Design of Cold-Formed Steel Framed Building on Concrete Podium

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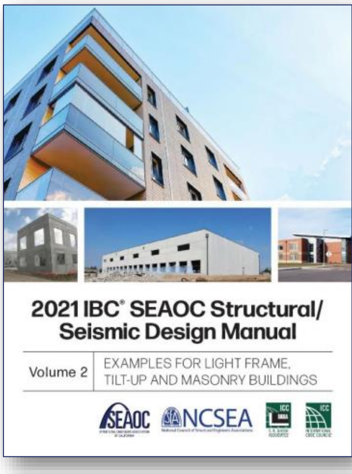
Agenda

- SSDM CFS building design example topics
- 2021 IBC/ 2022 CBC & reference AISI standards
- CFS light-frame construction
- CFS framed shear wall design example
- CFS wall stud bracing and design

CFS building design example topics



1. Building Geometry and Seismic Criteria
2. Roof and Floor Gravity Loads
3. Lateral Loading: Seismic
4. Diaphragm Flexibility
5. Flexible Diaphragm Condition
6. Building Classification: Regular or Irregular
7. Redundancy Factor
8. Redundancy Check for Building B
9. Selected Analytical Procedure
10. Distribution of Seismic Forces to Shear Walls
11. Sheathed CFS-Stud Shear Walls: Framing Materials
12. Shear Wall Design Example: Building B
13. Shear Wall Deflection
14. Discussion: Framing with Cold-Formed Steel

Cold-Formed Steel (CFS) Design References & 2021 IBC Provisions

CFS design standards & references

Download free specifications and standards at CFSEI website!

- AISI S100-16/S2-20 (Ref'd by 2021 IBC/ 2022 CBC), similar design provisions as S100-12 but re-formatted.
- AISI Standards (2020): *S240-20 CFS Structural Framing and S400-20 Seismic Design of CFS Structural Systems (Ref'd by 2021 IBC/ 2022 CBC, and S100).*
- AISI CFS Design Manual (D100-17) based on S100-16 *[Similar to AISC Steel Construction Manual]*
- AISI D110-16 CFS Framing Design Guide for S100 and S240 and AISI D113-19 Shear Wall Design Guide for S240 and S400
- Cold-Formed Steel Design, Fifth Edition
- SEAOC Structural/Seismic Design Manual – 2021 IBC Examples *(Force transfer & CFS Ex.'s in Vol. 2)*

** CFS support online: www.steel.org, www.steelframing.org, www.cfsei.org, www.buildsteel.org*

2021 IBC CFS design provisions

SECTION 2210
COLD-FORMED STEEL

2210.1 General. The design of cold-formed carbon and low-alloy steel structural members shall be in accordance with AISI S100. The design of cold-formed stainless-steel structural members shall be in accordance with ASCE 8. Cold-formed steel *light-frame construction* shall comply with Section 2211. Where required, the seismic design of cold-formed steel structures shall be in accordance with the additional provisions of Section 2210.2.

2210.1.1 Steel decks. The design and construction of cold-formed steel decks shall be in accordance with this section.

2210.1.1.1 Noncomposite steel floor decks. Noncomposite steel floor decks shall be permitted to be designed and constructed in accordance with ANSI/SDI-NC1.0.

2210.1.1.2 Steel roof deck. Steel *roof decks* shall be permitted to be designed and constructed in accordance with ANSI/SDI-RD1.0.

2210.1.1.3 Composite slabs on steel decks. Composite slabs of concrete and steel deck shall be permitted to be designed and constructed in accordance with SDI-C.

2210.2 Seismic requirements for cold-formed steel structures. Where a response modification coefficient, R, in accordance with ASCE 7, Table 12.2-1, is used for the design of cold-formed steel structures, the structures shall be designed and detailed in accordance with the requirements of AISI S100, ASCE 8, or, for cold-formed steel special-bolted moment frames, AISI S400.

CHAPTER 35
REFERENCED STANDARDS

AISI

American Iron and Steel Institute
25 Massachusetts Avenue, NW Suite 800
Washington, DC 20001

AISI S100—16(2020) w/S2—20: North American Specification for the Design of Cold-Formed Steel Structural Members, 2016 Edition (Reaffirmed 2020), with Supplement 2, 2020 Edition
1604.3.3, 1905.1.8, 2202.1, 2203.1, 2210.1, 2210.2

AISI S202—20: Code of Standard Practice for Cold-formed Steel Framing, 2020
2211.1.3.1

AISI S220—20: North American Standard for Cold-Formed Steel Nonstructural Framing
2202.1, 2203.1, 2211.2, Table 2506.2, Table 2507.2

AISI S230—2019: Standard for Cold-formed Steel Framing—Prescriptive Method for One- and Two-family Dwellings, 2019
1609.1.1, 1609.1.1.1, 2211.1.2

AISI S240—20: North American Standard for Cold-Formed Steel Structuring Framing, 2020
2202.1, 2203.1, 2211.1, 2211.1.1.1, 2211.1.3.3, Table 2506.2, Table 2507.2, Table 2603.12.1, Table 2603.12.2

AISI S400—20: North American Standard for Seismic Design of Cold-formed Steel Structural Systems, 2020
2210.2, 2211.1.1.1, 2211.1.1.2

8—17: Standard Specification for the Design of Cold-formed Stainless Steel Structural Members
1604.3.3, 2210.1, 2210.2

IBC CFS light-frame design provisions

SECTION 2211
COLD-FORMED STEEL
LIGHT-FRAME CONSTRUCTION

2211.1 Structural framing. For cold-formed steel *light-frame construction*, the design and installation of the following structural framing systems, including their members and connections, shall be in accordance with AISI S240, and Sections 2211.1.1 through 2211.1.3, as applicable:

1. Floor and roof systems.
2. Structural walls.
3. Shear walls, strap-braced walls and diaphragms that resist in-plane lateral loads.
4. Trusses.

2211.1.1 Seismic requirements for cold-formed steel structural systems. The design of cold-formed steel *light-frame construction* to resist seismic forces shall be in accordance with the provisions of Section 2211.1.1.1 or 2211.1.1.2, as applicable.

2211.1.1.1 Seismic Design Categories B and C. Where a response modification coefficient, *R*, in accordance with ASCE 7, Table 12.2-1 is used for the design of cold-formed steel *light-frame construction* assigned to *Seismic Design Category* B or C, the *seismic force-resisting system* shall be designed and detailed in accordance with the requirements of AISI S400.

Exception: The response modification coefficient, *R*, designated for "Steel systems not specifically detailed for seismic resistance, excluding cantilever column systems" in ASCE 7, Table 12.2-1, shall be permitted for systems designed and detailed in accordance with AISI S240 and need not be designed and detailed in accordance with AISI S400

2211.1.1.2 Seismic Design Categories D through F. In cold-formed steel *light-frame construction* assigned to *Seismic Design Category* D, E or F, the *seismic force-resisting system* shall be designed and detailed in accordance with AISI S400.

AISI CFS light-frame terminology

AISI S240-20-C
Commentary on the North American Standard for Cold-Formed Steel Structural Framing, 2020 Edition

A1 Scope and Applicability

In 2015, the provision that the Standard applies to applications where the specified minimum *base steel thickness* is not greater than 0.1180 inches (2.997 mm) was replaced with the provision that the Standard applies to *light-frame construction* applications, and a definition for the term *light-frame construction* was added to the Standard. *Cold-formed steel structural members* for *light-frame construction* applications are available in a variety of thicknesses. In 2020, standard thicknesses for *cold-formed steel structural members* previously defined in AISI S201 (AISI, 2017) were incorporated in AISI S240.

AISI S240-20
North American Standard For Cold-Formed Steel Structural Framing, 2020 Edition

A2 Definitions

Light-Frame Construction. Construction where the vertical and horizontal structural elements are primarily formed by a system of repetitive *cold-formed steel* or wood framing members.

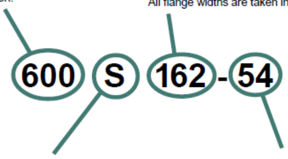
CFS framing terminology

SSMA Catalog

SFIA Catalog

Member Depth:
(Example: 6″ = 600 × 1/100 inches)
All member depths are taken in 1/100 inches.
For all "T" sections member depth is the inside to inside dimension.

Flange Width:
(Example: 1 5/8″ = 1.625″ = 162 × 1/100 inches)
All flange widths are taken in 1/100 inches.



Style:
(Example: Stud or Joist section = S)
The four alpha characters utilized by the designator system are:
S = Stud or Joist Sections
T = Track Sections
U = Channel Sections
F = Furring Channel Sections

Material Thickness:
(Example: 0.054 in. = 54 mils; 1 mil = 1/1000 in.)
Material thickness is the minimum base metal thickness in mils. Minimum base metal thickness represents 95% of the design thickness.

AISI S240

AISI S100 Specification

S100-12 to S100-16 reorganization

AISI S100-12 *(2015 IBC reference)*

A. General Provisions
B. Elements
C. Members
D. Structural Assemblies and Systems
E. Connections and Joints
F. Tests for Special Cases
G. Design for Fatigue

Appendix 1, Dsn. of CFS Structural Mbrs. using Direct Strength Method
App. 2, Second-Order Analysis
App. A, Provisions Applicable to the U.S. and Mexico
App. B, Provisions Applicable to Canada

AISI S100-16 *S100 new format, similar to AISC 360*

A. General Provisions
B. Design Requirements
C. Design for Stability
D. Members in Tension
E. Members in Compression
F. Members in Flexure
G. Members in Shear and Web Crippling
H. Members under Combined Forces
I. Assemblies and Systems
J. Connections and Joints
K. Strength for Special Cases
L. Design for Serviceability
M. Design for Fatigue

Appendix 1, Effective Width of Elements
App. 2, Elastic Buckling Analysis of Mbrs.
App. A, Provisions for U.S. and Mexico
App. B, Provisions for Canada

AISI S100 scope & applicability

A1 Scope, Applicability, and Definitions

A1.1 Scope

This *Specification* applies to the design of *structural members* cold-formed to shape from carbon or low-alloy steel sheet, strip, plate, or bar not more than 1 in. (25.4 mm) in *thickness* and used for *load*-carrying purposes in

(a) Buildings, and
(b) Structures other than buildings provided allowances are made for dynamic effects.

A3.1 Applicable Steels

This section shall apply to steels that are based on specifications providing mandatory mechanical properties and requiring test reports to confirm those properties.

Steels used in *structural members,* decks, and *connections* shall follow uses and restrictions outlined in this section and sub-sections, as applicable.

Exception: Requirements for steels used in composite slabs shall be in accordance with the *applicable building code*.

> **User Note:**
> Design of composite steel floor deck is governed by the *applicable building code* and standards published by the Steel Deck Institute (www.sdi.org).

Applicable steels have been grouped by their minimum elongation requirements over a two-inch (50-mm) gage length.

