed wide-flange, subject to the bracing limitations mentioned previously, the column may be as deep as a W36 (W920). When the boxed wide-flange column participates in orthogonal moment frames, neither the depth nor the width of the column is allowed to exceed 16 in. (400 mm), applying the same limit as a built-up box.

3. Bracket Limitations

The ASTM cast steel material specification used to manufacture the brackets is based on recommendations from the Steel Founders' Society of America (SFSA).

The cast brackets are configured and proportioned to resist applied loads in accordance with the limit states outlined by Gross et al. (1999). These limit states include column flange local buckling; bolt prying action; combined bending and axial loading on the bracket; shear; and additionally for the B-series, bolt bearing deformation and block shear rupture.

In tests representing new applications, the bracket column bolt holes were cast vertically short-slotted. The vertically slotted holes provide field installation tolerance. In tests representing a repair application, the holes were cast standard diameter. There has been no difference in connection performance using either type of cast hole (Adan and Gibb, 2009).

9.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

The reduction of column axial and moment strength due to the column bolt holes need not be considered when checking column-beam moment ratios. Research performed by Masuda et al. (1998) indicated that a 30 to 40% loss of flange area due to bolt holes showed only a corresponding 10% reduction in the yield moment strength.

9.5. BRACKET-TO-COLUMN FLANGE CONNECTION LIMITATIONS

In the prequalification tests, fasteners joining the bracket to the column flange were pretensioned ASTM F3125 Grade A490 or A490M bolts. The column bolt head can

be positioned on either the column or bracket side of the connection. Where possible, the column bolts are tightened prior to the bolts in the web shear tab.

When needed, finger shims between the bracket and column face allow for fit between the bracket and column contact surfaces. Tests indicated that the use of finger shims does not affect the performance of the connection.

Because the flanges of a box column are stiffened only at the corners, tightening of the column bolts can cause excessive local flange bending. Therefore, as shown in Figure C-9.1, a washer plate is required between the box column flange and the bracket.

As shown in Figure C-9.1, orthogonally connected beams framing into a box column are raised one-half of the column bolt spacing distance to avoid overlapping the column bolts.

9.6. BRACKET-TO-BEAM FLANGE CONNECTION LIMITATIONS

The cast steel brackets are not currently listed as a prequalified material in AWS D1.1/D1.1M (AWS, 2015). Therefore, the weld procedure specification (WPS) for the fillet weld joining the bracket to the beam flange is required to be qualified by test with the specific cast material.

Bolts joining the bracket to the beam flange in prequalification tests have been conducted with pretensioned ASTM F3125 Grade A490 or A490M bolts with the threads excluded from the shear plane. The beam bolt head can be positioned on either the beam or bracket side of the connection. Given the beam bolt pattern and hole size, it is necessary to use the bracket as a template when drilling the beam bolt holes. The holes must be aligned to permit insertion of the bolts without undue damage to the threads.

The brass washer plate prevents abrading of the beam and bracket contact surfaces. In the initial developmental stages of the connection, several specimens configured without the brass plate experienced flange net section rupture through the outermost bolt holes. Observation of the failed specimens indicated that fracture likely initiated at a notch created by the abrading contact surfaces near the hole. Furthermore, energy released through the beam-bracket slip-stick mechanism caused loud, intermittent bursts of noise, particularly at high levels of inelastic drift (Kasai and Bleiman, 1996). To overcome these problems, the brass plate was inserted between the bracket and the beam flange. The idea is based on the use of a brass plate as a special friction-based seismic energy dissipator (Grigorian et al., 1992). Although not intended to dissipate energy in the bolted bracket connection, the brass plate provides a smooth slip mechanism at the bracket-to-beam interface.

When bolting the bracket to a beam flange, a steel washer or clamp plate is positioned on the opposite side of the connected flange. The restraining force of the clamp plate prevents local flange buckling from occurring near the outermost bolt holes. In tests performed without the clamp plates, flange distortion increased the strains near the

holes. The increased strain caused necking and fracture through the flange net area. In similar tests performed with the clamp plates, yielding and fracture occurred outside the connected region through the flange gross area (Kasai and Bleiman, 1996).

9.7. BEAM WEB-TO-COLUMN CONNECTION LIMITATIONS

All of the bolted bracket connection tests were performed with a bolted web connection where pretensioned high-strength bolts were used. Therefore, the KBB is prequalified for a bolted beam web-to-column connection.

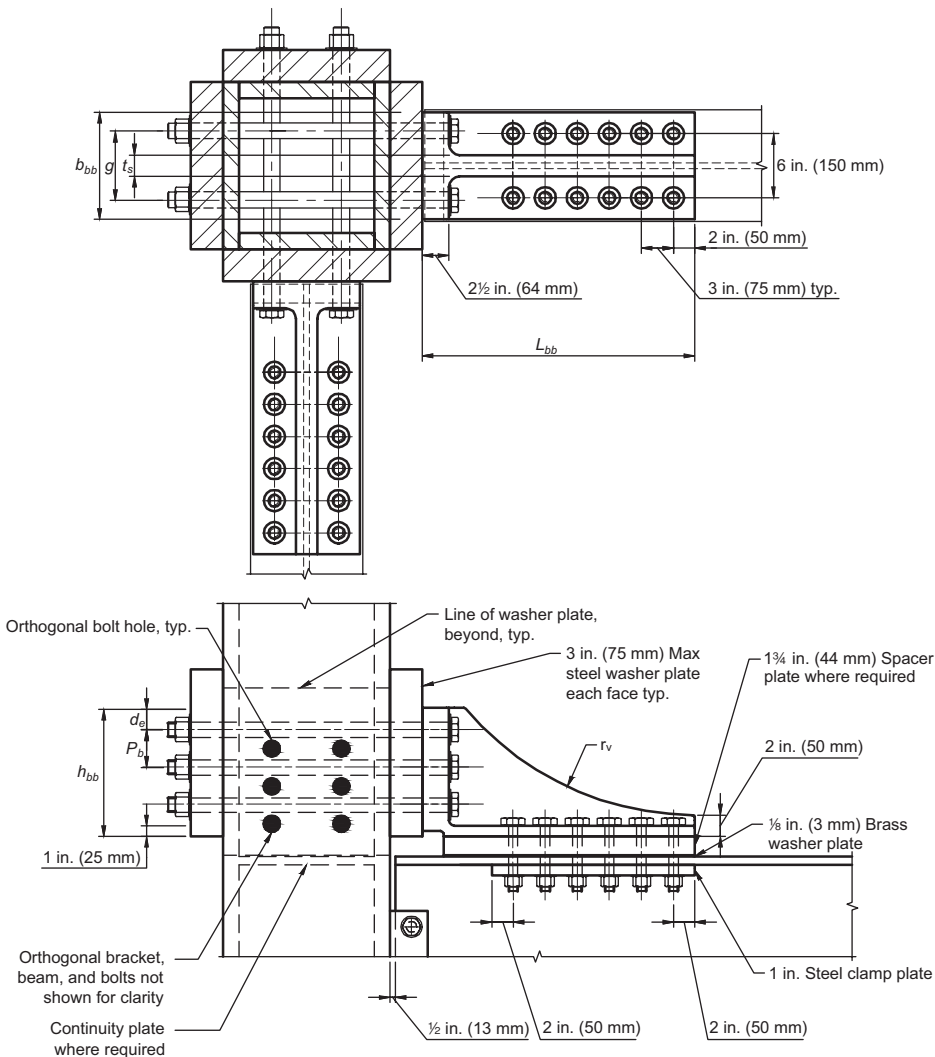


Fig. C-9.1. Box column connection detailing for KBB.

9.8. CONNECTION DETAILING

Both Figures 9.4 and 9.5 show the connection configured with continuity plates where required. The use of continuity plates is dictated by the need to satisfy prescribed limit states for the flange and web of the column. In a bolted connection, the configuration of the fasteners can impede the ability of the continuity plates to effectively address these limit states. The design intent for the KBB is to satisfy the prescribed limit states without continuity plates. In tests of wide flange columns without continuity plates, the absence of the continuity plates did not appear to promote local flange bending or lead to other detrimental effects (Adan and Gibb, 2009). However, in the absence of additional tests on deeper column sections, prequalification without continuity plates is limited to W12 (W310) and W14 (W360) sections.

9.9. DESIGN PROCEDURE

The design procedure for prequalified KBB connections is intended to develop the probable maximum moment capacity of the connecting beam. Test data indicate that connecting the brackets to the column and beam in accordance with the requirements herein allows the connection to resist this level of moment.

Tables C-9.1, C-9.1M, C-9.2, and C-9.2M can be used as a guide in selecting trial bracket-beam combinations in conjunction with **Steps 1** and **3**. The tables are based on beams that satisfy the limitations of Section 9.3.1 for ASTM A992/A992M or A572/A572M Grade 50 (Grade 345) wide-flange shapes.

Step 4 of the procedure requires computation of the shear force at the expected plastic hinge. This shear force is a function of the gravity load on the beam and the plastic moment strength. A calculation similar to that for the RBS moment connection is required for the case of a beam with a uniformly distributed gravity load as shown in Figure C-5.1. For the KBB, L_h is the distance between the expected plastic hinge locations and S_h is the distance from the face of the column to the hinge. The explanation associated with Equations C-5.8-1 and C-5.8-2 also applies to the KBB.

Step 6 is based on the limit state of bolt tensile rupture as defined in AISC *Specification* Section J3.6 (AISC, 2016b), where the required bolt tensile strength is determined in Equation 9.9-3.

Steps 7 and **11** of the procedure apply to rolled or built-up shapes with flange holes, proportioned based on flexural strength of the gross section. The flexural strength is limited in accordance with the limit state of flange tensile rupture as defined in AISC *Specification* Section F13.1. When the flange width is adequate, the tensile rupture limit state does not apply.

Step 8 of the procedure requires a column flange prying action check as outlined in AISC *Manual* Part 9. The computations include provisions from the research performed by Kulak et al. (1987).

Step 9 of the procedure is based on the limit state of column flange local bending as defined in AISC *Specification* Section J10.1. The limit state determines the strength

TABLE C-9.1
Recommended W-Series Bracket-Beam Combinations

Bracket Designation	Beam Designations
W1.0	W33×130, W30×124, W30×116, W24×131, W21×122, W21×111
W2.1	W30×108, W27×114, W27×102, W24×103, W21×93, W18×106, W18×97
W2.0	W27×94, W24×94, W24×84, W24×76, W21×83, W21×73, W21×68, W21×62, W18×86, W18×71, W18×65
W3.1	W24×62, W24×55, W21×57, W18×60, W18×55, W16×57
W3.0	W21×50, W21×44, W18×50, W18×46, W18×35, W16×50, W16×45, W16×40, W16×31

TABLE C-9.1M
Recommended W-Series Bracket-Beam Combinations

Bracket Designation	Beam Designations
W1.0	W840×193, W760×185, W760×173, W610×195, W530×182, W530×165
W2.1	W760×161, W690×170, W690×152, W610×153, W530×138, W460×158, W460×144
W2.0	W690×140, W610×140, W610×125, W610×113, W530×123, W530×109, W530×101, W530×92, W460×128, W460×106, W460×97
W3.1	W610×92, W610×82, W530×85, W460×89, W460×82, W410×85
W3.0	W530×74, W530×66, W460×74, W460×68, W460×52, W410×75, W410×67, W410×60, W410×46.1

TABLE C-9.2
Recommended B-Series Bracket-Beam Combinations

Bracket Designation	Beam Designations
B1.0	W33×130, W30×124, W30×116, W24×131, W21×122, W21×111
B2.1	W30×108, W27×114, W27×102, W27×94, W18×106, W18×97

TABLE C-9.2M
Recommended B-Series Bracket-Beam Combinations

Bracket Designation	Beam Designations
B1.0	W840×193, W760×185, W760×173, W610×195, W530×182, W530×165
B2.1	W760×161, W690×170, W690×152, W690×140, W460×158, W460×144

of the flange using a simplified yield line analysis. Yield line analysis is a method that determines the flexural load at which a collapse mechanism will form in a flat plate structure and employs the principle of virtual work to develop an upper bound solution for plate strength. Given the bolted bracket configuration, the solution can be simplified to determine the controlling yield line pattern that produces the lowest failure load. Because a continuity plate would interfere with the installation of the connecting bolts, the procedure requires that the column flange thickness adequately satisfies the limit state without the requirement to provide continuity plates.

Although **Step 9** requires a flange thickness that will adequately satisfy the column flange local bending limit state, the limit states of web local yielding, web crippling, and web compression buckling as defined in Sections J10.2, J10.3 and J10.5 of the AISC *Specification*, respectively, may also be applicable. In shallow seismically compact W12 (W310) and W14 (W360) sections, these additional limit states will not control. However, in some deeper sections, the additional limit states may govern. Therefore, **Step 10** requires continuity plates in the deeper sections to adequately address the limit states and to stabilize deep column sections. The plates are positioned at the same level as the beam flange as shown in Figures 9.4 and 9.5.

Step 12 of the procedure is based on the limit state of bolt shear rupture as defined in AISC *Specification* Section J3.6. When this connection first appeared in the 2009 Supplement No. 1 to AISC 358-05, a bolt shear overstrength factor of 1.1 was included in the denominator of Equation 9.9-9 based on research subsequently reported by Tide (2010). The 2010 AISC *Specification* has incorporated that factor into the tabulated shear strengths of bolts, necessitating its removal here.

The procedure outlined in **Step 12** omits a bolt bearing or tearout limit state check per Section J3.10 of the AISC *Specification* because the provisions of Sections 9.3.1(5) and 9.3.1(7) preclude the use of beams where the bolt bearing or tearout would limit the strength of the connection.

Step 14 of the procedure is based on the limit state of weld shear rupture as defined in AISC *Specification* Section J2.4. The procedure assumes a linear weld group loaded through the center of gravity.

Step 18 of the procedure is supplemental if the column is a built-up box configuration. The procedure is based on the limit state of yielding (plastic moment) as defined in AISC *Specification* Section F11.1. The design assumes a simply supported condition with symmetrical point loads applied at the bolt locations.

CHAPTER 10

CONXTECH CONXL MOMENT CONNECTION

10.1. GENERAL

The ConXtech® ConXL™ moment connection is designed to provide robust cost effective moment framing while eliminating field welding and facilitating fast frame erection. The patented ConXL fabrication and manufacturing process utilizes forged parts, welding fixtures and robotic welders to produce a standardized connection.

The collars and collar assemblies illustrated, and methodologies used in their fabrication and erection, are covered by one or more of the U.S. and foreign patents shown at the bottom of the first page of Chapter 10. Additional information on the ConXL connection can also be found at <http://www.conxtech.com>.

Prequalification of the ConXL moment connection is based on the 17 qualifying cyclic tests listed in Table C-10.1, as well as nonlinear finite element modeling of the connection. The test database includes five biaxial moment connection tests. These unprecedented biaxial moment connection tests subjected the framing in the orthogonal plane to a constant shear creating a moment across the column-beam joint equivalent to that created by the probable maximum moment at the plastic hinge of the primary beams, while the framing in the primary plane was simultaneously subjected to the qualifying cyclic loading specified by ANSI/AISC 341-05 Appendix S (AISC, 2005a) until failure occurred. Tests were conducted using a variety of column-to-beam strength ratios. Many tests were conducted with an intentionally reinforced column, consisting of a concrete-filled HSS with an embedded W12 (W310) inside the HSS, forcing all inelastic behavior out of the column. In one of the biaxial tests, simultaneous flexural yielding of the column was initiated during cycling. Typically, failures consist of low-cycle fatigue of a beam flange in the zone of plastic hinging, following extensive rotation and local buckling deformation.

The ConXL connection is a true biaxial moment connection capable of moment-connecting up to four beams to a column. All moment-connected columns require a full set of four collar flange top (CFT) pieces and four collar flange bottom (CFB) pieces at every beam-column moment connected joint, even if a column face has no beam present. Each column face with either a moment-connected beam or simply supported beam will have the full collar flange assembly [CFT, CFB, and collar web extension (CWX)] with the simply connected beam connected to the CWX with a standardized bolted connection.

Unlike more conventional moment frame design, which focuses on keeping the number of moment-resisting frames to a minimum for reasons of economy, the efficient ConXL system distributes the biaxial moment connection to nearly every beam-column-beam joint throughout the structure creating a distributed moment-resisting

TABLE C-10.1
Summary of ConXL Tests

Test No.	Test Condition	Column Size	Primary Axis Beam	Secondary Axis Beam	Rotation (rad)
1101	Planar	HSS 16×16× $\frac{5}{8}$ *	W18×76 RBS	N/A	0.05
1102	Planar	HSS 16×16× $\frac{5}{8}$ *	W18×119	N/A	0.05
1103	Planar	HSS 16×16× $\frac{5}{8}$ *	W24×84 RBS	N/A	0.06
1104	Planar	HSS 16×16× $\frac{5}{8}$ *	W24×104	N/A	0.05
1105	Planar	HSS 16×16× $\frac{5}{8}$ *	W24×117×6†	N/A	0.04
1106	Planar	HSS 16×16× $\frac{5}{8}$ *	W24×117×9†	N/A	0.04
1107	Planar	HSS 16×16× $\frac{5}{8}$	W21×62 RBS	N/A	0.04
1108	Planar	HSS 16×16× $\frac{5}{8}$	W21×62 RBS	N/A	0.06
1201	Planar	HSS 16×16× $\frac{5}{8}$ *	W30×108 RBS	N/A	0.05
1202	Planar	HSS 16×16× $\frac{5}{8}$ *	W30×108 RBS	N/A	0.05
1203	Planar	HSS 16×16× $\frac{5}{8}$ *	W30×90	N/A	0.04
1204	Planar	HSS 16×16× $\frac{5}{8}$ *	W30×90	N/A	0.04
2102	Biaxial	BU 16×16×1.25	W30×108 RBS	W30×148	0.05
2103	Biaxial	BU 16×16×1.25	W30×108 RBS	W30×148	0.06
2105	Biaxial	HSS 16×16× $\frac{1}{2}$	W21×55 RBS	W21×83	0.06
2106	Biaxial	BU 16×16×1.25**	W30×108 RBS	W30×148	0.05
2107	Biaxial	BU 16×16×1.25**	W30×108 RBS	W30×148	0.05
2111	Biaxial	BU 16×16×1.25***	W30×108 RBS	W30×148	0.047
2113	Biaxial	BU 16×16×1.25***	W30×108 RBS	W30×148	0.047

* Column consisted of HSS 16 with supplementary W12×136 housed within concrete fill.
 ** Built-up box fabricated using CJP groove welds.
 *** Built-up box fabricated using PJP groove welds with groove weld size equal to $\frac{3}{4}$ of flange thickness.
 † Beam flanges were trimmed to the indicated width in order to test the ability of the collar to withstand (a) narrow-flange beams [6 in. (150 mm) flange] and (b) maximum forces [9 in. (230 mm) flange].
 BU indicates built-up box section columns.

space frame. Thus, instead of a less redundant structure with more concentrated lateral force resistance, all or almost all beam-column connections are moment-resisting creating extensive redundancy. The distribution of moment connections throughout the structure also allows for reduced framing sizes and provides excellent floor vibration performance due to fixed-fixed beam end conditions. The highly distributed lateral force resistance also provides for reduced foundation loads and an inherently robust resistance to progressive collapse.

Finite element models of tested beam-column assemblies confirm that the contribution of concrete column fill can be accounted for using the gross transformed properties of the column. Beams and columns should be modeled without rigid end

offsets. Prescriptive reductions in beam stiffness to account for reduced beam section (RBS) property reductions are conservative for ConXL framing, as the RBS is located farther away from the column centerline than is typical of standard RBS connections. Therefore, modeling of ConXL assemblies employing RBS beams should model the reduced beam sections explicitly, rather than using prescriptive reductions in stiffness to account for the beam flange reduction.

Because ConXL systems have their lateral force resistance distributed throughout the structure, torsional resistance can be less than structures with required lateral force resistance concentrated on exterior lines. It is possible to minimize this effect by selecting stiffer members towards the building perimeter, to increase the torsional inertia.

10.2. SYSTEMS

The ConXL moment connection is unique in that it meets the prequalification requirements for special and intermediate moment frames in orthogonal intersecting moment-resisting frames. It can also be used in more traditional plane frame applications. These requirements are met with a single standardized connection.

