
Seismic Provisions for Structural Steel Buildings

PUBLIC REVIEW DRAFT dated **April 17, 2009**

Supersedes the *Seismic Provisions
for Structural Steel Buildings*
dated March 9, 2005,
Supplement No. 1 dated November 16, 2005,
and all previous versions

(Not yet) Approved by the
AISC Committee on Specifications and
issued by the AISC Board of Directors

DRAFT



AMERICAN INSTITUTE OF STEEL CONSTRUCTION

One East Wacker Drive, Suite 700
Chicago, Illinois 60601-1802

Copyright © 2010

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 in accordance with recognized engineering principles and is for general information only. 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 professional engineer, designer, or architect. The publication of the material contained herein is not intended as a representation or warranty on the part of the American Institute of Steel Construction 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. Anyone making use of this information assumes all liability arising from such use.

Caution must be exercised when relying upon other specifications and codes 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 Institute 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 a part of ANSI/AISC 341-10, *Seismic Provisions for Structural Steel Buildings*, but is included for informational purposes only.

The AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360-10) is intended to cover common design criteria. Accordingly, it is not feasible for it to also cover all of the special and unique problems encountered within the full range of structural design practice. This document, the AISC *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341-10) (hereafter referred to as the *Provisions*) is a separate consensus standard that addresses one such topic: the design and construction of structural steel and composite structural steel/reinforced concrete building systems for high-seismic applications.

A list of Symbols, and Glossary are part of this document. Terms that appear in the Glossary are generally italicized where they first appear in a sub-section, throughout these *Provisions*. A nonmandatory Commentary with background information is also provided.

The AISC Committee on Specifications, Task Committee 9—Seismic Provisions is responsible for the ongoing development of these *Provisions*. The AISC Committee on Specifications gives final approval of the document through an ANSI-accredited balloting process, and has enhanced these *Provisions* through careful scrutiny, discussion, and suggestions for improvement. AISC further acknowledges the significant contributions of several groups to the completion of this document: the Building Seismic Safety Council (BSSC), the SAC Joint Venture, the Federal Emergency Management Agency (FEMA), the National Science Foundation (NSF), and the Structural Engineers Association of California (SEAOC).

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

This specification was approved by the AISC Committee on Specifications:

James M. Fisher, Chairman
Edward E. Garvin, Vice Chairman
Hansraj G. Ashar

William F. Baker
John M. Barsom
William D. Bast
Reidar Bjorhovde
Roger L. Brockenbrough
Gregory G. Deierlein
Bruce R. Ellingwood
Michael D. Engelhardt
Shujin Fang
Steven J. Fennes
John W. Fisher
Theodore V. Galambos
Louis F. Geschwindner
Lawrence G. Griffis
John L. Gross
Jerome F. Hajjar
Patrick M. Hasset
Tony C. Hazel
Mark V. Holland
Ronald J. Janowiak
Richard C. Kaehler
Lawrence A. Kloiber
Lawrence F. Kruth
Jay W. Larson
Roberto T. Leon
James O. Malley
Sanjeev R. Malushte
David L. McKenzie
Duane K. Miller
Larry Muir
Thomas M. Murray
R. Shankar Nair
Jack E. Petersen
Douglas A. Rees-Evans
Thomas A. Sabol
Robert E. Shaw, Jr.
Donald R. Sherman
W. Lee Shoemaker
William A. Thornton
Raymond H.R. Tide
Chia-Ming Uang
Donald W. White
Cynthia J. Duncan, Secretary

The Committee gratefully acknowledges the following task committee (TC 9 – Seismic Design) for their development of this document.

James O. Malley, Chairman
C. Mark Saunders, Vice Chairman
Michel Bruneau
Gregory G. Deierlein
Richard M. Drake
Michael D. Engelhardt
Timothy P. Fraser
Subhash C. Goel
Jerome F. Hajjar
Ronald O. Hamburger
James R. Harris
Patrick M. Hassett
John D. Hooper
Brian T. Knight
Keith Landwehr
Roberto T. Leon
Sanjeev R. Malushte
Bonnie E. Manley
Clarkson W. Pinkham
John A. Rolfes
Rafael Sabelli
Thomas A. Sabol
Bahram M. Shahrooz
Robert E. Shaw, Jr.
W. Lee Shoemaker
Kurt D. Swensson
Robert Tremblay
Jamie Winans
Cynthia J. Duncan, Secretary

TABLE OF CONTENTS

SYMBOLS

GLOSSARY

ACRONYMS

PROVISIONS

A. GENERAL REQUIREMENTS

- A1. Scope
- A2. Referenced Specifications, Codes and Standards
- A3. Materials
 - 1. Material Specifications
 - 2. Expected Material Strength
 - 3. Heavy Sections
 - 4. Consumables for Welding
 - 4a. Seismic Force Resisting System Welds
 - 4b. Demand Critical Welds
 - 5. Concrete and Steel Reinforcement
- A4. Drawings and Specifications
 - 1. General
 - 2. Steel Construction
 - 3. Composite Construction

B. GENERAL DESIGN REQUIREMENTS

- B1. General Seismic Design Requirements
- B2. Loads and Load Combinations
- B3. Design Basis
 - 1. Required Strength
 - 2. Available Strength
- B4. System Type

C. ANALYSIS

- C1. General
- C2. Additional Requirements
- C3. Nonlinear Analysis

D. GENERAL MEMBER AND CONNECTION DESIGN REQUIREMENTS

- D1. Member Requirements
 - 1. Classification of Sections for Ductility
 - 1a. Section Requirements for Ductile Members
 - 1b. Width-to-Thickness Limitations of Steel Sections
 - 2. Stability Bracing of Members
 - 2a. Moderately Ductile Members

- 2b. Highly Ductile Members
- 2c. Special Bracing at Plastic Hinge Locations

- 3. Protected Zones
- 4. Columns
 - 4a. Required Strength
 - 4b. Encased Composite Columns
 - 4c. Filled Composite Columns
- 5. Composite Slab Diaphragms
 - 5a. Load Transfer
 - 5b. Nominal Shear Strength
- D2. Connections
 - 1. General
 - 2. Bolted Joints
 - 3. Welded Joints
 - 4. Continuity Plates & Stiffeners
 - 5. Column Splices
 - 5a. Location of Splices
 - 5b. Required Strength
 - 5c. Required Shear Strength
 - 5d. Structural Steel Splice Configurations
 - 5e. Splices in Encased Composite Columns
 - 6. Column Bases
 - 6a. Required Axial Strength
 - 6b. Required Shear Strength
 - 6c. Required Flexural Strength
 - 7. Composite Connections
 - 8. Steel Anchors
- D3. Deformation Compatibility of Non-SFRS Members and Connections
- D4. H-Piles
 - 1. Design Requirements
 - 2. Battered H-Piles
 - 3. Tension
 - 4. Protected Zone

- E. MOMENT-FRAME SYSTEMS**
- E1. Ordinary Moment Frames
 - 1. Scope
 - 2. Basis of Design
 - 3. Analysis
 - 4. System Requirements
 - 5. Members
 - 6. Connections
 - 6a. Demand Critical Welds
 - 6b. Requirements for FR Moment Connections

- 6c. Requirements for PR Moment Connections
- E2. Intermediate Moment Frames
 - 1. Scope
 - 2. Basis of Design
 - 3. Analysis
 - 4. System Requirements
 - 4a. Stability Bracing of Beams
 - 5. Members
 - 5a. Width-Thickness Limitations
 - 5b. Beam Flanges
 - 5c. Protected Zones
 - 5d. Beams
 - 6. Connections
 - 6a. Demand Critical Welds
 - 6b. Beam-to-Column Connection Requirements
 - 6c. Conformance Demonstration
 - 6d. Required Shear Strength
 - 6e. Continuity Plates
 - 6f. Column Splices
- E3. Special Moment Frames
 - 1. Scope
 - 2. Basis of Design
 - 3. Analysis
 - 4. System Requirements
 - 4a. Moment Ratio
 - 4b. Stability Bracing of Beams
 - 4c. Stability Bracing at Beam-to-Column Connections
 - 5. Members
 - 5a. Width-Thickness Limitations
 - 5b. Beam Flanges
 - 5c. Protected Zones
 - 5d. Beams
 - 6. Connections
 - 6a. Demand Critical Welds
 - 6b. Beam-to-Column Connection Requirements
 - 6c. Conformance Demonstration
 - 6d. Required Shear Strength
 - 6e. Panel Zone
 - 6f. Continuity Plates
 - 6g. Column Splices
- E4. Special Truss Moment Frames
 - 1. Scope
 - 2. Basis of Design
 - 3. Analysis
 - 3a. Special Segment
 - 3b. Non-Special Segment

- 4. System Requirements
 - 4a. Special Segment
 - 4b. Stability Bracing of Trusses
 - 4c. Stability Bracing of Truss-to-Column Connections
 - 5. Members
 - 5a. Special Segment Members
 - 5b. Expected Vertical Shear Strength of Special Segment
 - 5c. Width-Thickness Limitations
 - 5d. Built-Up Chord Members
 - 5e. Protected Zones
 - 6. Connections
 - 6a. Connections of Diagonal Web Members in the Special Segment
 - 6b. Column Splices
- E5. Ordinary Cantilever Column Systems
- 1. Scope
 - 2. Basis of Design
 - 3. Analysis
 - 4. System Requirements
 - 4a. Columns
 - 4b. Stability Bracing of Columns
 - 5. Members
 - 5a. Width-Thickness Limitations
 - 5b. Column Flanges
 - 5c. Protected Zones
 - 6. Connections
 - 6a. Column Bases
- E6. Special Cantilever Column Systems
- 1. Scope
 - 2. Basis of Design
 - 3. Analysis
 - 4. System Requirements
 - 4a. Columns
 - 4b. Stability Bracing of Columns
 - 5. Members
 - 5a. Width-Thickness Limitations
 - 5b. Column Flanges
 - 5c. Protected Zones
 - 6. Connections
 - 6a. Column Bases
- F. BRACED-FRAME AND SHEAR-WALL SYSTEMS**
- F1. Ordinary Concentrically Braced Frames
- 1. Scope

- 2. Basis of Design
- 3. Analysis
- 4. System Requirements
 - 4a. V-Braced and Inverted-V-Braced Frames
 - 4b. K-Braced Frames
- 5. Members
 - 5a. Width-Thickness Limitations
 - 5b. Slenderness
- 6. Connections
 - 6a. Diagonal Brace Connections
- 7. Ordinary Concentrically Braced Frames above Seismic Isolation Systems
 - 7a. System Requirements
 - 7b. Members
- F2. Special Concentrically Braced Frames
 - 1. Scope
 - 2. Basis of Design
 - 3. Analysis
 - 4. System Requirements
 - 4a. Lateral Force Distribution
 - 4b. V- and Inverted V-Braced Frames
 - 4c. K-Braced Frames
 - 4d. Tension-Only Frames
 - 5. Members
 - 5a. Width-Thickness Limitations
 - 5b. Diagonal Braces
 - 5c. Protected Zones
 - 6. Connections
 - 6a. Required Strength of Brace Connections
 - 6b. Beam-to-Column Connections
 - 6c. Column Splices
 - 6d. Demand Critical Welds
- F3. Eccentrically Braced Frames
 - 1. Scope
 - 2. Basis of Design
 - 3. Analysis
 - 4. System Requirements
 - 4a. Link Rotation Angle
 - 4b. Bracing of Link
 - 5. Members
 - 5a. Links
 - 5b. Diagonal Braces
 - 5c. Beam Outside Link
 - 5d. Columns
 - 5e. Protected Zones

- 6. Connections
 - 6a. Link-to-Column Connections
 - 6b. Diagonal Brace Connections
 - 6c. Beam-to-Column Connections
 - 6d. Column Splices
 - 6e. Demand Critical Welds
- F4. Buckling-Restrained Braced Frames
 - 1. Scope
 - 2. Basis of Design
 - 2a. Brace Strength
 - 3. Analysis
 - 4. System Requirements
 - 4a. V- and Inverted V-Braced Frames
 - 4b. K-Braced Frames
 - 4c. Bracing Connections at Beam-to-Column Connections
 - 5. Members
 - 5a. Diagonal Braces
 - 5b. Width-Thickness Limitations of Beams and Columns
 - 5c. Protected Zones
 - 6. Connections
 - 6a. Diagonal Brace Connections
 - 6b. Column Splices
 - 6c. Demand Critical Welds
 - 6d. Beam-to-Column Connections
- F5. Special Plate Shear Walls
 - 1. Scope
 - 2. Basis of Design
 - 3. Analysis
 - 4. System Requirements
 - 4a. Stiffness of Boundary Elements
 - 4b. HBE-to-VBE Connection Moment Ratio
 - 4c. Bracing
 - 4d. Openings in Webs
 - 5. Members
 - 5a. Webs
 - 5b. Width-Thickness Limitations
 - 5c. Protected Zone
 - 6. Connections
 - 6a. Connections of Webs to Boundary Elements
 - 6b. HBE-to-VBE Connections
 - 6c. Column Splices
 - 7. Perforated Webs
 - 7a. Regular Layout of Circular Perforations
 - 7b. Reinforced Corner Cut-Out

- G. COMPOSITE MOMENT FRAME SYSTEMS**
- G1. Composite Ordinary Moment Frames
 - 1. Scope
 - 2. Basis of Design
 - 3. Analysis
 - 4. System Requirements
 - 5. Members
 - 6. Connections
 - G2. Composite Intermediate Moment Frames
 - 1. Scope
 - 2. Basis of Design
 - 3. Analysis
 - 4. System Requirements
 - 4a. Stability Bracing of Beams
 - 5. Members
 - 5a. Detailing Requirements
 - 5b. Columns
 - 5c. Beam Flanges
 - 5d. Protected Zones
 - 6. Connections
 - 6a. Demand Critical Welds
 - 6b. Beam-to-Column Connection Requirements
 - 6c. Required Shear Strength
 - 6d. Continuity Plates and Diaphragms
 - 6e. Column Splices
 - G3. Composite Special Moment Frames
 - 1. Scope
 - 2. Basis of Design
 - 3. Analysis
 - 4. System Requirements
 - 4a. Moment Ratio
 - 4b. Stability Bracing of Beams
 - 4c. Stability Bracing at Beam-to-Column Connections
 - 5. Members
 - 5a. Detailing Requirements
 - 5b. Columns
 - 5c. Beam Flanges
 - 5d. Protected Zones
 - 6. Connections
 - 6a. Demand Critical Welds
 - 6b. Beam-to-Composite Column Connection Requirements
 - 6c. Required Shear Strength
 - 6d. Connection Diaphragm Plates
 - 6e. Column Splices
 - G4. Composite Partially-Restrained Moment Frames
 - 1. Scope

- 2. Basis of Design
- 3. Analysis
- 4. System Requirements
- 5. Members
- 5a. Columns
- 5b. Beams
- 6. Connections
- 6a. Required Strength
- 6b. Connection Rotational Capacity
- 6c. Connection Strength
- 6d. Conformance Demonstration
- 6e. Column Splices
- H. COMPOSITE BRACED-FRAME AND SHEAR-WALL SYSTEMS**
- H1. Composite Ordinary Braced Frames
 - 1. Scope
 - 2. Basis of Design
 - 3. Analysis
 - 4. System Requirements
 - 5. Members
 - 5a. Detailing Requirements
 - 5b. Columns
 - 5c. Braces
 - 6. Connections
- H2. Composite Special Concentrically Braced Frames
 - 1. Scope
 - 2. Basis of Design
 - 3. Analysis
 - 4. System Requirements
 - 5. Members
 - 5a. Detailing Requirements
 - 5b. Columns
 - 5c. Braces
 - 6. Connections
 - 6a. Required Strength of Brace Connections
 - 6b. Beam-to-Column Connections
 - 6c. Column Splices
- H3. Composite Eccentrically Braced Frames
 - 1. Scope
 - 2. Basis of Design
 - 3. Analysis
 - 4. System Requirements
 - 5. Members
 - 5a. Links
 - 5b. Columns
 - 6. Connections

- 6a. Beam-to-Column Connections
- H4. Composite Ordinary Reinforced Concrete Shear Walls with Steel Elements
 - 1. Scope
 - 2. Basis of Design
 - 3. Analysis
 - 4. System Requirements
 - 5. Members
 - 5a. Links
 - 5b. Columns
 - 6. Connections
 - 6a. Beam-to-Column Connections
- H5. Composite Special Reinforced Concrete Shear Walls with Steel Elements
 - 1. Scope
 - 2. Basis of Design
 - 3. Analysis
 - 4. System Requirements
 - 5. Members
 - 5a. Ductile Elements
 - 5b. Boundary Members
 - 5c. Steel Coupling Beams
 - 5d. Composite Coupling Beams
 - 6. Connections
- H6. Composite Steel Plate Shear Walls
 - 1. Scope
 - 2. Basis of Design
 - 3. Analysis
 - 3a. Webs
 - 3b. Other Members and Connections
 - 4. System Requirements
 - 4a. Steel Plate Thickness
 - 4b. Stiffness of Vertical Boundary Elements
 - 4c. HBE-to-VBE Connection Moment Ratio
 - 4d. Lateral Bracing
 - 4e. Openings in Webs
 - 4f. Detailing Requirements
 - 5. Members
 - 5a. Webs
 - 5b. Concrete Stiffening Elements
 - 5c. Boundary Members
 - 5d. Detailing Requirements
 - 6. Connections
 - 6a. Demand Critical Welds
 - 6b. Connections of Steel Plate to Boundary Elements
 - 6c. HBE-to-VBE Connections

6d. Connections of Steel Plate to Reinforced Concrete Panel

I. FABRICATION, ERECTION, QUALITY CONTROL AND QUALITY ASSURANCE

- I1. Shop and Erection Drawings
 - 1. Shop Drawings for Steel Construction
 - 2. Erection Drawings for Steel Construction
 - 3. Shop and Erection Drawings for Composite Construction
- I2. Fabrication and Erection
 - 1. Protected Zone
 - 2. Bolted Joints
 - 3. Welded Joints
 - 4. Continuity Plates and Stiffeners
 - 5. Weld Tabs
- I3. Quality Control and Quality Assurance
 - 1. Inspection and Nondestructive Testing Personnel
 - 2. Contractor Documents
 - 2a. Documents to be Submitted for Steel Construction
 - 2b. Documents to be Available for Review for Steel Construction
 - 2c. Documents to be Submitted for Composite Construction
 - 3. Quality Assurance Agency Documents
 - 4. Inspection Points and Frequencies
 - 4a. Observe (O)
 - 4b. Perform (P)
 - 4c. Document (D)
 - 5. Welding Inspection
 - 5a. Visual Welding Inspection
 - 5b. Nondestructive Testing (NDT) of Welds
 - 6. Inspection of Bolting
 - 7. Other Steel Structure Inspections
 - 8. Inspection of Composite Structures
 - 9. Inspection of Piling

J. QUALIFICATIONS AND PREQUALIFICATION TESTING PROVISIONS

- J1. Prequalification of Beam-to-Column and Link-to-Column Connections
 - 1. Scope
 - 2. General Requirements
 - 2a. Basis of Prequalification
 - 2b. Authority for Prequalification
 - 3. Testing Requirements
 - 4. Prequalification Variables

- 4a. Beam or Link Parameters
- 4b. Column Parameters
- 4c. Beam (or Link)-Column Relations
- 4d. Continuity Plates
- 4e. Welds
- 4f. Bolts
- 4g. Workmanship
- 4h. Additional Connection Details
- 5. Design Procedure
- 6. Prequalification Record
- J2. Cyclic Tests for Qualification of Beam-to-Column and Link-to-Column Connections
 - 1. Scope
 - 2. Test Subassembly Requirements
 - 3. Essential Test Variables
 - 3a. Source of Inelastic Rotation
 - 3b. Size of Members
 - 3c. Connection Details
 - 3d. Continuity Plates
 - 3e. Steel Strength
 - 3f. Welds
 - 3g. Bolts
 - 4. Loading History
 - 4a. General Requirements
 - 4b. Loading Sequence for Beam-to-Column Moment Connections
 - 4c. Loading Sequence for Link-to-Column Connections
 - 5. Instrumentation
 - 6. Materials Testing Requirements
 - 6a. Tension Testing Requirements for Structural Steel
 - 6b. Methods of Tension Testing for Structural Steel
 - 6c. Weld Metal Testing Requirements
 - 7. Test Reporting Requirements
 - 8. Acceptance Criteria
- J3. Qualifying Cyclic Tests of Buckling-Restrained Braces
 - 1. Scope
 - 2. Subassembly Test Specimen
 - 3. Brace Test Specimen
 - 3a. Design of Brace Test Specimen
 - 3b. Manufacture of Brace Test Specimen
 - 3c. Similarity of Brace Test Specimen and Prototype
 - 3d. Connection Details
 - 3e. Materials
 - 3f. Connections
 - 4. Loading History
 - 4a. General Requirements

- 4b. Test Control
- 4c. Loading Sequence
- 5. Instrumentation
- 6. Materials Testing Requirements
- 6a. Tension Testing Requirements
- 6b. Methods of Tension Testing
- 7. Test Reporting Requirements
- 8. Acceptance Criteria

COMMENTARY

DRAFT

SYMBOLS

The symbols listed below shall be used in addition to or replacements for those in the AISC *Specification for Structural Steel Buildings*. Where there is a duplication of the use of a symbol between this Specification and the AISC *Specification for Structural Steel Buildings*, the symbol listed herein shall take precedence. The section or table number in the right-hand column refers to where the symbol is first used.

<u>Symbol</u>	<u>Definition</u>	<u>Reference</u> (to be provided in final)
A_b	Cross-sectional area of a horizontal boundary element (HBE), in. ² (mm ²)	
A_c	Cross-sectional area of a vertical boundary element (VBE), in. ² (mm ²)	
A_f	Flange area, in. ² (mm ²)	
A_g	Gross area, in. ² (mm ²)	
A_{lw}	Link web area (excluding flanges), in. ² (mm ²)	
A_s	Cross-sectional area of the structural steel core, in. ² (mm ²)	
A_s	Area of transfer reinforcement in coupling beam, in. ² (mm ²)	
A_{sc}	Area of the yielding segment of steel core, in. ² (mm ²)	
A_{sh}	Minimum area of tie reinforcement, in. ² (mm ²)	
A_{sp}	Horizontal area of stiffened steel plate in composite steel plate shear wall, in. ² (mm ²)	
A_{st}	Horizontal cross-sectional area of <i>link</i> stiffener, in. ² (mm ²)	
A_{tb}	Area of transfer reinforcement in coupling beam, in. ² (mm ²)	
C_a	Ratio of required strength to available strength	
C_d	Coefficient relating relative brace stiffness and curvature	
D	Dead load due to the weight of the structural elements and permanent features on the building, kips (N)	
D	Outside diameter of round HSS, in. (mm)	
E	Earthquake load	
E	Modulus of elasticity of steel, $E = 29,000$ ksi (200 000 MPa)	
F_y	Specified minimum yield stress of the type of steel to be used, ksi (MPa). As used in the <i>Specification</i> , "yield stress" denotes either the minimum specified yield point (for those steels that have a yield point) or the specified yield strength (for those steels that do not have a yield point).	

F_{yb}	F_y of a beam, ksi (MPa)
F_{yc}	F_y of a column, ksi (MPa)
F_{yh}	Specified minimum yield stress of the ties, ksi (MPa)
F_{ysc}	Specified minimum yield stress of the steel core, or actual yield stress of the steel core as determined from a coupon test, ksi (MPa)
F_u	Specified minimum tensile strength, ksi (MPa)
H	Height of story, which may be taken as the distance between the centerline of floor framing at each of the levels above and below, or the distance between the top of floor slabs at each of the levels above and below, in. (mm)
H_c	Clear height of story, which may be taken as the distance between the bottom of the flange of the beam above and either the top of the flange of the beam below, or the top of the structural slab, if present, in. (mm)
I	Moment of inertia, in. ⁴ (mm ⁴)
I_c	Moment of inertia of a vertical boundary element (VBE) taken perpendicular to the direction of the web plate line, in. ⁴ (mm ⁴)
K	Effective length factor for prismatic member
L	Live load due to occupancy and moveable equipment, kips (kN)
	L Span length of the truss, in. (mm)
	L Distance between VBE centerlines, in. (mm)
L_b	Length between points which are either braced against lateral displacement of compression flange or braced against twist of the cross section, in. (mm).....
L_{cf}	Clear distance between column flanges, in. (mm)
L_e	Embedment length of coupling beam, in. (mm).....
L_h	Distance between plastic hinge locations, in. (mm)
L_p	Limiting laterally unbraced length for full plastic flexural strength, uniform moment case, in. (mm)
L_{pd}	Limiting laterally unbraced length for plastic analysis, in. (mm)
L_s	Length of the <i>special segment</i> , in. (mm)
M_a	Required flexural strength, using ASD load combinations, kip-in. (N-mm)
M_{av}	Additional moment due to shear amplification from the location of the plastic hinge to the column centerline based on ASD load combinations, kip-in. (N-mm).....
M_n	Nominal flexural strength, kip-in. (N-mm)
M_{nc}	Nominal flexural strength of the chord member of the special segment, kip-in. (N-mm)
$M_{n,PR}$	Nominal flexural strength of PR connection at a rotation of 0.02 rad, kip-in. (N-mm)
M_p	Nominal plastic flexural strength, kip-in. (N-mm)

M_{pa}	Nominal plastic flexural strength modified by axial load, kip-in. (N-mm)
M_{pb}	Nominal plastic flexural strength of the beam, kip-in. (N-mm)
M_{pc}	Nominal plastic flexural strength of the column, kip-in. (N-mm)
M_{pcc}^*	Nominal flexural strength of composite or reinforced concrete column with consideration of the required axial strength, kip-in. (N-mm)
$M_{p,exp}$	Expected plastic moment, kip-in. (N-mm)
$M_{p,exp}^*$	Expected flexural strength at the intersection of the beam and column centerlines, kip-in. (N-mm)
M_r	Expected flexural strength, kip-in. (N-mm)
M_{uv}	Additional moment due to shear amplification from the location of the plastic hinge to the column centerline based on LRFD load combinations, kip-in. (N-mm)
M_u	Required flexural strength, using LRFD load combinations, kip-in. (N-mm)
$M_{u,exp}$	Expected required flexural strength, kip-in. (N-mm)
M_{uv}	Moment due to shear amplification from the location of the plastic hinge to the column centerline, kip-in. (N-mm)
P_a	Required axial strength of a column using ASD load combinations, kips (N)
P_{ac}	Required compressive strength of columns using ASD load combinations, kips (N)
P_b	Required strength of lateral brace at ends of the link, kips (N)
P_b	Axial design strength of wall at balanced condition, kips (N) ..
P_c	Available axial strength of a column, kips (N)
P_n	Nominal axial strength of a column, kips (N)
P_n	Nominal compressive strength of the composite column calculated in accordance with the <i>Specification</i> , kips (N)
P_{nc}	Nominal axial compressive strength of diagonal members of the special segment, kips (N)
P_{nt}	Nominal axial tensile strength of diagonal members of the special segment, kips (N)
P_o	Nominal axial strength of a composite column at zero eccentricity, kips (N)
P_r	Required compressive strength, kips (N)
P_{rc}	Required compressive strength of columns using ASD or LRFD load combinations, kips (N)
P_u	Required axial strength using LRFD load combinations, kips (N)
P_u	Required axial strength of a composite column, kips (N)
P_{uc}	Required compressive strength of columns using LRFD load combinations, kips (N)
P_y	Nominal axial yield strength of a member, equal to $F_y A_g$, kips

	(N)
P_{ysc}	Axial yield strength of steel core, kips (N)
Q_1	Axial forces and moments generated by at least 1.25 times the expected nominal shear strength of the link
R	Seismic response modification coefficient
R_n	Nominal strength, kips (N)
R_t	Ratio of the expected tensile strength to the specified minimum tensile strength F_u , as related to overstrength in material yield stress R_y
R_u	Required strength
R_v	Panel zone nominal shear strength
R_y	Ratio of the expected yield stress to the specified minimum yield stress, F_y
S	Snow load, kips (N)
V_a	Required shear strength using ASD load combinations, kips (N)
V_{comp}	Nominal shear strength of composite coupling beam, kips (N)..
V_n	Nominal shear strength of a member, kips (N)
$V_{n,comp}$	Nominal shear strength of composite coupling beam, kips (N)
V_{ne}	Expected vertical shear strength of the special segment, kips (N)
V_{ns}	Nominal shear strength of the steel plate in a composite plate shear walls, kips (N)
V_p	Nominal shear strength of an active link, kips (N)
V_{pa}	Nominal shear strength of an active link modified by the axial load magnitude, kips (N)
V_u	Required shear strength using LRFD load combinations, kips (N)
Y_{con}	Distance from top of steel beam to top of concrete slab or encasement, in. (mm)
Y_{PNA}	Maximum distance from the maximum concrete compression fiber to the plastic neutral axis, in. (mm)
Z	Plastic section modulus of a member, in. ³ (mm ³)
Z_b	Plastic section modulus of the beam, in. ³ (mm ³)
Z_c	Plastic section modulus of the column, in. ³ (mm ³)
Z_x	Plastic section modulus x-axis, in. ³ (mm ³)
Z_{RBS}	Minimum plastic section modulus at the reduced beam section, in. ³ (mm ³)
b	Width of compression element as defined in <i>Specification</i> Section B4.1, in. (mm)
b_{cf}	Width of column flange, in. (mm)
b_f	Flange width, in. (mm)
b_w	Width of the concrete cross-section minus the width of the structural shape measured perpendicular to the direction of shear, in. (mm)

b_w	Wall width, in. (mm)
d	Nominal fastener diameter, in. (mm)
d	Overall beam depth, in. (mm)
d_c	Overall column depth, in. (mm)
d_z	Overall panel zone depth between continuity plates, in. (mm)
e	EBF link length, in. (mm)
f'_c	Specified compressive strength of concrete, ksi (MPa)
h	Clear distance between flanges less the fillet or corner radius for rolled shapes; and for built-up sections, the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used; for tees, the overall depth; and for rectangular HSS, the clear distance between the flanges less the inside corner radius on each side, in. (mm)
h	Distance between horizontal boundary element centerlines, in. (mm)
h_{cc}	Cross-sectional dimension of the confined core region in composite columns measured center-to-center of the transverse reinforcement, in. (mm)
h_o	Distance between flange centroids, in. (mm)
l	Unbraced length between stitches of built-up braces, in. (mm)
l	Unbraced length of compression or bracing member, in. (mm)
r	Governing radius of gyration, in. (mm)
r_y	Radius of gyration about y axis, in. (mm)
s	Spacing of transverse reinforcement measured along the longitudinal axis of the structural composite member, in. (mm)
t	Thickness of connected part, in. (mm)
t	Thickness of element, in. (mm)
t	Thickness of column web or doubler plate, in. (mm)
t_{bf}	Thickness of beam flange, in. (mm)
t_{cf}	Thickness of column flange, in. (mm)
t_f	Thickness of flange, in. (mm)
t_p	Thickness of panel zone including doubler plates, in. (mm)
t_w	Thickness of web, in. (mm)
w_z	Width of panel zone between column flanges, in. (mm)
ΣM^*_{pc}	Moment at beam and column centerline determined by projecting the sum of the nominal column plastic moment strength, reduced by the axial stress P_{uc}/A_g , from the top and bottom of the beam moment connection
ΣM^*_{pb}	Moment at the intersection of the beam and column centerlines determined by projecting the beam maximum developed moments from the column face. Maximum developed moments shall be determined from test results
ΣM^*_{pcc}	Sum of the nominal flexural strengths of composite columns above and below the joint at the intersection of the beam and column centerlines

$\Sigma M_{p,exp}^*$	Sum of the expected flexural strengths of steel beams or concrete-encased composite beams at the intersection of the beam and column centerlines
β	Compression strength adjustment factor
Δ	Design story drift
Δ_b	Deformation quantity used to control loading of test specimen (total brace end rotation for the subassembly test specimen; total brace axial deformation for the brace test specimen)
Δ_{bm}	Value of deformation quantity, Δ_b , corresponding to the design story drift
Δ_{by}	Value of deformation quantity, Δ_b , at first significant yield of test specimen
Ω	Safety factor
Ω_b	Safety factor for flexure = 1.67
Ω_c	Safety factor for compression = 1.67
Ω_o	Horizontal seismic overstrength factor
Ω_v	Safety factor for shear strength of panel zone of beam-to-column connections
α	Angle of diagonal members with the horizontal
α	Angle of web yielding in radians, as measured relative to the vertical
δ	Deformation quantity used to control loading of test specimen
δ_y	Value of deformation quantity δ at first significant yield of test specimen
ρ'	Ratio of stress due to required axial strength (P_r/A_g) to required stress due to required shear strength (V_r/A_w) of a link
$\lambda_{hd}, \lambda_{md}$	Limiting slenderness parameter for highly and moderately ductile compression elements, respectively
ϕ	Resistance factor
ϕ_b	Resistance factor for flexure
ϕ_c	Resistance factor for compression
ϕ_v	Resistance factor for shear
θ	Interstory drift angle, radians
γ_{total}	Link rotation angle
ω	Strain hardening adjustment factor

Glossary

The terms listed below shall be used in addition to those in the AISC *Specification for Structural Steel Buildings*. Some commonly used terms are repeated here for convenience. Glossary terms are generally *italicized* throughout these *Provisions* and *Commentary*, where they first appear within a section.

Notes:

- (1) Terms designated with † are common AISI-AISC terms that are coordinated between the two standards developers.
- (2) Terms designated with * are usually qualified by the type of *load effect*, for example, *nominal tensile strength*, *available compressive strength*, *design flexural strength*.
- (3) Terms designated with ** are usually qualified by the type of component, for example, *web local buckling*, *flange local bending*.

Adjusted brace strength. Strength of a brace in a *buckling-restrained braced frame* at deformations corresponding to 2.0 times the *design story drift*.

*Allowable strength**†. Nominal strength divided by the safety factor, R_n / Ω .

Applicable building code†. Building code under which the structure is designed.

Amplified seismic load. Horizontal component of earthquake load E multiplied by Ω_o , where E and the horizontal component of E are specified in the *applicable building code*.

Authority having jurisdiction. Organization, political subdivision, office or individual charged with the responsibility of administering and enforcing the provisions of this Standard.

*Available strength**†. *Design strength* or *allowable strength*, as appropriate.

ASD (Allowable Strength Design)†. Method of proportioning structural components such that the *allowable strength* equals or exceeds the *required strength* of the component under the action of the ASD load combinations.

ASD load combination†. Load combination in the *applicable building code* intended for *allowable strength design* (allowable stress design).

Boundary member. Portion along wall or diaphragm edge strengthened with structural steel sections and/or longitudinal steel reinforcement and transverse reinforcement.

Braced frame†. An essentially vertical truss system that provides resistance to lateral forces and provides stability for the structural system.

Brace test specimen. A single buckling-restrained brace element used for laboratory testing intended to model the brace in the *prototype*.

Buckling-restrained brace. A pre-fabricated, or manufactured, brace element consisting of a steel core and a buckling-restraining system as described in Section F4 and qualified by testing as required in Section J3.

- Buckling-restrained braced frame (BRBF)*. A diagonally braced frame employing buckling-restrained braces and meeting the requirements of Section F4.
- Buckling-restraining system*. System of restraints that limits buckling of the steel core in BRBF. This system includes the casing surrounding the steel core and structural elements adjoining its connections. The buckling-restraining system is intended to permit the transverse expansion and longitudinal contraction of the steel core for deformations corresponding to 2.0 times the *design story drift*.
- Cantilever column system (CCS)*. A seismic force resisting system in which the seismic forces are resisted by one or more columns that are cantilevered from the foundation, or from the diaphragm level below.
- Casing*. Element that resists forces transverse to the axis of the diagonal brace thereby restraining buckling of the core. The casing requires a means of delivering this force to the remainder of the buckling-restraining system. The casing resists little or no force along the axis of the diagonal brace.
- Collector*. Also known as drag strut, member that serves to transfer loads between floor diaphragms and the members of the *seismic force resisting system*.
- Column base*. Assemblage of structural shapes, plates, connectors, bolts and rods at the base of a column used to transmit forces between the steel superstructure and the foundation.
- Complete loading cycle*. A cycle of rotation taken from zero force to zero force, including one positive and one negative peak.
- Composite beam*. Structural steel beam in contact with and acting compositely with a reinforced concrete slab designed to act compositely for seismic forces.
- Composite brace*. Reinforced-concrete-encased structural steel section (rolled or built-up) or concrete-filled steel section used as a diagonal brace.
- Composite column*. Reinforced-concrete-encased structural steel section (rolled or built-up) or concrete-filled steel section used as a column.
- Composite eccentrically braced frame (C-EBF)*. Composite braced frame meeting the requirements of Section H3.
- Composite intermediate moment frame (C-IMF)*. Composite moment frame meeting the requirements of Section G2.
- Composite ordinary braced frame (C-OBF)*. Composite braced frame meeting the requirements of Section H1.
- Composite ordinary moment frame (C-OMF)*. Composite moment frame meeting the requirements of Section G1.
- Composite ordinary reinforced concrete shear wall with steel elements (C-ORCW)*. Composite shear walls meeting the requirements of Section H4.
- Composite partially-restrained moment frame (C-PRMF)*. Composite moment frame meeting the requirements of Section G4.

- Composite shear wall.* Reinforced concrete wall that has unencased or reinforced-concrete-encased structural steel sections as *boundary members*.
- Composite slab.* Reinforced concrete slab supported on and bonded to a formed steel deck that acts as a diaphragm to transfer load to and between elements of the *seismic force resisting system*.
- Composite special concentrically braced frame (C-SCBF).* Composite braced frame meeting the requirements of Section H2.
- Composite special moment frame (C-SMF).* Composite moment frame meeting the requirements of Section G3.
- Composite special reinforced concrete shear wall with steel elements (C-SRCW).* Composite shear walls meeting the requirements of Section H5.
- Composite steel plate shear wall (C-SPW).* Wall consisting of steel plate with reinforced concrete encasement on one or both sides that provides out-of-plane stiffening to prevent buckling of the steel plate and meeting the requirements of Section H6.
- Continuity plates.* Column stiffeners at the top and bottom of the *panel zone*; also known as transverse stiffeners.
- Contractor.* Fabricator or erector, as applicable.
- Coupling beam.* Structural steel or composite beam connecting adjacent reinforced concrete wall elements so that they act together to resist lateral loads.
- Demand critical weld.* Weld so designated by these *Provisions*.
- Design earthquake.* The earthquake represented by the *design response spectrum* as specified in the *applicable building code*.
- Design methodology.* A set of step-by-step procedures, based on calculation or experiment, used to determine sizes, lengths, and details in the design of buckling-restrained braces and their connections.
- Design story drift.* Calculated story drift, including the effect of expected inelastic action, due to design level earthquake forces as determined by the *applicable building code*.
- Design strength*†.* *Resistance factor* multiplied by the *nominal strength*, ϕR_n .
- Diagonal brace.* Inclined structural member carrying primarily axial force in a braced frame.
- Dual system.* Structural system with the following features: (1) an essentially complete space frame that provides support for gravity loads; (2) resistance to lateral load provided by moment frames (SMF, IMF or OMF) that are capable of resisting at least 25% of the base shear, and concrete or steel shear walls, or steel braced frames (EBF, SCBF or OCBF); and (3) each system designed to resist the total lateral load in proportion to its relative rigidity.
- Eccentrically braced frame (EBF).* Diagonally braced frame meeting the requirements of Section F3 that has at least one end of each diagonal brace

connected to a beam with a defined eccentricity from another beam-to-brace connection or a beam-to-column connection.

Encased composite beam. Composite beam completely enclosed in reinforced concrete.

Encased composite column. Structural steel column completely encased in reinforced concrete.

Engineer of record. Licensed professional responsible for sealing the contract documents.

Exempted column. Column not meeting the requirements of Equation E3-1 for SMF.

Expected yield strength. Yield strength in tension of a member, equal to the expected yield stress multiplied by A_g .

*Expected tensile strength *.* Tensile strength of a member, equal to the specified minimum tensile strength, F_u , multiplied by R_t .

Expected yield stress. Yield stress of the material, equal to the specified minimum yield stress, F_y , multiplied by R_y .

Face bearing plates. Stiffeners attached to structural steel beams that are embedded in reinforced concrete walls or columns. The plates are located at the face of the reinforced concrete to provide confinement and to transfer loads to the concrete through direct bearing.

Filled composite column. HSS filled with structural concrete.

Fully composite beam. Composite beam that has a sufficient number of shear connectors to develop the nominal plastic flexural strength of the composite section.

Highly ductile member. A member expected to undergo significant plastic rotation (more than 0.02 rad) under the *design earthquake*.

Inelastic deformation. The plastic portion of deformation imposed on an element, such as plastic axial deformation in a buckling-restrained brace

Horizontal boundary element (HBE). A beam with a connection to one or more web plates in a SPSW.

