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North American Standard for Cold-Formed Steel Framing -- Lateral Design, 2007 Edition with Supplement No 1. October 2009 (Reaffirmed 2012)

American Iron and Steel Institute

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American Iron and Steel Institute

AISI STANDARD

North American Standard for Cold-Formed Steel Framing – Lateral Design 2007 Edition with Supplement No. 1

October 2009 (Reaffirmed 2012)

Endorsed by:

Steel Framing Alliance™

DISCLAIMER

The material contained herein has been developed by the American Iron and Steel Institute Committee on Framing Standards. The Committee has made a diligent effort to present accurate, reliable, and useful information on cold-formed steel framing design and installation. 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*.

With anticipated improvements in understanding of the behavior of cold-formed steel framing and the continuing development of new technology, this material will 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 design professional. Indeed, in many jurisdictions, such 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.

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PREFACE

The American Iron and Steel Institute Committee on Framing Standards has developed AISI S213, the *North American Standard for Cold-Formed Steel Framing – Lateral Design*, to address the design of lateral force resisting systems to resist wind and seismic forces in a wide range of buildings constructed with cold-formed steel framing. This standard is intended for adoption and use in the United States, Canada and Mexico.

This standard provides an integrated treatment of Allowable Strength Design (ASD), Load and Resistance Factor Design (LRFD), and Limit States Design (LSD). This is accomplished by including the appropriate resistance factors (ϕ) for use with LRFD and LSD, and the appropriate factors of safety (Ω) for use with ASD. It should be noted that LSD is limited to Canada and LRFD and ASD are limited to Mexico and the United States.

Changes made in Supplement No. 1 to the 2007 Edition include the following:

- R_d values in Table A4-1 for diagonal *strap* braced (concentric) walls were adjusted to match the values approved by the Canadian National Committee on Earthquake Engineering (CANCEE) for inclusion in the National Building Code of Canada (NBCC) seismic provisions.
- Language in C1.1 was modified to clarify when design must comply with the special seismic requirements.
- The existing provisions on setbacks in Section C2, which the Committee thought should be limited to prescriptive methods with defined limits of applicability, were replaced with a requirement deemed to be more appropriate for a design standard.
- Adjustments were made to Table C2.1-3 for 0.027" steel sheet, one side, based on testing at the University of North Texas (Yu, 2007). Designation thickness for stud, track and blocking associated with the existing tabulated values was increased from 33 mils (min.) to 43 mils (min.). New values were added for designation thickness for stud, track and blocking equal to 33 mils (min.).
- Equation C2.1-1 for determining the design deflection of a blocked wood structural panel or sheet steel shear wall was consolidated for US Customary and SI units.
- The word "countersunk" was deleted and commentary added to clarify provisions for tapping screws to attach wood structural panel sheathing in Section C2.2.2.
- Language in C3.3.2 was modified to clarify when the uplift anchorage and boundary chords must comply with the special seismic requirements.
- The design provisions of Section C5.3, Seismic Forces Contributed by Masonry and Concrete Walls, and Section C5.4, Seismic Forces from Other Concrete or Masonry Construction, were relocated under Section C1, General.
- A definition for *amplified seismic load* was added under Section A2, Definitions.
- Equation D2.1-1 for determining the design deflection of a blocked wood structural panel diaphragm was consolidated for US Customary and SI units.

The report referenced above, *Steel Sheet Sheathing Options for Cold-Formed Steel Framed Shear Wall Assemblies Providing Shear Resistance* (Yu, 2007), is available as a free download from the American Iron and Steel Institute (www.steel.org) and Steel Framing Alliance (www.steelframing.org).

The Committee acknowledges and is grateful for the contributions of the numerous engineers, researchers, producers and others who have contributed to the body of knowledge on the subjects. The Committee wishes to also express its appreciation for the support of the

Steel Framing Alliance and the Canadian Sheet Steel Building Institute.

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Note (1) indicates AISI S213-07 only. Note (2) indicates Supplement No. 1 only.

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NORTH AMERICAN STANDARD FOR COLD-FORMED STEEL FRAMING – LATERAL DESIGN, 2007 EDITION WITH SUPPLEMENT NO. 1 OCTOBER 2009 (REAFFIRMED 2012)

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NORTH AMERICAN STANDARD FOR COLD-FORMED STEEL FRAMING –LATERAL DESIGN

A. GENERAL

A1 Scope

The design and installation of *cold-formed steel* light-framed *shear walls,* diagonal *strap bracing* (that is part of a structural wall) and *diaphragms* to resist wind, seismic and other in-plane lateral loads shall be in accordance with the provisions of AISI S200, the additional requirements of this standard and the *applicable building code*.

This standard shall not preclude the use of other materials, assemblies, structures or designs not meeting the criteria herein, when the other materials, assemblies, structures or designs demonstrate equivalent performance for the intended use to those specified in this standard. Where there is a conflict between this standard and other reference documents, the requirements contained within this standard shall govern.

This standard shall include Sections A through D inclusive.

A2 Definitions

Where terms appear in this standard in italics, such terms shall have meaning as defined in AISI S200 or as defined herein. Where a country is indicated in brackets following the definition, the definition shall apply only in the country indicated. Where terms are not italicized, such terms shall have ordinary accepted meaning in the context for which they are intended.

Amplified Seismic Load. Load determined in accordance with the *applicable building code* load combinations that include the system overstrength factor, Ω_o, for strength design (LRFD). [USA and Mexico]

Capacity Based Design. Method for designing a *seismic force resisting system* in which a) specific elements or mechanisms are designed 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]

Fiberboard. A fibrous, homogeneous panel made from lignocellulosic fibers (usually wood or cane) and having a density of less than 31 pounds per cubic foot (pcf) (497 kg/m^3) but more than 10 pcf (160 kg/m^3).

Seismic Force Resisting System. That part of the structural system that has been considered in the design to provide the required resistance to the earthquake forces and effects. [Canada]

Type II Shear Wall Segment. Section of *shear wall* (within a *Type II shear wall*) with full-height sheathing (i.e., with no openings) and which meets specific aspect ratio limits.