The exception associated with concrete structural slab placement at the column and collar assembly is based on testing conducted on the stiffened extended end-plate moment connection (Seek and Murray, 2005). Early testing by Murray of a bolted-end-plate specimen with a concrete slab in place failed by tensile rupture of the bolts. This was postulated to be the result of composite action between the beam and slab, resulting in increased beam flexural strength and increased demands on the bolt relative to calculated demands neglecting composite effects. Later testing referenced previously demonstrated that placement of a flexible material in the slab adjacent to the column sufficiently reduced this composite action and protected the bolts. Although ConXL connections have not been tested with slabs present, it is believed that the same protective benefits of the flexible material apply to this connection.

ConXL's highly distributed lateral force resistance reduces the need for metal deck/concrete fill to act as a diaphragm and drag forces to a limited number of moment resisting frames. Each moment-resisting column and connected beams resist a tributary lateral load and typically minimal concrete reinforcement or deck attachment is required.

10.3. PREQUALIFICATION LIMITS

1. Beam Limitations

Minimum beam depth is controlled by the collar dimensions and is 18 in. (460 mm). Maximum beam depth is controlled by strong-column weak-beam considerations and is limited to 30 in. (760 mm) for practical purposes. The flange width and thickness requirements are limited by the ability of the collar flange to accommodate the beam flange weld and also by the strength of the bolts. A key ConXL requirement for allowable beam sections is limiting the force delivered by the beam to the bolts connecting the collar flange/beam to the collar corner assemblies/column so as to

not overcome the pretension load applied to the bolts. This requirement is covered in detail in Section 10.8.

ConXL connections have been successfully tested without reduced beam section reductions in flange width and are qualified for use without such reductions. However, RBS cuts in beam flanges can be a convenient way to achieve strong column weak beam limitations without increasing column weight.

Lateral bracing of beams is in accordance with the AISC *Seismic Provisions*. During the biaxial moment connection tests, the test beams (W30×108 with 50% RBS, W21×55 with 50% RBS) were not braced at the RBS and were braced at the beam ends, 10 ft (3 m) from the column center.

All moment-connected beams are required to meet seismic compaction requirements of the AISC *Seismic Provisions*, if RBS beams are used, the width-to-thickness ratio is taken within its reduced flange width as permitted for RBS connections [Section 5.3.1(6)].

2. Column Limitations

The key requirement for ConXL moment columns is a square sectional dimension of 16 in. (400 mm). Section type (built-up box or HSS) can vary, as can steel strength and wall thickness. All columns used in ConXL moment connections are concrete-filled with either normal or lightweight concrete, having minimum compressive strength of 3,000 psi (21 MPa). Columns are typically filled with concrete at the job site after erection and bolting is complete. The concrete is pumped to the top of column and allowed to free-fall the full height of column, using the column as a tremie. There are no obstructions, stiffener plates, etc., within the column; thus, the column is similar to a tremie-pipe allowing the concrete an unobstructed path to its placement with excellent consolidation (Suprenant, 2001).

Two biaxial beam-column tests (Tests 2111 and 2113, Table C-10.1) were performed with built-up box section columns fabricated using PJP groove welds to join the box section flange and web plates as illustrated in Figure 10.5. In each case, the column marginally met biaxial strong column-weak beam requirements, and some limited yielding of column flanges was observed in later stages of the tests. Both tests reached total rotation demands of 0.047 rad without failure or noticeable loss of load carrying ability. In addition, a single cantilever column test (Test 1120) was conducted to evaluate the inelastic behavior of box columns with PJP groove welds. In this test, a 16-in.-square built-up box column with 1¼ in.-thick plates joined using a PJP groove weld size of 1½ in. was subjected to uniaxial ramped cyclic loading in a cantilever condition while maintaining an applied axial load of 640 kips (approximately 9-ksi axial stress). The specimen was loaded to six cycles of displacement each to story drift ratios of 0.00375, 0.005 and 0.0075; four cycles at 0.010 rad; and two cycles each at 0.015, 0.020, 0.030 and 0.045 rad. Initial yielding was observed to occur at displacements of 0.0075 rad. Minor bowing of the flange plates occurred at 0.045 rad. The test was terminated without failure and while still exhibiting positive

strain hardening after loading to two cycles at 0.045 rad. No evidence of distress to the PJP welds in any of these tests was observed.

3. Collar Limitations

Appendix B describes the forged steel material specification used to manufacture the collars. The forging process produces an initial collar (blank collar) slightly larger than the final overall dimensions. The collar is then machined to their manufacturing dimensions within the required tolerances.

10.4. COLLAR CONNECTION LIMITATIONS

The collars are the key elements of the ConXL connection. They are standardized components, and no further design or sizing of these components is required. The same components are used for all beams and columns. The same is true for the collar bolts, where the specification, size, and number of bolts always remain the same. The design procedure ensures that column-beam combinations used in the ConXL connection fall within the code requirements of these standard connection components.

The bolts used in the ConXL connection are 1¼-in.-diameter ASTM A574 bolts. These bolts are similar in chemistry and mechanical properties to ASTM F3125 Grade A490 bolts but have socket heads to accommodate their use in this connection. Metric bolts conforming to ASTM A574M have not been tested and are not prequalified for use in this connection. Pretensioning is performed to the requirements for 1¼-in.-diameter ASTM F3125 Grade A490 bolts [102 kips (454 kN) in accordance with Table J3.1 of the 2016 AISC *Specification*].

10.5. BEAM WEB-TO-COLLAR CONNECTION LIMITATIONS

The collar web extension (CWX) is ½ in. (38 mm) thick; thus, the minimum sized fillet weld between the CWX and beam web is a 5/16-in. (8-mm) fillet weld. This weld size for a two-sided fillet weld (each side of the web) should be sufficient for all allowable beams; this should be confirmed during the design procedure calculations.

10.6. BEAM FLANGE-TO-COLLAR FLANGE WELDING LIMITATIONS

Welding of the beam flange to the collar flange is performed in a proprietary ConX-tech beam weld fixture, which rotates the beam to allow access to the bottom flange for welding in the flat position. The beam weld fixture enables the manufacturing of the moment beam within ConXL tolerances.

10.7. COLUMN-BEAM RELATIONSHIP LIMITATIONS

The ConXL moment connection is a biaxial connection. Strong-column/weak-beam requirements specified by the AISC *Seismic Provisions* were formulated considering the typical planar framing prevalent in moment-frame construction following the 1994 Northridge earthquake. Because the ConXL connection is primarily used in intersecting moment frames, with biaxial behavior an inherent part of the design, the committee felt that it was imperative to require that columns have sufficient strength

to develop expected simultaneous flexural hinging in beams framing into all column faces. The biaxial calculation considers all moment beams attached to the column. This calculation is covered in detail in Section 10.8.

10.8. DESIGN PROCEDURE

Step 1. As with other connections, the first step in the design procedure is to compute the probable maximum moment at the plastic hinge. Note that differing C_{pr} factors are applied for RBS and non-RBS beams. The factor for non-RBS beams is compatible with the standard requirements in the AISC *Seismic Provisions* while that for RBS beams is compatible with the requirements of this standard for RBS connections.

Step 2. As with other connections, the equation given for computation of shear forces has to include consideration of gravity loads that are present. The equations presented in the design procedure assume uniform gravity loading. Modifications to these equations are necessary for cases with concentrated loads present. These modifications must satisfy static equilibrium requirements.

Step 3. The ConXL moment connection is a true biaxial moment connection; thus, the committee determined that columns must be sufficiently strong to permit simultaneous development of flexural hinging in all beams framing to a column, not just beams along a single plane. This biaxial column-beam moment evaluation is more conservative than current AISC *Seismic Provisions* requirements that consider plastic hinging of beams in a single plane only, even though columns supporting moment frames in orthogonal directions are possible with other connections using built-up box sections or other built-up column sections. In calculating the ConXL biaxial column-beam moment ratio, it is permitted to take the actual yield strength of the column material in lieu of the specified minimum yield stress, F_y , and to consider the full composite behavior of the column for axial load and flexural action (interstory drift analysis). The default formula for column strength provided in the design procedure assumes that equal strength beams are present on all faces of the connection. When some beams framing to a column are stronger than others, it is permitted to use basic principles of structural mechanics to compute the actual required flexural strength.

The design procedure also considers the critical beam strength as it relates to the column strength at locations just above the beam's top flange and just below the beam's bottom flange, where flexural demand on the columns are greatest. Flexural demand on the column within the panel zone is less than at these locations.

Step 5. The available tensile strength for the bolts used in the ConXL connection is specified as the minimum bolt pretension load. The purpose of assigning the minimum pretension load as the available bolt tensile strength is to prevent overcoming of bolt pretension, at least up to the bolt loading subjected by the probable maximum moment. The minimum bolt pretension load is 102 kips (454 kN). Bolts are checked for tension only because the frictional force developed by the bolt pretension will resist beam shear (see **Steps 6** and **7**).

Steps 6 and 7. Beam shear is resisted by the friction developed between the collar flanges and the collar corners. The collar flanges are clamped against the collar corner assemblies and column when the collar bolts are pretensioned. This pretension clamping force creates friction between the machined surfaces of the collar flanges and collar corners. The machined surfaces are classified as a Class B surface (unpainted blast-cleaned steel surfaces). From AISC *Specification* Section J3.8, the design frictional resistance per bolt is:

$$R_n = \mu D_u h_{sc} T_b n_s \quad (\text{Spec. Eq. J3-4})$$

$$\phi = 0.85 \text{ for oversized bolt holes}$$

$$\mu = 0.50$$

$$D_u = 1.13$$

$$h_{sc} = 1.0$$

$$T_b = 102 \text{ kips (454 kN)}$$

$$n_s = 1$$

$$\phi R_n = (0.85)(0.50)(1.13)(1.0)(102)(1) = 49.0 \text{ kips/bolt (218 kN/bolt)}$$

There are 16 bolts per beam end providing a total of 784 kips (3490 kN) of frictional resistance against shear. This frictional force is significantly greater than any beam shear developed by an allowable beam.

Steps 8 and 9. The available length of weld for the collar web extension and collar corner assemblies allow for minimum sized fillet welds to resist beam shear.

Steps 10 and 11. The collar corner assemblies provide additional strength to the column walls to resist panel zone shear. Without taking into consideration the contribution of the concrete fill, the column section along with the collar corner assemblies should provide sufficient strength for anticipated panel zone shear; this should be confirmed during the design procedure calculations.

CHAPTER 11

SIDEPLATE MOMENT CONNECTION

11.1. GENERAL

The SidePlate[®] moment connection is a post-Northridge connection system that uses a configuration of redundant interconnecting structural plate and fillet weld groups, which act as positive and discrete load transfer mechanisms to resist and transfer applied moment, shear and axial load from the connecting beam(s) to the column. This load transfer minimizes highly restrained conditions and triaxial strain concentrations that typically occur in flange-welded moment connection geometries. The connection system is used for both new and retrofit construction and for a multitude of design hazards such as earthquakes, extreme winds, and blast and progressive collapse mitigation.

The wide range of applications for SidePlate connection technology, including the methodologies used in the fabrication and erection shown herein, are protected by one or more U.S. and foreign patents identified at the bottom of the first page of Chapter 11. Information on the SidePlate moment connection can be found at www.sideplate.com. The connection is not prequalified when side plates of an unlicensed design and/or manufacturer are used.

SidePlate Systems Inc. developed, tested and validated SidePlate connection design methodology, design controls, critical design variables, and analysis procedures. The development of the SidePlate FRAME[®] configuration that employs the full-length beam erection method (which uses a full-length beam assembly fillet-welded in the field to a column assembly to achieve maximum shop fabrication and field erection efficiencies) builds off the research and testing history of its proven predecessor—the original SidePlate steel frame connection system that employs the link-beam erection method (which uses column tree assemblies with shop-installed beam stubs, which are then connected in the field to a link beam using CJP welds) and its subsequent refinements. It represents the culmination of an ongoing research and development program (since 1994), which has resulted in further performance enhancements: optimizing the use of connection component materials with advanced analysis methods and maximizing the efficiency, simplicity and quality control of its fabrication and erection processes. Following the guidance of the AISC *Seismic Provisions*, the validation of the SidePlate FRAME configuration consists of:

- (a) Analytical testing conducted by SidePlate Systems using nonlinear finite element analysis (FEA) for rolled shapes, plates and welds and validated inelastic material properties by physical testing.
- (b) Physical validation testing conducted at the Lehigh University Center for Advanced Technology for Large Structural Systems (ATLSS) (Hodgson et al.,

2010a, 2010b, and 2010c; a total of six cyclic tests) and at University of California, San Diego (UCSD), Charles Lee Powell Laboratories (Minh Huynh and Uang, 2012; a total of three cyclic tests). The purpose of these tests was to confirm global inelastic rotational behavior of parametrically selected member sizes, corroborated by analytical testing, and to identify, confirm and accurately quantify important limit state thresholds for critical connection components to objectively set critical design controls. The third cyclic test at UCSD was a biaxial moment connection test that subjected the framing in the orthogonal plane to a constant shear, creating a moment across the column-beam joint equivalent to that created by the probable maximum moment at the plastic hinge of the primary beam, while the framing in the primary plane was simultaneously subjected to the qualifying cycle loading specified by the AISC *Seismic Provisions* (AISC, 2016a). Tests on SidePlate moment connections, both uniaxial and biaxial applications, show that yielding is generally concentrated within the beam section just outside the ends of the two side plates. Peak strength of specimens is usually achieved at an interstory drift angle of approximately 0.03 to 0.05 rad. Specimen strength then gradually reduces due to local and lateral-torsional buckling of the beam. Ultimate failure typically occurs at interstory drift angles of approximately 0.04 to 0.06 rad by low-cycle fatigue fracture from local buckling of the beam flanges and web.

To ensure predictable, reliable and safe performance of the SidePlate FRAME configuration when subjected to severe load applications, the inelastic material properties, finite element modeling (FEM) techniques, and analysis methodologies that were used in its analytical testing were initially developed, corroborated and honed based on nonlinear analysis of prior full-scale physical testing of the original SidePlate connection. The prior physical testing consisted of a series of eight uniaxial cyclic tests, one biaxial cyclic test conducted at UCSD, and a separate series of large-scale arena blast tests and subsequent monotonic progressive collapse tests: two blast tests (one with and one without a concrete slab present), two blast-damaged progressive collapse tests, and one non-blast damaged test, conducted by the Defense Threat Reduction Agency (DTRA) of the U.S. Department of Defense (DoD), at the Kirtland Air Force Base, Albuquerque, NM. This extensive effort has resulted in the ability of SidePlate Systems to:

- (a) Reliably replicate and predict the global behavior of the SidePlate FRAME configuration compared to actual tests.
- (b) Explore, evaluate and determine the behavioral characteristics, redundancies, and critical limit state thresholds of its connection components.
- (c) Establish and calibrate design controls and critical design variables of the SidePlate FRAME configuration, as validated by physical testing.

Connection prequalification is based on the completion of several carefully prescribed validation testing programs, the development of a safe and reliable plastic capacity design methodology that is derived from ample performance data from

24 full-scale tests of which two were biaxial, and the judgment of the CPRP. The connection prequalification objectives have been successfully completed; the rudiments are summarized here:

- (a) System-critical limit states have been identified and captured by physical full-scale cyclic testing and corroborated through nonlinear FEA.
- (b) The effectiveness of identified primary and secondary component redundancies of the connection system has been demonstrated and validated through parametric performance testing—both physical and analytical.
- (c) Critical behavioral characteristics and performance nuances of the connection system and its components have been identified, captured and validated.
- (d) Material sub-models of inelastic stress/strain behavior and fracture thresholds of weld consumables and base metals have been calibrated to simulate actual behavior.
- (e) Sufficient experimental and analytical data on the performance of the connection system have been collected and assessed to establish the likely yield mechanisms and failure modes.
- (f) Rational nonlinear FEA models for predicting the resistance associated with each mechanism and failure mode have been employed and validated through physical testing.
- (g) Based on the technical merit of the preceding accomplishments, a rational ultimate strength design procedure has been developed based on physical testing, providing confidence that sufficient critical design controls have been established to preclude the initiation of undesirable mechanisms and failure modes and to secure expected safe levels of cyclic rotational behavior and deformation capacity of the connection system for a given design condition.

11.2. SYSTEMS

The SidePlate moment connection meets the prequalification requirements for special and intermediate moment frames in both traditional in-plane frame applications (one or two beams framing into a column) as well as orthogonal intersecting moment-resisting frames (corner conditions with two beams orthogonal to one another, as well as three or four orthogonal beams framing into the same column).

The SidePlate moment connection has been used in moment-resisting frames with skewed and/or sloped beams with or without skewed side plates, although such usage is outside of the scope of this standard.

SidePlate's unique geometry allows its use in other design applications where in-plane diagonal braces or diagonal dampers are attached to the side plates at the same beam-to-column joint as the moment-resisting frame while maintaining the intended SMF or IMF level of performance. When such dual systems are used, supplemental calculations must be provided to ensure that the connection elements (plates and

welds) have not only been designed for the intended SMF or IMF connection in accordance with the prequalification limits set herein, but also for the additional axial, shear and moment demands due to the diagonal brace or damper.

11.3. PREQUALIFICATION LIMITS

1. Beam Limitations

A wide range of beam sizes, including both wide flange and HSS beams, has been tested with the SidePlate moment connection. For wide-flange sections, the smallest beam size was a W18×35 (W460×52) and the heaviest a W40×297 (W1000×443). Beam compactness ratios have varied from that of a W18×35 (W460×52) with $b_f/2t_f = 7.06$ to a W40×294 (W1000×438) with $b_f/2t_f = 3.11$. For HSS beam members, tests have focused on small members such as the HSS7×4×½ (HSS177.8×101.6×12.7) having ratios of $b/t = 5.60$ and $h/t = 12.1$. As a result of the SidePlate testing programs, critical ultimate strength design parameters for the design and detailing of the SidePlate moment connection system have been developed for general project use. These requirements and design limits are the result of a detailed assessment of actual performance data coupled with independent physical validation testing and/or corroborative analytical testing of full-scale test specimens using nonlinear FEA. It was the judgment of the CPRP that the maximum beam depth and weight of the SidePlate moment connection would be limited to the nominal beam depth and approximate weight of the sections tested, as has been the case for all other connections.

Because the behavior and overall ductility of the SidePlate moment connection system is defined by the plastic rotational capacity of the beam, the limit state for the SidePlate moment connection system is ultimately the failure of the beam flange, away from the connection. Therefore, the limit of the beam's hinge-to-hinge span-to-depth ratio of the beam, L_h/d , is based on the demonstrated rotational capacity of the beam.

As an example, for test specimen 3 tested at Lehigh University (Hodgson et al., 2010c), the W40×294 (W1000×438) beam connected to the W36×395 (W920×588) column reached two full cycles at 0.06 rad of rotation (measured at the centerline of the column), which is significantly higher than the performance threshold of one cycle at 0.04 rad of rotation required for successful qualification testing by the AISC *Seismic Provisions*. Most of the rotation at that amplitude came from the beam rotation at the plastic hinge. With the rotation of the column at 0.06 rad, the measured rotation at the beam hinge was between 0.085 and 0.09 rad [see Figure C-11.1(a)]. The tested half-span was 14.5 ft (4.42 m), which represents a frame span of 29 ft (8.84 m) and an L_h/d ratio of 5.5. Assuming that 100% of the frame system's rotation comes from the beam's hinge rotation (a conservative assumption because it ignores the rotational contributions of the column and connection elements), it is possible to calculate a minimum span at which the frame drift requirement of one cycle at 0.04 rad is maintained, while the beam reaches a maximum of 0.085 rad of rotation. Making this calculation gives a minimum span of 20 ft (6.1 m) and an L_h/d ratio of 3. Making this same calculation for the tests of the W36×150 (W920×223) beam

[Minh Huynh and Uang, 2012; Figure C-11.1(b)] using an average maximum beam rotation of 0.08 rad of rotation, gives a minimum span of 18 ft 10 in. (5.74 m) and an L_h/d ratio of 3.2. Given that there will be variations in the performance of wide-flange beams due to local effects such as flange buckling, it is reasonable to set the lower bound L_h/d ratio for the SidePlate moment connection system at 4.5 for SMF using the U-shaped cover plate and 3.0 for IMF using the U-shaped cover plate, regardless of beam compactness. It should be noted that the minimum L_h/d ratio of 4.5 (where L_h is measured from the centerline of the beam's plastic hinges) typically equates to 6.7 as measured from the face of column to face of column when the typical side plate extension of 0.77d (shown as "Side plate {A} extension" in Figure 11.5) from face of column is used. The 6.7 ratio, which is slightly less than the 7.0 for other SMF moment connections, allows the potential for a deeper beam to be used in a shorter bay than other SMF moment connections.

