Approved American National Standard

ANSI

AISI S110-07/S1-09 (2012)

American
Iron and Steel
Institute

AISI STANDARD

Standard for Seismic Design of Cold-Formed Steel Structural Systems– Special Bolted Moment Frames with Supplement No. 1

October 2009 (Reaffirmed 2012)

The material contained herein has been developed by the American Iron and Steel Institute Committee on Specifications for the Design of Cold-Formed Steel Structural Members. The Committee has made a diligent effort to present accurate, reliable, and useful information on seismic design for cold-formed steel structures. The Committee acknowledges and is grateful for the contributions of the numerous researchers, engineers, and others who have contributed to the body of knowledge on the subject. Specific references are included in the *Commentary* on the *Standard*.

With anticipated improvements in understanding of the behavior of cold-formed steel and the continuing development of new technology, this material may eventually become dated. It is anticipated that AISI will publish updates of this material as new information becomes available, but this cannot be guaranteed.

The materials set forth herein are for general purposes only. They are not a substitute for competent professional advice. Application of this information to a specific project should be reviewed by a registered professional engineer. Indeed, in many jurisdictions, such a review is required by law. Anyone making use of the information set forth herein does so at their own risk and assumes any and all liability arising therefrom.

First Publishing – March 2010
Second Publishing – April 2014

Produced by American Iron and Steel Institute

PREFACE

The American Iron and Steel Institute's (AISI) Committee on Specifications for the Design of Cold-Formed Steel Structural Members has developed this first edition of the *Standard for Seismic Design of Cold-Formed Steel Structural Systems – Special Bolted Moment Frames* (hereinafter referred to as this *Standard* in general) in 2007. This *Standard* is intended to address the design and construction of cold-formed steel members and connections used in the seismic force resisting systems in buildings and other structures. In the 2007 edition with *Supplement No. 1*, the *Standard* is focused on the Special Bolted Moment Frame (SBMF) System, which is widely used in industrial work platforms. In addition, many seismic design requirements stipulated in this *Standard* are based on the ANSI/AISC 341-05, *Seismic Provisions for Structural Steel Buildings*, and ANSI/AISC 341s1-05, *Supplement No. 1,* developed by the American Institute of Steel Construction (AISC). The application of this *Standard* should be in conjunction with ANSI/AISI S100-07, *North American Specification for the Design of Cold-Formed Steel Structural Members* (hereinafter referred to as AISI S100).

Supplement No. 1 revisions and additions were made to ensure that the application of the design provisions is within the configurations used in the initial research of special bolted moment frames.

AISI Subcommittee 32, Seismic Design, of the Committee on Specifications, is responsible for the ongoing development of this *Standard*. The AISI Committee on Specifications gives the final approval of this document through an ANSI accredited balloting process. The membership of these committees follows this Preface.

The Committee acknowledges and is grateful to the numerous engineers, researchers, producers and others who have contributed to the body of knowledge on these subjects. AISI further acknowledges the permission of the American Institute of Steel Construction for adopting many provisions from its *Seismic Provisions for Structural Steel Buildings*.

American Iron and Steel Institute
October 2009

AISI Committee on Specifications

Subcommittee 32, Seismic Design

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STANDARD FOR SEISMIC DESIGN OF COLD-FORMED STEEL STRUCTURAL SYSTEMS – SPECIAL BOLTED MOMENT FRAMES WITH SUPPLEMENT NO. 1

A. GENERAL

A1 Limits of Applicability and Terms

A1.1 Scope

This *Standard for Seismic Design of Cold-Formed Steel Structural Systems – Special Bolted Moment Frames*, hereinafter referred to as this *Standard*, is applicable for the design and construction of cold-formed steel members and connections in *seismic force resisting systems* (SFRS) in buildings and *other structures*.

A1.2 Applicability

This *Standard* shall govern when *seismic response modification coefficient*, R, used to determine the seismic design forces, is taken as greater than 3, and the main seismic resisting system is the cold-formed steel – special bolted moment frame (CFS–SBMF) system as specified in this *Standard*.

This *Standard* shall be applied in conjunction with the ANSI/AISI S100, *North American Specification for the Design of Cold-Formed Steel Structural Members* and the *applicable building code*. Where there is no applicable building code, the *loads*, *load combinations*, system limitations and general design requirements shall be in accordance with ASCE/SEI 7.

When the *seismic response modification coefficient*, R, used to determine the seismic design forces, is equal to or less than 3, the cold-formed steel members and connections need only be designed in accordance with AISI S100.

Buildings and structures designed and constructed, and within the scope and limitations of the following documents, need not comply with this *Standard*:

- ANSI/AISC 341, *Seismic Provisions for Structural Steel Buildings*, American Institute of Steel Construction
- RMI, *Specification for the Design, Testing and Utilization of Industrial Steel Storage Racks*, Rack Manufacturers Institute
- AISI S213, *North American Cold-Formed Steel Framing–Lateral Design*, American Iron and Steel Institute

A1.3 Definitions

The terms defined in this section are italicized when they appear for the first time in each section except in the titles. Terms designated with ✦ are common AISC-AISI terms that are coordinated between the two standards developers. Terms defined by other standards developers are indicated in square brackets.

*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 stress design (*allowable strength design*).

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

Amplified Seismic Load. The horizontal component of earthquake load E multiplied by Ω_o, where E and the horizontal component of E are defined in the applicable building code. [ANSI/AISC 341]

Applicable Building Code✦. Building code under which the structure is designed.

Authority Having Jurisdiction. Organization, political subdivision, office or individual charged with the responsibility of administering and enforcing the provisions of this *Standard*. [ANSI/AISC 341]

Available Strength✦. Design strength or *allowable strength* as appropriate.

Bearing✦. Limit state of local compressive yielding due to the action of a member bearing against another member or surface.

Connection✦. Combination of structural elements and *joints* used to transmit forces between two or more members.

Contract Document. Document including, but not limited to, plans and specifications, which defines the responsibilities of the parties involved in bidding, purchasing, designing, supplying, and installing cold-formed steel members and systems.

Design Earthquake. The earthquake represented by the design response spectrum as specified in the applicable building code. [ANSI/AISC 341]

Design Strength✦. *Resistance factor* multiplied by the *nominal strength*, ϕR_n.

Design Story Drift. Amplified story drift (drift under the *design earthquake*, including the effects of inelastic action), determined as specified in the applicable building code except as modified by this *Standard*.

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

Expected Tensile Strength. The probable tensile strength of the material, equal to the *specified minimum tensile strength*, F_u, multiplied by R_t.

Joint✦. Area where two or more ends, surfaces or edges are attached. Categorized by type of fastener or weld used and the method of force transfer.

Load✦. Force or other action that results from the weight of building materials, occupants and their possessions, environmental effects, differential movement, or restrained dimensional changes.

Load Effect✦. Forces, stresses, and deformations produced in a structural component by the applied loads.

LRFD Load Combination✦. Load combination in the applicable building code intended for strength design (Load and Resistance Factor Design).

Modified Expected Yield Stress. The probable yield stress of the material, equal to the *specified minimum yield stress*, F_y, multiplied by $R_{re}R_{cf}R_y$.

Moment Frame✦. Framing system that provides resistance to lateral loads and provides stability to the structural system primarily by shear and flexure of the framing members and their connections.

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 the *load effects*, as determined in accordance with this *Standard*.

Occupancy Category. A category assigned by the applicable building code or ASCE/SEI 7, which is used to determine structural requirements based on occupancy.

Owner. An individual or entity organizing and financing the design and construction of a project.

Owner's Representative. An *owner* or individual designated contractually to act for the owner.

Other Structures. Structures designed and constructed in a manner similar to buildings, with building-like vertical and lateral load-resisting elements.

Professional Engineer. An individual who is registered or licensed to practice his/her respective design profession as defined by the statutory requirements of the state in which the project is to be constructed. [AISI S200]

Quality Control. System of shop and field controls implemented by the fabricator and erector to ensure that contract and company fabrication and erection requirements are met. [ANSI/AISC 341]

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 this *Standard*.

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 unavoidable deviations of the nominal strength from the actual strength and for the manner and consequences of failure.

Seismic Design Category (SDC). Classification assigned to a structure by the applicable building code based on its *occupancy category* and the severity of the design earthquake ground motion at the site. [ASCE/SEI 7]

Seismic Force Resisting System. The assembly of structural elements in the building that resists seismic loads, including struts, collectors, chords, diaphragms and trusses.

Seismic Response Modification Coefficient, R. Factor that reduces seismic load effects to strength level as specified by the applicable building code. [ANSI/AISC 341]

Snug-Tightened Bolt. Bolt in a joint in which tightness is attained by either a few impacts of an impact wrench, or the full effort of a worker with an ordinary spud wrench, that brings the connected plies into firm contact.

Specified Minimum Yield Stress. Lower limit of *yield stress* specified for a material as defined by ASTM.

Specified Minimum Tensile Strength. Lower limit of tensile strength specified for a material as defined by ASTM.

Structural Component✦. Member, connector, connecting element or assemblage.

A2 Seismic Design Requirements

A2.1 General

The *required strength* and other seismic provisions for *seismic design categories* (SDCs), and limitations on height and irregularity shall be determined in accordance with the *applicable* *building code*.

A2.2 Story Drift

The *design story drift* and story drift limits shall be determined as specified in the *applicable building code*, except as modified throughout this *Standard*.

A3 Loads and Load Combinations

The *loads* and *load combinations* shall be determined as specified in accordance with the *applicable building code.* Where *amplified seismic loads* are required by this *Standard*, the horizontal earthquake load E (as defined in the applicable building code) shall be multiplied by the system overstrength factor, Ω_o. Where the applicable building code does not contain design coefficients for CSF–SBMF systems, the provisions of Appendix 1 shall apply.

A4 Nominal Strength

The *nominal strength* of systems, members and *connections* shall be determined in accordance with AISI S100, except as modified throughout this *Standard*.

A5 Referenced Documents

The following documents or portions thereof are referenced in this *Standard* and shall be considered part of the requirements of this *Standard*.

1. American Institute of Steel Construction (AISC), One East Wacker Drive, Suite 700, Chicago, IL 60601-1802:

   ANSI/AISC 341-05, *Seismic Provisions for Structural Steel Buildings*, March 9, 2005

   ANSI/AISC 341s1-05, *Supplement No. 1,* November 16, 2005

   ANSI/AISC 360-05, *Specification for Structural Steel Buildings*, Chicago, IL, March 9, 2005

2. American Iron and Steel Institute (AISI), 1140 Connecticut Avenue, NW, Suite 705, Washington, DC 20036:

   AISI S100-07, *North American Specification for the Design of Cold-Formed Steel Structural Members*, 2007

   AISI S213-07, *Lateral Standard for Cold-Formed Steel Framing–Lateral Design*, 2007

3. American Society of Civil Engineers, 1801 Alexander Bell Drive, Reston, Virginia 20191-4400:

   ASCE/SEI 7-05, *Minimum Design Loads for Buildings and Other Structures,* 2005

4. American Society for Testing and Materials (ASTM), 100 Barr Harbor Drive, West Conshohocken, PA 19428-2959:

   ASTM A36/A36M-05, Standard Specification for Carbon Structural Steel

   ASTM A242/A242M-04e1, Standard Specification for High-Strength Low-Alloy Structural Steel

   ASTM A283/A283M-03, Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates

ASTM A500-03a, Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes

ASTM A529/A529M-05, Standard Specification for High-Strength Carbon-Manganese Steel of Structural Quality

ASTM A572/A572M-06, Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel

ASTM A588/A588M-05, Standard Specification for High-Strength Low-Alloy Structural Steel with 50 ksi [345 MPa] Minimum Yield Point to 4-in. [100 mm] Thick

ASTM A606-04, Standard Specification for Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance

ASTM A653/A653M-06a, Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process

ASTM A792/A792M-05, Standard Specification for Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process