A3.1.1 Steels With a Specified Minimum Elongation of Ten Percent or Greater (Elongation ≥ 10%)

Steel grades listed below, as well as any other steel for structural applications, are permitted to be used without restriction under the provisions of this *Specification* provided:

S100-12 vs. S100-16 section reference

Section Numbering Comparison — AISI S100-12 Versus AISI S100-16

AISI S100-12 Section Numbers	Section Title	AISI S100-16 Section Numbers
A.	**GENERAL PROVISIONS**	A.
A1	Scope, Applicability, and Definitions	A1
A1.1	Scope	A1.1
A1.2	Applicability	A1.2
A1.3	Definitions	A1.3
A1.4	Units of Symbols and Terms	A1.4
A2	Material	A3
A2.1	Applicable Steels	A3.1
A2.1.1	Steels With a Specified Minimum Elongation of Ten Percent or Greater (Elongation ≥ 10%)	A3.1.1
A2.1.2	Steels With a Specified Minimum Elongation From Three Percent to Less Than Ten Percent (3% ≤ Elongation < 10%)	A3.1.2
A2.1.3	Steels With a Specified Minimum Elongation of Less Than Three Percent (Elongation < 3%)	A3.1.3
A2.2	Other Steels	A3.2
A2.3	Permitted Uses and Restrictions of Applicable Steels	A3.1
A2.3.1	Steels With a Specified Minimum Elongation of Ten Percent or Greater (Elongation ≥ 10%)	A3.1.1
D.	**STRUCTURAL ASSEMBLIES AND SYSTEMS**	I
D1	Built-Up Sections	I1
D1.1	Flexural Members Composed of Two Back-to-Back C-Sections	I1.1
D1.2	Compression Members Composed of Two Sections in Contact	I1.2
D1.3	Spacing of Connections in Cover-Plated Sections	I1.3
D2	Mixed Systems	I3
D3	Lateral and Stability Bracing	C2
D3.1	Symmetrical Beams and Columns	C2.1
D3.2	C-Section and Z-Section Beams	C2.2
D3.2.1	Neither Flange Connected to Sheathing That Contributes to the Strength and Stability of the C- or Z- Section	C2.2.1
D3.3	Bracing of Axially Loaded Compression Members	C2.3
D4	Cold-Formed Steel Light-Frame Construction	I4
D4.1	All-Steel Design of Wall Stud Assemblies	I4.1
D5	Floor, Roof, or Wall Steel Diaphragm Construction	I2
D6	Metal Roof and Wall Systems	I6
D6.1	Purlins, Girts and Other Members	I6.2

AISI S240 and S400 CFS Framing Standards

AISI S240-20

AISI S240: North American Standard for Cold-Formed Steel Structural Framing *(S240 first adopted into 2018 IBC)*

- **Consolidated** into single standard in 2015:
  - **AISI S200:** General Provisions
  - **AISI S210:** Floor and Roof System Design
  - **AISI S211:** Wall Stud Design
  - **AISI S212:** Header Design
  - **AISI S213:** Lateral Design
  - **AISI S214:** Truss Design

S240 & S400 provides performance provisions for framing assemblies

A1.2.3 *Cold-formed steel* framing for floor and roof systems, and structural walls, as listed in items A1.2.1(a) and A1.2.1(b), are also permitted to be designed solely in accordance with AISI S100 [CSA S136].

S200 series vs. S240-15 section reference

x AISI S240-15

Section Reference Between AISI S240 and AISI S200, S210, S211, S212, S213, and S214

AISI S240 Section	Title	Source Standard	Section
	Buckling		
B4.2.2.2	Distortional Buckling	new	n/a
B4.2.3	Bending	new (editorial)	n/a
B4.2.3.1	Lateral-Torsional Buckling	S210	B1.4.2
B4.2.3.2	Distortional Buckling	new	n/a
B4.2.4	Shear	S210	B1.4.3
B4.2.5	Web Crippling	S210	B1.4.4
B4.2.6	Axial Load and Bending	S210	B1.4.5
B4.2.7	Bending and Shear	S210	B1.4.6
B4.2.8	Bending and Web Crippling	S210	B1.4.7
B4.3	Roof Truss Design	S210	B2
B4.4	Bearing Stiffeners	S210	B3.1
B4.5	Bracing Design	S210	B4
B4.6	Roof Diaphragm Design	S210	B5
B5	Lateral Force-Resisting Systems	new (editorial)	n/a
B5.1	Scope	new (editorial)	n/a
B5.2	Shear Wall Design	S213	C
B5.2.1	General	S213	C1
B5.2.1.1	Type I Shear Walls	S213	C2
B5.2.1.2	Type II Shear Walls	S213	C3
B5.2.2	Nominal Strength [Resistance]		
B5.2.2.1	Type I Shear Walls	S213	C2
B5.2.2.2	Type II Shear Walls	S213	C3
B5.2.3	Available Strength [Factored Resistance]	S213	C2
B5.2.4	Collectors and Anchorage	S213	C3.3
B5.2.5	Design Deflection	S213	C2.1.1

S240-20 scope and applicability

A1.1 Scope

This Standard applies to the design, manufacture, installation, and quality of *structural members* and *connections* utilized in *cold-formed steel light-frame construction* applications.

A1.2 Applicability

A1.2.1 The design and installation of *cold-formed steel* framing for the following systems shall be in accordance with AISI S100 [CSA S136] and this Standard:

(a) floor and roof systems in buildings,
(b) structural walls in buildings,
(c) *shear walls*, *strap braced walls* and *diaphragms* to resist in-plane lateral *loads*, and
(d) *trusses* for *load*-carrying purposes in buildings.

A1.2.2 *Cold-formed steel structural members* and *connections* in *seismic force-resisting systems* and *diaphragms* shall be designed in accordance with the additional provisions of AISI S400 in the following cases:

(a) In the United States and Mexico, in *seismic design categories* (SDC) D, E, or F, or wherever the *seismic response modification coefficient*, R, used to determine the seismic design forces is taken other than 3.
(b) In Canada, where the design spectral response acceleration S(0.2) as specified in the NBCC is greater than 0.12 and the seismic force modification factors, R_dR_o, used to determine the seismic design forces, are taken as greater than or equal to 1.56.

A1.2.3 *Cold-formed steel* framing for floor and roof systems, and structural walls, as listed in items A1.2.1(a) and A1.2.1(b), are also permitted to be designed solely in accordance with AISI S100 [CSA S136].

A1.2.4 This Standard shall govern over other standards, including those referenced in this Standard, in matters pertaining to elements falling within the scope of this Standard, as defined in Section A1.1. Where conflicts between this Standard and the *applicable building code* occur, the requirements of the *applicable building code* shall govern. In areas without an *applicable building code*, this Standard defines the minimum acceptable standards for elements falling within the scope of this Standard, as defined in Section A1.1.

A1.2.5 This Standard does not preclude the use of other *approved* materials, assemblies, structures or designs of equivalent performance.

A1.2.6 This Standard includes Chapters A through F and Appendix 1 in their entirety.

S240 definitions

A2 Definitions, A2.1 Terms

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

Chord. Member of a *shear wall*, *strap braced wall* or *diaphragm* that forms the perimeter, interior opening, discontinuity or re-entrant corner.

Chord Stud. Axial *load-bearing studs* located at the ends of *Type I shear walls* or *Type II shear wall segments* or *strap braced walls*.

Collector. Also known as a drag strut, a member parallel to the applied *load* that serves to transfer forces between *diaphragms* and members of the *lateral force-resisting* system or distributes forces within the *diaphragm*.

Lateral Force-Resisting System. The structural elements and *connections* required to resist racking and overturning due to wind forces or seismic forces, or other predominantly horizontal forces, or combination thereof, imposed upon the structure in accordance with *the applicable building code*.

Required Strength. Forces, stresses, and deformations produced in a structural component, determined by either structural analysis, for the *LRFD* or *ASD load combinations*, as appropriate, or as specified by this Standard.

Seismic Force-Resisting System. That part of the structural system that has been selected in the design to provide energy dissipation and the required resistance to seismic forces prescribed in the *applicable building code*.

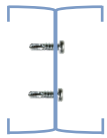
S240 built-up section design

B1.3 Built-Up Section Design

Built-up sections shall be evaluated in accordance with Section I1 of AISI S100 [CSA S136] and the additional requirements of Sections B1.3.1 and B1.3.2, as applicable.

B1.3.1 For either all steel design or sheathing braced design, the *available strength* [*factored resistance*] of built-up sections shall be determined in accordance with Section I1.2 of AISI S100 [CSA S136].

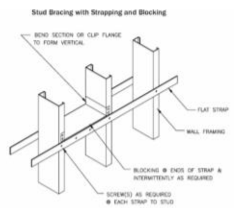
Exception: Where a built-up axial *load* bearing section comprised of two *studs* oriented back-to-back forming an I-shaped cross-section is seated in a *track* in accordance with the requirements of Section C3.4.3 and the top and bottom end bearing detail of the *studs* consists of support by steel or concrete components with adequate strength and stiffness to preclude relative end slip of the two built-up *stud* sections, the compliance with the end *connection* provisions of AISI S100 Section I1.2(b) is not required.



Courtesy: Dr. Roger

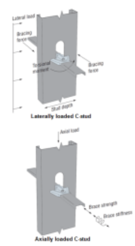
S240 wall stud bracing

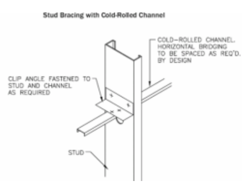
Wall Stud Bracing: Steel Based Design



Strap & Blocking

2016 AISI Design Guide (D110-16) *Bracing Design Examples*





Channel & Clip Angle

Model No.	Stud Depth (in.)	Stud Thickness mil (ga.)	Laterally Loaded C-Stud: Allowable Torsional Moment (in.-lbs.)		Axially Loaded C-Stud: Allowable Brace Strength (lbs.)		Brace Stiffness (lbs. / in.)	
			Min.	Max.	Min.	Max.	Min.	Max.
SUBH3.25	3.625	33 (20)	320	345	230	370	1,450	1,985
		43 (18)	355	430	255	420	2,780	4,035
		54 (16)	420	455	290	475	2,925	3,975
MSUBH3.25		54 (16)	550	800	435	630	3,440	4,015
		68 (14)	640	860	485	695	4,040	6,145
		97 (12)	670	860	515	770	6,860	14,265
SUBH3.25	6.000	33 (20)	275	385	110	110	605	605
		43 (18)	295	525	230	250	1,050	1,205
		54 (16)	350	550	275	415	1,130	1,700
MSUBH3.25		54 (16)	565	895	385	430	1,630	1,695
		68 (14)	655	925	455	620	1,860	2,655
		97 (12)	690	960	505	765	4,070	4,090
SUBH3.25		43 (18)	255	570	190	190	505	535

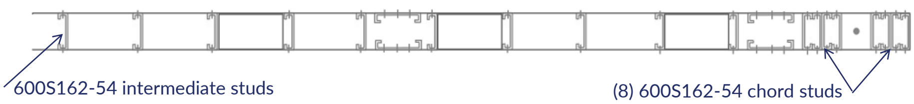
Proprietary Solutions

S240 wall stud bracing

Wall Stud Bracing: Steel Based Design

AISI S240 B3.4.1: Intermediate Brace Design

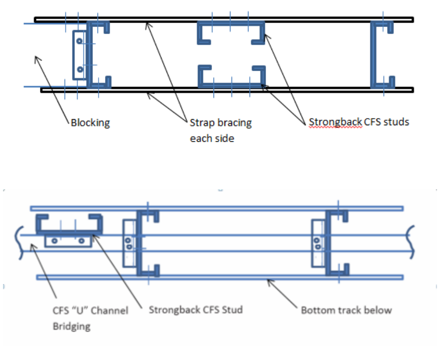
- For **axial** loaded members, each intermediate brace shall be designed for 2% of the design compression load in the member.
- For **combined bending and axial** loads, each intermediate brace shall be designed for the combined brace force determined by C2.2.1 of S100 **and** 2% of design compression force in member.
- Brace forces are **accumulative between anchorage points** (S240 Comm. Section B3.4).