Inelastic rotation. The permanent or plastic portion of the rotation angle between a beam and the column or between a link and the column of the test specimen, measured in radians.

Intermediate boundary element (IBE). A member, other than a beam or column, that provides resistance to web plate tension adjacent to an opening in a SPSW.

Intermediate moment frame (IMF). Moment frame system that meets the requirements of Section E2.

Intermediate seismic systems. Seismic systems designed assuming moderate inelastic action occurs in some members under the design earthquake.

Interstory drift angle. Interstory displacement divided by story height, radians.

Inverted-V-braced frame. See *V-braced frame*.

k-area. The *k-area* is the region of the web that extends from the tangent point of the web and the flange-web fillet (AISC “*k*” dimension) a distance of 1½ in. (38 mm) into the web beyond the “*k*” dimension.

K-braced frame. A braced-frame configuration in which braces connect to a column at a location with no diaphragm or other out-of-plane support.

Link. In EBF, the segment of a beam that is located between the ends of the connections of two diagonal braces or between the end of a diagonal brace and a column. The length of the *link* is defined as the clear distance between the ends of two diagonal braces or between the diagonal brace and the column face.

Link intermediate web stiffeners. Vertical web stiffeners placed within the link in EBF.

Link rotation angle. Inelastic angle between the link and the beam outside of the *link* when the total story drift is equal to the *design story drift*.

Link shear design strength. Lesser of the available shear strength of the *link* based on the flexural or shear strength of the link member.

Load-carrying reinforcement. Reinforcement in composite members designed and detailed to resist the required loads.

Lowest Anticipated Service Temperature (LAST). The lowest 1-hour average temperature with a 100-year mean recurrence interval.

LRFD (Load and Resistance Factor Design) †. Method of proportioning structural components such that the *design strength* equals or exceeds the *required strength* of the component under the action of the *LRFD load combinations*.

LRFD Load Combination†. Load combination in the *applicable building code* intended for strength design (*load and resistance factor design*).

Material test plate. A test specimen from which steel samples or weld metal samples are machined for subsequent testing to determine mechanical properties.

Measured flexural resistance. Bending moment measured in a beam at the face of the column, for a beam-to-column test specimen tested in accordance with Section J2.

Member brace. Member that provides stiffness and strength to control movement of another member out-of-the plane of the frame at the braced points.

Moderately ductile member A member expected to undergo moderate plastic rotation (0.02 rad or less) under the *design earthquake*.

Nominal load†. Magnitude of the *load* specified by the *applicable building code*.

Nominal strength†.* Strength of a structure or component (without the *resistance factor* or *safety factor* applied) to resist load effects, as determined in accordance with the *Specification*.

- Occupancy Category.* Classification assigned to a structure based on its use as specified by the *applicable building code*.
- Ordinary concentrically braced frame (OCBF).* Diagonally braced frame meeting the requirements of Section F1 in which all members of the braced-frame system are subjected primarily to axial forces.
- Ordinary moment frame (OMF).* Moment frame system that meets the requirements of Section E1.
- Ordinary seismic systems.* Seismic systems designed assuming limited (less than moderate) inelastic action occurs in some members under the design earthquake.
- Overstrength factor, Ω_o .* Factor specified by the *applicable building code* in order to determine the amplified seismic load, where required by these *Provisions*.
- Partially composite beam. Unencased composite beam* with a nominal flexural strength controlled by the strength of the shear stud connectors.
- Partially-restrained composite connection.* Partially restrained (PR) connections as defined in the *Specification* that connect partially or *fully composite beams* to steel columns with flexural resistance provided by a force couple achieved with steel reinforcement in the slab and a steel seat angle or similar connection at the bottom flange.
- Plastic Hinge.* Yielded zone that forms in a structural member when the plastic moment is attained. The member is assumed to rotate further as if hinged, except that such rotation is restrained by the plastic moment.
- Prequalified connection.* Connection that complies with the requirements of Section J1 or ANSI/AISC 358.
- Protected zone.* Area of members or connections of members in which limitations apply to fabrication and attachments.
- Prototype.* The tested connection or diagonal brace design that is to be used in the building (SMF, IMF, EBF, and BRBF).
- Provisions.* Refers to this document, the AISC *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341).
- Quality assurance plan.* Written description of qualifications, procedures, quality inspections, resources and records to be used to provide assurance that the structure complies with the engineer's quality requirements, specifications and contract documents.
- Reduced beam section.* Reduction in cross section over a discrete length that promotes a zone of inelasticity in the member.
- Reinforced-concrete-encased shapes.* Structural steel sections encased in reinforced concrete.
- Required strength*.* Forces, stresses, and deformations acting on a structural component, determined by either structural analysis, for the *LRFD or ASD*

load combinations, as appropriate, or as specified by the *Specification* and these *Provisions*.

Restraining bars. Steel reinforcement in composite members that is not designed to carry required loads, but is provided to facilitate the erection of other steel reinforcement and to provide anchorage for stirrups or ties. Generally, such reinforcement is not spliced to be continuous.

Resistance factor, ϕ . Factor that accounts for unavoidable deviations of the *nominal strength* from the actual strength and for the manner and consequences of failure.

Safety factor, Ω . Factor that accounts for deviations of the actual strength from the nominal strength, deviations of the actual load from the *nominal load*, uncertainties in the analysis that transforms the load into a load effect, and for the manner and consequences of failure.

Seismic design category. Classification assigned to a building by the *applicable building code* based upon its *occupancy category* and the design spectral response acceleration coefficients.

Seismic force resisting system (SFRS). That part of the structural system that has been considered in the design to provide the required resistance to the seismic forces prescribed in SEI/ASCE 7.

Seismic response modification coefficient, R. Factor that reduces seismic load effects to strength level as specified by the *applicable building code*.

Special concentrically braced frame (SCBF). Diagonally braced frame meeting the requirements of Section F2 in which all members of the braced-frame system are subjected primarily to axial forces.

Special moment frame (SMF). Moment frame system that meets the requirements of Section E3.

Special plate shear wall (SPSW). Plate shear wall system that meets the requirements of Section F5.

Special seismic systems. Seismic systems designed assuming significant inelastic action occurs in some members under the design earthquake.

Special truss moment frame (STMF). Truss moment frame system that meets the requirements of Section E4.

Specification. Refers to the AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360).

Steel core. Axial-force-resisting element of diagonal braces in BRBF. The steel core contains a yielding segment and connections to transfer its axial force to adjoining elements; it may also contain projections beyond the casing and transition segments between the projections and yielding segment.

Subassembly test specimen. The combination of the brace, the connections and testing apparatus that replicate as closely as practical the axial and flexural deformations of the brace in the *prototype*.

Tested connection. Connection that complies with the requirements of Section J2.

Test setup. The supporting fixtures, loading equipment, and lateral bracing used to support and load the Test Specimen.

Test specimen. Brace test specimen or subassemblage test specimen.

Test subassemblage. The combination of the test specimen and pertinent portions of the test setup.

Total link rotation angle. The relative displacement of one end of the link with respect to the other end (measured transverse to the longitudinal axis of the undeformed link), divided by the link length. The total link rotation angle includes both elastic and inelastic components of deformation of the link and the members attached to the link ends.

V-braced frame. Concentrically braced frame (SCBF, OCBF, BRBF, C-OBF or C-SCBF) in which a pair of diagonal braces located either above or below a beam is connected to a single point within the clear beam span. Where the diagonal braces are below the beam, the system is also referred to as an *inverted-V-braced frame*.

Vertical boundary element (VBE). A column with a connection to one or more web plates in a SPSW.

X-braced frame. Concentrically braced frame (OCBF, SCBF, C-OBF or C-SCBF) in which a pair of diagonal braces crosses near the mid-length of the diagonal braces.

Y-braced frame. Eccentrically braced frame (EBF and C-EBF) in which the stem of the Y is the *link* of the EBF system.

DRAFT

ACRONYMS

The following acronyms appear in the AISC *Seismic Provisions for Structural Steel Buildings*. The acronyms are written out where they first appear within a Section.

ACI (American Concrete Institute)
ANSI (American National Standards Institute)
ASCE (American Society of Civil Engineers)
ASD (allowable strength design)
ASNT (American Society for Nondestructive Testing)
ASTM (American Society for Testing of Materials)
AWS (American Welding Society)
BRBF (buckling-restrained braced frame)
CAC-A (air carbon arc cutting)
C-CBF (composite special concentrically braced frame)
CCS (cantilever column system)
C-EBF (composite eccentrically braced frame)
C-IMF (composite intermediate moment frame)
CJP (complete joint penetration)
C-OBF (composite ordinary braced frame)
C-OMF (composite ordinary moment frame)
C-ORCW (composite ordinary reinforced concrete shear wall with steel elements)
C-PRMF (composite partially restrained moment frame)
CPRP (connection prequalification review panel)
C-SCBF (composite special concentrically braced frame)
C-SMF (composite special moment frame)
C-SPW (composite steel plate shear wall)
C-SRCW (composite special reinforced concrete shear walls with steel elements)
CVN (Charpy V-notch)
EBF (eccentrically braced frame)
FCAW (flux cored arc welding)
FEMA (Federal Emergency Management Agency)
GMAW (gas metal arc welding)
HBE (horizontal boundary element)
HSS (hollow structural section)
IBE (intermediate boundary element)
IMF (intermediate moment frame)
LAST (lowest anticipated service temperature)
LRFD (load and resistance factor design)
MT (magnetic particle testing)
NDT (nondestructive testing)
OCBF (ordinary concentrically braced frame)
OMF (ordinary moment frame)
QA (quality assurance)
QC (quality control)

PJP (partial joint penetration)
RBS (reduced beam section)
RCSC (Research Council on Structural Connections)
SAW (submerged arc welding)
SCBF (special concentrically braced frame)
SDC (seismic design category)
SFRS (seismic force resisting system)
SMAW (shielded metal arc welding).
SMF (special moment frame)
SPSW (special plate shear wall)
SRC (steel-reinforced concrete)
STMF (special truss moment frame)
UT (ultrasonic testing)
VBE (vertical boundary element)
WPQR (welder performance qualification records)
WPS (welding procedure specifications)

DRAFT

DRAFT

CHAPTER A

GENERAL REQUIREMENTS

This chapter states the scope of the *Provisions*, summarizes referenced specification, code, and standard documents, and provides requirements for materials and contract documents.

The chapter is organized as follows:

- A1. Scope
- A2. Referenced Specifications, Codes and Standards
- A3. Materials
- A4. Drawings And Specifications

A1. SCOPE

The *Seismic Provisions for Structural Steel Buildings*, hereafter referred to as these Provisions, shall govern the design, fabrication and erection of structural steel members and connections in the *seismic force resisting systems* (SFRS), and splices and bases of columns in gravity framing systems of buildings and other structures with *moment frames, braced frames* and *shear walls*. Other structures are defined as those structures designed, fabricated and erected in a manner similar to buildings, with building-like vertical and lateral force-resisting-elements. These *Provisions* shall apply for design of seismic force resisting systems of structural steel or of structural steel acting compositely with reinforced concrete, unless specifically exempted by the applicable building code.

Wherever these provisions refer to the *applicable building code* and there is no local building code, the loads, *load combinations*, system limitations, and general design requirements shall be those in SEI/ASCE 7.

User Note: SEI/ASCE 7 (Table 12.2-1, Item H) specifically exempts structural steel systems, but not composite systems, in Seismic Design Categories B and C if they are designed according to the *Specification for Structural Steel Buildings* and the seismic loads are computed using an *R* factor of 3. For Seismic Design Category A, SEI/ASCE 7 does specify lateral forces to be used as the seismic loads and effects, but these calculations do not involve the use of an *R* factor. Thus for Seismic Design Category A it is not necessary to define a seismic force resisting system that meets any special requirements and these *Provisions* do not

39 apply.

40

41 **User Note:** Composite seismic force resisting systems include those
42 systems with at least some members of structural steel acting compositely
43 with reinforced concrete, as well as systems in which structural steel
44 members and reinforced concrete members act together to be a composite
45 system.

46 These *Provisions* shall be applied in conjunction with the AISC
47 *Specification for Structural Steel Buildings*, hereafter referred to as the
48 *Specification*. Members and connections of the SFRS shall satisfy the
49 requirements of the applicable building code, the *Specification*, and these
50 *Provisions*.

51 *Building Code Requirements for Structural Concrete* (ACI 318), as
52 modified in these *Provisions*, shall be used for the design of reinforced
53 concrete components in composite construction. For the SFRS in
54 composite construction incorporating reinforced concrete components
55 designed according to ACI 318, the requirements of *Specification* Section
56 B3.3, Design for Strength Using Load and Resistance Factor Design,
57 shall be used.

58 When the design is based upon elastic analysis, the stiffness properties of
59 the component members of composite construction shall reflect their
60 condition at the onset of significant yielding of the structure.

61

62 **[SECTION A2 BELOW IS NOT INCLUDED IN THE APRIL 09 BALLOT]**

63 **A2. REFERENCED SPECIFICATIONS, CODES AND STANDARDS**

64 The documents referenced in these *Provisions* shall include those listed in
65 *Specification* Section A2 with the following additions:

66

67 American Concrete Institute (ACI)
68 *Details and Detailing of Concrete Reinforcement*, ACI 315-99
69 *Manual of Engineering and Placing Drawings for Reinforced Concrete*
70 *Structures*, ACI 315-R-94
71 *ACI Detailing Manual*, ACI SP-66
72 *Monolithic Joints in Concrete Structures*, ACI 352

73 American Institute of Steel Construction (AISC)
74 *Specification for Structural Steel Buildings*, ANSI/AISC 360-2010
75 *Prequalified Connections for Special and Intermediate Steel Moment*
76 *Frames for Seismic Applications*, ANSI/AISC 358-10, including
77 Supplement No. 1

78 American Welding Society (AWS)

79	<i>Structural Welding Code–Seismic Supplement, ANSI/AWS</i>
80	<i>D1.8/D1.8M:2009</i>
81	<i>Standard Methods for Mechanical Testing of Welds- U.S. Customary,</i>
82	<i>ANSI/AWS B4.0-98</i>
83	<i>Standard Methods for Mechanical Testing of Welds – Metric Only,</i>
84	<i>ANSI/AWS B4.0M:2000</i>
85	<i>Structural Welding Code-Reinforcing Steel, AWS D1.4-2005</i>

DRAFT
DRAFT

86

87 **A3. MATERIALS**88 **1. Material Specifications**

89 Structural steel used in the *seismic force resisting system* (SFRS) shall
90 meet the requirements of *Specification* Section A3.1a, except as modified
91 in these *Provisions*. The specified minimum yield stress of steel to be
92 used for members in which inelastic behavior is expected shall not
93 exceed 50 ksi (345 MPa) for systems defined in Chapters E, F, G and H
94 except that for systems defined in Sections E1, F1, G1, H1 and H4 this
95 limit shall not exceed 55 ksi (380 MPa). Either of these specified
96 minimum yield stress limits are permitted to be exceeded when the
97 suitability of the material is determined by testing or other rational
98 criteria. This limitation does not apply to columns or other members
99 which are not the primary intended yielding element(s) as indicated in the
100 section, Basis of Design, for the given system.

101 **User Note:** Specified minimum yield stress of columns in *special*
102 *moment frame* (SMF) systems may exceed 50 ksi (345 MPa) if inelastic
103 yielding at the column base is anticipated, as the columns are not a
104 primary intended yielding element.

105 The structural steel used in the SFRS described in Chapters E, F, G and H
106 shall meet one of the following ASTM Specifications:

107 A36/A36M

108 A53/A53M

109 A500 (Gr. B or C)

110 A501

111 A529/A529M

112 A572/A572M [Gr. 42 (290), 50 (345) or 55 (380)]

113 A588/A588M

114 A913/A913M [Gr. 50 (345), 60 (415) or 65 (450)]

115 A992/A992M

116 A1011 HSLAS Gr. 55 (380)

117 A1043/A1043M

118 The structural steel used for column base plates shall meet one of the
119 preceding ASTM specifications or ASTM A283/A283M Grade D.

120 ASTM A572/A572M Gr. 42 (290) is not permitted in members where the
121 load effect of the member or its connections is based upon the *required*
122 *strength* of the member as determined per Section A3.2.

123 Other steels and nonsteel materials in *buckling-restrained braced frames*
124 are permitted to be used subject to the requirements of Sections F4 and
125 J3.

126 **User Note:** This section only covers material properties for structural
127 steel used in the SFRS and included in the definition of structural steel
128 given in Section 2.1 of the AISC *Code of Standard Practice*. Other steel,
129 such as cables for permanent bracing, is not covered.

130 2. Expected Material Strength

131 When required in these *Provisions*, the required strength of an element (a
132 member or a connection of a member) shall be determined from the
133 *expected yield stress*, $R_y F_y$, of the member or an adjoining member, as
134 applicable, where F_y is the specified minimum yield stress of the grade of
135 steel to be used in the member and R_y is the ratio of the expected yield
136 stress to the specified minimum yield stress, F_y , of that material.

137 When required to determine the *nominal strength*, R_n , for limit states
138 within the same member from which the required strength is determined,
139 the expected yield stress, $R_y F_y$, and the *expected tensile strength*, $R_t F_u$, are
140 permitted to be used in lieu of F_y and F_u , respectively, where F_u is the
141 specified minimum tensile strength and R_t is the ratio of the expected
142 tensile strength to the specified minimum tensile strength, F_u , of that
143 material.

144 **User Note:** In several instances a member, or a connection limit state
145 within that member, is required to be designed for forces corresponding
146 to the expected strength of the member itself. Such cases include
147 determination of the nominal strength, R_n , of the beam outside of the link
148 in EBF, diagonal brace rupture limit states (block-shear rupture and net-
149 section rupture in the diagonal brace in SCBF), etc. In such cases it is
150 permitted to use the expected material strength in the determination of
151 available member strength. For connecting elements and for other
152 members, specified material strength should be used.

153 The values of R_y and R_t for various steel and concrete materials are given
154 in Table A3.1. Other values of R_y and R_t shall be permitted if the values
155 are determined by testing of specimens, similar in size and source to the
156 materials to be used, conducted in accordance with the testing
157 requirements per the ASTM specifications for the specified grade of
158 steel.

159

160

TABLE A3.1
 R_y and R_t Values for Steel and Concrete Materials

Application	R_y	R_t
Hot-rolled structural shapes and bars:		
• ASTM A36/A36M,	1.5	1.2
• ASTM A1043/1043M Gr. 36 (248)	1.3	1.1
• ASTM A572/572M Gr. 50 (345) or 55 (380), ASTM A913/A913M Gr. 50 (345), 60 (415), or 65 (450), ASTM A588/A588M, ASTM A992/A992M,	1.1	1.1
• ASTM A1043/A1043M Gr. 50 (345)	1.2	1.1
• ASTM A529 Gr. 50 (345)	1.2	1.2
• ASTM A529 Gr. 55 (380)	1.1	1.2
Hollow structural sections (HSS):		
• ASTM A500 (Gr. B or C),	1.4	1.3
• ASTM A501		
Pipe:		
• ASTM A53/A53M	1.6	1.2
Plates, Strips and Sheets:		
• ASTM A36/A36M,	1.3	1.2
• ASTM A1043/1043M Gr. 36 (248)	1.3	1.1
• A1011 HSLAS Gr. 55 (380),	1.1	1.1
• ASTM A572/A572M Gr. 50 (345), Gr. 55 (380), ASTM A588/A588M,	1.1	1.2
• ASTM 1043/1043M Gr. 50 (345)	1.2	1.1
• Steel Reinforcement:		
• ASTM A615,	1.25	1.25
• ASTM A706		
•		
• Structural Concrete:		
• Structural Concrete	See ACI	See ACI

•	318 Table 5.3.2.2	318 Table 5.3.2.2
---	-------------------------	-------------------------

161 **3. Heavy Sections**

162 For structural steel in the SFRS, in addition to the requirements of
 163 *Specification* Section A3.1c, hot rolled shapes with flanges 1½ in. thick
 164 (38 mm) and thicker shall have a minimum Charpy V-notch toughness of
 165 20 ft-lbs (27 J) at 70 °F (21 °C), tested in the alternate core location as
 166 described in ASTM A6 Supplementary Requirement S30. Plates 2 in.
 167 (50 mm) thick and thicker shall have a minimum Charpy V-notch
 168 toughness of 20 ft-lbs (27 J) at 70 °F (21 °C), measured at any location
 169 permitted by ASTM A673, Frequency P, where the plate is used in the
 170 following:

- 171 (a) Members built-up from plate
- 172 (b) Connection plates where inelastic strain under seismic loading is
 173 expected
- 174 (c) The *steel core* of buckling restrained braces

175 **User Note:** Examples of connection plates where inelastic behavior is
 176 expected include, but are not limited to, gusset plates intended to function
 177 as a hinge and allow out-of-plane buckling of diagonal braces, some
 178 bolted flange plates for moment connections, some end plates for bolted
 179 moment connections, and some column base plates designed as a pin.

180 **4. Consumables for Welding**

181 **4a. Seismic Force Resisting System Welds**

182 All welds used in members and connections in the SFRS shall be made
 183 with filler metals meeting the requirements specified in AWS D1.8
 184 Clause 6.3. AWS D1.8 Clauses 6.3.5, 6.3.6, 6.3.7 and 6.3.8 shall apply
 185 only to demand critical welds..
 186
 187

188 **4b. Demand Critical Welds**

189 Welds designated as demand critical shall be made with filler metals
 190 meeting the requirements specified in AWS D1.8 Clause 6.3.
 191
 192

193
194
195
196
197
198
199

User Note:
AWS D1.8 requires that all seismic force resisting system welds are to be made with filler metals classified using AWS A5 standards that achieve the following mechanical properties:

Filler Metal Classification Properties for Seismic Force Resisting System Welds		
Property	Classification	
	70 ksi (480 MPa)	80 ksi (550 MPa)
Yield Strength, ksi (MPa)	58 (400) min.	68 (470) min.
Tensile Strength, ksi (MPa)	70 (480) min.	80 (550) min.
Elongation, %	22 min.	19 min.
CVN Toughness, ft-lbf (J)	20 (27) min. @ 0 °F (-18 °C) ^a	
^a Filler metals classified as meeting 20 ft-lbf (27 J) min. at a temperature lower than 0 °F (-18 °C) also meet this requirement.		

200
201
202
203
204
205

In addition to the above requirements, AWS D1.8 requires, unless otherwise exempted from testing, that all demand critical welds are to be made with filler metals receiving Heat Input Envelope Testing that achieve the following mechanical properties in the weld metal:

Mechanical Properties for Demand Critical Welds		
Property	Classification	
	70 ksi (480 MPa)	80 ksi (550 MPa)
Yield Strength, ksi (MPa)	58 (400) min.	68 (470) min.
Tensile Strength, ksi (MPa)	70 (480) min.	80 (550) min.
Elongation (%)	22 min.	19 min.
CVN Toughness, ft-lbf (J)	40 (54) min. @ 70 °F (20 °C) ^{b, c}	
^b For LAST of +50 °F (+10 °C). For LAST less than + 50 °F (+10 °C), see AWS D1.8 Clause 6.3.6. ^c Tests conducted in accordance to AWS D1.8 Annex A meeting 40 ft-lbf (54 J) min. at a temperature lower than +70 °F (20 °C) also meet this requirement.		

206
207
208
209
210
211

5. Concrete and Steel Reinforcement

Concrete and steel reinforcement used in composite components in composite intermediate or special SFRS of Sections G2, G3, G4, H2, H3, H5 and H6 shall meet the requirements of ACI 318, Chapter 21. Concrete and steel reinforcement used in composite components in composite

212 ordinary SFRS of Sections G1, H1 and H4 shall meet the requirements of
213 ACI 318, Section 21.1.1.5.

214

215 **A4. DRAWINGS AND SPECIFICATIONS**

216 **1. General**

217 Structural design drawings and specifications shall indicate the work to
218 be performed, and include items required by the *Specification*, the AISC
219 *Code of Standard Practice for Steel Buildings and Bridges*, the
220 *applicable building code*, and the following, as applicable:

- 221 (1) Designation of the SFRS.
- 222 (2) Identification of the members and connections that are part of the
223 SFRS.
- 224 (3) Locations and dimensions of *protected zones*.
- 225 (4) Connection details between the diaphragm and the SFRS, when
226 concrete floor slabs serve as diaphragms.
- 227 (5) Shop drawing and erection drawing requirements not addressed in
228 Section I1.

229 **User Note:** For connections and applications for which details are not
230 specifically addressed by the *Provisions*, The *engineer of record* should
231 include applicable requirements in the design drawings and
232 specifications. Any requirements for shop drawings and erection
233 drawings beyond the requirements of these *Provisions* should be clearly
234 indicated in the design drawings and specifications.

235 **2. Steel Construction**

236 In addition to the requirements of Section A4.1, structural design
237 drawings and specifications for steel construction shall indicate the
238 following items, as applicable:

- 239 (1) Configuration of the connections.
- 240 (2) Connection material specifications and sizes.
- 241 (3) Locations of *demand critical welds*.
- 242 (4) Locations where gusset plates are to be detailed to accommodate
243 inelastic rotation.
- 244 (5) Locations of connection plates requiring Charpy V-notch (CVN)
245 toughness in accordance with Section A3.3(b).

- 246 (6) *Lowest anticipated service temperature (LAST)* of the steel
247 structure, if the structure is not enclosed and maintained at a
248 temperature of 50 °F (10 °C) or higher.
- 249 (7) Locations where weld backing is required to be removed.
- 250 (8) Locations where fillet welds are required when weld backing is
251 permitted to remain.
- 252 (9) Locations where fillet welds are required to reinforce groove
253 welds or to improve connection geometry.
- 254 (10) Locations where weld tabs are required to be removed.
- 255 (11) Splice locations where tapered transitions are required.
- 256
- 257 (12) The shape of weld access holes, if a shape other than those
258 provided for in the *Specification* is required.
- 259 (13) Joints or groups of joints in which a specific assembly order,
260 welding sequence, welding technique or other special precautions
261 are required to be submitted to the engineer of record.
262

3. Composite Construction

264 In addition to the requirements of Section A4.1, and the requirements of
265 Section A4.2 as applicable for the steel components of reinforced
266 concrete or composite elements, structural design drawings, and
267 specifications for composite construction shall indicate the following
268 items, as applicable:

- 269 (1) Bar placement, cutoffs, lap and mechanical splices, hooks and
270 mechanical anchorage, placement of ties and other transverse
271 reinforcement.
- 272 (2) Requirements for dimensional changes resulting from
273 temperature changes, creep and shrinkage.
- 274 (3) Location, magnitude and sequencing of any prestressing or post-
275 tensioning present.
- 276 (4) Location of steel headed stud anchors and welded reinforcing bar
277 anchors.

279 GENERAL DESIGN REQUIREMENTS

280 This chapter addresses the general requirements for the seismic design of steel
281 structures that are applicable to all chapters of the *Provisions*.

282 This chapter is organized as follows:

- 283
- 284 B1. General Seismic Design Requirements
 - 285 B2. Loads and Load Combinations
 - 286 B3. Design Basis
 - 287 B4. System Type
- 288

289 **B1. GENERAL SEISMIC DESIGN REQUIREMENTS**

290 The *required strength* and other seismic provisions for *seismic design*
291 *categories* (SDCs), *occupancy categories*, and the limitations on height
292 and irregularity shall be as specified in the *applicable building code*.

293 The *design story drift* and the limitations on story drift shall be
294 determined as required in the *applicable building code*.

295 **B2. LOADS AND LOAD COMBINATIONS**

296 The loads and *load combinations* shall be as stipulated by the *applicable*
297 *building code*. Where *amplified seismic loads* are required by these
298 *Provisions*, the seismic load effect including the overstrength factor, E_m ,
299 shall be applied as prescribed by the applicable building code

300 For the *seismic force resisting system* (SFRS), in composite construction,
301 incorporating reinforced concrete components designed according to the
302 requirements of ACI 318, the requirements of *Specification* Section B3.3,
303 Design for Strength Using Load and Resistance Factor Design, shall be
304 used.

305 **User Note:** When not defined in the applicable building code, Ω_o should
306 be taken from SEI/ASCE 7.

307 **B3. DESIGN BASIS**308 **1. Required Strength**

309 The *required strength* of structural members and connections shall be the
310 greater of:

311 (1) The required strength as determined by structural analysis for the
312 appropriate *load combinations*, as stipulated in the *applicable*
313 *building code*, and in Chapter C.

314 (2) The required strength given in Chapters D, E, F, G and H.

315 **2. Available Strength**

316 The *available strength* is stipulated as *design strength*, ϕR_n , for design
317 according to the provisions for *load and resistance factor design* (LRFD)
318 and *allowable strength*, R_n/Ω , for design according to the provisions for
319 *allowable strength design* (ASD). The available strength of systems,
320 members, and connections shall be determined according to the
321 *Specification*, except as modified throughout these *Provisions*.

322 **B4. SYSTEM TYPE**

323 The seismic force resisting system (SFRS) shall contain one or more
324 moment frame, braced frame or shear wall systems conforming to the
325 requirements of one of the seismic systems designated in Chapters E, F,
326 G and H.

DRAFT

327

CHAPTER C

328

ANALYSIS

329

330 This chapter addresses design related analysis requirements. The chapter is
331 organized as follows:

332 C1. General

333 C2. Additional Requirements

334 C3. Nonlinear Analysis

335

336 **C1. GENERAL**

337 An analysis conforming to the requirements of the *applicable building*
338 *code* and the *Specification* shall be performed for design of the system.

339

340 When the design is based upon elastic analysis, the stiffness properties of
341 component members of steel systems shall be based on elastic sections
342 and those of composite systems shall include the effects of cracked
343 sections.

344

345 **C2. ADDITIONAL REQUIREMENTS**

346 Additional analysis shall be performed as specified in Chapters E, F, G
347 and H of these *Provisions*.

348 **C3. NONLINEAR ANALYSIS**

349 When nonlinear analysis is used to satisfy the requirements of these
350 *Provisions*, it shall be performed in accordance with Chapter 16 of
351 SEI/ASCE 7.

352

353

354

CHAPTER D

355 GENERAL MEMBER AND CONNECTION DESIGN REQUIREMENTS

356

357 This chapter addresses general requirements for the design of members and
358 connections.

359 The chapter is organized as follows:

360 D1. Member Requirements

361 D2. Connections

362 D3. Deformation Compatibility of Non-SRFS Members and Connections

363 D4. H-Piles

364 **D1. MEMBER REQUIREMENTS**

365 Members of moment frames, braced frames and shear walls in the
366 seismic force resisting system (SFRS) shall comply with the *Specification*
367 and this section.

368 **1. Classification of Sections for Ductility**

369 Certain members of the SFRS that are expected to undergo inelastic
370 deformation under the design earthquake are designated in these
371 provisions as *moderately ductile members* or *highly ductile members*.
372 When required for the systems defined in Chapters E, F, G, H and
373 Section D4, these members shall comply with this section.

374 **1a. Section Requirements for Ductile Members**

375 Structural steel sections for both *moderately ductile members* and *highly*
376 *ductile members* shall have flanges continuously connected to the web or
377 webs. Webs and flanges of built-up sections shall be continuously
378 connected by fillet or groove welds. Welds of webs to flanges of built-up
379 sections used as link beams in *eccentrically braced frames* shall be
380 complete-joint-penetration groove welds. Built-up sections used in
381 *intermediate moment frames* and *special moment frames* shall comply
382 with Section 2.3.2 of ANSI/AISC 358.

383 Encased structural steel sections shall comply with the requirements of
384 Section D1.4b(1) for moderately ductile members and Section D1.4b(2)
385 for highly ductile members.

386 Filled composite sections shall comply with the requirements of Section
387 D1.4c for both moderately and highly ductile members.

388 Concrete sections shall comply with the requirements of ACI 318 Section
389 21.3 for moderately ductile members and ACI 318 Section 21.6 for
390 highly ductile members.

391 **1b. Width-to-Thickness Limitations of Steel Sections**

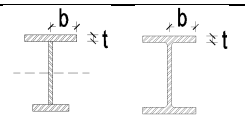
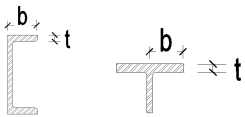
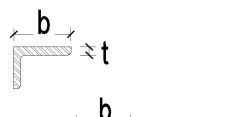
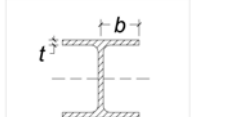
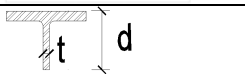
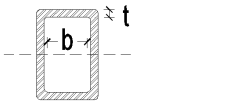
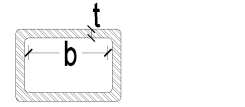
392 For members designated as *moderately ductile members*, the width-
393 thickness ratios of compression elements shall not exceed the limiting
394 width-thickness ratios, λ_{md} , from Table D1.1.

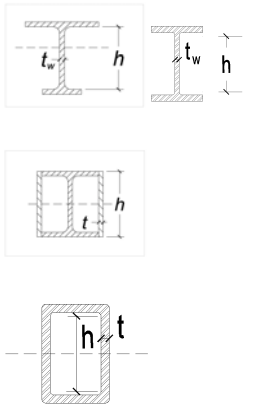
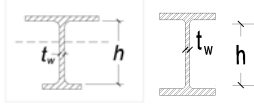
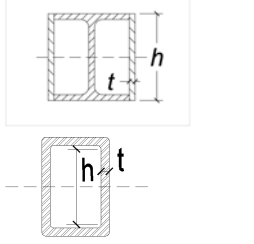
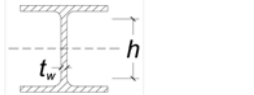
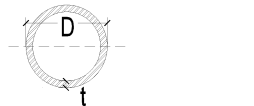
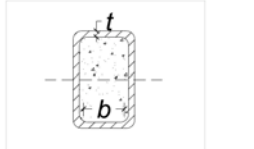
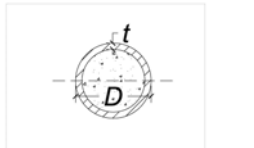
395 For members designated as *highly ductile members*, the width-thickness
396 ratios of compression elements shall not exceed the limiting width-
397 thickness ratios, λ_{hd} , from Table D1.1.

398

DRAFT

TABLE D1.1
Limiting Width-Thickness Ratios for Compression Elements

Description of Element	Width-Thickness Ratio	Limiting Width-Thickness Ratio		Example	
		λ_{hd} Highly Ductile Members	λ_{md} Moderately Ductile Members		
Unstiffened Elements	Flanges of rolled or built-up I-shaped sections, channels and tees	b/t	$0.30 \sqrt{E/F_y}$	$0.38 \sqrt{E/F_y}$	
	Legs of single angles or double angle members with separators				
	Outstanding legs of pairs of angles in continuous contact				
	Flanges of H-pile sections per Section D4				
Stems of tees ^[a]	d/t	$0.30 \sqrt{E/F_y}$	$0.38 \sqrt{E/F_y}$		
Stiffened Elements	Flanges of boxed I-shaped sections and built-up box sections	b/t	$0.60 \sqrt{E/F_y}$	$1.12 \sqrt{E/F_y}$	
	Walls of rectangular HSS	b/t	$0.55 \sqrt{E/F_y}$	$0.64 \sqrt{E/F_y}^{[b]}$	

	Members subject to flexure or combined flexure and compression: Webs of rolled or built-up I-shaped sections, ^(d) Side plates of boxed I-shaped sections, Webs of built-up box sections	h/t_w h/t h/t	For $C_a \leq 0.125$ $2.45 \sqrt{E/F_y} (1 - 0.93C_a)$ For $C_a > 0.125$ $0.77 \sqrt{E/F_y} (2.93 - C_a)$ $\geq 1.49 \sqrt{E/F_y}$ where $C_a = \frac{P_u}{\phi_b P_y}$ (LRFD) $C_a = \frac{\Omega_b P_a}{P_y}$ (ASD)	For $C_a \leq 0.125$ $3.76 \sqrt{E/F_y} (1 - 2.75C_a)$ For $C_a > 0.125$ $1.12 \sqrt{E/F_y} (2.33 - C_a)$ $\geq 1.49 \sqrt{E/F_y}$ where $C_a = \frac{P_u}{\phi_b P_y}$ (LRFD) $C_a = \frac{\Omega_b P_a}{P_y}$ (ASD)	
	Webs of rolled or built-up I-shaped sections subject to uniform compression	h/t_w	$1.49 \sqrt{E/F_y}$	$1.49 \sqrt{E/F_y}$	
	Side plates of boxed I-shaped sections, and webs of built-up box sections subject to uniform compression ^(e)	h/t	Columns: $1.49 \sqrt{E/F_y}$ Diagonal Braces: $0.60 \sqrt{E/F_y}$	Columns: $1.49 \sqrt{E/F_y}$ Diagonal Braces: $0.70 \sqrt{E/F_y}$	
	Webs of H-Pile sections	h/t_w	$0.94 \sqrt{E/F_y}$	not applicable	
	Walls of round HSS	D/t	$0.038 E / F_y$	$0.044 E / F_y$ [c]	
Composite Elements	Rectangular filled composite member	b/t	$1.4 \sqrt{E / F_y}$	$2.26 \sqrt{E / F_y}$	
	Round filled composite member	D/t	$0.076 E / F_y$	$0.15 E / F_y$	

^[a] For tee shaped compression members, the limiting width-to-thickness ratio for the stem of the tee can be increased to $0.38 \sqrt{E/F_y}$ if either of the following conditions are satisfied:

- (1) Buckling of the compression member occurs about the plane of the stem.
- (2) The compression load is transferred at end connections to only the outside face of the flange of the tee resulting in reduced compression stresses at the tip of the stem of the tee due to flexure from the eccentric connection.

^[b] The limiting width to thickness ratio of walls of rectangular HSS members used as beams or columns shall not exceed $1.12 \sqrt{E/F_y}$

^[c] The limiting diameter to thickness ratio of round HSS members used as beams or columns shall not exceed $0.07 E/F_y$.

^[d] For beams in OMF, IMF or SMF systems, where C_a is less than 0.125, the limiting ratio h/t_w shall not exceed $2.45 \sqrt{E/F_y}$.

DRAFT

400

401

402

403 **2. Stability Bracing of Members**

404 Where required in Chapters E, F, G and H, stability bracing of structural
 405 steel, concrete-encased or concrete-filled members subject to flexure
 406 shall be provided to restrain lateral-torsional buckling as required in this
 407 section.

408 **User Note:** In addition to the requirements in Chapters E, F, G and H to
 409 provide stability bracing for various beam members such as intermediate
 410 and special moment frame beams, stability bracing is also required for
 411 columns in the *cantilever column systems* (CCS) in Sections E5 and E6.

412 **2a. Moderately Ductile Members**

413 (1) The bracing of moderately ductile steel members shall satisfy the
 414 following requirements:

415 (i) Both flanges of members shall be laterally braced or the
 416 member cross section shall be torsionally braced.

417 (ii) Member bracing shall meet the requirements of Appendix
 418 6 of the *Specification* for lateral or torsional bracing of
 419 beams, where the expected flexural strength of the
 420 member shall be:

$$421 \quad M_r = M_u = R_y Z F_y \text{ (LRFD)} \quad \text{(D1-1a)}$$

422 or **DRAFT**

$$423 \quad M_r = M_u = R_y Z F_y / 1.5 \text{ (ASD)} \quad \text{(D1-1b)}$$

424 where

$$425 \quad C_d = 1.0$$

426 (iii) Member bracing shall have a maximum spacing of

$$427 \quad L_b = 0.17 r_y E / F_y \quad \text{(D1-2)}$$

428 (2) The bracing of moderately ductile concrete-encased and concrete-
 429 filled composite members shall satisfy the following
 430 requirements:

431 (i) Both flanges of members shall be laterally braced or the
 432 member cross-section shall be torsionally braced..

433 (ii) Lateral bracing shall meet the requirements of Appendix
 434 6 of the *Specification* for lateral or torsional bracing of
 435 beams, where $M_r = M_u = M_p$ of the member as specified in
 436 Section G2.6c, and $C_d = 1.0$.

437 (iii) Member bracing shall have a maximum spacing of

$$438 \quad L_b = 0.17r_y E / F_y \quad (D1-3)$$

439 using the properties of the steel section, except that in the
 440 calculation for concrete encased composite members, r_y
 441 shall not be taken as less than 0.3 times the overall depth
 442 of the cross section in the plane of buckling.

443 2b. Highly Ductile Members

444 In addition to the requirements of Sections D1.2a(1)(i) and (ii), and
 445 D1.2a(2)(i) and (ii), the bracing of *highly ductile members* shall have a
 446 maximum spacing of $L_b = 0.086r_y E / F_y$. For concrete-encased and
 447 concrete-filled composite members, the properties of the steel section
 448 shall be used, except that in the calculation for concrete encased
 449 composite members, r_y shall not be taken as less than 0.3 times the
 450 overall depth of the cross section in the plane of buckling.