ASTM A847/A847M-05, Standard Specification for Cold-Formed Welded and Seamless High-Strength, Low-Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance

ASTM A875/A875M-05, Standard Specification for Steel Sheet, Zinc-5% Aluminum Alloy-Coated by the Hot-Dip Process

ASTM A1003/A1003M-05, Standard Specification for Steel Sheet, Carbon, Metallic- and Nonmetallic-Coated for Cold-Formed Framing Members

ASTM A1008/A1008M-05b, Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability, Solution Hardened and Bake Hardenable

ASTM A1011/A1011M-05a, Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability

5. American Welding Society (AWS), 550 N.W. LeJeune Road, Miami, Florida 33135:

AWS D1.1/D1.1M-2006, *Structural Welding Code-Steel*

AWS D1.3-98, *Structural Welding Code-Sheet Steel*

6. Rack Manufacturers Institute (RMI), 8720 Red Oak Blvd., Suite 201, Charlotte, NC 28217:

RMI, *Specification for the Design, Testing and Utilization of Industrial Steel Storage Racks*, Rack Manufacturers Institute, 2004

B. MATERIALS

B1 Material Specifications

The use of steels intended for structural applications in this *Standard* shall be as defined by the following specifications of the American Society for Testing and Materials, subject to the additional limitations specified in Section D:

ASTM A36/A36M, Standard Specification for Carbon Structural Steel

ASTM A242/A242M, Standard Specification for High-Strength Low-Alloy Structural Steel

ASTM A283/A283M, Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates

ASTM A500 (Grade B or C), Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes

ASTM A529/A529M, Standard Specification for High-Strength Carbon-Manganese Steel of Structural Quality

ASTM A572/A572M (Grade 42 (290), 50 (345), or 55 (380)), Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel

ASTM A588/A588M, Standard Specification for High-Strength Low-Alloy Structural Steel with 50 ksi [345 MPa] Minimum Yield Point to 4-in. [100 mm] Thick

ASTM A606, Standard Specification for Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance

ASTM A653/A653M (SS Grades 33 (230), 37 (255), 40 (275), and 50 (340) Class 1 and Class 3; HSLAS Types A and B, Grades 40 (275), 50 (340), 60 (410)), Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process

ASTM A792/A792M (Grades 33 (230), 37 (255), 40 (275), and 50 Class 1 (340 Class 1)), Standard Specification for Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process

ASTM A847, Standard Specification for Cold-Formed Welded and Seamless High-Strength, Low-Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance

ASTM A875/A875M (SS Grades 33 (230), 37 (255), 40 (275), and 50 (340) Class 1 and Class 3; HSLAS Types A and B, Grades 50 (340), 60 (410)), Standard Specification for Steel Sheet, Zinc-5% Aluminum Alloy-Coated by the Hot-Dip Process

ASTM A1003/A1003M (Grades ST33H, ST37H, ST40H, ST50H), Standard Specification for Steel Sheet, Carbon, Metallic- and Nonmetallic-Coated for Cold-Formed Framing Members

ASTM A1008/A1008M (SS Grades 25 (170), 30 (205), 33 (230) Types 1 and 2, and 40 (275) Types 1 and 2; HSLAS Classes 1 and 2, Grades 45 (310), 50 (340), 55 (380), 60 (410), and 65 (450)); HSLAS-F Grades 50 (340), 60 (410)), Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, Solution Hardened, and Bake Hardenable

ASTM A1011/A1011M (SS Grades 30 (205), 33 (230), 36 (250) Types 1 and 2, 40 (275), 45 (310), 50 (340), and 55 (380); HSLAS Classes 1 and 2, Grades 45 (310), 50 (340), 55 (380), 60 (410), and 65 (450)); HSLAS-F Grades 50 (340), and 60 (410)), Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability

B1.1 Material Properties for Determining Required Strength

Where required in this *Standard*, the *required strength* of a connection or member shall be determined from the modified *expected yield stress*, $R_{re}R_{cf}R_yF_y$ and the *expected tensile strength*, R_tF_u, of the connected member, where F_y is the *specified minimum yield stress* of the grade of steel to be used, F_u is the *specified minimum tensile strength* of the grade of steel to be used, and R_y and R_t are factors given in Table B1.1, unless otherwise modified in Chapter D.

The factor to account for the increase in yield stress due to cold work of forming averaged over the cross section, R_{cf}, shall be taken as 1.10.

Alternately, R_{cf} shall be permitted to be determined in accordance with Section A7.2 of AISI S100, except that the calculated R_{cf} shall be taken greater than or equal to 1.1.

The factor considering the inelastic reserve capacity for a compact section in bending, R_{re}, shall be determined as follows:

For $\lambda < 0.673$,

$$R_{re} = M_{no}/M_y \qquad (Eq.\ B1.1\text{-}1)$$

For $\lambda \geq 0.673$ and for other than bending members

$$R_{re} = 1$$

where

λ = Slenderness of compression flange of member considered, as defined in accordance with AISI S100

M_{no} = Nominal strength determined in accordance with AISI S100, Section C3.1.1(b)

M_y = S_eF_y, nominal flexural yield strength

where

S_e = Effective section modulus at yield stress, F_y

F_y = *Specified minimum yield stress*

Where both the *required strength* and the *available strength* calculations are made for the same member or connecting element, the modified *expected yield stress*, $R_{re}R_{cf}R_yF_y$, of the connected member, and the *expected tensile strength*, R_tF_u, of the connected member, shall be permitted to be used for determination of the *available strength*.

Values of R_y and R_t, other than those listed in Table B1.1, shall be permitted to be used, if the values are determined by testing specimens representative of the product thickness and source, and such tests are conducted in accordance with the ASTM requirements for the specified grade of steel in Section B1.

Table B1.1
R_y and R_t Values for Various Product Types

Steel	R_y	R_t
Plates and bars:		
A36/A36M, A283/A283M	1.3	1.2
A242/A242M, A529/A529M, A572/A572M, A588/A588M	1.1	1.2
Hollow Structural Sections:		
A500 and A847	1.4	1.3
Sheet and strip (A606, A653/A653M, A792/A792M, A875, A1003/A1003M, A1008/A1008M, A1011/A1011M):		
F_y< 37 ksi (255 MPa)	1.5	1.2
37ksi (255MPa) ≤ F_y< 40 ksi (275 MPa)	1.4	1.1
40ksi (275MPa) ≤ F_y<50 ksi (340 MPa)	1.3	1.1
F_y ≥ 50 ksi(340 MPa)	1.1	1.1

Note:

R_y = Ratio of *expected yield stress* to *specified minimum yield stress.*

R_t = Ratio of *expected tensile strength* to *specified minimum tensile strength.*

F_y = *Specified minimum yield stress*

C. CONNECTIONS, JOINTS, AND FASTENERS

Connections, *joints* and fasteners that are part of the *seismic force resisting system* shall meet the requirements of AISI S100, except as modified in Chapter C or Chapter D of this *Standard*.

Connections for members that are a part of the seismic force resisting system shall be configured such that a ductile limit-state in the member or at the joint controls the design.

C1 Bolted Joints

Bolts shall be high-strength bolts, and bolted *joints* shall not be designed to share load in combination with welds.

The bearing strength of bolted joints shall be provided using standard holes or short-slotted holes perpendicular to the line of force, unless an alternative hole-type is approved by a *professional engineer*.

C2 Welded Joints

Welded *joints* shall be permitted to join members that are a part of the *seismic force resisting system*, in accordance with AISI S100.

C3 Other Joints and Connections

Alternative *joints* and *connections* shall be permitted if the *professional engineer* demonstrates performance equivalent to the permitted joints and connections of this *Standard*.

D. SYSTEMS

D1 Cold-Formed Steel Special Bolted Moment Frames (CFS–SBMF)

Cold-formed steel–special bolted moment frames (CFS–SBMF) systems shall withstand inelastic deformations through friction and bearing at their bolted *connections*. Beams, columns, and connections shall satisfy the requirements in this section. CFS–SBMF systems shall be limited to one-story structures, no greater than 35 feet in height, without column splices. The CFS–SBMF shall engage all columns supporting the roof or floor above. A single size beam and single size column with the same bolted moment connection detail shall be used for each frame. The frame shall be supported on a level floor or foundation.

D1.1 Beam-to-Column Connections

D1.1.1 Connection Limitations

Beam-to-column *connections* in CFS–SBMF systems shall be bolted connections with 1-in. (25 mm) diameter *snug-tight high-strength bolts*. The bolt spacing and edge distance shall be in accordance with the limits of AISI S100, Section E3. The 8-bolt configuration shown in Table D1-1 shall be used. The faying surfaces of the beam and column in the bolted moment connection region shall be free of lubricants or debris.

D1.1.2 Bolt Bearing Plates

The use of bolt bearing plates on beam webs in CFS–SBMF systems shall be permitted to increase the bearing strength of the bolt. Bolt bearing plates shall be welded to the beam web. The edge distance of bolts shall be in accordance with the limits of AISI S100, Section E3.

D1.2 Beams and Columns

D1.2.1 Beam Limitations

In addition to the requirements of Section D1.2.3, beams in CFS–SBMF systems shall be ASTM A653 Grade 55 galvanized cold-formed steel C-section members with lips, designed in accordance with Chapter C of AISI S100. The beams shall have a minimum design thickness of 0.105 in. (2.67 mm). The beam depth shall not be less than 12 in. (305 mm) or greater than 20 in. (508 mm). The flat depth-to-thickness ratio of the web shall not exceed $6.18\sqrt{E/F_y}$. When single C-section beams are used, torsional effects shall be accounted for in the design.

D1.2.2 Column Limitations

In addition to the requirements of Section D1.2.3, columns in CFS–SBMF systems shall be ASTM A500 Grade B cold-formed steel hollow structural section (HSS) members painted with a standard industrial finished surface, and designed in accordance with Chapter C of AISI S100. The column depth shall not be less than 8 in. (203 mm) or greater than 12 in. (305 mm). The flat depth-to-thickness ratio shall not exceed $1.40\sqrt{E/F_y}$.

D1.2.3 Required Strength

D1.2.3.1 Beams and Columns

The *required strength* of beams and columns in CFS–SBMF systems shall be determined from the *expected moment* developed at the bolted connection. The *expected moment*, M_e, shall be determined as follows:

$$M_e = h(V_S + R_t V_B) \quad (Eq.\ D1.2.3.1\text{-}1)$$

where

h = Story height

R_t = Ratio of *expected tensile strength* to *specified minimum tensile strength*

V_S = Column shear corresponding to the slip strength of the bolt group

V_B = Column shear corresponding to the bearing strength of the bolt group

(1) *Slip Component of Column Shear, V_S*

The value of V_S shall be determined by Eq. D1.2.3.1-2.

$$V_S = C_S kNT/h \quad (Eq.\ D1.2.3.1\text{-}2)$$

where

C_S = Value from Table D1-1

k = Slip coefficient

= 0.33

N = 1 for single-channel beams

= 2 for double-channel beams

T = 10 kips (44.5kN) for 1-in. (25.4 mm) diameter bolts, unless the use of a higher value is approved by the *authority having jurisdiction*.