A3 Symbols and Notations

C = Boundary chord force (tension/compression) (lbs, kN)
C_a = Shear resistance adjustment factor from Table C3.2-1
E = Effect of horizontal and vertical seismic forces as defined in the *applicable building code*

F_a = Acceleration-based site coefficient, as defined in *NBCC* [Canada]
F_v = Velocity-based site coefficient, as defined in *NBCC* [Canada]
I_E = Earthquake importance factor of the structure, as defined in *NBCC* [Canada]
L_i = Width of *Type II shear wall segment* (inches, mm)
R = Seismic response modification coefficient as defined by the *applicable building code*
R_n = *Nominal strength*
R_d = Ductility-related force modification factor reflecting the capability of a structure to dissipate energy through inelastic behavior, to be used with *NBCC* [Canada]
R_o = Overstrength-related force modification factor accounting for the dependable portion of reserve strength in a structure, to be used with *NBCC* [Canada]
$S_a(T)$ = 5% damped spectral response acceleration, expressed as a ratio to gravitational acceleration, for a period T, as defined in *NBCC* [Canada]
V = Shear force in *Type II shear wall* (lbs, kN)
h = Height of a *shear wall* measured as (1) the maximum clear height from top of foundation to bottom of *diaphragm* framing above, or (2) the maximum clear height from top of a *diaphragm* to bottom of *diaphragm* framing above
ΣL_i = Sum of widths of *Type II shear wall segments* (feet, m)
v = Unit shear force (plf, kN/m)
w = Width of a *shear wall,* pier or *diaphragm* in the direction of application of force measured as the sheathed dimension of the *shear wall,* pier or *diaphragm*
ϕ = *Resistance factor* to be used in determining the *design strength* in *LRFD* and *LSD*
Ω = *Safety factor* to be used in determining the *allowable strength* in *ASD*
Ω_o = System overstrength factor as defined by the *applicable building code*

A4 Loads and Load Combinations

Buildings or other structures and all parts therein shall be designed to safely support all loads that are expected to affect the structure during its life in accordance with the *applicable building code*. In the absence of an *applicable building code*, the loads, forces, and combinations of loads shall be in accordance with accepted engineering practice for the geographical area under consideration as specified by the applicable sections of *Minimum Design Loads for Buildings and Other Structures* (*ASCE 7)* in the United States and Mexico, and the *National Building Code of Canada* (*NBCC)* in Canada.

A4.1 Modification Factors and Limitations in Canada

Ductility related force modification factors, R_d, overstrength related force modification factors, R_o, and restrictions for *cold-formed steel* light-framed structures that are to be designed for seismic loads in conjunction with the *National Building Code of Canada* shall be as listed in Table A4-1. In addition, gypsum board shear walls shall not be used alone to resist lateral loads and the use of gypsum board in shear walls shall be limited to structures four stories or less, in accordance with Table A4-2.

Table A4-1
Canada
Design Coefficients and Factors for Seismic Force Resisting Systems in Canada

Type of Seismic Force Resisting System	R_d	R_o	Building Height (m) Limitations[1]				
			Cases Where $I_E F_a S_a(0.2)$				Cases Where $I_E F_v S_a(1.0)$
			< 0.2	≥0.2 to <0.35	≥0.35 to ≤0.75	>0.75	>0.3
Shear Walls[2]							
Screw connected shear walls: wood-based structural panel	2.5	1.7	20	20	20	20	20
Screw connected shear walls: wood based structural and gypsum panels in combination	1.5	1.7	20	20	20	20	20
Diagonal Strap Braced (Concentric) Walls[3]							
Limited ductility braced wall[4]	1.9	1.3	20	20	20	20	20
Conventional construction[5]	1.2	1.3	15	15	NP	NP	NP
Other Cold-Formed Steel SFRS(s) Not Listed Above	1.0	1.0	15	15	NP	NP	NP

1. NP = Not Permitted.
2. *Seismic Force Resisting System* specifically detailed for ductile seismic performance. *Capacity based design* approach is applied, assuming the sheathing connections act as the energy-dissipating element (See Section C5.1).
3. *Seismic Force Resisting System* specifically detailed so that all members of the bracing system are subjected primarily to axial forces. The eccentric effect due to single sided bracing is neglected for purposes of this classification, but shall be considered in accordance with C5.2.2.3.
4. *Seismic Force Resisting System* specifically detailed for ductile seismic performance. *Capacity based design* approach is applied, assuming the braces act as the energy-dissipating element (gross cross-section yielding). See Section C5.2.
5. Lateral system not specifically detailed for ductile seismic performance (*Capacity based design* approach not required. See Section C5.2).

Table A4-2
Canada
Maximum Percentage of Total Shear Forces Resisted by Gypsum Board in a Story

Story	Percentage of Shear Forces			
	Stories in Building			
	4	3	2	1
4^{th}	80	-	-	-
3^{rd}	60	80	-	-
2^{nd}	40	60	80	-
1^{st}	40	40	60	80

A5 Referenced Documents

The following documents or portions thereof are referenced within this standard and shall be considered part of the requirements of this document.