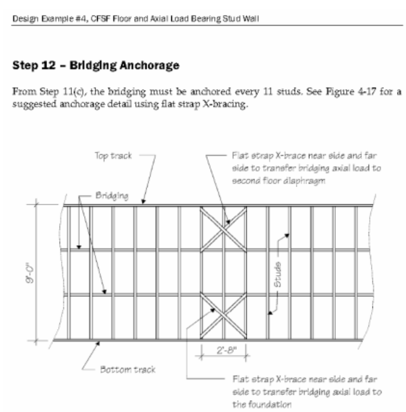


S240 wall stud bracing

Wall Stud Flange Bracing: Steel Based Design

Bridging Anchorage





Design Example #4, CFSF Floor and Axial Load Bearing Stud Wall

Step 12 – Bridging Anchorage

From Step 11(c), the bridging must be anchored every 11 studs. See Figure 4-17 for a suggested anchorage detail using flat strap X-bracing.

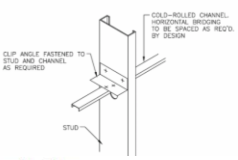
AISI S110 Figure

S240 wall stud bracing

Wall Stud Flange Bracing: Steel Based Design

Design using either AISI S240 **or** S100

Stud Bracing with Cold-Rolled Channel



AISI S100-16

C2.3 Bracing of Axially Loaded Compression Members

The *required brace strength* [brace force due to *factored loads*] and stiffness are permitted to be determined by a *second-order analysis* in accordance with the requirements of Section C1.

Alternatively, to provide an adequate intermediate brace (or braces) that will allow an individual concentrically loaded compression member to develop its *required axial strength* [compressive axial force due to *factored loads*], the *required strength* [brace force due to *factored loads*] acting on the brace (or braces) shall be calculated in accordance with Eq. C2.3-1.

$$\overline{P}_{rb} = 0.01\ \overline{P}_{ra} \qquad (Eq.\ C2.3\text{-}1)$$

where

$\overline{P}_{rb}$ = *Required brace strength* [brace force due to *factored loads*] to brace a single compression member with an axial *load* $\overline{P}_{ra}$

$\overline{P}_{ra}$ = *Required compressive axial strength* [compressive axial force due to *factored loads*] of individual concentrically loaded compression member to be braced, which is calculated in accordance with *ASD*, *LRFD*, or *LSD load combinations* depending on the design method used

The stiffness of each brace shall equal or exceed β_{rb}, as calculated in Eq. C2.3-2:

For *ASD*

$$\beta_{rb} = \frac{2[4-(2/n)]}{L_b}(\Omega \overline{P}_{ra}) \qquad (Eq.\ C2.3\text{-}2a)$$

Ω = 2.00

S240 wall stud bracing

AISI S240 Section B1.2.2.1(b): Sheathing Braced Design

B1.2.2 Wall Studs

B1.2.2.1 Wall *studs* shall be designed either on the basis of all steel design or on the basis of sheathing braced design, in accordance with the following:

(a) All Steel Design. Wall *stud* assemblies using all steel design shall be designed neglecting the structural *bracing* and composite-action contribution of the attached sheathings.

(b) Sheathing Braced Design. Wall *stud* assemblies using the sheathing braced design provisions of this Section shall have *structural sheathing* attached to both *flanges* of the wall *stud* or *structural sheathing* attached to the one *flange* and discrete bracing to the other *flange*. The *studs* shall be spaced no greater than 24 inches (610 mm) on center and be connected to the bottom and top *track* or other horizontal member(s) of the wall to provide lateral and torsional support to the wall *stud* in the plane of the wall. Wall *studs* with sheathing attached to both sides that is not identical shall be designed based on the assumption that the weaker of the two sheathings is attached to both sides.

B1.2.2.2 When sheathing braced design is used, the *construction documents* shall identify the *structural sheathing* as a structural element.

B1.2.2.3 For *curtain wall studs*, the combination of *structural sheathing* attached to one side of the wall *stud* and discrete bracing for the other *flange* is permitted. The spacing of discrete bracing shall be no greater than 8 ft (2.44 m) on center and the *studs* shall not be spaced greater than 24 inches (610 mm) on center. For design, the *nominal flexural strength* [*resistance*] shall be determined by Chapter F of AISI S100 [CSA S136]. When the compression *flange* has *structural sheathing* attached, the *available flexural strength* [*factored resistance*] is permitted to be the lesser of the strength determined in accordance with Section F3 with $F_n = F_y$ or $M_{ne} = M_y$ and the strength determined in accordance with Section F4 of AISI S100 [CSA S136], where the contribution of the sheathing rotational stiffness, determined in accordance with Appendix 1, is permitted.

B1.2.2.4 In the United States and Mexico, when sheathing braced design is used, the wall *studs* shall also be evaluated without the sheathing *bracing* for the following *load* combination:

$$1.2D + (0.5L \text{ or } 0.2S) + 0.2W \qquad (Eq.\ B1.2.2\text{-}1)$$

*S240 explicitly permits the use of sheathing bracing for one stud flange **and** discrete bracing (steel) for the other. **[New in 2015]***

AISI S400-20

AISI S400: North American Standard for Seismic Design of Cold-Formed Steel Structural Systems
(S400 first adopted into 2018 IBC)

- **Consolidated** into single standard in 2015:
  - **AISI S110:** Special Bolted Moment Frame
  - **AISI S213:** Lateral Design (*seismic provisions only*)

S400 scope and applicability

A1 Scope and Applicability

A1.1 Scope

This *Standard* is applicable for the design and construction of *cold-formed steel structural members* and *connections* in *seismic force-resisting systems* and *diaphragms* in buildings and *other structures*.

A1.2 Applicability

A1.2.1 This *Standard* shall be applied in conjunction with AISI S100 [CSA S136], AISI S240 and the *applicable building code*.

A1.2.2 In the absence of an *applicable building code*, the design requirements shall be in accordance with accepted engineering practice for the location under consideration as specified by the applicable sections of ASCE 7 in the United States and Mexico, or the *National Building Code of Canada* (NBCC) in Canada.

A1.2.3 In the United States and Mexico, in *Seismic Design Categories* B or C and where the *seismic response modification coefficient*, R, used to determine the seismic design forces is taken equal to 3, the *cold-formed steel structural members* and *connections* in *lateral force-resisting systems* need only be designed in accordance with AISI S100 or AISI S240, as applicable. In Canada, where the *seismic force modification factors*, R_dR_o, used to determine the seismic design forces, are taken as less than 1.56 or the design spectral response acceleration S(0.2) as specified in the NBCC is less than or equal to 0.12, the *cold-formed steel structural members* and *connections* in *lateral force-resisting systems* need only be designed in accordance with CSA S136 or AISI S240, as applicable.

User Note:
This *Standard* intends to exempt *lateral force-resisting system* only where the *Seismic Design Category* is B or C and the *seismic response modification coefficient*, R, equals 3. ASCE 7, Table 12.2-1, Line H exempts these steel systems from seismic detailing requirements in this *Standard* as long as they are designed in accordance with AISI S240 or AISI S100, as applicable. For *Seismic Design Category* A, it is not necessary to define a *seismic force-resisting system* that meets any special requirements and this *Standard* does not apply.

S400 definitions

A2 Definitions, A2.1 Terms

Capacity-Based Design. Design of *lateral force-resisting systems* according to capacity design principles to resist the maximum anticipated seismic loads.

User Note:

Capacity design principles for design of a *seismic force-resisting system* include all of the following: a) specific elements or mechanisms are design to dissipate energy; b) all other elements are sufficiently strong for this energy dissipation to be achieved; c) structural integrity is maintained; d) elements and *connections* in the horizontal and vertical load paths are designed to resist these seismic loads and corresponding principal and companion loads as defined by the NBCC; e) *diaphragms* and *collector* elements are capable of transmitting the *loads* developed at each level to the vertical *seismic force-resisting system*; and f) these *loads* are transmitted to the foundation. [Canada]

Designated Energy Dissipating Mechanism. Selected portion of the *seismic force-resisting system* designed and detailed to dissipate energy.

Lateral Force-Resisting System. The structural elements and *connections* required to resist racking and overturning due to wind forces or seismic forces, or other predominantly horizontal forces, or combination thereof, imposed upon the structure in accordance with the *applicable building code*.

Required Strength. Forces, stresses, and deformations produced in a *structural component*, determined by either structural analysis, for the *LRFD* or *ASD load combinations*, as appropriate, or as specified by this Standard.

Seismic Force-Resisting System. That part of the structural system that has been selected in the design to provide energy dissipation and the required resistance to seismic forces prescribed in the *applicable building code*.

S400 expected strength

A3.2 Expected Material Properties

A3.2.1 Material Expected Yield Stress [Probable Yield Stress]

Where required in this *Standard*, the expected strength [probable resistance] of a *connection* or *structural member* shall be determined using the expected yield stress [probable yield stress], R_yF_y, with R_y given in Table A3.2-1, unless otherwise modified in Chapter E and Chapter F.

Values of R_y, other than those listed in Table A3.2-1, are 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 requirements for the specified grade of steel in Section A3.1.

Table A3.2-1
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 Grade B	1.4	1.3
A500 Grade C	1.3	1.2
A1085	1.25	1.15
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

R_t Ratio of expected tensile strength and *specified minimum tensile strength*

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

A3.2.2 Material Expected Tensile Strength [Probable Tensile Strength]

Where required in this *Standard*, the expected strength [probable resistance] of a *connection* or *structural member* shall be determined using the expected tensile strength [probable tensile strength], R_tF_u with R_t given in Table A3.2-1, unless otherwise modified in Chapter E and Chapter F.

S400 WSP shear wall provisions

E1 CFS Light Frame Shear Walls Sheathed with Wood Structural Panels

- **E1.1 Scope**
- **E1.2 Basis of Design**
  - E1.2.1 Designated Energy-Dissipating Mechanism
  - E1.2.2 Seismic Design Parameters [Seismic Force Modification Factors and Limitations]
  - E1.2.3 Type I or Type II Shear Walls
  - E1.2.4 Seismic Load Effects Contributed by Masonry and Concrete Walls
- **E1.3 Shear Strength**
  - E1.3.1 Nominal Strength [Resistance]
  - E1.3.2 Available Strength [Factored Resistance]
  - E1.3.3 Expected Strength [Probable Resistance]
- **E1.4 Systems Requirements**
  - E1.4.1 Type I Shear Walls
  - E1.4.2 Type II Shear Walls

S400 DEDM & expected strength

E. SEISMIC FORCE-RESISTING SYSTEMS

E1 Cold-Formed Steel Light Frame Shear Walls Sheathed With Wood Structural Panels

E1.1 Scope

Cold-formed steel light frame *shear walls* sheathed with *wood structural panels* rated for shear resistance shall be designed in accordance with the requirements of this section.