All moment-connected beams are required to satisfy the width-to-thickness requirements of AISC *Seismic Provisions* Sections E2 and E3.

Required lateral bracing of the beam follows the AISC *Seismic Provisions*. However, due to the significant lateral and torsional restraint provided by the side plates as observed in past full-scale tests, for calculation purposes, the unbraced length of the beam is taken as the distance between the respective ends of each side plate extension (see Figures 11.10 through 11.15 for depictions of the alphabetical designations). As determined by the full-scale tests, no additional lateral bracing is required at or near the plastic beam hinge location.

The protected zone is defined as shown in Figures 11.6 and 11.7 and extends from the end of the side plate to one-half the beam depth beyond the plastic hinge location, which is located at one-third the beam depth beyond the end of the side plate. This definition is based on test observations that indicate yielding typically does not extend past 83% of the depth of the beam from the end of the side plate.

2. Column Limitations

SidePlate moment connections have been tested with W14 (W360), W16 (W410), W30 (W760), and W33 (W840) built-up I-sections and a rolled W36 (W920). Although no tested data are available for test specimens using built-up box columns, the side plates transfer the loads to the column in the same way as with wide-flange columns. The only difference is that the horizontal shear component at the top and bottom of the side plates {A} now transfer that horizontal shear directly into the face of the built-up box column web using a shop fillet weld, and thus, an internal horizontal shear plate or stiffener is not required. As such, built-up box columns are prequalified as long as they meet all applicable requirements of the AISC *Seismic Provisions*. There are no internal stiffener plates within the column, and there are no requirements that the columns be filled with concrete for either SMF or IMF applications. However, in some blast applications, there may be advantages to filling the HSS or built-box columns with concrete to strengthen the column walls in such extreme loading applications.

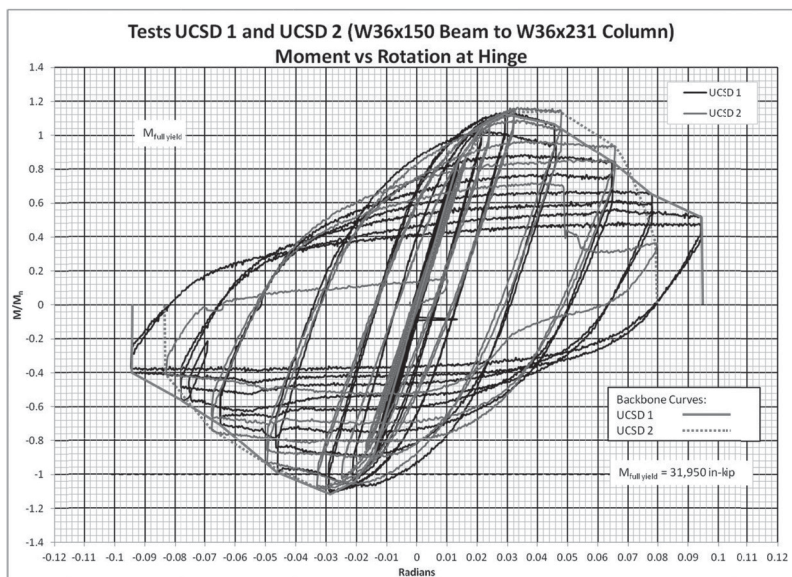
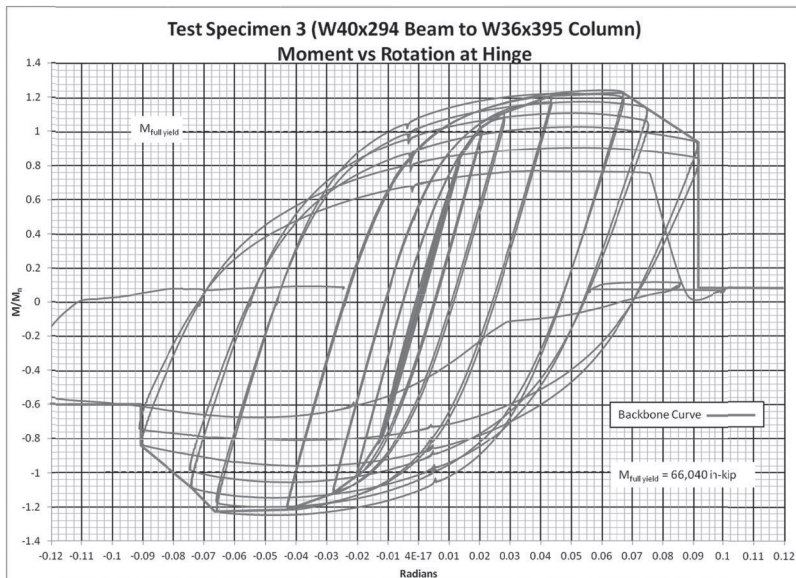


Fig. C-11.1. SidePlate tests—backbone curves for (a) W40×294 (W1000×438) beam; and (b) W36×150 (W920×223) beam (measured at the beam hinge location).

The behavior of SidePlate connections with cruciform columns is similar to uniaxial one- and two-sided moment connection configurations because the ultimate failure mechanism remains in the beam. Successful tests have been conducted on SidePlate connections with cruciform columns using W36 (W920) shapes with rolled or built-up structural tees.

For SMF systems, the column bracing requirements of AISC *Seismic Provisions* Section E3.4c.1 are satisfied when a lateral brace is located at or near the intersection of the frame beams and the column. Full-scale tests have demonstrated that the full-depth side plates provide the required indirect lateral bracing of the column flanges through the side plate-to-column flange welds and the connection elements that connect the column web to the side plates. Therefore, no additional direct lateral bracing of the column flanges is required.

3. Connection Limitations

All test specimens have used ASTM A572/A572M Grade 50 plate material. Nonlinear finite element parametric modeling of side plate extensions in the range of $0.65d$ to $1.0d$ has demonstrated similar overall connection and beam behavior when compared to the results of full-scale tests.

Because there is a controlled level of plasticity within the design of the two side plates, the side plate protected zones have been designated based upon test observations. Protected zones are indicated in Figures 11.6 and 11.7.

11.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

See Figures 11.10 through 11.15 for depictions of the alphabetical and numerical designations. The beams and columns selected must satisfy physical geometric compatibility requirements between the beam flange and column flange to allow sufficient lateral space for depositing fillet welds {5} along the longitudinal edges of the beam flanges that connect to the top and bottom cover plates {B}. Equations 11.4-1 and 11.4-1M assist designers in selecting appropriate final beam and column size combinations prior to the SidePlate connection actually being designed for a specific project.

Unlike more conventional moment frame designs that typically rely on the deformation of the column panel zone to achieve the required rotational capacity, SidePlate technology instead stiffens and strengthens the column panel zone by providing a minimum of three panel zones (the column web plus the two full-depth side plates). This configuration forces the vast majority of plastic deformation to occur through flange local buckling of the beam.

The column web must be capable of resisting the panel zone shear loads transferred from the horizontal shear plates {D} through the pair of shop fillet welds {3}. The strength of the column web is thereby calculated and compared to the ultimate strength of the welds {3} on both sides of the web. To be acceptable, the panel zone shear strength of the column must be greater than the strength of the two welds. This ensures that the limit state will be failure of the welds as opposed to failure of the

column web. The following calculation and check is built into the SidePlate connection design software:

$$\frac{R_u}{R_n} < 1.0 \quad (\text{C-11.4-1})$$

where

R_n = nominal strength of column web panel zone in accordance with AISC *Specification* Section J10.6b, kips (N)

R_u = ultimate strength of fillet welds {3} from horizontal shear plates to column web, kips (N)

$$R_n = 0.60F_y d_c t_{cw} \left(1 + \frac{3b_{fc} t_{fc}^2}{d_{sp} d_c t_{cw}} \right) \quad (\text{from } Spec. \text{ Eq. J10-11})$$

where

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

d_c = depth of column, in. (mm)

d_{sp} = depth of SidePlate, in. (mm)

t_{cw} = thickness of column web, in. (mm)

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

In determining the SMF column-beam moment ratio to satisfy strong column/weak beam design criteria, the beam-imposed moment, M_{pb}^* , is calculated at the column centerline using statics (i.e., accounting for the increase in moment due to shear amplification from the location of the plastic hinge to the center of the column, due to the development of the plastic moment capacity, M_{pr} , of the beam at the plastic hinge location) and then linearly decreased to one-quarter the column depth above and below the extreme top and bottom fibers of the side plates. This location is used for determination of the column strength as the column is unlikely to form a hinge within the panel zone due to the presence and strengthening effects of the two side plates.

This requirement need not apply if any of the exceptions articulated in AISC *Seismic Provisions* Section E3.4a are satisfied. The calculation and check is included in the SidePlate connection design software.

11.5. CONNECTION WELDING LIMITATIONS

Fillet welds joining the connection plates to the beam and column provided on all of the SidePlate test specimens have been made by either the self-shielded flux cored arc welding process (FCAW-S or FCAW-G) with a few specimens using the submerged arc welding process (SAW) for certain shop fillet welds. Other than the original three prototype tests in 1994 and 1995 that used a non-notch-tough weld electrode, tested electrodes satisfy minimum Charpy V-notch toughness as required by the 2010 AISC *Seismic Provisions*. Test specimens that included either a field complete-joint-penetration groove-welded beam-to-beam splice or field fillet welds specifically utilized E70T-6 for the horizontal position and E71T-8 for the vertical position.

11.6. CONNECTION DETAILING

Figures 11.10 through 11.12 show typical one- and two-sided moment connection details used for shop fabrication of the column with fillet welds. Tests have shown that the horizontal shear plate {D} need not be welded to the column flanges for successful performance of the connection. However, if there are orthogonal forces being transferred through the connection from collector, chord or cantilever beams, then fillet welds connecting the horizontal shear plates and the column flanges are required.

Tests have shown that the use of oversized bolt holes in the side plates, located near their free end (see Figure C-11.2), does not affect the performance of the connection because beam moments and shears are transferred through fillet welds. Bolts from the side plate to the vertical shear element are only required for erection of the full-length beam assembly prior to field welding of the connection.

Figure 11.13 shows the typical full-length beam detail used for shop fabrication of the beam with fillet welds. Multiple options can be used to create the vertical shear element, such as a combination of angles and plates or simply bent plates. Figure 11.14 shows the typical full-length beam-to-side plate detail used for field erection of the beam with fillet welds.

11.7. DESIGN PROCEDURE

The design procedure for the SidePlate connection system is based on results from both physical testing and detailed nonlinear finite element modeling. The procedure uses an ultimate strength design approach to size the plates and welds in the connection, incorporating strength, plasticity and fracture limits. For welds, an ultimate strength analysis incorporating the instantaneous center of rotation is used (as described in the AISC *Steel Construction Manual* Part 8). Overall, the design process is consistent with the expected seismic behavior of an SMF system: Lateral drifts due to seismic loads induce moments and shear forces in the columns and beams. Where these moments exceed the yield capacity of a beam, a plastic hinge will form. While the primary yield mechanism is plastic bending in the beam, a balanced design approach allows for secondary plastic bending to occur within the side plates. Ultimately, the location of the hinge in the beam directly affects the amplification of load (i.e., moment and shear from both seismic and gravity) that is resisted by the components of the connection, the column panel zone, and the column (as shown in Figure C-11.2). The capacity of each connection component can then be designed to resist its respective load demands induced by the seismic drift (including any increase due to shear amplification as measured from the beam plastic hinge).

For the SidePlate moment connection, all of the connection details, including the sizing of connection plates and fillet welds, are designed and provided by engineers at SidePlate Systems Inc. The design of these details is based upon basic engineering principles, plastic capacities validated by full-scale testing, and nonlinear finite element analysis. A description of the design methods is presented in **Step 7**. The initial design procedure for the engineer of record in designing a project with SidePlate moment connections largely involves:

- Sizing the frame's beams and columns, shown in **Steps 1 and 2**.
- Checking applicable building code requirements and performing a preliminary compliance check with all prequalification limitations, shown in **Steps 3 and 4**.
- Verifying that the SidePlate moment connections have been designed with the correct project data as outlined in **Step 5** and are compliant with all prequalification limits, including final column-beam relationship limitations as shown in **Steps 6, 7 and 8**.

Step 1. Equations 11.4-1/11.4-1M should be used as a guide in selecting beam and column section combinations during design iterations.

Satisfying these equations minimizes the possibility of incompatible beam and column combinations that cannot be fabricated and erected or that may not ultimately satisfy column-beam moment ratio requirements.

Step 2. The SidePlate connection design forces a plastic hinge to form in the beam beyond the extension of the side plates from the face of the column (dimension A in Figure 11.5). Because inelastic behavior is forced into the beam at the hinge, the effective span of the beam is reduced, thus increasing the lateral stiffness and strength of the frame (see Figure C-11.3). This increase in stiffness and strength provided by the two parallel side plates should be simulated when creating elastic models of the steel frame. Many commercial structural analysis software programs have a built-in feature for modeling the stiffness and strength of the SidePlate connection.

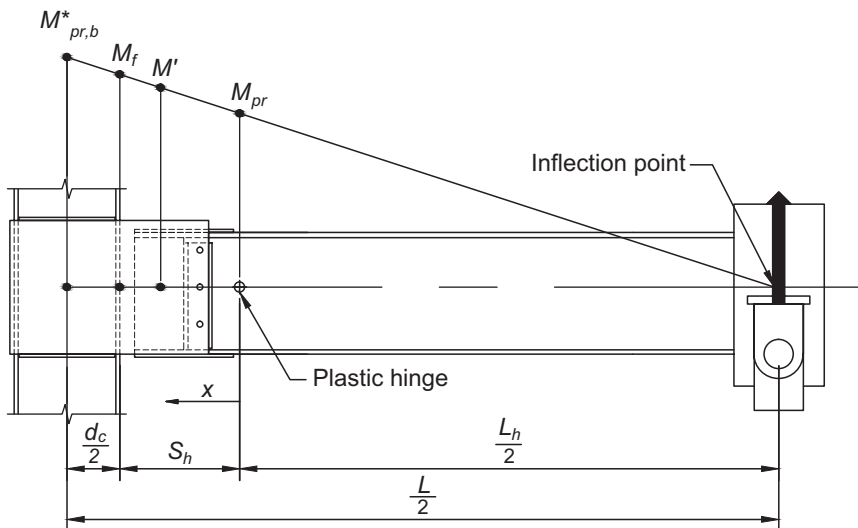


Fig. C-11.2. Amplification of maximum probable plastic hinge moment, M_{pr} , to the column face.

Step 5. Some structural engineers design moment-frame buildings with a lateral-only computer analysis. The results are then superimposed with results from additional lateral and vertical load analysis to check beam and column stresses. Because these additional lateral and vertical loads can affect the design of the SidePlate moment connection, they must also be submitted with the lateral-only model forces. Such additional lateral and vertical loads include drag and chord forces, factored shear loads at the plastic hinge location due to gravity loads on the moment frame beam itself, loads from gravity beams framing into the face of the side plates, and gravity loads from cantilever beams (including vertical loads due to earthquakes) framing into the face of the side plates.

There are instances where an in-plane lateral drag or chord axial force needs to transfer through the SidePlate moment connection, as well as instances where it is necessary to transfer lateral drag or chord axial forces from the orthogonal direction through the SidePlate moment connection. In such instances, these loads must be submitted in order to properly design the SidePlate moment connection for these conditions.

Step 6 of the procedure requires SidePlate Systems to review the information received from the structural engineer, including the assumptions used in the generation of final beam and column sizes to ensure compliance with all applicable building code requirements and prequalification limitations contained herein. Upon reaching concurrence with the structural engineer of record that beam and column sizes are acceptable and final, SidePlate Systems designs and details all of the SidePlate moment connections for a specific project in accordance with **Step 7**.

The SidePlate moment connection design procedure is based on the idealized primary behavior of an SMF system: the formation of a plastic hinge in the beam, outside of the connection. Although the primary yield mechanism is development of a plastic hinge in the beam near the end of the side plate, secondary plastic behavior (plastic

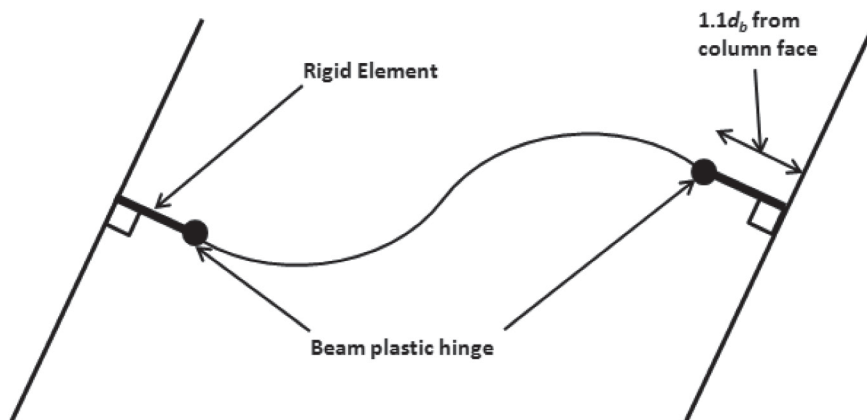


Fig. C-11.3. Increased frame stiffness with reduction in effective span of the beam.

moment capacity) is developed within the side plates themselves, at the face of the column. Overall, a balanced design is used for the connection components to ensure that the plastic hinge will form at the predetermined location. The demands on the connection components are a function of the strain-hardened moment capacity of the beam, the gravity loads carried by the beam, and the relative locations of each component and the beam's plastic hinge. Connection components closer to the column centerline are subjected to increased moment amplification compared to components located closer to the beam's plastic hinge as illustrated in Figure C-11.2.

Step 7 of the process requires that SidePlate Systems design and detail the connection components for the actions and loads determined in Step 6. The procedure uses an ultimate strength design approach to size plates and welds that incorporates strength, plasticity and fracture limits. For welds, an ultimate strength analysis incorporating the instantaneous center of rotation is used (as described in the AISC *Steel Construction Manual* Part 8). Overall, the design process is consistent with the expected seismic behavior of an SMF system as described previously.

The SidePlate moment connection components are divided into four distinct design groups:

- (a) Load transfer out of the beam.
- (b) Load transfer into the side plates {A}.
- (c) Design of the side plates {A} at the column face.
- (d) Load transfer into the column.

The transfer of load out of the beam is achieved through welds {4} and {5}. The loads are, in turn, transferred through the vertical shear elements {E} and cover plates {B} into the side plates {A} by welds {6} and {7}, respectively. The load at the column face (gap region) is resisted solely by the side plates {A}, which transfers the load directly into the column through weld {2} and indirectly through weld {3} through the combination of weld {1} and the horizontal shear plates {D}. At each of the four design locations, the elements are designed for the combination of moment, M_{group} , and shear, V_u .

Connection Design

Side Plate {A}. To achieve the balanced design for the connection—the primary yield mechanism developing in the beam outside of the connection with secondary plastic behavior within the side plates—the required minimum thickness of the side plate is calculated using an effective side plate plastic section modulus, Z_{eff} , generated from actual side plate behavior obtained from stress and strain profiles along the depth of the side plate, as recorded in test data and nonlinear analysis (see Figure C-11.4). The moment capacity of the plates, $M_{n,sp}$, is then calculated using the simplified Z_{eff} and an effective plastic stress, F_{ye} , of the plate. Allowing for yielding of the plate as observed in testing and analyses (Figure C-11.5) and comparing to the design demand M_{group} calculated at the face of column gives:

$$\frac{M_{group}}{M_{n,sp}} \leq 1.0 \quad (C-11.7-1)$$

where

$$M_{n,sp} = F_{ye} Z_{eff}$$

To ensure the proper behavior of the side plates and to preclude undesirable limit states such as buckling or rupture of the side plates, the ratio of the gap distance between the end of the beam and the face of the column to the side plate thickness is kept within a range for all connection designs. The optimum gap-to-thickness ratio has been derived based upon the results of full-scale testing and parametric nonlinear analysis.

Cover Plate {B}. The thickness of the cover plates {B} is determined by calculating the resultant shear force demand, R_u , from the beam moment couple as:

$$R_u = (M_{group}/d) \quad (C-11.7-2)$$

and by calculating the vertical shear loads, resisted through the critical shear plane of the cover plates {B}.