1. AHA A194.1-85, *Cellulosic Fiberboard*, 1985 Edition, American Hardwood Association, Palatine, IL.
2. AISI S100-07, *North American Specification for the Design of Cold Formed Steel Structural Members*, American Iron and Steel Institute, Washington, DC.
3. AISI S200-07, *North American Standard for Cold-Formed Steel Framing – General Provisions*, American Iron and Steel Institute, Washington, DC.
4. ASCE 7-05 Including Supplement No. 1, *Minimum Design Loads for Buildings and Other Structures*, American Society of Civil Engineers, Reston, VA.
5. ASTM A1003/A1003M-05, *Standard Specification for Steel Sheet, Carbon, Metallic- and Nonmetallic-Coated for Cold-Formed Framing Members*, ASTM International, West Conshohocken, PA.
6. ASTM C208-95(2001), *Standard Specification for Cellulosic Fiber Insulating Board*, ASTM International, West Conshohocken, PA.
7. ASTM C954-04, *Standard Specification for Steel Drill Screws for the Application of Gypsum Panel Products or Metal Plaster Bases to Steel Studs From 0.033 in. (0.84 mm) to 0.112 in. (2.84 mm) in Thickness*, ASTM International, West Conshohocken, PA.
8. ASTM C1002-04, *Standard Specification for Steel Self-Piercing Tapping Screws for the Application of Gypsum Panel Products or Metal Plaster Bases to Wood Studs or Steel Studs*, ASTM International, West Conshohocken, PA.
9. ASTM C1396/C1396M–06, *Standard Specification for Gypsum Board*, ASTM International, West Conshohocken, PA.
10. ASTM C1513-04, *Standard Specification for Steel Tapping Screws for Cold-Formed Steel Framing Connections*, ASTM International, West Conshohocken, PA.
11. ASTM E2126-05, *Standard Test Methods for Cyclic (Reversed) Load Test for Shear Resistance of Walls for Buildings*, ASTM International, West Conshohocken, PA.
12. CAN/CSA-O325.0-92 (R2003), *Construction Sheathing*. Canadian Standards Association, Mississauga, Ontario, Canada.
13. CAN/CSA-S136-07, *North American Specification for the Design of Cold-Formed Steel Structural Members*, Canadian Standards Association, Mississauga, Ontario, Canada.
14. CSA-O121-M1978 (R2003), *Douglas Fir Plywood*. Canadian Standards Association, Mississauga, Ontario, Canada.
15. CSA-O151-04, *Canadian Softwood Plywood*. Canadian Standards Association, Mississauga, Ontario, Canada.
16. DOC PS 1-07, *Structural Plywood*, United States Department of Commerce, National Institute of Standards and Technology, Gaithersburg, MD.
17. DOC PS 2-04, *Performance Standard for Wood-Based Structural-Use Panels*, United States Department of Commerce, National Institute of Standards and Technology, Gaithersburg, MD.
18. NBCC 2005, *National Building Code of Canada*, 2005 Edition, National Research Council of Canada, Ottawa, Ontario, Canada.

B. GENERAL DESIGN REQUIREMENTS

B1 General

The proportioning, design and detailing of *cold-formed steel* light-framed systems, members, connections and connectors shall be in accordance with AISI S100 [CSA S136], AISI S200 and the referenced documents except as modified by the provisions of this standard.

The lateral force resisting systems shall be subject to the limitations in the *applicable building code*.

B2 Shear Resistance Based on Principles of Mechanics

The shear resistance of *diaphragms*, diagonal *strap bracing* and *shear walls* is permitted to be calculated by principles of mechanics using values of fastener strength and sheathing shear resistance. The *nominal strength* [*nominal resistance*] so calculated defines the maximum resistance that the *diaphragm*, *shear wall*, or diagonal *strap bracing* is capable of developing. *Available strength* [*factored resistance*] shall be computed based on the wind and seismic force requirements in the *applicable building code*. Calculated values for systems defined in this standard shall be scaled to the values in this standard.

B3 Framing and Anchorage

Boundary members, *chords*, *collectors* and connections thereto shall be proportioned to transmit the induced forces and, where required by this standard, the following:

(a) ***In the United States and Mexico***: *Amplified seismic loads*.

(b) ***In Canada***: Probable seismic resistance of the shear wall or diagonal *strap bracing* following a capacity based design approach.

C. WALLS

C1 General

The design of *shear walls* or systems using diagonal *strap bracing* that resist wind, seismic or other in-plane lateral loads, as permitted, shall comply with the requirements of this section. *Shear walls* shall be classified as either *Type I shear walls*, which shall meet the requirements of Section C2, or shall be classified as *Type II shear walls*, which shall meet the requirements of Section C3. Diagonal *strap bracing*, as part of a structural wall, shall meet the requirements of Section C4.

Where setbacks of structural walls create an offset between them on an upper and lower story, the floor diaphragm and floor framing shall be designed to transfer overturning and shear forces through the offset in accordance with this standard and the *applicable building code*.

C1.1 Seismic Requirements

The design shall comply with these provisions exclusive of those in Section C5 when the following is applicable:

(a) ***In the United States and Mexico:*** The seismic response modification coefficient, R, (for steel systems) is taken equal to or less than 3, in accordance with the *applicable building code*.

(b) ***In Canada:*** R_dR_o is taken equal to or less than 2 for sheathed *shear walls*, and equal to or less than 1.625 for diagonal strap braced walls, in accordance with the *applicable building code*. For sheathed *shear walls*, the height restrictions in Table A4-1 shall apply. For diagonal strap braced walls, the height restrictions corresponding to conventional construction shall apply.

The design shall comply with these provisions inclusive of those in Section C5 when the following is applicable:

(a) ***In the United States and Mexico:*** The seismic response modification coefficient, R, (for steel systems) is taken greater than 3, in accordance with the *applicable building code*.

(b) ***In Canada:*** R_dR_o is taken greater than 2 for sheathed *shear walls*, and greater than 1.625 for diagonal strap braced walls, in accordance with the *applicable building code*. The height restrictions in Table A4-1 shall apply.

C1.2 Seismic Forces Contributed by Masonry and Concrete Walls

Shear walls, diagonal *strap bracing* and *diaphragms* shall be permitted to be used to resist seismic forces contributed by masonry or concrete walls in structures under the following conditions:

(a) *Cold-formed steel* floor and roof members shall be permitted to be used in *diaphragms* to resist horizontal seismic forces contributed by masonry or concrete walls in structures over one story in height, provided such forces do not result in torsional force distribution through the *diaphragm*.