(2) *Bearing Component of Column Shear, V_B*

The value of V_B shall be determined as follows:

$$\left(\frac{V_B}{V_{B,max}}\right)^2 + \left(1 - \frac{\Delta_B}{\Delta_{B,max}}\right)^{1.43} = 1 \quad (Eq.\ D1.2.3.1\text{-}3)$$

where

V_B = Bearing column shear of a bolt group

$V_{B,max}$ = Column shear producing the maximum bearing strength of a bolt group

$= C_B N R_0 / h$ (*Eq*. D1.2.3.1-4)

Δ = Design story drift

Δ_B = Component of design story drift causing bearing deformation in a bolt group

$= \Delta - \Delta_S - \dfrac{nM_e}{hK} \geq 0$ (*Eq*. D1.2.3.1-5)

$\Delta_{B,max}$ = Component of design story drift corresponding to the deformation of the bolt group at maximum bearing strength

$= C_{B,0} C_{DB} h$ (*Eq*. D1.2.3.1-6)

Δ_S = Component of design story drift corresponding to bolt slip deformation

$= C_{DS} h_{os} h$ (*Eq*. D1.2.3.1-7)

C_B, C_{DS}, and $C_{B,0}$ = values from Table D1-1

C_{DB} = Value from Table D1-2

d = Bolt diameter

F_u = Tensile strength of connected component

h_{os} = Hole oversize

K = Structural lateral stiffness

M_e = Expected moment at a bolt group

n = Number of columns in a frame line

R_0 = Governing value of dtF_u of connected components

t = Thickness of connected component

Alternate methods of calculating V_S and V_B shall be permitted if such methods are acceptable to the authority having jurisdiction.

Table D1-1
Values of Coefficients C_S, C_{DS}, C_B, and $C_{B,0}$

Bolt spacing, in.			C_S (ft)	C_{DS} (1/ft)	C_B (ft)	$C_{B,0}$ (in./ft)
a	b	c				
2½	3	4¼	2.37	5.22	4.20	0.887
3	6		3.34	3.61	5.88	0.625
3	10		4.53	2.55	7.80	0.475
2½	3	6¼	2.84	4.66	5.10	0.792
3	6		3.69	3.44	6.56	0.587
3	10		4.80	2.58	8.50	0.455

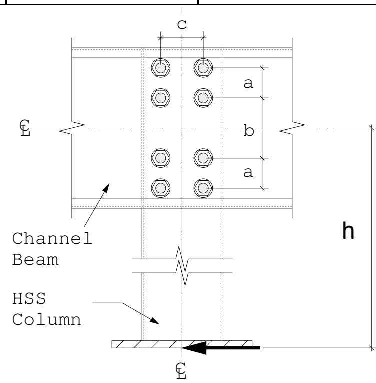

Table D1-2
Bearing Deformation Adjustment Factor C_{DB}

Relative Bearing Strength, R_{BS}	0.0	0.4	0.5	0.6	0.7	0.8	0.9	1.0
C_{DB}	1.00	1.10	1.16	1.23	1.33	1.46	1.66	2.00

where

Relative Bearing Strength (R_{BS}) = $(tF_u)_{(weaker)} / (tF_u)_{(stronger)}$, where weaker components correspond to that with a smaller tF_u value.

t = Thickness of beam or column component

F_u = Tensile strength of beam or column

D1.2.3.2 Bolt Bearing Plates

Bolt bearing plates shall be welded to the beam web and be designed for the following *required strength*, M_{bp}:

$$M_{bp} = \frac{M_e}{N}\left(\frac{t_p}{t_w + t_p}\right) \qquad (Eq.\ D1.2.3.2\text{-}1)$$

where

M_e = Expected moment at a bolt group

t_p = Thickness of bolt bearing plate

t_w = Thickness of beam web

D1.3 Design Story Drift

Where the *applicable building code* does not contain design coefficients for CSF-SBMF systems, the provisions of Appendix 1 shall apply.

For structures having a period shorter than T_S, as defined in the *applicable building code*, alternate methods of computing Δ shall be permitted, provided such alternate methods are acceptable to the *authority having jurisdiction*.

D1.4 P-Δ Effects

P-Δ effects shall be considered in accordance with the requirements of the *applicable building code*.

E. QUALITY ASSURANCE AND QUALITY CONTROL

The fabricator shall provide *quality control* procedures to the extent that the fabricator deems necessary to ensure that the work is performed in accordance with this *Standard*. In addition to the fabricator's *quality control* procedures, material and workmanship at all times shall be permitted to be subject to inspection by qualified inspectors representing the owner. If such inspection by the *owner's representatives* will be required, it shall be so stated in the *contract documents*.

E1 Cooperation

As far as possible, the inspection by *owner's representatives* shall be made at the fabricator's plant. The fabricator shall cooperate with the inspector, permitting access for inspection to all places where work is being done. The *owner's* inspector shall schedule this work for minimum interruption to the work of the fabricator.

E2 Rejections

Material or workmanship not in conformance with the provisions of this *Standard* shall be permitted to be rejected at any time during the progress of the work.

The fabricator shall receive copies of all reports furnished to the *owner* by the inspection agency.

E3 Inspection of Welding

The inspection of welding shall be in accordance with the provisions of AWS D1.1 and AWS D1.3, as applicable. When visual inspection is required to be performed by AWS certified welding inspectors, it shall be specified in the *contract documents*. When nondestructive testing is required, the process, extent, and standards of acceptance shall be defined in the contract documents.

E4 Inspection of Bolted Connections

Connections shall be inspected to verify that the fastener components are as specified and that the joint plies have been drawn into firm contact. A representative sample of bolts shall be evaluated using an ordinary spud wrench, to ensure that the bolts in the connections have been tightened to a level equivalent to that of the full effort of a worker with such wrench.

E5 Identification of Steel

The fabricator shall be able to demonstrate by a written procedure and by actual practice a method of material identification, visible at least through the "fit-up" operation, for the main structural elements of each shipping piece.

APPENDIX 1: SEISMIC DESIGN COEFFICIENTS

This appendix contains design coefficients, system limitations and design parameters for *seismic force resisting systems* (*SFRS*) that are included in this *Standard*, but are not yet defined in the *applicable building code*. The values presented in Table 1.2-1 in this appendix shall only be used when neither the *applicable building code* nor ASCE/SEI 7 contain such values.

1.1 Symbols

The following symbols are used in this appendix.

C_d Deflection amplification factor

Ω_o System overstrength factor

R Response modification coefficient

1.2 Design Coefficients and Factors for Basic Seismic Force Resisting Systems

TABLE 1.2-1
Design Coefficients and Factors for Basic Seismic Force Resisting Systems

Basic Seismic Force Resisting System	Response Modification Coefficient R	System Overstrength Factor Ω_o	Deflection Amplification Factor C_d	Height Limit (ft)			
				Seismic Design Category			
				B & C	D	E	F
Building Frame Systems							
Cold-formed steel–special bolted moment frames[c]	3.5	3.0 [a]	3.5 [b]	35	35	35	35

[a] The seismic load effect with overstrength, E_{mh}, is permitted to be based on the expected strength determined in accordance with Section D1.2.3.

[b] Also see Section D1.3.

[c] Cold-formed steel–special bolted moment frame is limited to one-story in height.

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AISI STANDARD

Commentary on Standard for Seismic Design of Cold-Formed Steel Structural Systems– Special Bolted Moment Frames with Supplement No. 1

October 2009 (Reaffirmed 2012)

The material contained herein has been developed by the American Iron and Steel Institute Committee on Specifications for the Design of Cold-Formed Steel Structural Members. The Committee has made a diligent effort to present accurate, reliable, and useful information on seismic design for cold-formed steel structures. The Committee acknowledges and is grateful for the contributions of the numerous researchers, engineers, and others who have contributed to the body of knowledge on the subject. Specific references are included in the *Commentary* on the *Standard*.

With anticipated improvements in understanding of the behavior of cold-formed steel and the continuing development of new technology, this material may eventually become dated. It is anticipated that AISI will publish updates of this material as new information becomes available, but this cannot be guaranteed.

The materials set forth herein are for general purposes only. They are not a substitute for competent professional advice. Application of this information to a specific project should be reviewed by a registered professional engineer. Indeed, in many jurisdictions, such a review is required by law. Anyone making use of the information set forth herein does so at their own risk and assumes any and all liability arising therefrom.

First Publishing – March 2010
Second Publishing – April 2014

Produced by American Iron and Steel Institute

PREFACE

The American Iron and Steel Institute's (AISI) Committee on Specifications for the Design of Cold-Formed Steel Structural Members has developed a *Standard for Seismic Design of Cold-Formed Steel Structural Systems* (hereinafter referred to as this *Standard* in general). This document provides a *Commentary* on the 2007 edition of this *Standard*, which should be used in combination with AISI S100, *North American Specification for the Design of Cold-Formed Steel Structural Members,* issued by the Institute.

The *Commentary* presents a record of the reasoning behind, justification for the various provisions of this *Standard*, and a brief discussion on the characteristics of cold-formed steel structural members and connections used in seismic force resisting systems for buildings and other structures. The readers who wish to have additional information should refer to the cited publications listed in References.

Revisions and additions to the *Commentary* were made to coordinate with *Supplement No. 1* of the *Standard*.

The assistance and close cooperation of the AISI Committee on Specifications under the Chairmanship of Mr. Roger L Brockenbrough and Vice Chairmanship of Mr. Richard Haws, and the Subcommittee on Seismic Design under the former Chairmanship of Dr. Reidar Bjorhovde are gratefully acknowledged. The Institute also acknowledges and is grateful for the contributions of numerous engineers, researchers, producers, and others who have contributed to the development of this new *Standard* with the *Commentary*.

American Iron and Steel Institute
October 2009

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COMMENTARY TO STANDARD FOR SEISMIC DESIGN OF COLD-FORMED STEEL STRUCTURAL SYSTEMS – SPECIAL BOLTED MOMENT FRAMES WITH SUPPLEMENT NO. 1, OCTOBER 2009

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COMMENTARY ON STANDARD FOR SEISMIC DESIGN OF COLD-FORMED STEEL STRUCTURAL SYSTEMS–SPECIAL BOLTED MOMENT FRAMES WITH SUPPLEMENT NO. 1

INTRODUCTION

The cold-formed steel design standards and specifications for the applications of cold-formed steel in high seismic force regions have been developed for rack structures (RMI, 2004) and cold-formed steel framing (AISI, 2007b). However, for many other cold-formed steel structures, a seismic design standard was needed.

In 2003, a seismic design subcommittee of the Committee on Specifications was formed under AISI to develop seismic design provisions to be used for the design and construction of cold-formed steel members and other structures that were not previously covered. The first edition of the *Standard for Seismic Design of Cold-Formed Steel Structural Systems–Special Bolted Moment Frames*, hereinafter referred to as the *Standard*, was completed in 2007. This first edition was developed based on the 2005 Edition of the ANSI/AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2005), and the research work (Uang and Sato, 2007) on cold-formed steel special bolted moment frames system as a seismic force resisting system. The Committee has prepared this *Standard* using the best available knowledge to date. It is intended that this *Standard* be used in conjunction with the ASCE/SEI 7-05 (ASCE, 2005).

A. GENERAL

A1 Limits of Applicability and Terms

A1.1 Scope

Structural steel building systems in seismic regions are generally expected to dissipate seismic input energy through controlled inelastic deformations of the structure. This *Standard* supplements AISI S100, *North American Specification for the Design of Cold-Formed Steel Structural Members*, hereinafter referred to as AISI S100 for such applications. The seismic design loads specified in the model building codes have been developed considering the energy dissipation generated during inelastic response.

A1.2 Applicability

It should be noted that this *Standard* was developed specifically for cold-formed steel structures and buildings with the cold-formed steel special bolted moment frames as the main *seismic force resisting system*. This *Standard* does not apply to rack structures and cold-formed steel framing. Rack structures should be designed in accordance with the latest edition of *Design Testing and Utilization of Industrial Steel Storage Racks* by RMI (2004), and cold-formed steel framing should be designed in accordance with the latest edition of AISI S213, *Standard for Cold-Formed Steel Framing – Lateral Design* (AISI, 2007b). For hot-rolled steel buildings and structures, ANSI/AISC 341, *Seismic Provisions for Structural Steel Buildings* (AISC, 2005), should be followed.

This *Standard* is intended to be mandatory for buildings and other structures in *Seismic Design Categories* (SDCs) D through F. In general, for structures in SDC A, B and C, the designer is given a choice to either solely use AISI S100 (AISI, 2007) and the response modification coefficient, R, given in the *applicable building code* or ASCE/SEI 7 for "structural steel buildings not specifically detailed for seismic resistance" (typically R = 3), or the designer may choose to assign a higher value for R to a system detailed for seismic resistance and follow the requirements of this *Standard*.