E1.2 Basis of Design

Cold-formed steel light frame *shear walls* sheathed with *wood structural panels* are expected to withstand seismic demands primarily through deformation in the *connection* between the *wood structural panel* sheathing and the *cold-formed steel structural members.*

E1.2.1 Designated Energy-Dissipating Mechanism *[New in 2015]*

The *structural member*-to-sheathing *connection* and the *wood structural panel* sheathing itself are the *designated energy-dissipating mechanism* in this system.

E1.3.3 Expected Strength [Probable Resistance] *[Revised in 2020]*

The expected strength [probable resistance] ($\Omega_E V_n$) shall be determined from the *nominal strength* [*resistance*] in accordance with this section.

In the United States and Mexico, the expected strength factor, Ω_E, for *shear walls* sheathed with *wood structural panels* shall be:

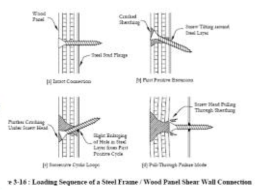
$$\Omega_E = \frac{1.1 v_n + v_{finish}}{v_n} \leq 1.8 \qquad (Eq.\ E1.3.3\text{-}1)$$

where

v_n = *Nominal shear strength* per unit length as specified in Table E1.3-1, lb/ft (kN/m)

v_{finish} = Mean shear strength per unit length of the wall finish system applied to the *shear wall*, not permitted to be less than $0.1v_n$

Expected strength load typically not less than overstrength load combinations



Courtesy: Boudreault, 2005. Report on shear wall testing at McGill University, Montreal

S400 shear wall required strength

E1.4.1.2 Capacity Protected Components

Collectors, chord studs, other vertical *boundary elements*, *hold-downs* and anchorage connected thereto, and all other *components* and *connections* of the *shear wall* that are not part of the *designated energy-dissipating mechanism* are *capacity protected components*.

B3 Design Basis

The *available strength* [*factored resistance*] of the designated *seismic force-resisting system* shall be greater than or equal to the *required strength* [effects of *factored loads*] determined from the applicable *load* combinations. To ensure the performance of the designated *seismic force-resisting system*, *capacity protected componenents* shall be designed as follows:

For the United States and Mexico, the *required strength* of *capacity protected components* shall be determined from the *expected strength* of the *seismic force-resisting system*, but need not exceed the *load effect* determined from the applicable *load* combinations including seismic *load* with overstrength. The *available strength* of the capacity protected components shall be greater than or equal to the *required strength*.

[New in 2020]

[New in 2015]

User Note:
In the United States and Mexico, per Section F2.3, the *diaphragm chords* and *diaphragms* are required to be designed for the *loads from the applicable building code* (without consideration of expected strength) and the *collectors* are required to be designed for the expected strength of the *seismic force-resisting system* but need not exceed the seismic *load* effect, including overstrength.

Type 1 shear wall deflection

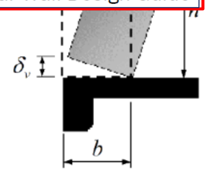
Flexure + Shear + Non-linear Effects + Hold-down

$$\delta = \frac{2vh^3}{3EA_c b} + \omega_1\omega_2\frac{vh}{\rho G t_{sheathing}} + \omega_1^{5/4}\omega_2\omega_3\omega_4\left(\frac{v}{\beta}\right)^2 + \frac{h}{b}\delta_v \qquad (Eq.\ E1.4.1.4\text{-}1)$$

Deflection of a blocked wood or sheet steel shear wall eq. variables:

The above calculations account for all of the deflection contributions provided in AISI S400, Eq. E1.4.1.4-1. However, rotation of the Level 2 shear wall caused by the Level 1 shear wall deformations will add to the total lateral deflection of the Level 2 shear. The lower level chord axial deformations and hold-down vertical deflections should all be considered.

AISI D113-19, Cold-Formed Steel Shear Wall Design Guide



$t_{sheathing}$ = Nominal panel thickness
t_{stud} = Stud designation thickness
v = Shear demand
δ_v = Vertical deformation of anchorage/attachment details
β = 67.5 Plywood, 55 OSB, and 29.12($t_{sheathing}$/0.018) sheet steel
ρ = 1.85 Plywood, 1.05 OSB, 0.075($t_{sheathing}$/0.018)
ω_1 = s/6
ω_2 = 0.033/t_{stud}
ω_3 = ((h/b)/2)^1/2
ω_4 = 1 for wood sheathing, (33/F_y)^1/2

S400 diaphragm provisions

F. Diaphragms

F1 General

- F1.1 Scope
- F1.2 Design Basis
- F1.3 Required Strength
  - F1.3.1 Diaphragm Stiffness
  - F1.3.2 Seismic Load Effects Including Overstrength
- F1.4 Shear Strength
  - F1.4.1 Nominal Strength *(By prin. of mech. or tabulated)*
    - F1.4.1.1 Diap. Sheathed with WSPs
    - F1.4.1.2 Diap. Sheathed with Profiled Steel Panels
  - F1.4.2 Available Strength

F2 CFS Diaphragms Sheathed with WSPs

- F2.1 Scope
- F2.2 Additional Design Requirements
  - F2.2.1 Seismic Detailing Requirements
  - F2.2.2 Seismic Load Effects Contributed by Masonry and Concrete Walls
- F2.3 Required Strength
  - F2.3.1 Diaphragm Stiffness
- F2.4 Shear Strength
  - F2.4.1 Nominal Strength *(Table F2.4-1)*
  - F2.4.2 Available Strength
  - F2.4.3 Design Deflection *(S240 Eq. B5.4.2.4-1)*
- F2.5 Requirements Where R is Greater Than Three

F3 Bare Steel Deck Diaphragms

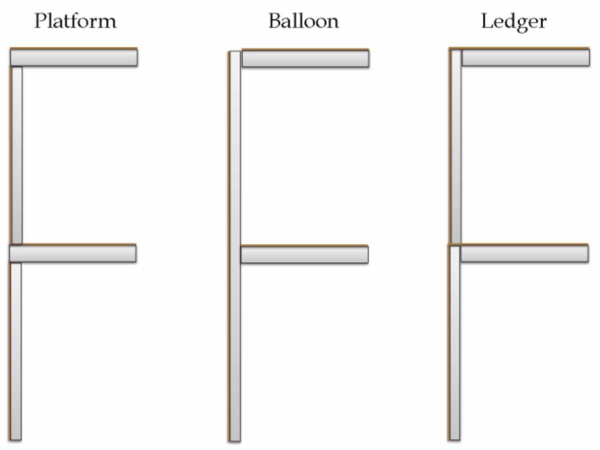
[New in 2020]

Table F2.4-1
Nominal Shear Strength (v_n) per Unit Length for Diaphragms Sheathed With Wood Structural Panel Sheathing [1, 2]
***United States and Mexico* (lb/ft)**

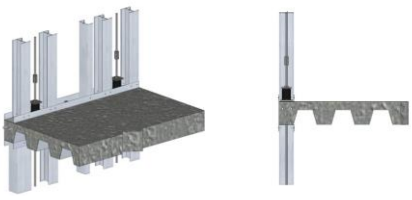
Sheathing	Thickness (in.)	Blocked: Screw spacing at diaphragm boundary edges and at all continuous panel edges (in.) 6 / Screw spacing at all other panel edges (in.) 6	4 / 6	2.5 / 4	2 / 3	Unblocked (Screws spaced maximum of 6 in. on all supported edges): Load perpendicular to unblocked edges and continuous panel joints	All other configurations
Structural I	3/8	768	1022	1660	2045	685	510
	7/16	768	1127	1800	2255	755	565
	15/32	925	1232	1970	2465	825	615
C-D, C-C and other graded wood structural panels	3/8	690	920	1470	1840	615	460
	7/16	760	1015	1620	2030	680	505
	15/32	832	1110	1770	2215	740	555

CFS Light-Frame Construction

CFS Framing Types



CFS Framing Types

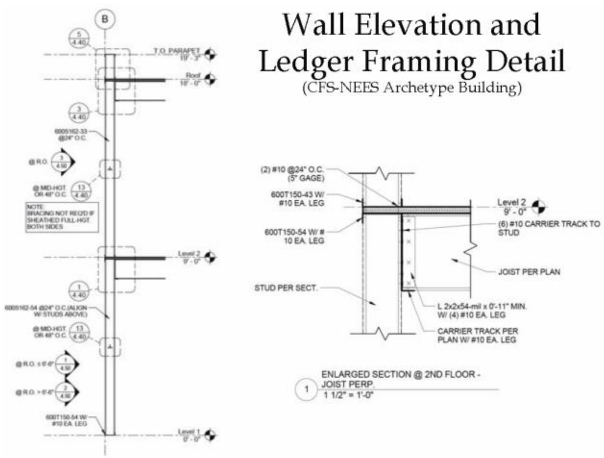


Platform Framed

Platform Framed

Ledger Framed

CFS Framing Types



Courtesy Dr. Schafer, Johns Hopkins

CFSF SW: Design Method

General Design Procedure:

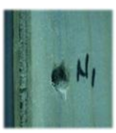
1. Determine design **loads** (gravity, wind, seismic, lateral earth pressure, etc.)
2. Determine **shear wall type/ fasteners** based on published shear wall shear strength determined using AISI S240 or S400.
3. Size **boundary members** and **supporting elements** of the shear wall.
4. Determine the **overturning** and **shear restraint** required.
5. Check the story **drift** and adjust the design as required.
6. Design the **collector** to shear wall connection
7. Select **hold-down steel anchor rod material and diameter** per AISI S240 or S400 *(amplified seismic load < available strength for R ⁄ 3)* and design conc. anchorage per ACI 318 Chap. 17.

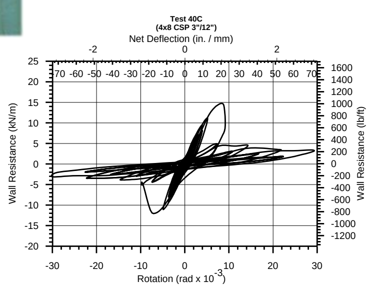
CFSF SW: Sheathing

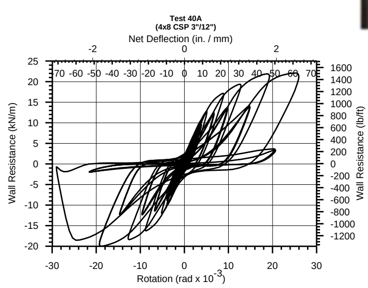
- Mechanical fasteners attaching sheathing to framing members. Sheathing types:
  - **Wood structural panel** (WSP) *[R = 6.5, Max. A.R. 4:1]*
  - **Steel sheet** *[R = 6.5, Max. A.R. 4:1]*
  - **Gypsum** *[R = 2.0, SDC A-D, Max. A.R. 2:1]*
  - **Fiberboard** *[R = 2.0, SDC A-D, Max. A.R. 1:1]*



CFSF SW: Fasteners







≈ 50% of sheathing screws overdriven

100% sheathing screws flush

Effect of overdriving sheathing fasteners (3/8 in. plywood)

Courtesy C.A. Rogers, McGill University, Montréal

CFSF SW: Fasteners

C4.1.4 Overdriven Screws in Shear Walls and Diaphragms Sheathed With Wood Structural Panels

Designation of overdriven screw fasteners in *shear walls* and *diaphragms* sheathed with *wood structural panels* shall be in accordance with Section C4.1.4.1. Remediation of overdriven screw fasteners shall be made in accordance with Section C4.1.4.2. Where all overdriven screws are not remediated, the calculated *nominal strength* [*nominal resistance*] shall be determined in accordance with Section C4.1.4.3.