451 2c. Special Bracing at Plastic Hinge Locations

452 Special bracing shall be located adjacent to expected plastic hinge
 453 locations where required by Chapters E, F, G or H.

454 (1) For structural steel members, such bracing shall satisfy the
 455 following requirements:

456 (i) Both flanges of members shall be laterally braced or the
 457 member cross section shall be torsionally braced..

458 (ii) The *required strength* of lateral bracing of each flange
 459 provided adjacent to plastic hinges shall be:

$$460 \quad P_u = 0.06 R_y Z F_y / h_o \text{ (LRFD)} \quad (D1-4a)$$

461 or

$$462 \quad P_a = (0.06/1.5) R_y Z F_y / h_o \text{ (ASD)} \quad (D1-4b)$$

463 where

464 h_o = distance between flange centroids, in.

465 The *required strength* of torsional bracing provided
 466 adjacent to plastic hinges shall be:

$$467 \quad M_u = 0.06 R_y Z F_y \text{ (LRFD)} \quad (D1-5a)$$

468 or

469 $M_a = 0.06 R_y Z F_y$ (ASD) (D1-5b)

470 (iii) The required bracing stiffness shall meet the requirements
471 of Appendix 6 of the *Specification* for lateral or torsional
472 bracing of beams with $C_d=1.0$ and where the expected
473 flexural strength of the member shall be:

474 $M_r = M_u = R_y Z F_y$ (LRFD) (D1-6a)

475 or

476 $M_r = M_a = R_y Z F_y / 1.5$ (ASD) (D1-6b)

477 (2) For concrete-encased and concrete-filled composite members,
478 such bracing shall satisfy the following requirements:

479 (i) Both flanges of members shall be laterally braced or the
480 member cross section shall be torsionally braced.

481 (ii) The *required strength* of lateral bracing provided adjacent
482 to plastic hinges shall be

483 $P_u = 0.06 M_p / h_o$ (D1-7)

484 of the member, where M_p is determined in accordance
485 with Section G2.6c.

486 The required strength for torsional bracing provided
487 adjacent to plastic hinges shall be $M_u = 0.06 M_p$ of the
488 member.

489 (iii) The required bracing stiffness shall meet the requirements
490 of Appendix 6 of the *Specification* for lateral or torsional
491 bracing of beams where $M_r = M_u = M_p$ of the member is
492 determined in accordance with Section G2.6c, and $C_d =$
493 1.0.

494 3. Protected Zones

495 Discontinuities from fabrication and erection procedures and from other
496 attachments are prohibited in the area of a member or a connection
497 element designated as a *protected zone* by these *Provisions* or
498 ANSI/AISC 358. See Section I2.1 for specific limitations.

499 Exception: Welded shear studs and other connections are permitted in
500 protected zones when designated in ANSI/AISC 358, or as otherwise
501 determined with a connection prequalification in accordance with Section
502 J1, or as determined in a program of qualification testing in accordance
503 with Sections J2 and J3.

504 **4. Columns**

505 Columns in moment frames, braced frames and shear walls shall satisfy
506 the requirements of this section.

507 **4a. Required Strength**

508
509 The *required strength* of columns in the SFRS shall satisfy both of the
510 following:

511
512 (1) The load effect resulting from the analysis requirements for the
513 applicable system per Sections E, F, G and H.

514
515 (2) The compressive axial strength and tensile strength as determined
516 using the load combinations stipulated in the applicable building code
517 including the amplified seismic load. It is permitted to neglect
518 applied moments in this determination unless the moment results
519 from a load applied to the column between points of lateral support.
520 The required axial compressive strength and tensile strength need not
521 exceed either of the following:

522 (a) The maximum load transferred to the column by the
523 system, including the effects of material overstrength and
524 strain hardening.

525 (b) The forces corresponding to the resistance of the
526 foundation to overturning uplift.

527 **4b. Encased Composite Columns**

528 Encased composite columns shall satisfy the requirements of
529 *Specification* Chapter I, in addition to the requirements of this section.
530 Additional requirements, as specified for *moderately ductile members*
531 and *highly ductile members* in Sections D1.4b(1) and (2) shall apply as
532 required in the descriptions of the composite seismic systems in Chapters
533 G and H.

534 **(1) Moderately Ductile Members**

535 Encased composite columns used as moderately ductile members shall
536 satisfy the following requirements:

537 (i) The maximum spacing of transverse reinforcement at the
538 top and bottom shall be the least of the following:

539 (a) one-half the least dimension of the section

540 (b) 8 longitudinal bar diameters

541 (c) 24 tie bar diameters

- 542 (d) 12 in. (300 mm)
- 543 (ii) This spacing shall be maintained over a vertical distance
544 equal to the greatest of the following lengths, measured from each
545 joint face and on both sides of any section where flexural yielding
546 is expected to occur:
- 547 (a) one-sixth the vertical clear height of the column
- 548 (b) the maximum cross-sectional dimension
- 549 (c) 18 in. (450 mm)
- 550 (iii) Tie spacing over the remaining column length shall not
551 exceed twice the spacing defined in Section D1.4b(1)(i).
- 552 (iv) Splices and end bearing details for *encased composite*
553 *columns* in ordinary seismic systems shall meet the requirements
554 of the *Specification* and ACI 318 Section 7.8.2. The design shall
555 comply with ACI 318 Sections 21.1.6 and 21.1.7. The design
556 shall consider any adverse behavioral effects due to abrupt
557 changes in either the member stiffness or the nominal *tensile*
558 *strength*. Locations of such adverse behavioral effects shall
559 include transitions to reinforced concrete sections without
560 embedded structural steel members, transitions to bare structural
561 steel sections, and column bases.
- 562 (v) Welded wire fabric shall be prohibited as transverse
563 reinforcement in *moderately ductile members*.

564 (2) Highly Ductile Members

565 Encased composite columns used as highly ductile members shall
566 satisfy Section D1.4b(1) in addition to the following
567 requirements:

- 568 (i) Longitudinal *load-carrying reinforcement* shall meet the
569 requirements of ACI 318 Section 21.6.3.
- 570 (ii) Transverse reinforcement shall be hoop reinforcement as
571 defined in ACI 318 Chapter 21 and shall satisfy the
572 following requirements:
- 573 (1) The minimum area of tie reinforcement, A_{sh} , shall
574 be:

$$A_{sh} = 0.09h_{cc} s \left(1 - \frac{F_y A_s}{P_n} \right) \left(\frac{f'_c}{F_{yh}} \right) \quad (D1-8)$$

576 where

577	h_{cc}	=	cross-sectional dimension of the
578			confined core measured center-to-
579			center of the tie reinforcement, in.
580			(mm)
581	s	=	spacing of transverse
582			reinforcement measured along the
583			longitudinal axis of the structural
584			member, in. (mm)
585	F_y	=	specified minimum yield stress of
586			the structural <i>steel core</i> , ksi (MPa)
587	A_s	=	cross-sectional area of the
588			structural steel core, in. ² (mm ²)
589	P_n	=	nominal compressive strength of
590			the composite column calculated
591			in accordance with the
592			<i>Specification</i> , kips (N)
593	f'_c	=	specified compressive strength of
594			concrete, ksi (MPa)
595	F_{yh}	=	specified minimum yield stress of
596			the ties, ksi (MPa)

597 Equation D1-8 need not be satisfied if the *nominal*
 598 *strength* of the concrete-encased structural steel
 599 section alone is greater than the *load effect* from a
 600 load combination of $1.0D+0.5L$.

601 (2) The maximum spacing of transverse
 602 reinforcement along the length of the column shall
 603 be the lesser of six longitudinal load-carrying bar
 604 diameters or 6 in. (150 mm).

605 (3) When specified in Sections D1.4b(1)(ii), (iii) or
 606 (iv), the maximum spacing of transverse
 607 reinforcement along the member length shall be
 608 the lesser of one-fourth the least member
 609 dimension or 4 in. (100 mm). Confining
 610 reinforcement shall be spaced not more than 14 in.
 611 (350 mm) on center in the transverse direction.

612 (iii) *Encased composite columns* in braced frames with
 613 required compressive strengths, without consideration of
 614 the amplified seismic loads, greater than $0.2P_n$ shall have
 615 transverse reinforcement as specified in Section
 616 D1.4b(2)(ii)(3) over the total element length. This

617 requirement need not be satisfied if the *nominal strength*
618 of the concrete-encased steel section alone is greater than
619 the *load effect* from a load combination of $1.0D+0.5L$.

620 (iv) *Composite columns* supporting reactions from
621 discontinued stiff members, such as walls or braced
622 frames, shall have transverse reinforcement as specified in
623 Section D1.4b(2)(ii)(3) over the full length beneath the
624 level at which the discontinuity occurs if the required
625 compressive strengths, without consideration of the
626 amplified seismic loads, exceeds $0.1P_n$. Transverse
627 reinforcement shall extend into the discontinued member
628 for at least the length required to develop full yielding in
629 the concrete-encased steel section and longitudinal
630 reinforcement. This requirement need not be satisfied if
631 the *nominal strength* of the concrete-encased steel section
632 alone is greater than the *load effect* from a load
633 combination of $1.0D+0.5L$.

634 (v) *Encased composite columns* used in a C-SMF shall satisfy
635 the following requirements:

636 (1) Transverse reinforcement shall meet the
637 requirements in Section D1.4b(2)(ii) at the top
638 and bottom of the column over the region
639 specified in Section D1.4b(1).

640 (2) The strong-column/weak-beam design
641 requirements in Section G3.4a shall be satisfied.
642 Column bases shall be detailed to sustain inelastic
643 flexural hinging.

644 (3) The *required shear strength* of the column shall
645 meet the requirements of ACI 318 Section
646 21.6.5.1.

647 (vi) When the column terminates on a footing or mat
648 foundation, the transverse reinforcement as specified in
649 this section shall extend into the footing or mat at least 12
650 in. (300 mm). When the column terminates on a wall, the
651 transverse reinforcement shall extend into the wall for at
652 least the length required to develop full yielding in the
653 concrete-encased shape and longitudinal reinforcement.
654

655 4c. Filled Composite Columns

656 This section applies to columns that meet the limitations of *Specification*
657 Section I2.2. Such columns shall be designed to meet the requirements of
658 *Specification* Chapter I, except that the *nominal shear strength* of the

659 composite column shall be the *nominal shear strength* of the structural
660 steel section alone, based on its effective shear area.

661 **5. Composite Slab Diaphragms**

662
663 The design of composite floor and roof slab diaphragms for seismic
664 effects shall meet the following requirements.

665 **5a. Load Transfer**

666 Details shall be provided to transfer loads between the diaphragm and
667 boundary members, collector elements, and elements of the horizontal
668 framing system.

669 **5b. Nominal Shear Strength**

670 The nominal in-plane shear strength of composite diaphragms and
671 concrete slab on steel deck diaphragms shall be taken as the nominal
672 shear strength of the reinforced concrete above the top of the steel deck
673 ribs in accordance with ACI 318 excluding Chapter 22. Alternatively, the
674 composite diaphragm nominal shear strength shall be determined by in-
675 plane shear tests of concrete-filled diaphragms.

676 **D2. CONNECTIONS**

677 **1. General**

678 Connections, joints and fasteners that are part of the SFRS shall comply
679 with *Specification* Chapter J, and with the additional requirements of this
680 section.

681 Splices and bases of columns that are not part of the SFRS shall meet the
682 requirements of Sections D2.5a, D2.5c and D2.6.

683 Where *protected zones* are designated in connection elements by these
684 *Provisions* or ANSI/AISC 358, they shall meet the requirements of
685 Sections D1.3 and I2.1.

686 **2. Bolted Joints**

687 Bolted joints shall satisfy the following requirements:

688 (i) All bolts shall be installed as pretensioned high strength bolts.
689 Faying surfaces shall meet the requirements for slip-critical
690 faying surfaces in accordance with *Specification* Section J3.8
691 with a Class A or better surface.

692
693 Exceptions: Connection surfaces are permitted to have coatings
694 with a slip coefficient less than that of a Class A faying surface
695 for the following:

696
697
698
699
700
701
702
703

(1) End plate moment connections conforming to the requirements of Section E1, or ANSI/AISC 358

(2) Bolted joints where the load effects due to seismic are transferred either by tension in bolts or by compression bearing but not by shear in bolts.

704
705
706
707

User Note: The designation of connections as slip critical will meet the requirements for pretensioned bolts and slip-critical faying surfaces but will result in additional requirements for inspection.

708
709

- (ii) Bolt holes shall be standard holes or short-slotted holes perpendicular to the applied load.

710
711
712
713
714
715

Exception: For diagonal braces specified in Sections F1, F2, F3 and F4, oversized holes are permitted in one connection ply only when the connection is designed as a slip-critical joint for the required brace connection strength in Sections F1, F2, F3 and F4, and where slip is considered a strength limit state per *Specification* Section J3.8.

716
717
718
719
720
721
722
723

User Note: Diagonal brace connections with oversized holes must also satisfy other limit states including bolt bearing and bolt shear for the required strength of the connection as defined in Sections F1, F2, F3 and F4. Alternative hole types are permitted if designated in the ANSI/AISC 358, or if otherwise determined in a connection prequalification in accordance with Section J1, or if determined in a program of qualification testing in accordance with Section J2 or Section J3.

724
725
726
727

- (iii) The available shear strength of bolted joints using standard holes shall be calculated as that for bearing-type joints in accordance with *Specification* Sections J3.6 and J3.10. The nominal bearing strength at bolt holes shall not be taken greater than $2.4dtF_u$.

728
729

- (iv) Bolts and welds shall not be designed to share force in a joint or the same force component in a connection.

730
731
732
733
734
735
736

User Note: A member force, such as a diagonal brace axial force, must be resisted at the connection entirely by one type of joint (in other words, either entirely by bolts or entirely by welds). A connection in which bolts resist a force that is normal to the force resisted by welds, such as a moment connection in which welded flanges transmit flexure and a bolted web transmits shear, is not considered to be sharing the force.

737 3. Welded Joints

738 Welded joints shall be designed in accordance with Chapter J of the
739 *Specification*. Welding shall be performed in accordance with Section
740 I2.3. Welding materials shall be in accordance with Section A3.4.

741 4. Continuity Plates and Stiffeners

742 The design of *continuity plates* and stiffeners located in the webs of
743 rolled shapes shall allow for the reduced contact lengths to the member
744 flanges and web based on the corner clip sizes in Section I2.4.

745 5. Column Splices

746 5a. Location of Splices

747 For all building columns, including those not designated as part of the
748 SFRS, column splices shall be located 4 ft (1.2 m) or more away from the
749 beam-to-column flange connections. When the column clear height
750 between beam-to-column flange connections is less than 8 ft (2.4 m),
751 splices shall be at half the clear height.

752 Exceptions:

753 1) Column splices with webs and flanges joined by complete-joint-
754 penetration groove welds are permitted to be located closer to the
755 beam-to-column flange connections, but not less than the depth of
756 the column.

757 2) Splices in composite columns.

758 **User Note:** Where possible, splices should be located at least 4 ft (1.2m)
759 above the finished floor elevation to permit installation of perimeter
760 safety cables prior to erection of the next tier and to improve
761 accessibility.

762 5b. Required Strength

763 The *required strength* of column splices in the SFRS shall be the greater
764 of:

765 (a) The required strength of the columns, including that determined
766 from Chapters E, F, G and H, and Section D1.4a; or,

767 (b) The required strength determined using the *load combinations*
768 stipulated in the *applicable building code* including the *amplified*
769 *seismic load*. The required strength need not exceed the
770 maximum loads that can be transferred to the splice by the
771 system.

772 In addition, welded column splices in which any portion of the column is
773 subject to a calculated net tensile load effect determined using the load
774 combinations stipulated in the applicable building code, including the

775 amplified seismic load, shall satisfy all of the following requirements:

- 776 (1) The *available strength* of partial-joint-penetration (PJP) groove
777 welded joints, if used, shall be at least equal to 200% of the
778 required strength.
- 779 (2) The available strength for each flange splice shall be at least
780 equal to $0.5R_yF_yb_{ftf}$ (LRFD) or $(0.5/1.5)R_yF_yb_{ftf}$ (ASD), as
781 applicable, where R_yF_y is the *expected yield stress* of the column
782 material and b_{ftf} is the area of one flange of the smaller column
783 connected.
- 784 (3) Where welded butt joint splices are made with complete-joint-
785 penetration groove welds, tapered transitions are required
786 between flanges of unequal thickness or width per AWS D1.8
787 Clause 4.2 when tension stress at any location in the smaller
788 flange exceeds $0.30F_y$ (LRFD) or $0.20F_y$ (ASD).

789

790 **User Note:** Butt splices subject to tension greater than 33% of the
791 expected yield strength under any load combination should have
792 tapered transitions. The stress concentration at a nontapered
793 transition, with a 90° corner, could cause local yielding when the
794 tensile stress exceeds 33% of yield. Lower levels of stress would
795 be acceptable with the stress concentration from a nontapered
796 transition.

797 **5c. Required Shear Strength**

798 For all building columns including those not designated as part of the
799 SFRS, the required shear strength of column splices with respect to both
800 orthogonal axes of the column shall be M_{pc}/H (LRFD) or $M_{pc}/1.5H$
801 (ASD), as applicable, where M_{pc} is the lesser nominal plastic flexural
802 strength of the column sections for the direction in question, and H is the
803 story height.

804 The required shear strength of splices of columns in the SFRS shall be
805 the greater of the above requirement or the required shear strength
806 determined per Section D2.5b(a) and (b).

807 **5d. Structural Steel Splice Configurations**

808 Structural steel column splices are permitted to be either bolted or
809 welded, or welded to one column and bolted to the other. Splice
810 configurations shall meet any specific requirements in Chapters E, F, G
811 or H.

812 Where columns of the SFRS use bolted web splices, plates or channels
813 shall be used on both sides of the column web.

814 For welded butt joint splices made with groove welds, weld tabs shall be
815 removed in accordance with AWS D1.8 Clause 6.10. Steel backing of
816 groove welds need not be removed.

817 **5e. Splices in Encased Composite Columns**

818 For encased composite columns, column splices shall conform to Section
819 D1.4b and ACI 318 Section 21.6.3.2.

820 **6. Column Bases**

821 The *required strength* of column bases, including those that are not part
822 of the SFRS, shall be calculated in accordance with this section

823 **User Note:** The only requirement for bases of columns that are not part
824 of the SFRS is the required shear strength per Section D2.5c.

825 The available strength of steel elements at the column base, including
826 base plates, anchor rods, stiffening plates, and shear lug elements shall be
827 in accordance with the *Specification*.

828 Where columns are welded to base plates with groove welds, weld tabs
829 and weld backing shall be removed.

830 The available strength of concrete elements at the column base, including
831 anchor rod embedment and reinforcing steel, shall be in accordance with
832 ACI 318, Appendix D.

833 **User Note:** When using concrete reinforcing steel as part of the
834 anchorage embedment design, it is important to understand the anchor
835 failure modes and provide reinforcement that is developed on both sides
836 of the expected failure surface. See ACI 318, Appendix D, including
837 Commentary.

838 **6a. Required Axial Strength**

839 The *required axial strength* of column bases, including their attachment
840 to the foundation, shall be the summation of the vertical components of
841 the required connection *strengths* of the steel elements that are connected
842 to the column base, but not less than the greater of:

843 (a) The column axial load calculated using the *load combinations* of the
844 *applicable building code*, including the *amplified seismic load*.

845 (b) The required axial strength for column splices, as prescribed in
846 Section D2.5.

847 **User Note:** The vertical components can include both the axial load
848 from columns and the vertical component of the axial load from diagonal
849 members framing into the column base. Section D2.5 includes references
850 to Section D1.4a and Chapters E, F, G and H. Where diagonal braces

851 frame to both sides of a column, the effects of compression brace
852 buckling should be considered in the summation of vertical components.
853 See Section F2.3.

854 **6b. Required Shear Strength**

855 The *required shear strength* of column bases, including their attachments
856 to the foundations, shall be the summation of the horizontal component of
857 the required strengths of the steel elements that are connected to the
858 column base as follows:

859 (1) For diagonal braces, the horizontal component shall be
860 determined from the required strength of diagonal brace
861 connections for the SFRS.

862 (2) For columns, the horizontal component shall be equal to the
863 required shear strength for column splices prescribed in Section
864 D2.5c.

865 Exception: Single story columns with simple end connections at
866 both ends need not comply with D2.6b(2).

867 **User Note:** The horizontal components can include the shear load from
868 columns and the horizontal component of the axial load from diagonal
869 members framing into the column base. Section D2.5 includes references
870 to Section D1.4a and Chapters E, F, G and H.

871 **6c. Required Flexural Strength**

872 The *required flexural strength* of column bases, including their
873 attachment to the foundation, shall be the summation of the required
874 connection strengths of the steel elements that are connected to the
875 column base as follows:

876 (1) For diagonal braces, the *required flexural strength* shall be at
877 least equal to the required flexural strength of diagonal brace
878 connections.

879 (2) For columns, the *required flexural strength* shall be at least equal
880 to the lesser of the following:

881 (a) $1.1R_yF_yZ$ (LRFD) or $(1.1/1.5)R_yF_yZ$ (ASD), as applicable,
882 of the column, or

883 (b) the moment calculated using the *load combinations* of the
884 *applicable building code*, including the *amplified seismic*
885 *load*.

886 **User Note:** Moments at column to column base connections designed as
887 simple connections may be ignored provided the connection has
888 sufficient ductility to accommodate the rotation resulting from the lateral
889 drift of the structure as calculated per the applicable building code.

890 7. Composite Connections

891 This section applies to connections in buildings that utilize composite or
892 dual steel and concrete systems wherein seismic load is transferred
893 between structural steel and reinforced concrete components. Methods
894 for calculating the connection strength shall meet the requirements in this
895 section. Unless the connection strength is determined by analysis or
896 testing, the models used for design of connections shall satisfy the
897 following requirements:

898 (1) Force shall be transferred between structural steel and
899 reinforced concrete through:

900 (a) direct bearing from internal bearing mechanisms;

901 (b) shear connection;

902 (c) shear friction with the necessary clamping force
903 provided by reinforcement normal to the plane of shear
904 transfer; or

905 (d) a combination of these means.

906 The contribution of different mechanisms is permitted
907 to be combined only if the stiffness and deformation
908 capacity of the mechanisms are compatible. Any
909 potential bond strength between structural steel and
910 reinforced concrete shall be ignored for the purpose of
911 the connection force transfer mechanism.

912 (2) The nominal bearing and shear-friction strengths shall meet the
913 requirements of ACI 318 Chapters 10 and 11. Unless a higher
914 strength is substantiated by cyclic testing, the nominal bearing and
915 shear-friction strengths shall be reduced by 25% for the composite
916 seismic systems described in Sections G3, H2, H3, H5 and H6.

917 (3) *Face bearing plates* consisting of stiffeners between the flanges
918 of steel beams shall be provided when beams are embedded in
919 reinforced concrete columns or walls.

920 (4) The *nominal shear strength* of concrete-encased steel panel-zones
921 in beam-to-column connections shall be calculated as the sum of
922 the *nominal strengths* of the structural steel and confined
923 reinforced concrete shear elements as determined in Section
924 E3.6e and ACI 318 Section 21.7, respectively.

925 (5) Reinforcement shall be provided to resist all tensile forces in
926 reinforced concrete components of the connections. Additionally,

927 the concrete shall be confined with transverse reinforcement. All
 928 reinforcement shall be fully developed in tension or compression,
 929 as applicable, beyond the point at which it is no longer required to
 930 resist the forces. Development lengths shall be determined in
 931 accordance with ACI 318 Chapter 12. Additionally, development
 932 lengths for the systems described in Sections G3, H2, H3, H5 and
 933 H6 shall meet the requirements of ACI 318 Section 21.7.5.

934 (6) Composite connections shall satisfy the following additional
 935 requirements:

936 (i) When the slab transfers horizontal diaphragm forces, the
 937 slab reinforcement shall be designed and anchored to
 938 carry the in-plane tensile forces at all critical sections in
 939 the slab, including connections to collector beams,
 940 columns, diagonal braces and walls.

941 (ii) For connections between structural steel or *composite*
 942 *beams* and reinforced concrete or *encased composite*
 943 *columns*, transverse hoop reinforcement shall be provided
 944 in the connection region of the column to meet the
 945 requirements of ACI 318 Section 21.7, except for the
 946 following modifications:

947 (1) Structural steel sections framing into the
 948 connections are considered to provide
 949 confinement over a width equal to that of *face*
 950 *bearing plates* welded to the beams between the
 951 flanges.

952 (2) Lap splices are permitted for perimeter ties when
 953 confinement of the splice is provided by *face*
 954 *bearing plates* or other means that prevents
 955 spalling of the concrete cover in the systems
 956 described in Sections G1, G2, H1 and H4.

957 (3) The longitudinal bar sizes and layout in reinforced
 958 concrete and *composite columns* are detailed to
 959 minimize slippage of the bars through the beam-
 960 to-column connection due to high force transfer
 961 associated with the change in column moments
 962 over the height of the connection.

963 8. Steel Anchors

964 Where steel headed stud anchors or welded reinforcing bar anchors are
 965 part of the intermediate or special SFRS of Sections G2, G3, G4, H2, H3,
 966 H5 and H6, their shear and tensile strength shall be reduced by 25%
 967 from the specified strengths given in *Specification* Chapter I.
 968

969 **User Note:** The 25% reduction is not necessary for gravity and collector
970 components in structures with intermediate or special seismic force
971 resisting systems designed for the amplified seismic load.

972 **D3. DEFORMATION COMPATIBILITY OF NON-SFRS MEMBERS**
973 **AND CONNECTIONS**

974 Where deformation compatibility of members and connections that are
975 not part of the *seismic force resisting system* (SFRS) is required by the
976 *applicable building code*, these elements shall be designed to resist the
977 combination of gravity load effects and the effects of deformations
978 occurring at the story drift calculated in accordance with the applicable
979 building code.

980 **User Note:** SEI/ASCE 7 stipulates the above requirement for structural
981 steel members and connections. Flexible shear connections that allow
982 member end rotations per Section J1.2 of the *Specification* shall be
983 considered to meet these requirements. Inelastic deformations are
984 permitted in connections or member provided they are self limiting and
985 do not create instability in the member. See Commentary for further
986 discussion.

987

988 **D4. H PILES**

989 **1. Design Requirements**

990 Design of H-piles shall comply with the requirements of the *Specification*
991 regarding design of members subjected to combined loads. H-piles shall
992 meet the requirements for *highly ductile members* of Section D1.1

993 **2. Battered H-Piles**

994 If battered (sloped) and vertical piles are used in a pile group, the vertical
995 piles shall be designed to support the combined effects of the dead and
996 live loads without the participation of the battered piles.

997 **3. Tension**

998 Tension in each pile shall be transferred to the pile cap by mechanical
999 means such as shear keys, reinforcing bars or studs welded to the
1000 embedded portion of the pile.

1001 **4. Protected Zone**

1002
1003 At each pile, the length equal to the depth of the pile cross section located
1004 directly below the bottom of the pile cap shall be designated as a
1005 *protected zone* meeting the requirements of Sections D1.3 and I2.1.

1006

1007

CHAPTER E

1008

MOMENT-FRAME SYSTEMS

1009

1010 This chapter provides the basis of design and the requirements pertaining to
1011 system configuration, members and connections for steel ordinary, intermediate,
1012 special and special truss moment frames, and for steel cantilever column systems.

1013 The chapter is organized as follows:

1014

- 1015 E1. Ordinary Moment Frames
- 1016 E2. Intermediate Moment Frames
- 1017 E3. Special Moment Frames
- 1018 E4. Special Truss Moment Frames
- 1019 E5. Ordinary Cantilever Column Systems
- 1020 E6. Special Cantilever Column Systems

1021

1022 **E1. ORDINARY MOMENT FRAMES**

1023 **1. Scope**

1024 *Ordinary moment frames* (OMF) of structural steel shall be designed in
1025 conformance with this section.

1026 **2. Basis of Design**

1027 OMF shall be designed to withstand minimal inelastic drift.

1028 **3. Analysis**

1029 There are no additional requirements beyond those in the *Specification*
1030 and the *applicable building code*.

1031 **4. System Requirements**

1032 There are no additional requirements beyond the *Specification*.

1033 **5. Members**

1034 There are no limitations on width-thickness ratios of members for OMF,
1035 beyond those in the *Specification*. There are no requirements for lateral
1036 bracing of members or joints in OMF, beyond those in the *Specification*.
1037 There are no designated *protected zones* for OMF members. Structural
1038 steel beams in OMF are permitted to be composite with a reinforced
1039 concrete slab to resist gravity loads.

1040 Trusses shall be permitted as beams in OMF. The required strength of
1041 truss members shall be based on the maximum forces that can be

1042 delivered to the truss by the system, including overstrength and strain
1043 hardening. Truss connections shall be fully-restrained (FR) connections.

1044 **6. Connections**

1045 Beam-to-column connections are permitted to FR or partially-restrained
1046 (PR) moment connections in accordance with this section.

1047 **6a. Demand Critical Welds**

1048
1049 Complete-joint-penetration welds of beam flanges to columns shall be
1050 *demand critical welds* in accordance with Section A3.4b.

1051 **User Note:** Single-sided partial-joint-penetration groove welds and
1052 single-sided fillet welds should be avoided for resisting tensile forces in
1053 connections.

1054 **6b. Requirements for FR Moment Connections**

1055 FR moment connections that are part of the *seismic force resisting system*
1056 (SFRS) shall meet at least one of the following requirements:

1057 (a) FR moment connections shall be designed for a *required flexural*
1058 *strength* that is equal to $1.1R_yM_p$ (LRFD) or $(1.1/1.5)R_yM_p$ (ASD), as
1059 appropriate, of the beam.

1060 The *required shear strength*, V_u or V_a , as appropriate, of the connection
1061 shall be determined using the following quantity for the earthquake load
1062 effect E

$$1063 \quad E = 2[1.1 R_y M_p] / L_{cf} \quad (E1-1)$$

1064 where **DRAFT**

1065 R_y = ratio of expected yield stress to the specified
1066 minimum yield stress, F_y

1067 M_p = $F_y Z$, kip-in. (N-mm)

1068 L_{cf} = clear length of beam, in.
1069 (mm)

1070 (b) FR moment connections shall be designed for a required flexural
1071 strength and a required shear strength equal to the maximum moment and
1072 corresponding shear that can be transferred to the connection by the
1073 system, including the effects of material overstrength and strain
1074 hardening.

1075 **User Note:** Factors that may limit the maximum moment and
1076 corresponding shear that can be transferred to the connection include:

1077 (1) The strength of the columns, and

1078 (2) The resistance of the foundations to uplift

1079 For options (1) and (2) above, continuity plates should be provided as
1080 required by Sections J10.1, J10.2 and J10.3 of the *Specification*. The
1081 bending moment used to check for continuity plates should be the same
1082 bending moment used to design the beam-to-column connection; in other
1083 words, either $1.1R_yM_p$ (LRFD) or $(1.1/1.5)R_yM_p$ (ASD) or the maximum
1084 moment that can be transferred to the connection by the system.

1085 (c) FR moment connections between wide flange beams and the
1086 flange of wide flange columns shall either meet the requirements of
1087 Section E2.6 or E3.6, or shall meet the following requirements:

1088 (1) All welds at the beam-to-column connection shall meet the
1089 requirements of Chapter 3 of ANSI/AISC 358.

1090 (2) Beam flanges shall be connected to column flanges using
1091 complete-joint-penetration (CJP) groove welds.

1092 (3) The shape of weld access holes shall be in accordance with
1093 Section 6.9.1.2 of AWS D1.8. Weld access hole quality requirements
1094 shall be in accordance with Clause 6.9.2 of AWS D1.8.

1095 (4) Continuity plates shall meet the requirements of Section
1096 2.4.4 of ANSI/AISC 358, including Supplement No. 1.
1097 Continuity plates shall be at least as wide as the flanges of
1098 the wider connected beam.

1099
1100 Exception: The welded joints of the continuity plates to the
1101 column flanges are permitted to be complete-joint-
1102 penetration groove welds, two-sided partial-joint-
1103 penetration groove welds, or two-sided fillet welds. The
1104 *required strength* of these joints shall not be less than the
1105 *available strength* of the contact area of the plate with the
1106 column flange.

1107 (5) The beam web shall be connected to the column flange using
1108 either a CJP groove weld extending between weld access holes, or using
1109 a bolted single plate shear connection designed for *required shear*
1110 *strength* per Equation E1-1.

1111 **User Note:** For FR moment connections, panel zone shear strength
 1112 should be checked according to Section J10.6 of the *Specification*. The
 1113 required shear strength of the panel zone should be based on the beam
 1114 end moments computed from the load combinations stipulated by the
 1115 *applicable building code*, not including the amplified seismic load.

1116 **6c. Requirements for PR Moment Connections**

1117 PR moment connections are permitted when the following requirements
 1118 are met:

1119 (1) Such connections shall be designed for the maximum moment
 1120 and shear from the applicable load combinations as described in
 1121 Sections B2 and B3.

1122 (2) The stiffness, strength and deformation capacity of PR moment
 1123 connections shall be considered in the design, including the effect on
 1124 overall frame stability.

1125 (3) The nominal flexural strength of the connection, $M_{n,PR}$ shall be
 1126 no less than 50% of M_p of the connected beam.

1127 Exception: For one-story structures, $M_{n,PR}$ shall be no less than
 1128 50% of M_p of the connected column.

1129 (4) For PR moment connections, V_u or V_a as appropriate, shall be
 1130 determined per Section E1.6b(a) with M_p in Equation E1-1 taken as
 1131 $M_{n,PR}$.

1132 **E2. INTERMEDIATE MOMENT FRAMES**

1133 **1. Scope**

1134 *Intermediate moment frames* (IMF) of structural steel shall be designed in
 1135 conformance with this section.

1136 **2. Basis of Design**

1137 IMF shall be designed to provide limited inelastic drift capacity through
 1138 flexural yielding of the IMF beams, , and columns, and shear yielding of
 1139 the column panel zones. Design of connections of beams to columns,
 1140 including panel zones and *continuity plates*, shall be based on connection
 1141 tests that provide the performance required by Section E2.6b, and
 1142 demonstrate this conformance as required by Section E2.6c.

1143 **User Note:** Panel zone shear strength should be checked according to
 1144 Section J10.6 of the *Specification*. The required shear strength of the
 1145 panel zone should be based on the beam end moments computed from the

1146 load combinations stipulated by the *applicable building code*, not
1147 including the amplified seismic load.

1148 **3. Analysis**

1149 There are no additional requirements beyond those in the *Specification*
1150 and the *applicable building code*.

1151 **4. System Requirements**

1152 **4a. Stability Bracing of Beams**

1153 Beams shall be braced to meet the requirements for *moderately ductile*
1154 *members* in Section D1.2.

1155 In addition, member braces shall be placed near concentrated forces,
1156 changes in cross-section, and other locations where analysis indicates that
1157 a plastic hinge will form during inelastic deformations of the IMF. The
1158 placement of lateral bracing shall be consistent with that documented for
1159 a *prequalified connection* designated in ANSI/AISC 358, including
1160 Supplement No. 1, or as otherwise determined in a connection
1161 prequalification in accordance with Section J1, or in a program of
1162 qualification testing in accordance with Section J2.

1163 The *required strength* of lateral bracing provided adjacent to plastic
1164 hinges shall be as required by Section D1.2c.

1165 **5. Members**

1166 **5a. Width-Thickness Limitations**

1167 Beam and column members shall satisfy the requirements for *moderately*
1168 *ductile members*, as defined in Section D1.1b, unless otherwise qualified
1169 by tests.

1170 **5b. Beam Flanges**

1171 Abrupt changes in beam flange area shall not be permitted in plastic
1172 hinge regions. The drilling of flange holes or trimming of beam flange
1173 width shall not be permitted unless testing or qualification demonstrates
1174 that the resulting configuration can develop stable plastic hinges. The
1175 configuration shall be consistent with a *prequalified connection*
1176 designated in ANSI/AISC 358, including Supplement No. 1, or as
1177 otherwise determined in a connection prequalification in accordance with
1178 Section J1, or in a program of qualification testing in accordance with
1179 Section J2.

1180 **5c. Protected Zones**

1181 The region at each end of the beam subject to inelastic straining shall be
1182 designated as a *protected zone*, and shall meet the requirements of
1183 Section D1.3. The extent of the protected zone shall be as designated in
1184 ANSI/AISC 358, including Supplement No. 1, or as otherwise
1185 determined in a connection prequalification in accordance with Section
1186 J1, or as determined in a program of qualification testing in accordance
1187 with Section J2.

1188 **User Note:** The plastic hinging zones at the ends of IMF beams should
1189 be treated as protected zones. The plastic hinging zones should be
1190 established as part of a prequalification or qualification program for the
1191 connection, per Section E2.6c. In general, for unreinforced connections,
1192 the protected zone will extend from the face of the column to one half of
1193 the beam depth beyond the plastic hinge point.

1194 **5d. Beams**

1195 Structural steel beams in IMF are permitted to be composite with a
1196 reinforced concrete slab to resist gravity loads.

1197 **6. Connections**

1198 **6a. Demand Critical Welds**

1199 Unless otherwise designated by ANSI/AISC 358, including Supplement
1200 No. 1, or otherwise determined in a connection prequalification in
1201 accordance with Section J1, or as determined in a program of
1202 qualification testing in accordance with Section J2, complete-joint-
1203 penetration groove welds of beam flanges and beam webs to columns
1204 shall be *demand critical* welds as described in Section A3.4b.

1205 **User Note:** For the designation of *demand critical welds*, standards such
1206 as ANSI/AISC 358 and tests addressing specific connections and joints
1207 should be used in lieu of the more general terms of these *Provisions*.
1208 Where these *Provisions* indicate that a particular weld is designated
1209 demand critical, but the more specific standard or test does not make such
1210 a designation, the more specific standard or test should govern. Likewise,
1211 these standards and tests may designate welds as demand critical that are
1212 not identified as such by these *Provisions*.

1213 **6b. Beam-to-Column Connection Requirements**

1214 Beam-to-column connections used in the SFRS shall satisfy the following
1215 requirements:

- 1216 (1) The connection shall be capable of sustaining an *interstory drift*
1217 *angle* of at least 0.02 rad.

1218 (2) The *measured flexural resistance* of the connection, determined
 1219 at the column face, shall equal at least $0.80M_p$ of the connected
 1220 beam at an *interstory drift angle* of 0.02 rad.

1221 **6c. Conformance Demonstration**

1222 Beam-to-column connections used in the SFRS shall satisfy the
 1223 requirements of Section E2.6b by one of the following:

1224 (a) Use of IMF connections designed in accordance with ANSI/AISC
 1225 358, including Supplement No. 1.

1226 (b) Use of a connection prequalified for IMF in accordance with
 1227 Section J1.

1228 (c) Provision of qualifying cyclic test results in accordance with
 1229 Section J2. Results of at least two cyclic connection tests shall be
 1230 provided and are permitted to be based on one of the following:

1231 (i) Tests reported in the research literature or documented
 1232 tests performed for other projects that represent the
 1233 project conditions, within the limits specified in Section
 1234 J2.

1235 (ii) Tests that are conducted specifically for the project and
 1236 are representative of project member sizes, material
 1237 strengths, connection configurations, and matching
 1238 connection processes, within the limits specified in
 1239 Section J2.

1240 **6d. Required Shear Strength**

1241 The *required shear strength* of the connection shall be determined using
 1242 the following quantity for the earthquake load effect E :

$$1243 \quad E = 2[1.1R_yM_p]/L_h \quad (E2-1)$$

1244 where

1245 $R_y =$ ratio of the expected yield stress to the specified
 1246 minimum yield stress, F_y

1247 $M_p = F_y Z =$ nominal plastic flexural strength, kip-in. (N-
 1248 mm)

1249 $L_h =$ distance between beam plastic hinge locations, in. (mm)

1250 Exception: In lieu of Equation E2-1, the *required shear strength* of the
 1251 connection shall be as specified in ANSI/AISC 358, or as otherwise
 1252 determined in a connection prequalification in accordance with Section
 1253 J1, or in a program of qualification testing in accordance with Section J2.

1254 **6e. Continuity Plates**

1255 *Continuity plates* shall be consistent with the *prequalified connection*
 1256 designated in ANSI/AISC 358, including Supplement No. 1, or as
 1257 otherwise determined in a connection prequalification in accordance with
 1258 Section J1, or as determined in a program of qualification testing in
 1259 accordance with Section J2.

1260 **6f. Column Splices**

1261 Column splices shall comply with the requirements of Section D2.5.
 1262 Where welds are used to make the splice, they shall be complete- joint-
 1263 penetration groove welds.