A1.3 Definitions

Terms defined in this section are self-explanatory.

A2 Seismic Design Requirements

A2.1 General

When designing structures to resist earthquake motions, each structure is categorized based upon its occupancy to establish the potential earthquake hazard that it represents. Determining the *available strength* differs significantly in each specification or model building code. The primary purpose of this *Standard* is to provide information necessary to determine the required and *available strengths* of cold-formed steel structures. The following discussion provides a basic overview of how several seismic codes or specifications categorize structures and how they determine the *required strength* and stiffness. For the variables required to assign *seismic design categories*, limitations of height, vertical and horizontal irregularities, site characteristics, etc., the *applicable building code* should be consulted.

In ASCE/SEI 7 (ASCE, 2005), structures are assigned to one of four *occupancy categories*. Category IV, for example, includes essential facilities. Structures are then assigned to a *Seismic Design Category* based upon the occupancy categories and the seismicity of the site. Seismic design categories A, B and C are generally applicable to structures with low to moderate seismic risk, and special seismic provisions like those in this *Standard* are optional. However, special seismic provisions are mandatory in seismic design categories D, E, and F, which cover areas of high seismic risk.

A2.2 Story Drift

For non-seismic applications, story drift limits (like deflection limits) are commonly used in design to ensure the serviceability of the structure. These limits vary because they depend upon the structural usage and contents. As an example, for wind loads, such serviceability limit states are regarded as a matter of engineering judgment rather than absolute design limits (Fisher and West, 1990), and no specific design requirements are given in AISI S100.

The situation is somewhat different when considering seismic effects. Research has shown that story drift limits, although primarily related to serviceability, also improve frame stability (P-Δ effects) and seismic performance because of the resulting additional strength and stiffness. Although some model building codes, load standards and resource documents contain specific seismic drift limits, there are major differences among them as to how the limit is specified and applied. Nevertheless, drift control is important to both the serviceability and the stability of the structure. As a minimum, the designer should use the drift limits specified in the *applicable building code*.

The analytical model used to estimate building drift should accurately account for the stiffness of the frame elements and *connections* and other structural and nonstructural elements that materially affect the drift.

A3 Loads and Load Combinations

The *required strength* of a *seismic force resisting system* should be determined in accordance with the *applicable building code*. An amplification or overstrength factor Ω_o applied to the horizontal portion of the earthquake load E is prescribed in the *applicable building code*. If the *applicable building code* does not include the systems covered in this *Standard*, the overstrength factors in Appendix 1 of this *Standard* should be used.

A4 Nominal Strength

The *nominal strength* of systems, members and connections should be determined in accordance with AISI S100, except as modified by this *Standard*. See Section D1 of the *Commentary* for further details on special requirements.

A5 Referenced Documents

The specifications, codes and standards referenced in this *Standard* are listed with appropriate edition dates that were used in the development of the *Standard*.

B. MATERIALS

B1 Material Specifications

The ASTM steel designations and grades that are permitted by this *Standard* are based on those listed in ANSI/AISC 341 and in AISI S100. However, some grades within designations were excluded to ensure a higher level of ductility and reserve strength for inelastic seismic loadings.

Grades excluded include A500 hollow structural sections Grades A and D; A572 /A572M Grades 60 (415) and 65 (450); and Grades 70 (480) and 80 (550) of the various sheet specifications (A653/A653M, A875/A875M, A1008/A1008M, and A1011/A1011M). The remaining grades provide a F_u/F_y ratio not less than 1.15 and an elongation in 2 in. (50 mm) not less than 12 percent except for a few cases. The elongation is 11 and 9 percent for A1011/A1011M Grades 50 (340) and 55 (380), respectively, in thicknesses from 0.064 in. (2.5 mm) to 0.025 in. (0.65 mm). The elongation is 10 percent and the ratio 1.08 for all ST grades of A1003/A1003M.

Because this *Standard* is limited to material not more than 1 in. (25.4 mm) thick, it was not considered necessary to specify notch toughness requirements.

B1.1 Material Properties for Determination of Nominal Strength

The basis for the R_y and R_t values is as follows. A recent study was made of typical properties of as-produced plate (Brockenbrough, 2003). The database included a significant quantity of relatively thin material (some supplied in coil form). The ratio of the mean yield stress to the *specified minimum yield stress*, and the ratio of the mean tensile strength to the *specified minimum tensile strength*, were as follows:

Table C-B1.1
Ratios of Mean-to-Specified Yield Stress and Mean-to-Specified Tensile Strength

ASTM Designation	Thickness Range, in. (mm)	No. of Data Items	Ratio of Mean-to-Specified Yield Stress	Ratio of Mean-to-Specified Tensile Strength
A36/A36M	0.188-0.75 (4.78-19.0)	14,900	1.30	1.17
A572/A772M Grade 50 (340)	0.188-0.50 (4.78-12.7)	1,161	1.17	1.18
A588/A588M	0.312-2.00 (7.70-50.8)	1,501	1.18	1.15

These values were generally supported by a subsequent study that included limited additional data and a review of existing data (Liu, et al, 2006). Rounded values were adopted for this *Standard*, which agree with those for plate material in ANSI/AISC 341. Although no data for the other plate steels listed in Table B1.1 of this *Standard* were available, it was considered likely that the ratios for A242/A242M, A283/A283M, and A529/A529M steel would be in the same range.

The R_y and R_t ratios for hollow structural sections, A500 and A847 steel, were taken from ANSI/AISC 341. The R_y and R_t ratios for all sheet and strip grades (A606, A653/A653M, A792/A792M, A875, A1003/A1003M, A1008/A1008M, and A1011/A1011M) were selected to agree with those for strap bracing in AISI S213. These strap bracing ratios are based on a 1995 study made by Bethlehem Steel for the U.S. Army Corps of Engineers on ASTM A653 (ASTM, 2002) material. In this study, data were gathered from two galvanized coating lines, where the conditions of the lines varied significantly so as to provide a good range of test results. However, the user is cautioned that while over 1000 coils were included in the study, individual sample size (grade/coating) varied from as few as 30 to as many as 717 coils. An individual sample may include several thicknesses for a given sample grade and coating.

R_{cf}, a factor to account for the increase in yield stress due to cold work of forming averaged over the cross section, was taken as 1.10 based on a review of typical cold-formed channel sections. This was deemed to be a representative value for fully effective sections. It is somewhat conservative for sections that are not fully effective, because the more limited effects of cold working are included indirectly in the basic strength equations for those sections.

C. CONNECTIONS, JOINTS, AND FASTENERS

Connections, *joints* and fasteners that are part of the *seismic force resisting system* should be designed in accordance with AISI S100, except as modified in Chapter C of this *Standard*.

Tension or shear fracture, bolt shear, and block shear rupture are examples of limit states that generally result in non-ductile failure of connections. As such, these limit states are undesirable as the controlling limit state for connections that are part of the seismic force resisting system. Accordingly, it is required that connections be configured such that a ductile limit state in the member or connection, such as yielding or bearing deformation, controls the *available strength*.

C1 Bolted Joints

This *Standard* prohibits the bolted *joints* being designed to share the load in combination with welds. Due to the potential of full load reversal and the likelihood of inelastic deformations in connecting elements, bolts may exceed their slip resistances under significant seismic loads. Welds that are in a common shear plane to these bolts will likely not deform sufficiently to allow the bolts to slip into bearing, particularly if subject to load reversal. Consequently, the welds will tend to resist the entire force and may fail if they were not designed as such.

The potential for full reversal of design load and the likelihood of inelastic deformations of members and/or connected parts necessitates that bolts in joints of the *seismic force resisting system* be tightened to at least the snug tight condition.

Earthquake motions are such that slip cannot and need not be prevented. To prevent excessive deformations of bolted joints due to slip between the connected plies under earthquake motions, the use of holes in bolted joints in the *seismic force resisting system* is limited to standard holes and short-slotted holes with the direction of the slot perpendicular to the line of force.

C2 Welded Joints

The general requirements for welded *joints* are given in AWS D1.1 (AWS, 2006) and AWS D1.3 (AWS, 1998), as applicable, wherein a Welding Procedure Specification (WPS) is required for all welds. When the typically thin elements of cold-formed structures in tension are joined by welding, it is almost always in single pass flare bevel welds. Many operations during fabrication, erection, and the subsequent work of other trades have the potential to create discontinuities in the *seismic force resisting system*. When located in regions of potential inelasticity, such discontinuities should be repaired by the responsible subcontractor. Discontinuities should also be repaired in other regions of the seismic force resisting system when the presence of the discontinuity would be detrimental to the system performance. Repair may be unnecessary for some discontinuities.

C3 Other Joints and Connections

Alternative *joints* and *connections* are permitted by this *Standard* if they are justified by the *professional engineer*.

Alternative joints must, as a minimum, provide the same performance as the joints permitted by this *Standard*.

D. SYSTEMS

D1 Cold-Formed Steel – Special Bolted Moment Frames (CFS–SBMF)

Cold-formed steel special bolted moment frame (CFS–SBMF) systems are expected to experience substantial inelastic deformation during significant seismic events. It is expected that most of the inelastic deformation will take place at the bolted *connections*, due to slip and bearing. To achieve this, beams and columns should have sufficient strength when subjected to the forces resulting from the motion of the *design earthquake*. Hong and Uang (2004) tested a total of nine full-scale beam-column specimens; see Table C-D1-1 for the test matrix. These specimens simulated a portion of an interior beam-to-column subassembly with a column height of 8.25 ft (2.51 m) and a bay width of 11 ft (3.35 m). Contrary to a general belief that cold-formed steel lack ductility, this testing program demonstrated that this type of system actually can develop significant ductility. Figure C-D1-1 illustrates the typical hysteresis behavior. All specimens developed a story drift capacity significantly larger than the 0.04 radians required for Special Moment Frames (SMF) in the ANSI/AISC 341 (AISC, 2005).

The height limitation of 35 feet is based on practical use only and not from any limits on the CFS–SBMF system strength. It is possible for the CFS–SBMF system to meet drift limits and support the loads associated with larger system heights, provided that members are sized accordingly and the design methods contained within this *Standard* are adhered to. The *Standard* was developed assuming that the CFS–SBMF system uses the same-size beams and same-size columns throughout. It was also assumed that the system would engage all primary columns, which support the roof or floor above, and that those columns would be supported on a level floor or foundation.

In 2009, the *Standard* was revised to reflect these assumptions in the requirements for the system.

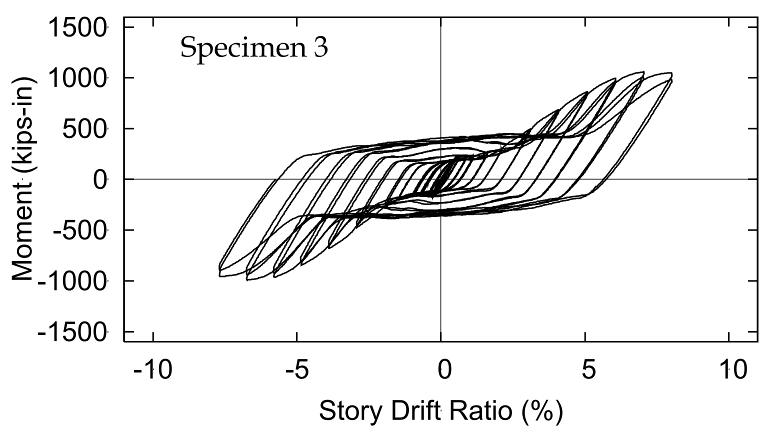


Figure C-D1-1 Typical Hysteresis Behavior of CFS–SBMF Systems (Hong and Uang, 2004)

Table C-D1-1
Test Matrix

Specimen No.	Beam	Column	Bearing Plate in. (mm)	Bolt configuration✦, in.		
				a, in. (mm)	b, in. (mm)	c, in. (mm)
1, 2	2C12 × $3^1/_2$ × 0.105	HSS8 × 8 × $^1/_4$	0.135 (3.43)	$2^1/_2$ (63.5)	3 (76.2)	$4^1/_4$ (108)
3	2C16 × $3^1/_2$ × 0.105	HSS8 × 8 × $^1/_4$	N/A	3 (76.2)	6 (152)	$4^1/_4$ (108)
4	2C16 × $3^1/_2$ × 0.105	HSS8 × 8 × $^1/_4$	0.135 (3.43)	3 (76.2)	6 (152)	$4^1/_4$ (108)
5, 6, 7	2C16 × $3^1/_2$ × 0.135	HSS8 × 8 × $^1/_4$	N/A	3 (76.2)	6 (152)	$4^1/_4$ (108)
8, 9	2C20 × $3^1/_2$ × 0.135	HSS10 × 10 × $^1/_4$	N/A	3 (76.2)	10 (254)	$6^1/_4$ (159)

Note: ✦ 1 in. (25.4 mm) diameter A325 bearing type high-strength bolts.
See Figure C-D1.1-1 for definitions of dimensions a, b, and c.