C4.1.4.1 A screw fastener shall be deemed overdriven where the flat outer surface of the head is driven below the surface of the panel beyond 1/8 in. (3.18 mm) for a nominal panel thickness of 7/16 in. (11.1 mm) or greater, or beyond 1/16 in. (1.59 mm) for a nominal panel thickness less than 7/16 in. (11.1 mm).

C4.1.4.2 Overdriven screw fasteners are permitted to be remediated by removal and replacement with a screw with a larger head diameter. Unscrewing an overdriven screw fastener until it is no longer considered overdriven in accordance with Section C4.1.4.1 is not permitted.

C4.1.4.3 For *shear walls* or *diaphragms* with overdriven screw fasteners, the calculated *nominal strength* [*nominal resistance*] shall be determined in accordance with either Section C4.1.4.3.1 or Section C4.1.4.3.2, as applicable.

C4.1.4.3.1 No strength reduction for overdriven fasteners is required where all the following criteria are met:

(1) In any of the four corners of an individual *wood structural panel*, none of the three fasteners closest to a panel's corner are overdriven;

(2) No more than 10% of fasteners along the perimeter, excluding the panel corner fasteners, are overdriven; and

(3) No more than 20% of fasteners in the field of the panel are overdriven.

C4.1.4.3.2 The *nominal shear strength* [*nominal resistance*] shall be multiplied by 0.75 where all the following criteria are met:

(1) In any of the four corners of an individual *wood structural panel*, none of the three fasteners closest to a panel's corner are overdriven;

(2) No more than 25% of fasteners along the perimeter, excluding the panel corner fasteners, are overdriven; and

(3) No more than 50% of fasteners in the field of the panel are overdriven.

AISI S240 C4.1.4

[New in 2020]

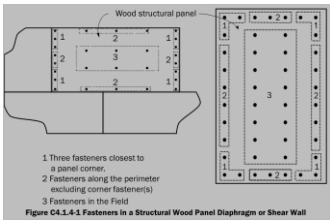
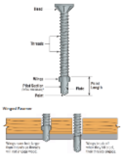


Figure C4.1.4-1 Fasteners in a Structural Wood Panel Diaphragm or Shear Wall

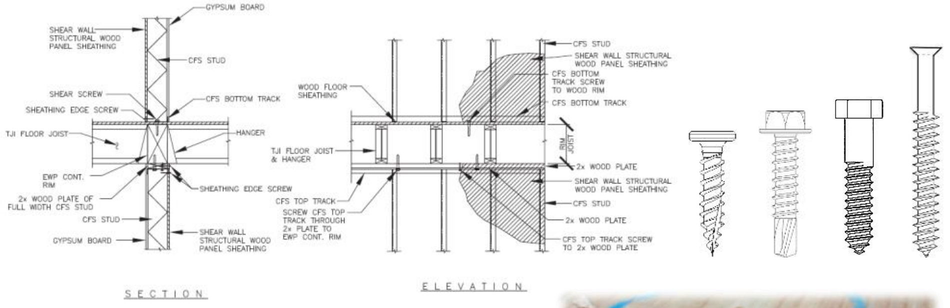
CFSF SW: Fasteners

Full-scale wood sheathed CFS framed shear wall tests comparing assemblies

- 2 in. and 6 in. on center panel edge fastener spacing
- Winged versus non-winged tipped self-tapping screws
- 20% design shear strength reduction



CFSF SW: Fasteners



Screw attaching CFS track through wood sheathing into wood rim joist:

- Pre-drill track and use wood screw or lag screw
- NDS values do not apply to self-drilling tapping screw as drill tip removes more material and will lead to additional movement than typical wood screw or lag screw

CFSF SW: Fasteners

CFSF SW: Overturning

Dead Load Resisting Shear Wall Overturning

There are three methods designers use to determine the amount of dead load along the shear wall that can be used to counter the shear-wall uplift force.

1. Neglect the dead load for conditions when it is minimal (example: floor framing parallel to shear wall), which is the most conservative approach for the uplift side, but not the compression side of the shear wall.

2. Consider the shear wall as a rigid body where the dead load is used to resist the uplift and used as part of the gravity load demand in addition to the overturning compression load. Some justify this using a deep-beam analogy.

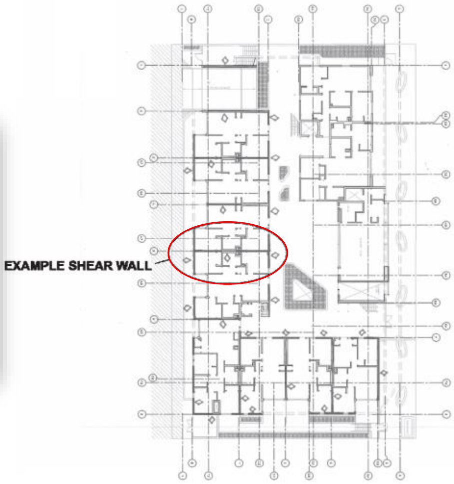
3. Consider the rim and top track as a beam on an elastic foundation. Such an analysis shows that typically only the dead load within a few stud bays can be mobilized to resist uplift. Further discussion can be found in FEMA 451 Chapter 10 as well as in the book *Structural Design of Low-Rise Buildings*, Section 6.7.

This design example uses the rigid body method although good arguments could be made in support of the "beam on elastic foundation" approach. Therefore, the dead load of and tributary to the wall is used to resist the uplift, and also it is used as part of the gravity-load demand to the chord-stud assembly on the compression side of the shear wall.

Shear Wall Design Example

SW Dsn. Ex.: Location and dimensions

Three-Story Light-Frame Multifamily Building Design Using Cold-Formed Steel Wall Framing and Wood Floor and Roof Framing



Building B

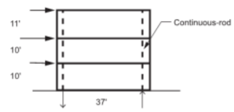


Figure 3-14. Option 1: Full-length, stacked shear wall

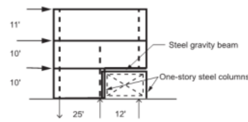


Figure 3-15. Option 2: Short first-floor shear wall

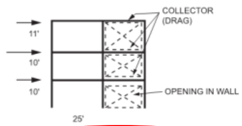


Figure 3-16. Option 3: Short, stacked shear wall

SW Dsn. Ex.: Parameters

One interior three-story, stacking shear wall in Building B, Zone B1 has been selected for this design example. The CFS-framed shear wall design will follow the requirements of AISI S240 and S400.

Shear wall parameters:

1. Classification: Type I.
2. Wall sheathing: Wood structural panels (DOC PS-1 and PS-2): plywood or OSB.
3. Wall studs: CFS studs (54 mil [16 gauge]): 600S162-54 (F_y = 50 ksi) and (68 mil [14 gauge]): 600S200-68 (F_y = 50 ksi).
4. Continuous rod tie-down (hold-down) system: Resists tension (uplift) from overturning and comprises continuous rods, coupler nuts, nuts, bearing plates, take-up devices, cripple studs, and bridge blocks.
5. Chord studs: Resist compression from gravity and overturning; CFS studs.
6. Wall base connection: Elevated floors: screws.
   Podium level: anchor bolts in a 12-inch-thick structural concrete slab.

Table 3-21. Shear wall design information (LRFD)

Level	Tributary Area to Shear Wall Line (ft^2)	Story Level Design Force *v* (psf)	Shear Wall Story Forces (lb)	Σ Shear Wall Force (lb)	ASD Forces (0.7F)
3rd Floor	(23 ft 8 in) × 40 ft = 947	10.4	947 × 10.4 = 9849	9849	6895
2nd Floor	(23 ft 8 in) × 45 ft = 1065	11.4	1065 × 11.4 = 12,141	21,990	15,393
1st Floor	(23 ft 8 in) × 45 ft = 1065	5.7	1065 × 5.7 = 6071	28,061	19,643

SW Dsn. Ex.: Assembly Selection

Table 3-22. Shear wall strengths

Wall Sheathing	Fastener Spacing at Panel Edges (inches) 6	4	3	2	Stud Thickness (mils)	Sheathing Screw Size
Table E1.3-1: Nominal Shear Strength (pounds per foot)—Sheathing One Side Only						
15/32 Structural I (4 ply)	890	1330	1775	2190	54/68	8/10
Table E1.3-1: Available Strength Design Values (pounds) $\phi = 0.6$						
15/32 Structural I (4 ply)	534	798	1065	1314	54/68	8/10
Table E1.3-1: Available ASD values (pounds) $\Omega = 2.5$						
15/32 Structural I (4 ply)	356	532	710	876	54/68	8/10

Table 3-25. Option 3: Short-length shear walls each floor level (L = 25 feet)

	LRFD				ASD			
Level	Σ Shear Wall Force[1] (F) (lb)	Shear (F/L) (plf)	Design Capacity[2] (plf)	#8 or #10 Screw Spacing (in)	Σ Shear Wall Forces ASD = (0.7F) (lb)	Shear (F/L) (plf)	Design Capacity[2] (plf)	#8 or #10 Screw Spacing (in)
3rd Floor	9849	394	534	6	6895	276	356	6
2nd Floor	21,990	880	1065	3	15,393	616	710	3
1st Floor	28,061	1123	1314	2	19,643	786	876	2

1. Shear wall forces are from Table 3-21.
2. Design capacities are from Table 3-22.

SW Dsn. Ex.: Chord Studs & OT Res.

S400 Sections E1.4.1.2 and E2.4.1.2 require that the shear wall CFS chord studs (vertical boundary members) and the uplift anchorage have the available strength to resist the amplified seismic load (required strength). For ease of reference in this design example, the term amplified seismic load will mean the lesser of

1. The load determined from the ASCE 7 seismic load combinations with the overstrength factor Ω_0.
2. The expected strength factor, Ω_E, for WSP-sheathed shear walls per AISI S400 Section E1.3.3 is determined from the following equation. This design example uses Ω_E equal to 1.8.

$$\Omega_E = \frac{1.1v_n + v_{\text{finish}}}{v_n} \leq 1.8 \qquad \text{AISI S400 E1.3.3-1}$$

where

v_n = *Nominal shear strength* per unit length as specified in Table E1.3-1, lb/ft (kN/m)

v_{finish} = Mean shear strength per unit length of the wall finish system applied to the shear wall, not permitted to be less than $0.1v_n$

Since the floors and roof diaphragms are considered to be flexible in this example, the overstrength factor, Ω_0, is equal to 2.5 per ASCE 7 Table 12.2-1 Footnote b.

The available strength of the collectors, chord studs, other vertical boundary elements, hold-downs and anchorage connected to the shear wall, and all other components and connections of the shear wall shall be greater than or equal to the amplified seismic load (required strength).

SW Dsn. Ex.: Chord Studs & OT Res.