(b) Wood structural panel or steel sheet sheathed *shear walls* shall be permitted to be used to provide resistance to seismic forces in two-story structures of masonry or concrete walls, provided the following requirements are met:

  (1) Story-to-story wall heights shall not exceed 12 feet (3.66 m).

(2) *Diaphragms* shall not be considered to transmit lateral forces by torsional force distribution or cantilever past the outermost supporting *shear wall*.

(3) Combined deflections of *diaphragms* and *shear walls* shall not permit per story drift of supported masonry or concrete walls to exceed 0.7% of the story height at *LRFD design* [*LSD factored*] *load* levels.

(4) Wood structural panel sheathing in *diaphragms* shall have all unsupported edges blocked. Wood structural panel or steel sheet sheathing for both stories of *shear walls* shall have all unsupported edges blocked and, for the lower story, shall have a minimum thickness of 15/32" (12 mm) wood structural panel or 0.027" (0.683 mm) steel sheet sheathing.

(5) There shall be no out-of-plane horizontal offsets between the first and second stories of wood structural panel or steel sheet sheathed *shear walls* or diagonal *strap bracing*.

C1.3 Seismic Forces from Other Concrete or Masonry Construction

Cold-formed steel members and systems shall be permitted to be designed to resist seismic forces from other concrete or masonry components, including but not limited to: chimneys, fireplaces, concrete or masonry veneers, and concrete floors.

C2 Type I Shear Walls

A *Type I shear wall* shall be fully sheathed and shall be provided with *hold-down anchors* at each end of the wall segment. *Type I shear walls* sheathed with wood structural or sheet steel panels are permitted to have openings, between *hold-down anchors* at each end of a wall segment, where details are provided to account for force transfer around openings.

The *nominal strength* [*nominal resistance*] for *Type I shear walls*, as shown in Tables C2.1-1 through C2.1-5 for wind, seismic and other in-plane lateral loads, as permitted and applicable, shall be permitted to establish *available strength* for such walls. The *available strength* [*factored resistance*] shall be determined using the *safety factor* (Ω) or the *resistance factor* (ϕ) as set forth in Section C2.1.

The height to width aspect ratio (h/w) of a wall pier in a *Type I shear wall* with openings shall be limited to a maximum of 2:1. The height of a wall pier (h) shall be defined as the height of the opening adjacent to the sheathed wall. The width of a wall pier (w) shall be defined as the sheathed width of the pier adjacent to the opening. The width of a wall pier shall not be less than 24 inches (610 mm).

The height to width aspect ratio (h/w) of the *Type I shear wall* shall not exceed the values in Tables C2.1-1, C2.1-2, C2.1-3, C2.1-4 and C2.1-5, unless permitted in a footnote to the table. The width of a *Type I shear wall* shall not be less than 24 inches (610 mm).

C2.1 Available Strength (Factored Resistance)

The *available strength* [*factored resistance*] shall be determined by using the *nominal strength* [*nominal resistance*] shown in Tables C2.1-1 through C2.1-5, as permitted and applicable, and dividing by the appropriate *safety factor* (Ω) or multiplying by the appropriate *resistance factor* (ϕ), as follows:

$\Omega = 2.50$ for ASD (seismic)

$\Omega = 2.00$ for ASD (wind or other in-plane lateral loads)

$\phi = 0.60$ for LRFD (seismic)

$\phi = 0.65$ for LRFD (wind or other in-plane lateral loads)

ϕ = 0.70 for LSD (except as noted below)

ϕ = 0.60 for LSD (gypsum sheathed walls)

Where a height to width aspect ratio (h/w) of a *shear wall* segment is greater than the tabulated value, as permitted in footnotes to Tables C2.1-1, C2.1-3 and C2.1-4, the *available strength* [*factored resistance*] shall be multiplied by 2w/h, but in no case shall the height to width aspect ratio (h/w) exceed 4:1.

The *available strength* [*factored resistance*] for shear panels with different sheathing materials and fastener configurations applied to the same side of a wall is not cumulative. For walls with material of the same type and *nominal strength* [*nominal resistance*] applied to opposite faces of the same wall, the *available strength* [*factored resistance*] of material of the same capacity is cumulative. Where the material *nominal strengths* [*nominal resistances*] are not equal, the *available strength* [*factored resistance*] shall be either two times the *available strength* [*factored resistance*] of the material with the smaller value or shall be taken as the value of the stronger side, whichever is greater. Summing the *available strengths* [*factored resistance*] of dissimilar material applied to opposite faces or to the same wall line is not allowed unless permitted by Table C2.1-1

C2.1.1 Design Deflection

The deflection of a blocked wood structural panel or sheet steel *shear wall* fastened throughout shall be permitted to be calculated in accordance with the following:

$$\delta = \frac{2vh^3}{3E_sA_cb} + \omega_1\omega_2\frac{vh}{\rho G t_{sheathing}} + \omega_1^{5/4}\omega_2\omega_3\omega_4\left(\frac{v}{\beta}\right)^2 + \frac{h}{b}\delta_v \qquad (Eq.\ C2.1\text{-}1)$$

where

A_c = Gross cross-sectional area of *chord* member, in square inches (mm^2)

b = Width of the *shear wall*, in inches (mm)

E_s = Modulus of elasticity of steel = 29,500,000 psi (203,000 MPa)

G = Shear modulus of sheathing material, in pounds per square inch (MPa)

h = Wall height, in inches (mm)

s = Maximum fastener spacing at panel edges, in inches (mm)

$t_{sheathing}$ = Nominal panel thickness, in inches (mm)

t_{stud} = Framing designation thickness, in inches (mm)

v = Shear demand (V/b), in pounds per linear inch (N/mm)

V = Total lateral load applied to the *shear wall*, in pounds (N)

β = 67.5 for plywood and 55 for OSB for U.S. Customary ($lb/in^{1.5}$)

= 2.35 for plywood and 1.91 for OSB for SI units ($N/mm^{1.5}$)

= 41.67 ($t_{sheathing}$/0.018) for sheet steel (for $t_{sheathing}$ in inches) ($lb/in^{1.5}$)