The critical shear plane is defined as a section cut through the cover plate {B} adjacent to the boundary of weld {7}, as shown in Figure C-11.6. Hence, the thickness, t_{cp} , of the cover plates is:

$$t_{cp} = \frac{R_u}{2(0.6)F_{ye}L_{crit}} \quad (C-11.7-3)$$

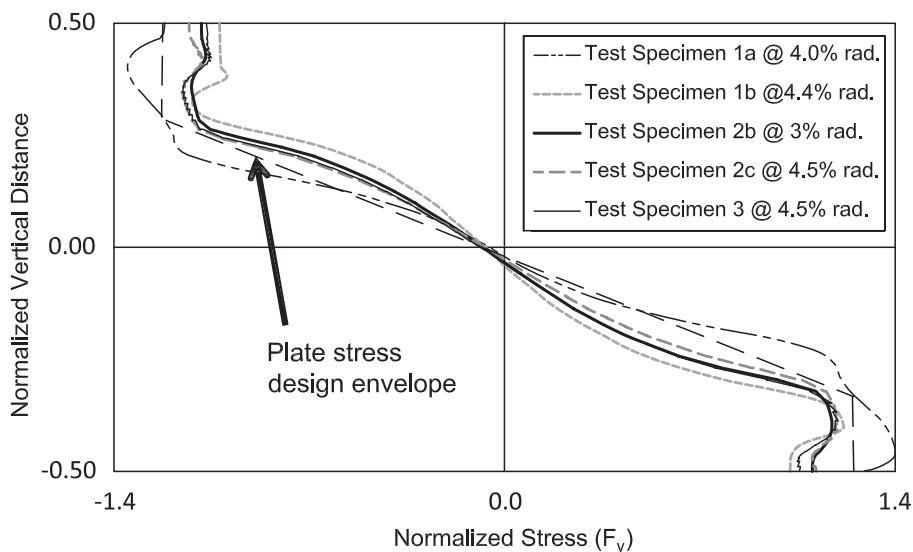


Fig. C-11.4. Stress profile along depth of side plate at the column face at maximum load cycle.

where

L_{crit} = length of critical shear plane through cover plate as shown in Figure C-11.6, in. (mm)

Vertical Shear Element (VSE) {F}. The thickness of the VSE {F} (which may include angles {E} and/or bent plates {C}, see Figures 11.10 through 11.15) is determined as the thickness required to transfer the vertical shear demand from the beam web into the side plates {A}. The vertical shear force demand, V_u , at this load transfer comes from the combination of the capacities of the cover plates and the VSE. The minimum thickness of the VSE, t_{vse} , to resist the vertical shear force is computed as follows:

$$t_{vse} = \frac{V_u'}{2(0.6)F_y d_{pl}} \quad (C-11.7-4)$$

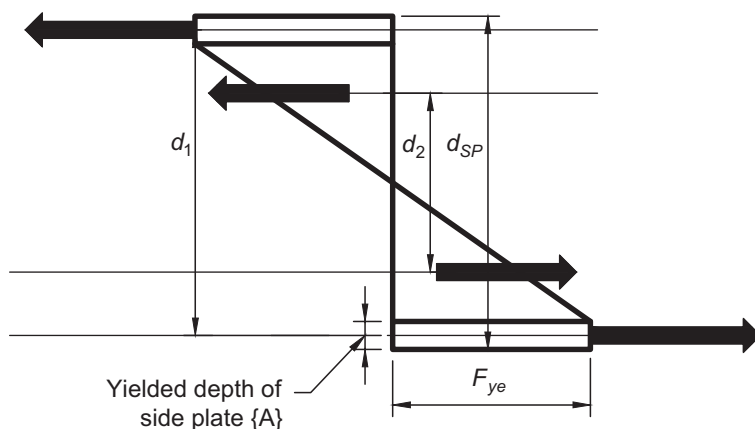


Fig. C-11.5. Idealized plastic stress distribution for computation of the effective plastic modulus, Z_{eff} , of the side plate.

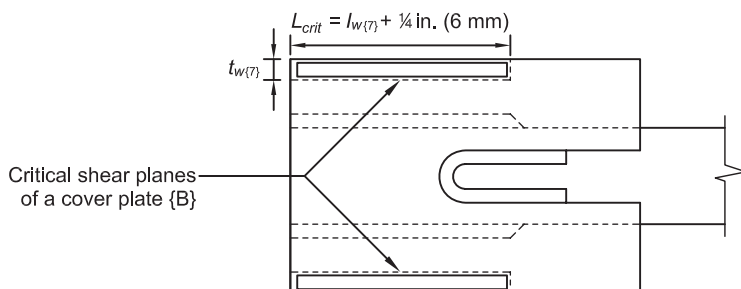


Fig. C-11.6. Critical shear plane of cover plate {B}.

where

V'_u = calculated vertical shear demand resisted by VSE, kips (N)

d_{pl} = depth of vertical shear element, in. (mm)

Horizontal Shear Plate (HSP) {D}. The thickness of the HSP {D} (see Figures 11.10 through 11.15) is determined as the thickness required to transfer the horizontal shear demand from the top (or bottom) of the side plate into the column web. The shear demand on the HSP is calculated as the design load developed through the fillet weld connecting the top (or bottom) edge of the side plate to the HSP (weld {1}). The demand force is determined using an ultimate strength analysis of the weld group at the column (weld {1} and weld {2}) as described in the following section.

$$t_{hsp} = \frac{V''_u}{(0.6)F_y l_{pl}} \quad (C-11.7-5)$$

where

V''_u = calculated horizontal shear demand delivered by weld {1} to the HSP, kips (N)

l_{pl} = effective length of horizontal shear plate, in. (mm)

Welds. Welds are categorized into three weld groups and sized using an ultimate strength analysis.

The weld groups are categorized as follows (see Figures 11.10 through 11.15): fillet welds from the beam flange to the cover plate (weld {5} and weld {5a}) and the fillet weld from the beam web to the VSE (weld {4}) constitute weld group 1. Fillet welds

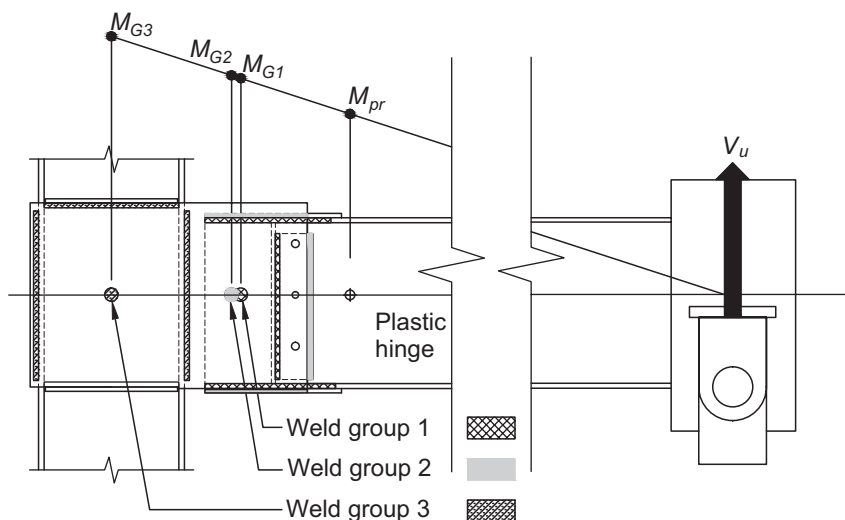


Fig. C-11.7. Location of design weld groups and associated moment demand ($M_{G\#}$).

from the cover plate to the side plate (weld {7}) and fillet welds from the VSE to the side plate (weld {6}) constitute weld group 2. Fillet welds from the side plate to the HSP (weld {1}), fillet welds from the side plate to the column flange tips (weld {2}) and fillet welds from the HSP to the column web (weld {3}) make up weld group 3. Refer to Figure C-11.7.

The ultimate strength design approach for the welds incorporates an instantaneous center of rotation method as shown in Figure C-11.8 and described in the AISC *Steel Construction Manual* Part 8.

At each calculation iteration, the nominal shear strength, R_n , of each weld group, for a determined eccentricity, e , is compared to the demand from the amplified moment to the instantaneous center of the group, $V_{pr}e$. The process is continued until equilibrium is achieved. Because the process is iterative, SidePlate Systems engineers use a design spreadsheet to compute the weld sizes required to achieve the moment and shear capacity needed for each weld group to resist the amplified moment and vertical shear demand, M_{group} and V_u , respectively.

Step 8 requires that the engineer of record review calculations and drawings supplied by SidePlate engineers to ensure that all project-specific moment connection designs have been appropriately completed and that all applicable project-specific design loads, building code requirements, building geometry, and beam-to-column combinations have been satisfactorily addressed.

The Connection Prequalification Review Panel (CPRP) has prequalified the SidePlate moment connection after reviewing the proprietary connection design procedure contained in the SidePlate FRAME Connection Design Software (version 5.2, revised January 2013), as summarized here. In the event that SidePlate FRAME connection designs use a later software version to accommodate minor format changes in

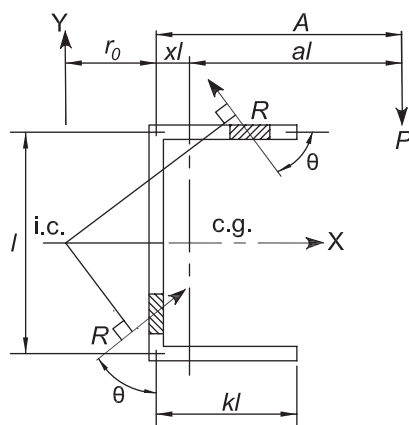


Fig. C-11.8. Instantaneous center of rotation of a sample weld group.

the software's user input summary and output summary, the SidePlate connection designs will be accompanied by a SidePlate validation report that demonstrates that the design dimensions, lengths and sizes of all plates and welds generated using the CPRP-reviewed connection design procedure remain unchanged from that obtained using the later version connection design software. Representative beam sizes to be included in the validation report are W36×150 (W920×223) and W40×294 (W1000×438).

CHAPTER 12

SIMPSON STRONG-TIE STRONG FRAME MOMENT CONNECTION

12.1. GENERAL

The Simpson Strong-Tie® Strong Frame® moment connection uses patented Yield-Link™ structural fuse technology to create a field bolted, partially restrained (PR) moment connection for strong-axis wide-flange beam-to-column connections. The Yield-Link elements are bolt-on, bolt-off easily replaceable elements that absorb inelastic demands, rather than forming plastic hinges in the members, and allow for rapid structural resiliency to be achieved if necessary. The connection eliminates field welding, and the frame behavior afforded by the connection enables frames to be designed without the need for flange bracing on the beams. This is particularly useful in structures where providing flange bracing can be difficult (such as when integrated into wood structures) or is an undesirable architectural intrusion. Connection testing qualified the use of snug-tight bolts for typical field-installed bolts, simplifying bolt installation, inspection and frame erection.

The connection centers around the Yield-Link (Link) structural fuse performance and a capacity-based design procedure that, under lateral loading, pushes inelastic demand into the Links rather than the members. Unlike other prequalified special moment frame (SMF) connections, little if any inelastic behavior is expected in the members. The Link is a modified T-stub and serves to transfer moment from the beam to the column. The flange of the Link bolts to the column flange with four snug-tight ASTM F3125 Grade A325 or A325M bolts (pretensioned ASTM F3125 Grade A325, A325M or F1852 bolt assemblies are also permitted). The stem of the Link bolts to the beam flange with pretensioned ASTM F3125 Grade A325, A325M, A490, A490M, F1852 or F2280 bolt assemblies. In between the connection to the beam and column, the stem of the Link is elongated and contains a section with reduced area that defines the location of yielding in the Link. This reduced area controls the axial strength of the Link and provides for very reliable estimates of the yield and ultimate moment strength of the beam to column connection. To prevent buckling of the yielding section of the Link when in compression, a buckling restraint plate (BRP) is placed over the Link and bolted to the beam flange on either side of the reduced-area section of the Link. The BRP uses snug tight ASTM F3125 Grade A325 or A325M bolts that pass through a spacer plate that fills the gap between the bottom of the BRP and the near surface of the beam flange. The web of the beam connects to the column via a single-plate shear connection. The connection uses an arrangement of bolts that permit transfer of shear and axial forces between the beam and column, while at the same time limiting the transfer of moment. This is accomplished by having a central pivot point defined by a center bolt passing through standard holes in both the beam

web and the shear plate and by having the upper and lower bolts in the shear plate pass through horizontal slots in the shear plate and standard holes in the beam web. This arrangement creates a hinge in the beam web to column flange connection and defines the effective rotation point for the plastic hinge. Shear-plate bolts are permitted to be snug-tight ASTM F3125 Grade A325 or A325M or pretensioned ASTM F3125 Grade A325, A325M or F1852. The hinge is used to transfer net axial force from the beam to column, so in addition to shear- and moment-related design provisions found in other prequalified moment connections, this connection also contains design provisions for axial load transfer.

Initial qualification testing consisted of a series of nine reversed cyclic tests according to Section E3.6c of ANSI/AISC 341-10 (AISC, 2010a) covering three configurations, each with three replications. Each test consisted of a single-story, single-bay frame with lateral loads (in-plane shear) introduced into the top flange of the beam through a wood nailer connected to the beam flange. Only one end of the beam used the Strong Frame connection, and the remaining beam-to-column and column-to-test bed connections were pinned. This configuration was chosen for testing the connection over the typical cantilever beam configuration for two primary reasons: It allowed beam axial loads to be driven through the joint to enable verification of both the axial and moment related design provisions, and it permitted observation of the beam flange response when flange bracing was omitted. The testing resulted in all frames reaching a drift level of 0.05 rad without a loss of strength greater than 20% of the nominal plastic moment strength, M_p , satisfying the requirements of Section E3.6b of ANSI/AISC 341-10. For this connection M_p is calculated using the yielding area of the links and the connection geometry rather than the beam properties.

At the current time there are no other PR connections that have been prequalified as an SMF connection, a situation not directly addressed in ANSI/AISC 341-10. Accordingly, even though the initial testing met the SMF connection performance requirements of ANSI/AISC 341-10, additional requirements were created to demonstrate the suitability of the connection and the design procedure for use as SMF or IMF connections in high-seismic applications.

The first supplemental requirement was to assess the connection performance through a component equivalency evaluation using the procedures found in FEMA P-795, *Quantification of Building Seismic Performance Factors: Component Equivalency Methodology* (FEMA, 2011). An independent study was commissioned to perform the FEMA P-795 evaluation, comparing the Strong Frame connection to the reduced beam section (RBS) connection, resulting in two changes to the design procedure. The Link flange-to-stem weld was required to develop the full strength of the unreduced portion of the stem at the column side—it had been previously designed for the probable maximum tensile strength of the reduced yielding area—and a single thickness of stem material, ½ in. (13 mm), was selected—initially different thicknesses were considered. Six additional tests similar to those described previously (three reversed cyclic tests according to ANSI/AISC 341-10 and three monotonic tests) were then conducted to verify the performance with the amended design and detailing procedure.

Although not required by ANSI/AISC 341-10, the monotonic tests were conducted to satisfy FEMA P-795 requirements. The purpose of the monotonic testing is to better understand the collapse behavior of buildings using the connection by investigating the interstory drift capacity afforded by the connection. The results of the cyclic tests again showed that the connection meets the performance requirements of ANSI/AISC 341-10 and that the ultimate limit state was as predicted: a net section fracture in the reduced portion of the Links. The results of the monotonic tests showed that the connection has tremendous displacement capacity, the tests being stopped at 9.5% interstory drift without failure or decreasing from peak capacity. The conclusion from the FEMA P-795 study is that the Strong Frame connection is equivalent to the prequalified RBS connection. It should be noted that for all the tests—initial and secondary, cyclic and monotonic—yielding initiated from about 1 to 1.5% interstory drift as is typically expected of frames with SMF connections.

Even with the successful FEMA P-795 evaluation, a second supplemental requirement was added to look more at system behavior rather than the individual connection behavior as was the focus of the FEMA P-795 evaluation. To address this additional requirement, a series of nonlinear response history analyses were performed using a suite of ground motions and a suite of archetype buildings to compare the seismic response of buildings using the Strong Frame connection to buildings using a prequalified connection. The connection chosen for comparison was again the RBS connection. As before, an independent study was commissioned, with designs for both systems minimized to the extent allowed by the respective design procedures. The study included the development of archetype designs for representative steel moment frames for a two-story, four-story, and six-story building using ASCE/SEI 7-10, ANSI/AISC 360-10 (AISC, 2010b) and ANSI/AISC 341-10 and was evaluated using seven scaled ground motion pairs. The study demonstrated that the performances of the Strong Frame and comparable RBS structures were very similar and within acceptable levels. No collapses were predicted by the analysis. The most severe response was recorded for the two-story RBS archetype, which exhibited a maximum story drift ratio for one record of nearly 5%. Mean story drift response for both the Strong Frame and RBS structures averaged approximately 2.3%, and the mean plus one standard deviation response averaged 3% for the Strong Frame structures and 2.8% for the RBS structures.

In addition to the cyclic and monotonic testing specifically used to qualify the Strong Frame connection, other large-scale shake table test programs have employed the connection. Steel frames using the Strong Frame connection were part of the 2009 NEESWood Capstone tests at Japan's E-Defense facility in Miki, Japan (van de Lindt et al., 2009). The full-scale seven-story structure consisted of first-story steel framing using the Strong Frame connection, which supported a six-story wood light-frame structure on top and had a plan dimension of 40 ft by 60 ft (12 m by 18 m). More recently, steel frames using the Strong Frame connection were employed as retrofit elements in the first story of a four-story full-scale light frame wood building built to simulate a typical San Francisco-style wood structure with a soft/weak first story due to ground-level parking. Known as the NEESSoft project (Pryor et al., 2014), the

building was successfully tested at the NEES@UCSD outdoor shake table under a variety of different ground motions.

12.3. PREQUALIFICATION LIMITS

1. Beam Limitations

A number of different beam sizes were used in the frame tests, with the largest being W16 (W410) profiles. Because the capacity-based design procedure forces inelastic behavior into the connection rather than the beam, in general, the width-to-thickness requirements of the AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360-16) (AISC, 2016b) apply. However, because the connection does rely on the beam flange and web to form part of the buckling restraint assembly for the yielding portion of the Link, the beam flange is required to have a minimum thickness of 0.40 in. (10 mm) and the width-to-thickness value cannot exceed λ_r in AISC *Specification* Table B4.1a. Additionally, the capacity-based design procedure and connection performance (no plastic hinging in the beam) allows the beam stability bracing to be designed in accordance with the AISC *Specification*. The protected zone encompasses the shear connection and yielding portions of the connection, specifically the Yield-Links, and elements of the connection in contact with both.

2. Column Limitations

A number of different column sizes were used in the frame tests, with the largest being W18 (W460) profiles. Because only strong-axis connections were tested, beams are required to connect to column flanges. Where frames are detailed so as to create plastic hinging at the column base, the width-to-thickness requirements for highly ductile members apply in the first story. Otherwise, the requirements of the AISC *Specification* apply. Column lateral bracing requirements in the AISC *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341-16) (AISC, 2016a) are to be satisfied. An exception is provided to allow bracing the column at the level of the top flange of the beam only if additional limits are placed on the column flexural design strength provisions of the AISC *Specification* to ensure the columns remain elastic outside the panel zones. The limits are noted in Step 13.2 of the Section 12.9 Design Procedure requirements.

3. Bolting Limitations

The connection testing specifically prequalified a number of bolts in the connection to be installed snug-tight. These include the Link flange-to-column flange bolts and the shear-plate bolts. These same bolts may also be installed pretensioned if desired. The buckling restraint plate bolts are required to be installed only snug-tight. The Link stem-to-beam bolts are required to be installed pretensioned to prevent slip that would occur under design loads. In the prequalification testing, slip would typically start between 2% to 3% interstory drift, at which point the bolts went into bearing. No special preparation of either the Links or the beam flange surfaces in the test frames was done. The only prequalification requirement is that faying surfaces not be painted.