ASCE 7 Section 12.4.3 Seismic Load Effects, including Overstrength; Sections 2.3.6 and 2.4.5 Basic Combinations with Seismic Load Effects

Only a portion of the dead load can be used for resisting shear wall overturning forces, and for light-frame wood-frame construction, the overturning resisting forces would be based on Section 2.3.6 equations.

ASCE 7 Sections 2.3.6 and 2.4.5

LFRD: $0.9D - E_v + E_{mh}$

$[(0.9 - 0.2S_{DS})D + \rho Q_E + 1.6H] \rightarrow (0.9 - 0.2S_{DS})D = [0.9 - 0.2(1.117)]D = 0.676D$

ASD: $0.6D - 0.7E_v + 0.7E_{mh}$

$[(0.6 - 0.14S_{DS})D + 0.7\rho Q_E + H] \rightarrow (0.6 - 0.14S_{DS})D = [0.6 - 0.14(1.117)]D = 0.444D$

Per AISI S400 Section E1.4.1.2, the chord studs and uplift anchorage must have the available strength to resist the lesser of (1) the expected strength of the shear wall or (2) the load determined using the ASCE 7 seismic load combinations, including the overstrength factor, Ω_0.

ASCE 7 Section 12.4.3

LRFD: $[(0.9 - 0.2S_{DS})D + \Omega_0 Q_E + 1.6H] \rightarrow (0.9 - 0.2S_{DS})D = [0.9 - 0.2(1.117)]D = 0.676D$

ASD: $[(0.6 - 0.14S_{DS})D + 0.7(\Omega_0 Q_E) + H] \rightarrow (0.6 - 0.14S_{DS})D = [0.6 - 0.14(1.117)]D = 0.444D$

In either case, the dead load used to resist the overturning is the same, but the uplift loads will be more significant when the overstrength factor, Ω_0, is included or uplift forces are derived using the expected shear strength of the shear walls.

SW Dsn. Ex.: Chord Studs & OT Res.

Table 3-32. Option 3 shear wall (wall length = 25 ft)

	Resisting Dead Load Moments (RM)				
Level	Wall height (h) (ft)	Σ Dead Load (W_{TD}) (plf)	Σ Shear Wall Resisting Moment (RM) (ft-lb)	LRFD 0.676RM (ft-lb)	ASD 0.444RM (ft-lb)
3rd Floor	11	153	47,813	32,321	21,229
2nd Floor	10	617	192,813	130,341	85,609
1st Floor	10	1081	337,813	228,361	149,989

1. Resisting moment = $(W_{TD})(L \times L)/2 = (153)(25)(25)/2 = 47{,}813$ lb.

W_{TD} = Total wall and floor weight (Table 3-37)

Table 3-35A. Option 3 shear wall

	Amplified Uplift Forces (LRFD)				
Level	Wall Length (L) (ft)	Wall Length (L_E) (ft)	Σ Shear Wall Ω_0 OTM (ft-lb)	0.676RM (ft-lb)	Story-Level Uplift (lb)
3rd Floor	25	23	270,848	32,321	10,371
2nd Floor	25	23	820,598	130,341	30,011
1st Floor	25	23	1,522,123	228,361	56,251

1. Uplift = $(\text{OTM} - 0.676\text{RM})/(L_E) = (270{,}848 - 32{,}321)/23 = 10{,}371$
2. $L_E = L$ Effective = 25 ft – 1 ft – 1 ft = 23 ft

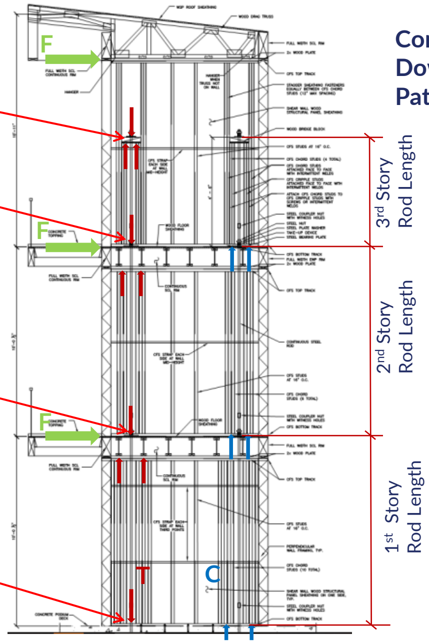
SW Dsn. Ex.: OT Restraint (T)

3rd story OT restraint
Bridge Block Assembly to Rod

2nd story OT restraint
Bearing Plate to Rod

1st story OT restraint
Bearing Plate to Rod

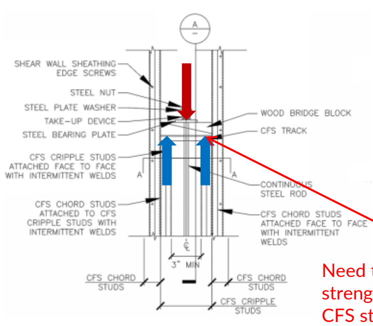
OT Restraint System
Anchorage



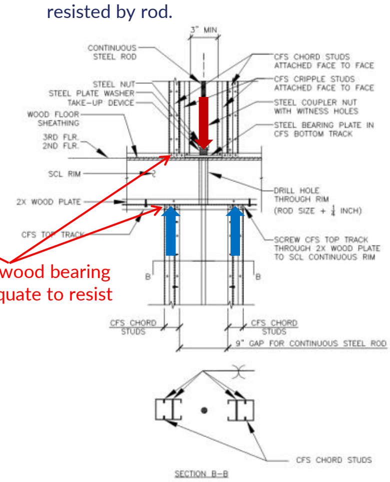
Continuous Rod Tie-Down System Load Path

SW Dsn. Ex.: OT Restraint (T)

Continuous Rod Tie-Down System



Bearing Plate Load Path: Sheathing fasteners (for SW below bearing plate) to chord studs to rim to bearing plate and resisted by rod.



Need to check wood bearing strength is adequate to resist CFS stud load

Bridge Block Load Path: Sheathing fasteners (for SW at story bridge block is located) to chord studs to cripple studs to bridge block to bearing plate and resisted by rod.

Figure 3-21. Typical chord-stud assembly of the floor line. Continuous rod tie-down (hold-down) system chord studs at the second and third floors.

SW Dsn. Ex.: OT Restraint (T)

Continuous Rod Tie-Down Design – Steel Rod Selection

- Continuous rod tie-down systems designed for strength, but also designed to ensure code drift compliance
- ICC-ES AC316 limit is 0.20" max. vertical displacement between restraints (includes rod elongation and take-up device deflection)
- Steel rod selection governed by strength in this example

Table 3-36. Continuous rod tie-down system required, provided strength, and rod sizes (Option 3: Shear wall)—LRFD

Level	Floor-to-Floor Height (L) (ft)	Story-Level Amplified Seismic Uplift Force[(1)] (P_{amp}) (lb)	Rod Gross Area Required[(4)]/Area Provided (A_b) for Strength (in^2)	Story-Level Seismic Uplift Force[(2)] (P) (lb)	Rod Net Tensile Area Required[(3)]/Area Provided (A_e) for Vertical Story Displacement[(6)] (in^2)	Rod Diameter Provided (in) (rod tensile strength [ksi])
3rd Floor	11	10,371	0.318/0.442	3305	0.063/0.334	¾ (F_u = 58 ksi)
2nd Floor	10	30,011	0.920/0.994	8604	0.150/0.763	1⅛ (F_u = 58 ksi)
2nd Floor	10	30,011	0.427/0.442	8604	0.150/0.334	¾ (F_u = 125 ksi)
1st Floor	10	56,251	0.800/0.994	16,542	0.288/0.763	1⅛ (F_u = 125 ksi)

1. Story-level amplified seismic uplift force is from Table 3-35A.
2. Story-level seismic uplift force is from Table 3-35B.
3. Rod material: ASTM A36 $F_y = 36$ ksi $F_u = 58$ ksi, $E = 29{,}000{,}000$
   ASTM A193 B7 $F_y = 105$ ksi $F_u = 125$ ksi, $E = 29{,}000{,}000$
4. Rod available strength (LRFD) = $0.75(0.75F_u)A_b$, so $A_{req} = P_{amp}/(0.56F_u)$ from AISC 360 Equation J3-1, Table J.3.2, and corresponding commentary.
5. System vertical story displacement limit = 0.170 = 0.20 inch − 0.030 inch take-up device initial seating, Δ_R, and design deflection, Δ_A, per manufacturer's evaluation report (ASD) × 1.4 = 0.170 × 1.4 = 0.238 inch (LRFD). System vertical story displacement limit = δ = 0.238 inch.

$$\delta = PL/A_{req}E$$

$$A_{req} = PL/0.238(E)$$

6. Rod net tensile area tabulated in the 15th Edition of AISC *Steel Construction Manual* Table 7-17 or per the following equation: $A_e = 0.7854 \times [d - (0.9743/n)]^2$ where n = threads per inch and d = rod diameter.

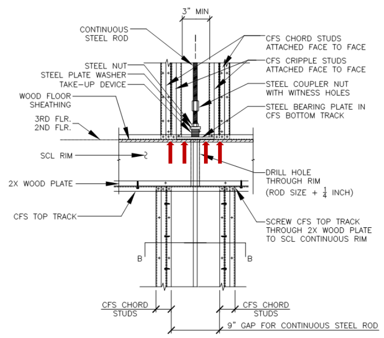
SW Dsn. Ex.: OT Restraint (T)

Tie-Down System Steel Bearing Plate Design

1. Determine bearing plate work space (e.g., between studs and track flanges)
2. Determine bearing plate size based on wood bearing strength
3. Determine bearing plate thickness based on its steel flexural and shear strength

Wood Bearing:

- Wood is not designed using the amplified seismic load, thus, size bearing plate based on wood bearing using ASD or LRFD seismic demand.
- Wood bearing based on average compression resistance at 0.04" of deformation. Divide nominal load by 1.67 for ASD or multiply for 0.9 for LRFD, per AWC NDS Tables 2.3.5 and 2.3.6.

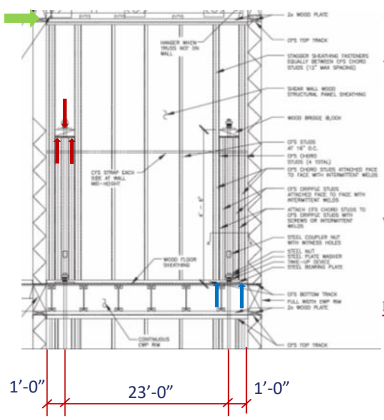


SW Dsn. Ex.: OT Restraint (T)

Tie-Down System Steel Bearing Plate Design & Detailing

- Multi-story CFS framed projects often use ledger framing
- Bearing plates designed to span stud gap
- Bearing plates should extend full width of stud and full length of stud pack. Coordination with panelizer important!
- Bearing plates often are 1 ½” to 1 ¾” thick so track flanges need to be taller for screw to track connection
- Some use thicker studs to align floor-to-floor
- Bearing plate dapped into wood diaphragm requires shear transfer & plate might be thicker than sheathing

SW Dsn. Ex.: Chord Studs (C)



3rd Story Shear Wall

Chord-Stud Assembly Compression Loads

The design forces shown are for the Option 3 shear wall (LRFD).