= 1.45 ($t_{sheathing}$/0.457) for sheet steel (for $t_{sheathing}$ in mm) ($N/mm^{1.5}$)

δ = Calculated deflection, in inches (mm)

δ_v = Vertical deformation of anchorage/attachment details, in inches (mm)

ρ = 1.85 for plywood and 1.05 for OSB

= 0.075($t_{sheathing}$/0.018) for sheet steel (for $t_{sheathing}$ in inches)

= 0.075($t_{sheathing}$/0.457) for sheet steel (for $t_{sheathing}$ in mm)

ω_1 = s/6 (for s in inches) and s/152.4 (for s in mm) (*Eq.* C2.1-2a)

ω_2 = 0.033/t_{stud} (for t_{stud} in inches) and 0.838/t_{stud} (for t_{stud} in mm) (*Eq.* C2.1-2b)

$\omega_3 = \sqrt{\frac{(h/b)}{2}}$ (*Eq.* C2.1-3)

ω_4 = 1 for wood structural panels

$= \sqrt{\frac{33}{F_y}}$ (for F_y in ksi) (*Eq.* C2.1-4a)

$= \sqrt{\frac{227.5}{F_y}}$ (for F_y in MPa) for sheet steel (*Eq.* C2.1-4b)

C2.2 Limitations for Tabulated Systems

The lateral resistant systems listed in Tables C2.1-1, C2.1-2, C2.1-3, C2.1-4 and C2.1-5 shall conform to the following requirements:

1. *Studs* shall be *C-shape* members with a minimum thickness of 33-mil, minimum *flange* width of 1-5/8 inches (41.3 mm), minimum *web* depth of 3-1/2 inches (89 mm) and minimum *edge stiffener* of 3/8 inches (9.5 mm) unless otherwise noted.
2. *Track* shall be a minimum thickness of 33 mils with a minimum *flange* width of 1-1/4 inches (31.8 mm) and a minimum *web* depth of 3-1/2 inches (89 mm) unless otherwise noted.
3. Framing screws shall be a minimum No. 8 in accordance with ASTM C1513.
4. Fasteners along the edges in shear panels shall be placed in from panel edges not less than the following, as applicable:
   (a) ***In the United States and Mexico***: 3/8 inches (9.5 mm).
   (b) ***In Canada***: 12.5 mm (1/2 inch).
5. Panel thicknesses shown shall be minimums.
6. Panels less than 12 inches (305 mm) wide shall not be used.
7. Maximum framing spacing shall be 24 inches (610 mm) on center.
8. Unless otherwise noted, all sheathing edges shall be attached to framing members or *blocking*. Where used as *blocking*, flat strapping shall be a minimum thickness of 33 mils with a minimum width of 1-1/2 inches (38.1 mm) and shall be either installed on top of or below sheathing. For other than steel sheathing, the screws shall be installed through the sheathing to the *blocking*.

C2.2.1 Sheet Steel Sheathing in the United States and Mexico

Steel sheets, attached to *cold-formed steel* framing, shall be permitted to resist horizontal forces produced by wind, seismic or other in-plane lateral loads subject to the following:

1. Steel sheets shall have a minimum *base steel thickness* as shown in Tables C2.1-1 or C2.1-3, and shall be of the following grade of steel: ASTM A1003 Structural Grade 33 (Grade 230) Type H.
2. *Nominal strengths* shall be given in Tables C2.1-1 for wind and other in-plane lateral loads and Table C2.1-3 for seismic loads. Table C2.1-3 shall also be permitted for calculating the *nominal strength* for wind and other in-plane lateral loads.
3. Steel sheets shall be permitted to be applied either parallel to or perpendicular to framing.
4. In lieu of *blocking*, panel edges shall be permitted to be overlapped and attached to each other with screw spacing as required for panel edges. Where such a connection is used, tabulated design values shall be reduced 30 percent.

5. Screws used to attach steel sheets shall be a minimum No. 8 or No. 10 in accordance with Table C2.1-3.

C2.2.2 Wood Structural Panel Sheathing

Cold-formed steel framed wall systems, sheathed with wood structural panels, shall be permitted to resist horizontal forces produced by wind, seismic or other in-plane lateral loads subject to the following:

1. Wood structural panels shall be manufactured using exterior glue and shall comply with the following, as applicable:
   (a) ***In the United States and Mexico:*** DOC PS 1 or PS 2.
   (b) ***In Canada:*** CSA O121, O151 or CAN/CSA O325.0.
2. *Nominal strengths* [*resistances*] shall be as given in the following tables, as applicable:
   (a) ***In the United States and Mexico:*** Tables C2.1-1, for wind and other in-plane lateral loads and Table C2.1-3, for seismic loads. Table C2.1-3 shall also be permitted for calculating the *nominal strength* for wind and other in-plane lateral loads.
   (b) ***In Canada:*** Table C2.1-4 for wind, seismic and other in-plane lateral loads.
3. Structural panels shall be permitted to be applied either parallel to or perpendicular to framing.
4. Wood structural panels shall be attached to steel framing with a minimum No. 8, tapping screws with a minimum head diameter of 0.285 inch (7.24 mm) or No. 10, tapping screws with a minimum head diameter of 0.333 inch (8.46 mm), in accordance with Table C2.1-3.
5. Screws used to attach wood structural panel sheathing shall be in accordance with ASTM C1513.
6. ***In the United States and Mexico:*** Where 7/16" OSB is specified, 15/32" Structural 1 Sheathing (plywood) shall be permitted for the values in Table C2.1-1 (wind loads).
7. Increases of the *nominal strengths* shown in Tables C2.1-1 and C2.1-3 as allowed by other standards shall not be permitted.