1264 When bolted column splices are used, they shall have a *required flexural*
 1265 *strength* that is at least equal to $R_y F_y Z_x$ (LRFD) or $R_y F_y Z_x / 1.5$ (ASD), as
 1266 appropriate, of the smaller column. The *required shear strength* of
 1267 column web splices shall be at least equal to $\Sigma M_{pc} / H$ (LRFD) or
 1268 $\Sigma M_{pc} / 1.5H$ (ASD), as appropriate, where ΣM_{pc} is the sum of the nominal
 1269 plastic flexural strengths of the columns above and below the splice.

1270 Exception: The *required strength* of the column splice need not exceed
 1271 that determined by a nonlinear analysis as specified in Chapter C
 1272 considering appropriate stress concentration factors or fracture mechanics
 1273 stress intensity factors.

1274 **E3. SPECIAL MOMENT FRAMES**

1275 **1. Scope**

1276 *Special moment frames* (SMF) of structural steel shall be designed in
 1277 conformance with this section.

1278 **2. Basis of Design**

1279 SMF shall be designed to provide significant inelastic drift capacity
 1280 through flexural yielding of the SMF beams and limited yielding of
 1281 column panel zones. Except where otherwise permitted in this section,
 1282 columns shall be designed to be generally stronger than the fully yielded
 1283 and strain-hardened beams or girders. Flexural yielding of columns at the
 1284 base shall be permitted. Design of connections of beams to columns,
 1285 including panel zones and *continuity plates*, shall be based on connection

1286 tests that provide the performance required by Section E3.6b, and
 1287 demonstrate this conformance as required by Section E3.6c.
 1288

1289 **3. Analysis**

1290 There are no additional requirements beyond those in the *Specification*
 1291 and the *applicable building code*.

1292 **4. System Requirements**

1293 **4a. Moment Ratio**

1294 The following relationship shall be satisfied at beam-to-column
 1295 connections:

$$\frac{\Sigma M_{pc}^*}{\Sigma M_{pb}^*} > 1.0 \quad (E3-1)$$

1296
 1297 where

1298 ΣM_{pc}^* = the sum of the moments in the column above and
 1299 below the joint at the intersection of the beam and
 1300 column centerlines. ΣM_{pc}^* is determined by
 1301 summing the projections of the nominal flexural
 1302 strengths of the columns (including haunches where
 1303 used) above and below the joint to the beam
 1304 centerline with a reduction for the axial force in the
 1305 column. It is permitted to determine ΣM_{pc}^* as
 1306 follows:

$$\Sigma M_{pc}^* = \Sigma Z_c(F_{yc} - P_{uc}/A_g) \text{ (LRFD)} \quad (E3-2a)$$

1307
 1308 or **DRAFT**

$$\Sigma Z_c[(F_{yc}/1.5) - P_{ac}/A_g] \text{ (ASD)}, \quad (E3-2b)$$

1309
 1310 as appropriate.

1311 When the centerlines of opposing beams in the same
 1312 joint do not coincide, the mid-line between
 1313 centerlines shall be used.

1314 ΣM_{pb}^* = the sum of the moments in the beams at the
 1315 intersection of the beam and column centerlines.
 1316 ΣM_{pb}^* is determined by summing the projections of
 1317 the expected flexural strengths of the beams at the
 1318 plastic hinge locations to the column centerline. It is
 1319 permitted to determine ΣM_{pb}^* as follows:

$$\Sigma M_{pb}^* = \Sigma(1.1R_y F_{yb} Z_b + M_{uv}) \text{ (LRFD)} \quad (E3-3a)$$

- 1321 or
- 1322 $\Sigma M^*_{pb} = \Sigma[(1.1/1.5)R_y F_{yb} Z_b + M_{av}]$ (ASD), (E3-3b)
- 1323 as appropriate.
- 1324 Alternatively, it is permitted to determine ΣM^*_{pb}
- 1325 consistent with a *prequalified connection* design as
- 1326 designated in ANSI/AISC 358, or as otherwise
- 1327 determined in a connection prequalification in
- 1328 accordance with Section J1, or in a program of
- 1329 qualification testing in accordance with Section J2.
- 1330 When connections with reduced beam sections are
- 1331 used, it is permitted determine ΣM^*_{pb} as follows:
- 1332 $\Sigma M^*_{pb} = \Sigma(1.1R_y F_{yb} Z_{RBS} + M_{uv})$ (LRFD) (E3-4a)
- 1333 or
- 1334 $\Sigma M^*_{pb} = \Sigma[(1.1/1.5)R_y F_{yb} Z_{RBS} + M_{av}]$ (ASD), (E3-4b)
- 1335 as appropriate.
- 1336 A_g = gross area of column, in.² (mm²)
- 1337 F_{yc} = specified minimum yield stress of column, ksi (MPa)
- 1338 M_{av} = the additional moment due to shear amplification
- 1339 from the location of the plastic hinge to the column
- 1340 centerline, based on ASD load combinations, kip-in.
- 1341 (N-mm)
- 1342 M_{uv} = the additional moment due to shear amplification
- 1343 from the location of the plastic hinge to the column
- 1344 centerline, based on LRFD load combinations, kip-
- 1345 in. (N-mm)
- 1346 P_{ac} = required compressive strength using ASD load
- 1347 combinations, including the amplified seismic load,
- 1348 kips (N)
- 1349 P_{uc} = required compressive strength using LRFD load
- 1350 combinations, including the amplified seismic load,
- 1351 kips (N)
- 1352 Z_b = plastic section modulus of the beam, in.³ (mm³)
- 1353 Z_c = plastic section modulus of the column, in.³ (mm³)
- 1354 Z_{RBS} = minimum plastic section modulus at the reduced
- 1355 beam section, in.³ (mm³)

1356 Exception: This requirement shall not apply if the conditions in (a) or
1357 (b) are satisfied.

1358 (a) Columns with $P_{rc} < 0.3 P_c$ for all *load combinations* other than
1359 those determined using the *amplified seismic load* that satisfy
1360 either of the following:

1361 (i) Columns used in a one-story building or the top story of a
1362 multistory building.

1363 (ii) Columns where: (1) the sum of the *available shear*
1364 *strengths* of all *exempted columns* in the story is less than
1365 20% of the sum of the available shear strengths of all
1366 moment frame columns in the story acting in the same
1367 direction; and (2) the sum of the available shear strengths
1368 of all *exempted columns* on each moment frame column
1369 line within that story is less than 33% of the available
1370 shear strength of all moment frame columns on that
1371 column line. For the purpose of this exception, a column
1372 line is defined as a single line of columns or parallel lines
1373 of columns located within 10% of the plan dimension
1374 perpendicular to the line of columns.

1375 For design according to *Specification* Section B3.3 (LRFD),

1376
$$P_c = F_{yc}A_g, \text{ kips (N)}$$

1377
$$P_{rc} = P_{uc}, \text{ required compressive strength,}$$

1378
$$\text{using LRFD load combinations, kips}$$

1379
$$\text{(N)}$$

1380 For design according to *Specification* Section B3.4 (ASD),

1381
$$P_c = F_{yc}A_g/1.5, \text{ kips (N)}$$

1382
$$P_{rc} = P_{ac}, \text{ required compressive strength,}$$

1383
$$\text{using ASD load combinations, kips}$$

1384
$$\text{(N)}$$

1385 (b) Columns in any story that has a ratio of available shear strength
1386 to *required shear strength* that is 50% greater than the story above.

1387 **4b. Stability Bracing of Beams**

1388
1389 Beams shall be braced to meet the requirements for *highly ductile*
1390 *members* in Section D1.2b.

1391 In addition, member braces shall be placed near concentrated forces,
1392 changes in cross-section, and other locations where analysis indicates that
1393 a plastic hinge will form during inelastic deformations of the SMF. The
1394 placement of lateral bracing shall be consistent with that documented for

1395 a *prequalified connection* designated in ANSI/AISC 358, including
 1396 Supplement No. 1, or as otherwise determined in a connection
 1397 prequalification in accordance with Section J1, or in a program of
 1398 qualification testing in accordance with Section J2.

1399 The *required strength* of lateral bracing provided adjacent to plastic
 1400 hinges shall be as required by Section D1.2c.

1401 4c. Stability Bracing at Beam-to-Column Connections

1402 (1) Braced Connections

1403
 1404 When the webs of the beams and column are co-planar, and a column is
 1405 shown to remain elastic outside of the panel zone, column flanges at
 1406 beam-to-column connections shall require stability bracing only at the
 1407 level of the top flanges of the beams. It shall be permitted to assume that
 1408 the column remains elastic when the ratio calculated using Equation E3-1
 1409 is greater than 2.0.

1410
 1411 When a column cannot be shown to remain elastic outside of the panel
 1412 zone, the following requirements shall apply:

- 1413 (a) The column flanges shall be laterally braced at the levels
 1414 of both the top and bottom beam flanges. Stability
 1415 bracing shall be permitted to be either direct or indirect.

1416 **User Note:** Direct lateral support (bracing) of the column
 1417 flange is achieved through use of member braces or other
 1418 members, deck and slab, attached to the column flange at
 1419 or near the desired bracing point to resist lateral buckling.
 1420 Indirect lateral support refers to bracing that is achieved
 1421 through the stiffness of members and connections that are
 1422 not directly attached to the column flanges, but rather act
 1423 through the column web or stiffener plates.

- 1424 (b) Each column-flange member brace shall be designed for a
 1425 *required strength* that is equal to 2% of the available
 1426 beam flange strength $F_y b_f t_{bf}$ (LRFD) or $F_y b_f t_{bf} / 1.5$
 1427 (ASD), as appropriate.

1428 (2) Unbraced Connections

1429
 1430 A column containing a beam-to-column connection with no member
 1431 bracing transverse to the seismic frame at the connection shall be
 1432 designed using the distance between adjacent member braces as the
 1433 column height for buckling transverse to the seismic frame and shall
 1434 conform to *Specification* Chapter H, except that:
 1435

1436 (1) The required column strength shall be determined from the
1437 appropriate *load combinations* in the *applicable building code*, except
1438 that *E* shall be permitted to be taken as the lesser of:

1439 (i) the *amplified seismic load*

1440 (ii) 125% of the frame *available strength* based upon
1441 either the beam *available flexural strength* or
1442 panel zone *available shear strength*

1443 (2) The slenderness L/r for the column shall not exceed 60.

1444 (3) The column *required flexural strength* transverse to the
1445 seismic frame shall include that moment caused by the
1446 application of the beam flange force specified in Section
1447 E3.4c(1)(b) in addition to the second-order moment due
1448 to the resulting column flange lateral displacement.

1449 5. Members

1450 5a. Width-Thickness Limitations

1451 Beam and column members shall meet the requirements of Section D1.1b
1452 and Table D1.1 for *highly ductile members*, unless otherwise qualified by
1453 tests.

1454 5b. Beam Flanges

1455 Abrupt changes in beam flange area shall not be permitted in plastic
1456 hinge regions. The drilling of flange holes or trimming of beam flange
1457 width shall not be permitted unless testing or qualification demonstrates
1458 that the resulting configuration can develop stable plastic hinges to the
1459 required interstory drift angle. The configuration shall be consistent with
1460 a *prequalified connection* designated in ANSI/AISC 358, including
1461 Supplement No. 1, or as otherwise determined in a connection
1462 prequalification in accordance with Section J1, or in a program of
1463 qualification testing in accordance with Section J2.

1464 5c. Protected Zones

1465 The region at each end of the beam subject to inelastic straining shall be
1466 designated as a *protected zone*, and shall meet the requirements of
1467 Section D1.3. The extent of the protected zone shall be as designated in
1468 ANSI/AISC 358, including Supplement No. 1, or as otherwise
1469 determined in a connection prequalification in accordance with Section
1470 J1, or as determined in a program of qualification testing in accordance
1471 with Section J2.

1472 **User Note:** The plastic hinging zones at the ends of SMF beams should
1473 be treated as protected zones. The plastic hinging zones should be

1474 established as part of a prequalification or qualification program for the
 1475 connection, per Section E3.6c. In general, for unreinforced connections,
 1476 the protected zone will extend from the face of the column to one half of
 1477 the beam depth beyond the plastic hinge point.

1478
 1479 **5d. Beams**

1480
 1481 Structural steel beams in SMF are permitted to be composite with a
 1482 reinforced concrete slab to resist gravity loads.

1483
 1484 **6. Connections**

1485 **6a. Demand Critical Welds**

1486 Unless otherwise designated by ANSI/AISC 358, including Supplement
 1487 No. 1, or otherwise determined in a connection prequalification in
 1488 accordance with Section J1, or as determined in a program of
 1489 qualification testing in accordance with Section J2, complete-joint-
 1490 penetration groove welds of beam flanges and beam webs to columns
 1491 shall be *demand critical* welds as described in Section A3.4b.

1492 **User Note:** For the designation of demand critical welds, standards such
 1493 as ANSI/AISC 358 and tests addressing specific connections and joints
 1494 should be used in lieu of the more general terms of these *Provisions*.
 1495 Where these *Provisions* indicate that a particular weld is designated
 1496 demand critical, but the more specific standard or test does not make such
 1497 a designation, the more specific standard or test should govern. Likewise,
 1498 these standards and tests may designate welds as demand critical that are
 1499 not identified as such by these *Provisions*.

1500 **6b. Beam-to-Column Connection Requirements**

1501 Beam-to-column connections used in the *seismic force resisting system*
 1502 (SFRS) shall satisfy the following two requirements:

- 1503 (1) The connection shall be capable of sustaining an *interstory drift*
 1504 *angle* of at least 0.04 rad.
- 1505 (2) The *measured flexural resistance* of the connection, determined
 1506 at the column face, shall equal at least $0.80M_p$ of the connected
 1507 beam at an interstory drift angle of 0.04 rad.

1508
 1509 **6c. Conformance Demonstration**

1510 Beam-to-column connections used in the SFRS shall satisfy the
 1511 requirements of Section E3.6b by one of the following:

- 1512 (a) Use of SMF connections designed in accordance with ANSI/AISC
1513 358, including Supplement No. 1.
- 1514 (b) Use of a connection prequalified for SMF in accordance with
1515 Section J1.
- 1516 (c) Provision of qualifying cyclic test results in accordance with Section
1517 J2. Results of at least two cyclic connection tests shall be provided
1518 and shall be based on one of the following:
- 1519 (i) Tests reported in the research literature or documented tests
1520 performed for other projects that represent the project
1521 conditions, within the limits specified in Section J2.
- 1522 (ii) Tests that are conducted specifically for the project and are
1523 representative of project member sizes, material strengths,
1524 connection configurations, and matching connection
1525 processes, within the limits specified in Section J2.

1526 **6d. Required Shear Strength**

1527 The *required shear strength* of the connection shall be determined using
1528 the following quantity for the earthquake load effect E :

$$1529 \quad E = 2[1.1R_y M_p]/L_h \quad (E3-5)$$

1530 where

1531 R_y = ratio of the expected *yield stress* to the specified
1532 minimum *yield stress*, F_y

1533 M_p = nominal plastic flexural strength, kip-in. (N-mm)

1534 L_h = distance between plastic hinge locations, in. (mm)

1535 Exception: In lieu of Equation E3-5, the *required shear strength* of the
1536 connection shall be as specified in ANSI/AISC 358, including
1537 Supplement No. 1, or as otherwise determined in a connection
1538 prequalification in accordance with Section J1, or in a program of
1539 qualification testing in accordance with Section J2.

1540 **6e. Panel Zone**

1541 (1) Required Shear Strength

1542
1543 The required thickness of the panel zone shall be determined in
1544 accordance with the method used in proportioning the panel zone of the
1545 tested or *prequalified connection*. As a minimum, the *required shear*
1546 *strength* of the panel zone shall be determined from the summation of the
1547 moments at the column faces as determined by projecting the expected
1548 moments at the plastic hinge points to the column faces. The design shear

1549 strength shall be $\phi_v R_v$ and the *allowable shear strength* shall be R_v/Ω_v
 1550 where

1551
 1552
$$\phi_v = 1.0 \text{ (LRFD)} \quad \Omega_v = 1.50 \text{ (ASD)}$$

1553
 1554 and the *nominal shear strength*, R_v , according to the limit state of shear
 1555 yielding, is determined as specified in *Specification* Section J10.6.

1556 (2) Panel Zone Thickness

1557
 1558 The individual thicknesses, t , of column webs and *doubler plates*, if used,
 1559 shall conform to the following requirement:

1560
 1561
$$t \geq (d_z + w_z)/90 \quad \text{(E3-6)}$$

1562
 1563 where

1564 t = thickness of column web or doubler plate, in.
 1565 (mm)

1566 d_z = panel zone depth between *continuity plates*, in.
 1567 (mm)

1568 w_z = panel zone width between column flanges, in.
 1569 (mm)

1570
 1571 Alternatively, when local buckling of the column web and doubler plate
 1572 is prevented by using plug welds joining them, and dividing the plate to
 1573 conform with Equation E3-1, the total panel zone thickness shall satisfy
 1574 Equation E3-1. When plug welds are required, a minimum of four plug
 1575 welds shall be provided.

1576
 1577 (3) Panel Zone Doubler Plates

1578
 1579 Doubler plates shall be welded to the column flanges to develop the
 1580 available shear strength of the full doubler plate thickness, using either a
 1581 complete-joint-penetration groove-welded or fillet-welded joint. When
 1582 doubler plates are placed against the column web, they shall be welded
 1583 across the top and bottom edges to develop the proportion of the total
 1584 force that is transmitted to the doubler plate. When doubler plates are
 1585 placed away from the column web, they shall be placed symmetrically in
 1586 pairs and welded to continuity plates to develop the proportion of the
 1587 total force that is transmitted to the doubler plate.

1588
 1589 **User Note:** When a doubler plate interferes with connecting
 1590 continuity plates directly to the column web, the designer must
 1591 provide a load path that satisfies the requirements of ANSI/AISC
 1592 358 Section 2.4.4b. This may be accomplished by sizing the
 1593 doubler plate such that it is capable of developing the required
 1594 strength of the continuity plate to column web connection.
 1595 Alternatively, the doubler plate can stop inside the continuity

1596 plates. A similar load path must be provided when the web plate
 1597 for a beam perpendicular to the column web connects to a doubler
 1598 plate.
 1599

1600 **6f. Continuity Plates**

1601 Continuity plates shall be consistent with the prequalified connection
 1602 designated in ANSI/AISC 358, including Supplement No. 1, or as
 1603 otherwise determined in a connection prequalification in accordance with
 1604 Section J1, or as determined in a program of qualification testing in
 1605 accordance with Section J2.

1606 **6g. Column Splices**

1607
 1608 Column splices shall comply with the requirements of Section D2.5.
 1609 Where groove welds are used to make the splice, they shall be complete-
 1610 joint-penetration groove welds.

1611 When bolted column splices are used, they shall have a *required flexural*
 1612 *strength* that is at least equal to $R_y F_y Z_x$ (LRFD) or $R_y F_y Z_x / 1.5$ (ASD), as
 1613 appropriate, of the smaller column. The *required shear strength* of
 1614 column web splices shall be at least equal to $\Sigma M_{pc} / H$ (LRFD) or
 1615 $\Sigma M_{pc} / 1.5H$ (ASD), as appropriate, where ΣM_{pc} is the sum of the nominal
 1616 plastic flexural strengths of the columns above and below the splice.

1617 Exception: The *required strength* of the column splice considering
 1618 appropriate stress concentration factors or fracture mechanics stress
 1619 intensity factors need not exceed that determined by a nonlinear analysis
 1620 as specified in Chapter C.

1621 **E4. SPECIAL TRUSS MOMENT FRAMES**

1622
 1623 **1. Scope**
 1624

1625 *Special truss moment frames* (STMF) shall meet the requirements in this
 1626 Section.

1627
 1628 **2. Basis of Design**
 1629

1630 STMF shall be designed to withstand significant inelastic deformation
 1631 within a *special segment* of the truss when subjected to the forces from
 1632 the motions of the *design earthquake*. STMF shall be limited to span
 1633 lengths between columns not to exceed 65 ft (20 m) and overall depth not
 1634 to exceed 6 ft (1.8 m). The columns and truss segments outside of the
 1635 special segments shall be designed to remain elastic under the forces that

1636 can be generated by the fully yielded and strain-hardened special
1637 segment.

1638
1639 **3. Analysis**

1640
1641 **3a. Special Segment**

1642
1643 The *required vertical shear strength* of the special segment shall be
1644 calculated for the appropriate *load combinations* in the *applicable*
1645 *building code*.

1646
1647 **3b. Non-Special Segment**

1648
1649 The *required strength* of non-special segment members and connections
1650 shall be calculated based on the appropriate *load combinations* in the
1651 applicable building code, replacing the earthquake load term, *E*, with the
1652 lateral forces necessary to develop the *expected vertical shear strength* of
1653 the special segment acting at mid-length and defined in Section E4.5b.
1654 Second order effects at maximum design drift shall be included.

1655
1656 **4. System Requirements**

1657
1658 **4a. Special Segment**

1659 Each horizontal truss that is part of the SFRS shall have a *special*
1660 *segment* that is located between the quarter points of the span of the truss.
1661 The length of the special segment shall be between 0.1 and 0.5 times the
1662 truss span length. The length-to-depth ratio of any panel in the special
1663 segment shall neither exceed 1.5 nor be less than 0.67.

1664 Panels within a special segment shall either be all Vierendeel panels or all
1665 X-braced panels; neither a combination thereof nor the use of other truss
1666 diagonal configurations is permitted. Where diagonal members are used
1667 in the special segment, they shall be arranged in an X pattern separated
1668 by vertical members. Diagonal members within the special segment shall
1669 be made of rolled flat bars of identical sections. Such diagonal members
1670 shall be interconnected at points where they cross. The interconnection
1671 shall have a *required strength* equal to 0.25 times the nominal *tensile*
1672 *strength* of the diagonal member. Bolted connections shall not be used for
1673 diagonal members within the special segment.

1674 Splicing of chord members shall not be permitted within the special
1675 segment, nor within one-half the panel length from the ends of the special
1676 segment.

1677 The *required axial strength* of the diagonal web members in the special
1678 segment due to dead and live loads within the special segment shall not
1679 exceed $0.03F_yA_g$ (LRFD) or $(0.03/1.5)F_yA_g$ (ASD), as appropriate.

1680 **4b. Stability Bracing of Trusses**

1681 Each flange of the chord members shall be laterally braced at the ends of
1682 the special segment. The *required strength* of the lateral brace shall be

1683
$$P_u = 0.06 R_y F_y A_f \text{ (LRFD)} \quad \text{(E4-1a)}$$

1684 or

1685
$$P_a = (0.06/1.5) R_y F_y A_f \text{ (ASD)} \quad \text{(E4-1b)}$$

1686 where

1687 A_f = gross area of the flange of the *special segment* chord
1688 member, in.² (mm²)

1689 **4c. Stability Bracing of Truss-to-Column Connections**

1690 The columns shall be laterally braced at the levels of top and bottom
1691 chords of the trusses connected to the columns. The lateral braces shall
1692 have required strength of

1693
$$P_u = 0.02 R_y P_{nc} \text{ (LRFD)} \quad \text{(E4-2a)}$$

1694 or

1695
$$P_a = (0.02/1.5) R_y P_{nc} \text{ (ASD)} \quad \text{(E4-2b)}$$

1696 where

1697 P_{nc} = nominal compressive strength of the chord member at
1698 the ends, kips (N)

1699 **4d. Stiffness of Stability Bracing**

1702 The required brace stiffness shall meet the provisions of Section 6.2 of
1703 Appendix 6 of the *Specification*, where

1704
$$P_r = R_y P_{nc} \text{ (LRFD)} \quad \text{(E4-3a)}$$

1705 or

1706
$$P_r = R_y P_{nc} / 1.5 \text{ (ASD)} \quad \text{(E4-3b)}$$

1707 **5. Members**

1709 **5a. Special Segment Members**

1710 The available shear strength of the *special segment* shall be calculated as
1711 the sum of the available shear strength of the chord members through

1712 flexure, and of the shear strength corresponding to the available tensile
 1713 strength and 0.3 times the available compressive strength of the diagonal
 1714 members, when they are used. The top and bottom chord members in the
 1715 special segment shall be made of identical sections and shall provide at
 1716 least 25% of the required vertical shear strength.

1717 The *required axial strength* of the chord members shall not exceed 0.45
 1718 times ϕP_n (LRFD) or P_n / Ω (ASD), as appropriate,

$$1719 \quad \phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

1720 where

$$1721 \quad P_n = F_y A_g \quad (\text{E4-4})$$

1722 **5b. Expected Vertical Shear Strength of Special Segment**

1723 The *expected vertical shear strength* of the special segment, V_{ne} (LRFD)
 1724 or $V_{ne} / 1.5$ (ASD), as appropriate, at mid-length, shall be given as:

$$1725 \quad V_{ne} = \frac{3.60 R_y M_{nc}}{L_s} + 0.036 EI \frac{(L - L_s)}{L_s^3} + R_y (P_{nt} + 0.3 P_{nc}) \sin \alpha \quad (\text{E4-4})$$

1726 where

1727 M_{nc} = nominal flexural strength of a chord member of the
 1728 special segment, kip-in. (N-mm)

1729 E = modulus of elasticity of a chord member of the special
 1730 segment, kip/in.² (N/mm²)

1731 I = moment of inertia of a chord member of the special
 1732 segment, in.⁴ (mm⁴)

1733 L = span length of the truss, in. (mm)

1734 L_s = length of the special segment, in. (mm)

1735 P_{nt} = nominal *tensile strength* of a diagonal member of the
 1736 special segment, kips (N)

1737 P_{nc} = nominal *compressive strength* of a diagonal member of
 1738 the special segment, kips (N)

1739 α = angle of diagonal members with the horizontal

1740 **5c. Width-Thickness Limitations**

1741 Chord members and diagonal web members within the special segment
1742 shall meet the requirements of Section D1.1b for highly ductile members.
1743 The width-thickness ratio of flat bar diagonal members shall not exceed
1744 2.5.

1745 **5d. Built-Up Chord Members**

1746
1747 Spacing of stitching for built-up chord members in the special segment
1748 shall not exceed $0.04Er_y/F_y$, where r_y is the radius of gyration of
1749 individual components about their weak axis.

1750
1751 **5e. Protected Zones**

1752 The region at each end of a chord member within the special segment
1753 shall be designated as protected zone meeting the requirements of Section
1754 D1.3. The protected zone shall extend over a length equal to two times
1755 the depth of the chord member from the connection with the web
1756 members. Vertical and diagonal web members from end to end of the
1757 special segments shall be protected zones.

1758
1759 **6. Connections**

1760 **6a. Connections of Diagonal Web Members in the Special Segment**

1761 The end connection of diagonal web members in the special segment
1762 shall have a required strength that is at least equal to the *expected yield*
1763 *strength* of the web member, $R_yF_yA_g$ (LRFD) or $R_yF_yA_g / 1.5$ (ASD), as
1764 appropriate.

1765 **6b. Column Splices**

1766
1767 Column splices shall comply with the requirements of Section D2.5.
1768 Where welds are used to make the splice, they shall be complete-joint-
1769 penetration groove welds.

1770 When bolted column splices are used, they shall have a *required flexural*
1771 *strength* that is at least equal to $R_yF_yZ_x$ (LRFD) or $R_yF_yZ_x / 1.5$ (ASD), as
1772 appropriate, of the smaller column. The *required shear strength* of
1773 column web splices shall be at least equal to $\Sigma M_{pc}/H$ (LRFD) or
1774 $\Sigma M_{pc}/1.5H$ (ASD), as appropriate, where ΣM_{pc} is the sum of the nominal
1775 plastic flexural strengths of the columns above and below the splice.

1776 Exception: The *required strength* of the column splice considering
1777 appropriate stress concentration factors or fracture mechanics stress
1778 intensity factors need not exceed that determined by a nonlinear analysis
1779 as specified in Chapter C.

1780

1781 **E5. ORDINARY CANTILEVER COLUMN SYSTEMS**1782 **1. Scope**

1783 *Ordinary Cantilever Column Systems* (OCCS) shall be designed in
1784 conformance with this section.

1785 **2. Basis of Design**

1786 OCCS shall be designed to provide minimal inelastic drift capacity
1787 through flexural yielding of the columns.
1788

1789 **3. Analysis**

1790 There are no additional requirements beyond those in the *Specification*
1791 and the *applicable building code*.

1792 **4. System Requirements**1793 **4a. Columns**

1794
1795 Columns shall be designed using the load combinations including the
1796 amplified seismic load. The *required strength*, P_{uc} , shall not exceed
1797 $0.15P_y$.

1798 **4b. Stability Bracing of Columns**

1799
1800 There are no additional requirements beyond the *Specification*.

1801 **5. Members**1802 **5a. Width-Thickness Limitations**

1803 There are no additional requirements beyond the *Specification*.

1804 **5b. Column Flanges**

1805 There are no additional requirements beyond the *Specification*.

1806 **5c. Protected Zones**

1807 There are no designated *protected zones*.

1808 **6. Connections**1809 **6a. Column Bases**

1810 There are no additional requirements beyond the *Specification*.

1811

1812 **E6. SPECIAL CANTILEVER COLUMN SYSTEMS**1813 **1. Scope**

1814 *Cantilever column systems* (CCS) shall be designed in conformance with
1815 this section.

1816 **2. Basis of Design**

1817 CCS shall be designed to provide limited inelastic drift capacity through
1818 flexural yielding of the columns.

1819 **3. Analysis**

1820 There are no additional requirements beyond those in the *Specification*
1821 and the *applicable building code*.

1822 **4. System Requirements**1823 **4a. Columns**

1824 Columns shall be designed using the load combinations including the
1825 amplified seismic load. The *required strength*, P_{uc} , shall not exceed
1826 $0.15P_y$.
1827
1828

1829 **4b. Stability Bracing of Columns**

1830 Columns shall be braced to meet the requirements for *moderately ductile*
1831 *members* in Section D1.2a.
1832

1833 **5. Members**1834 **5a. Width-Thickness Limitations**

1835 Column members shall meet the requirements of Section D1.1b and
1836 Table D1.1 for *highly ductile members*.

1837 **5b. Column Flanges**

1838 Abrupt changes in column flange area are prohibited in the *protected*
1839 *zone* as designated in Section E3.5c.

1840 **5c. Protected Zones**

1841 The region at the base of the column subject to inelastic straining shall be
1842 designated as a *protected zone*, and shall meet the requirements of
1843 Section D1.3. The length of the *protected zone* shall be two times the
1844 column depth, unless otherwise substantiated by testing.

1845 **6. Connections**

1846 **6a. Column Bases**

1847 Column bases shall be designed in accordance with Section D2.6.

DRAFT

1848

CHAPTER F

1849

BRACED-FRAME AND SHEAR-WALL SYSTEMS

1850

1851

1852 This chapter provides the basis of design and the requirements for analysis, the
 1853 system, members and connections for steel ordinary, special, eccentrically, and
 1854 buckling-restrained braced frames, as well as special plate shear walls.

1855 The chapter is organized as follows:

1856

- 1857 F1. Ordinary Concentrically Braced Frames
- 1858 F2. Special Concentrically Braced Frames
- 1859 F3. Eccentrically Braced Frames
- 1860 F4. Buckling-Restrained Braced Frames
- 1861 F5. Special Plate Shear Walls

1862

1863 **F1. ORDINARY CONCENTRICALLY BRACED FRAMES**1864 **1. Scope**

1865 *Ordinary concentrically braced frames (OCBF)* shall be designed in
 1866 conformance with this section.

1867 In seismically isolated structures, OCBF above the isolation system shall
 1868 meet the requirements of Sections F1.4b, F1.5, F1.6 and F1.7 and need
 1869 not meet the requirements of Section F1.4a.

1870 **2. Basis of Design**

1871 This Section is applicable to braced frames that consist of concentrically
 1872 connected members. Minor eccentricities less than the beam depth are
 1873 permitted if they are accounted for in the member design by
 1874 determination of eccentric moments using the *amplified seismic load*.
 1875 OCBF are expected to withstand limited inelastic deformations in their
 1876 members and connections when subjected to the forces resulting from the
 1877 motions of the *design earthquake*.

1878 **3. Analysis**

1879 There are no additional requirements beyond those in the *Specification*
 1880 and the *applicable building code*.

1881 **4. System Requirements**1882 **4a. V-Braced and Inverted-V-Braced Frames**

1883 Beams in V-type and inverted V-type OCBF shall be continuous at brace
 1884 connections away from the beam-column connection and shall meet the
 1885 following requirements:

1886 (1) The *required strength* shall be determined based on the *load*
 1887 *combinations* of the *applicable building code* assuming that the
 1888 braces provide no support of dead and live loads. For load
 1889 combinations that include earthquake effects, the earthquake
 1890 effect, E , on the member shall be determined as follows:

1891 (i) The forces in braces in tension shall be assumed to be the
 1892 least of the following:

- 1893 1. The expected strength of the brace, $R_y F_y A_g$
- 1894 2. The load effect based upon the amplified seismic
 1895 load
- 1896 3. The maximum force that can be developed by the
 1897 system

1898 (ii) The forces in braces in compression shall be assumed to
 1899 be equal to $0.3P_n$.

1900 (2) As a minimum, one set of lateral braces is required at the point of
 1901 intersection of the braces, unless the member has sufficient out-
 1902 of-plane strength and stiffness to ensure stability between
 1903 adjacent brace points.

1904 **4b. K-Braced Frames**

1905 K-type braced frames are not permitted for OCBF.

1906 **5. Members**

DRAFT

1907 **5a. Width-Thickness Limitations**

1908 Braces shall meet the requirements of Section D1.1a for *moderately*
 1909 *ductile members*.

1910 **5b. Slenderness**

1911 Braces in V or inverted-V configurations shall have $Kl / r \leq 4\sqrt{E/F_y}$.

1912

1913

1914 **6. Connections**

1915 **6a. Diagonal Brace Connections**

1916 The *required strength* of diagonal brace connections is the load effect
 1917 based upon the *amplified seismic load*.
 1918

1919 Exception: The required strength of the brace connection need not
 1920 exceed the following:

- 1921 (1) The maximum force that can be developed by the system
- 1922 (2) In tension, the *expected yield strength* of the brace, determined as
 1923 $R_y F_y A_g$ (LRFD) or $R_y F_y A_g / 1.5$ (ASD), as appropriate
- 1924 (3) In compression, 1.1 times the expected brace strength in
 1925 compression. The expected brace strength in compression is
 1926 permitted to be taken as the lesser of $R_y F_{cr} A_g$ and $F_E A_g$ where F_{cr}
 1927 and F_E are determined from *Specification* Chapter E.
- 1928 (4) When oversized holes are used, the required strength for the limit
 1929 state of bolt slip need not exceed a *load effect* based upon using
 1930 the *load combinations* stipulated by the applicable building code,
 1931 not including the *amplified seismic load*.

1932 **User Note:** In seismic design, slip is a strength limit state.

1933

1934 **7. Ordinary Concentrically Braced Frames above Seismic Isolation**
 1935 **Systems**

1936 **7a. System Requirements**

1937 Beams in V-type and inverted V-type braced frames shall be continuous
 1938 between columns.

1939 **7b. Members**

1940 Braces shall have $KL/r \leq 4\sqrt{E/F_y}$

1941

1942 **F2. SPECIAL CONCENTRICALLY BRACED FRAMES**

1943

1944 **1. Scope**

1945

1946 *Special concentrically braced frames* (SCBF) shall be designed in
 1947 conformance with this section. This Section is applicable to braced
 1948 frames that consist of concentrically connected members. Minor
 1949 eccentricities less than the beam depth are permitted if the resulting
 1950 member and connection forces are addressed in the design and do not
 1951 change the expected source of system ductility: buckling and tension
 1952 yielding of braces.
 1953

1954 **2. Basis of Design**
 1955

1956 SCBF shall be designed so that inelastic deformations under the design
 1957 earthquake will occur primarily as brace buckling and yielding in tension.

1958

1959 **3. Analysis**

1960 Columns, beams and connections in SCBF shall be designed to resist
 1961 seismic forces from the *load combinations* in the *applicable building*
 1962 *code* where the seismic load, E , on each element is the larger force
 1963 determined from the following two analyses:

1964 (i) An analysis in which all braces are assumed to resist forces
 1965 corresponding to their expected strength in compression or in
 1966 tension.

1967 (ii) An analysis in which all braces in tension are assumed to resist
 1968 forces corresponding to their expected strength and all braces in
 1969 compression are assumed to resist their expected post-buckling
 1970 strength.

1971 Braces shall be determined to be in compression or tension
 1972 neglecting the effects of gravity loads. Analyses shall consider
 1973 both directions of frame loading.

1974 The expected brace strength in tension is $R_y F_y A_g$.

1975 The expected brace strength in compression is permitted to be
 1976 taken as the lesser of $R_y F_{cr} A_g$ and $F_E A_g$ where F_{cr} and F_E are
 1977 determined from the *Specification* Chapter E and the brace length
 1978 is taken from brace end to brace end.

1979 The expected post-buckling brace strength shall be taken as a
 1980 maximum of 0.3 times the expected brace strength in
 1981 compression.

1982 **User Note:** Braces with a slenderness ratio of 200 (the maximum
 1983 permitted by Section F2.5b) buckle elastically for permissible materials;
 1984 the value of $0.3 F_{cr}$ for such braces is 2.1 ksi. This value may be used in
 1985 Section F2.3(ii) for braces of any slenderness and a liberal estimate of the
 1986 required strength of framing members will be obtained. Alternatively, 0
 1987 ksi may also be used to simplify the analysis.

1988 Exceptions:

1989 (1) It is permitted to neglect flexural forces resulting from seismic
 1990 drift in this determination. Moment resulting from a load applied
 1991 to the column between points of lateral support must be
 1992 considered.

1993 (2) The required strength of columns need not exceed the least of the
 1994 following:

1995

- 1996 (a) The forces determined using load combinations stipulated
 1997 by the applicable building code including the amplified
 1998 seismic load, applied to a building frame model in which
 1999 all compression braces have been removed.
- 2000 (b) The forces corresponding to the resistance of the found-
 2001 ation to overturning uplift.
- 2002 (c) Forces determined from nonlinear analysis as defined in
 2003 Section C3.
 2004

2005 4. System Requirements

2006 4a. Lateral Force Distribution

2007 Along any line of braces, braces shall be deployed in alternate directions
 2008 such that, for either direction of force parallel to the braces, at least 30%
 2009 but no more than 70% of the total horizontal force along that line is
 2010 resisted by braces in tension, unless the *available strength* of each brace
 2011 in compression is larger than the *required strength* resulting from the
 2012 application of the appropriate *load combinations* stipulated by the
 2013 *applicable building code* including the *amplified seismic load*. For the
 2014 purposes of this provision, a line of braces is defined as a single line or
 2015 parallel lines with a plan offset of 10% or less of the building dimension
 2016 perpendicular to the line of braces.

2017 4b. V- and Inverted V-Braced Frames

2018 Beams that are intersected by braces away from beam-to-column
 2019 connections shall comply with the following requirements:

- 2020 (1) Beams shall be continuous between columns.
- 2021 (2) Beams shall be braced to meet the requirements for *moderately*
 2022 *ductile members* in Section D1.2a.
 2023

2024 As a minimum, one set of lateral braces is required at the point of
 2025 intersection of the V-type (or inverted V-type) braced frames, unless the
 2026 beam has sufficient out-of-plane strength and stiffness to ensure stability
 2027 between adjacent brace points.

2028 **User Note:** One method of demonstrating sufficient out-of-plane strength
 2029 and stiffness of the beam is to apply the bracing force defined in Equation
 2030 A-6-7 of Appendix 6 of the *Specification* to each flange so as to form a
 2031 torsional couple; this loading should be in conjunction with the flexural
 2032 forces defined in item (1) above. The stiffness of the beam (and its
 2033 restraints) with respect to this torsional loading should be sufficient to
 2034 satisfy Equation A-6-8 of the *Specification*.

2035 **4c. K-Braced Frames**

2036 K-type braced frames are not permitted for SCBF.

2037 **4d. Tension-Only Frames**

2038 Tension-only frames are not permitted in SCBF.

2039

User Note: Tension-only braced frames are those in which the brace compression resistance is neglected in the design and the braces are designed for tension forces only.

2041

2042

2043

2044 **5. Members**2045 **5a. Width-Thickness Limitations**

2046 Columns and braces shall meet the requirements of Section D1.1b for
2047 *highly ductile members*. Beams shall meet the requirements of Section
2048 D1.1b for *moderately ductile members*

2049 **5b. Diagonal Braces**

2050 Braces shall comply with the following requirements:

2051 (1) Slenderness: Braces shall have a slenderness ratio $Kl/r \leq 200$.

2052 (2) Built-up Braces: The spacing of stitches shall be such that the
2053 slenderness ratio l/r of individual elements between the stitches
2054 does not exceed 0.4 times the governing slenderness ratio of the
2055 built-up member.

2056

2057 The sum of the available shear strengths of the stitches shall equal
2058 or exceed the available *tensile strength* of each element. The
2059 spacing of stitches shall be uniform. Not less than two stitches
2060 shall be used in a built-up member. Stitches shall not be located
2061 within the middle one-fourth of the clear brace length.