Roof-Level Design Loads (Third-Floor Shear Walls)

Wall gravity-compression loads:

Dead load = 153 plf

Live load = 0 plf (roof live load not used in load combinations)

Load combination = $(1.2 + 0.2(S_{DS}))D + 0.5L$ — ASCE 7 Eq 6 §2.3.6

= [1.2 + 0.2(1.117)](153) + 0.5(0) = 218 plf

Chord stud boundary gravity load = (25-ft-long wall/2)(218 plf) = 2725 lb

Assumed chord stud boundary width = 2 ft

Wall seismic forces (amplified seismic loads – compression loads):

OTM (including the seismic overstrength factor Ω_0) = 270,848 ft-lb

Lever arm = centerline of hold-down to centerline of hold-down = 25 ft – 2(1 ft) = 23 ft

$C = T$ = 270,848/23 = 11,776 lb

Expected shear wall strength – compression loads:

Wall length = 25 ft, #8 screws at 6 in o.c., $V_{nominal} = V_n$ = 890 plf, Ω_E = 1.8

OTM = (25 ft)(890)(1.8)(11 ft)/1000 = 440.55 kip-ft

Lever arm = centerline of hold-down to centerline of hold-down = 25 – 2(1 ft) = 23 ft

C = 440,550/23 = 19,154 lb > 11,776 lb; therefore, use 11,776 lb

SW Dsn. Ex.: Chord Studs (C)

Table 3-38. Option 3 shear wall: summary of CFS chord stud differential uplift and accumulative downward compression design loads

Level	OTM Differential Uplift Loads (kips)	Load Type[1]	OTM Compression Downward Loads (kips)	Load Type[2]	Gravity Loads (kips)
3rd Floor	10.371	amplified differential	11.776	amplified	2.725
2nd Floor	19.640	amplified differential	35.679	amplified	13.938
1st Floor	26.240	amplified differential	66.180	amplified	25.138

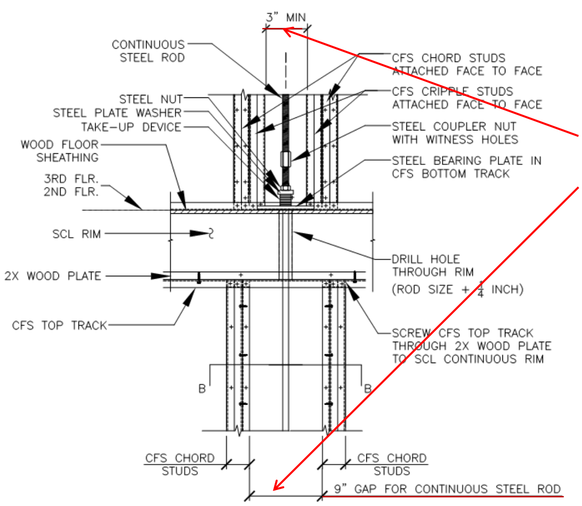
1. The uplift loads are the amplified differential story loads from Table 3-37.
2. The downward compression design loads are the smaller of the calculated overstrength and expected loads.

5 Stud Bracing: Example uses bracing mid-point at top 2 floors and third points at first floor.

Table 3-39. Option 3 shear wall: CFS chord stud size, number, and strength

Level	Stud Height[1] (ft)	Individual Chord Stud Size	Available Stud Strength[5] (kips)	Number of Chord Studs	Available Chord Assembly Axial Strength (kips)	Amplified LRFD[2] Axial Design Load (kips)	Chord Stud Assembly Width[4]
3rd Floor	10	600S162-54(50)	4.48	4	17.92	14.489	(8 × 1.625 in) + 3 in = 16 in
2nd Floor	9	600S200-68(50)	9.34	6	56.0	49.617	(6 × 2.0 in) + 9 in = 21 in
1st Floor	9	600S200-68(50)	11.1	10	110.8	91.318	(10 × 2.0 in) + 9 in = 29 in

SW Dsn. Ex.: Chord Studs (C)



Gap between studs to be adequate to install uplift restraint system, including continuous rod, coupler nut, and take-up device.

SW Dsn. Ex.: Stud Bracing

Steel Stud Steel Bracing Systems

U-Channel bridging through stud web punchout attached to CFS stud with connector

a. Requires coordination with building elements in stud bay
b. Installs using one side of wall
c. Does not bump out sheathing
d. Bracing for axial loaded studs requires periodic anchorage to structure (e.g., strongbacks, diagonal strap bracing, etc.)
e. Bracing of laterally loaded studs do not require periodic anchorage (system in equilibrium as torsion in stud resisted by U-channel bending)

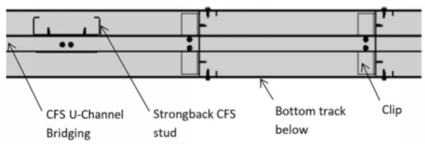


Figure 3-27. CFS stud-wall U-channel bridging—strongback anchorage (plan view)

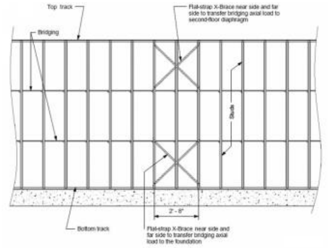


Figure 3-29. CFS stud-wall flat-strap bracing—diagonal strap bracing anchorage (elevation view)

SW Dsn. Ex.: Stud Bracing

Steel Stud Steel Bracing Systems

Flat strap bracing installed on each face of the CFS stud

a. May be installed at other locations than stud punchout
b. Required on both sides of wall
c. Bumps out sheathing
d. Bracing for axial loaded studs requires periodic anchorage to structure (e.g., strongbacks, diagonal strap bracing, etc.) (same load direction in stud flanges)
e. Bracing for laterally loaded studs requires design of periodic blocking or periodic anchorage to structure (opposite load direction in stud flanges)

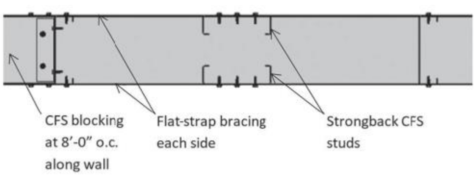


Figure 3-28. CFS stud-wall flat-strap bracing and blocking—strongback anchorage (plan view)

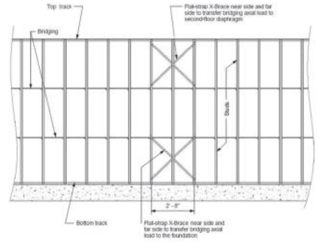
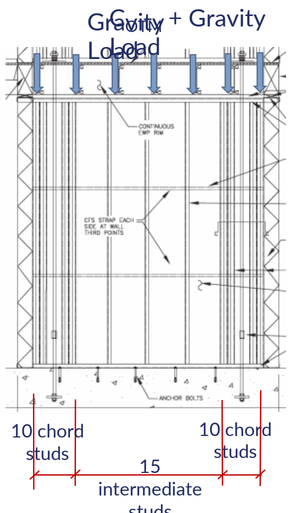


Figure 3-29. CFS stud-wall flat-strap bracing—diagonal strap bracing anchorage (elevation view)

SW Dsn. Ex.: Stud Bracing



1st Story Shear Wall
NTS

Bracing for axially loaded compression studs

- S240 Section B3.4 requires the brace strength to be 2% of axial load on stud.
- Axial bracing force is cumulative; periodic anchorage required.

Two Axial Load Cases:

1. Bracing force from CFS compression chord studs with the amplified seismic load and the CFS intermediate studs for half the wall length with the LRFD gravity load.
2. Bracing force from all the studs with the LRFD gravity load along the entire wall length.

SW Dsn. Ex.: Stud Bracing

Case 1: Determining CFS stud compressive load using seismic load combinations with Ωo

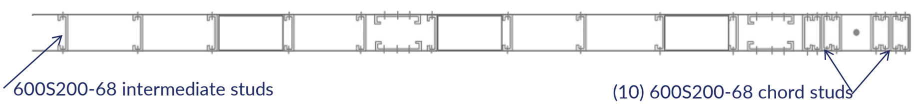
Chord stud pack (10 - 600S200-68 x 9'-0") amplified LRFD axial load	= 91.3 kips	(Table 3-39)
Intermediate stud LRFD axial load = 2011 plf x 16" o.c. / 12"/ft	= 2.68 kips	(Pg. 222)

Strap-to-stud connection design force

$P_{br,2\ chord}$ = 0.02P = 0.02(91.3 kips / 10 chord studs)
= 0.183 kips / 2 straps (each side of stud) = 0.092 kips

$P_{br,2\ inter}$ = 0.02P = 0.02(2.68 kips) = 0.054 kips / 2 straps (each side of stud) = 0.027 kips

- Attach CFS strap to CFS stud flange with #8 self-drilling tapping screw with minimum LRFD shear strength of 114 lbs at individual chord studs and 27 lbs at intermediate studs.



SW Dsn. Ex.: Stud Bracing

Case 1: Determining CFS stud compressive load using seismic load combinations with Ωo

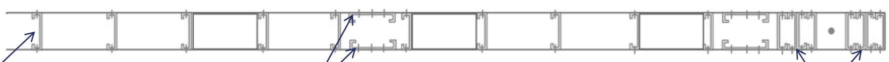
Strongback bracing system anchorge design force

P_{anch} = (0.092 kips x 10 chord studs) + (0.027 kips x 8 intermediate studs in ½ of shear wall)
= 1.136 kips

$P_{anch\ sb}$ = 1.136 kips / 4 pairs of strongback locations along wall length
= 0.284 kips at 1/3 pt. bracing height of 3 ft and 6 ft

Horizontal Straps
- Designed for LRFD tension load of 284 lbs
- Use 1 ½" wide x 33 mil (20 ga) strap (Fy=33 ksi) with (2) #8 screws to strongback stud each side

Strongback studs
- Designed unbraced for full hgt. of 9'-0" with point loads from bracing at 3 ft and 6 ft (no axial load)
- Attached to top and bottom track with clip angles
- Use (4) pairs of 800S162-54 CFS strongback studs near chord studs and then equally spaced



SW Dsn. Ex.: Deflection

ASCE 7 Table 12.12-1 limits the story drift for Risk Category II three-story buildings to $0.025h_{sx}$, where h_{sx} is the story height. This three-story light-frame building is considered to comply with the Table 12.12-1 requirement that interior walls, partitions, ceilings, and exterior walls have been designed to accommodate the story drifts. If it did not, or the number of stories was five stories or more, the story drift would be limited to $0.020h_{sx}$.