C2.2.3 Gypsum Board Panel Sheathing

Cold-formed steel framed wall systems, sheathed with gypsum board, shall be permitted to resist horizontal forces produced by wind or seismic loads subject to the following:

1. Gypsum board shall comply with ASTM C1396/C1396M.
2. *Nominal strengths* [*resistances*] shall be as given in the following tables, as applicable:
   (a) ***In the United States and Mexico:*** Table C2.1-2.
   (b) ***In Canada:*** Table C2.1-5.
3. Gypsum board shall be applied perpendicular to framing with strap *blocking* behind the horizontal joint and with solid *blocking* between the first two end *studs*, at each end of the wall, or applied vertically with all edges attached to framing members. Unblocked assemblies are permitted provided the *nominal strength* [*resistance*] values are multiplied by 0.35.
4. Screws used to attach gypsum board shall be a minimum No. 6 in accordance with ASTM C954 or ASTM C1002, as applicable.

C2.2.4 Fiberboard Panel Sheathing in the United States and Mexico

Cold-formed steel framed wall systems, sheathed with *fiberboard*, shall be permitted to resist horizontal forces produced by wind or seismic loads in *seismic design categories* A, B and C subject to the following:

1. *Fiberboard* panels shall comply with AHA A194.1 or ASTM C 208.
2. *Nominal* shear strengths shall be given in Table C2.1-2.
3. *Fiberboard* shall be applied perpendicular to framing with strap *blocking* behind the horizontal joint and with solid *blocking* between the first two end *studs*, at each end of the wall, or applied vertically with all edges attached to framing members.
4. Screws used to attach *fiberboard* shall be a minimum No. 8 in accordance with ASTM C1513. Head style shall be selected to provide a flat bearing surface in contact with the sheathing with a head diameter not less than 0.43 inches (10.9 mm). Screws shall be driven so that their flat bearing surface is flush with the surface of the sheathing.

Table C2.1-1
United States and Mexico
Nominal Shear Strength (R_n) for Wind and Other In-Plane Loads for Shear Walls [1,4,6,7,8]
(Pounds Per Foot)

Assembly Description	Maximum Aspect Ratio (h/w)	Fastener Spacing at Panel Edges[2] (inches)			
		6	4	3	2
15/32" structural 1 sheathing (4-ply), one side	2:1	1065 [3]	-	-	-
7/16" rated sheathing (OSB), one side	2:1	910 [3]	1410	1735	1910
7/16" rated sheathing (OSB), one side oriented perpendicular to framing	2:1	1020	-	-	-
7/16" rated sheathing (OSB), one side	2:1 [5]	-	1025	1425	1825
0.018" steel sheet, one side	2:1	485	-	-	-
0.027" steel sheet, one side	4:1	-	1,000 [9]	1085 [9]	1170 [9]
	2:1 [5]	647	710	778	845

1. *Nominal strengths* shall be multiplied by the *resistance factor* (ϕ) to determine *design strength* or divided by the *safety factor* (Ω) to determine *allowable strengths* as set forth in Section C2.1.
2. Screws in the field of the panel shall be installed 12 inches (305 mm) o.c. unless otherwise shown.
3. Where fully blocked gypsum board is applied to the opposite side of this assembly, per Table C2.1-2 with screw spacing at 7 inches (178 mm) o.c. edge and 7 inches (178 mm) o.c. field, these *nominal strengths* are permitted to be increased by 30%.
4. See Section C2.1 for requirements for sheathing applied to both sides of wall.
5. *Shear wall* height to width aspect ratio's (h/w) greater than 2:1, but not exceeding 4:1, shall be permitted provided the *nominal strength* is multiplied by 2w/h. See Section C2.1.
6. Shear values are permitted for use in seismic design where the seismic response modification factor, R, is taken equal to or less than 3, subject to the limitations in Section C1.1.
7. For wood structural panel sheathed shear walls, tabulated R_n values shall be applicable for short-term load duration (wind loads). For other in-plane lateral loads of normal or permanent load duration as defined by the AF&PA NDS, the values in the table above for wood structural panel sheathed shear walls shall be multiplied by 0.63 (normal) or 0.56 (permanent).
8. For SI: 1" = 25.4 mm, 1 foot = 0.305 m, 1 lb = 4.45 N.
9. For these assemblies, the *designation thickness* of *stud*, *track* and *blocking* shall be a minimum of 43 mils.

Table C2.1-2
United States and Mexico
Nominal Shear Strength (R_n) for Wind and Seismic Loads
for Shear Walls Faced with Gypsum Board or Fiberboard [1,2,3,4]
(Pounds Per Foot)

Assembly Description	Maximum Aspect Ratio (h/w)	Fastener Spacing at Panel Edges/Field (inches)						
		7/7	4/4	4/12	8/12	4/6	3/6	2/6
½” gypsum board on one side of wall; studs max. 24” o.c.	2:1	290	425	295	230	-	-	-
½” fiberboard on one side of wall; studs max. 24” o.c.	1:1	-	-	-	-	425	615	670

1. *Nominal strengths* shall be multiplied by the *resistance factor* (ϕ) to determine *design strength* or divided by the *safety factor* (Ω) to determine *allowable strengths* as set forth in Section C2.1.
2. See Section C2.1 for requirements for sheathing applied to both sides of wall.
3. For gypsum or fiberboard sheathed shear walls, tabulated values shall be applicable for short-term load duration only (wind or seismic loads).
4. For SI: 1” = 25.4 mm, 1 foot = 0.305 m, 1 lb = 4.45 N.