D1.1 Beam-to-Column Connections

Cold-formed steel special bolted moment frame (CFS–SBMF) systems are comprised of cold-formed steel, single- or double-channel beams, and hollow structural section (HSS) columns. The beams and columns are connected by snug-tight high-strength bolts. Typical detail for this type of connection is shown in Figure C-D1.1-1. Bearing plates can be used to increase the bearing strength of the bolt.

Components of story drift due to the deformation of beam and column, and bolt slippage and bearing for a typical test specimen are shown in Figure C-D1.1-2 (Hong and Uang, 2004). The inelastic deformation was mainly from the slip and bearing deformations of the bolted connection. By properly limiting the width-thickness ratios for both the beam and column, inelastic action in these members can be prevented.

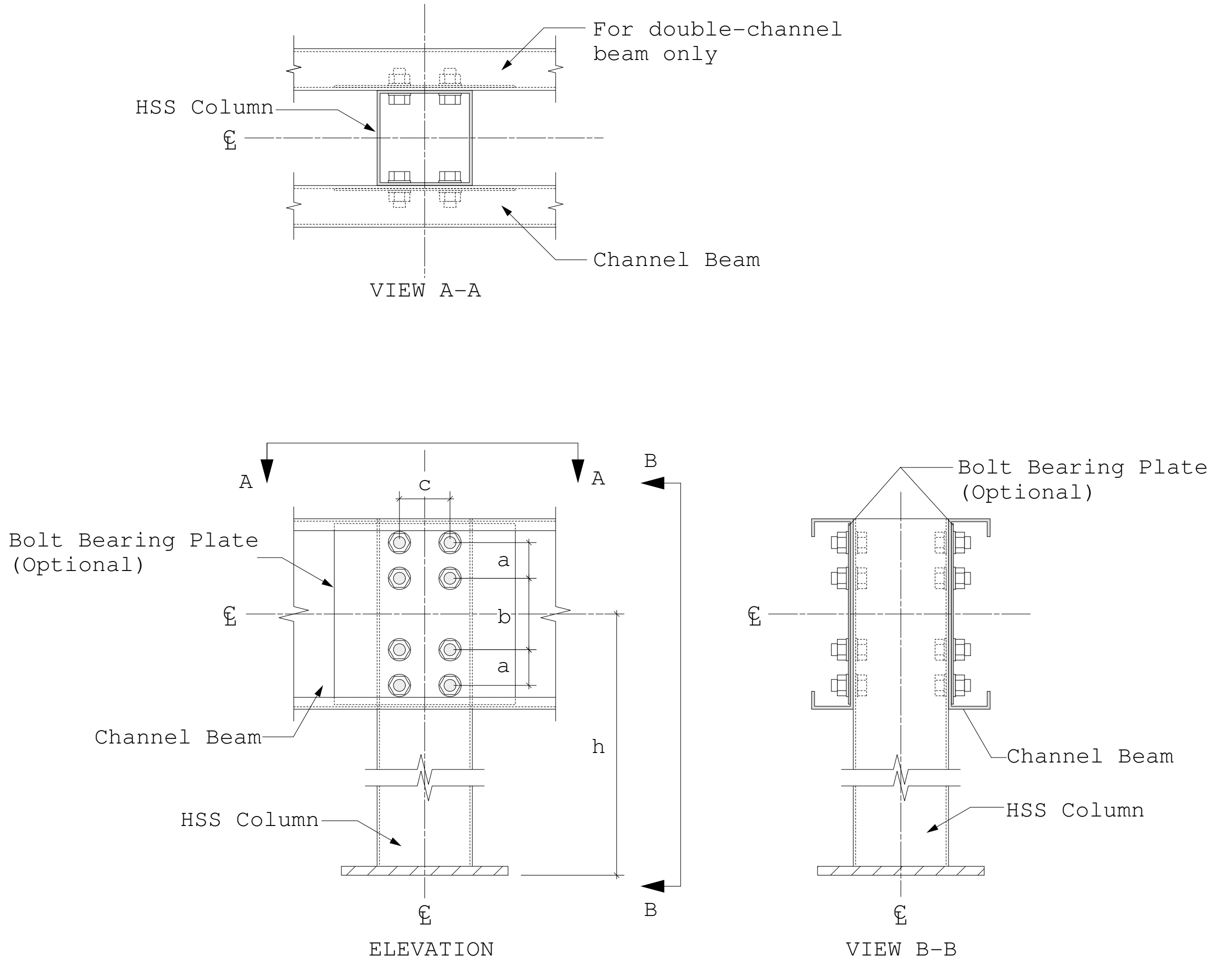
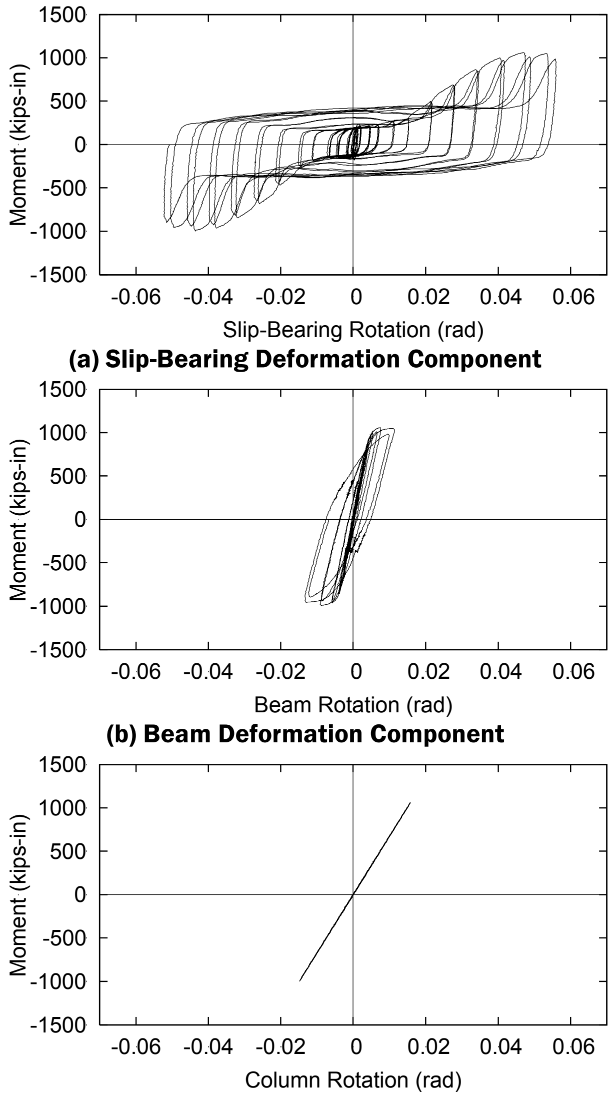


Figure C-D1.1-1 Typical CFS–SBMF Systems Bolted Connection



(a) Slip-Bearing Deformation Component

(b) Beam Deformation Component

(c) Column Deformation Component

Figure C-D1.1-2 Components of Story Drift (Hong and Uang, 2004)

D1.1.1 Connection Limitations

In 2009, modifications were made for consistency with the test database.

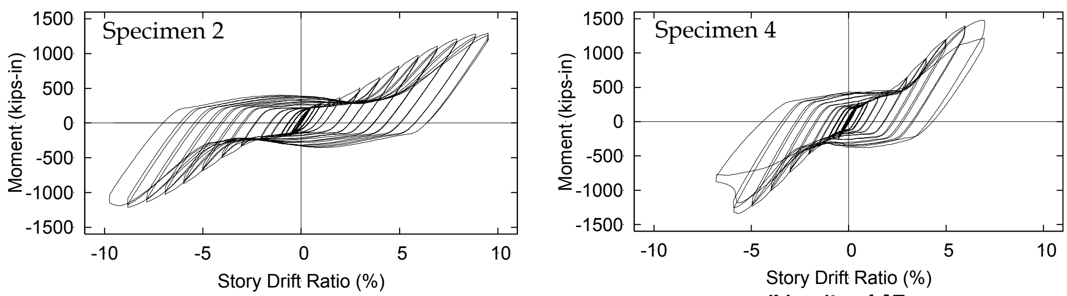
D1.2 Beams and Columns

The test matrix in Table C-D1-1 was developed to allow for the effect of local buckling on strength degradation.

D1.2.1 Beam Limitations

Unlike the strong column-weak beam concept adopted in the ANSI/AISC 341 for Special Moment Frame design, buckling of a cold-formed steel beam is the most undesirable failure mode in CFS–SBMF systems. As shown in Figure C-D1.2-1, rapid strength degradation would occur when the beam web flat depth-to-thickness ratio (w/t) is 147. Two measures are taken to avoid such strength degradation: (1) limit the design story drift ratio to no greater than 0.05, and (2) limit the w/t ratio to no greater than $6.18\sqrt{E/F_y}$.

In 2009, ASTM A653 was specified for cold-formed steel C-section members based on the test database. In addition, limitations on the beam depth, thickness, and surface treatment were added to reflect the test database.



(a) w/t = 109 **(b) w/t = 147**

Figure C-D1.2-1 Beam Local Buckling Effect on Strength Degradation (Hong and Uang, 2004)

A single channel beam configuration is permitted by AISI S110; however, only the double channel beam configuration has been tested to date. Since the single channel configuration is unsymmetrical, it could possibly induce torsion into the channel and column. In 2009, further clarification was added requiring designers to demonstrate that the torsional effect is properly taken into account when the design uses a single-channel beam.

Typically, the beam top flanges are connected to a floor deck (normally steel deck and plywood). This will resist the small torsion in the column due to the load on one side only. Also, designers should include in their column check the ability to add the torsion stress to the bending and axial load stresses to ensure a properly designed column.

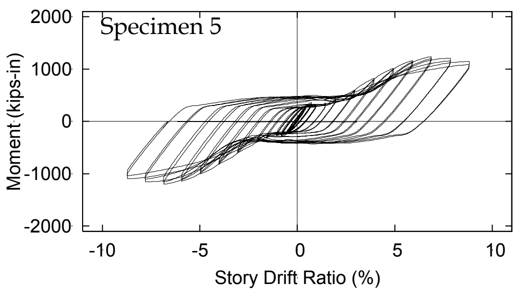
If a system is constructed without deck attached to the beam flanges, the torsion forces should be included in the column design.

Consider a seismic force at the top of the column which is typically 2 to 3 kips. The seismic force would result in a torsional moment of (4 x 3 = 12 in-kips (1.36 m-kN) or 5 x 3 = 15 in-kips (1.69 m-kN)). The seismic moment in the column is in the range of 360 to 600 in-kips (40.7 to 67.8 m-kN) with axial loads of 30 to 50 kips (133 to 222 kN). In this case, the torsional moment would not control the design.