2062

2063 Exception: Where the buckling of braces about their critical
2064 bucking axis does not cause shear in the stitches, the spacing of
2065 the stitches shall be such that the slenderness ratio l/r of the
2066 individual elements between the stitches does not exceed 0.75
2067 times the governing slenderness ratio of the built-up member.

2068

2069 (3) The brace effective net area shall not be less than the brace gross
2070 area. Where reinforcement on braces is used the following requirements
2071 shall apply:

2072

2073 (i) The specified minimum yield strength of the reinforcement
2074 shall be at least the specified minimum yield strength of the brace.

2075
 2076 (ii) The connections of the reinforcement to the brace shall have
 2077 sufficient strength to develop the expected reinforcement strength
 2078 on each side of a reduced section.
 2079

2080 **5c. Protected Zones**

2081 The protected zone of SCBF shall satisfy Section D1.3 and include the
 2082 following:

2083 (1) For braces, the center one-quarter of the brace length and a zone
 2084 adjacent to each connection equal to the brace depth in the plane
 2085 of buckling.

2086 (2) Elements that connect braces to beams and columns.
 2087

2088
 2089 **6. Connections**

2090 **6a. Required Strength of Brace Connections**

2091 The *required strength* in tension, compression and flexure of brace
 2092 connections (including beam-to-column connections if part of the braced-
 2093 frame system) shall be determined as required below. These required
 2094 strengths are permitted to be considered independently without
 2095 interaction.

2096 (1) Required Tensile Strength: The *required tensile strength* is the
 2097 lesser of the following:

2098 (a) The *expected yield strength*, in tension, of the brace,
 2099 determined as $R_y F_y A_g$ (LRFD) or $R_y F_y A_g / 1.5$ (ASD), as
 2100 appropriate.

2101 Exception: Braces need not comply with the
 2102 requirements of Equation J4-1 and J4-2 of the
 2103 *Specification* for this loading.

2104 **User Note:** This exception applies to braces where the
 2105 section is reduced or where the net section is effectively
 2106 reduced due to shear lag. A typical case is a slotted HSS
 2107 brace at the gusset plate connection. Section F2.5b
 2108 requires braces with holes or slots to be reinforced such
 2109 that the effective net area exceeds the gross area.

2110 The brace strength used to check connection limit states,
 2111 such as brace block shear, may be determined using
 2112 expected material properties as permitted by Section
 2113 A3.2.

2114 (b) The maximum load effect, indicated by analysis, that can
2115 be transferred to the brace by the system.

2116 When oversized (OVS) holes are used, the required strength for
2117 the limit state of bolt slip need not exceed a load effect based
2118 upon using the *load combinations* stipulated by the *applicable*
2119 *building code*, including the *amplified seismic load*.

2120 **User Note:** For other limit states the loadings of (i) and (ii) apply.
2121 In seismic design slip is a strength limit state.

2122 (2) Required Compressive Strength: Brace connections shall be
2123 designed for a *required compressive strength* based on buckling
2124 limit states that is at least equal to 1.1 times the expected brace
2125 strength in compression (LRFD) or (1.1/1.5) times the expected
2126 brace strength in compression (ASD), as appropriate, where the
2127 expected brace strength in compression is as defined in Section
2128 F2.3(ii).

2129 (3) Accommodation of Brace Buckling: Brace connections shall be
2130 designed to withstand the flexural forces or rotations imposed by
2131 brace buckling. Connections satisfying either of the following
2132 provisions are deemed to satisfy this requirement:

2133 (i) Required Flexural Strength: Brace connections designed
2134 to withstand the flexural forces imposed by brace
2135 buckling shall have an available flexural strength of at
2136 least $1.1R_yM_p$ (LRFD) or $(1.1/1.5)R_yM_p$ (ASD), as
2137 appropriate, of the brace about the critical buckling axis.

2138 (ii) Rotation Capacity: Brace connections designed to
2139 withstand the rotations imposed by brace buckling
2140 shall have sufficient rotation capacity to accommodate the
2141 required rotation at the *design story drift*. Inelastic
2142 rotation of the connection is permitted.

2143 **User Note:** Accommodation of inelastic rotation is
2144 typically accomplished by means of a single gusset plate
2145 with the brace terminating before the line of restraint. The
2146 detailing requirements for such a connection are described
2147 in the commentary.

2148 6b. Beam-to-Column Connections

2149 Where a brace or gusset plate connects to both members at a beam-to-
2150 column connection, the connection shall conform to one of the following:

2151

- 2152 (1) The connection shall have sufficient rotation capacity to
 2153 accommodate the required rotation at a minimum story drift of
 2154 2.5% of the story height; or
- 2155 (2) The connection shall be designed to resist a moment equal to the
 2156 lesser of the following:
- 2157 (i) A moment corresponding to $1.1R_yF_yZ$ of the beam
- 2158 (ii) A moment corresponding to $\Sigma 1.1R_yF_yZ$ of the column
- 2159 This moment shall be considered in combination with the
 2160 required strength of the brace connection and beam connection,
 2161 including the amplified diaphragm collector forces.

2162

2163 6c. Column Splices

2164

2165 In addition to meeting the requirements of Section D2.5, column splices
 2166 in SCBF shall be designed to develop at least 50% of the lesser available
 2167 flexural strength of the connected members.

2168

2169 The *required shear strength* shall be $\Sigma M_{pc} / H_c$ (LRFD) or $\Sigma M_{pc} / 1.5 H_c$
 2170 (ASD), as appropriate,

2171

2172 where

2173

2174 ΣM_{pc} = sum of the nominal plastic flexural strengths,
 2175 $F_y Z_c$, of the columns above and below the splice,
 2176 kip-in. (N-mm)

2177 H_c = clear height of the column between beam
 2178 connections, including a structural slab, if
 2179 present., in. (mm)

2180

2181 6d. Demand Critical Welds

2182 Groove welds at column splices shall be *demand critical welds* as
 2183 described in Section A3.4b.

2184

2185 F3. ECCENTRICALLY BRACED FRAMES

2186

2187 1. Scope

2188 *Eccentrically braced frames* (EBF) shall be designed in conformance
 2189 with this section. Minor eccentricities less than the beam depth are
 2190 permitted in the brace connection away from the link if the resulting
 2191 member and connection forces are addressed in the design and do not

2192 change the expected source of system ductility: buckling and tension
2193 yielding of braces.

2194 2. Basis of Design

2195 EBF shall be designed so that inelastic deformations under the design
2196 earthquake will occur primarily in the links.

2197 Where links connect directly to columns, the link-to-column connection
2198 shall provide the performance and shall meet the conformance
2199 demonstration requirements of Section F3.6a.

2200 3. Analysis

2201 The *required strength* of diagonal braces and their connections, beams
2202 outside links, and columns shall be based on the load combinations in the
2203 *applicable building code*. For load combinations that include earthquake
2204 effects, the earthquake effect E for columns, diagonal braces, connections
2205 and beams outside links shall be taken as the forces developed in the
2206 member assuming the forces at the end of the link connected to the
2207 member correspond to the *adjusted link shear strength*. The adjusted
2208 link shear strength shall be taken as R_y times the link nominal shear
2209 strength given in Section F3.5a(2) multiplied by 1.25 for I-shaped links
2210 and 1.4 for box links.

2211 Exceptions:

2212 (1) The earthquake effect, E , is permitted to be taken as 0.88 times
2213 the forces determined above for the design of the following
2214 members:

- 2215
- 2216 (a) Beams outside links; and
 - 2217 (b) Columns in frames of three or more stories of bracing.

2218 (2) It is permitted to neglect flexural forces resulting from seismic
2219 drift in this determination. Moment resulting from a load applied
2220 to the column between points of lateral support must be
2221 considered.

2222 (3) The required strength of columns need not exceed the lesser of
2223 the following:

- 2224 (a) Forces corresponding to the resistance of the foundation to
2225 overturning uplift.

2231 (b) Forces as determined from nonlinear analysis as defined
2232 in Section C3.

2233 The inelastic link rotation angle shall be determined from the inelastic
2234 story drift. Alternatively, the inelastic link rotation angle may be
2235 determined from nonlinear analysis as defined in Section C3.

2236 **User Note:** The seismic load effect E used in the design of EBF
2237 members, such as the required axial strength used in the equations in
2238 Section F3.5, should be calculated from the analysis above.

2239

2240 4. System Requirements

2241 4a. Link Rotation Angle

2242 The link rotation angle is the inelastic angle between the link and the
2243 beam outside of the link when the total story drift is equal to the *design*
2244 *story drift*, Δ . The link rotation angle shall not exceed the following
2245 values:

2246 (1) 0.08 rad for links of length $1.6M_p/V_p$ or less.

2247 (2) 0.02 rad for links of length $2.6M_p/V_p$ or greater.

2248 The value determined by linear interpolation between the above values
2249 for links of length between $1.6M_p/V_p$ and $2.6M_p/V_p$.

2250 4b. Bracing of Link

2251 Bracing shall be provided at both the top and bottom link flanges at the
2252 ends of the link for I-shaped sections. Bracing shall have an available
2253 strength and stiffness as required for expected plastic hinge locations by
2254 Section D1.2b for *highly ductile members*.

2255 5. Members

2256 5a. Links

2257 Link beams subject to shear and flexure due to eccentricity between the
2258 intersections of brace centerlines and the beam centerline (or between the
2259 intersection of the brace and beam centerlines and the column centerline
2260 for links attached to columns) shall be provided. The link beam shall be
2261 considered to extend from brace connection to brace connection for
2262 center links and from brace connection to column face for link-to-column
2263 connections except as permitted by Section F3.6a.

2264 (1) Limitations

2265

2266 *Links* shall be I-shaped cross-sections (rolled wide-flange sections
2267 or built-up sections), or built-up box sections. HSS sections shall
2268 not be used as links.

2269
2270 Links shall meet the requirements of Section D1.1b for *highly*
2271 *ductile members*.

2272
2273 Exception: Flanges of links with I-shaped sections with link
2274 lengths, $e \leq 1.6 M_p/V_p$ are permitted to meet the requirements for
2275 *moderately ductile members*.

2276
2277 The web or webs of a link shall be single thickness. Doubler-
2278 plate reinforcement and web penetrations are not permitted.

2279 For links made of built-up cross-sections, complete-joint-
2280 penetration groove welds shall be used to connect the web (or
2281 webs) to the flanges.

2282 Links of built-up box sections shall have a moment of inertia, I_y ,
2283 about an axis in the plane of the EBF limited to $I_y > 0.67 I_x$, where
2284 I_x is the moment of inertia about an axis perpendicular to the
2285 plane of the of the EBF.

2286 (2) Shear Strength

2287
2288 The link *design shear strength*, $\phi_v V_n$, and the *allowable shear*
2289 *strength*, V_n/Ω_v , shall be the lower value obtained according to the
2290 *limit states* of *shear yielding* in the web and *flexural yielding* in
2291 the gross section. For both limit states:

$$2292 \phi_v = 0.90 \text{ (LRFD)} \quad \Omega_v = 1.67 \text{ (ASD)}$$

2293
2294 For shear yielding:

$$2295 V_n = V_p \quad \text{(F3-1)}$$

2296
2297 where

$$2300 V_p = 0.6 F_y A_{lw} \text{ for } P_r/P_c \leq 0.15 \quad \text{(F3-2)}$$

$$2301 V_p = 0.6 F_y A_{lw} \sqrt{1 - (P_r/P_c)^2} \text{ for } P_r/P_c > 0.15 \quad \text{(F3-3)}$$

$$2302 A_{lw} = (d - 2t_f)t_w \text{ for I-shaped link sections} \quad \text{(F3-4)}$$

$$2303 A_{lw} = 2(d - 2t_f)t_w \text{ for box link sections} \quad \text{(F3-5)}$$

$$2304 P_r = P_u \text{ (LRFD) or } P_a \text{ (ASD), as appropriate}$$

$$2305 P_u = \text{required axial strength using LRFD load combinations, kips (N)}$$

$$2306 P_a = \text{required axial strength using ASD load combinations, kips (N)}$$

$$2307 P_c = P_y \text{ (LRFD) or } P_y/1.5 \text{ (ASD) as appropriate}$$

$$P_y = \text{nominal axial yield strength} = F_y A_g \quad (\text{F3-6})$$

2312

2313 For flexural yielding:

2314

$$V_n = 2M_p/e \quad (\text{F3-7})$$

2316

2317 where

$$M_p = F_y Z \text{ for } P_r/P_c \leq 0.15 \quad (\text{F3-8})$$

$$M_p = F_y Z \left[\frac{1 - P_r/P_c}{0.85} \right] \text{ for } P_r/P_c > 0.15 \quad (\text{F3-9})$$

2320

 $e = \text{link length, in. (mm)}$

2321

2322

2323

2324

User Note: The requirements of Section F3.5a(2) and (3) have been reformatted from the 2005 *Seismic Provisions for Structural Steel Buildings* for clarity and simplicity. However, no change to the requirements is entailed in this reformatting.

2325

(3) Link Length

2326

2327

If $P_r/P_c > 0.15$, the length of the link shall be limited as follows:

2328

When $\rho' \leq 0.5$

2329

$$e \leq \frac{1.6M_p}{V_p} \quad (\text{F3-10})$$

2330

When $\rho' > 0.5$

2331

$$e \leq \frac{1.6M_p}{V_p} (1.15 - 0.3\rho') \quad (\text{F3-11})$$

2332

2333

where

DRAFT

2334

$$\rho' = \frac{P_r/P_c}{V_r/V_c} \quad (\text{F3-12})$$

2335

 $V_r = V_u$ (LRFD) or V_a (ASD), as appropriate, kips (N)

2336

 $V_u =$ required shear strength based on LRFD load

2337

combinations, kips (N)

2338

 $V_a =$ required shear strength based on ASD load

2339

combinations, kips (N)

2340

 $V_c = V_y$ (LRFD) or $V_y/1.5$ (ASD), as appropriate, kips

2341

(N)

2342

 $V_y =$ nominal shear yield strength, kips (N)

2343

$$= 0.6F_y A_{tw} \quad (\text{F3-13})$$

User Note: For links with low axial force there is no upper limit on link length. The limitations on link rotation angle in Section F3.4a result in a practical lower limit on link length.

(4) Link Stiffeners for I-shaped cross-sections

Full-depth web stiffeners shall be provided on both sides of the link web at the diagonal brace ends of the link. These stiffeners shall have a combined width not less than $(b_f - 2t_w)$ and a thickness not less than $0.75t_w$ or 3/8 in. (10 mm), whichever is larger, where b_f and t_w are the link flange width and link web thickness, respectively.

Links shall be provided with intermediate web stiffeners as follows:

- (i) Links of lengths $1.6M_p/V_p$ or less shall be provided with intermediate web stiffeners spaced at intervals not exceeding $(30t_w-d/5)$ for a link rotation angle of 0.08 rad or $(52t_w-d/5)$ for link rotation angles of 0.02 rad or less. Linear interpolation shall be used for values between 0.08 and 0.02 rad.
- (ii) Links of length greater than $2.6M_p/V_p$ and less than $5M_p/V_p$ shall be provided with intermediate web stiffeners placed at a distance of 1.5 times b_f from each end of the link.
- (iii) Links of length between $1.6M_p/V_p$ and $2.6M_p/V_p$ shall be provided with intermediate web stiffeners meeting the requirements of (i) and (ii) above.

Intermediate web stiffeners are not required in links of length greater than $5M_p/V_p$.

Intermediate web stiffeners shall be full depth. For links that are less than 25 in. (635 mm) in depth, stiffeners are required on only one side of the link web. The thickness of one-sided stiffeners shall not be less than t_w or 3/8 in. (10 mm), whichever is larger, and the width shall be not less than $(b_f/2) - t_w$. For links that are 25 in. (635 mm) in depth or greater, similar intermediate stiffeners are required on both sides of the web.

The *required strength* of fillet welds connecting a link stiffener to the link web is $A_{st}F_y$ (LRFD) or $A_{st}F_y/1.5$ (ASD), as appropriate, where A_{st} is the horizontal cross-sectional area of the link stiffener. The *required strength* of fillet welds connecting the

2386 stiffener to the link flanges is $A_{st}F_y / 4$ (LRFD) or $A_{st}F_y / 4(1.5)$
 2387 (ASD).

2388 (5) Link Stiffeners for box sections

2389 Full-depth web stiffeners shall be provided on one side of each
 2390 link web at the diagonal brace connection. These stiffeners may
 2391 be welded to the outside or inside face of the link webs. These
 2392 stiffeners shall each have a width not less than $b/2$, where b is the
 2393 inside width of the box. These stiffeners shall each have a
 2394 thickness not less than $0.75 t_w$ or $1/2$ in. (13 mm), whichever is
 2395 less.

2396 Box links shall be provided with intermediate web stiffeners as
 2397 follows:

- 2398 (a) For links of length $1.6M_p/V_p$ or less and with web depth-
 2399 to-thickness ratio, h/t_w , greater than or equal to
 2400 $0.64\sqrt{E/F_y}$, full-depth web stiffeners shall be provided
 2401 on one side of each link web, spaced at intervals not
 2402 exceeding $20t_w - (d - 2t_f)/8$.
- 2403 (b) For links of length $1.6M_p/V_p$ or less and with web depth-
 2404 to-thickness ratio, h/t_w less than $0.64\sqrt{E/F_y}$, no
 2405 intermediate web stiffeners are required.
- 2406 (c) For links of length greater than $1.6M_p/V_p$, no intermediate
 2407 web stiffeners are required.

2408 Intermediate web stiffeners shall be full depth, and may be
 2409 welded to the outside or inside face of the link webs.

2410
 2411 The *required strength* of fillet welds connecting a link stiffener to
 2412 the link web is $A_{st}F_y$ (LRFD) or $A_{st}F_y / 1.5$ (ASD), as appropriate,
 2413 where A_{st} is the horizontal cross-sectional area of the link
 2414 stiffener.

2415
 2416 **User Note:** Stiffeners of box links need not be welded to link
 2417 flanges.
 2418

2419 **5b. Diagonal Braces**

2420 Brace members shall satisfy width-thickness limitations in Section D1.1b
 2421 for *moderately ductile members*.

2422 **5c. Beam Outside Link**

2423 Where the beam outside of the link is a different section from the link,
 2424 the beam shall satisfy the width-thickness limitations in Section D1.1b
 2425 for *moderately ductile members*.

2426 **User Note:** The diagonal brace and beam segment outside of the link are
 2427 intended to remain essentially elastic under the forces generated by the
 2428 fully yielded and strain hardened link. Both the diagonal brace and beam
 2429 segment outside of the link are typically subject to a combination of large
 2430 axial force and bending moment, and therefore should be treated as
 2431 beam-columns in design, where the available strength is defined by
 2432 Chapter H of the *Specification*.

2433 Where the beam outside the link is the same member as the link, its
 2434 strength may be determined using expected material properties as
 2435 permitted by Section A3.2.

2436

2437 **5d. Columns**

2438 Column members shall satisfy width-thickness limitations in Section
 2439 D1.1b for *highly ductile members*.

2440 **5e. Protected Zones**

2441 Links in EBFs are a protected zone, and shall satisfy the requirements of
 2442 Section D1.3.

2443 **6. Connections**

2444 **6a. Link-to-Column Connections**

2447 (1) Requirements

2448 **DRAFT**

2449 Link-to-column connections shall be fully restrained (FR)
 2450 moment connections and shall satisfy the following requirements:

2451 (i) The connection shall be capable of sustaining the link
 2452 rotation angle specified in Section F3.4a.

2453 (ii) The shear resistance of the connection, measured at the
 2454 required link rotation angle, shall be at least equal to the
 2455 nominal shear strength of the link, V_n , as defined in
 2456 Section F3.5a(2).

2457 (iii) The flexural resistance of the connection, measured at the
 2458 required link rotation angle, shall be at least equal to the
 2459 moment corresponding to the nominal shear strength of
 2460 the link, V_n , as defined in Section F3.5a(2).

2461
 2462
 2463

- 2464
2465 (2) Conformance Demonstration
2466
2467 Link-to-column connections shall satisfy the above requirements
2468 by one of the following:
- 2469 (a) Use a connection *prequalified* for EBF in accordance with
2470 Section J1.

2471 **User Note:** There are no prequalified link-to-column
2472 connections.

- 2473 (b) Provide qualifying cyclic test results in accordance with
2474 Section J2. Results of at least two cyclic connection tests
2475 shall be provided and are permitted to be based on one of
2476 the following:
- 2477 (i) Tests reported in research literature or
2478 documented tests performed for other projects that
2479 are representative of project conditions, within the
2480 limits specified in Section J2.
- 2481 (ii) Tests that are conducted specifically for the
2482 project and are representative of project member
2483 sizes, material strengths, connection
2484 configurations, and matching connection material
2485 properties, within the limits specified in Section
2486 J2.

2487
2488 Exception: Cyclic testing of the reinforced connection is
2489 not required if the following conditions are met:

- 2490
2491 (1) Reinforcement at the beam-to-column connection
2492 at the link end precludes yielding of the beam
2493 over the reinforced length.
- 2494 (2) The *available strength* of the reinforced section
2495 and the connection equals or exceeds the *required*
2496 *strength* calculated based upon *adjusted link shear*
2497 *strength* as described in Section F3.3.
- 2498 (3) The link length (taken as the beam segment from
2499 the end of the reinforcement to the brace
2500 connection) does not exceed $1.6M_p/V_p$.
- 2501 (4) Full depth stiffeners as required in Section
2502 F3.5a(4) are placed at the link-to-reinforcement
2503 interface.

2504
2505 **6b. Diagonal Brace Connections**

2506 When oversized (OVS) holes are used, the required strength for the limit
2507 state of bolt slip need not exceed a load effect based upon using the *load*

2508 *combinations stipulated by the applicable building code, including the*
 2509 *amplified seismic load.*

2510 Connections of braces designed to resist a portion of the link end moment
 2511 shall be designed as fully restrained.

2512 **6c. Beam-to-Column Connections**

2513 Where a brace or gusset plate connects to both members at a beam-to-
 2514 column connection, the connection shall conform to one of the following:

2515 (1) The connection shall have sufficient capacity to accommodate
 2516 the required rotation at a minimum story drift of 2.5% of the story
 2517 height; or

2518 (2) The connection shall be designed to resist a moment simultaneous
 2519 with the required strength of the brace connection, equal to the
 2520 lesser of the following:

2521 (i) A moment corresponding to $1.1R_yF_yZ$ of the beam.

2522 (ii) A moment corresponding to $\Sigma 1.1R_yF_yZ$ of the column.

2523 This moment shall be considered in combination with the
 2524 required strength of the brace connection and beam connection,
 2525 including the amplified diaphragm collector forces.

2526

2527 **6d. Column Splices**

2528

2529 In addition to meeting the requirements of Section D2.5, column splices
 2530 in EBF shall be designed to develop at least 50% of the lesser available
 2531 flexural strength of the connected members.

2532

2533 The *required shear strength* shall be $\Sigma M_{pc} / H_c$ (LRFD) or $\Sigma M_{pc} / 1.5 H_c$
 2534 (ASD), as appropriate,

2535

2536

2537

where

2538 $\Sigma M_{pc} =$ the sum of the nominal plastic flexural strengths
 2539 $F_{yc}Z_c$ of the columns above and below the splice,
 2540 kip-in. (N-mm)

2541 $H_c =$ is the clear height of the column between beam
 2542 connections, including a structural slab, if present.
 2543 in. (mm)

2544 **6e. Demand Critical Welds**

2545 The following welds are *demand critical* welds, and shall satisfy the
2546 requirements of Section A3.4b:

2547 (1) Welds attaching the link flanges and the link web to the column
2548 where links connect to columns,

2549 (2) Welds connecting the webs to the flanges in built-up link beams.

2550 (3) Groove welds at column splices.

2551 **F4. BUCKLING-RESTRAINED BRACED FRAMES**2552 **1. Scope**

2553
2554
2555 *Buckling-restrained braced frames* (BRBF) shall be designed in
2556 conformance with this section. Minor eccentricities less than the beam
2557 depth are permitted if the resulting member and connection forces are
2558 addressed in the design and do not change the expected source of system
2559 ductility: compression and tension yielding of braces.

2560 **2. Basis of Design**

2561
2562 The analysis of BRBF shall be as required by Section F4.3. Braces shall
2563 provide the performance required by Section F4.2a, and demonstrate this
2564 conformance as required by Section F4.5a(3). Braces shall be designed,
2565 tested, and detailed to accommodate expected deformations. Expected
2566 deformations are those corresponding to a story drift of at least 2% of the
2567 story height or two times the design story drift, whichever is larger, in
2568 addition to brace deformations resulting from deformation of the frame
2569 due to gravity loading.

2570 BRBF shall be designed so that inelastic deformations under the design
2571 earthquake will occur primarily as brace yielding in tension and
2572 compression.

2573 **2a. Brace Strength**

2574 The adjusted brace strength shall be established on the basis of testing as
2575 described in this section.

2576 Where required by these *Provisions*, brace connections and adjoining
2577 members shall be designed to resist forces calculated based on the
2578 adjusted brace strength.

2579 The adjusted brace strength in compression shall be $\beta\omega R_y P_{ysc}$,

2580 where

2581 β = compression strength adjustment factor

2582 ω = strain hardening adjustment factor

2583 P_{ysc} = axial yield strength of steel core, ksi (MPa)

2584 The adjusted brace strength in tension shall be $\omega R_y P_{ysc}$.

2585 Exception: The factor R_y need not be applied if P_{ysc} is established using
2586 yield stress determined from a coupon test.

2587 The compression strength adjustment factor, β , shall be calculated as the
2588 ratio of the maximum compression force to the maximum tension force
2589 of the test specimen measured from the qualification tests specified in
2590 Section J3.4c for the expected deformations. The larger value of β from
2591 the two required brace qualification tests shall be used. In no case shall β
2592 be taken as less than 1.0.

2593 The strain hardening adjustment factor, ω , shall be calculated as the ratio
2594 of the maximum tension force measured from the qualification tests
2595 specified in Section J3.4c (for the expected deformations) to P_{ysc} of the
2596 test specimen. The larger value of ω from the two required qualification
2597 tests shall be used. Where the tested *steel core* material does not match
2598 that of the prototype, ω shall be based on coupon testing of the prototype
2599 material.

2600 3. Analysis

2601 Buckling-restrained braces shall not be considered as resisting gravity
2602 forces.

2603 Columns, beams, and connections in BRBF shall be designed to resist
2604 seismic forces from the *load combinations* in the *applicable building*
2605 *code* where the seismic load E is determined from an analysis in which
2606 all braces are assumed to resist forces corresponding to their adjusted
2607 strength in compression or in tension.

2608 Braces shall be determined to be in compression or tension neglecting the
2609 effects of gravity loads. Analyses shall consider both directions of frame
2610 loading.

2611 The adjusted brace strength in tension shall be as given in Section F4.2a.

2612 Exceptions:

2613 (1) It is permitted to neglect flexural forces resulting from seismic
 2614 drift in this determination. Moment resulting from a load applied to the
 2615 column between points of lateral support must be considered.

2616 (2) The required strength of columns need not exceed the lesser of the
 2617 following:

2618 (i) The forces corresponding to the resistance of the found-
 2619 ation to overturning uplift.

2620 (ii) Forces as determined from nonlinear analysis as defined
 2621 in Section C3.

2622
 2623 The brace deformation shall be determined from the inelastic story drift
 2624 and shall include the effects of beam vertical flexibility. Alternatively, the
 2625 brace deformation may be determined from nonlinear analysis as defined
 2626 in Section C3.

2627

2628 4. System Requirements

2629 4a. V- and Inverted V-Braced Frames

2630 V-type and inverted-V-type braced frames shall meet the following
 2631 requirements:

2632 (1) The *required strength* of beams intersected by braces, their
 2633 connections, and supporting members shall be determined based
 2634 on the *load combinations* of the *applicable building code*
 2635 assuming that the braces provide no support for dead and live
 2636 loads. For *load combinations* that include earthquake effects, the
 2637 vertical and horizontal earthquake effect, E , on the beam shall be
 2638 determined from the adjusted brace strengths in tension and
 2639 compression.

2640 (2) Beams shall be continuous between columns. Beams shall be
 2641 braced to meet the requirements for *moderately ductile members*
 2642 in Section D1.1. As a minimum, one set of lateral braces is
 2643 required at the point of intersection of the V-type (or inverted V-
 2644 type) braces, unless the beam has sufficient out-of-plane strength
 2645 and stiffness to ensure stability between adjacent brace points.

2646 **User Note:** The beam has sufficient out-of-plane strength and
 2647 stiffness if the beam bent in the horizontal plane meets the
 2648 required brace strength and required brace stiffness for column
 2649 nodal bracing as prescribed in the *Specification*. P_u may be taken
 2650 as the required compressive strength of the brace.

2651 For purposes of brace design and testing, the calculated maximum
 2652 deformation of braces shall be increased by including the effect of the

2653 vertical deflection of the beam under the loading defined in Section
2654 F4.4a(1).

2655 **4b. K-Braced Frames**

2656 K-type braced frames are not permitted for BRBF.

2657 **4c. Bracing Connections at Beam-to-Column Connections**

2658 Bracing connections at beam-to-column connections shall be designed to
2659 withstand an out-of-plane earthquake force E equal to 6% of the adjusted
2660 diagonal brace compression strength. The required bracing stiffness shall
2661 meet the provisions of Equation A-6-4 of Appendix 6 of the *Specification*
2662 where P_r is the adjusted diagonal brace compression strength.

2663 **5. Members**

2664 **5a. Diagonal Braces**

2666 (1) Assembly

2667 Braces shall be composed of a structural *steel core* and a system that
2668 restrains the *steel core* from buckling.

2669 (i) Steel Core

2670 Plates used in the *steel core* that are 2 in. (50 mm) thick
2671 or greater shall satisfy the minimum notch toughness
2672 requirements of Section A3.3.

2673 Splices in the *steel core* are not permitted.

2674 (ii) Buckling-Restraining System

2675 The buckling-restraining system shall consist of the
2676 casing for the *steel core*. In stability calculations, beams,
2677 columns, and gussets connecting the core shall be
2678 considered parts of this system.

2679 The buckling-restraining system shall limit local and
2680 overall buckling of the *steel core* for the expected
2681 deformations.

2682 **User Note:** Conformance to this provision is
2683 demonstrated by means of testing as described in Section
2684 F4.2b.

2685 (2) Available Strength

2686
2687 The *steel core* shall be designed to resist the entire axial force in
2688 the brace.

2689
2690 The brace design axial strength, ϕP_{ysc} (LRFD), and the brace
2691 allowable axial strength, P_{ysc} / Ω (ASD), in tension and

2692 compression, according to the limit state of yielding, shall be
 2693 determined as follows:

$$2694 \quad P_{ysc} = F_{ysc} A_{sc} \quad (F4-1)$$

$$2695 \quad \phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)}$$

2696
 2697 where

2700 F_{ysc} = specified minimum yield stress of the *steel core*,
 2701 or actual yield stress of the *steel core* as
 2702 determined from a coupon test, ksi (MPa)

2703 A_{sc} = net area of *steel core*, in.² (mm²)

2704 **User Note:** Load effects calculated based on adjusted brace
 2705 strengths should not be amplified by the overstrength factor, Ω_o .

2706 (3) Conformance Demonstration

2707 The design of braces shall be based upon results from qualifying
 2708 cyclic tests in accordance with the procedures and acceptance
 2709 criteria of Section J3. Qualifying test results shall consist of at
 2710 least two successful cyclic tests: one is required to be a test of a
 2711 brace subassembly that includes brace connection rotational
 2712 demands complying with Section J3.2 and the other shall be
 2713 either a uniaxial or a subassembly test complying with Section
 2714 J3.3. Both test types shall be based upon one of the following:

2715 (a) Tests reported in research or documented tests performed
 2716 for other projects.

2717 (b) Tests that are conducted specifically for the project.

2718 Interpolation or extrapolation of test results for different member
 2719 sizes shall be justified by rational analysis that demonstrates
 2720 stress distributions and magnitudes of internal strains consistent
 2721 with or less severe than the tested assemblies and that considers
 2722 the adverse effects of variations in material properties.
 2723 Extrapolation of test results shall be based upon similar
 2724 combinations of *steel core* and buckling-restraining system sizes.
 2725 Tests are permitted to qualify a design when the provisions of
 2726 Section J3 are met.

2727 **5b. Width-Thickness Limitations of Beams and Columns**

2728 Beam and column members shall meet the requirements of Section D1.1b
 2729 for *highly ductile members*.

2730 **5c. Protected Zones**

2731 The protected zone shall include the *steel core* of braces and elements
 2732 that connect the *steel core* to beams and columns, and shall satisfy the
 2733 requirements of Section D1.3.

2734 6. Connections

2735 6a. Diagonal Brace Connections

2736 (1) Required Strength

2737
 2738 The *required strength* of brace connections in tension and
 2739 compression (including beam-to-column connections if part of
 2740 the braced-frame system) shall be 1.1 times the adjusted brace
 2741 strength in compression (LRFD) or 1.1/1.5 times the adjusted
 2742 brace strength in compression (ASD) where the adjusted brace
 2743 strength is as defined in Section F4.2a.

2744 When oversized (OVS) holes are used, the required strength for
 2745 the limit state of bolt slip need not exceed a load effect based
 2746 upon using the *load combinations* stipulated by the *applicable*
 2747 *building code*, including the *amplified seismic load*.
 2748

2749 (2) Gusset-Plate Requirements

2750
 2751 The design of connections shall include considerations of local
 2752 and overall buckling. Lateral bracing consistent with that used in
 2753 the tests upon which the design is based is required.

2754 **User Note:** This provision may be met by designing the gusset
 2755 plate for a transverse force consistent with transverse bracing
 2756 forces determined from testing, by adding a stiffener to it to resist
 2757 this force, or by providing a brace to the gusset plate. Where the
 2758 supporting tests did not include transverse bracing, no such
 2759 bracing is required. Any attachment of bracing to the steel core
 2760 must be included in the qualification testing.

2761 6b. Column Splices

2762 In addition to meeting the requirements of Section D2.5, column splices
 2763 in BRBF shall be designed to develop at least 50% of the lesser available
 2764 flexural strength of the connected members, determined based on the
 2765 limit state of yielding. The *required shear strength* shall be $\Sigma M_{pc} / H_c$
 2766 (LRFD) or $\Sigma M_{pc} / 1.5H_c$ (ASD), as appropriate,

2767 where

2768 $\Sigma M_{pc} =$ sum of the nominal plastic flexural strengths,
 2769 $Z_c F_{yc}$, of the columns above and below the splice,
 2770 kip-in. (N-mm)

2771 $H_c =$ clear height of the column between beam
 2772 connections, including a structural slab, if present,
 2773 in. (mm)

2774 **6c. Demand Critical Welds**

2775 Groove welds of column splices shall be *demand critical welds* as
 2776 described in Section A3.4b.

2777 **6d. Beam-to-Column Connections**

2778 Where a brace or gusset plate connects to both members at a beam-to-
 2779 column connection, the connection shall conform to one of the following:

2780 (a) The connection shall have sufficient capacity to accommodate the
 2781 required rotation at a minimum story drift of 2.5% of the story
 2782 height; or

2783 (b) The connection shall be designed to resist a moment equal to the
 2784 lesser of the following:

2785 (i) A moment corresponding to $1.1R_yF_yZ$ of the beam.

2786 (ii) A moment corresponding to $\Sigma 1.1R_yF_yZ$ of the column.

2787 This moment shall be considered in combination with the
 2788 required strength of the brace connection and beam connection,
 2789 including the amplified diaphragm collector forces.
 2790
 2791

2792 **F5. SPECIAL PLATE SHEAR WALLS**

2793 **1. Scope**

2795 *Special plate shear walls* (SPSW) shall be designed in conformance with
 2796 this section.
 2797

2798 **2. Basis of Design**

2799 SPSW are expected to withstand significant inelastic deformations in the
 2800 webs when subjected to the forces resulting from the motions of the
 2801 *design earthquake*. SPSW shall be designed so that inelastic
 2802 deformations under the design earthquake will occur as web plate
 2803 yielding and as plastic-hinge formation in the ends of horizontal
 2804 boundary elements (HBEs).

2805 **3. Analysis**

2806 The webs of SPSW shall not be considered as resisting gravity forces.

2807 HBEs, VBEs and connections in SPSW shall be designed to resist
 2808 seismic forces from the *load combinations* in the *applicable building*
 2809 *code* where the seismic load E is determined from the following analysis:

2810 An analysis in which all webs are assumed to resist forces corresponding
 2811 to their expected strength in tension at an angle α as determined in
 2812 Section F5.5a and HBE are resisting flexural forces corresponding to
 2813 flexural forces at each end equal to $1.1R_yM_p$ (LRFD) or $(1.1/1.5)R_yM_p$
 2814 (ASD). Webs shall be determined to be in tension neglecting the effects
 2815 of gravity loads.

2816 The expected web yield stress shall be taken as R_yF_y . Where perforated
 2817 walls are used, the effective expected tension stress is as defined in
 2818 Section F5.7a.

2819 **User Note:** Shear forces per Equation E1-1 must be included in this
 2820 analysis. Designers should be aware that in some cases forces from the
 2821 analysis in the applicable building code will govern the design of HBE.

2822 **User Note:** Shear forces in beams and columns are likely to be high and
 2823 shear yielding may be a governing limit state.

2824 | 2825 4. System Requirements

2826 4a. Stiffness of Boundary Elements

2827 The vertical boundary elements (VBEs) shall have moments of inertia
 2828 about an axis taken perpendicular to the plane of the web, I_c , not less than
 2829 $0.0031 t_w h^4 / L$. The horizontal boundary elements (HBEs) shall have
 2830 moments of inertia about an axis taken perpendicular to the plane of the
 2831 web, I_b , not less than $0.0031 L^4 / h$ times the difference in web plate
 2832 thicknesses above and below,

2833 where

2834 t_w = thickness of the web, in. (mm)

2835 h = distance between HBE centerlines, in. (mm)

2836 I_b = moment of inertia of a HBE taken perpendicular to the
 2837 direction of the web plate line, in.⁴ (mm⁴)

2838 I_c = moment of inertia of a VBE taken perpendicular to the
 2839 direction of the web plate line, in.⁴ (mm⁴)

2840 L = distance between VBE centerlines, in. (mm)

2841 4b. HBE-to-VBE Connection Moment Ratio

2842 The beam-column moment ratio provisions in Section E3.4a shall be met
 2843 for all HBE/VBE intersections without consideration of the effects of the
 2844 webs.

2845 **4c. Bracing**

2846 HBE shall be braced to meet the requirements for *moderately ductile*
 2847 *members* in Section D1.2a.

2848 **4d. Openings in Webs**

2849 Openings in webs shall be bounded on all sides by intermediate boundary
 2850 elements extending the full width and height of the panel respectively,
 2851 unless otherwise justified by testing and analysis or permitted by Section
 2852 F5.7.

2853 **5. Members**

2854 **5a. Webs**

2855 The panel design shear strength, ϕV_n (LRFD), and the *allowable shear*
 2856 *strength*, V_n/Ω (ASD), according to the limit state of shear yielding, shall
 2857 be determined as follows:

$$2858 \quad V_n = 0.42 F_y t_w L_{cf} \sin 2\alpha \quad (\text{F5-1})$$

$$2859 \quad \phi = 0.90 \quad (\text{LRFD}) \quad \Omega = 1.67 \quad (\text{ASD})$$

2860 where

2861 t_w = thickness of the web, in. (mm)

2862 L_{cf} = clear distance between column flanges, in. (mm)

2863 α = angle of web yielding in radians, as measured relative to the
 2864 vertical. The angle of inclination, α , is permitted to be taken as 40° , or
 2865 may be calculated as follows:

$$2866 \quad \tan^4 \alpha = \frac{1 + \frac{t_w L}{2A_c}}{1 + t_w h \left(\frac{1}{A_b} + \frac{h^3}{360I_c L} \right)} \quad (\text{F5-2})$$

2867 h = distance between HBE centerlines, in. (mm)

2868 A_b = cross-sectional area of a HBE, in.² (mm²)

2869 A_c = cross-sectional area of a VBE, in.² (mm²)

2870 I_c = moment of inertia of a VBE taken perpendicular to the
2871 direction of the web plate line, in.⁴ (mm⁴)

2872 L = distance between VBE centerlines, in. (mm)

2873 **5b. Width-Thickness Limitations**

2874 HBE, VBE and intermediate boundary elements shall meet the
2875 requirements of Section D1.1b for *highly ductile members*.

2876 **5c. Protected Zone**

2877 The protected zone of SPSW shall satisfy Section D1.3 and include the
2878 following:

2879 (1) The webs of SPSW.

2880 (2) Elements that connect webs to HBEs and VBEs.