Table C2.1-3
United States and Mexico
Nominal Shear Strength (R_n) for Seismic and Other In-Plane Loads for Shear Walls [1,4,7,8]
(Pounds Per Foot)

Assembly Description	Max. Aspect Ratio (h/w)	Fastener Spacing at Panel Edges[2] (inches)				Designation Thickness[5,6] of Stud, Track and Blocking (mils)	Required Sheathing Screw Size
		6	4	3	2		
15/32" Structural 1 sheathing (4-ply), one side	2:1[3]	780	990	-	-	33 or 43	8
	2:1	890	1330	1775	2190	43 or 54	8
						68	10
7/16" OSB, one side	2:1[3]	700	915	-	-	33	8
	2:1[3]	825	1235	1545	2060	43 or 54	8
	2:1	940	1410	1760	2350	54	8
	2:1	1232	1848	2310	3080	68	10
0.018" steel sheet, one side	2:1	390	-	-	-	33 (min.)	8
0.027" steel sheet, one side	4:1	-	1000	1085	1170	43 (min.)	8
	2:1[3]	647	710	778	845	33 (min.)	8

1. *Nominal strength* shall be multiplied by the *resistance factor* (ϕ) to determine *design strength* or divided by the *safety factor* (Ω) to determine *allowable strength* as set forth in Section C2.1.
2. Screws in the field of the panel shall be installed 12 inches (305 mm) o.c. unless otherwise shown.
3. *Shear wall* height to width aspect ratios (h/w) greater than 2:1, but not exceeding 4:1, shall be permitted provided the *nominal strength* values are multiplied by 2w/h. See Section C2.1.
4. See Section C2.1 for requirements for sheathing applied to both sides of wall.
5. Unless noted as (min.), substitution of a *stud* or *track* of a different *designation thickness* is not permitted.
6. Wall *studs* and *track* shall be of ASTM A1003 Structural Grade 33 (Grade 230) Type H steel for members with a *designation thickness* of 33 and 43 mils, and A1003 Structural Grade 50 (Grade 340) Type H steel for members with a *designation thickness* equal to or greater than 54 mils.
7. For wood structural panel sheathed shear walls, tabulated R_n values applicable for short-term load duration (seismic loads). For other in-plane lateral loads of normal or permanent load duration as defined by the AF&PA NDS, the values in the table above for wood structural panel sheathed shear walls shall be multiplied by 0.63 (normal) or 0.56 (permanent).
8. For SI: 1" = 25.4 mm, 1 foot = 0.305 m, 1 lb = 4.45 N.

Table C2.1-4
Canada
Nominal Shear Resistance (R_n) for Wind, Seismic and Other In-Plane Lateral Loads for Shear Walls Sheathed with Wood Structural Panels [1,4,7,8]
(kN/m)

Assembly Description	Max. Aspect Ratio (h/w)	Fastener Spacing at Panel Edges[2] (mm)			Designation Thickness[5,6] of Stud, Track and Blocking (mils)	Required Sheathing Screw Size
		150	100	75		
9.5 mm CSP Sheathing	2:1[3]	8.5	11.8	14.2	43 (min.)	8
12.5 mm CSP Sheathing	2:1[3]	9.5	13.0	19.4	43 (min.)	8
12.5 mm DFP Sheathing	2:1[3]	11.6	17.2	22.1	43 (min.)	8
9 mm OSB 2R24/W24	2:1[3]	9.6	14.3	18.2	43 (min.)	8
11 mm OSB 1R24/2F16/W24	2:1[3]	9.9	14.6	18.5	43 (min.)	8

1. *Nominal resistance* shall be multiplied by the *resistance factor* (ϕ) to determine the factored shear resistance as set forth in Section C2.1.
2. Screws in the field of the panel shall be installed 300 mm o.c. unless otherwise shown.
3. *Shear wall* height to width aspect ratios (h/w) greater than 2:1, but not exceeding 4:1, shall be permitted provided the *nominal resistance* values are multiplied by 2w/h. See Section C2.1.
4. See Section C2.1 for requirements for sheathing applied to both sides of wall.
5. Unless noted as (min.), substitution of a *stud* or *track* of a different *designation thickness* is not permitted.
6. Wall *studs* and *track* shall be of ASTM A1003 Structural Grade 230 (Grade 33) Type H steel for members with a *designation thickness* of 33 and 43 mils, and A1003 Structural Grade 340 (Grade 50) Type H steel for members with a *designation thickness* equal to or greater than 54 mils.
7. Tabulated R_n values shall be applicable for short-term load duration (wind or seismic loads). For other in-plane lateral loads of permanent term load duration (dead) tabulated R_n values shall be multiplied by 0.56. For standard term load duration (snow and occupancy) tabulated R_n values shall be multiplied by 0.80. For other permanent and standard load combinations where the specified dead load is greater than the specified standard term load tabulated R_n values shall be multiplied by a factor equal to $0.8 - 0.43 \log (D/ST) \geq 0.56$, where D = specified dead load and ST = specified standard term load based on snow or occupancy loads acting alone or in combination.
8. For U.S. Customary Units: 1 mm = 0.0394", 1 m = 3.28 feet, 1 N = 0.225 lb.

Table C2.1-5
Canada
Nominal Shear Resistance (R_n) for Wind and Seismic Loads for Shear Walls Sheathed with Gypsum Board [1,2,3,4,5]
(kN/m)

Assembly Description	Maximum Aspect Ratio (h/w)	Fastener Spacing at Panel Edges/Field (mm)		
		100/300	150/300	200/300
12.5 mm gypsum board on one side of wall; studs max. 600 mm o.c.	2:1	3.4	3.1	2.7

1. *Nominal resistance* shall be multiplied by the *resistance factor* (ϕ) to determine the *factored resistance* as set forth in Section C2.1.
2. See Section C2.1 for requirements for sheathing applied to both sides of wall.
3. Unblocked assemblies shall be permitted provided the *nominal resistance* values above are multiplied by 0.35.
4. Tabulated values shall be applicable for short-term load duration only (wind or seismic loads). Gypsum sheathed shear walls shall not be permitted for other load durations.
5. For U.S. Customary Units: 1 mm = 0.0394", 1 m = 3.28 feet, 1 N = 0.225 lb.

C3 Type II Shear Walls

Type II shear walls sheathed with wood structural panels or sheet steel shall be permitted to resist wind and seismic loads when designed in accordance with this section. *Type II shear walls* shall meet the requirements for *Type I shear walls* except as revised by this section.