12.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

The requirements for the Strong Frame connection are similar to those of other prequalified SMF connections. M_{pr} , however, is calculated based on the probable maximum tensile strength of the Links, $M_{pr} = P_{r-link} (d + t_{stem})$, where P_{r-link} is the probable maximum tensile strength of the Link calculated as the product of the yield area; specified minimum tensile strength, F_u ; and the ratio of the expected tensile strength to the specified minimum tensile strength, R_t . When Links are fabricated from ASTM A572/A572M Grade 50 (345) plate material, this approach results in a 23% higher estimate of demand than what would be calculated if an approach equivalent to that of other SMF connections was used (Equation 2.4-1). Basing connection demand on the section properties and the expected tensile strength is used in numerous places in the design procedure and produces similarly higher demands when compared to typical SMF requirements. This is consistent with the overall goal of keeping nearly all inelastic demand in the replaceable Yield-Link elements and creating little if any inelastic demand in the members. Using this higher demand also applies to the evaluation of panel zone strength, which for the Strong Frame connection is done in accordance with the AISC *Specification* rather than the AISC *Seismic Provisions*. One effect of this requirement is the use of the AISC *Specification* $\phi = 0.90$ rather than the AISC *Seismic Provision* $\phi_v = 1.00$ (AISC *Seismic Provisions* Section E3.6e), in conjunction with nominal resistance, R_n , calculated in accordance with AISC *Specification* Section J10.6. Added to the differences in how M_{pr} is calculated results in panel zone shear demands approximately 26% higher than would be calculated if typical SMF design methodologies were used.

12.5. CONTINUITY PLATES

The need for continuity plates is determined in the design procedure by basing demand on the expected tensile strength of the Links as discussed in Commentary Section 12.4 and design strength as determined by the AISC *Specification*. As was used successfully in the qualification testing, fillet welds are permitted at the web and flanges of the column.

12.6. YIELD-LINK FLANGE-TO-STEM WELD LIMITATIONS

As discussed previously, initially the design demand for this weld was based on the expected tensile strength of the reduced portion of the Link. While this did permit the qualification testing to successfully meet the performance requirements of the Seismic Provisions, the ultimate limit state for some of the tests was failure of this weld rather than the more desirable failure in the yielding area of the Link. As a result of the additional requirement to pass the FEMA P-795 component equivalency evaluation, which compared the Strong Frame connection performance to that of an RBS connection, this weld was changed to require either complete-joint-penetration groove welds or double-sided fillet welds that develop the tensile strength of the unreduced portion of the Link.

12.7. FABRICATION OF YIELD-LINK CUTS

The fabrication requirements reflect production quality necessary to ensure the proper performance of the links.

12.8. CONNECTION DETAILING

The requirements of this section reflect the tested conditions and common allowances where appropriate. The connection is detailed to accommodate up to 0.07-rad rotation, which, along with frame flexibility, will accommodate the expected interstory drift without affecting any connection element other than the Yield-Links. Shear plate connection welds are required to develop the strength of the shear plate, and Yield-Link thickness is limited to material nominally $\frac{1}{2}$ in. (13 mm) thick and fabricated from one of the three permitted steel grades.

The stems of the pair of Yield-Links at each connection must be fabricated from the same heat of material to ensure minimum variability in actual F_y and F_u for the pair of Links in a connection. This is because imbalance of the Link strengths can drive additional force into the central pivot bolt of the connection. This force is parallel to and can be cumulative with the net axial connection force in the beam, which is also resisted by the central pivot bolt. Rather than include an explicit design procedure to accommodate unbalanced Link strength, it was decided at this time to simply use material from the same heat for the stems of each pair of Links at a given connection.

In general, the topic of the potential adverse effects of unequal strength in the Links, or flanges, of a moment connection is not limited to just the Strong Frame connection. While the central pivot design of the Strong Frame connection in essence attempts to maintain the location of the plastic neutral axis at the centerline of the beam even if the Links are of different strengths—and thus create relatively even strain demands in each link for a given connection rotation—the same is not true for traditional welded up shapes that may have different flange strengths and form plastic hinges in the beam cross section. The neutral axis of the plastic section would shift toward the flange with higher strength, and uneven strain demands in the flanges would result. However, the effect on inelastic performance for this condition has not been studied, and currently there are no requirements to control flange strength in SMF connections using built-up sections subject to plastic hinging.

12.9. DESIGN PROCEDURE

The design procedure for the Strong Frame connection parallels the design concepts for frames with other moment connections but is adapted to the specific configuration of the connection. Connection moment strength is controlled by the strength of the Yield-Links and shear strength by the strength of the shear-plate connection. This allows beams to be designed, if desired, to be unbraced yet stable under the combined effects of expected ultimate connection moment strength, gravity loads, and axial load resulting from lateral loading. Unlike some historical PR moment connections, the Strong Frame connection is proportioned to remain elastic under the combined effects of design lateral and vertical loads, with the Yield-Links only experiencing

inelastic behavior during seismic events in which the real seismic forces are expected to exceed the unamplified design seismic forces (Rex and Goverdhan, 2000). This permits the use of typical elastic analysis procedures similar to other SMF connections. However, like some historical PR moment connections, the beams are designed as simple span for gravity loads (Geschwindner and Disque, 2005). This facilitates post-earthquake repairs, should they be needed, by ensuring the beam is proportioned to support its design gravity loads even if the Links are removed during replacement. In addition to the various strength checks for frame members and elements of the connection, the PR nature of the Strong Frame connection requires a detailed stiffness check using actual connection stiffness to ensure lateral drift limits are met. This means that the lateral stiffness-to-mass and lateral yield strength-to-mass ratios are required to be the same as any other frame using SMF connections. As such, the code equations for base shear and period estimation are equally applicable to frames using the Strong Frame connection as they are to frames using other SMF connections. This was verified as part of the nonlinear response history study comparing Strong Frame and RBS connections discussed previously. For each of the archetype structures, the periods of the RBS frames and Strong Frame frames were virtually identical.

The design process can be iterative, and **Step 1** begins with suggestions on how to create trial values for sizes of the frame members and provides an initial estimate of story drift which is explicitly checked later in the design procedure. In addition to designing the beam as simply supported, **Step 2** also suggests a deflection limit on the beam to limit member end rotations that would affect the connection.

Step 5 determines the width of the yielding portion of the Link based on the permitted thickness of $\frac{1}{2}$ in. (13 mm) and subject to limitations that include a maximum width of $3\frac{1}{2}$ in. (88 mm), which corresponds to the strongest Yield-Link that has been qualified. Testing showed that for the approved steel grades, if the length of the straight portion of the yielding section of the Link is proportioned such that the strain demand in that section does not exceed 0.085 when the connection is subjected to a rotation of 0.05 rad, the Link will possess sufficient toughness to enable the connection to meet the cyclic test performance requirements of the AISC *Seismic Provisions*, and this is reflected in **Step 6**.

In **Step 7**, the Link expected yield strength and probable maximum tensile strength are computed. The value of R_t is specified as 1.2 to reflect the proper value from AISC *Seismic Provisions* Table A3.1 for ASTM A572/A572M Grade 50 (345) plates, strips and sheets. If the Link is fabricated from hot-rolled structural shapes of ASTM A992/A992M or A913/A913M Grade 50 (345) as permitted, the tabulated value of $R_t = 1.1$ is used.

In **Step 8**, the Link-to-beam flange connection is designed. Both here and in the web shear-plate connection, bolt bearing is required to be designed using bearing values that limit deformation at the bolt hole. The purpose of this is to again drive the inelastic response into the reduced portion of the Link and to keep other areas of the connection outside of the link essentially damage free to facilitate Link replacement should it be desired after a seismic event.

In **Step 9.2**, the required Yield-Link flange thickness, for a no prying action condition with a force limited by the probable maximum tensile strength of the Link as reflected in the calculation of r_l in **Step 9.1**, is determined.

Step 11 is a procedure for calculating the actual connection stiffness for use in checking frame drift and connection behavior. The Link stiffness is calculated as three springs in series, where the springs represent the stiffnesses of the Link flange in bending, the yielding portion of the link stem under axial load and the nonyielding portion of the Link stem under axial load. Once the axial stiffness of the Links is computed, the connection can either be modeled with appropriate geometry using discrete axial elements to represent the top and bottom links at a connection, or an equivalent rotational spring can be determined and used in the modeling. As seen in Figure C-12.1, this approach has been shown to be very effective for modeling both the elastic and inelastic behavior of the connection (Pryor and Murray, 2013).

Step 11.2 requires that the frame, using the actual Strong Frame connection properties, meets the required drift limit and that the connection response is elastic under design load combinations (not including amplified seismic load combinations). The calculation of required shear in **Step 12** is analogous to that used in designing RBS connections. Because a plastic hinge is not formed in the beam in Strong Frame connections, the value of L_h is the distances between the rotational points in the shear plate connections rather than between the centers of plastic hinges. The user is directed to the Commentary for Chapter 5, *Reduced Beam Section (RBS) Moment Connection*, for additional information.

Required member checks are in **Step 13**. **Step 13.1** requires the beams to be checked using the AISC *Specification* under combined demand that consists of the maximum probable end moments, axial forces considering either the maximum that the system can deliver, or from amplified seismic loads and gravity loads. If the designer chooses,

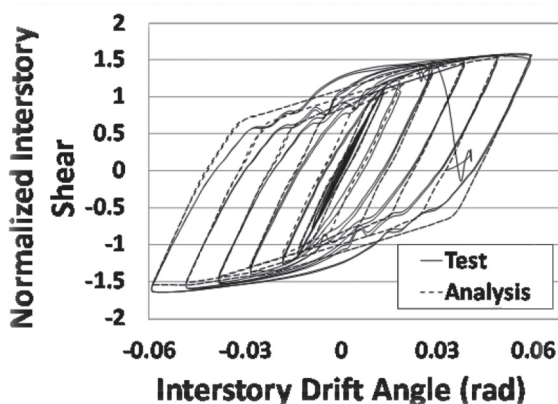


Fig. C-12.1 Testing vs. FEA analysis for frame modeled with all material nonlinearity in the Yield-Link elements.

beam size can be selected to meet the requirements of the AISC *Specification* under this combined loading without lateral bracing. In **Step 13.2**, column design demand is determined from load combinations that include seismic effects derived from either the maximum that the system can deliver or the amplified seismic loads. The design strength of the column outside the panel zone is not permitted to exceed $\phi_b F_y S_x$, where $\phi_b = 0.90$ even if otherwise permitted by AISC *Specification* Section F2 when column bracing is only provided at the level of the top flange of the beam.

In **Step 15**, the shear plate and beam web are designed in accordance with the AISC *Specification* to permit hinging about a central rotation point while resisting the beam shear and axial force demand determined from capacity-based design principles. In **Step 15.1**, note that the bolt shear demand is controlled by the shear force on the center bolt in the connection because it takes its full portion of the vertical shear reaction in combination with any axial loads being transferred from the beam to the column, combined using the square root of the sum of the squares (vector sum) rule.

Analogous to a beam flange force, in **Step 16**, the maximum probable axial strength of the Yield-Link is used to calculate panel zone shear demand. As is the case for typical connections, Link strengths are summed for double-sided connections.

Borrowing from the Bolted Unstiffened and Stiffened Extended End-Plate Moment Connections provisions in Chapter 6, **Step 18** provides an analogous design procedure for checking the column flanges for flexural yielding based on the maximum probable tensile strength of the Yield-Link.

If the design strength of the column web or flange without continuity plates or stiffeners is insufficient to support the maximum probable tensile strength of the Yield-Links, the design requirements for the stiffeners or continuity plates are in **Step 19**. Fillet welds are permitted at both column web and flange connections to the continuity plates or stiffeners.

CHAPTER 13

DOUBLE-TEE MOMENT CONNECTION

13.1. GENERAL

The double-tee provisions were written primarily based on testing performed at the Georgia Institute of Technology. The series of six connection tests used a W14×145 (W360×216) column and either a W21×44 (W530×66) or W24×55 (W610×82) beam (Swanson, 1999). The tests are summarized in Table C-13.1. This series of six full-scale assembly tests was supplemented by a series of 48 tests on T-stub components conducted at the Georgia Institute of Technology (Swanson, 1999; Swanson and Leon, 2000). None of the tested configurations included a concrete slab, and all of the tested configurations included a single-plate shear connection between the web of the beam and the column flange.

Research work conducted at the University of Wyoming (McManus and Pucket, 2010) was also considered in development of these double-tee provisions. Testing associated with this program included 22 T-stub component tests with very wide gages between the two rows of bolts connecting the T-flange to the column flange and T-stubs with very thick shims between the T-flange and column flange. These shims were arranged to provide a gap between the T-flange and column flange such that inelastic bending of the of the T-flange could occur not only when the T-stub was subjected to tension but also when it was subjected to compression.

Research work conducted at the University of Cincinnati on T-stubs built-up from thicker plates was also considered in the development of these provisions (Hantouche et al., 2013; Hantouche et al., 2015). The intent of this work was to allow for the use of built-up T-stub components in the double-tee connections. However, since the testing that was performed at the University of Cincinnati did not include full-scale assemblies, the CPRP elected to not permit built-up T-stubs in this edition of the provisions.

A series of five full-scale beam-column subassemblies was tested at the University of Texas at Austin in 1996 (Ulloa Barbaran, 1996; Larson, 1996). These assemblies consisted of cantilever W36×150 (W920×223) beams connected to pinned-pinned W14×426 (W360×634) columns using a shear tab to resist shear and, in most cases, fully-bolted T-stubs to resist moment. The first specimen was designed with a shear-only connection—with a shear tab but without T-stubs—so as to investigate the contribution of the shear tab and beam web to the moment strength of the connection. The second and third specimens were designed both with a shear tab and T-stubs, but the T-stubs were configured to provide only a partial strength beam connection. The fourth and fifth specimens were designed with both a shear tab and T-stubs with the tees in the fourth specimen proportioned to transmit 100% of the beam moment to the column and the tees in the fifth specimen proportioned to transmit approximately 125% of the beam's plastic moment (Ulloa Barbaran, 1996). The connections were

TABLE C-13.1.**Summary of Prequalification Tests on Double-Tee Connections**

Test	Beam	Column	T-stub	Bolts
FS-03	W21×44	W14×145	W16×45	7/8-in. A490
FS-04	W21×44	W14×145	W16×45	1-in. A490
FS-05	W24×55	W14×145*	W16×100	7/8-in. A490
FS-06	W24×55	W14×145*	W16×100	1-in. A490
FS-07	W24×55	W14×145*	W21×93	7/8-in. A490
FS-08	W24×55	W14×145*	W21×93	1-in. A490

* Column was stiffened with continuity and doubler plates.

loaded by applying a displacement to the end of the cantilever beam. The fourth and fifth specimens are most germane to the topic of prequalification of double-tee connections. The fourth specimen failed when the bolts connecting the T-stubs to the column flange fractured. After disassembling the connection, small fractures near the bolt holes in the flange of the beam were noted. Testing of the fifth specimen was stopped when a fracture of the beam flange was noticed. None of the specimens performed adequately to be considered for use in SMF or IMF systems.

Research on bolted flange plate (BFP) connections conducted at the University of Illinois (Schneider and Teeraparbong, 2002) and at the University of California at San Diego (Sato et. al., 2007) was also considered because the connection of the flange plate to the beam flange in a BFP connection is nearly identical to the connection of the T-stem to the beam flange in a double-tee connection.

13.2. SYSTEMS

None of the tested configurations included a concrete slab; thus, the slab must be isolated from the column to avoid developing unintended and unproven mechanism in the connection as built during a seismic event. Further, no shear studs were present in the tested configuration; therefore, as a result, shear studs are precluded in the as-built connections in a conservative but consistent decision aimed at preventing potential fractures from initiating at the stud welds in regions expected to undergo significant inelastic strains. Given that the distance from the face of the column to one beam depth beyond the shear bolts farthest from the column may be a large distance, the requirement that studs be omitted in this region may create difficulties for composite beam systems. However, the requirement is felt necessary to preserve the anticipated performance of the connection. It is speculated that the presence of the bolts and nuts connecting the beam flange to the T-stem (the shear bolts) will transfer some shear between the steel and concrete without compromising the seismic performance of the connection.

Double-tee connections tested at Georgia Tech ranged in stiffness from connections that were clearly fully restrained (FR) to nearly partially restrained (PR). The AISC

Seismic Provisions explicitly permit PR connections in OMFs but are silent on the use of PR connections in IMF and SMF. This Standard precludes the use of PR double-tee connections until the issue can be considered in more detail. Step 13 of the double-tee design procedure includes a check of the connection stiffness to ensure that it satisfies the FR criteria.

13.3. PREQUALIFICATION LIMITS

Prequalification limits for the beam are based on the configurations successfully tested at Georgia Tech. Limitations for the column are based on limitation for other prequalified connection types.

13.5. CONNECTION DETAILING

1. T-Stub Material Specifications

T-stubs tested at Georgia Tech were cut from W-shapes rolled from either ASTM A572 Grade 50 or dual grade ASTM A36/A572 Grade 50 steel. In all cases, the yield and tensile strengths of the steel used for the T-stubs were consistent with ASTM A572 Grade 50 (345) or ASTM A992/A992M material. Details are available in Swanson (1999).

2. Continuity Plates

All of the experiments upon which these provisions are based were conducted with continuity plates present. As a result, continuity plates are required in the prequalified double-tee connections. The continuity plates are required to control column-flange bending that may result in column-flange prying forces in the tension bolts between the T-flange and column flange. While T-flange prying is considered explicitly in the design procedure, column-flange prying is not. Further, when two rows of four tension bolts are used, it is speculated that the absence of continuity plates may result in column-flange bending that could produce larger forces in the interior tension bolts than the exterior tension bolts. More research is needed before the requirement of including continuity plates can be relaxed.

4. Bolts

All of the tests used as a basis for prequalification employed two identical T-stubs and bolt patterns that were arranged symmetrically with respect to the axes of the beam and column (with the exception of the web bolts, which are slightly off center with respect to the plane of the beam web and column web). Further, all prequalification tests used two rows of bolts between the T-stem and beam flange and between the T-flange and column flange. Significant questions remain regarding whether additional rows of bolts between the T-flange and column flange would participate in carrying load without the presence of stiffeners on the T-stub.

All of the six full-scale assemblies tested at Georgia Tech made use of ASTM F3125 Grade A490 bolts, but a number of the T-stub component tests included ASTM F3125 Grade A325 fasteners. As a result, Grade A325, A325M, A490 or A490M bolts,

or their tension control equivalents, are permitted in the double-tee connections. While all of the tested configurations used the same diameter and grade of fasteners throughout the connections, there is no requirement that all bolts in the prequalified connection be the same diameter or grade. It is expected that all bolts will likely be Grade A490 or A490M, but there may be advantages to using larger-diameter tension bolts between the T-flange and column flange and smaller-diameter shear bolts between the T-stem and beam flange.

Oversize holes are not permitted in the beam flange, because if they are used, it is difficult to achieve adequate moment capacity in the beam at the row of shear bolts farthest from the column face.

Of the 22 experiments conducted at the University of Wyoming, 18 components failed with a fracture of the flange of the T-stub. Of the 60 T-stub components tested at Georgia Tech (including the 12 T-stubs in the full-scale assembly tests), no fractures in the flanges were observed. An analysis of the 82 experiments revealed that the T-stubs that failed via flange fracture all had a ratio of tension bolt gage to T-flange thickness (g_{tb}/t_{ft}) that was greater than 10. No flange fractures were observed in T-stub components with a ratio of g_{tb}/t_{ft} of less than 9. After considering the data, the CPRP elected to limit the ratio of g_{tb}/t_{ft} to be not greater than 7.0, a value that includes all of the components tested at Georgia Tech.

All bolts are required to be installed as pretensioned fasteners and faying surfaces are required to be prepared as they would be in slip-critical connections. The connections are not intended to be designed as slip-critical, however. The required pretension and faying surface preparations are intended to provide a reasonable level of friction to prevent slip at service loads and provide slip resistance conducive to energy dissipation at design loads. Prevention of slip at design loads is not considered desirable as this would limit a robust energy dissipation mechanism of the connection.

5. T-Stub Shims

The thickness of shims between the T-flange and column flange is limited to a maximum of $\frac{1}{4}$ in. (6 mm) to preclude the use of shims that would be similar in configuration to the shimmed configuration in the University of Wyoming experiments where the T-flange was permitted to flex both when the T-stub was in tension and when it was in compression. Further, the use of shims that extend the full width and breadth of the T-flange is encouraged.

13.6. DESIGN PROCEDURE

Step 1. The determination of the probable maximum moment is consistent with other prequalified moment connections, such as the bolted flange-plate connection, based on C_{pr} , which is described in Section 2.4.3.