2881

2882 (3) The plastic hinging zones at each end of HBEs, over a region
2883 ranging from the face of the column to one beam depth beyond
2884 the face of the column, or as otherwise specified in Section E3.5c.

2885

2886 **6. Connections**

2887 **6a. Connections of Webs to Boundary Elements**

2888 The *required strength* of web connections to the surrounding HBE and
2889 VBE shall equal the *expected yield strength*, in tension, of the web
2890 calculated at an angle, α .

2891 **6b. HBE-to-VBE Connections**

2892 HBE-to-VBE connections shall satisfy the requirements of Section E1.6.

2893 (1) Required Strength

2894 The *required shear strength*, of a HBE-to-VBE connection shall be
2895 determined using a earthquake load effect, E , equal to the shear
2896 calculated from Equation E1-1 together with the shear resulting from the
2897 *expected yield strength* in tension of the webs yielding at an angle, α .

2898 (2) Panel Zones

2899

2900 The VBE panel zone next to the top and base HBE of the SPSW shall
2901 comply with the requirements in Section E3.6e.

2902 **6c. Column Splices**

2903 In addition to meeting the requirements of Section D2.5, column splices
 2904 in SPSW shall be designed to develop at least 50% of the lesser available
 2905 flexural strength of the connected members, determined based on the
 2906 limit state of yielding. Groove welds of column splices shall be *demand*
 2907 *critical* welds as described in Section A3.4b.

2908 7. Perforated Webs

2909 7a. Regular Layout of Circular Perforations

2910
 2911 A perforated plate conforming to this section is permitted to be used as
 2912 the web of a SPSW. Perforated webs shall have a regular pattern of holes
 2913 of uniform diameter spaced evenly over the entire web-plate area in an
 2914 array pattern so that holes align diagonally at a uniform angle to vertical.
 2915 Edges of openings shall have a surface roughness of 500 micro-inches
 2916 (13 microns) or less.

2917 (1) Strength

2918
 2919 The panel design shear strength, ϕV_n (LRFD), and the *allowable*
 2920 *shear strength*, V_n/Ω (ASD), according to the limit state of shear
 2921 yielding, shall be determined as follows for perforated webs:

$$2922 \quad V_n = 0.42 F_y t_w L_{cf} (1 - 0.7 D/S_{diag}) \quad (F5-3)$$

$$2923 \quad \phi = 0.90 \quad (\text{LRFD}) \quad \Omega = 1.67 \quad (\text{ASD})$$

2924 where

2925
 2926 D = the diameter of the holes, in. (mm)
 2927 S_{diag} = the shortest center-to-center distance between the
 2928 holes, in. (mm)

2929 (2) Spacing

2930
 2931 The spacing, S_{diag} , shall be at least $1.67D$.
 2932

2933
 2934 The distance between the first holes and web connections to the
 2935 HBEs and VBEs shall be at least D , but not exceeding
 2936 $(D+0.7S_{diag})$.
 2937

2938 (3) Stiffness

2939
 2940 The stiffness of such regularly perforated infill plates shall be
 2941 calculated using an effective web-plate thickness, t_{eff} , is given
 2942 by:
 2943

2944

$$t_{eff} = \frac{1 - \frac{\pi}{4} \left(\frac{D}{S_{diag}} \right)}{1 - \frac{\pi}{4} \left(\frac{D}{S_{diag}} \right) \left(1 - \frac{N_r D \sin \alpha}{H_c} \right)} t_w \quad (F5-4)$$

2945

2946

where

2947

H_c = clear column (and web-plate) height between
beam flanges, in. (mm)

2948

2949

t_w = web-plate thickness, in. (mm)

2950

N_r = the number or horizontal rows of perforations

2951

α = angle of the shortest center-to-center lines in the
opening array to vertical, degrees

2952

2953

(4) Effective expected tension stress

2954

The effective expected tension stress to be used in place of the
effective tension stress for analysis per Section F5.3 is $R_y F_y (1 - 0.7 D/S_{diag})$.

2955

2956

2957

7b. Reinforced Corner Cut-out

2958

2959

Quarter-circular cut-outs are permitted at the corners of the webs
provided that the webs are connected to a reinforcement arching plate
following the edge of the cut-outs. The plates shall be designed to allow
development of the full strength of the solid web and maintain its
resistance when subjected to deformations corresponding to the design
story drift. This is deemed to be achieved if the following conditions are
met.

2960

2961

2962

2963

2964

2965

2966

2967

(a) Design for tension

2968

The arching plate shall have the *available strength* to resist the axial
tension force resulting from web-plate tension in the absence of other
forces.

2969

2970

2971

2972

2973

$$P_u = \frac{R_y F_y t_w R^2}{4e} \quad (\text{LRFD}) \quad (F5-5a)$$

2974

2975

or

2976

2977

$$P_a = \frac{R_y F_y t_w R^2 / 1.5}{4e} \quad (\text{ASD}), \text{ as appropriate} \quad (F5-5b)$$

2978

2979

where

2980

2981

R = the radius of the cut-out, in. (mm)

2982
$$e = R(1 - \sqrt{2}/2), \text{ in. (mm)} \quad (\text{F5-6})$$

2983

2984 HBES and VBES shall be designed to resist the tension axial forces acting
 2985 at the end of the arching reinforcement.

2986

2987 (b) Design for beam-to-column connection forces

2988

2989 The arching plate shall have the available strength to resist the combined
 2990 effects of axial force and moment resulting from connection deformation
 2991 in the absence of other forces. These forces are:

2992
$$P_u = \frac{15 E I_y}{4 e^2} \frac{\Delta}{h} \quad (\text{LRFD}) \quad (\text{F5-7a})$$

2993 or

2994

2995
$$P_a = \frac{15 E I_y}{1.5(4 e^2)} \left(\frac{\Delta}{h} \right) \quad (\text{ASD}), \text{ as appropriate} \quad (\text{F5-7b})$$

2996

2997
$$M_{uy} = \frac{P_u R}{2} \quad (\text{LRFD}) \quad (\text{F5-8a})$$

2998 or

2999
$$M_{ay} = \frac{P_a R}{2} \quad (\text{ASD}), \text{ as appropriate} \quad (\text{F5-8b})$$

3000 where

3001

3002 E = modulus of elasticity, ksi (MPa)

3003 I_y = moment of inertia of the plate, in.⁴ (mm⁴)

3004 h = the story height, in. (mm)

3005 Δ = the design story drift, in. (mm)

3006

3007

CHAPTER G

3008

COMPOSITE MOMENT FRAME SYSTEMS

3009

3010 **G1. COMPOSITE ORDINARY MOMENT FRAMES (C-OMF)**3011 **1. Scope**

3012 *Composite ordinary moment frames (C-OMF) shall be designed in*
3013 *conformance with this section. This Section is applicable to moment*
3014 *frames with fully-restrained (FR) connections that consist of either*
3015 *composite or reinforced concrete columns and structural steel, concrete-*
3016 *encased composite, or composite beams.*

3017 **2. Basis of Design**

3018 C-OMF are expected to withstand minimal inelastic drift.

3019 **User Note:** Composite ordinary moment frames, comparable to
3020 reinforced concrete ordinary moment frames, are only permitted in
3021 Seismic Design Categories B or below in ASCE 7. This is in contrast to
3022 steel ordinary moment frames, which are permitted in higher Seismic
3023 Design Categories. The design requirements are commensurate with
3024 providing minimal ductility in the members and connections.

3025 **3. Analysis**

3026 There are no additional requirements beyond those in the *Specification*
3027 and the *applicable building code*.

3028 **4. System Requirements**

3029 There are no additional requirements beyond those in the *Specification*
3030 and the *applicable building code*.

3031 **5. Members**

3032 There are no additional requirements for steel or composite members
3033 beyond those in the *Specification*. Reinforced concrete columns shall
3034 meet the requirements of ACI 318, excluding Chapter 21.

3035 **6. Connections**

3036

3037

3038

Connections shall be fully-restrained (FR). Connections shall be designed
for the applicable load combinations as described in Sections B2 and B3..

3039 Beam-to-column connection design strengths shall be determined
3040 according to the *Specification* and Section D2.7.

3041 **G2. COMPOSITE INTERMEDIATE MOMENT FRAMES (C-IMF)**

3042 **1. Scope**

3043 *Composite intermediate moment frames (C-IMF)* shall be designed in
3044 conformance with this section. This Section is applicable to moment
3045 frames with fully-restrained (FR) connections that consist of either
3046 composite or reinforced concrete columns and structural steel, concrete-
3047 encased composite or *composite beams*.

3048 **2. Basis of Design**

3049 C-IMF shall be designed to provide limited inelastic drift capacity
3050 through flexural yielding of the C-IMF beams, column panel zones, and
3051 columns.

3052 **User Note:** Composite intermediate moment frames, comparable to
3053 reinforced concrete intermediate moment frames, are only permitted in
3054 Seismic Design Categories C or below in SEI/ASCE 7. This is in
3055 contrast to steel intermediate moment frames, which are permitted in
3056 higher Seismic Design Categories. The design requirements are
3057 commensurate with providing limited ductility in the members and
3058 connections.

3059 **3. Analysis**

3060 There are no additional requirements beyond those in the *Specification*
3061 and the *applicable building code*.

3062 **4. System Requirements**

3063 **4a. Stability Bracing of Beams**

3064 Beams shall be braced to meet the requirements for *moderately ductile*
3065 *members* in Section D1.2a.

3066 In addition, member braces shall be placed near concentrated loads,
3067 changes in cross-section, and other locations where analysis indicates that
3068 a plastic hinge will form during inelastic deformations of the C-IMF.

3069 The *required strength* of stability bracing provided adjacent to plastic
3070 hinges shall be as required by Section D1.2c.

3071 **5. Members**

3072 **5a. Detailing Requirements**

3073 Steel and composite members shall meet the requirements of Sections
3074 D1.2, D1.4b and D1.4c for *moderately ductile members*.

3075 **5b. Columns**

3076 Steel, composite, and reinforced concrete columns shall meet the
3077 requirements of Section D1.4a.

3078 **5c. Beam Flanges**

3079 Abrupt changes in the beam flange area are not permitted in plastic hinge
3080 regions. The drilling of flange holes or trimming of beam flange width is
3081 not permitted unless testing or qualification demonstrates that the
3082 resulting configuration can develop stable plastic hinges.

3083 **5d. Protected Zones**

3084 The region at each end of the beam subject to inelastic straining shall be
3085 designated as a protected zone, and shall meet the requirements of
3086 Section D1.3.

3087 **User Note:** The plastic hinge zones at the ends of C-IMF beams should
3088 be treated as *protected zones*. In general, the protected zone will extend
3089 from the face of the composite column to one-half of the beam depth
3090 beyond the plastic hinge point.

3091 **6. Connections**

3092
3093 Connections shall be fully-restrained (FR) and shall meet the
3094 requirements of Section D2 and this section.

3095 **User Note:** All subsections of Section D2 are relevant for C-IMF.

3096 **6a. Demand Critical Welds**

3097 Complete-joint-penetration groove welds of beam flanges, diaphragm
3098 plates, shear plates, and beam webs to columns shall be *demand critical*
3099 welds as described in Section A3.4b.

3100 **6b. Beam-to-Column Connection Requirements**

3101 Beam-to-composite column connections used in the SFRS shall satisfy
3102 the following requirements:

3103 (1) The connection shall be capable of sustaining an *interstory drift*
3104 *angle* of at least 0.02 radians.

3105 (2) The *measured flexural resistance* of the connection, determined at
3106 the column face, shall equal at least $0.80M_p$ of the connected beam

3107 at an *interstory drift angle* of 0.02 rad, where M_p is defined as the
 3108 nominal flexural strength of the steel, concrete-encased or
 3109 composite beams and shall meet the requirements of *Specification*
 3110 Chapter I.

3111 6c. Required Shear Strength

3112 The *required shear strength* of the connection, V_u , shall be determined
 3113 using the following quantity for the earthquake load effect E :

$$3114 \quad E = 2[1.1M_{p,exp}]/L_h \quad (G2-1)$$

3115 where $M_{p,exp}$ is the expected flexural strength of the steel, concrete-
 3116 encased or composite beams. For concrete-encased or composite beams,
 3117 $M_{p,exp}$ shall be calculated using the plastic stress distribution or the strain
 3118 compatibility method. Appropriate R_y factors shall be used for different
 3119 elements of the cross-section while establishing section force equilibrium
 3120 and calculating the flexural strength. L_h shall be equal to the distance
 3121 between beam plastic hinge locations, in. (mm).

3122 **User Note:** For steel beams, $M_{p,exp}$ in Equation G2-1 may be taken as
 3123 $R_y M_p$ of the beam.

3124 6d. Continuity Plates and Diaphragms

3125 Connection diaphragm plates are permitted for filled composite columns
 3126 both external to the column and internal to the column.

3127 Where diaphragm plates are used, the thickness of the plates shall be at
 3128 least the thickness of the beam flange.

3129 The diaphragm plates shall be welded around the full perimeter of the
 3130 column using either complete-joint-penetration welds or two sided fillet
 3131 welds. The *required strength* of these joints shall not be less than the
 3132 *available strength* of the contact area of the plate with the column sides.

3133 Internal diaphragms shall have circular openings sufficient for placing the
 3134 concrete.

3135 6e. Column Splices

3136 In addition to the requirements of Section D2.5, column splices shall
 3137 comply with the requirements of this section. Where groove welds are
 3138 used to make the splice, they shall be complete-joint-penetration groove
 3139 welds. When column splices are not made with groove welds, they shall
 3140 have a *required flexural strength* that is at least equal to the moment
 3141 capacity M_{pcc}^* of the smaller composite column. The *required shear*
 3142 *strength* of column web splices shall be at least equal to $\Sigma M_{pcc}^*/H$, where
 3143 ΣM_{pcc}^* is the sum of the nominal flexural strengths of the composite

3144 columns above and below the splice. M_{pcc}^* is defined as the nominal
 3145 flexural strength of the composite or reinforced concrete column. For
 3146 composite columns, the nominal flexural strength shall meet the
 3147 requirements of *Specification* Chapter I with consideration of the required
 3148 axial strength, P_{rc} . For reinforced concrete columns, the nominal flexural
 3149 strength shall be calculated based on the provisions of ACI 318 with
 3150 consideration of the required axial strength, P_{rc} .

3151 G3. COMPOSITE SPECIAL MOMENT FRAMES (C-SMF)

3152 1. Scope

3153 *Composite special moment frames* (C-SMF) shall be designed in
 3154 conformance with this section. This Section is applicable to moment
 3155 frames with fully-restrained (FR) connections that consist of either
 3156 composite or reinforced concrete columns and either structural steel or
 3157 concrete-encased composite, or *composite beams*.

3158 2. Basis of Design

3159 C-SMF shall be designed to provide significant inelastic drift capacity
 3160 through flexural yielding of the C-SMF beams and limited yielding of the
 3161 column panel zones. Except where otherwise permitted in this section,
 3162 columns shall be designed to remain essentially elastic when subjected to
 3163 forces associated with fully yielded and strain hardened beams or girders.
 3164 Flexural yielding columns at the base shall be permitted.

3165 3. Analysis

3166 There are no additional requirements beyond those in the *Specification*
 3167 and the *applicable building code*.

3168 4. System Requirements

3169 4a. Moment Ratio

3170 The following relationship shall be satisfied at beam-to-column
 3171 connections:

$$3172 \frac{\sum M_{pcc}^*}{\sum M_{p,exp}^*} > 1.0 \quad (G3-1)$$

3173 where

3174 $\sum M_{pcc}^*$ = the sum of the nominal flexural strengths of
 3175 the composite columns above and below the
 3176 joint at the intersection of the beam and
 3177 column centerlines. $\sum M_{pcc}^*$ is determined by
 3178 summing the projections of the nominal
 3179 flexural strengths of the columns (including

3180 haunches where used) above and below the
 3181 joint to the beam centerline with a reduction
 3182 for the axial force in the column, where
 3183 M_{pcc}^* is defined in Section G2.6e. When the
 3184 centerlines of opposing beams in the same
 3185 joint do not coincide, the mid-line between
 3186 centerlines shall be used.

3187 $\Sigma M_{p,exp}^*$ = the sum of the expected flexural strengths of
 3188 the steel beams or concrete-encased
 3189 composite beams at the intersection of the
 3190 beam and column centerlines. $\Sigma M_{p,exp}^*$ is
 3191 determined by summing the projections of
 3192 the expected flexural strengths of the beams
 3193 at the plastic hinge locations to the column
 3194 centerline. It is permitted to take $\Sigma M_{p,exp}^* =$
 3195 $\Sigma(1.1M_{p,exp} + M_{uv})$, where $M_{p,exp}$ is calculated
 3196 as specified in Section G2.6c.

3197 M_{uv} = moment due to shear amplification from the
 3198 location of the plastic hinge to the column
 3199 centerline

3200 Exception: The exceptions of Section E3.4a shall apply with the
 3201 following modifications.

3202 (1) For exception (1), the force limit in Section E3.4a shall be $P_{rc} <$
 3203 $0.1P_c$.

3204 (2) *Composite columns* exempted from the requirement of Equation
 3205 G3-1 shall have transverse reinforcement that meets the
 3206 requirements of Section D1.4b(2)(ii).

3207 **4b. Stability Bracing of Beams**

3208 Beams shall be braced to meet the requirements for *highly ductile*
 3209 *members* in Section D1.2b.

3210 In addition, stability bracing shall be placed near concentrated forces,
 3211 changes in cross-section, and other locations where analysis indicates that
 3212 a plastic hinge will form during inelastic deformations of the C-SMF.

3213 The *required strength* of stability bracing provided adjacent to plastic
 3214 hinges shall be as required by Section D1.2c.

3215 **4c. Stability Bracing at Beam-to-Column Connections**

3216 Composite columns with unbraced connections shall satisfy the
 3217 requirements of Section E3.4c(2).

3218 **5. Members**3219 **5a. Detailing Requirements**

3220 Steel and composite members shall meet the requirements of Sections
3221 D1.1, D1.4b and D1.4c for *highly ductile members*.

3222
3223 Exception: Reinforced concrete-encased beams shall satisfy the
3224 requirements for Section D1.1 for *moderately ductile members* if the
3225 reinforced concrete cover is at least 2 in. (50 mm) and confinement is
3226 provided by hoop reinforcement in regions where plastic hinges are
3227 expected to occur under seismic deformations. Hoop reinforcement shall
3228 meet the requirements of ACI 318 Section 21.5.3.

3229
3230 Concrete-encased composite beams that are part of C-SMF shall also
3231 meet the following requirements. The distance from the maximum
3232 concrete compression fiber to the plastic neutral axis shall not exceed the
3233 following requirement

$$3234 \quad Y_{PNA} = \frac{Y_{con} + d_b}{1 + \left(\frac{1,700 F_y}{E} \right)} \quad (G3-2)$$

3235 where

3236 Y_{con} = distance from the top of the steel beam to the top of
3237 the concrete, in. (mm)

3238 d_b = depth of the steel beam, in. (mm)

3239 F_y = specified minimum yield stress of the steel beam, ksi
3240 (MPa)

3241 E = elastic modulus of the steel beam, ksi (MPa)
3242

3243 **5b. Columns**

3244 Steel, composite, and reinforced concrete columns shall meet the
3245 requirements of Section D1.4a.

3246 **5c. Beam Flanges**

3247 Abrupt changes in beam flange area are not permitted in plastic hinge
3248 regions. The drilling of flange holes or trimming of beam flange width is
3249 not permitted unless testing or qualification demonstrates that the
3250 resulting configuration can develop stable plastic hinges to the required
3251 interstory drift angle.

3252 **5d. Protected Zones**

3253 The region at each end of the beam subject to inelastic straining shall be
 3254 designated as a protected zone, and shall meet the requirements of
 3255 Section D1.3.

3256 **User Note:** The plastic hinge zones at the ends of C-SMF beams should
 3257 be treated as *protected zones*. In general, the protected zone will extend
 3258 from the face of the composite column to one-half of the beam depth
 3259 beyond the plastic hinge point.

3260 **6. Connections**

3261
 3262 Connections shall be fully-restrained (FR) and shall meet the
 3263 requirements of Section D2 and this section.

3264 **User Note:** All subsections of Section D2 are relevant for C-SMF.

3265 **6a. Demand Critical Welds**

3266 Complete-joint-penetration groove welds of beam flanges, diaphragm
 3267 plates, shear plates, and beam webs to columns shall be *demand critical*
 3268 welds as described in Section A3.4b.

3269 **6b. Beam-to-Composite Column Connection Requirements**

3270 Beam-to-composite column connections used in the *seismic load force*
 3271 *resisting system* (SLRS) shall satisfy the following two requirements:

- 3272 (1) The connection shall be capable of sustaining an *interstory drift*
 3273 *angle* of at least 0.04 rad.
- 3274 (2) The *measured flexural resistance* of the connection, determined
 3275 at the column face, shall equal at least $0.80M_p$ of the connected
 3276 beam at an *interstory drift angle* of 0.04 rad, where M_p is
 3277 calculated as in Section G2.6b.

3278 The two requirements above shall be satisfied as follows:

- 3279 (1) When beams are interrupted at the connection, the connections shall
 3280 be qualified using test results obtained in accordance with Section J2.
 3281 Results of at least two cyclic connection tests shall be provided, and
 3282 shall be based on one of the following:
 3283
- 3284 (i) Tests reported in research literature or documented tests
 3285 performed for other projects that represent the project
 3286 conditions, within the limits specified in J2.
 - 3287 (ii) Tests that are conducted specifically for the project and are
 3288 representative of project member sizes, material strengths,

3289 connection configurations, and matching connection
3290 processes, within the limits specified by J2.

3291

3292 (2) When beams are uninterrupted or continuous through the composite
3293 or reinforced concrete column, beam flange welded joints are not
3294 used, and the connection is not otherwise susceptible to premature
3295 fracture, the performance requirements of Section G3.6b shall be
3296 demonstrated by qualifying test results in accordance with (1), or
3297 other substantiating data.

3298 Connections that accommodate the required *interstory drift angle* within
3299 the connection elements and provide the *measured flexural resistance*
3300 and shear strengths specified in Section G3.6c are permitted. In addition
3301 to satisfying the requirements noted above, the design shall demonstrate
3302 that any additional drift due to connection deformation can be
3303 accommodated by the structure. The design shall include analysis for
3304 stability effects of the overall frame, including second-order effects.

3305 **6c. Required Shear Strength**

3306 The *required shear strength* of the connection, V_u , shall be determined
3307 using the following quantity for the earthquake load effect E .

$$3308 \quad E = 2[1.1M_{p,exp}]/L_h \quad (G3-2)$$

3309 where $M_{p,exp}$ is the expected flexural strength of the steel, concrete-
3310 encased, or composite beams. For concrete-encased or composite beams,
3311 $M_{p,exp}$ shall be calculated according to Section G2.6c, and L_h shall be the
3312 distance between plastic hinge locations, in. (mm).

3313 **6d. Connection Diaphragm Plates**

3314 The continuity plates or diaphragms used for infilled column moment
3315 connections shall meet the requirements of Section G2.6d.

3316 **6e. Column Splices**

3317 Composite column splices shall satisfy the requirements of Section
3318 G2.6e.

3319 **G4. COMPOSITE PARTIALLY RESTRAINED MOMENT FRAMES** 3320 **(C-PRMF)**

3321 **1. Scope**

3322 *Composite partially restrained moment frames* (C-PRMF) shall be
3323 designed in conformance with this section. This Section is applicable to
3324 moment frames that consist of structural steel columns and *composite*

3325 *beams* that are connected with partially-restrained (PR) moment
3326 connections that meet the requirements in *Specification* Section B3.6b(b).

3327 **2. Basis of Design**

3328 C-PRMF shall be designed so that yielding occurs in the ductile
3329 components of the composite PR beam-to-column moment connections.
3330 Limited yielding is permitted at other locations, such as column base
3331 connections.

3332 **3. Analysis**

3333 Analysis shall meet the requirements of Chapter C as modified in this
3334 section.

3335 Connection flexibility and composite beam action shall be accounted for
3336 in determining the dynamic characteristics, strength and drift of C-
3337 PRMF.

3338 For purposes of analysis, the stiffness of beams shall be determined with
3339 an effective moment of inertia of the composite section.

3340 **4. System Requirements**

3341 There are no additional requirements beyond the *Specification*.

3342 **5. Members**

3343 **5a. Columns**

3344 Steel columns shall meet the requirements of Section D1.4a and shall
3345 meet the requirements of Sections D1.1 for *highly ductile members*.

3346 **5b. Beams**

3347 *Composite beams* shall be unencased, fully composite and shall meet the
3348 requirements of *Specification* Chapter I. A solid slab shall be provided
3349 for a distance of 12 in. around the column.

3350 **6. Connections**

3351
3352 Connections shall be partially-restrained (PR) and shall meet the
3353 requirements of Section D2 and this section.

3354 **User Note:** All subsections of Section D2 are relevant for C-PRMF.

3355 **6a. Required Strength**

3356
3357 The *required strength* of the beam-to-column PR moment connections
3358 shall be determined considering the effects of connection flexibility and
3359 second-order moments. In addition, composite connections shall have a

3360 nominal strength that is at least equal to 50% of M_p , where M_p is the
3361 nominal plastic flexural strength of the connected structural steel beam
3362 ignoring composite action.

3363

3364 **6b. Connection Rotational Capacity**

3365 Beam-to-composite column connections used in the SFRS shall satisfy
3366 the following requirements:

3367 (1) The connection shall be capable of sustaining an interstory drift
3368 angle of at least 0.02 rad.

3369 (2) The *measured flexural resistance* of the connection determined at
3370 the column face shall increase monotonically to a value of at least
3371 $0.5M_p$ of the connected beam at an *interstory drift angle* of 0.02
3372 rad, where M_p is defined as the nominal flexural strength of the
3373 composite beam and shall meet the requirements of *Specification*
3374 Chapter I.

3375 The connections shall be qualified for the above requirements using test
3376 results obtained in accordance with Section J2. Results of at least two
3377 cyclic connection tests shall be provided, and shall be based on one of the
3378 following:

3379 (a) Tests reported in research literature or documented tests
3380 performed for other projects that represent the project conditions,
3381 within the limits specified in Section J2.

3382

3383 (b) Tests that are conducted specifically for the project and are
3384 representative of project member sizes, material strengths,
3385 connection configurations, and matching connection processes,
3386 within the limits specified by Section J2.
3387

3388 **6c. Connection Strength**

3389 Nominal connection strength, $M_{n,PR}$, shall be taken as the bending
3390 strength at 0.02 rad during the first excursion at which that rotation is
3391 exceeded.

3392 **6d. Conformance Demonstration**

3393 The ability of the connection to reach the *rotational capacity* specified
3394 above shall be substantiated by cyclic testing.

3395 **6e. Column Splices**

3396 Composite column splices shall satisfy the requirements of Section
3397 G2.6e.

3398

CHAPTER H

3399

COMPOSITE BRACED FRAME AND SHEAR WALL SYSTEMS

3400

3401

3402 **H1. COMPOSITE ORDINARY BRACED FRAMES (C-OBF)**3403 **1. Scope**

3404 *Composite ordinary braced frames (C-OBF)* shall be designed in
3405 conformance with this section. Columns shall be structural steel, encased
3406 composite, filled composite or reinforced concrete members. Beams shall
3407 be either structural steel or composite beams. Braces shall be structural
3408 steel or filled composite members. This Section is applicable to braced
3409 frames that consist of concentrically connected members. Minor
3410 eccentricities are permitted if they are accounted for in the design.

3411 **2. Basis of Design**

3412 C-OBF are expected to withstand limited inelastic deformations in their
3413 members and connections when subjected to the forces resulting from the
3414 motions of the *design earthquake*. C-OBF shall meet the requirements of
3415 Section F1, except as modified in this Section.

3416 **User Note:** Composite ordinary braced frames, comparable to other
3417 steel braced frames designed per the *Specification* using $R = 3$, are only
3418 permitted in Seismic Design Categories C or below in SEI/ASCE 7. This
3419 is in contrast to steel ordinary braced frames, which are permitted in
3420 higher Seismic Design Categories. The design requirements are
3421 commensurate with providing minimal ductility in the members and
3422 connections.

3423 **3. Analysis**

3424 There are no additional requirements beyond those in the *Specification*
3425 and the *applicable building code*.

3426 **4. System Requirements**

3427 There are no additional requirements beyond the *Specification* and the
3428 *applicable building code*.

3429 **5. Members**3430 **5a. Detailing Requirements**

3431 There are no additional requirements beyond the *Specification* and the
3432 *applicable building code*.

3433 **5b. Columns**

3434 There are no additional requirements beyond the *Specification* and the
3435 *applicable building code*. Reinforced concrete columns shall meet the
3436 requirements of ACI 318, excluding Chapter 21.

3437 **5c. Braces**

3438 There are no additional requirements beyond the *Specification* for
3439 structural steel and filled composite braces.

3440 **6. Connections**

3441 Connections shall meet the requirements of Section D2.7 and the
3442 *Specification* and the *applicable building code*.

3443 **H2. COMPOSITE SPECIAL CONCENTRICALLY BRACED FRAMES**
3444 **(C-SCBF)**

3445 **1. Scope**

3446 *Composite special concentrically braced frames (C-SCBF)* shall be
3447 designed in conformance with this section. Columns shall be encased or
3448 filled composite. Beams shall be either structural steel or composite
3449 beams. Braces shall be structural steel or filled composite members. This
3450 Section is applicable to braced frames that consist of concentrically
3451 connected members. Minor eccentricities are permitted if they are
3452 accounted for in the design.

3453 **2. Basis of Design**

3454 C-SCBF shall be designed to provide inelastic drift capacity through
3455 cyclic buckling and tension yielding of braces when subjected to the
3456 forces resulting from the motions of the *design earthquake*. SCBF shall
3457 be designed so that inelastic deformations under the design earthquake
3458 will occur primarily as brace buckling and yielding in tension.

3459 **3. Analysis**

3460 The analysis requirements for C-SCBF shall satisfy the analysis
3461 requirements of Section F2.3.

3462 **4. System Requirements**

3463 The system requirements for C-SCBF shall satisfy the system
3464 requirements of Section F2.4.

3465 **5. Members**

3466 **5a. Detailing Requirements**

3467 Steel or composite columns or braces shall meet the requirements of
 3468 Section D1.1, D1.4b and D1.4c for *highly ductile members*. Steel or
 3469 composite beams shall meet the requirements of Section D1.1 for
 3470 *moderately ductile members*.

3471 **User Note:** In order to satisfy the compactness requirement of Section
 3472 F2.5a the actual width-to-thickness ratio of square and rectangular filled
 3473 composite braces may be multiplied by a factor, $[(0.264 + 0.0082KL/r)]$,
 3474 for KL/r between 35 and 90, KL/r being the effective slenderness ratio of
 3475 the brace.

3476 **5b. Columns**

3477 Composite columns shall meet the requirements of Section D1.4a.

3478 **5c. Braces**

3479 Structural steel and filled composite braces shall meet the requirements
 3480 for SCBF of Section F2.5b. The radius of gyration in Section F2.5b shall
 3481 be taken as that of the steel section alone.

3482 **6. Connections**

3483 Design of connections in C-SCBF shall be based on Section D2 and the
 3484 provisions of this section.

3485 **6a. Required Strength of Brace Connections**

3486 The required strength of brace connections shall meet the
 3487 requirements of Section F2.6a, except that design of C-SCBF is limited
 3488 to the load and resistance factor design (LRFD) method.

3489 **6b. Beam-to-Column Connections**

3490 Beam-to-column connections shall conform to one of the following:

- 3491 (1) Beam-to-column connections shall meet the requirements for FR
 3492 moment connections as specified in Sections D2 and G2.6; or
- 3493 (2) Beam-to-column connections shall have sufficient rotation capacity
 3494 to accommodate the required rotation at a minimum story drift of
 3495 2.5% of the story height.

3496 **6c. Column Splices**

3497 Column splices in C-SCBF shall be designed following the requirements
 3498 of Section G2.6e.
 3499

3500 **H3. COMPOSITE ECCENTRICALLY BRACED FRAMES (C-EBF)**

3501 **1. Scope**
 3502

3503 *Composite eccentrically braced frames (C-EBF)* shall be designed in
3504 conformance with this section. Columns shall be encased composite or
3505 filled composite. Beams shall be structural steel. Braces shall be
3506 structural steel. This Section is applicable to braced frames for which
3507 one end of each brace intersects a beam at an eccentricity from the
3508 intersection of the centerlines of the beam and an adjacent brace.

3509

3510 2. Basis of Design

3511 C-EBF shall be designed so that inelastic deformations under the *design*
3512 *earthquake* will occur primarily as shear yielding in the *links*. Diagonal
3513 braces, columns and beam segments outside of the link shall be designed
3514 to remain essentially elastic under the maximum forces that can be
3515 generated by the fully yielded and strain-hardened link. The *available*
3516 *strength* of members shall meet the requirements in the *Specification*,
3517 except as modified in this Section. C-EBF shall meet the requirements of
3518 Section F3.2, except as modified in this section.

3519 3. Analysis

3520 The analysis requirements for C-EBF shall satisfy the analysis
3521 requirements of Section F3.3.

3522 4. System Requirements

3523 The system requirements for C-EBF shall satisfy the system requirements
3524 of Section F3.4.

3525

3526 5. Members

3527

3528 The member requirements of C-EBF shall satisfy the member
3529 requirements of Section F3.5 except as noted below.

3530 5a. Links

3531 Links shall be unencased structural steel and shall meet the requirement
3532 for *eccentrically braced frame (EBF) links* in Section F3.5a. Beams
3533 containing the link are permitted to act compositely with the floor slab
3534 using steel anchors along all or any portion of the beam if the composite
3535 action is considered when determining the nominal strength of the link.

3536

3537 5b. Columns

3538

3539 Composite columns shall meet the requirements of Section D1.1 for
3540 *highly ductile members*.

3541

3542 Composite columns shall meet the requirements of Section D1.4.

3543

3544 6. Connections

3545 The connection requirements of C-EBF shall satisfy the connection

3546 requirements of Section F3.6 except as noted below.
3547

3548 **6a. Beam-to-Column Connections**

3549 Beam-to-column connections shall conform to one of the following:

3550 (1) Beam-to-column connections shall meet the requirements for fully-
3551 restrained (FR) ordinary moment connections as specified in Sections
3552 D2 and G1.6; or

3553 (2) Beam-to-column connections shall have sufficient rotation capacity to
3554 accommodate the required rotation at a minimum story drift of 2.5% of
3555 the story height.

3556

3557 **H4. COMPOSITE ORDINARY REINFORCED CONCRETE SHEAR**
3558 **WALLS WITH STEEL ELEMENTS (C-ORCW)**

3559 **1. Scope**

3560 Composite ordinary reinforced concrete shear walls with steel elements
3561 (C-ORCW) shall be designed in conformance with this section. This
3562 Section is applicable when reinforced concrete walls are composite with
3563 structural steel elements, either as infill panels, such as reinforced
3564 concrete walls in structural steel frames where unencased or concrete-
3565 encased structural steel sections act as *boundary members* for the walls,
3566 or as structural steel or encased composite *coupling beams* that connect
3567 two or more adjacent reinforced concrete walls.

3568 **2. Basis of Design**

3569 Reinforced concrete walls shall meet the requirements of ACI 318
3570 excluding Chapter 21. The design is based on limited inelastic action
3571 consistent with ACI 318 excluding Chapter 21.

3572 **3. Analysis**

3573 Analysis shall meet the requirements of Chapter C as modified in this
3574 section.

3575 (a) Uncracked effective stiffness values for elastic analysis shall be
3576 assigned according to ACI 318 Chapter 10 for wall piers and
3577 composite coupling beams.

3578 (b) When *concrete-encased shapes* function as boundary members,
3579 the analysis shall be based upon a transformed concrete section
3580 using elastic material properties.

3581 (c) The flexibility of the connection between coupling beams and
3582 wall piers and the effect of shear distortions of the coupling beam

3583 and walls shall be taken into account.

3584 4. System Requirements

3585 In *coupled walls*, coupling beams are permitted to yield over the height of
3586 the structure. The coupling beam–wall connection shall develop the
3587 expected flexural and shear strengths of the coupling beam.

3588 In *coupled walls*, it is permitted to redistribute coupling beam forces
3589 vertically to adjacent floors. The shear in any individual coupling beam
3590 should not be reduced by more than 20% of the elastically determined
3591 value. The sum of the coupling beam shear resistance over the height of
3592 the building shall be greater than or equal to the sum of the elastically
3593 determined values.

3594 5. Members

3595 5a. Boundary Members

3596 Boundary members shall meet the requirements of this Section:

3597 (1) The *required axial strength* of the *boundary member* shall be
3598 determined assuming that the shear forces are carried by the
3599 reinforced concrete wall and the entire gravity and overturning
3600 forces are carried by the boundary members in conjunction with
3601 the shear wall.

3602 (2) When the concrete-encased structural steel boundary member
3603 qualifies as a *composite column* as defined in *Specification*
3604 Chapter I, it shall be designed as a composite column to meet the
3605 requirements of Chapter I of the *Specification*.

3606 (3) Headed studs or welded reinforcement anchors shall be provided
3607 to transfer required shear strengths between the structural steel
3608 boundary members and reinforced concrete walls. Headed studs,
3609 if used, shall meet the requirements of *Specification* Chapter I.
3610 Welded reinforcement anchors, if used, shall meet the
3611 requirements of AWS D1.4.

3612 5b. Steel Coupling Beams

3613 (1) Structural Steel Coupling Beams
3614 Structural steel coupling beams that are used between adjacent reinforced
3615 concrete walls shall meet the requirements of the *Specification* and this
3616 Section. The requirements apply to wide flange steel coupling beams.

3617 (a) Steel coupling beams shall comply with the requirements
3618 of Section D1.1 for *moderately ductile members*.

3619 (b) Coupling beams shall have an embedment length that is
3620 sufficient to develop the nominal shear strength, V_n ,

3621 computed from Equation H4-1.

$$3622 \quad V_n = \frac{2R_y M_p}{g} \leq R_y V_p \quad (\text{H4-1})$$

3623 where

3624 V_n = nominal shear strength, kips (kN)

3625 $M_p = F_y Z$, kip-in. (N-mm)

3626 $V_p = 0.6F_y A_{tw}$, kips (kN)

3627 g = coupling beam clear span, in. (mm)

3628 (c) The embedment length shall be computed from Equation
 3629 H4-2 and H4-2M. The embedment length shall be
 3630 considered to begin inside the first layer of confining
 3631 reinforcement in the wall boundary member.

$$3632 \quad R_y V_n = 1.54 \sqrt{f'_c} \left(\frac{b_w}{b_f} \right)^{0.66} \beta_1 b_f L_e \left[\frac{0.58 - 0.22\beta_1}{0.88 + \frac{g}{2L_e}} \right] \quad (\text{H4-2})$$

$$3633 \quad R_y V_n = 0.004 \sqrt{f'_c} \left(\frac{b_w}{b_f} \right)^{0.66} \beta_1 b_f L_e \left[\frac{0.58 - 0.22\beta_1}{0.88 + \frac{g}{2L_e}} \right] \quad (\text{S.I.}) \quad (\text{H4-2M})$$

3633

3634 where

3635 b_w = thickness of wall pier, in. (mm)

3636 b_f = beam flange width, in. (mm)

3637 f'_c = concrete compressive strength, ksi (MPa)

3638 L_e = embedment length, in. (mm)

3639 β_1 = factor relating depth of equivalent rectangular
 3640 compressive stress block to neutral axis depth, as defined
 3641 in ACI 318.

3642 (d) Vertical wall reinforcement with *nominal axial strength*
 3643 equal to the *nominal shear strength* of the coupling beam
 3644 shall be placed over the embedment length of the beam
 3645 with two-thirds of the steel located over the first half of
 3646 the embedment length. This wall reinforcement shall
 3647 extend a distance of at least one tension development
 3648 length above and below the flanges of the coupling beam.
 3649 It is permitted to use vertical reinforcement placed for

3650 other purposes, such as for vertical boundary members, as
3651 part of the required vertical reinforcement.

3652 (2) Composite Coupling Beams
3653

3654 Encased composite sections serving as *coupling beams* shall meet the
3655 requirements of Section H4.5b(1) as modified in this Section:

3656 (a) Coupling beams shall have an embedment length into the
3657 reinforced concrete wall that is sufficient to develop the
3658 nominal shear strength, $V_{n,comp}$, computed from Equation
3659 H4-3.

$$3660 \quad V_{n,comp} = \frac{2M_{p,exp}}{g} \leq V_{comp} \quad (H4-3)$$

3661 where

3662 $M_{p,exp}$ = expected flexural strength of composite coupling
3663 beam. For concrete-encased or composite beams, $M_{p,exp}$
3664 shall be calculated using the plastic stress distribution or
3665 the strain compatibility method. Appropriate R_y factors
3666 shall be used for different elements of the cross-section
3667 while establishing section force equilibrium and
3668 calculating the flexural strength.