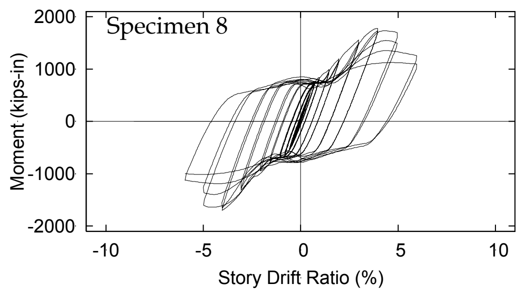
D1.2.2 Column Limitations

Column buckling is not as detrimental as beam buckling in terms of strength degradation, partly because the HSS column section is comprised of stiffened elements. When a slender section in accordance with the ANSI/AISC 360 (AISC, 2005) is used, test results show that significant strength degradation may occur (see Figure C-D1.2-2). This undesirable failure mode can be avoided by limiting both the flat width-to-thickness ratio to $1.40\sqrt{E/F_y}$ and the maximum story drift to 3 percent of the story height.

In 2009, to reflect limitations of the test database, ASTM A500 for hollow structural section (HSS) members painted with a standard industrial finished surface was specified for columns. Upper and lower limits on the column depths were added as well to mirror the limitations of the tests.



(a) w/t = 31



(b) w/t = 40

Figure C-D1.2-2 Column Local Buckling Effect on Strength Degradation (Hong and Uang, 2004)

D1.2.3 Required Strength

D1.2.3.1 Beams and Columns

To ensure that inelastic action will only occur at the bolted *connections*, capacity design principles should be followed to calculate the maximum force that can be developed in these connections at the design story drift. Beams and columns are then designed to remain essentially elastic based on this maximum force.

It is common that all the beams in CFS–SBMF are the same size and so are all the columns. All the beam and column connections have the same bolt configuration. This leads to the assumption of the desirable yield mechanism with the expected distribution of column shears as shown in Figure C-D1.2-3(a). The lateral load response of one column is shown in Figure C-D1.2-3(b). At the *design story drift*, Δ, the column shear is $(V_S + R_tV_B)$, and the expected moment at the bolt group is

$$M_e = h(V_S + R_t V_B) \tag{C-D1.2-1}$$

where h is story height, and R_t is the factor given in *Standard* Table B1.1.

In the above equation, V_S is the column shear that causes the bolt group to slip [Point a in Figure C-D1.2-3(b)]; R_t is the ratio of *expected tensile strength* to *specified minimum tensile strength*. The bolt hole oversize allows the bolt group to rotate by an amount, which produces a component of story drift of Δ_S in Figure C-D1.2-3(b), until bolt bearing occurs (Point b). To overcome the *bearing* resistance, the additional column shear required to reach the design story drift (Point c) is defined as R_tV_B.

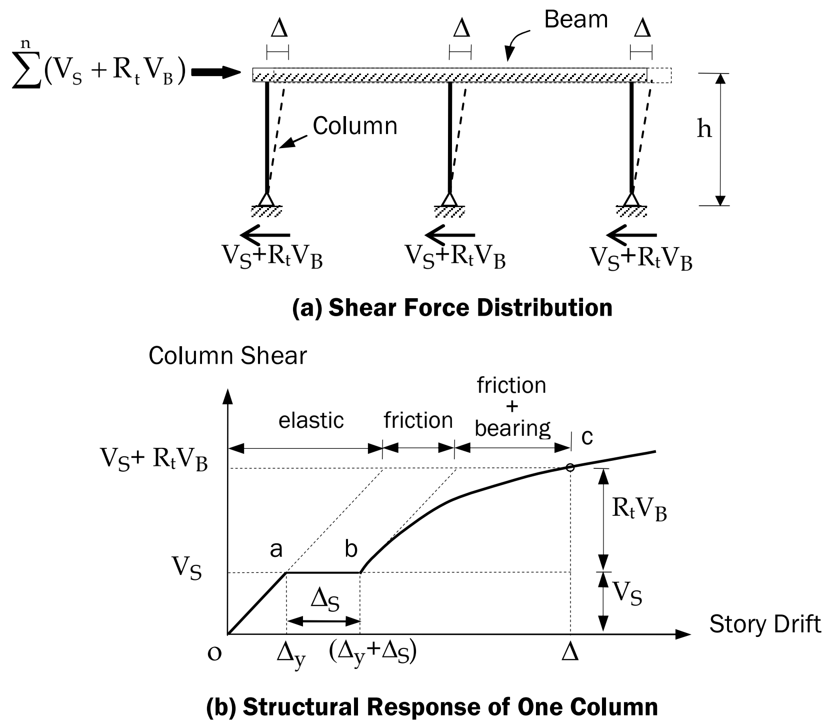


Figure C-D1.2-3 General Structural Response of CFS–SBMF System

Figure C-D1.2-4 shows a bolt group with an eccentric shear at the column base. The instantaneous center (IC) of rotation concept (Crawford and Kulak, 1971) can be applied to compute the required response quantities. At the bolt level, the slip resistance of one bolt, R_S, is

$$R_S = kT \tag{C-D 1.2-2}$$

where k = slip coefficient, T = *snug-tight bolt* tension. A value of k = 0.33 is assumed, and the value of T ranges from 10 (44.5 kN) to 25 kips (111 kN) for 1-in. (25.4 mm) diameter snug-tight bolts. For design purposes, a value of T equal to 10 kips (44.5 kN) is recommended for 1-in. (25.4 mm) diameter snug-tight bolts.

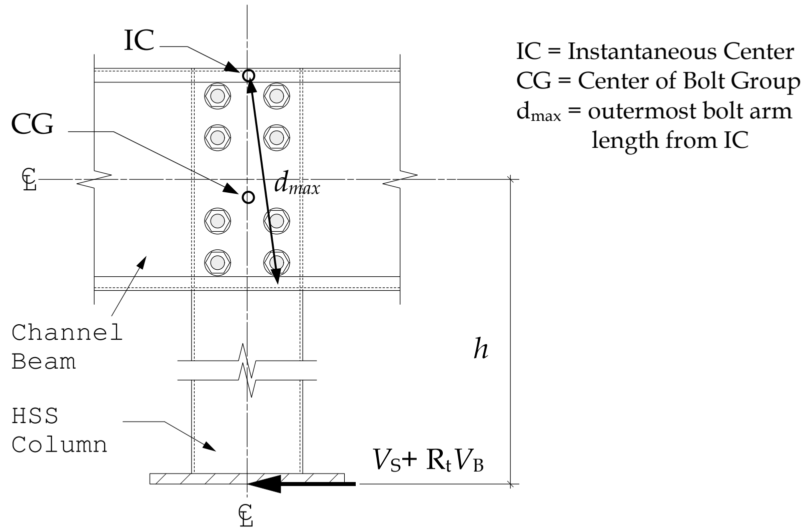


Figure C-D1.2-4 Bolt Group in Eccentric Shear

The slip range, Δ_S, in Figure C-D1.2-3(b) is a function of the bolt hole oversize and can be computed as

$$\Delta_S = \frac{2h_{os}h}{d_{max}} \tag{C-D1.2-3}$$

where h_{os} = hole oversize (difference between hole diameter and bolt diameter), and d_{max} = outermost bolt arm length from instantaneous center (IC).

The bearing resistance of a bolt is

$$R_B = R_{ult}(1-e^{-\mu\delta})^{\lambda} \tag{C-D1.2-4}$$

where δ = bearing deformation, R_{ult} = ultimate bearing strength, e = 2.718, μ and λ = regression coefficients. For application in cold-formed steel special bolted moment frame systems, μ = 5 and λ = 0.55 gave a reasonable correlation to available test results (Sato and Uang, 2007).

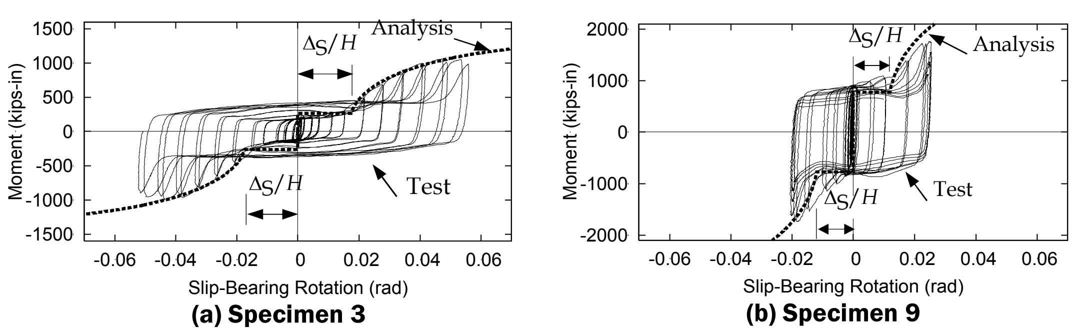


Figure C-D1.2-5 Sample Correlation of Bolted Connection Response

Based on the above procedure, sample correlation of two test specimens is shown in Figure C-D1.2-5.

Values of V_S and Δ_S can be computed by using the instantaneous center of rotation theory, and Table C-D1.2-1 shows the results for some commonly used bolt configurations and story heights. Equations (D1.2.3.1-2) and (D1.2.3.1-7) of the *Standard* are derived from regression analysis of Table C-D1.2-1 to facilitate design.

Next, consider V_B in Eq. C-D1.2-1 (or *Standard* Eq. D1.2.3.1-1). Referring to Point c in Figure C-D1.2-3(b), the design story drift (Δ) is composed of three components: (1) the recoverable elastic component which is related to the lateral stiffness, K, of the frame, (2) the slip component, Δ_S, from *Standard* Eq. D1.2.3.1-7, and (3) the bearing component:

$$\Delta_B = \Delta - \Delta_S - \frac{nM_e}{hK} \quad \text{(C-D1.2-5)}$$

where n = number of columns in a frame line (i.e., number of bays plus 1), and M_e is the expected moment at a bolt group as defined in *Standard* Section D1.2.3.

Applying the instantaneous center of rotation concept to the eccentrically loaded bolt group in Figure C-D1.2-4 by using the bolt bearing relationship in Eq. C-D1.2-4, the relationship between the bearing component of the story drift, Δ_B, and the bearing component of the column shear, V_B, can be established. Figure C-D1.2-7(a) shows a sample result. For a given story height, the last point of each curve represents the ultimate limit state when the bearing deformation of the outermost bolt reaches 0.34 in. (8.6 mm).

Values of $V_{B,max}$ and $\Delta_{B,max}$ for some commonly used bolt configurations and story heights are computed. *Standard* Eqs. D1.2.3.1-4 and D1.2.3.1-6 are derived from regression analysis of Table C-D1.2-2 to facilitate design.

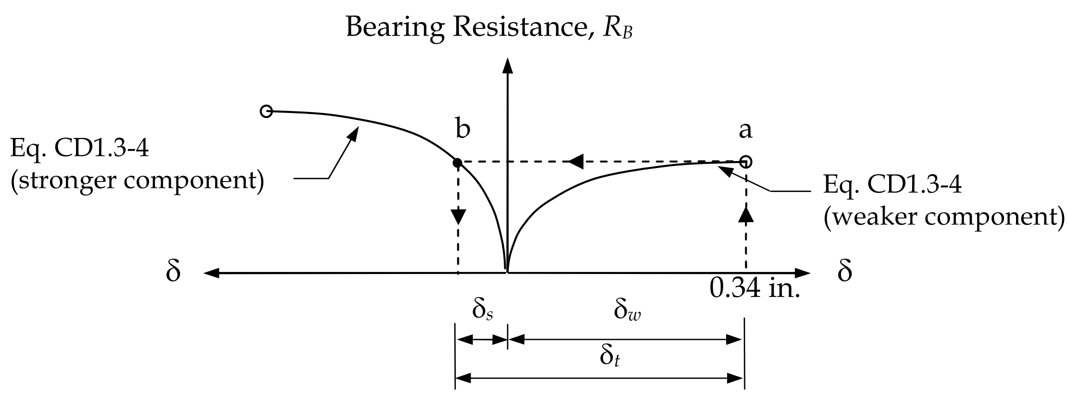


Figure C-D1.2-6 Bolt Bearing Deformations in Stronger and Weaker Components

The Bearing Deformation Adjustment Factor, C_{DB}, in Eq. C-D1.2-7 accounts for the additional contribution of bearing deformation from the stronger component.