The shear wall deflection (δ) is calculated per ASCE 7 Equation 12.8-15:

$$\delta_x = C_d \delta_{xe} / I_e$$

Shear wall deflection calculation:

First story: chord studs A_{Gross}, Ac = (10) – 600S200-68 = 10 × 0.7643 in² = 7.643 in² AISI D100 TI-2

Second B story: chord studs A_{Gross}, Ac = (6) – 600S200-68 = 6 × 0.7643 in² = 4.586 in² AISI D100 TI-2

Second A story: chord studs A_{Gross}, Ac = (6) – 600S200-68 = 6 × 0.7643 in² = 4.586 in² AISI D100 TI-2

Third story: chord studs A_{Gross}, Ac = (4) – 600S162-54 = 4 × 0.5560 in² = 2.224 in² AISI D100 TI-2

Structural I (4 ply, 32/16) plywood sheathing shear modulus per AISI S400 Commentary Section E1.4.1.4 and AWC SDPWS Commentary Table C4.2.3A:

$$G = G_v t_v / t = (45{,}500 \text{ lb/in})/0.469 \text{ in} = 97{,}015 \text{ psi}$$

SW Dsn. Ex.: Deflection

Table 3-46. Option 3 shear wall: deflection variables

Floor Level	Wall Height, h (in)	Wall Width, b (in)	LRFD Required Strength, v (lb/in)	LRFD Story Level Uplift Force, P (lb)	End Studs Gross Area, A_c (in^2)	Sheathing Thickness, t (in)	Sheathing Shear Modulus, G (psi)	Rod Net Tensile Area (A_e)(in^2)/ LRFD Rod Elongation(2,3) d_{v1} (in)	Take-Up Device Deflection(4) d_{v2} (in)
3rd	132 (54)(1)	300	32.8	3305	2.224	0.469	97,015	0.334/0.018	0.030
2nd A	120	300	73.3	8604	4.586	0.469	97,015	0.763/0.046	0.030
2nd B	120	300	73.3	8604	4.586	0.469	97,015	0.334/0.107	0.030
1st	120	300	93.5	16,542	7.643	0.469	97,015	0.763/0.088	0.030

See notes for Table 3-47.

Table 3-47. Option 3 shear wall: deflection variables

Floor Level	CFS Modulus of Elasticity, E_s (psi)	Fastener Spacing, s (in)	Framing Designation Thickness t_{stud} (in)	Sheathing Factor, ρ	Fastener Spacing Factor, $\omega_1 = s/6$	Framing Designation Thickness Factor, $\omega_2 = 0.033/t_{stud}$	Aspect Ratio Factor, $\omega_3 = ((h/b)/2)^{0.5}$	Sheathing Material Factor, ω_4
3rd	29.5E6	6	0.054	1.85	1.00	0.611	0.469	1
2nd A	29.5E6	3	0.068	1.85	0.50	0.485	0.447	1
2nd B	29.5E6	3	0.068	1.85	0.50	0.485	0.447	1
1st	29.5E6	2	0.068	1.85	0.33	0.485	0.447	1

Notes for Table 3-47:

1. Rod length at third-floor bridge block = 4 ft 6 in = 54 in.
2. Vertical rod elongation = $\delta_{v1} = PL/A_eE$.
3. Modulus of elasticity (E) for tie rod = 29,000,000 psi.
4. Reference manufacturer's evaluation report evaluated to ICC-ES AC316 for take-up device initial (seating increment) deflection, Δ_R, and allowable load deflection, Δ_A, to determine the total device deflection, Δ_T, which for this design example is δ_{v2}. $\Delta_T = \delta_{v2} = \Delta_R + \Delta_A (P_D/P_A)$. Shrinkage and estimated building settlement are to be taken as part of the deflection computation if a take-up device is not used.
5. Total vertical deformation of uplift anchorage system = δ_v = rod elongation (δ_{v1}) + total take-up device deflection (δ_{v2}) + stud bearing crushing at wood floor + bearing plate crushing at wood floor. The wood crushing terms are not shown in this design example, but should be added.

SW Dsn. Ex.: Deflection

Table 3-46. Option 3 shear wall: deflection variables

Floor Level	Wall Height, h (in)	Wall Width, b (in)	LRFD Required Strength, v (lb/in)	LRFD Story Level Uplift Force, P (lb)	End Studs Gross Area, A_c (in^2)	Sheathing Thickness, t (in)	Sheathing Shear Modulus, G (psi)	Rod Net Tensile Area (A_e)(in^2)/ LRFD Rod Elongation(2,3) d_{v1} (in)	Take-Up Device Deflection(4) d_{v2} (in)
3rd	132 (54)(1)	300	32.8	3305	2.224	0.469	97,015	0.334/0.018	0.030

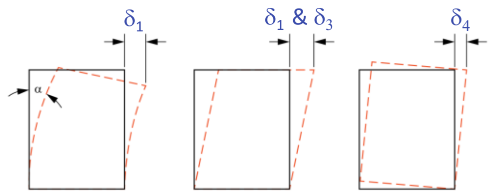
4. Reference manufacturer's evaluation report evaluated to ICC-ES AC316 for take-up device initial (seating increment) deflection, Δ_R, and allowable load deflection, Δ_A, to determine the total device deflection, Δ_T, which for this design example is δ_{v2}. $\Delta_T = \delta_{v2} = \Delta_R + \Delta_A (P_D/P_A)$. Shrinkage and estimated building settlement are to be taken as part of the deflection computation if a take-up device is not used.
5. Total vertical deformation of uplift anchorage system = δ_v = rod elongation (δ_{v1}) + total take-up device deflection (δ_{v2}) + stud bearing crushing at wood floor + bearing plate crushing at wood floor. The wood crushing terms are not shown in this design example, but should be added.

2nd B	29.5E6	3	0.068	1.85	0.50	0.485	0.447	1
1st	29.5E6	2	0.068	1.85	0.33	0.485	0.447	1

Notes for Table 3-47:

1. Rod length at third-floor bridge block = 4 ft 6 in = 54 in.
2. Vertical rod elongation = $\delta_{v1} = PL/A_eE$.
3. Modulus of elasticity (E) for tie rod = 29,000,000 psi.
4. Reference manufacturer's evaluation report evaluated to ICC-ES AC316 for take-up device initial (seating increment) deflection, Δ_R, and allowable load deflection, Δ_A, to determine the total device deflection, Δ_T, which for this design example is δ_{v2}. $\Delta_T = \delta_{v2} = \Delta_R + \Delta_A (P_D/P_A)$. Shrinkage and estimated building settlement are to be taken as part of the deflection computation if a take-up device is not used.
5. Total vertical deformation of uplift anchorage system = δ_v = rod elongation (δ_{v1}) + total take-up device deflection (δ_{v2}) + stud bearing crushing at wood floor + bearing plate crushing at wood floor. The wood crushing terms are not shown in this design example, but should be added.

SW Dsn. Ex.: Deflection



For 3rd-floor shear wall:

$$\delta_1 = \frac{2vh^3}{3E_sA_cb} = 0.0026 \text{ in}$$

$$\delta_2 = \omega_1\omega_2\frac{vh}{\rho G t_{sheathing}} = 0.0314 \text{ in}$$

$$\delta_3 = \omega_1^{5/4}\omega_2\omega_3\omega_4\left(\frac{v}{\beta}\right)^2 = 0.0677 \text{ in}, \ \beta = 67.5 \text{ for plywood sheathing}$$

AISI S400 §E1.4.1.4

$$\delta_4 = \frac{h}{b}\delta_v = \frac{132}{300}(0.018 + 0.030) = 0.0211 \text{ in}$$

There are additional contributors to the deflection of multistory shear walls that need to be taken into account, including rotation at the top of the shear wall below; axial shortening and lengthening of the compression and tension chords, respectively, of the shear wall below; and hold-down deflection of the shear walls below.

SW Dsn. Ex.: Deflection

Table 3-48. *Summary of top-of-wall deflections*

Floor Level	Flexure δ_1 (in)	Shear δ_2 (in)	Non-linear δ_3 (in)	Tie-down δ_4 (in)	Rot. of SW below δ_{rot} (in)	Chords of SW below δ_{chrds} (in)	Tie-down of SW below δ_{TDS} (in)	Total story deflection δ_{Total} (in)	Design Story Drift δ_x (in)	Allowable Story Drift Δ_a (in)	Allow. Story drift check $\delta_{Total} < \delta_x$
3rd 1	0.0026	0.0314	0.0677	0.0211	0.0055	0.0094	0.1020	0.2397	0.9587	3.3	Yes, OK
2nd A	0.0021	0.0254	0.1076	0.0304	0.0024	0.0061	0.0304	0.2043	0.8172	3.0	Yes, OK
2nd B	0.0021	0.0254	0.1076	0.0548	0.0024	0.0061	0.0472	0.2455	0.9820	3.0	Yes, OK
1st	0.0016	0.0213	0.1041	0.0472	—	—	—	0.1742	0.6970	3.0	Yes, OK

1. 3rd-floor shear wall top-of-wall deflection calculation based on 2nd B rod size.

Each line of vertical elements of the seismic-force-resisting system complies with allowable story drifts per ASCE 7 Table 12.12-1.

$\Delta_a = 0.025h_{sx} = 0.025(120 \text{ in}) = 3.0 \text{ in}$ (first and second floors)
$= 0.025(132 \text{ in}) = 3.3 \text{ in}$ (third floor)

$\delta_x = C_d\,\partial_{xe}/I_e$

$I_e = 1.0$

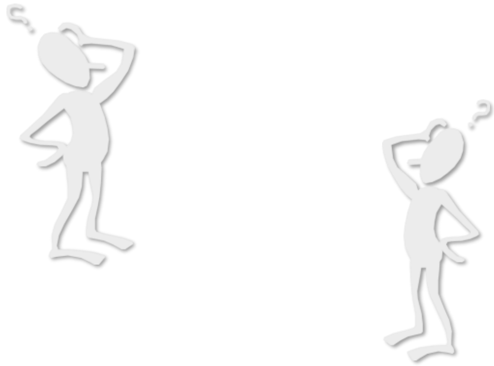
CFS vs. wood framed shear walls

1. Lateral Standard: AISI S240 and S400 rather than AWC SDPWS.
2. Shear wall & diaphragm provisions similar to wood with some differences.
3. Shear wall & diaphragm strength values similar to wood with some differences.
4. Shear wall & diaphragm deflection equations similar to wood with some differences.
5. R ≠ 3 shear walls require collector connections, chord studs, tie-downs *available strength* greater than min. of *expected strength* and seismic load combos with Ω_0.
6. Continuous rod tie-down systems offer greater strength than typical holdowns for the higher demands in multi-story construction.
7. Axially and laterally loaded studs may require intermediate bracing. Bracing force is cumulative for axially loaded studs so requires periodic anchorage to the structure.
8. Shear transfer design and detailing may be challenging for hybrid structures.
9. Design concrete anchorage for the bolt selected as required by AISI S240 and S400 (e.g., 1" dia ASTM 193 B7 (Fu=125 ksi)) and then design anchorage per ACI 318.

Code and standard summary

- **2021 IBC** references **AISI S100-16/S2-20**, **S240-20**, and **S400-20** for the design provisions for CFS structures.
- **S100-16** reformatted in a similar format as AISC 360, but most design provisions similar to those in S100-12. ***However, -16 may result in less available strength for some sections.***
- **S240** consolidated most of the previous S200 series standards for CFS structural framing, but most design provisions similar to those in the S200 series.
- **S400** consolidated S110 (CFS moment frame) and S213 (diap., strap brace and shear wall systems) into a seismic design standard, but most design provisions similar to those standards. ***However, some changes will result in larger seismic demand and members.***
- **S202** (Standard Practice), **S220** (Nonstructural Members), and **S230** Prescriptive for 1- and 2-Family Dwellings) are the other AISI standards adopted by the 2021 IBC.

Questions?



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