Step 2. The maximum shear bolt diameter is determined that will allow the full plastic moment of the beam to be developed at the shear bolts farthest from the column face, while precluding a net-section rupture of the beam flange in tension. The underlying model in the double-tee connection differs from the bolted flange

plate connection and Kaiser bolted bracket connection. The underlying model for the bolted flange plate and Kaiser bolted bracket connections is the same that appears in Section F13 of the AISC *Specification for Structural Steel Buildings* (AISC, 2016b), which compares the gross yielding strength of the flange to the net section rupture strength of the flange. In Section F13 of the AISC *Specification*, if it is found that net section strength governs, then a method is provided to reduce available flexural strength of the beam. In the bolted flange plate and Kaiser bolted bracket connections, a moment strength less than the full plastic moment is not acceptable, so the analogous checks in those design procedures are reformulated so as to determine the maximum bolt diameter while ensuring that a strength reduction is not required. The check in **Step 2** of the double-tee design procedure, gross yielding is compared to net-section fracture, but the comparison is made on the plastic flexural section instead of on the axial section of the tension flange. Again, a flexural strength less than the full plastic moment is not acceptable, so no provision was envisioned for computing a reduced strength when the net fracture strength governs.

The checks described previously are predicated on the assumption that there are two bolts in standard holes in the tension flange of the beam. The difference in basis between bolted flange plate and double-tee design procedures is based on experimental observations at Georgia Tech that beams that did not satisfy the design check of Section F13 in the AISC *Specification*—and, thus, had bolts larger in diameter than would be permitted in the bolted flange plate design procedure—were still able to develop their full plastic moment at the critical net section computed using measured yield stresses. Based on that observation, the bolted flange plate version of the beam net-section check was deemed to be too conservative, thus, the double-tee version of the check was introduced. A thorough discussion of the issue is provided by Swanson (2016).

In the event that the check in Equation 13.6-2 is not satisfied (e.g., larger bolts diameter are required), reinforcement of the beam could be provided. It should be noted, however, that none of the experiments upon which these provisions are based included any reinforcement of the beam flanges at the critical net section. Thus, reinforcing the beam flange is not within the scope of this prequalified connection design procedure.

The check can be made on a section assuming that there are holes in both the tension flange and compression flange of the beam, which is simpler as is shown in Equation 13.6-3, or it can be assumed that only the holes in tension are critical, which is more complicated but does allow for slightly larger diameter bolts.

Step 3. Bolt shear will govern in most cases, using typical bolt spacing and end distances. The bearing and tearout strength of the T-stem and beam flange will be checked in **Step 18** after additional dimensions of the connection are determined. A simple calculation can be performed at this stage to determine bolt spacing and end distance that will guarantee that shear of the bolt will govern, which will maximize the strength of the shear bolts.

Step 4. Equation 13.6-5 is used to estimate the number of shear bolts that will be required to connect the T-stem to the beam flange. The quantity $1.25M_{pr}$ in the

numerator is in place as a preliminary estimate of the moment at the face of column, M_f , which cannot be determined until the number of shear bolts is known.

Step 5. The location of the plastic hinge relative to the column face is determined in this design step. This is based on spacing and end distance of the shear bolts and the setback distance of the end of the beam relative to the column face, which will be much larger than the $\frac{1}{2}$ in. (12 mm) that is typically used for bolted flange plate connections and simple shear connections to allow space for the T-flanges between the end of the beam and column face.

Step 6. Calculation of the shear force at the plastic hinge is consistent with the AISC *Seismic Provisions*. The shear force required to develop a full plastic mechanism in the beam is based on E_{cl} as shown in Equation E1-1 of the AISC *Seismic Provisions* and combined with gravity loads, as is indicated.

Step 7. The moment at the face of the column is determined by adding the probable maximum at the plastic hinge, M_{pr} , to the moment created by shear force at the plastic hinge acting over the distance S_h .

Step 8. The maximum probable flange force is determined by dividing the moment at the column face, M_f , by the distance between centerlines of the stems of the top and bottom T-stubs. Because the thickness of the T-stems has not yet been determined, this distance is estimated as $1.05d_b$. The expected flange force will be computed using the actual distance between the T-stem in Step 14 after the actual stem thickness is known.

Step 9. The size of the T-stem is determined based on gross yielding and net fracture in tension, and gross yielding or buckling in compression. The gross section of the T-stem at the first row of shear bolts will be determined by the width of the T-stub and the thickness of the T-stem. The width of the T-stub will be governed by the spacing of the bolts between the T-flange and column flange. As a general rule of thumb, assume that the T-stub will be slightly narrower than the width of the column flange. The lesser of the actual T-stub width and the Whitmore width is used in calculating section properties. If the T-stem is not tapered, the actual width is simply the T-stub width, W_T . If the T-stem is tapered, then the actual width will depend on the geometry of the taper.

The typical gages for beam sections published in the AISC *Steel Construction Manual* can generally be used for the shear bolts. In some cases, it may be advantageous to use a wider gage as long as edge distance requirements are met, which will increase the Whitmore width of the stem and allow more entering and tightening clearance, but this may lower the block shear strength of the beam flange.

The strength of the T-stem will be backchecked in **Step 16** after the section from which the T-stubs are to be cut is selected in **Step 12**.

Step 10. Equation 13.6-16 provides a lower bound estimate of the required diameter of the tension bolts based on the assumption that there is no prying in the T-flange. Larger bolts may be required depending on the level of prying present in the T-flange

and may be desirable to allow a ductile prying mechanism to form in the T-flange (Swanson, 2002).

Step 11. The provisions in this step are based on the prying model developed by Swanson (Swanson, 2002). The strength of the tension bolts and T-flange will be confirmed in **Step 17** after the final T-stub dimensions are determined in **Step 12**. In general, the smallest gage possible for the tension bolts is desirable from a strength point of view; however, larger gages can provide additional ductility in the T-flange at the expense of higher prying forces. The gage of the tension bolts will be governed by the entering and tightening clearances of the tension bolts relative to the beam flange, T-stem and shear bolts. Care must also be taken to ensure that there is sufficient space between the top and bottom T-stubs for the shear tab.

The thickness of the T-flange is also a critical parameter and influences the strength of the T-stub greatly. A solution envelope for a typical T-stub is shown in Figure C-13.1 and is the result of plotting a T-stub's flange capacity as a function of the flange thickness. The bold line OABC defines the capacity of the flange and tension bolts, and the region below this line represents an adequate design. Segment OA defines the flange mechanism strength and is calculated as ϕT_1 using Equation 13.6-46. Segment AB, computed as ϕT_2 using Equation 13.6-47, is referred to as a mixed-mode failure because yielding of the T-flange and bolt failure are both expected. Segment BC represents the conventional strength of the bolts without prying and is computed as ϕT_3 using Equation 13.6-48. Point A in the chart is generally considered to represent a balanced failure because the full strength of the flange is exhausted at the same time

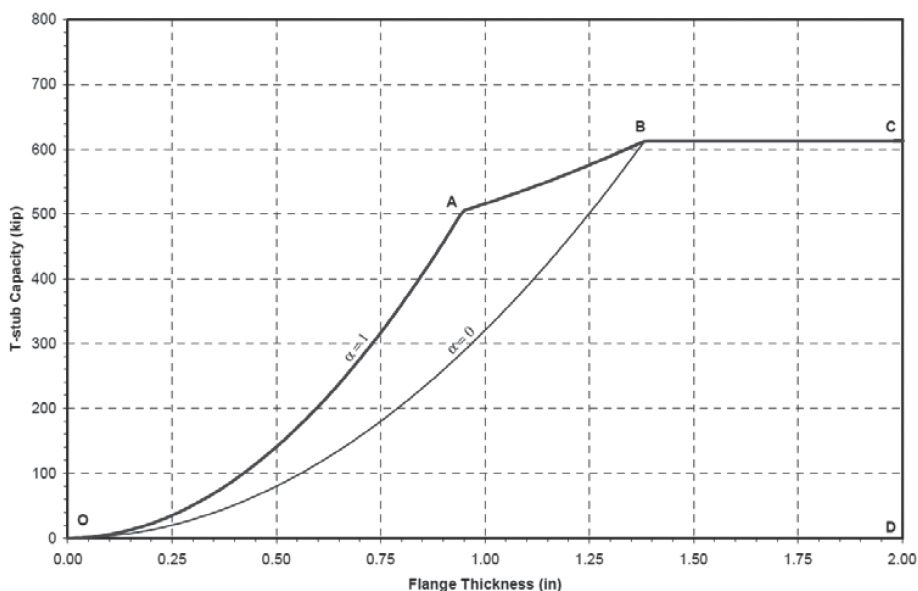


Fig. C-13.1. Relationship of T-stub capacity to flange thickness.

that the bolt forces, including prying, become critical. The flange thickness associated with the point B is often referred to as the critical thickness, $t_{f,crit}$, because a T-stub with a flange thickness greater than that will have negligible prying and will develop the full tensile strength of the bolts.

The initial tension bolt diameter determined in **Step 10** using Equation 13.6-16 is based on setting ϕT_3 equal to F_{pr}/n_{tb} and solving for d_{tb} . The initial flange thickness determined in **Step 11** using Equation 13.6-21 corresponds to Segment AB of Figure C-13.1 and is found by setting ϕT_2 equal to F_{pr}/n_{tb} and solving for t_{ft} . The flange thickness associated with a balanced failure, point A in Figure C-13.1, can be found by setting ϕT_1 equal to ϕT_2 and solving for t_{ft} , as is shown in Equation 13.6-25. The strength of flanges thinner than the thickness given in Equation 13.6-25 will be governed by a plastic flange mechanism. The critical flange thickness associated with point B in Figure C-13.1, can be found by setting the flange strength for a mixed-mode failure, ϕT_2 , equal to the flange strength in the absence of prying, ϕT_3 , and solving for t_{ft} , as shown in Equation 13.6-26.

The dimension a is estimated in **Step 11** as shown in Equation 13.6-18. After the final T-stub dimensions are determined in **Step 12**, the T-flange and tension bolt strength is backchecked in **Step 17**, using a different value of a as calculated in Equation 13.6-52. When determining the gage of the tension bolts, g_{tb} , keep in mind entering and tightening clearances for the tension bolts, particularly in the side of the T-stem adjacent to the beam where the flange of the beam can interfere with tightening of the tension bolts. Using a smaller tension bolt gage reduces prying forces in the bolts, but if the gage is too small, it may be very challenging, if not impossible, to pretension the tension bolts.

Step 12. Select a W-shape that the T-stubs can be cut from. The depth of the W-shape must be sufficiently large to accommodate the distance from the face of the column to the end of the T-stems, which is S_h plus the end distance of the shear bolts in the T-stem. The web thickness and flange thickness of the W-shape must be large enough to accommodate the values of t_{st} and t_{fl} computed in **Steps 9** and **11**, respectively. Finally, the flange width of the W-shape must be large enough to accommodate the gage of the tension bolts and the required edge distance.

Step 13. The initial stiffness of the connection is computed based on the stiffness model proposed by Swanson (1999) and is compared to a minimum stiffness of $18EI/L$ that is required for a FR connection. It is expected that most connections designed on the basis of developing the full plastic moment of the beam will be stout enough to qualify as FR. Still, this check is required for the double-tee connection.

Step 14. After the final dimensions of the T-stubs and connection are determined, the expected flange force, F_f , is computed based on actual dimensions instead of estimated dimensions.

Step 15. After the expected flange force, F_f , has been determined and the actual dimensions of the connection are known, the strength of the shear bolts is checked to ensure that they are adequate.

Step 16. After the expected flange force, F_f , has been determined and the actual dimensions of the connection are known, the strength of the T-stem is checked to ensure that it is adequate.

Step 17. After the expected flange force, F_f , has been determined and the actual dimensions of the connection are known, the strength of the T-flange and tension bolts is checked to ensure that they are adequate. Shear and tension interaction in the tension bolts need not be considered. Because the shear plate will undoubtedly be much stiffer with respect to vertical forces than the T-stem, it is safely assumed that the web bolts carry all of the beam's shear force to the column while the tension bolts carry only tension resulting from the beam's moment. This can be demonstrated by considering the ratio of the stiffness of the shear plate to the stiffness of the T-stems on the top and bottom flanges of the beam.

$$\frac{K_{\text{shear plate}}}{K_{T\text{-stems}}} = \frac{12L_{sp}^3 t_{sp}}{(2)(12)W_T t_{st}^3}$$

If it is assumed that $L_{sp} \approx W_T$, $t_{st} \approx 2t_{sp}$, and $L_{sp} \approx 20 t_{sp}$, then this ratio reduces to 25 indicating that the shear connection is 25 times stiffer than the flange connections with respect to resisting vertical forces.

Step 18. Bearing and tearout of the shear bolts relative to the T-stem and beam flange is checked consistent with Section J3.10 of the AISC *Specification* using actual dimensions and the expected flange force, F_f .

Step 19. Block shear of the T-stem and beam flange should be checked in a manner consistent with Section J4.3 of the AISC *Specification*. For the purpose of this design, the block shear failure shall be considered a ductile failure mode.

The “alternate block shear” mechanism illustrated in Figure 13.7 need not be checked. For this failure mechanism to form, a net rupture of the beam flange must occur, and this is precluded by the check in **Step 2**. In numerous documented tests of double-tee and bolted flange plate connections, this alternative block shear mechanism has not been observed.

Step 20. Because of the large setback required, the shear connection will most likely need to be designed as an “extended” shear tab, particularly when one considers that the shear connection should be designed with sufficient strength in the event of the failure of the T-stubs. Most importantly, the length of the shear connection, L_{sc} , should be determined so as to fit between the flanges of the T-stubs allowing ample clearance.

Consistent with AISC *Specification* Section J4.3, block shear of the beam web should be checked for the failure path shown in Figure C-13.2. For the purpose of this design, the block shear failure shall be considered a ductile failure mode.

Step 21. The strength of the column flange adjacent to the T-stub is considered in **Step 21**. A yield line analysis is the basis for the strength in Equations 13.6-55 through 13.6-61. The yield-line pattern is shown in Figure 13.8, wherein it is assumed

that the top T-stub is in tension. With the requirement that both T-stubs be identical, checking only one location is satisfactory.

Note that the yield-line pattern shown in Figure 13.8 is applicable to both the cases of T-stubs with eight tension bolts or four tension bolts. In the case where eight tension bolts are used, the yield pattern is defined by the inner four bolts and the associated gage, g_{ic} , shown in the figure.

Step 22. In checking the column for web local yielding and web local crippling consistent with Sections J10.2 and J10.3 of the AISC *Specification*, respectively, the bearing length, l_b , can be taken as $5k_T + t_{st}$, where k_T is the k dimension of the T-stub and t_{st} is the T-stub stem thickness. Given the requirement for continuity plates, however, web local yielding and web local buckling are not expected to control.

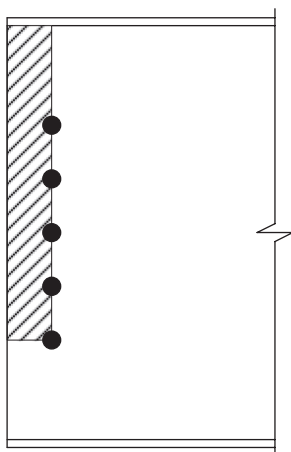


Fig. C-13.2. Block shear failure mode for the beam web.

APPENDIX A

CASTING REQUIREMENTS

A1. CAST STEEL GRADE

The cast steel grade is selected for its ability to provide ductility similar to that of rolled steel. The material has a specified minimum yield and tensile strength of 50 ksi (354 MPa) and 80 ksi (566 MPa), respectively. The ASTM specification requires the castings be produced in conjunction with a heat treatment process that includes normalizing and stress relieving. It also requires each heat of steel meet strict mechanical properties. These properties include the specified tensile and yield strengths, as well as elongation and area reduction limitations.

A2. QUALITY CONTROL (QC)

See Commentary Section 3.7.

2. First Article Inspection (FAI) of Castings

The intent of this section is that at least one casting of each pattern undergo first article inspection (FAI). When a casting pattern is replaced or when the rigging is modified, FAI is to be repeated.

3. Visual Inspection of Castings

All casting surfaces shall be free of adhering sand, scales, cracks, hot tears, porosity, cold laps and chaplets. All cored holes in castings shall be free of flash and raised surfaces. The ASTM specification includes acceptance criteria for the four levels of surface inspection. Level I is the most stringent criteria. The Manufacturers Standardization Society (MSS) specification includes a set of reference comparators for the visual determination of surface texture, surface roughness and surface discontinuities.

4. Nondestructive Testing (NDT) of Castings

These provisions require the use of nondestructive testing (NDT) to verify the castings do not contain indications that exceed the specified requirements.

Radiographic testing (RT) is capable of detecting internal discontinuities and is specified only for the FAI. The ASTM specifications contain referenced radiographs and five levels of RT acceptance. The lower acceptance levels are more stringent and are typically required on high-performance aerospace parts such as jet engine turbine blades or on parts that may leak, such as valves or pumps. Level III is considered the industry standard for structurally critical components.

Ultrasonic testing (UT) is also capable of detecting internal discontinuities and is specified for production castings. The ASTM specification includes seven levels of UT acceptance. The lower acceptance levels are more stringent and are typically reserved for machined surfaces subject to contact stresses, such as gear teeth. Level 3 is considered the industry standard for structurally critical components.

The areas to be covered by RT or UT are those adjacent to the junctions of:

- (1) The vertical flange and the horizontal flange.
- (2) The vertical flange and the vertical stiffener.
- (3) The horizontal flange and the vertical stiffener.

Magnetic particle testing (MT) is required to detect other forms of discontinuities on or near the surface of the casting. The ASTM specification includes five levels of MT acceptance. The lower levels are more stringent and are typically reserved for pressure vessels. Level V is considered the industry standard for structurally critical components.

Shrinkage is one of the more commonly occurring internal discontinuities and is a result of metal contraction in the mold during solidification. Shrinkage is avoided using reservoirs of molten metal known as risers that compensate for the volumetric contraction during solidification. Numerical modeling of solidification and prediction of shrinkage have been the focus of a number of investigations performed in conjunction with the Steel Founders' Society of America (SFA). Niyama et al. (1982) developed a criterion that relates the casting temperature gradient and cooling rate. Based on the Niyama criterion, Hardin et al. (1999) developed a correlation between casting simulation and radiographic testing. Subsequently, Carlson et al. (2003) determined that variation in internal porosity (shrinkage) was related to the pattern and rigging of the casting mold.

Based on these conclusions, the provisions require RT and MT on the FAI of castings to verify that the pattern and rigging are capable of producing a satisfactory casting. Subsequent castings manufactured with the same pattern and rigging require UT and MT to verify production consistency.

Research performed by Briggs (1967) on the effect of discontinuities found that castings perform satisfactorily under loads in excess of service requirements even with discontinuities of considerable magnitude. Testing demonstrated fatigue and static failures occurred at the location of maximum stress regardless of the presence of discontinuities in other sections.

6. Tensile Requirements

Coupons or keel blocks for tensile testing shall be cast and treated from the same batch of representative castings. Each test specimen shall have complete documentation and traceability. If the specimens fail to meet required specifications, then all the castings they represent shall be rejected.

A3. MANUFACTURER DOCUMENTS

Submittal documents allow a thorough review on the part of the patent holder, engineer of record, the authority having jurisdiction, and outside consultants, if required.

APPENDIX B

FORGING REQUIREMENTS

There is no Commentary for this Appendix.

REFERENCES

The following references have been reviewed as a basis for the prequalification of the connections described in this Standard. Although some references are not specifically cited in this Standard, they have been reviewed by the AISC Connection Prequalification Review Panel and are listed here to provide an archival record of the basis for this Standard in accordance with the requirements of Chapter K of the AISC *Seismic Provisions*.

ALL CONNECTIONS

- AISC (2005a), *Seismic Provisions for Structural Steel Buildings*, ANSI/AISC 341-05, American Institute of Steel Construction, Chicago, IL.
- AISC (2005b), *Specification for Structural Steel Buildings*, ANSI/AISC 360-05, American Institute of Steel Construction, Chicago, IL.
- AISC (2005c), *Steel Construction Manual*, 13th Ed., American Institute of Steel Construction, Chicago, IL.
- AISC (2010a), *Seismic Provisions for Structural Steel Buildings*, ANSI/AISC 341-10, American Institute of Steel Construction, Chicago, IL.
- AISC (2010b), *Specification for Structural Steel Buildings*, ANSI/AISC 360-10, American Institute of Steel Construction, Chicago, IL.
- AISC (2016a), *Seismic Provisions for Structural Steel Buildings*, ANSI/AISC 341-16, American Institute of Steel Construction, Chicago, IL.
- AISC (2016b), *Specification for Structural Steel Buildings*, ANSI/AISC 360-16, American Institute of Steel Construction, Chicago, IL.
- Anderson, J.C. and Linderman, R.R. (1991), “Steel Beam to Box Column Connections,” Report No. CE 91-03, Department of Civil Engineering, University of Southern California, Los Angeles, CA.
- ATC (1992), *Guidelines for Cyclic Seismic Testing of Components of Steel Structures*, ATC-24, Applied Technology Council, Redwood City, CA.
- AWS (2010), *Criteria for Describing Oxygen-Cut Surfaces*, AWS C4.1:2010, American Welding Society, Miami, FL.
- AWS (2015), *Structural Welding Code—Steel*, AWS D1.1/D1.1M:2015, American Welding Society, Miami, FL.
- AWS (2016), *Structural Welding Code—Seismic Supplement*, AWS D1.8/D1.8M:2016, American Welding Society, Miami, FL.