Refer to Point a in Figure C-D1.2-6, where the ultimate bearing deformation [= 0.34 in. (8.6 mm)] of the weaker component is reached. Since the bearing forces of the bolt between both the weaker and stronger components are identical, it can be shown that the corresponding bearing deformation of the stronger component (i.e., Point b) is

$$\delta_s = -\frac{1}{5}\ln\left[1 - 0.817\left(\frac{(tF_u)_w}{(tF_u)_s}\right)^{1.82}\right] \quad \text{(C-D1.2-6)}$$

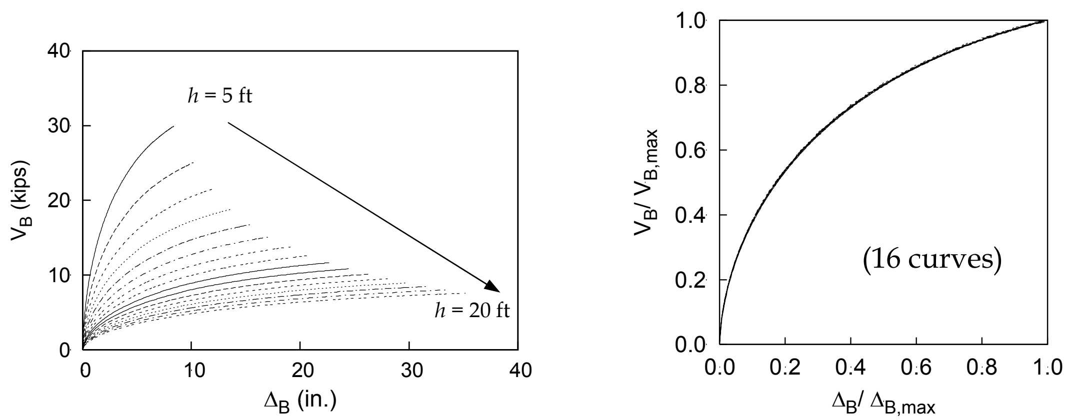
The C_{DB} factor represents the ratio between the total bearing deformation and 0.34 in. (8.6 mm).

$$C_{DB} = \frac{0.34 + \delta_s}{0.34} = 1.0 - 0.588\ln\left[1 - 0.817\left(\frac{(tF_u)_w}{(tF_u)_s}\right)^{1.82}\right] \quad \text{(C-D1.2-7)}$$

Note that the $\Delta_{B,0}$ values correspond to the maximum drift deformation when the bearing deformation is contributed by the weaker component only.

Normalizing each curve in Figure C-D1.2-7(a) by its own ultimate limit state, Figure C-D1.2-7(b) shows that a normalized relationship between V_B and Δ_B can be established:

$$\left(\frac{V_B}{V_{B,max}}\right)^2 + \left(1 - \frac{\Delta_B}{\Delta_{B,max}}\right)^{1.43} = 1 \quad \text{(C-D1.2-8)}$$



(Column: HSS8 × 8 × 1/4, Beam: 2C12 × 3 1/2 × 0.105, Bearing Plate: 0.135 in.)

(a) Bearing Response **(b) Normalized Bearing Response**

Figure C-D1.2-7 Sample Result of Bearing Response

Iteration is required to compute the expected moment, M_e, in Eq. C-D1.2-1. A flowchart is provided in step 4 of Section D1.5. The following value is suggested as the initial value for Δ_B:

$$\Delta_B = \frac{[\Delta - (\Delta_S + \Delta_y)]K}{nV_{B,max}/\Delta_{B,max} + K} \quad \text{(C-D1.2-9)}$$

where Δ_y is the story drift at point a in Figure C-D1.2-3(b).

D1.3 Design Story Drift

From Figure C-D1.2-3, the design story drift, Δ, resulting from the motion of the *design earthquake* is needed to compute the required force in the beams and columns. The *design story drift* is generally computed in accordance with the *applicable building code* but modified by using an empirical Deflection Amplification Factor, C_d,. The basis of the C_d factor in *Standard* for a CFS–SBMF system follows.

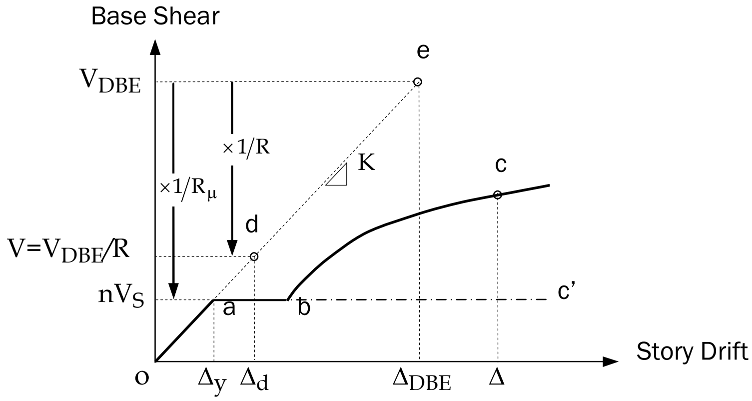


Figure C-D1.3-1 General Response of CFS–SBMF System

Figure C-D1.3-1 shows the general response of a CFS–SBMF system. For design purposes, the elastic seismic force produced by the design earthquake (Point e) is reduced by a Response Modification Coefficient, R, of 3.5; the corresponding story drift at Point d is Δ_d. The bolted connections actually slip at Point a, producing pseudo-yielding at a base shear of nV_S, where V_S is computed from *Standard* Section D1.2.3, and n is the number of columns in a frame line. The ratio between the base shears at Point e and a is the system ductility reduction factor:

$$R_\mu = \frac{V_{DBE}}{nV_S} \qquad \text{(C-D1.3-1)}$$

where V_{DBE} is the elastic base shear corresponding to the Design Basis Earthquake, and R_μ is the system ductility reduction factor.

The ratio between the story drifts at Points c and a is defined as the system ductility factor:

$$\mu = \frac{\Delta}{\Delta_y} \qquad \text{(C-D1.3-2)}$$

Newmark and Hall (1982) proposed a relationship between μ and R_μ for a single-degree-of-freedom system that responds in an elasto-perfectly plastic (EPP) manner (path o-a-b-c′):

$$R_{\mu(N\text{-}H)} = \begin{cases} \mu & \text{for } T \geq T_S \\ \sqrt{2\mu - 1} & \text{for } T \leq T_C \end{cases} \qquad \text{(C-D1.3-3)}$$

where T_S is defined in the *applicable building code*, and $T_C = T_S\sqrt{2\mu - 1}/\mu$. Since the actual response of a CFS–SBMF system exhibits a significant hardening (path o-a-b-c) when the bolts are in bearing, for a given ductility factor it is expected that the ductility reduction factor should be higher than that given in Eq. C-D1.3-3. A parametric study was conducted, and the result in Table C-D1.3-1 shows that it is reasonable to assume the following (Sato and Uang, 2007):

$$R_\mu = 1.2R_{\mu(N-H)} \qquad \text{(C-D1.3-4)}$$

For the period not shorter than T_S (i.e., $T \geq T_S$), the above equation gives $R_\mu = 1.2\mu$. Using the relationships in Figure C-D1.3-1,

$$\Delta = \mu\Delta_y = \frac{R_\mu}{1.2}\Delta_y = \frac{V_{DBE}}{1.2(nV_S)}\left(\frac{nV_S}{K}\right)$$

$$= \frac{V_{DBE}}{1.2K} = 0.83\Delta_{DBE} = (0.83R)\Delta_d \tag{C-D1.3-5}$$

that is, the Deflection Amplification Factor, C_d, is 0.83R. For an R value of 3.5, the value of C_d is about 3.0. Based upon recommendations from the Provisions Update Committee (PUC) of Building Seismic Safety Council (BSSC), however, the value of C_d has been conservatively increased to 3.5.

For $T \leq T_C$, a simple expression for C_d cannot be derived. Following a similar procedure would give the following for the design story drift (Sato and Uang, 2007).

$$\Delta = \frac{1}{2K}\left(nV_S + 0.7\frac{V_{DBE}^{\,2}}{nV_S}\right) \tag{C-D1.3-6}$$

where

$$T_C = T_S\left(\frac{nV_S}{V_{DBE}}\sqrt{\frac{2V_{DBE}}{nV_S} - 1}\right) \tag{C-D1.3-7}$$

For structures having a period between T_S and T_C, Δ can be determined from linear interpolation.

Table C-D1.3-1
Average Value of R_μ Ratio

Ductility Factor	$\mu = 4$	$\mu = 6$	$\mu = 8$
$R_{\mu(actual)}/R_{\mu(EPP)}$	1.14	1.23	1.26

In 2009, the drift limit in AISI S110 was deleted in favor of the current allowable story drift in ASCE/SEI 7, which limits the drift to a range from 0.025h for Occupancy Category I and II buildings and structures to as little as 0.015h for Occupancy Category IV buildings and structures. The intent of these drift limits is to control damage to nonstructural components that are attached to the *lateral force resisting system*. However, Footnote c of Table 12.12-1 in ASCE/SEI 7 waives the drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. This footnote is certainly valid in the case of most CFS–SBMF systems, which are commonly used in industrial platforms. However, for non-structural components that are susceptible to drift damage, the more stringent drift limits specified in Table 12.12-1 in ASCE/SEI 7 should be applied.

D1.4 P-Δ Effects

P-Δ effects should conform to the requirements of the *applicable building code*.

D1.5 Design Procedure

Step 1 – Preliminary design

Perform a preliminary design of the beams, columns, and bolted connections by considering all basic load combinations in the applicable building code. In determining the earthquake load, use a rational method to determine the structural period.

Step 2 – Compute both the base shear (nV_S) that causes the bolt groups to slip and the slip range (Δ_S) in terms of story drift

For a given configuration of the bolt group, *Standard* Eqs. D1.2.3.1-2 and D1.2.3.1-7 can be used to compute both V_S and Δ_S. n represents the number of columns in a frame line.

Step 3 – Compute design story drift, Δ

Follow the *applicable building code* to compute the design story drift, where the Deflection Amplification Factor is given in the *Standard*. For structures with $T \le T_C$, Eq. C-D1.3-6 can be used.

Step 4 – Perform capacity design check of beams and columns

Beams and columns should be designed based on special seismic load combinations of the *applicable building code*; the seismic load effect with overstrength, E_{mh}, can be replaced by the required strength in *Standard* Section D1.2.3. Iteration is required to compute the expected moment, M_e in *Standard* Section D1.2.3. The flowchart in Figure C-D1.5-1 can be used for this purpose.