3669 V_{comp} = nominal shear strength of the encased composite
3670 coupling beam as computed by Equation H4-4 and H4-
3671 4M, ksi (kN)

$$3672 \quad V_{comp} = R_y V_n + \left(2\sqrt{f_c'} b_{wc} d_c + \frac{A_s f_{yt} d_c}{s} \right) \quad (H4-4)$$

$$3672 \quad V_{comp} = R_y V_n + \left(0.166\sqrt{f_c'} b_{wc} d_c + \frac{A_s f_{yt} d_c}{s} \right) \quad (\text{S.I.}) \quad (H4-4M)$$

3673 where

3674 A_s = area of transverse reinforcement, in.² (mm²)

3675 b_{wc} = width of concrete encasement, in. (mm)

3676 d_c = effective depth of concrete encasement

3677 f_{yt} = yield strength of transverse reinforcement, ksi (MPa)

3678 s = spacing of transverse reinforcement, in. (mm)

3679 (b) The required embedment length shall be computed from
3680 Equation H4-2 and H4-2M by using $V_{n,comp}$ instead of V_n .

3681 **6. Connections**

3682 There are no additional requirements beyond Section H4.5.

3683

3684 **H5. COMPOSITE SPECIAL REINFORCED CONCRETE SHEAR**
3685 **WALLS WITH STEEL ELEMENTS**3686 **1. Scope**3687 *Composite special reinforced concrete shear walls composite with steel*
3688 *elements (C-SRCW) shall be designed in conformance with this section.*3689 **2. Basis of Design**3690 C-SRCW systems shall meet the requirements of Section H4 and the shear
3691 wall requirements of ACI 318 including Chapter 21, except as modified in
3692 this Section. The design is based on inelastic actions consistent with
3693 Chapter 21 of ACI 318.3694 **3. Analysis**3695 Analysis requirements of Section H4.3 shall be met with the
3696 following exceptions.3697 (a) Cracked effective stiffness values for elastic analysis shall be
3698 assigned according to ACI 318 Chapter 10 practice for wall piers
3699 and composite coupling beams.3700 (b) Effects of shear distortion of the steel coupling beam shall be
3701 taken into account.3702 **4. System Requirements**

3703

3704 System requirements stated in Section H4.4 shall be satisfied with the
3705 following exceptions:3706 (a) In *coupled walls*, coupling beams shall yield over the height of
3707 the structure followed by yielding at the base of the wall piers.3708 (b) In *coupled walls*, the total required compressive axial strength in
3709 a wall pier, computed as the sum of the required strengths
3710 attributed to the walls from the gravity load components of the
3711 lateral load combination plus the sum of the expected beam shear
3712 strengths increased by a factor of 1.1 to reflect the effects of
3713 strain hardening ($1.1R_yV_n$) of all the coupling beams framing into
3714 the walls, shall not exceed the axial design strength of the wall at
3715 the balanced condition, P_b .

3716 **5. Members**3717 **5a. Ductile Elements**

3718 Coupling beams are protected zones, and shall satisfy the requirements of
3719 Section D1.3. Welding on steel coupling beams is permitted for
3720 attachment of stiffeners, as required in Section F3.5a(4).

3721 **5b. Boundary Members**

3722 Unencased structural steel columns shall meet the requirements of
3723 Section H4.5b(1).

3724 In addition to the requirements of Sections H4.3(b) and H4.5a(2), the
3725 requirements in this Section shall apply to walls with reinforced-
3726 concrete-encased structural steel *boundary members*. Concrete-encased
3727 structural steel boundary members that qualify as *composite columns* in
3728 *Specification* Chapter I shall meet the *highly ductile member*
3729 requirements of Section D1.4b(2). Otherwise, such members shall be
3730 designed as composite compression members to meet the requirements of
3731 ACI 318 Section 10.13 including the special seismic requirements for
3732 boundary members in ACI 318 Section 21.9.6. Transverse reinforcement
3733 for confinement of the composite boundary member shall extend a
3734 distance of $2h$ into the wall, where h is the overall depth of the boundary
3735 member in the plane of the wall.

3736 Headed studs or welded reinforcing bar anchors shall be provided as
3737 specified in Section H4.5a(3).

3738 **5c. Steel Coupling Beams**

3739 In addition to the requirements of Section H4.5b, structural steel *coupling*
3740 *beams* shall meet the requirements of Section F3.5a. When required in
3741 Section F3.5a(3), the coupling rotation shall be assumed as 0.08 rad
3742 unless a smaller value is justified by rational analysis of the inelastic
3743 deformations that are expected under the *design earthquake*. *Face*
3744 *bearing plates* shall be provided on both sides of the coupling beams at
3745 the face of the reinforced concrete wall. These stiffeners shall meet the
3746 detailing requirements of Section F3.5a(4).

3747 Steel coupling beams shall comply with the requirements of Section D1.1
3748 for highly ductile members.

3749 The expected shear strength for which the embedment length is
3750 calculated in Equation H4-1 shall be increased by a factor of 1.1 to reflect
3751 the effects of strain hardening ($1.1R_yV_n$).

3752 Vertical wall reinforcement as specified in Section H4.5b(1)(d) shall be
3753 confined by transverse reinforcement that meets the requirements for
3754 *boundary members* of ACI 318 Section 21.9.6.
3755

3756 Embedded steel members shall be provided with two regions of vertical
 3757 transfer reinforcement attached to both the top and bottom flanges of the
 3758 embedded member. The first region shall be located to coincide with the
 3759 location of longitudinal wall reinforcing bars closest to the face of the
 3760 wall. The second shall be placed a distance no less than $d/2$ from the
 3761 termination of the embedment length. All transfer reinforcement bars
 3762 shall be fully developed where they engage the coupling beam flanges. It
 3763 is permitted to use straight, hooked or mechanical anchorage to provide
 3764 development. It is permitted to use mechanical couplers welded to the
 3765 flanges to attach the vertical transfer bars. The area of vertical transfer
 3766 reinforcement required is computed by Equation H5-1:

$$3767 \quad A_{tb} \geq 0.03 f_c' L_e b_f / f_y \quad (H5-1)$$

3769 where

3770 A_{tb} = area of transfer reinforcement required in each of
 3771 the first and second regions attached to each
 3772 of the top and bottom flanges, in.² (mm²)
 3773 b_f = beam flange width, in. (mm)
 3774 f_c' = concrete compressive strength, ksi (MPa)
 3775 f_y = yield strength of transfer reinforcement, ksi
 3776 (MPa)
 3777 L_e = embedment length, in. (mm)

3779 The area of vertical transfer reinforcement shall not exceed that
 3780 computed by Equation H5-2:

$$3781 \quad \Sigma A_{tb} < 0.08 L_e b_w - A_s \quad (H5-2)$$

3782 where

3783 ΣA_{tb} = total area of transfer reinforcement provided both
 3784 the first and second regions attached to either the
 3785 top or bottom flange, in.² (mm²)
 3786 A_s = area of longitudinal wall reinforcement provided
 3787 over the embedment length L_e , in.² (mm²)
 3788 b_w = wall width, in. (mm)

3792 5d. Composite Coupling Beams

3793 *Encased composite sections serving as coupling beams* shall meet the
 3794 requirements of Section H5.5c except the requirements of Section
 3795 F3.5a(3) need not be met, and use Equation H5-3 instead of Equation H4-
 3796 4. For all composite encased coupling beams,

$$3797 \quad V_e = 1.1R_y V_n + 1.56 \left(2\sqrt{f_c} b_{wc} d_c + \frac{A_s f_{yr} d_c}{s} \right) \quad (\text{H5-3})$$

$$V_e = 1.1R_y V_n + 1.56 \left(0.166\sqrt{f_c} b_{wc} d_c + \frac{A_s f_{yr} d_c}{s} \right) \quad (\text{S.I.}) \quad (\text{H5-3M})$$

3798 **6. Connections**

3799 Groove welds at column splices shall be demand critical welds as
3800 described in Section A3.4b.

3801 **H6. COMPOSITE STEEL PLATE SHEAR WALLS**

3802 **1. Scope**

3803 *Composite steel plate shear walls* (C-SPW) shall be designed in
3804 conformance with this section. *Composite steel plate shear walls* consist
3805 of steel plates with reinforced concrete encasement on one or both sides of
3806 the plate, or steel plates on both sides of reinforced concrete infill, and
3807 structural steel or composite *boundary members*.

3808 **2. Basis of Design**

3809 C-SPW are expected to withstand significant inelastic deformations in the
3810 webs. The horizontal boundary elements (HBE) and vertical boundary
3811 elements (VBE) adjacent to the composite webs shall be designed to
3812 remain essentially elastic under the maximum forces that can be
3813 generated by the fully yielded steel webs along with the reinforced
3814 concrete webs after the steel web has fully yielded, except that plastic
3815 hinging at the ends of HBE is permitted.

3816 **3. Analysis**

DRAFT

3817 **3a. Webs**

3818 Steel webs shall be designed to resist the seismic load E determined from
3819 the analysis required by the applicable building code. The analysis shall
3820 account for openings in the web.

3821 **3b. Other Members and Connections**

3822 Columns, beams, and connections in C-SPW shall be designed to resist
3823 seismic forces determined from an analysis that includes the expected
3824 strength of the steel webs in shear, $0.6R_y F_y A_{sp}$, and any reinforced
3825 concrete portions of the wall active at the design story drift. The vertical
3826 boundary elements (VBE) are permitted to yield at the base.

3827 **4. System Requirements**

3828 **4a. Steel Plate Thickness**

3829 Steel plates with thickness less than 3/8 in. (9.5 mm) are not permitted.

3830 **4b. Stiffness of Vertical Boundary Elements**3831
3832 The VBE shall meet the requirements of Section F5.4a.3833 **4c. HBE-to-VBE Connection Moment Ratio**3834 The beam-column moment ratio shall meet the requirements of Section
3835 F5.4b.3836 **4d. Bracing**

3837 The bracing shall meet the requirements of Section F5.4c.

3838 **4e. Openings in Webs**3839 *Boundary members* shall be provided around openings in shear wall webs
3840 as required by analysis.3841 **4f. Detailing Requirements**3842 Steel and composite HBE and VBE shall meet the requirements of
3843 Section D1.1 for *highly ductile members*.3844 **5. Members**3845 **5a. Webs**3846 The *design shear strength*, ϕV_{ns} , for the limit state of shear yielding with
3847 a composite plate conforming to Section H6.5b shall be taken as:

3848

3849
$$V_n = 0.6A_{sp}F_y \quad (\text{H6-1})$$

3850
$$\phi = 0.90 \text{ (LRFD)}$$

3851 where

3852 V_n = nominal shear strength of the steel plate, kips (N)3853 A_{sp} = horizontal area of stiffened steel plate, in.² (mm²)3854 F_y = specified minimum yield stress of the plate, ksi
3855 (MPa)3856 The *design shear strength* of C-SPW with a plate that does not meet the
3857 stiffening requirements in Section H6.5b shall be based upon the strength
3858 of the plate as given in Section F5.5 and meet the requirements of

3859 *Specification* Sections G2 and G3.

3860 **5b. Concrete Stiffening Elements**

3861 The steel plate shall be adequately stiffened by encasement or attachment
3862 to a reinforced concrete panel. Conformance to this requirement shall be
3863 demonstrated with an elastic plate buckling analysis showing that the
3864 composite wall can resist a nominal shear force equal to V_{ns} .

3865 The concrete thickness shall be a minimum of 4 in. (100 mm) on each
3866 side when concrete is provided on both sides of the steel plate and 8 in.
3867 (200 mm) when concrete is provided on one side of the steel plate.
3868 Headed shear stud connectors or other mechanical connectors shall be
3869 provided to prevent local buckling and separation of the plate and
3870 reinforced concrete. Horizontal and vertical reinforcement shall be
3871 provided in the concrete encasement to meet or exceed the requirements
3872 in ACI 318 Section 14.3. The reinforcement ratio in both directions shall
3873 not be less than 0.0025. The maximum spacing between bars shall not
3874 exceed 18 in. (450 mm).

3875 **5c. Boundary Members**

3876 Structural steel and composite *boundary members* shall be designed to
3877 resist the *expected shear strength* of steel plate and any reinforced
3878 concrete portions of the wall active at the design story drift. Composite
3879 and reinforced concrete boundary members shall also meet the
3880 requirements of Section H5.5b. Steel boundary members shall also meet
3881 the requirements of Section F5.

3882 **5d. Detailing Requirements**

3883 HBE and VBE shall meet the requirements of Section D1.1 for *highly*
3884 *ductile members*.

3885 **6. Connections**

3886 **6a. Demand Critical Welds**

3887 Welds that connect the shear wall webs to the boundary elements and
3888 groove welds at column splices shall be *demand critical* welds as
3889 described in Section A3.4b.

3890 **6b. Connections of Steel Plate to Boundary Elements**

3891 The steel plate shall be continuously welded or bolted on all edges to the
3892 structural steel framing and/or steel *boundary members*, or the steel
3893 component of the composite boundary members. Welds and/or slip-
3894 critical high-strength bolts are required to develop the nominal shear
3895 strength of the plate shall be provided.

3896 **6c. HBE-to-VBE Connections**

3897 HBE-to-VBE connections shall meet the requirements of Section F5.

3898 **6d. Connections of Steel Plate to Reinforced Concrete Panel**

3899 The shear connectors between the steel plate and the reinforced concrete
3900 panel shall be designed to prevent its overall buckling. Shear connectors
3901 shall be designed to satisfy the following conditions:

3902 (1) Tension in the connector

3903 The shear connector shall be designed to resist the tension force resulting
3904 from inelastic local buckling of the steel plate.

3905 (2) Shear in the connector

3906 The shear connectors collectively shall be designed to transfer expected
3907 strength in shear of steel plate or reinforced concrete panel, whichever is
3908 smaller.

3909

3910

3911

3912

DRAFT

3913
3914
3915
3916
3917
3918
3919
3920
3921
3922
3923
3924
3925
3926
3927
3928
3929
3930
3931
3932
3933
3934
3935
3936
3937
3938
3939
3940
3941
3942
3943
3944
3945
3946
3947
3948
3949
3950
3951
3952
3953
3954
3955
3956
3957
3958
3959

CHAPTER I

FABRICATION, ERECTION, QUALITY CONTROL AND QUALITY ASSURANCE

This chapter addresses requirements for fabrication, erection, quality control and quality assurance. All requirements of *Specification* Section M shall apply, unless specifically modified by these *Provisions*.

The chapter is organized as follows:

- I1. Shop and Erection Drawings
- I2. Fabrication and Erection
- I3. Quality Control and Quality Assurance

II. SHOP AND ERECTION DRAWINGS

1. Shop Drawings for Steel Construction

Shop drawings and erection drawings shall indicate the work to be performed, and include items required by the *Specification*, the AISC *Code of Standard Practice for Steel Buildings and Bridges*, the *applicable building code*, the requirements of Sections A4.1 and A4.2, and the following, as applicable:

- (1) Locations of pretensioned bolts
- (2) Locations of Class A, or better, faying surfaces
- (3) Gusset plates drawn to scale when they are designed to accommodate inelastic rotation
- (4) Weld access hole dimensions, surface profile and finish requirements
- (5) Locations where weld backing is required to be removed
- (6) Locations where fillet welds are required where backing is permitted to remain
- (7) Locations where fillet welds are required to reinforce groove welds, or where fillet welds are used to improve connection geometry
- (8) Locations where weld tabs are required to be removed
- (9) Nondestructive testing (NDT) where performed by the fabricator

2. Erection Drawings for Steel Construction

Erection drawings shall indicate the work to be performed, and include items required by the *Specification*, the AISC *Code of Standard Practice*

3960 for Steel Buildings and Bridges, the applicable building code, the
3961 requirements of Sections A4.1 and A4.2, and the following, as applicable:

- 3962
- 3963 (1) Locations of pretensioned bolts
- 3964 (2) Locations where weld backing is to be removed.
- 3965 (3) Locations where fillet welds are required where backing is
- 3966 permitted to remain
- 3967 (4) Locations where fillet welds are required to reinforce groove
- 3968 welds, or where fillet welds are used to improve connection
- 3969 geometry.
- 3970 (5) Locations where weld tabs are to be removed.
- 3971 (6) Those joints or groups of joints in which a specific assembly
- 3972 order, welding sequence, welding technique or other special
- 3973 precautions are required.
- 3974

3975 **3. Shop and Erection Drawings for Composite Construction**

3976
3977 Shop drawings and erection drawings for the steel components of
3978 composite steel-concrete construction shall meet the requirements of
3979 Sections I1.1 and I1.2.

3980
3981 For reinforced concrete and composite steel-concrete construction, the
3982 provisions of ACI 315-99 *Details and Detailing of Concrete*
3983 *Reinforcement*, ACI 315-R-94 *Manual of Engineering and Placing*
3984 *Drawings for Reinforced Concrete Structures*, and ACI SP-66 *ACI*
3985 *Detailing Manual*, including modifications required by Chapter 21 of
3986 ACI 318-05, and ACI 352 *Monolithic Joints in Concrete Structures* shall
3987 be satisfied. The shop drawings and erection drawings shall meet the
3988 requirements of Section A4.3.

DRAFT

3990
3991 **I2. FABRICATION AND ERECTION**

3992
3993 **1. Protected Zone**

3994
3995 Where a protected zone is designated by these *Provisions* or ANSI/AISC
3996 358, it shall comply with the following:

- 3997
- 3998 (i) Within the protected zone, discontinuities created by fabrication
- 3999 or erection operations, such as tack welds, erection aids, air-arc
- 4000 gouging and thermal cutting, shall be repaired as required by the
- 4001 *engineer of record*.
- 4002
- 4003 (ii) Welded shear studs and decking attachments that penetrate the
- 4004 beam flange shall not be placed on beam flanges within the
- 4005 protected zone. Arc spot welds as required to secure decking shall
- 4006 be permitted.
- 4007

4008 (iii) Welded, bolted, screwed or shot-in attachments for perimeter
 4009 edge angles, exterior facades, partitions, duct work, piping or
 4010 other construction shall not be placed within the protected zone.
 4011

4012 Exception: Other attachments are permitted where designated or
 4013 approved by the engineer of record. See Section D1.3.
 4014

User Note: AWS D1.8 clause 6.15 contains requirements for weld
 4016 removal and the repair of gouges and notches in the protected zone.
 4017

4018 2. Bolted Joints

4019
 4020 Bolted joints shall satisfy the requirements of Section D2.2.
 4021

4022 3. Welded Joints

4023
 4024 Welding shall be performed in accordance with a welding procedure
 4025 specification (WPS) as required in *Structural Welding Code–Steel* (AWS
 4026 D1.1/D1.1M), hereafter referred to as AWS D1.1/D1.1M, and approved
 4027 by the engineer of record. The WPS variables shall be within the
 4028 parameters established by the filler metal manufacturer.
 4029

4030 In addition to the above requirements, *demand critical welds* shall be
 4031 made in accordance with AWS D1.8 clauses 6.1.2, 6.1.3, 6.2.1 and 6.4.
 4032

4033 Welds shall be made using materials as prescribed in Section A3.4.
 4034

4035 Welding details, welding personnel qualification, fabrication and erection
 4036 shall satisfy the requirements of *Structural Welding Code – Seismic*
 4037 *Supplement* (AWS D1.8), hereafter referred to as AWS D1.8, except as
 4038 follows:
 4039

4040 (1) Weld tab removal shall meet the requirements of AWS D1.8,
 4041 except at the outboard ends of weld tabs for continuity plates,
 4042 where $\frac{1}{4}$ in. (6 mm) of weld tab material and weld metal is
 4043 permitted to remain. See Section I2.5.
 4044

4045 (2) AWS D1.8 clauses relating to fabrication shall apply equally to
 4046 shop fabrication welding and to field erection welding.
 4047

User Note: AWS D1.8 was specifically written to provide additional
 4049 requirements for the welding of seismic force resisting systems, and has
 4050 been coordinated wherever possible with these *Provisions*. AWS D1.8
 4051 requirements related to fabrication and erection are organized as follows,
 4052 including normative (mandatory) annexes:

- 4053 1. General Requirements
- 4054 2. Reference Documents
- 4055 3. Definitions

- 4056 4. Welded Connection Details
 4057 5. Welder Qualification
 4058 6. Fabrication
 4059 Annex A. WPS Heat Input Envelope Testing of Filler Metals for
 4060 Demand Critical Welds
 4061 Annex B. Intermix CVN Testing of Filler Metal Combinations
 4062 (where one of the filler metals is FCAW-S)
 4063 Annex C. Supplemental Welder Qualification for Restricted
 4064 Access Welding
 4065 Annex D. Supplemental Testing for Extended Exposure Limits for
 4066 FCAW Filler Metals
 4067

4068 AWS D1.8 requires the complete removal of all weld tab material,
 4069 leaving only base metal and weld metal at the edge of the joint. This is to
 4070 remove any weld discontinuities at the weld ends, as well as facilitate
 4071 magnetic particle testing (MT) of this area. At continuity plates, these
 4072 *Provisions* permit a limited amount of weld tab material to remain
 4073 because of the reduced strains at continuity plates, and any remaining
 4074 weld discontinuities in this weld end region would likely be of little
 4075 significance. Also, weld tab removal sites at continuity plates are not
 4076 subjected to MT.

4077
 4078 AWS D1.8 Clause 6 is entitled "Fabrication, but the intent of AWS is that
 4079 all provisions of AWS D1.8 apply equally to fabrication and erection
 4080 activities as described in the *Specification* and in these *Provisions*."
 4081

4082 **4. Continuity Plates and Stiffeners**

4083
 4084 Corners of continuity plates and stiffeners placed in the webs of rolled
 4085 shapes shall be detailed in accordance with AWS D1.8 Clause 4.1.

4086 **5. Weld Tabs**

4087
 4088 Weld tabs shall be in accordance with AWS D1.8 Clause 6.10.

4089
 4090 Exception: At the outboard ends of continuity plate to column welds,
 4091 weld tabs and weld metal need not be removed closer than ¼ in. (6 mm)
 4092 from the continuity plate edge.
 4093

4094 **13. QUALITY CONTROL AND QUALITY ASSURANCE**

4095
 4096 Quality control (QC) and quality assurance (QA) shall be provided as
 4097 specified in this Section.
 4098

4099
 4100 When required by the applicable building code or contract documents,
 4101 the engineer of record shall prepare a quality assurance plan (QAP). The
 4102 quality assurance plan shall identify the specific quality control and
 4103 quality assurance tasks to be performed on the project respectively by the

4104 contractor and the QA agency. The contractor and quality assurance
 4105 agency shall perform those tasks as identified in the QAP. The QAP shall
 4106 include the requirements of this section.
 4107

User Note: The quality assurance plan in Section I3 is considered
 4108 adequate and effective for most seismic force resisting systems and
 4109 should be used without modification. The quality assurance plan is
 4110 intended to ensure that the seismic force resisting system is significantly
 4111 free of defects that would greatly reduce the ductility of the system.
 4112 There may be cases (for example, nonredundant major transfer members,
 4113 or where work is performed in a location that is difficult to access) where
 4114 supplemental testing might be advisable. Additionally, where the
 4115 contractor's quality control program has demonstrated the capability to
 4116 perform some tasks this plan has assigned to quality assurance,
 4117 modification of the plan could be considered.
 4118
 4119

4120 **1. Inspection and Nondestructive Testing Personnel**
 4121

4122 Visual welding inspection and nondestructive testing (NDT) shall be
 4123 conducted in accordance with a written practice by personnel qualified in
 4124 accordance with AWS D1.8 Clause 7.2.

4125 Bolting inspection shall be conducted in accordance with a written
 4126 practice by qualified personnel.
 4127

User Note: The recommendations of the ICC Model Program for Special
 4128 Inspection should be considered a minimum requirement to establish the
 4129 qualifications of a bolting inspector.
 4130
 4131

4132 **2. Contractor Documents**
 4133

4134 **2a. Documents to be Submitted for Steel Construction**
 4135

4136 The following documents shall be submitted for review by the *engineer*
 4137 *of record* or designee, prior to fabrication or erection, as applicable:
 4138

- 4139 (1) Shop drawings
- 4140 (2) Erection drawings
- 4141 (3) Welding procedure specifications (WPS)
- 4142 (4) Copies of the manufacturer's typical certificate of conformance
 4143 for all electrodes, fluxes and shielding gasses to be used.
- 4144 (5) For demand critical welds, applicable manufacturer's
 4145 certifications that the filler metal meets the supplemental notch
 4146 toughness requirements, as applicable. Should the filler metal
 4147 manufacturer not supply such supplemental certifications, the
 4148 contractor shall have the necessary testing performed and provide
 4149 the applicable test reports.
- 4150 (6) Manufacturer's product data sheets or catalog data for SMAW,

- 4151 FCAW and GMAW composite (cored) filler metals to be used.
 4152 (7) Bolt installation procedures
 4153 (8) Specific assembly order, welding sequence, welding technique or
 4154 other special precautions for joints or groups of joints where such
 4155 items are designated to be submitted to the engineer of record
 4156

2b. Documents to be Available for Review for Steel Construction

4158
 4159 The following documents shall be available for review by the engineer of
 4160 record or designee prior to fabrication or erection, as applicable, unless
 4161 specified to be submitted:
 4162

- 4163 (1) Material test reports for structural steel, bolts, shear connectors,
 4164 and welding materials
 4165 (2) Inspection procedures
 4166 (3) Nonconformance procedure
 4167 (4) Material control procedure
 4168 (5) Welder performance qualification records (WPQRs), including
 4169 any supplemental testing requirements
 4170 (6) QC Inspector qualifications
 4171

4172 The contractor shall retain the document(s) for at least one year after
 4173 substantial completion of construction.
 4174

2c. Documents to be Submitted for Composite Construction

4175
 4176 The following documents shall be submitted for review by the engineer
 4177 of record or designee, prior to concrete production or placement, as
 4178 applicable:
 4179

- 4180
 4181 (1) Concrete mix design and test reports for the mix design
 4182 (2) Reinforcing steel shop drawings
 4183 (3) Concrete placement sequences, techniques and restriction
 4184

2d. Documents to be Available for Review for Composite Construction

4185
 4186 The following documents shall be available for review by the engineer of
 4187 record or designee prior to fabrication or erection, as applicable, unless
 4188 specified to be submitted:
 4189

- 4190
 4191 (1) Material test reports for reinforcing steel
 4192 (2) Inspection procedures
 4193 (3) Nonconformance procedure
 4194 (4) Material control procedure
 4195 (5) Welder performance qualification records (WPQRs) as required
 4196 by AWS D1.4
 4197 (6) QC Inspector qualifications
 4198

4199 The contractor shall retain the document(s) for at least one year after
4200 substantial completion of construction.

4201

4302 **3. Quality Assurance Agency Documents**

4203

4204 The agency responsible for quality assurance shall submit the following
4205 documents to the authority having jurisdiction, the engineer of record,
4206 and the owner or owner's designee:

4207

4208 (1) QA agency's written practices for the monitoring and control
4209 of the agency's operations. The written practice shall include:

4210 (i) The agency's procedures for the selection and
4211 administration of inspection personnel, describing the
4212 training, experience and examination requirements for
4213 qualification and certification of inspection personnel,
4214 and

4215 (ii) The agency's inspection procedures, including general
4216 inspection, material controls, and visual welding
4217 inspection.

4218 (2) Qualifications of management and QA personnel designated
4219 for the project.

4220 (3) Qualification records for Inspectors and NDT technicians
4221 designated for the project.

4222 (4) NDT procedures and equipment calibration records for NDT to
4223 be performed and equipment to be used for the project.

4224 (5) For composite construction, concrete testing procedures and
4225 equipment

4226 (6) Daily or weekly inspection reports.

4227 (7) Nonconformance reports.

4228

4229 **4. Inspection Points and Frequencies**

4230

4231 Inspection points and frequencies of quality control (QC) and quality
4232 assurance (QA) tasks and documentation for the seismic force resisting
4233 system (SFRS) shall be as provided in the following tables.

4234

4235 The following entries are used in the tables:

4236

4237 **4a. Observe (O)**

4238

4239 The inspector shall observe these functions on a random, daily basis.
4240 Welding operations need not be delayed pending observations. For shop
4241 fabrication, where a task is noted to be performed by both QC and QA,
4242 the frequency of QA inspections is permitted to be reduced following
4243 demonstrated performance by the fabricator in meeting the requirements
4244 of these *Provisions*, as evaluated and approved by the engineer of record
4245 and the authority having jurisdiction.

4246

4247 **4b. Perform (P)**

4248
4249 These inspections shall be performed prior to the final acceptance of the
4250 item. Where a task is noted to be performed by both QC and QA,
4251 coordination of the inspection function between QC and QA is permitted
4252 so that the inspection functions need be performed by only one party.
4253 Where QA is to rely upon inspection functions performed by QC, the
4254 approval of the engineer of record and the authority having jurisdiction is
4255 required.

4256
4257 **4c. Document (D)**

4258
4259 The inspector shall prepare reports indicating that the work has been
4260 performed in accordance with the contract documents. The report need
4261 not provide detailed measurements for joint fit-up, WPS settings,
4262 completed welds, or other individual items listed in the Tables in Section
4263 I3.5a. For shop fabrication, the report shall indicate the piece mark of the
4264 piece inspected. For field work, the report shall indicate the reference
4265 grid lines and floor or elevation inspected. Work not in compliance with
4266 the contract documents and whether the noncompliance has been
4267 satisfactorily repaired shall be noted in the inspection report.

4268
4269 **5. Welding Inspection**

4270
4271 Welding inspection and nondestructive testing shall satisfy the
4272 requirements of this section and AWS D1.8.

4273
4274 **User note:** AWS D1.8 was specifically written to provide additional
4275 requirements for the welding of seismic force resisting systems, and has
4276 been coordinated when possible with these *Provisions*. AWS D1.8
4277 requirements related to inspection and nondestructive testing are
4278 organized as follows, including normative (mandatory) annexes:
4279 1. General Requirements
4280 7. Inspection
4281 Annex E. Supplemental Ultrasonic Technician Testing
4282 Annex F. Supplemental Magnetic Particle Testing Procedures
4283 Annex G. Flaw Sizing by Ultrasonic testing

4284
4285 **5a. Visual Welding Inspection**

4286
4287 Visual inspection of welding shall be the primary method used to confirm
4288 that the procedures, materials and workmanship incorporated in
4289 construction are those that have been specified and approved for the
4290 project. All requirements of AWS D1.1/D1.1M for statically loaded
4291 structures shall apply to the designated welds, except as specifically
4292 modified by AWS D1.8.

4293
4294 Visual welding inspection shall be performed by both quality control and

4295 quality assurance personnel. As a minimum, tasks shall be as listed in
 4296 Tables I3-1, I3-2 and I3-3.

4297
 4298
 4299

TABLE I3-1
Visual Inspection Tasks Prior to Welding

Visual Inspection Tasks Prior to Welding	QC		QA	
	Task	Doc.	Task	Doc.
Material identification (Type/Grade)	O	-	O	-
Welder identification system	O	-	O	-
Fit-up of Groove Welds (including joint geometry) - Joint preparation - Dimensions (alignment, root opening, root face, bevel) - Cleanliness (condition of steel surfaces) - Tacking (tack weld quality and location) - Backing type and fit (if applicable)	P/O**	-	O	-
Configuration and finish of access holes	O	-	O	-
Fit-up of Fillet Welds - Dimensions (alignment, gaps at root) - Cleanliness (condition of steel surfaces) - Tacking (tack weld quality and location)	P/O**	-	O	-
** Following performance of this inspection task for ten welds to be made by a given welder, with the welder demonstrating adequate understanding of requirements and possession of skills and tools to verify these items, the Perform designation of this task shall be reduced to Observe, and the welder shall perform this task. Should the inspector determine that the welder has discontinued adequate performance of this task, the task shall be returned to Perform until such time as the Inspector has re-established adequate assurance that the welder will perform the inspection tasks listed.				

4300
 4301
 4302

TABLE I3-2
Visual Inspection Tasks during Welding

Visual Inspection Tasks During Welding	QC		QA	
	Task	Doc.	Task	Doc.
WPS followed - Settings on welding equipment - Travel speed - Selected welding materials - Shielding gas type/flow rate - Preheat applied - Interpass temperature maintained (min/max.) - Proper position (F, V, H, OH) - Intermix of filler metals avoided unless approved	O	-	O	-
Use of qualified welders	O	-	O	-
Control and handling of welding consumables - Packaging - Exposure control	O	-	O	-
Environmental conditions - Wind speed within limits - Precipitation and temperature	O	-	O	-
Welding techniques	O	-	O	-

- Interpass and final cleaning - Each pass within profile limitations - Each pass meets quality requirements				
No welding over cracked tacks	O	-	O	-

4303

4304

4305

TABLE I3-3
Visual Inspection Tasks after Welding

Visual Inspection Tasks After Welding	QC		QA	
	Task	Doc.	Task	Doc.
Welds cleaned	O	-	O	-
Size length, and location of welds	P	-	P	-
Welds meet visual acceptance criteria - Crack prohibition - Weld/base-metal fusion - Crater cross-section - Weld profiles and size - Undercut - Porosity	P	D	P	D
Placement of reinforcement fillets	P	D	P	D
Backing removed, weld tabs removed and finished, <u>and fillet welds added</u> (if required)	P	D	P	D
Repair activities	P	-	P	D

4306

4307

4308

4309

4310

4311

4312

4313

4314

4315

4316

4317

4318

4319

4320

4321

4322

4323

4324

4325

4326

4327

4328

4329

4330

5b. Nondestructive Testing (NDT) of Welds

Nondestructive testing of welds as described in the following shall be performed by quality assurance personnel:

(1) k-Area NDT

Where welding of doubler plates, continuity plates, or stiffeners has been performed in the k-area, the web shall be tested for cracks using magnetic particle testing (MT). The MT inspection area shall include the k-area base metal within 3 in. (75 mm) of the weld. The MT shall be performed no sooner than 48 hours following completion of the welding.

(2) CJP Groove Weld NDT

Ultrasonic testing (UT) shall be performed on 100% of CJP groove welds in materials $\frac{5}{16}$ in. (8 mm) thick or greater. Ultrasonic testing in materials less than $\frac{5}{16}$ in. (8 mm) thick is not required. Weld discontinuities shall be accepted or rejected on the basis of criteria of AWS D1.1/D1.1M Table 6.2. Magnetic particle testing shall be performed on 25% of all beam-to-column CJP groove welds. The rate of UT and MT is permitted to be reduced in accordance with Section I3.5b(7) and (8), respectively.

- 4331
4332 Exception: For ordinary moment frames, UT and MT of CJP
4333 groove welds are required only for demand critical welds.
4334
- 4335 (3) Base Metal NDT for Lamellar Tearing and Laminations
4336
4337 After joint completion, base metal thicker than 1½ in. (38 mm)
4338 loaded in tension in the through thickness direction in tee and
4339 corner joints, where the connected material is greater than ¾ in.
4340 (19 mm) and contains CJP groove welds, shall be ultrasonically
4341 tested for discontinuities behind and adjacent to the fusion line of
4342 such welds. Any base metal discontinuities found within $t/4$ of the
4343 steel surface shall be accepted or rejected on the basis of criteria
4344 of AWS D1.1/D1.1M Table 6.2, where t is the thickness of the
4345 part subjected to the through-thickness strain.
4346
- 4347 (4) Beam Cope and Access Hole NDT
4348
4349 At welded splices and connections, thermally cut surfaces of
4350 beam copes and access holes shall be tested using magnetic
4351 particle testing or penetrant testing, when the flange thickness
4352 exceeds 1½ in. (38 mm) for rolled shapes, or when the web
4353 thickness exceeds 1½ in. (38 mm) for built-up shapes.
4354
- 4355 (5) Reduced Beam Section Repair NDT
4356
4357 Magnetic particle testing shall be performed on any weld and
4358 adjacent area of the reduced beam section (RBS) cut surface that
4359 has been repaired by welding, or on the base metal of the RBS cut
4360 surface if a sharp notch has been removed by grinding.
4361
- 4362 (6) Weld Tab Removal Sites
4363
4364 At the end of welds where weld tabs have been removed,
4365 magnetic particle testing shall be performed on the same beam-to-
4366 column joints receiving UT as required under Section I3.5b(2).
4367 The rate of MT is permitted to be reduced in accordance with
4368 Section I3.5b(8). MT of continuity plate weld tabs removal sites
4369 is not required.
4370
- 4371 (7) Reduction of Percentage of Ultrasonic Testing
4372
4373 Except for demand critical welds, the amount of ultrasonic testing
4374 is permitted to be reduced if approved by the engineer of record
4375 and the authority having jurisdiction. The nondestructive testing
4376 rate for an individual welder or welding operator is permitted to
4377 be reduced to 25%, provided the reject rate is demonstrated to be
4378 5% or less of the welds tested for the welder or welding operator.

4379 A sampling of at least 40 completed welds for a job shall be made
 4380 for such reduction evaluation. Reject rate is the number of welds
 4381 containing rejectable defects divided by the number of welds
 4382 completed. For evaluating the reject rate of continuous welds over
 4383 3 ft (1 m) in length where the effective throat thickness is 1 in.
 4384 (25 mm) or less, each 12 in. (300 mm) increment or fraction
 4385 thereof shall be considered as one weld. For evaluating the reject
 4386 rate on continuous welds over 3 ft (1 m) in length where the
 4387 effective throat thickness is greater than 1 in. (25 mm), each 6 in.
 4388 (150 mm) of length or fraction thereof shall be considered one
 4389 weld.

4390

4391 (8) Reduction of Percentage of Magnetic Particle Testing

4392

4393 The amount of MT on CJP groove welds is permitted to be
 4394 reduced if approved by the engineer of record and the authority
 4395 having jurisdiction. The MT rate for an individual welder or
 4396 welding operator is permitted to be reduced to 10%, provided the
 4397 reject rate is demonstrated to be 5% or less of the welds tested for
 4398 the welder or welding operator. A sampling of at least 20
 4399 completed welds for a job shall be made for such reduction
 4400 evaluation. Reject rate is the number of welds containing
 4401 rejectable defects divided by the number of welds completed.
 4402 This reduction is prohibited on welds in the k-area, at repair sites,
 4403 backing removal sites, and access holes.

4404

4405 (9) Documentation

4406

4407 All NDT performed shall be documented. For shop fabrication,
 4408 the NDT report shall identify the tested weld by piece mark and
 4409 location in the piece. For field work, the NDT report shall
 4410 identify the tested weld by location in the structure, piece mark,
 4411 and location in the piece.

4412

4413 **6. Inspection of Bolting**

4414

4415 Observation of bolting operations shall be the primary method used to
 4416 confirm that the procedures, materials and workmanship incorporated in
 4417 construction are those that have been specified and approved for the
 4418 project. As a minimum, the tasks shall be as listed in Tables I3-4, I3-5
 4419 and I3-6.

4420

4421

4422

TABLE I3-4
Inspection Tasks Prior To Bolting

Inspection Tasks Prior To Bolting	QC		QA	
	Task	Doc.	Task	Doc.

Inspection Tasks Prior To Bolting	QC		QA	
	Task	Doc.	Task	Doc.
Proper bolts selected for the joint detail	O	-	O	-
Proper bolting procedure selected for joint detail	O	-	O	-
Connecting elements, including the appropriate faying surface condition and hole preparation, if specified, meet applicable requirements	O	-	O	-
Pre-installation verification testing observed for fastener assemblies and methods used	P	D	O	D
Proper storage provided for bolts, nuts, washers and other fastener components	O	-	O	-

4423

4424

4425

TABLE I3-5
Inspection Tasks during Bolting

Inspection Tasks During Bolting	QC		QA	
	Task	Doc.	Task	Doc.
Fastener assemblies placed in all holes and washers (if required) are positioned as required	O	-	O	-
Joint brought to the snug tight condition prior to the pretensioning operation	O	-	O	-
Fastener component not turned by the wrench prevented from rotating	O	-	O	-
Bolts are pretensioned progressing systematically from most rigid point toward free edges	O	-	O	-

4426

4427

4428

TABLE I3-6
Inspection Tasks after Bolting

Inspection Tasks After Bolting	QC		QA	
	Task	Doc.	Task	Doc.
Document accepted and rejected connections	P	D	P	D

4429

4430 7. Other Steel Structure Inspections

4431 Where applicable, the following inspection tasks listed in Table I3-7 shall
4432 be performed:

4433

4434

4435

TABLE I3-7
Other Inspection Tasks

Other Inspection Tasks	QC		QA	
	Task	Doc.	Task	Doc.
RBS requirements, if applicable	P	D	P	D
- contour and finish				
- dimensional tolerances				
Protected zone - no holes and unapproved attachments made by	P	D	P	D

contractor				
------------	--	--	--	--

4436
4437
4438
4439
4440
4441
4442
4443
4444
4445
4446
4447
4448
4449
4450
4451
4452
4453

User Note: The protected zone should be inspected by others following completion of the work of other trades, including those involving curtainwall, mechanical, electrical, plumbing and interior partitions.