- Ball, S.C., Uang, C.M., Kim, D.W. and Sim, H.B. (2010), “Cyclic Testing of Steel Non-orthogonal RBS Moment Connections for LAX Tom Bradley International Terminal Modernization Program,” *Proceedings of the 79th Annual SEAOC Convention*, Indian Wells, CA, Structural Engineers Association of California, Sacramento, CA.
- Barsom, J. and Korvink, S. (1998), “Effects of Strain Hardening and Strain Aging on the K-Region of Structural Shapes,” Report No. SAC/BD-98/02, SAC Joint Venture, Sacramento, CA.
- Chi, W.M., Deierlein, G. and Ingrassia, A. (1997), “Finite Element Fracture Mechanics Investigation of Welded Beam-Column Connections,” Report No. SAC/BD-97/05, SAC Joint Venture, Sacramento, CA.
- FEMA (1997), *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*, FEMA 302, Part 1—Provisions, Federal Emergency Management Agency, Washington, DC.
- FEMA (2000a), *Recommended Post-Earthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings*, FEMA 352, Federal Emergency Management Agency, Washington, DC.
- FEMA (2000b), *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*, FEMA 350, Federal Emergency Management Agency, Washington, DC.
- FEMA (2000c), *Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings*, FEMA 351, Federal Emergency Management Agency, Washington, DC.
- FEMA (2000d), *Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications*, FEMA 353, Federal Emergency Management Agency, Washington, DC.
- FEMA (2000e), *State of the Art Report on Connection Performance*, FEMA 355D, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.
- FEMA (2000f), *State of the Art Report on Systems Performance of Steel Moment Frames Subject to Earthquake Ground Shaking*, FEMA 355C, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.
- FEMA (2011), *Quantification of Building Seismic Performance Factors: Component Equivalency Methodology*, FEMA P-795, Federal Emergency Management Agency, Washington, DC.
- Kim, T., Stojadinović, B. and Whittaker, A.S. (2008), “Seismic Performance of Pre-Northridge Welded Steel Moment Connections to Built-Up Box Columns,” *Journal of Structural Engineering*, ASCE, Vol. 134, pp. 289–299.
- Prinz, G.S. and Richards, P.W. (2016), “Demands on Reduced Beam Section Connections with Out-of-Plane Skew,” *Journal of Structural Engineering*, American Society of Civil Engineers, Vol. 142, No. 1.

- RCSC (2014), *Specification for Structural Joints Using High-Strength Bolts*, Research Council on Structural Connections, Chicago, IL.
- Tsai, K.C. and Popov, E.P. (1986), “Two Beam-to-Column Web Connections,” Report No. UCB/EERC-86/05, Earthquake Engineering Research Center, University of California at Berkeley, April.
- Tsai, K.C. and Popov, E.P. (1988), “Steel Beam-Column Joints in Seismic Moment Resisting Frames,” Report No. UCB/EERC-88/19, Earthquake Engineering Research Center, University of California at Berkeley, November.

CHAPTER 5

REDUCED BEAM SECTION (RBS) MOMENT CONNECTION

- Chambers, J.J., Almudhafer, S. and Stenger, F. (2003), “Effect of Reduced Beam Section Frames Elements on Stiffness of Moment Frames,” *Journal of Structural Engineering*, ASCE, Vol. 129, No. 3.
- Chen, S.J. and Chao, Y.C. (2001), “Effect of Composite Action on Seismic Performance of Steel Moment Connections with Reduced Beam Sections,” *Journal of Constructional Steel Research*, Elsevier Science Publishers, Vol. 57.
- Chen, S.J., Yeh, C.H. and Chu, J.M. (1996), “Ductile Steel Beam-to-Column Connections for Seismic Resistance,” *Journal of Structural Engineering*, ASCE, Vol. 122, No. 11, pp. 1,292–1,299.
- Engelhardt, M.D. (1999), “The 1999 T.R. Higgins Lecture: Design of Reduced Beam Section Moment Connections,” *Proceedings 1999 North American Steel Construction Conference*, Toronto, Canada, pp. 1-1 to 1-29, American Institute of Steel Construction, Chicago, IL.
- Engelhardt, M.D., Fry, G., Jones, S., Venti, M. and Holliday, S. (2000), “Behavior and Design of Radius-Cut Reduced Beam Section Connections,” Report No. SAC/BD-00/17, SAC Joint Venture, Sacramento, CA.
- Engelhardt, M.D., Winneberger, T., Zekany, A.J. and Potyraj, T.J. (1998), “Experimental Investigation of Dogbone Moment Connections,” *Engineering Journal*, AISC, Vol. 35, No. 4, 4th Quarter, pp. 128–139.
- FEMA (2000g), *State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes*, FEMA 355e, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.
- Gilton, C., Chi, B. and Uang, C.M. (2000), “Cyclic Response of RBS Moment Connections: Weak Axis Configuration and Deep Column Effects,” Report No. SAC/BD-00/23, SAC Joint Venture, Sacramento, CA.
- Grubbs, K.V. (1997), “The Effect of the Dogbone Connection on the Elastic Stiffness of Steel Moment Frames,” M.S. Thesis, Department of Civil Engineering, The University of Texas at Austin.

- Iwankiw, N.R. and Carter, C. (1996), "The Dogbone: A New Idea to Chew On," *Modern Steel Construction*, AISC, April.
- Kim, D.W., Sim, H.B. and Uang, C.M. (2010), "Cyclic Testing of Non-Orthogonal Steel Moment Connections for LAX TBIT Modifications," Report No. TR-09/04, Department of Structural Engineering, University of California, San Diego, La Jolla, CA.
- Lee, C.H., Kim, J.H., Jeon, S.W. and Kim, J.H. (2004), "Influence of Panel Zone Strength and Beam Web Connection Method on Seismic Performance of Reduced Beam Section Steel Moment Connections," *Proceedings of the CTBUH 2004 Seoul Conference—Tall Buildings for Historical Cities*, Council on Tall Buildings and Urban Habitat, Bethlehem, PA.
- Moore, K.S., Malley, J.O. and Engelhardt, M.D. (1996), "Design of Reduced Beam Section (RBS) Moment Connections," *Steel Tips*, Structural Steel Education Council, Moraga, CA.
- Okahashi, Y. (2003), "Reduced Beam Section Connection without Continuity Plates," M.S. Thesis, Department of Civil and Environmental Engineering, University of Utah.
- Plumier, A. (1990), "New Idea for Safe Structures in Seismic Zones," *IABSE Symposium—Mixed Structures Including New Materials*, Brussels, Belgium.
- Plumier, A. (1997), "The Dogbone: Back to the Future," *Engineering Journal*, AISC, Vol. 34, No. 2, 2nd Quarter, pp. 61–67.
- Popov, E.P., Yang, T.S. and Chang, S.P. (1998), "Design of Steel MRF Connections Before and After 1994 Northridge Earthquake," *International Conference on Advances in Steel Structures*, Hong Kong, December 11–14, 1996. Also in: *Engineering Structures*, Elsevier Science Publishers, Vol. 20, No. 12, pp. 1,030–1,038.
- Ricles, J.M., Zhang, X., Lu, L.W. and Fisher, J. (2004), "Development of Seismic Guidelines for Deep Column Steel Moment Connections," ATLSS Report No. 04–13, Lehigh University, Bethlehem, PA.
- Shen, J., Kitjasetanphun, T. and Srivanich, W. (2000), "Seismic Performance of Steel Moment Frames with Reduced Beam Sections," *Journal of Constructional Steel Research*, Elsevier Science Publishers, Vol. 22.
- Shen, J., Astaneh-Asl, A. and McCallen, D.B. (2002), "Use of Deep Columns in Special Steel Moment Frames," *Steel Tips*, Structural Steel Education Council, Moraga, CA.
- Suita, K., Tamura, T., Morita, S., Nakashima, M. and Engelhardt, M.D. (1999), "Plastic Rotation Capacity of Steel Beam-to-Column Connections Using a Reduced Beam Section and No Weld Access Hole Design—Full Scale Tests for Improved Steel Beam-to-Column Subassemblies—Part 1," *Structural Journal*, Architectural Institute of Japan, No. 526, pp. 177–184, (in Japanese).
- Tremblay, R., Tchebotarev, N. and Filiatrault, A. (1997), "Seismic Performance of RBS Connections for Steel Moment Resisting Frames: Influence of Loading Rate and Floor Slab," *Proceedings, Stessa '97*, Kyoto, Japan.

- Tsai, K.C., Chen, W.Z. and Lin, K.C. (1999), “Steel Reduced Beam Section to Weak Panel Zone Moment Connections,” *Proceedings: Workshop on Design Technologies of Earthquake-Resistant Moment-Resisting Connections in Steel Buildings*, May 17–18, 1999, Taipei, Taiwan (in Chinese).
- Uang, C.M. and Fan, C.C. (1999), “Cyclic Instability of Steel Moment Connections with Reduced Beam Section,” Report No. SAC/BD-99/19, SAC Joint Venture, Sacramento, CA.
- Uang, C.M. and Richards, P. (2002), “Cyclic Testing of Steel Moment Connections for East Tower of Hoag Memorial Hospital Presbyterian,” Third Progress Report, University of California, San Diego, CA.
- Yu, Q.S., Gilton, C. and Uang, C.M. (2000), “Cyclic Response of RBS Moment Connections: Loading Sequence and Lateral Bracing Effects,” Report No. SAC/BD-00/22, SAC Joint Venture, Sacramento, CA.
- Zekioglu, A., Mozaffarian, H., Chang, K.L., Uang, C.M. and Noel, S. (1997), “Designing after Northridge,” *Modern Steel Construction*, American Institute of Steel Construction, Chicago, IL.
- Zekioglu, A., Mozaffarian, H. and Uang, C.M. (1997), “Moment Frame Connection Development and Testing for the City of Hope National Medical Center,” *Building to Last—Proceedings of Structures Congress XV*, Portland, American Society of Civil Engineers, Reston, VA.

CHAPTER 6

BOLTED UNSTIFFENED AND STIFFENED EXTENDED END-PLATE MOMENT CONNECTIONS

- Abel, M.S. and Murray, T.M. (1992a), “Multiple Row, Extended Unstiffened End-Plate Connection Tests,” Research Report CE/VPI-ST-92/04, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA.
- Abel, M.S. and Murray, T.M. (1992b), “Analytical and Experimental Investigation of the Extended Unstiffened Moment End-Plate Connection with Four Bolts at the Beam Tension Flange,” Research Report CE/VPI-ST-93/08, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA.
- Adey, B.T., Grondin, G.Y. and Cheng, J.J.R. (1997), “Extended End Plate Moment Connections under Cyclic Loading,” Structural Engineering Report No. 216, Department of Civil and Environmental Engineering, University of Alberta, Edmonton, Alberta, Canada.
- Adey, B.T., Grondin, G.Y. and Cheng, J.J.R. (1998), “Extended End Plate Moment Connections under Cyclic Loading,” *Journal of Constructional Steel Research*, Elsevier Science Publishers, Vol. 46, pp. 1–3.
- Adey, B.T., Grondin, G.Y. and Cheng, J.J.R. (2000), “Cyclic Loading of End Plate Moment Connections,” *Canadian Journal of Civil Engineering*, National Research Council of Canada, Vol. 27, No. 4, pp. 683–701.

- Agerskov, H. (1976), "High Strength Bolted Connections Subject to Prying," *Journal of the Structural Division*, ASCE, Vol. 102, No. ST1, pp. 161–175.
- Agerskov, H. (1977), "Analysis of Bolted Connections Subject to Prying," *Journal of the Structural Division*, ASCE, Vol. 103, No. ST11, pp. 2,145–2,163.
- Ahuja, V. (1982), "Analysis of Stiffened End-Plate Connections Using Finite Element Method," M.S. Thesis, School of Civil Engineering and Environmental Science, University of Oklahoma, Norman, OK.
- Bahaari, M.R. and Sherbourne, A.N. (1993), "Modeling of Extended End-plate Bolted Connections," *Proceedings of the National Steel Structures Congress*, pp. 731–736, American Institute of Steel Construction, Chicago, IL.
- Bjorhovde, R., Brozzetti, J. and Colson, A. (1987) "Classification of Connections," *Connections in Steel Structures—Behaviour, Strength and Design*, Elsevier Science Publishers, London, U.K., pp. 388–391.
- Bjorhovde, R., Colson, A. and Brozzetti, J. (1990), "Classification System for Beam-to-Column Connections," *Journal of Structural Engineering*, ASCE, Vol. 116, No. 11, pp. 3,059–3,076.
- Borgsmiller, J.T. and Murray, T.M. (1995), "Simplified Method for the Design of Moment End-Plate Connections," Research Report CE/VPI-ST-95/19, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA.
- Bursi, O.S. and Leonelli, L. (1994), "A Finite Element Model for the Rotational Behavior of End Plate Steel Connections," *SSRC Proceedings 1994 Annual Task Group Technical Session*, pp. 162–175, Lehigh University, Bethlehem, PA.
- Carter, C.J. (1999), *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications*, Design Guide No. 13, American Institute of Steel Construction, Chicago, IL.
- Castellani, A., Castiglioni, C.A., Chesi, C. and Plumier, A. (1998), "A European Research Program on the Cyclic Behaviour of Welded Beam to Column Connections," *Proceedings of the NEHRP Conference and Workshop on Research on the Northridge, California Earthquake of January 17, 1994*, Vol. III-B, pp. 510–517, National Earthquake Hazards Reduction Program, Washington, DC.
- Coons, R.G. (1999), "Seismic Design and Database of End Plate and T-stub Connections," M.S. Thesis, University of Washington, Seattle, WA.
- Disque, R.O. (1962), "End-Plate Connections," *National Engineering Conference Proceedings*, American Institute of Steel Construction, pp. 30–37.
- Douty, R.T. and McGuire, S. (1965), "High Strength Bolted Moment Connections," *Journal of the Structural Division*, ASCE, Vol. 91, No. ST2, pp. 101–126.
- Fleischman, R.B., Chasten, C.P., Lu, L-W and Driscoll, G.C. (1991), "Top-and-Seat Angle Connections and End-Plate Connections: Snug vs. Fully Pretensioned Bolts," *Engineering Journal*, AISC, Vol. 28, No. 1, 1st Quarter, pp. 18–28.

- Ghassemieh, M. (1983), "Inelastic Finite Element Analysis of Stiffened End-Plate Moment Connections," M.S. Thesis, School of Civil Engineering and Environmental Science, University of Oklahoma, Norman, OK.
- Ghobarah, A., Korol, R.M. and Osman, A. (1992), "Cyclic Behavior of Extended End-Plate Joints," *Journal of Structural Engineering*, ASCE, Vol. 118, No. 5, pp. 1,333–1,353.
- Ghobarah, A., Osman, A. and Korol, R.M. (1990), "Behaviour of Extended End-Plate Connections under Cyclic Loading," *Engineering Structures*, Elsevier Science Publishers, Vol. 12, pp. 15–26.
- Granstrom, A. (1980), "Bolted End-Plate Connections," Stalbyggnads Institute SBI Report 86.3, pp. 5–12.
- Griffiths, J.D. (1984), "End-Plate Moment Connections—Their Use and Misuse," *Engineering Journal*, AISC, Vol. 21, No. 1, 1st Quarter, pp. 32–34.
- Hasan, R., Kishi, N. and Chen, W.F. (1997), "Evaluation of Rigidity of Extended End-Plate Connections," *Journal of Structural Engineering*, ASCE, Vol. 123, No. 12, pp. 1,595–1,602.
- Hendrick, D., Kukreti, A. and Murray, T. (1984), "Analytical and Experimental Investigation of Stiffened Flush End-Plate Connections with Four Bolts at the Tension Flange," Research Report FSEL/MBMA 84-02, Fears Structural Engineering Laboratory, University of Oklahoma, Norman, OK.
- Hendrick, D., Kukreti, A. and Murray, T. (1985), "Unification of Flush End-Plate Design Procedures," Research Report FSEL/MBMA 85-01, Fears Structural Engineering Laboratory, University of Oklahoma, Norman, OK.
- Johnstone, N.D. and Walpole, W.R. (1981), "Bolted End-Plate Beam-to-Column Connections Under Earthquake Type Loading," Report 81-7, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.
- Kato, B. and McGuire, W.F. (1973), "Analysis of T-Stub Flange-to-Column Connections," *Journal of the Structural Division*, ASCE, Vol. 99 No. ST5, pp. 865–888.
- Kennedy, N.A., Vinnakota, S. and Sherbourne, A.N. (1981), "The Split-Tee Analogy in Bolted Splices and Beam-Column Connections," *Proceedings of the International Conference on Joints in Structural Steelwork*, pp. 2.138–2.157.
- Kline, D., Rojiani, K. and Murray, T. (1989), "Performance of Snug Tight Bolts in Moment End-Plate Connections," MBMA Research Report, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA. Revised July 1995.
- Korol, R.M., Ghobarah, A. and Osman, A. (1990), "Extended End-Plate Connections Under Cyclic Loading: Behaviour and Design," *Journal of Constructional Steel Research*, Elsevier Science Publishers, Vol. 16, No. 4, pp. 253–279.
- Krishnamurthy, N. (1978), "A Fresh Look at Bolted End-Plate Behavior and Design," *Engineering Journal*, AISC, Vol. 15, No. 2, 2nd Quarter, pp. 39–49.

- Krishnamurthy, N. and Graddy, D. (1976), "Correlation between 2- and 3-Dimensional Finite Element Analysis of Steel Bolted End Plate Connections," *Computers and Structures*, Vol. 6, No. 4/5, pp. 381–389.
- Kukreti, A.R., Ghassemieh, M. and Murray, T.M. (1990), "Behavior and Design of Large-Capacity Moment End-Plates," *Journal of Structural Engineering*, ASCE Vol. 116, No. 3, pp. 809–828.
- Kukreti, A.R., Murray, T.M. and Abolmaali, A. (1987), "End-Plate Connection Moment-Rotation Relationship," *Journal of Constructional Steel Research*, Elsevier Science Publishers, Vol. 8, pp. 137–157.
- Mann, A.P. and Morris, L.J. (1979), "Limit Design of Extended End-Plate Connections," *Journal of the Structural Division*, ASCE, Vol. 105, No. ST3, pp. 511–526.
- Meng, R.L. (1996), "Design of Moment End-Plate Connections for Seismic Loading," Doctoral Dissertation, Virginia Polytechnic Institute and State University, Blacksburg, VA.
- Meng, R.L. and Murray, T.M. (1997), "Seismic Performance of Bolted End-Plate Moment Connections," *Proceedings of the 1997 National Steel Construction Conference*, American Institute of Steel Construction, pp. 30-1 to 30-14.
- Morrison, S.J., Astaneh-Asl, A. and Murray, T. (1985), "Analytical and Experimental Investigation of the Extended Stiffened Moment End-Plate Connection with Four Bolts at the Beam Tension Flange," Research Report FSEL/MBMA 85-05, Fears Structural Engineering Laboratory, University of Oklahoma, Norman, OK.
- Morrison, S.J., Astaneh-Asl, A. and Murray, T. (1986), "Analytical and Experimental Investigation of the Multiple Row Extended $\frac{1}{3}$ Moment End-Plate Connection with Eight Bolts at the Beam Tension Flange," Research Report FSEL/MBMA 86-01, Fears Structural Engineering Laboratory, University of Oklahoma, Norman, OK.
- Murray, T.M. (1986), "Stability of Gable Frame Panel Zone Plates," *Proceedings of the Structural Stability Research Council Annual Technical Session*, pp. 317–325, Structural Stability Research Council, Bethlehem, PA.
- Murray, T.M. (1988), "Recent Developments for the Design of Moment End-Plate Connections," *Journal of Constructional Steel Research*, Vol. 10, pp. 133–162.
- Murray, T.M. (1990), *Extended End-Plate Moment Connections*, Design Guide No. 4, American Institute of Steel Construction, Chicago, IL.
- Murray, T.M., Kline, D.P. and Rojiani, K.B. (1992), "Use of Snug-Tightened Bolts in End-Plate Connections," *Connections in Steel Structures II: Behavior, Strength and Design*, Edited by R. Bjorhovde et al., American Institute of Steel Construction, Chicago, IL, pp. 27–34.
- Murray, T.M. and Kukreti, A.R. (1988), "Design of 8-Bolt Stiffened Moment End Plates," *Engineering Journal*, AISC, Vol. 25, No. 2, 2nd Quarter, Chicago, IL, pp. 45–52.