Step 5 – Check P-Δ effects

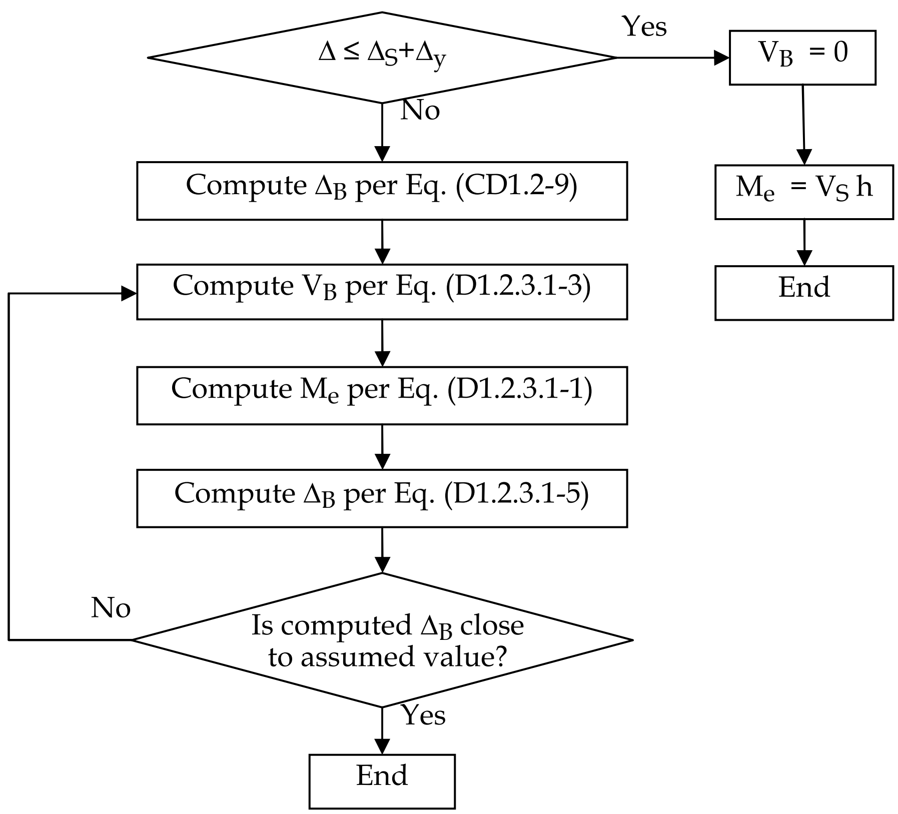


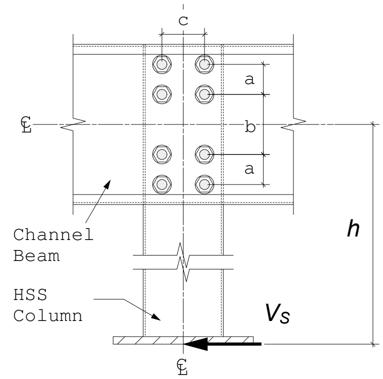
Figure C-D1.5-1 Flowchart for Computing Expected Moment M_e

Table C-D1.2-1
Values of G_S and G_{DS} for Eccentrically Loaded Bolt Groups

$$V_S = N \times G_S \times R_s$$
$$\Delta_S = G_{DS} \times h_{os}$$

N = 1 for single-channel beams
= 2 for double-channel beams

where
V_S = column shear causing slip
R_S = slip strength per bolt
(=k×T)
k = slip coefficient
T = snug-tight bolt tension
h = story height, ft
a, b, and c = bolt spacing, in.
Δ_S = slip drift due to slip
G_S, G_{DS} = coefficient tabulated below
h_{os}= hole oversize



c, in.	*h*, ft	Bolt spacing *a* and *b*, in.					
		a = 2-1/2, *b* = 3		*a* = 3, *b* = 6		*a* = 3, *b* = 10	
		G_S	G_{DS}	G_S	G_{DS}	G_S	G_{DS}
4-1/4	8	0.296	40.5	0.416	26.6	0.562	17.6
	9	0.264	45.8	0.370	30.3	0.501	20.1
	10	0.237	51.0	0.333	34.0	0.452	22.7
	11	0.216	56.3	0.303	37.7	0.411	25.3
	13	0.183	66.9	0.257	45.1	0.349	30.6
	15	0.158	77.5	0.223	52.6	0.303	35.9
	17	0.139	88.1	0.197	60.1	0.268	41.4
	19	0.125	98.7	0.176	67.6	0.240	46.9
	21	0.113	109	0.159	75.1	0.217	52.5
	23	0.103	120	0.145	82.6	0.198	58.1
	25	0.0946	130	0.134	90.2	0.182	63.7
	27	0.0879	141	0.124	97.7	0.169	69.3
	29	0.0818	152	0.115	105	0.157	75.0
	31	0.0763	162	0.108	113	0.147	80.7
	33	0.0714	173	0.101	120	0.138	86.4
	35	0.0678	183	0.0955	128	0.130	92.1
6-1/4	8	0.355	36.2	0.460	25.8	0.597	18.2
	9	0.315	40.9	0.410	29.3	0.531	20.9
	10	0.284	45.6	0.369	32.9	0.479	23.5
	11	0.259	50.4	0.335	36.4	0.436	26.2
	13	0.218	59.8	0.284	43.5	0.370	31.6
	15	0.189	69.3	0.246	50.5	0.321	37.0
	17	0.167	78.7	0.217	57.6	0.283	42.5
	19	0.150	88.2	0.194	64.7	0.253	48.0
	21	0.135	97.6	0.176	71.8	0.229	53.5
	23	0.124	107	0.161	78.9	0.210	59.0
	25	0.114	117	0.148	85.9	0.193	64.6
	27	0.105	126	0.137	93.0	0.179	70.1
	29	0.0977	135	0.127	100	0.166	75.7
	31	0.0915	145	0.119	107	0.156	81.2
	33	0.0859	154	0.112	114	0.146	86.8
	35	0.0810	164	0.105	121	0.138	92.4

Table C-D1.2-2

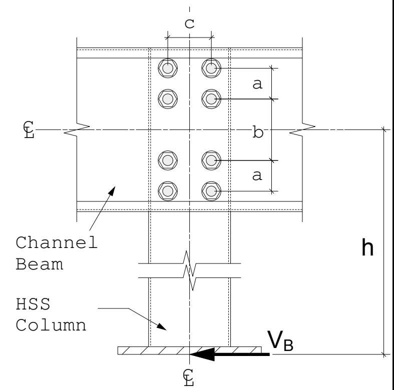
Values G_B and $\Delta_{B,0}$ for Eccentrically Loaded Bolt Groups

$$V_{B,max} = N \times G_B \times R_0$$

$$\Delta_{B,max} = C_{DB} \times \Delta_{B,0}$$

N = 1 for single-channel beams
= 2 for double-channel beams

where
$V_{B,max}$ = column shear causing bolt maximum bearing
R_0 = governing values of dtF_u of connected components
F_u = tensile strength
t = bearing thickness
d = bolt diameter
G_B = coefficient tabulated below
$\Delta_{B,0}$ = maximum bearing drift deformation
C_{DB} = bearing deformation adjustment



c, in.	*h*, ft	Bolt spacing *a* and *b*, in.					
		a = 2-1/2, *b* = 3		*a* = 3, *b* = 6		*a* = 3, *b* = 10	
		G_B	$\Delta_{B,0}$, in.	G_B	$\Delta_{B,0}$, in.	G_B	$\Delta_{B,0}$, in.
4-1/4	8	0.524	6.92	0.728	4.77	0.983	3.50
	9	0.466	7.81	0.649	5.40	0.878	4.00
	10	0.420	8.71	0.586	6.04	0.794	4.49
	11	0.381	9.61	0.533	6.68	0.724	4.98
	13	0.323	11.4	0.453	7.95	0.616	5.97
	15	0.281	13.2	0.393	9.23	0.536	6.96
	17	0.247	15.0	0.347	10.5	0.474	7.95
	19	0.222	16.8	0.311	11.8	0.425	8.94
	21	0.200	18.6	0.281	13.1	0.385	9.92
	23	0.183	20.4	0.257	14.3	0.352	10.9
	25	0.169	22.2	0.237	15.6	0.325	11.9
	27	0.156	24.0	0.220	16.9	0.301	12.9
	29	0.145	25.8	0.204	18.2	0.281	13.9
	31	0.136	27.6	0.191	19.5	0.262	14.9
	33	0.127	29.4	0.180	20.7	0.247	15.8
	35	0.120	31.2	0.169	22.0	0.233	16.8
6-1/4	8	0.637	6.17	0.814	4.48	1.05	3.36
	9	0.566	6.97	0.725	5.08	0.935	3.82
	10	0.510	7.77	0.654	5.68	0.845	4.29
	11	0.464	8.57	0.595	6.28	0.771	4.76
	13	0.393	10.2	0.504	7.48	0.655	5.70
	15	0.341	11.8	0.438	8.68	0.570	6.65
	17	0.302	13.4	0.387	9.88	0.504	7.59
	19	0.269	15.0	0.347	11.1	0.452	8.54
	21	0.244	16.6	0.314	12.3	0.410	9.48
	23	0.222	18.2	0.287	13.5	0.374	10.4
	25	0.205	19.8	0.264	14.7	0.345	11.4
	27	0.189	21.4	0.244	15.9	0.319	12.3
	29	0.176	23.0	0.228	17.1	0.298	13.3
	31	0.165	24.6	0.213	18.3	0.279	14.2
	33	0.154	26.2	0.201	19.5	0.262	15.2
	35	0.146	27.8	0.189	20.7	0.247	16.1

E. QUALITY ASSURANCE AND QUALITY CONTROL

Quality assurance and quality control procedures as set forth in this *Standard* are essential for seismic systems.

Snug-tightened bolts are specified as is customary for this type of construction. However, a departure from traditional practice is to require that the bolt tightness be checked on a representative sample of bolts. This is because a modest level of tightness is required to develop the expected level of slip resistance in the *connections*. An ordinary spud wrench is used to make this check. It should be noted that fully pretensioned bolts, such as is required in slip-critical connections in heavier construction, are not suitable for cold-formed steel structural systems. The higher levels of tensioning for those applications are usually controlled by the turn-of-nut method, but the rotations specified are not applicable to cold-formed steel because they are based on greater grip lengths than those typically encountered with the thinner material. The turn-of-nut and other methods are outlined by the Research Council on Structural Connections.

APPENDIX I

In 2009, the system overstrength factor, Ω_o was decreased at 3.0 and deflection amplification factor, C_d, was increased to 3.5. These changes reflect recommendations from the BSSC PUC.

REFERENCES

American Institute of Steel Construction (2005), ANSI/AISC 341, *Seismic Provisions for Structural Steel Buildings*. Chicago, IL.

American Iron and Steel Institute (2007), AISI S100, *North American Specification for the Design of Cold-Formed Steel Structural Members*, Washington, DC.

American Iron and Steel Institute (2007b), AISI S213, *North American Standard for Cold-Formed Steel Framing – Lateral Design*, Washington, DC.

American Society of Civil Engineering (2005), ASCE/SEI 7-05, *Minimum Design Loads for Buildings and Other Structures*.

American Welding Society (1998), *Structural Welding Code – Sheet Steel*, ANSI/AWD D1.3-98, Miami, FL.

Brockenbrough, R. L. (2003), "MTR Survey of Plate Material Used in Structural Fabrication," Engineering Journal, AISC.

Crawford, S.F. and G.L. Kulak (1971), "Eccentrically Loaded Bolted Connections," *Journal of the Structural Division*, ASCE, Vol. 97, ST3, pp. 765-783.

Hong, J.K. and C.M Uang (2004). "Cyclic Testing of Cold-Formed Steel Moment Connection for Pre-Fabricated Mezzanines," *Report No. TR-04/03*, University of California, San Diego, La Jolla, CA.

International Code Council Inc. (2006), *International Building Code*. Whittier, CA.

Liu, J., R. Sabelli, R. L. Brockenbrough, T. P. Fraser (2007), "Expected Yield Stress and Tensile Strength Ratios for Determination of Expected Member Capacity in the 2005 AISC Seismic Provisions," Engineering Journal, AISC.

Rack Manufacturers Institute (2004), *Specification for the Design, Testing and Utilization of Industrial Steel Storage Racks*, Charlotte, NC.

Sato, A. and C.M. Uang (2007), "Development of a Seismic Design Procedure for Cold-Formed Special Bolted Frames," *Report No. SSRP-07/16*, University of California, San Diego, La Jolla, CA.

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