8. Inspection of Composite Structures

Where applicable, inspection of structural steel used in composite structures shall comply with the requirements of Chapter I.

Where applicable, inspection of reinforced concrete shall comply with the requirements of ACI 318 and the applicable requirements of Section I3.5a for welded reinforcing steel.

Where applicable to the type of composite construction, the following minimum inspection tasks shall be as listed in Tables I3-8, I3-9 and I3-10.

4454
4455
4456
4457
4458
4459
4460
4461

TABLE I3-8
Inspection of Composite Structures Prior to Concrete Placement

Inspection of Composite Structures Prior to Concrete Placement	QC		QA	
	Task	Doc	Task	Doc.
Material identification of reinforcing steel (Type/Grade)	O	-	O	-
Confirmation of acceptable carbon equivalent for reinforcing steel not complying with requirements of ASTM A706	O	-	O	-
Confirmation of proper reinforcing steel size, spacing and orientation	O	-	O	--
Confirmation that reinforcing steel has not been rebent in the field	O	-	O	-
Confirmation that reinforcing steel has been tied and supported as required	O	-	O	-
Confirmation that required reinforcing steel clearances have been provided	O	-	O	-
Confirmation that composite member has required size	O	-	O	-

4462

4463
4464
4465

TABLE I3-9
Inspection of Composite Structures during Concrete Placement

Inspection of Composite Structures during Concrete Placement	QC		QA	
	Task	Doc	Task	Doc.
Concrete: Material identification (mix design, compressive strength, maximum large aggregate size, maximum slump)	O	D	O	D
Limits on water added at the truck or pump	O	D	O	D
Proper placement techniques to limit segregation	O	-	O	-

4466

4467
4468
4469

TABLE I3-10
Inspection of Composite Structures after Concrete Placement

Inspection of Composite Structures After Concrete Placement	QC		QA	
	Task	Doc	Task	Doc.
Achievement of minimum specified concrete compressive strength at specified age	-	D	-	D

4470

9. Inspection of Piling

4472
4473

Where applicable, the following inspection tasks listed in Table I3-11 shall be performed:

4474

4475

4476

4477

DRAFT
TABLE I3-11
Inspection of Piling

Inspection of Piling	QC		QA	
	Task	Doc	Task	Doc.
Protected zone - no holes and unapproved attachments made by contractor	P	D	P	D

4478

4479

4480

CHAPTER J

4481

QUALIFICATIONS AND PREQUALIFICATION TESTING

4482

PROVISIONS

4483

4484 This chapter addresses requirements for qualifications and prequalification
4485 testing. This chapter is organized as follows:

4486 J1. Prequalification of Beam-to-Column and Link-to-Column
4487 Connections

4488 J2. Cyclic Tests for Qualification of Beam-to-Column and Link-to—
4489 Column Connections

4490 J3. Qualifying Cyclic Tests of Buckling Restrained Braces

4491

4492 **J1. PREQUALIFICATION OF BEAM-TO-COLUMN AND LINK-**
4493 **TO-COLUMN CONNECTIONS**

4494 **1. Scope**

4495 This section contains minimum requirements for prequalification of
4496 beam-to-column moment connections in special moment frames (SMF),
4497 intermediate moment frames (IMF), and link-to-column connections in
4498 eccentrically braced frames (EBF). Prequalified connections are
4499 permitted to be used, within the applicable limits of prequalification,
4500 without the need for further qualifying cyclic tests. When the limits of
4501 prequalification or design requirements for prequalified connections
4502 conflict with the requirements of these Provisions, the limits of
4503 prequalification and design requirements for prequalified connections
4504 shall govern.

4505 **2. General Requirements**

4506 **2a. Basis for Prequalification**

4507 Connections shall be prequalified based on test data satisfying Section
4508 J1.3, supported by analytical studies and design models. The combined
4509 body of evidence for prequalification must be sufficient to assure that the
4510 connection can supply the required *interstory drift angle* for SMF and
4511 IMF systems, or the required *link rotation angle* for EBF, on a consistent
4512 and reliable basis within the specified limits of prequalification. All
4513 applicable limit states for the connection that affect the stiffness, strength
4514 and deformation capacity of the connection and the *seismic force*

4515 *resisting system (SFRS)* must be identified. These include rupture related
 4516 limit states, stability related limit states, and all other limit states pertinent
 4517 for the connection under consideration. The effect of design variables
 4518 listed in Section J1.4 shall be addressed for connection prequalification.

4519 **2b. Authority for Prequalification**

4520 Prequalification of a connection and the associated limits of
 4521 prequalification shall be established by a connection prequalification
 4522 review panel (CPRP) approved by the authority having jurisdiction.

4523 **3. Testing Requirements**

4524 Data used to support connection prequalification shall be based on tests
 4525 conducted in accordance with Section J2. The CPRP shall determine the
 4526 number of tests and the variables considered by the tests for connection
 4527 prequalification. The CPRP shall also provide the same information
 4528 when limits are to be changed for a previously prequalified connection.
 4529 A sufficient number of tests shall be performed on a sufficient number
 4530 of nonidentical specimens to demonstrate that the connection has the
 4531 ability and reliability to undergo the required interstory drift angle for
 4532 SMF and IMF and the required link rotation angle for EBF, where the
 4533 link is adjacent to columns. The limits on member sizes for
 4534 prequalification shall not exceed the limits specified in Section J2.3b.

4535 **4. Prequalification Variables**

4536 In order to be prequalified, the effect of the following variables on
 4537 connection performance shall be considered. Limits on the permissible
 4538 values for each variable shall be established by the CPRP for the
 4539 prequalified connection.

4540 **4a. Beam or Link Parameters**

- 4541 (1) Cross-section shape: wide flange, box, or other.
- 4542 (2) Cross-section fabrication method: rolled shape, welded shape, or
 4543 other.
- 4544 (3) Depth.
- 4545 (4) Weight per foot.
- 4546 (5) Flange thickness.
- 4547 (6) Material specification.
- 4548 (7) Span-to-depth ratio (for SMF or IMF), or link length (for EBF).
- 4549 (8) Width-thickness ratio of cross-section elements.

- 4550 (9) Lateral bracing.
- 4551 (10) Other parameters pertinent to the specific connection under
4552 consideration.
- 4553 **4b. Column Parameters**
- 4554 (1) Cross-section shape: wide flange, box, or other.
- 4555 (2) Cross-section fabrication method: rolled shape, welded shape, or
4556 other.
- 4557 (3) Column orientation with respect to beam or link: beam or link is
4558 connected to column flange, beam or link is connected to column
4559 web, beams or links are connected to both the column flange and
4560 web, or other.
- 4561 (4) Depth.
- 4562 (5) Weight per foot.
- 4563 (6) Flange thickness.
- 4564 (7) Material specification.
- 4565 (8) Width-thickness ratio of cross-section elements.
- 4566 (9) Lateral bracing.
- 4567 (10) Other parameters pertinent to the specific connection under
4568 consideration.
- 4569 **4c. Beam (or Link) – Column Relations**
- 4570 (1) Panel zone strength.
- 4571 (2) Doubler plate attachment details.
- 4572 (3) Column-beam (or link) moment ratio.
- 4573 **4d. Continuity Plates**
- 4574 (1) Identification of conditions under which continuity plates are
4575 required.
- 4576 (2) Thickness, width and depth.
- 4577 (3) Attachment details.
- 4578 **4e. Welds**
- 4579 (1) Location, extent (including returns), type (CJP, PJP, fillet, etc.)
4580 and any reinforcement or contouring required.

- 4581 (2) Filler metal classification strength and notch toughness.
 4582 (3) Details and treatment of weld backing and weld tabs.
 4583 (4) Weld access holes: size, geometry and finish.
 4584 (5) Welding quality control and quality assurance beyond that
 4585 described in Chapter I, including NDT method, inspection
 4586 frequency, acceptance criteria and documentation requirements.

4587 **4f. Bolts**

- 4588 (1) Bolt diameter.
 4589 (2) Bolt grade: ASTM A325, A490, or other.
 4590 (3) Installation requirements: pretensioned, snug-tight, or other.
 4591 (4) Hole type: standard, oversize, short-slot, long-slot, or other.
 4592 (5) Hole fabrication method: drilling, punching, sub-punching and
 4593 reaming, or other.
 4594 (6) Other parameters pertinent to the specific connection under
 4595 consideration.

4596 **4g. Workmanship**

4597 All workmanship parameters that exceed AISC, RCSC and AWS
 4598 requirements, pertinent to specific connection under consideration, as
 4599 follows:

- 4600 (1) Surface roughness of thermal cut or ground edges.
 4601 (2) Cutting tolerances.
 4602 (3) Presence of holes, fasteners or welds for attachments.

4603 **4h. Additional Connection Details**

4604 All variables pertinent to the specific connection under consideration, as
 4605 established by the CPRP.

4606 **5. Design Procedure**

4607 A comprehensive design procedure must be available for a prequalified
 4608 connection. The design procedure must address all applicable limit states
 4609 within the limits of prequalification.

4610 **6. Prequalification Record**

4611 A prequalified connection shall be provided with a written

- 4612 prequalification record with the following information:
- 4613 (1) General description of the prequalified connection and drawings
4614 that clearly identify key features and components of the
4615 connection.
- 4616 (2) Description of the expected behavior of the connection in the
4617 elastic and inelastic ranges of behavior, intended location(s) of
4618 inelastic action, and a description of limit states controlling the
4619 strength and deformation capacity of the connection.
- 4620 (3) Listing of systems for which connection is prequalified: SMF,
4621 IMF or EBF.
- 4622 (4) Listing of limits for all prequalification variables listed in Section
4623 J1.4.
- 4624 (5) Listing of demand critical welds.
- 4625 (6) Definition of the region of the connection that comprises the
4626 protected zone.
- 4627 (7) Detailed description of the design procedure for the connection, as
4628 required in Section J1.5.
- 4629 (8) List of references of test reports, research reports and other
4630 publications that provided the basis for prequalification.
- 4631 (9) Summary of quality control and quality assurance procedures.

4632 **J2. CYCLIC TESTS FOR QUALIFICATION OF BEAM-TO-COLUMN**
4633 **AND LINK-TO-COLUMN CONNECTIONS**

4634 **1. Scope**

4635 This section includes requirements for qualifying cyclic tests of beam-to-
4636 column moment connections in special and intermediate moment frames
4637 and link-to-column connections in *eccentrically braced frames*, when
4638 required in these *Provisions*. The purpose of the testing described in this
4639 section is to provide evidence that a beam-to-column connection or a
4640 link-to-column connection satisfies the requirements for strength and
4641 *interstory drift angle* or *link rotation angle* in these *Provisions*.
4642 Alternative testing requirements are permitted when approved by the
4643 engineer of record and the *authority having jurisdiction*.

4644 This section provides minimum recommendations for simplified test
4645 conditions.

4646

4647 **2. Test Subassemblage Requirements**

4648 The *test subassemblage* shall replicate as closely as is practical the
 4649 conditions that will occur in the prototype during earthquake loading. The
 4650 test subassemblage shall include the following features:

- 4651 (1) The test specimen shall consist of at least a single column with
 4652 beams or links attached to one or both sides of the column.
- 4653 (2) Points of inflection in the test assemblage shall coincide
 4654 approximately with the anticipated points of inflection in the
 4655 Prototype under earthquake loading.
- 4656 (3) Lateral bracing of the test subassemblage is permitted near load
 4657 application or reaction points as needed to provide lateral stability
 4658 of the test subassemblage. Additional lateral bracing of the test
 4659 subassemblage is not permitted, unless it replicates lateral bracing
 4660 to be used in the prototype.

4661 **3. Essential Test Variables**

4662 The test specimen shall replicate as closely as is practical the pertinent
 4663 design, detailing, construction features, and material properties of the
 4664 prototype. The following variables shall be replicated in the test
 4665 specimen.

4666 **3a. Sources of Inelastic Rotation**

4667 The inelastic rotation shall be computed based on an analysis of test
 4668 specimen deformations. Sources of inelastic rotation include yielding of
 4669 members, yielding of connection elements and connectors, and slip
 4670 between members and connection elements. For beam-to-column
 4671 moment connections in special and intermediate moment frames,
 4672 inelastic rotation is computed based upon the assumption that inelastic
 4673 action is concentrated at a single point located at the intersection of the
 4674 centerline of the beam with the centerline of the column. For link-to-
 4675 column connections in eccentrically braced frames, inelastic rotation
 4676 shall be computed based upon the assumption that inelastic action is
 4677 concentrated at a single point located at the intersection of the centerline
 4678 of the link with the face of the column.

4679 *Inelastic rotation* shall be developed in the test specimen by inelastic
 4680 action in the same members and connection elements as anticipated in the
 4681 prototype (in other words, in the beam or link, in the column panel zone,
 4682 in the column outside of the panel zone, or in connection elements)

4683 within the limits described below. The percentage of the total inelastic
 4684 rotation in the test specimen that is developed in each member or
 4685 connection element shall be within 25% of the anticipated percentage of
 4686 the total inelastic rotation in the prototype that is developed in the
 4687 corresponding member or connection element.

4688 **3b. Size of Members**

4689 The size of the beam or link used in the test specimen shall be within the
 4690 following limits:

4691 (1) The depth of the test beam or link shall be no less than
 4692 90% of the depth of the prototype beam or link.

4693 (2) The weight per foot of the test beam or link shall be no
 4694 less than 75% of the weight per foot of the prototype beam
 4695 or link.

4696 The size of the column used in the test specimen shall properly represent
 4697 the inelastic action in the column, as per the requirements in Section
 4698 J2.3a. In addition, the depth of the test column shall be no less than 90%
 4699 of the depth of the prototype column.

4700 Extrapolation beyond the limitations stated in this Section shall be
 4701 permitted subject to qualified peer review and approval by the *authority*
 4702 *having jurisdiction*.

4703 **User Note:** Based upon the above criteria, beam depth and column
 4704 depths up to and including 11% greater than that tested should be
 4705 permitted for the prototype. Weight per foot of the beam or link up to and
 4706 including 33% greater than that tested should be permitted for the
 4707 prototype.

4708 **3c. Connection Details**

4709 The connection details used in the test specimen shall represent the
 4710 prototype connection details as closely as possible. The connection
 4711 elements used in the test specimen shall be a full-scale representation of
 4712 the connection elements used in the prototype, for the member sizes
 4713 being tested.

4714 **3d. Continuity Plates**

4715 The size and connection details of continuity plates used in the test
 4716 specimen shall be proportioned to match the size and connection details
 4717 of continuity plates used in the prototype connection as closely as
 4718 possible.

4719 **3e. Steel Strength**

4720 The following additional requirements shall be satisfied for each member
 4721 or connection element of the test specimen that supplies inelastic rotation
 4722 by yielding:

- 4723 (1) The yield strength shall be determined as specified in
 4724 Section J2.6a. The use of yield stress values that are reported
 4725 on certified material test reports in lieu of physical testing is
 4726 prohibited for purposes of this Section.
- 4727 (2) The yield strength of the beam flange as tested in accordance
 4728 with Section J2.6a shall not be more than 15% below $R_y F_y$
 4729 for the grade of steel to be used for the corresponding
 4730 elements of the prototype.
- 4731 (3) The yield strength of the columns and connection elements
 4732 shall not be more than 15% above or below $R_y F_y$ for the
 4733 grade of steel to be used for the corresponding elements of
 4734 the prototype. $R_y F_y$ shall be determined in accordance with
 4735 Section A3.2.

User Note: Based upon the above criteria, steel of the specified grade with a F_y of up to and including 1.15 times the $R_y F_y$ for the steel tested should be permitted in the prototype. In production, this limit should be checked using the values stated on the steel manufacturer's material test reports.

4741 3f. Welded Joints

4742 Welds on the test specimen shall satisfy the following requirements:

- 4743 (1) Welding shall be performed in conformance with Welding
 4744 Procedure Specifications (WPS) as required in AWS
 4745 D1.1/D1.1M. The WPS essential variables shall meet the
 4746 requirements in AWS D1.1/D1.1M and shall be within the
 4747 parameters established by the filler-metal manufacturer. The
 4748 tensile strength and Charpy V-notch (CVN) toughness of the
 4749 welds used in the *test assembly* shall be determined by tests
 4750 as specified in Section J2.6c, made using the same filler
 4751 metal classification, manufacturer, brand or trade name,
 4752 diameter, and average heat input for the WPS used on the
 4753 *test specimen*. The use of tensile strength and CVN
 4754 toughness values that are reported on the manufacturer's
 4755 typical certificate of conformance in lieu of physical testing
 4756 is prohibited for purposes of this section.
- 4757 (2) The specified minimum tensile strength of the filler metal
 4758 used for the test specimen shall be the same as that to be
 4759 used for the welds on the corresponding prototype. The
 4760 tensile strength of the deposited weld as tested in accordance

4761 with Section J2.6c shall not exceed the tensile strength
 4762 classification of the filler metal specified for the prototype
 4763 by more than 25 ksi (125 MPa).

4764 **User Note:** Based upon the criteria in (2) above, should the
 4765 tested tensile strength of the weld metal exceed 25 ksi (125
 4766 MPa) above the specified minimum tensile strength, the
 4767 prototype weld should be made with a filler metal and WPS
 4768 that will provide a tensile strength no less than 25 ksi below
 4769 the tensile strength measured in the *material test plate*.
 4770 When this is the case, the tensile strength of welds resulting
 4771 from use of the filler metal and the WPS to be used in the
 4772 prototype should be determined by using an all-weld-metal
 4773 tension specimen. The test plate is described in AWS D1.8
 4774 clause A6 and shown in AWS D1.8 Figure A.1.

4775 (3) The specified minimum CVN toughness of the filler metal
 4776 used for the test specimen shall not exceed that to be used
 4777 for the welds on the corresponding prototype. The tested
 4778 CVN toughness of the weld as tested in accordance with
 4779 Section J2.6c shall not exceed the minimum CVN toughness
 4780 specified for the prototype by more than 50%, nor 25 ft-lbs
 4781 (34 kJ), whichever is greater.

4782 **User Note:** Based upon the criteria in (3) above, should the
 4783 tested CVN toughness of the weld metal in the material test
 4784 specimen exceed the specified CVN toughness for the *test*
 4785 *specimen* by 25 ft-lbs (34 kJ) or 50%, whichever is greater,
 4786 the prototype weld should be made with a filler metal and
 4787 WPS that will provide a CVN toughness that is no less than
 4788 25 ft-lbs (34 kJ) or 33% lower, whichever is lower, below
 4789 the CVN toughness measured in the *material test plate*.
 4790 When this is the case, the weld properties resulting from the
 4791 filler metal and WPS to be used in the prototype should be
 4792 determined using five CVN test specimens. The test plate is
 4793 described in AWS D1.8 clause A6 and shown in AWS D1.8
 4794 Figure A.1.

4795 (4) The welding positions used to make the welds on the test
 4796 specimen shall be the same as those to be used for the
 4797 prototype welds.

4798 (5) Details of weld backing, weld tabs, access holes, and similar
 4799 items used for the test specimen welds shall be the same as
 4800 those to be used for the corresponding prototype welds.
 4801 Weld backing and weld tabs shall not be removed from the
 4802 test specimen welds unless the corresponding weld backing
 4803 and weld tabs are removed from the prototype welds.

- 4804 (6) Methods of inspection and nondestructive testing and
 4805 standards of acceptance used for test specimen welds shall
 4806 be the same as those to be used for the prototype welds.

4807 **User Note:** The filler metal used for production of the prototype is
 4808 permitted to be of a different classification, manufacturer, brand or trade
 4809 name, and diameter, provided that Sections J2.3f(3) and J2.3f(4) are
 4810 satisfied. To qualify alternate filler metals, the tests as prescribed in
 4811 Section J2.6c should be conducted.

4812 **3g. Bolted Joints**

4813 The bolted portions of the test specimen shall replicate the bolted
 4814 portions of the prototype connection as closely as possible. Additionally,
 4815 bolted portions of the test specimen shall satisfy the following
 4816 requirements:

- 4817 (1) The bolt grade (for example, ASTM A325, A325M, ASTM
 4818 A490, A490M, ASTM F1852, ASTM F2280) used in the test
 4819 specimen shall be the same as that to be used for the prototype,
 4820 except that heavy hex bolts may be substituted for twist-off-type
 4821 tension control bolts of equal minimum specified tensile strength,
 4822 and vice versa.
- 4823 (2) The type and orientation of bolt holes (standard, oversize, short
 4824 slot, long slot, or other) used in the test specimen shall be the
 4825 same as those to be used for the corresponding bolt holes in the
 4826 prototype.
- 4827 (3) When inelastic rotation is to be developed either by yielding or by
 4828 slip within a bolted portion of the connection, the method used to
 4829 make the bolt holes (drilling, sub-punching and reaming, or other)
 4830 in the test specimen shall be the same as that to be used in the
 4831 corresponding bolt holes in the prototype.
- 4832 (4) Bolts in the test specimen shall have the same installation
 4833 (pretensioned or other) and faying surface preparation (no
 4834 specified slip resistance, Class A or B slip resistance, or other) as
 4835 that to be used for the corresponding bolts in the prototype.

4836 **4. Loading History**

4837 **4a. General Requirements**

4838 The test specimen shall be subjected to cyclic loads according to the
 4839 requirements prescribed in Section J2.4b for beam-to-column moment
 4840 connections in special and intermediate moment frames, and according to
 4841 the requirements prescribed in Section J2.4c for link-to-column
 4842 connections in eccentrically braced frames.

4843 Loading sequences other than those specified in Sections J2.4b and J2.4c
 4844 may be used when they are demonstrated to be of equivalent or greater
 4845 severity.

4846 **4b. Loading Sequence for Beam-to-Column Moment Connections**

4847 Qualifying cyclic tests of beam-to-column moment connections in special
 4848 and intermediate moment frames shall be conducted by controlling the
 4849 interstory drift angle, θ , imposed on the test specimen, as specified
 4850 below:

- 4851 (1) 6 cycles at $\theta = 0.00375$ rad
- 4852 (2) 6 cycles at $\theta = 0.005$ rad
- 4853 (3) 6 cycles at $\theta = 0.0075$ rad
- 4854 (4) 4 cycles at $\theta = 0.01$ rad
- 4855 (5) 2 cycles at $\theta = 0.015$ rad
- 4856 (6) 2 cycles at $\theta = 0.02$ rad
- 4857 (7) 2 cycles at $\theta = 0.03$ rad
- 4858 (8) 2 cycles at $\theta = 0.04$ rad

4859 Continue loading at increments of $\theta = 0.01$ rad, with two cycles of
 4860 loading at each step.

4861 **4c. Loading Sequence for Link-to-Column Connections**

4862 Qualifying cyclic tests of link-to-column moment connections in
 4863 *eccentrically braced frames* shall be conducted by controlling the *total*
 4864 *link rotation angle*, γ_{total} , imposed on the test specimen, as follows:

- 4865 (1) 6 cycles at $\gamma_{total} = 0.00375$ rad
- 4866 (2) 6 cycles at $\gamma_{total} = 0.005$ rad
- 4867 (3) 6 cycles at $\gamma_{total} = 0.0075$ rad
- 4868 (4) 6 cycles at $\gamma_{total} = 0.01$ rad
- 4869 (5) 4 cycles at $\gamma_{total} = 0.015$ rad
- 4870 (6) 4 cycles at $\gamma_{total} = 0.02$ rad
- 4871 (7) 2 cycles at $\gamma_{total} = 0.03$ rad
- 4872 (8) 1 cycle at $\gamma_{total} = 0.04$ rad

4873 (9) 1 cycle at $\gamma_{total} = 0.05$ rad

4874 (10) 1 cycle at $\gamma_{total} = 0.07$ rad

4875 (11) 1 cycle at $\gamma_{total} = 0.09$ rad

4876 Continue loading at increments of $\gamma_{total} = 0.02$ rad, with one cycle of
4877 loading at each step.

4878 **5. Instrumentation**

4879 Sufficient instrumentation shall be provided on the test specimen to
4880 permit measurement or calculation of the quantities listed in Section J2.7.

4881 **6. Testing Requirements for Material Specimens**

4882 **6a. Tension Testing Requirements for Structural Steel Material**
4883 **Specimens**

4884 Tension testing shall be conducted on samples taken from *material test*
4885 *plates* in accordance with Section J2.6b. The material test plates shall be
4886 taken from the steel of the same heat as used in the *test specimen*.
4887 Tension-test results from certified material test reports shall be reported,
4888 but shall not be used in lieu of physical testing for the purposes of this
4889 Section. Tension testing shall be conducted and reported for the
4890 following portions of the test specimen:

4891 (1) Flange(s) and web(s) of beams and columns at standard
4892 locations.

4893 (2) Any element of the connection that supplies inelastic
4894 rotation by yielding.

4895 **6b. Methods of Tension Testing for Structural Steel Material Specimens**

4896 Tension testing shall be conducted in accordance with ASTM A6/A6M,
4897 ASTM A370, and ASTM E8, with the following exceptions:

4898 (1) The yield strength, F_y , that is reported from the test shall
4899 be based upon the yield strength definition in ASTM
4900 A370, using the offset method at 0.002 strain.

4901 (2) The loading rate for the tension test shall replicate, as
4902 closely as practical, the loading rate to be used for the test
4903 specimen.

4904 **6c. Testing Requirements for Weld Metal Material Specimens**

4905 Weld metal testing shall be conducted on samples extracted from the
4906 *material test plate*, made using the same filler metal classification,

4907 manufacturer, brand or trade name and diameter, and using the same
 4908 average heat input as used in the welding of the *test specimen*. The
 4909 tensile strength and CVN toughness of weld material specimens shall be
 4910 determined in accordance with *Standard Methods for Mechanical Testing*
 4911 *of Welds* (ANSI/AWS B4.0). The use of tensile strength and CVN
 4912 toughness values that are reported on the manufacturer's typical
 4913 certificate of conformance in lieu of physical testing shall be prohibited
 4914 for use for purposes of this section.

4915 The same WPS shall be used to make the test specimen and the material
 4916 test plate. The material test plate shall use base metal of the same grade
 4917 and type as was used for the test specimen, although the same heat need
 4918 not be used. If the average heat input used for making the material test
 4919 plate is not within $\pm 20\%$ of that used for the test specimen, a new
 4920 material test plate shall be made and tested.

4921 7. Test Reporting Requirements

4922 For each test specimen, a written test report meeting the requirements of
 4923 the *authority having jurisdiction* and the requirements of this Section shall
 4924 be prepared. The report shall thoroughly document all key features and
 4925 results of the test. The report shall include the following information:

- 4926 (1) A drawing or clear description of the test subassembly,
 4927 including key dimensions, boundary conditions at loading and
 4928 reaction points, and location of lateral braces.
- 4929 (2) A drawing of the connection detail showing member sizes, grades
 4930 of steel, the sizes of all connection elements, welding details
 4931 including filler metal, the size and location of bolt holes, the size
 4932 and grade of bolts, and all other pertinent details of the connection.
- 4933 (3) A listing of all other essential variables for the test specimen, as
 4934 listed in Section J2.3.
- 4935 (4) A listing or plot showing the applied load or displacement history
 4936 of the test specimen.
- 4937 (5) A listing of all welds to be designated *demand critical*.
- 4938 (6) Definition of the region of the member and connection to be
 4939 designated a *protected zone*.
- 4940 (7) A plot of the applied load versus the displacement of the test
 4941 specimen. The displacement reported in this plot shall be
 4942 measured at or near the point of load application. The locations on
 4943 the test specimen where the loads and displacements were
 4944 measured shall be clearly indicated.

- 4945 (8) A plot of beam moment versus *interstory drift angle* for beam-to-
 4946 column moment connections; or a plot of link shear force versus
 4947 link rotation angle for link-to-column connections. For beam-to-
 4948 column connections, the beam moment and the interstory drift
 4949 angle shall be computed with respect to the centerline of the
 4950 column.
- 4951 (9) The interstory drift angle and the total inelastic rotation developed
 4952 by the test specimen. The components of the test specimen
 4953 contributing to the total inelastic rotation due to yielding or slip
 4954 shall be identified. The portion of the total inelastic rotation
 4955 contributed by each component of the test specimen shall be
 4956 reported. The method used to compute inelastic rotations shall be
 4957 clearly shown.
- 4958 (10) A chronological listing of significant test observations, including
 4959 observations of yielding, slip, instability, and rupture of any
 4960 portion of the test specimen as applicable.
- 4961 (11) The controlling failure mode for the test specimen. If the test is
 4962 terminated prior to failure, the reason for terminating the test shall
 4963 be clearly indicated.
- 4964 (12) The results of the material specimen tests specified in Section J2.6.
- 4965 (13) The Welding Procedure Specifications (WPS) and welding
 4966 inspection reports.
- 4967 Additional drawings, data, and discussion of the test specimen or test
 4968 results are permitted to be included in the report.

4969 8. Acceptance Criteria

4970 The test specimen must satisfy the strength and *interstory drift angle* or
 4971 link rotation angle requirements of these *Provisions* for the *special*
 4972 *moment frame*, *intermediate moment frame*, or *eccentrically braced*
 4973 *frame* connection, as applicable. The test specimen must sustain the
 4974 required interstory drift angle or link rotation angle for at least one
 4975 complete loading cycle.

4976 4977 J3. QUALIFYING CYCLIC TESTS OF BUCKLING-RESTRAINED 4978 BRACES

4979 4980 1. Scope

4981 This section includes requirements for qualifying cyclic tests of
 4982 individual buckling-restrained braces and buckling-restrained brace
 4983 subassemblages, when required in these provisions. The purpose of the
 4984

4985 testing of individual braces is to provide evidence that a buckling-
 4986 restrained brace satisfies the requirements for strength and inelastic
 4987 deformation by these provisions; it also permits the determination of
 4988 maximum brace forces for design of adjoining elements. The purpose of
 4989 testing of the brace subassembly is to provide evidence that the brace-
 4990 design can satisfactorily accommodate the deformation and rotational
 4991 demands associated with the design. Further, the subassembly test is
 4992 intended to demonstrate that the hysteretic behavior of the brace in the
 4993 subassembly is consistent with that of the individual brace elements
 4994 tested uniaxially.

4995
 4996 Alternative testing requirements are permitted when approved by the
 4997 engineer of record and the *authority having jurisdiction*.
 4998 This section provides only minimum recommendations for simplified test
 4999 conditions.

5000 2. Subassembly Test Specimen

5001
 5002 The *subassembly test specimen* shall satisfy the following
 5003 requirements:

- 5004 (1) The mechanism for accommodating inelastic rotation in the
 5005 subassembly test specimen brace shall be the same as that of the
 5006 *prototype*. The rotational deformation demands on the
 5007 subassembly test specimen brace shall be equal to or greater
 5008 than those of the prototype.
- 5009 (2) The axial yield strength of the steel core, $P_{y,sc}$, of the brace in the
 5010 subassembly test specimen shall not be less than that of the
 5011 prototype where both strengths are based on the core area, A_{sc} ,
 5012 multiplied by the yield strength as determined from a coupon test.
- 5013 (3) The cross-sectional shape and orientation of the steel core
 5014 projection of the subassembly test specimen brace shall be the
 5015 same as that of the brace in the prototype.
- 5016 (4) The same documented design methodology shall be used for
 5017 design of the subassembly as used for the prototype, to allow
 5018 comparison of the rotational deformation demands on the
 5019 subassembly brace to the prototype. In stability calculations,
 5020 beams, columns and gussets connecting the core shall be
 5021 considered parts of this system.
- 5022 (5) The calculated margins of safety for the prototype connection
 5023 design, steel core projection stability, overall buckling and other
 5024 relevant subassembly test specimen brace construction details,
 5025 excluding the gusset plate, for the prototype, shall equal or exceed
 5026 those of the subassembly test specimen construction.

5027 (6) Lateral bracing of the subassemblage test specimen shall replicate
5028 the lateral bracing in the prototype.

5029 (7) The *brace test specimen* and the prototype shall be manufactured
5030 in accordance with the same quality control and assurance
5031 processes and procedures.
5032

5033 Extrapolation beyond the limitations stated in this section shall be
5034 permitted subject to qualified peer review and approval by the
5035 *authority having jurisdiction*.

5036 3. Brace Test Specimen

5037
5038 The *brace test specimen* shall replicate as closely as is practical the
5039 pertinent design, detailing, construction features, and material properties
5040 of the *prototype*.

5041 3a. Design of Brace Test Specimen

5042 The same documented *design methodology* shall be used for the brace
5043 test specimen and the prototype. The design calculations shall
5044 demonstrate, at a minimum, the following requirements:

5045 (1) The calculated margin of safety for stability against overall
5046 buckling for the prototype shall equal or exceed that of the brace
5047 test specimen.

5048 (2) The calculated margins of safety for the brace test specimen and
5049 the prototype shall account for differences in material properties,
5050 including yield and ultimate stress, ultimate elongation, and
5051 toughness.

5052 3b. Manufacture of Brace Test Specimen

5053 The brace test specimen and the prototype shall be manufactured in
5054 accordance with the same quality control and assurance processes and
5055 procedures.

5056 3c. Similarity of Brace Test Specimen and Prototype

5057 The brace test specimen shall meet the following requirements:

5058 (1) The cross-sectional shape and orientation of the steel core shall be
5059 the same as that of the prototype.

5060 (2) The axial yield strength of the steel core, $P_{y_{sc}}$, of the brace test
5061 specimen shall not be less than 50% nor more than 120% of the
5062 prototype where both strengths are based on the core area, A_{sc} ,
5063 multiplied by the yield strength as determined from a coupon test.

5064 (3) The material for, and method of, separation between the steel core
 5065 and the buckling restraining mechanism in the brace test specimen
 5066 shall be the same as that in the prototype.

5067 Extrapolation beyond the limitations stated in this section shall be
 5068 permitted subject to qualified peer review and approval by the
 5069 *authority having jurisdiction.*

5070 **3d. Connection Details**

5071 The connection details used in the brace test specimen shall represent the
 5072 prototype connection details as closely as practical.

5073 **3e. Materials**

5074 **(1) Steel Core**

5075 The following requirements shall be satisfied for the steel core of
 5076 the brace test specimen:

5077 (a) The specified minimum yield stress of the brace test
 5078 specimen steel core shall be the same as that of the
 5079 prototype.

5080 (b) The measured yield stress of the material of the steel core
 5081 in the brace test specimen shall be at least 90% of that of
 5082 the prototype as determined from coupon tests.

5083 (c) The specified minimum ultimate stress and strain of the
 5084 brace test specimen steel core shall not exceed those of the
 5085 prototype.

5086 **(2) Buckling-Restraining Mechanism**

5087 Materials used in the *buckling-restraining mechanism* of the
 5088 brace test specimen shall be the same as those used in the
 5089 prototype.

5090 **3f. Connections**

5091 The welded, bolted, and pinned joints on the test specimen shall replicate
 5092 those on the prototype as close as practical.

5093 **4. Loading History**

5094 **4a. General Requirements**

5095 The *test specimen* shall be subjected to cyclic loads according to the
 5096 requirements prescribed in Sections J3.4b and J3.4c. Additional
 5097 increments of loading beyond those described in Section J3.4c are

5098 permitted. Each cycle shall include a full tension and full compression
5099 excursion to the prescribed deformation.

5100 **4b. Test Control**

5101 The test shall be conducted by controlling the level of axial or rotational
5102 deformation, Δ_b , imposed on the test specimen. As an alternate, the
5103 maximum rotational deformation may be applied and maintained as the
5104 protocol is followed for axial deformation.

5105 **4c. Loading Sequence**

5106 Loads shall be applied to the test specimen to produce the following
5107 deformations, where the deformation is the steel core axial deformation
5108 for the test specimen and the rotational deformation demand for the
5109 *subassembly test specimen* brace:

- 5110 (1) 2 cycles of loading at the deformation corresponding to $\Delta_b = \Delta_{by}$.
- 5111 (2) 2 cycles of loading at the deformation corresponding to $\Delta_b = 0.50$
5112 Δ_{bm} .
- 5113 (3) 2 cycles of loading at the deformation corresponding to $\Delta_b = 1$
5114 Δ_{bm} .
- 5115 (4) 2 cycles of loading at the deformation corresponding to $\Delta_b = 1.5$
5116 Δ_{bm} .
- 5117 (5) 2 cycles of loading at the deformation corresponding to $\Delta_b = 2.0$
5118 Δ_{bm} .
- 5119 (6) Additional complete cycles of loading at the deformation
5120 corresponding to $\Delta_b = 1.5\Delta_{bm}$ as required for the brace test
5121 specimen to achieve a cumulative inelastic axial deformation of at
5122 least 200 times the yield deformation (not required for the
5123 subassembly test specimen).

5124 The *design story drift* shall not be taken as less than 0.01 times the story
5125 height for the purposes of calculating Δ_{bm} . Other loading sequences are
5126 permitted to be used to qualify the test specimen when they are
5127 demonstrated to be of equal or greater severity in terms of maximum and
5128 cumulative inelastic deformation.

5129 **5. Instrumentation**

5130
5131 Sufficient instrumentation shall be provided on the *test specimen* to
5132 permit measurement or calculation of the quantities listed in Section
5133 J3.7.

5134 **6. Materials Testing Requirements**5135 **6a. Tension Testing Requirements**

5136 Tension testing shall be conducted on samples of steel taken from the
 5137 same heat of steel as that used to manufacture the steel core. Tension test
 5138 results from certified material test reports shall be reported but are
 5139 prohibited in place of material specimen testing for the purposes of this
 5140 Section. Tension-test results shall be based upon testing that is conducted
 5141 in accordance with Section J3.6b.

5142 **6b. Methods of Tension Testing**

5143 Tension testing shall be conducted in accordance with ASTM A6, ASTM
 5144 A370, and ASTM E8, with the following exceptions:

- 5145 (1) The yield stress that is reported from the test shall be based upon
 5146 the yield strength definition in ASTM A370, using the offset
 5147 method of 0.002 strain.
- 5148 (2) The loading rate for the tension test shall replicate, as closely as is
 5149 practical, the loading rate used for the *test specimen*.
- 5150 (3) The coupon shall be machined so that its longitudinal axis is
 5151 parallel to the longitudinal axis of the steel core.

5152 **7. Test Reporting Requirements**

5153 For each *test specimen*, a written test report meeting the requirements of
 5154 this Section shall be prepared. The report shall thoroughly document all
 5155 key features and results of the test. The report shall include the following
 5156 information:
 5157

- 5158 (1) A drawing or clear description of the test specimen, including key
 5159 dimensions, boundary conditions at loading and reaction points,
 5160 and location of lateral bracing, if any.
- 5161 (2) A drawing of the connection details showing member sizes, grades
 5162 of steel, the sizes of all connection elements, welding details
 5163 including filler metal, the size and location of bolt or pin holes, the
 5164 size and grade of connectors, and all other pertinent details of the
 5165 connections.
- 5166 (3) A listing of all other essential variables as listed in Sections J3.2 or
 5167 J3.3, as appropriate.
- 5168 (4) A listing or plot showing the applied load or displacement history.
- 5169 (5) A plot of the applied load versus the deformation, Δ_b . The method
 5170 used to determine the deformations shall be clearly shown. The

5171 locations on the *test specimen* where the loads and deformations
5172 were measured shall be clearly identified.

5173 (6) A chronological listing of significant test observations, including
5174 observations of yielding, slip, instability, transverse displacement
5175 along the test specimen and rupture of any portion of the test
5176 specimen and connections, as applicable.

5177 (7) The results of the material specimen tests specified in Section
5178 J3.6.

5179 (8) The manufacturing quality control and quality assurance plans
5180 used for the fabrication of the test specimen. These shall be
5181 included with the welding procedure specifications and welding
5182 inspection reports.

5183
5184 Additional drawings, data, and discussion of the test specimen or test
5185 results are permitted to be included in the report.

5186 **8. Acceptance Criteria**

5187
5188 At least one subassemblage test that satisfies the requirements of Section
5189 J3.2 shall be performed. At least one brace test that satisfies the
5190 requirements of Section J3.3 shall be performed. Within the required
5191 protocol range all tests shall satisfy the following requirements:

5192 (1) The plot showing the applied load vs. displacement history shall
5193 exhibit stable, repeatable behavior with positive incremental
5194 stiffness.

5195 (2) There shall be no rupture, brace instability or brace end connection
5196 failure.

5197 (i) For brace tests, each cycle to a deformation greater than Δ_{by} the
5198 maximum tension and compression forces shall not be less than
5199 the nominal strength of the core.

5200 (ii) For brace tests, each cycle to a deformation greater than Δ_{by} the
5201 ratio of the maximum compression force to the maximum tension
5202 force shall not exceed 1.3.

5203 Other acceptance criteria may be adopted for the *brace test specimen* or
5204 *subassemblage test specimen* subject to qualified peer review and
5205 approval by the *authority having jurisdiction*.

5206
5207