- Murray, T.M. and Shoemaker, W.L. (2002), *Flush and Extended Multiple Row Moment End Plate Connections*, Design Guide No. 16, American Institute of Steel Construction, Chicago, IL.
- Murray, T.M. and Sumner, E.A. (2003), *Extended End-Plate Moment Connections: Seismic and Wind Applications*, Design Guide No. 4, 2nd Ed., American Institute of Steel Construction, Chicago, IL.
- Nair, R., Birkemoe, P. and Munse, W. (1974), "High Strength Bolts Subject to Tension and Prying," *Journal of the Structural Division*, ASCE, Vol. 100, No. ST2, pp. 351–372.
- Packer, J. and Morris, L. (1977), "A Limit State Design Method for the Tension Region of Bolted Beam-Column Connections," *The Structural Engineer*, Vol. 55, No. 10, pp. 446–458.
- Popov, E. and Tsai, K.C. (1989), "Performance of Large Seismic Steel Moment Connections under Cyclic Loads," *Engineering Journal*, AISC, Vol. 12, No. 2, 2nd Quarter, pp. 51–60.
- Ryan, J.C. and Murray, T.M. (1999), "Evaluation of the Inelastic Rotation Capability of Extended End-Plate Moment Connections," Research Report No. CE/VPI-ST-99/13, submitted to Metal Building Manufacturers Association and American Institute of Steel Construction, Virginia Polytechnic Institute and State University, Blacksburg, VA.
- Salmon, C. and Johnson, J. (1980), *Steel Structures, Design and Behavior*, 2nd Ed., Harper & Row, New York, NY.
- Seek, M.W. and Murray, T.M. (2008), "Seismic Strength of Moment End-Plate Connections with Attached Concrete Slab," *Proceedings, Connections VI*, American Institute of Steel Construction, Chicago, IL, June 23–25.
- Srouji, R., Kukreti, A.R. and Murray, T.M. (1983a), "Strength of Two Tension Bolt Flush End-Plate Connections," Research Report FSEL/MBMA 83-03, Fears Structural Engineering Laboratory, University of Oklahoma, Norman, OK.
- Srouji, R., Kukreti, A.R. and Murray, T.M. (1983b), "Yield-Line Analysis of End-Plate Connections With Bolt Force Predictions," Research Report FSEL/MBMA 83-05, Fears Structural Engineering Laboratory, University of Oklahoma, Norman, OK.
- Sumner, E.A., Mays, T.W. and Murray, T.M. (2000a), "Cyclic Testing of Bolted Moment End-Plate Connections," Research Report No. CE/VPI-ST-00/03, SAC Report No. SAC/BD00/21, submitted to the SAC Joint Venture, Virginia Polytechnic Institute and State University, Blacksburg, VA.
- Sumner, E.A., Mays, T.W. and Murray, T.M. (2000b), "End-Plate Moment Connections: Test Results and Finite Element Method Validation," *Connections in Steel Structures IV, Proceedings of the Fourth International Workshop*, American Institute of Steel Construction, Chicago, IL, pp. 82–93.
- Sumner, E.A. and Murray, T.M. (2001), "Experimental Investigation of the MRE 1/2 End-Plate Moment Connection," Research Report No. CE/VPI-ST-01/14, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA.

- Sumner, E.A. and Murray, T.M. (2002), “Behavior of Extended End-Plate Moment Connections Subject to Cyclic Loading,” *Journal of Structural Engineering*, ASCE, Vol. 128, No. 4, pp. 501–508.
- Tsai, K.C. and Popov, E.P. (1990), “Cyclic Behavior of End-Plate Moment Connections,” *Journal of Structural Engineering*, ASCE, Vol. 116, No. 11, pp. 2,917–2,930.
- Yang, H., Tagawa, Y. and Nishiyama, I. (2003), “Elasto-Plastic Behavior of ‘New Composite Beam System’,” *Steel Structures*, Vol. 3, pp. 45–52.
- Young, J. and Murray, T.M. (1996), “Experimental Investigation of Positive Bending Moment Strength of Rigid Knee Connections,” Research Report No. CE/VPI-ST 9617, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA.

CHAPTER 7

BOLTED FLANGE PLATE (BFP) MOMENT CONNECTION

- Krawinkler, H., (1992), “Guidelines for Cyclic Seismic Testing of Components of Steel Structures,” Report ATC-24, Applied Technology Council, Redwood City, CA.
- Krawinkler, H., Gupta, A., Medina, R. and Luco, N. (2000), “Loading Histories for Seismic Performance Testing of SMRF Components and Assemblies,” Report SAC/BD-00/10, SAC Joint Venture, Sacramento, CA.
- Sato, A., Newell, J. and Uang, C.M. (2008), “Cyclic Behavior and Seismic Design of Bolted Flange Plate Steel Moment Connections,” *Engineering Journal*, AISC, Vol. 45, No. 4, 4th Quarter, pp. 221–232.
- Schneider, S.P. and Teeraparbwong, I. (1999), “SAC Task 7.09: Bolted Flange Plate Connections,” report submitted to the SAC Joint Venture by the University of Illinois, Urbana, IL.
- Seek, M.W. and Murray, T. M. (2008), “Seismic Strength of Moment End-Plate Connections with Attached Concrete Slab,” *Proceedings, Connections VI*, American Institute of Steel Construction, Chicago, IL, June 23–25.
- Swanson, J., Leon, R.D. and Smallridge, J. (2000), “Tests on Bolted Connections,” Report SAC/BD-00/04, SAC Joint Venture, Sacramento, CA.

CHAPTER 8

WELDED UNREINFORCED FLANGE–WELDED WEB (WUF-W) MOMENT CONNECTION

- Lee, D., Cotton, S.C., Dexter, R.J., Hajjar, J.F., Ye, Y. and Ojard, S.D. (2002), “Column Stiffener Detailing and Panel Zone Behavior of Steel Moment Frame Connections,” Structural Engineering Report No. ST-01-3.2, Department of Civil Engineering, University of Minnesota.

- Lee, D., Cotton, S.C., Hajjar, J., Dexter, R.J. and Ye, Y. (2005a), “Cyclic Behavior of Steel Moment-Resisting Connections Reinforced by Alternative Column Stiffener Details I. Connection Performance and Continuity Plate Detailing,” *Engineering Journal*, AISC, Vol. 42, No. 4, 4th Quarter, pp.189–214.
- Lee, D., Cotton, S.C., Hajjar, J., Dexter, R.J. and Ye, Y. (2005b), “Cyclic Behavior of Steel Moment-Resisting Connections Reinforced by Alternative Column Stiffener Details II. Panel Zone Behavior and Doubler Plate Detailing,” *Engineering Journal*, AISC, Vol. 42, No. 4, 4th Quarter, pp. 215–238.
- Ricles, J.M., Mao, C., Lu, L.W. and Fisher, J.W. (2000), “Development and Evaluation of Improved Details for Ductile Welded Unreinforced Flange Connections,” Report No. SAC/BD-00-24, SAC Joint Venture, Sacramento, CA.
- Ricles, J.M., Mao, C., Lu, L.W. and Fisher, J.W. (2002), “Inelastic Cyclic Testing of Welded Unreinforced Moment Connections,” *Journal of Structural Engineering*, ASCE, Vol. 128, No. 4.
- CHAPTER 9**
- KAISER BOLTED BRACKET (KBB) MOMENT CONNECTION**
- Adan, S.M. and Gibb, W. (2009), “Experimental Evaluation of Kaiser Bolted Bracket Steel Moment Resisting Connections,” *Engineering Journal*, AISC, Vol. 46, No. 3, 3rd Quarter, pp. 181–195.
- Grigorian, C.E., Yang, T.S. and Popov, E.P. (1992), “Slotted Bolted Connection Energy Dissipators,” EERC Report No. 92/10, Earthquake Engineering Research Center, University of California, Berkeley, CA.
- Gross, J.L., Engelhardt, M.D., Uang, C.M., Kasai, K. and Iwankiw, N.R. (1999), *Modification of Existing Welded Steel Moment Frame Connections for Seismic Resistance*, Design Guide No. 12, American Institute of Steel Construction, Chicago, IL.
- Kasai, K. and Bleiman, D. (1996), “Bolted Brackets for Repair of Damaged Steel Moment Frame Connections,” *7th US–Japan Workshop on the Improvement of Structural Design and Construction Practices: Lessons Learned from Northridge and Kobe*, Kobe, Japan.
- Kulak, G.L., Fisher, J.W. and Struik, J.H.A. (1987), *Guide to Design Criteria for Bolted and Riveted Joints*, 2nd Ed., Wiley, New York, NY.
- Masuda, H., Tamaka, A., Hirabayashi, K. and Genda, I. (1998), “Experimental Study on the Effect of Partial Loss of Sectional Area on the Static Characteristics of H-Beams,” *Journal of Structural and Construction Engineering* (Transaction of AIJ), Architectural Institute of Japan, No. 512, pp. 157–164, October (in Japanese).
- Newell, J. and Uang, C.M. (2006), “Cyclic Testing of Steel Moment Connections for the CALTRANS District 4 Office Building Seismic Rehabilitation,” UCSD Report No. SSRP-05/03, University of California, San Diego, CA.

- Ricles, J.M., Zhang, X., Lu, L.W. and Fisher, J. (2004), “Development of Seismic Guidelines for Deep Column Steel Moment Connections,” ATLSS Report No. 04-13, Lehigh University, Bethlehem, PA.
- Seek, M. W. and Murray, T.M. (2008), “Seismic Strength of Moment End-Plate Connections with Attached Concrete Slab,” *Proceedings, Connections VI*, American Institute of Steel Construction, Chicago, IL, June 23–25.
- Tide, R.H.R. (2010), “Bolt Shear Design Considerations,” *Engineering Journal*, AISC Vol. 47, No. 1, 1st Quarter, pp. 47–63.

CHAPTER 10

CONXTECH CONXL MOMENT CONNECTION

- AISC (2005a), *Seismic Provisions for Structural Steel Buildings*, ANSI/AISC 341-05, American Institute of Steel Construction, Chicago, IL.
- AISC (2005b), *Specification for Structural Steel Buildings*, ANSI/AISC 360-05, American Institute of Steel Construction, Chicago, IL.
- AISC (2010a), *Seismic Provisions for Structural Steel Buildings*, ANSI/AISC 341-10, American Institute of Steel Construction, Chicago, IL.
- AISC (2010b), *Specification for Structural Steel Buildings*, ANSI/AISC 360-10, American Institute of Steel Construction, Chicago, IL.
- Seek, M.W. and Murray, T.M. (2005), “Cyclic Test of 8-Bolt Extended Stiffened Steel Moment End Plate Connection with Concrete Structural Slab,” report submitted to the American Institute of Steel Construction, Virginia Polytechnic Institute and State University, Blacksburg, VA.
- Suprenant, B. (2001), “Free Fall of Concrete, ASCC Position Statement No. 17,” *Concrete International*, ACI, Vol. 23, No. 6.

CHAPTER 11

SIDEPLATE MOMENT CONNECTION

- GSA (2008), “GSA Steel Frame Bomb Blast & Progressive Collapse Test Program (2004–2007) Summary Report,” January 10, prepared by MHP Structural Engineers for the U.S. General Services Administration (GSA), Office of the Chief Architect (OCA), Washington, DC.
- Hodgson, I.C., Tahmasebi, E. and Ricles, J.M. (2010a), “Cyclic Testing of Beam-to-Column Assembly Connected with SidePlate FRAME Special Moment Frame Connections—Test Specimens 1A, 2A, and 2B,” ATLSS Report No. 10-12, December, Center for Advanced Technology for Large Structural Systems (ATLSS), Lehigh University, Bethlehem, PA.
- Hodgson, I.C., Tahmasebi, E. and Ricles, J.M. (2010b), “Cyclic Testing of Beam-to-Column Assembly Connected with SidePlate Steel Moment Frame Connection—Test Specimen 2C,” ATLSS Report No. 10-13, December, Center for Advanced Technology for Large Structural Systems (ATLSS), Lehigh University, Bethlehem, PA.

- Hodgson, I.C., Tahmasebi, E. and Ricles, J.M. (2010c), “Cyclic Testing of Beam-to-Column Assembly Connected with SidePlate FRAME Special Moment Frame Connections—Test Specimens 1B and 3,” ATLSS Report No. 10-14, December, Center for Advanced Technology for Large Structural Systems (ATLSS), Lehigh University, Bethlehem, PA.
- ICC (2013a), Independent Pre-Qualification summarized in Evaluation Report by ICC Evaluation Service, Inc. (ICC-ES ESR-1275), “SidePlate Steel Frame Connection Technology,” issued May 1.
- ICC (2013b), Independent Pre-Qualification summarized in Research Report by Engineering Research Section, Department of Building and Safety, City of Los Angeles (COLA RR 25393), “GENERAL APPROVAL—SidePlate Steel Frame Connection Technology for Special Moment Frame (SMF) and Intermediate Moment Frame (IMF) Systems,” issued April 1.
- LACO (1997), Independent Evaluation and Acceptance Report by the Los Angeles County Technical Advisory Panel on Steel Moment Resisting Frame Connection Systems (LACO-TAP SMRF Bulletin No. 3, Chapter 2), “SidePlate Connection System,” dated March 4.
- Latham, C.T., Baumann, M.A. and Seible, F. (2004), “Laboratory Manual,” Structural Systems Research Project Report No. TR-97/09, May, Charles Lee Powell Structural Research Laboratories, University of California, San Diego, La Jolla, CA.
- Minh Huynh, Q. and Uang, C.M. (2012), “Cyclic Testing of SidePlate Steel Moment Frame for SMF Applications,” Structural Systems Research Project Report No. TR-12-02, October, Charles Lee Powell Structural Research Laboratories, University of California, San Diego, La Jolla, CA.
- Richards, P. and Uang, C.M. (2003), “Cyclic Testing of SidePlate Steel Frame Moment Connections for the Sharp Memorial Hospital,” Structural Systems Research Project Report No. TR-2003/02, March, Charles Lee Powell Structural Research Laboratories, University of California, San Diego, La Jolla, CA.
- Richards, P. and Uang, C.M. (2003), “Cyclic Testing of SidePlate Steel Frame Moment Connections for Children’s Hospital Los Angeles,” Structural Systems Research Project Report No. TR-2003/03, May, Charles Lee Powell Structural Research Laboratories, University of California, San Diego, La Jolla, CA.
- Trautner, J.J. (1995), “Three-Dimensional Non-Linear Finite-Element Analysis of MNH-SMRF™ Prototype Moment Connection,” System Reliability of Steel Connections Research Report No. 1, Department of Civil Engineering, University of Utah, Salt Lake City, UT.
- Uang, C.M. and Latham, C.T. (1995), “Cyclic Testing of Full-Scale MNH-SMR Moment Connections,” Structural Systems Research Project Report No. TR-95/01, March, Charles Lee Powell Structural Research Laboratories, University of California, San Diego, La Jolla, CA.

Uang, C.M., Bondad, D. and Noel, S. (1996), “Cyclic Testing of the MNH-SMR Dual Strong Axes Moment Connection with Cruciform Column,” Structural Systems Research Project Report No. TR-96/04, May, Charles Lee Powell Structural Research Laboratories, University of California, San Diego, La Jolla, CA.

CHAPTER 12

SIMPSON STRONG-TIE STRONG FRAME MOMENT CONNECTION

Geschwindner, L.F. and Disque, R.O. (2005), “Flexible Moment Connections for Unbraced Frames Subject to Lateral Forces – A Return to Simplicity,” *Engineering Journal*, AISC, Vol. 42, No. 2, 2nd Quarter, pp. 99–112.

Pryor, S.E. and Murray, T.M. (2013), “Next Generation Partial Strength Steel Moment Frames for Seismic Resistance,” *Research, Development, and Practice in Structural Engineering and Construction*, V. Vimonsatit, A. Singh and S. Yazdani, eds., *Proceedings of the First Australasia and South-East Asia Structural Engineering and Construction Conference*, Perth, Australia, November 28–December 2, pp. 27–32.

Pryor, S.E., van de Lindt, J.W. and Bahmani, P. (2014), “Full-Scale Testing of Soft-Story Wood-Frame Buildings,” *Structural Engineer Online*, <http://cenews.com/article/9732/full-scale-testing-of-soft-story-wood-frame-buildings>, February.

Rex, C.O. and Goverdhan, A.V. (2000), “Design and Behavior of a Real PR Building,” *Connections in Steel Structures IV; Behavior, Strength & Design*, R. Leon and W.S. Easterling, eds., *Proceedings of the Fourth Workshop on Connections in Steel Structures*, Roanoke, VA, October 22–24, pp. 94–105.

Van de Lindt, J.W., Pryor, S.E. and Pei, S. (2009), “Shake Table Testing of a Seven-Story Mixed-Use Condominium at Japan’s E-Defense,” *Proceedings, Annual Convention, Structural Engineers Association of California*, San Diego, CA, September 23–26.

CHAPTER 13

DOUBLE-TEE MOMENT CONNECTION

Larson, P.C. (1996), “The Design and Behavior of Bolted Beam-to-Column Frame Connections Under Cyclical Loading,” Master’s Thesis, University of Texas at Austin, Austin, TX.

Hantouche, E.G., Kukreti, A.R., Rassati, G.A. and Swanson, J.A. (2013), “Modified Stiffness Model for Thick Flange in Built-Up T-stub Connections,” *Journal of Constructional Steel Research*, Vol. 81, February, pp. 76–85.

Hantouche, E.G., Kukreti, A.R., Rassati, G.A. and Swanson, J.A. (2015), “Prying Models for Strength in Thick-Flange Built-Up T-Stubs with Complete Joint Penetration and Fillet Welds,” *Journal of Structural Engineering*, ASCE, Vol. 141, No. 2.

McManus, P. and Pucket, J.A. (2010), *Economic and Serviceable Seismic Systems: Phase I – Ductile WT Moment Frames*, Final Report for the American Institute of Steel Construction, June, University of Wyoming, Laramie, WY.

- Swanson, J.A. (1999), *Characterization of the Strength, Stiffness, and Ductility Behavior of T-stub Connections*, Ph.D. Thesis, Georgia Institute of Technology, Atlanta, GA.
- Swanson, J.A. (2002) “Ultimate Strength Prying Models for Bolted T-stub Connections,” *Engineering Journal*, AISC, Vol. 39, No. 3, 3rd Quarter, pp. 136–147.
- Swanson, J.A. (2016) “Strength of Beams in Beam-to-Column Connections with Holes in the Tension Flange,” *Engineering Journal*, AISC, Vol. 53, No. 3, 3rd Quarter, pp. 159–172.
- Swanson, J.A. and Leon, R.T. (2000), “Bolted Steel Connections: Tests on T-stub Components,” *Journal of Structural Engineering*, ASCE, Vol. 126, No. 1, pp. 50–56.
- Ulloa Barbaran, F.V. (1996), *Tension Bolt Behavior in Moment Connections for Seismic Applications*, Master’s Thesis, University of Texas at Austin, Austin, TX.

APPENDIX A

CASTING REQUIREMENTS

- Briggs, C.W. (1967), “The Evaluation of Discontinuities in Commercial Steel Castings by Dynamic Loading to Failure in Fatigue,” Steel Founders’ Society of America (SFSA), Rocky River, OH.
- Carlson, K.D., Lin, Z., Hardin, R.A., Beckermann, C., Mazurkevich, G. and Schneider, M.C. (2003), “Modeling of Porosity Formation and Feeding Flow in Steel Casting,” *Proceedings of Modeling of Casting, Welding and Advanced Solidification Processes X*, May 25–30, Destin, FL.
- Hardin R.A., Ou, S., Carlson, K. and Beckermann, C. (1999), “Relationship between Casting Simulation and Radiographic Testing: Results from the SFSA Plate Casting Trials,” *Proceedings of the 1999 SFSA Technical and Operating Conference*, November 4–6, Chicago, IL.
- Niyama, E., Nchida, T., Marikawa, M. and Shigeki, S. (1982), “A Method of Shrinkage Prediction and its Application to Steel Castings Practice,” paper presented at *49th International Foundry Congress*, Chicago, IL.