C3.1 Limitations

The following limitations shall apply to the use of *Type II shear walls*:

1. A *Type II shear wall segment*, meeting the aspect ratio (h/w) limitations of Section C3.2.3, shall be located at each end of a *Type II shear wall*. Openings shall be permitted to occur beyond the ends of the *Type II shear wall*; however, the width of such openings shall not be included in the width of the *Type II shear wall*.
2. The *nominal strength* (*nominal resistance*) shall be based upon a panel edge screw spacing that is greater than or equal to 4 inches (100 mm) o.c. when the following is applicable:
   (a) ***In the United States and Mexico:*** The *Seismic Design Category* is other than A.
   (b) ***In Canada:*** The specified short period spectral acceleration ratio ($I_EF_aS_a(0.2)$) is greater than 0.167.
3. A *Type II shear wall* shall not have out of plane (horizontal) offsets. Where out of plane offsets occur, portions of the wall on each side of the offset shall be considered as separate *Type II shear walls*.
4. *Collectors* for shear transfer shall be provided for the full length of the *Type II shear wall*.
5. A *Type II shear wall* shall have uniform top of wall and bottom of wall elevations. *Type II shear walls* not having uniform elevations shall be designed by other methods.
6. *Type II shear wall* height, h, shall not exceed 20 feet (6.1 m).

C3.2 Type II Shear Wall—Design Shear Resistance

The *available strength* [*factored resistance*] of *Type II shear walls* shall be equal to the *adjusted shear resistance*, as determined by the provisions in Section C3.2.4, times the sum of the widths (ΣL_i) of the *Type II shear wall segments* and shall be calculated in accordance with the following:

C3.2.1 Percent Full-Height Sheathing

The percent of full-height sheathing shall be calculated as the sum of widths (ΣL_i) of *Type II shear wall segments* divided by the total width of the *Type II shear wall* including openings.

C3.2.2 Maximum Opening Height Ratio

The maximum opening height ratio shall be calculated by dividing the maximum opening clear height by the *shear wall* height, h.

C3.2.3 Unadjusted Shear Resistance

The *unadjusted shear resistance* shall be the *available strength* [*factored resistance*] calculated in accordance with Section C2.1, based upon the following, as applicable:

(a) ***In the United States and Mexico:*** *Nominal strengths* (R_n) in Tables C2.1-1 and C2.1-3.
(b) ***In Canada:*** *Nominal resistances* (R_n) in Table C2.1-4.

The aspect ratio (h/w) of *Type II shear wall segments* used in calculations shall comply with Section C2.1.

C3.2.4 Adjusted Shear Resistance

The *adjusted shear resistance* shall be calculated by multiplying the *unadjusted shear resistance* by the shear resistance adjustment factors of Table C3.2-1. For intermediate values of opening height ratio and percentages of full-height sheathing, the shear resistance adjustment factors shall be permitted to be determined by interpolation.

Table C3.2-1
Shear Resistance Adjustment Factor-C_a

	Maximum Opening Height Ratio [1]				
	1/3	1/2	2/3	5/6	1
Percent Full-Height Sheathing [2]	**Shear Resistance Adjustment Factor**				
10%	1.00	0.69	0.53	0.43	0.36
20%	1.00	0.71	0.56	0.45	0.38
30%	1.00	0.74	0.59	0.49	0.42
40%	1.00	0.77	0.63	0.53	0.45
50%	1.00	0.80	0.67	0.57	0.50
60%	1.00	0.83	0.71	0.63	0.56
70%	1.00	0.87	0.77	0.69	0.63
80%	1.00	0.91	0.83	0.77	0.71
90%	1.00	0.95	0.91	0.87	0.83
100%	1.00	1.00	1.00	1.00	1.00

1. See Section C3.2.2.
2. See Section C3.2.1.

C3.3 Anchorage and Load Path

Design of *Type II shear wall* anchorage and load path shall conform to the requirements of this section, or shall be calculated using principles of mechanics.

C3.3.1 Collectors and Anchorage for In-Plane Shear

The unit shear force, *v*, transmitted into the top and out of the base of the *Type II shear wall* full-height sheathing segments, and into *collectors* (drag struts) connecting *Type II shear wall segments*, shall be calculated in accordance with the following:

$$v = \frac{V}{C_a \sum L_i} \qquad (Eq.\ C3.3\text{-}1)$$

where

v = Unit shear force (plf, kN/m)

V = Shear force in *Type II shear wall* (lbs, kN)

C_a = Shear resistance adjustment factor from Table C3.2-1

ΣL_i = Sum of widths of *Type II shear wall segments* (feet, mm/1000)

C3.3.2 Uplift Anchorage at Type II Shear Wall Ends

Anchorage for uplift forces due to overturning shall be provided at each end of the *Type II shear wall*. Uplift anchorage and boundary chord forces shall be determined from *Eq*. C3.3-2.

In the United States and Mexico: When seismic response modification coefficient, R, is taken greater than 3, the uplift anchorage and boundary chords shall also comply with the requirements of Section C5.1.2.

In Canada: When R_dR_o is taken greater than 2 for wood sheathed shear walls, the uplift anchorage and boundary chords shall also comply with the requirements of Section C5.1.2.

$$C = \frac{Vh}{C_a \sum L_i} \qquad (Eq. \text{ C3.3-2})$$

where

C = Boundary chord force (tension/compression) (lbs, kN)

V = Shear force in *Type II shear wall* (lbs, kN)

h = *Shear wall* height (feet, mm/1000)

C_a = Shear resistance adjustment factor from Table C3.2-1

ΣL_i = Sum of widths of *Type II shear wall segments* (feet, mm/1000)

C3.3.3 Uplift Anchorage Between Type II Shear Wall Ends

In addition to the requirements of Section C3.3.2, *Type II shear wall* bottom plates, at full-height sheathing locations, shall be anchored for a uniform uplift force, *t*, equal to the unit shear force, *v*, determined in Section C3.3.1.

C3.3.4 Load Path

A load path to the foundation shall be provided for the uplift, shear, and compression forces as determined from Sections C3.3.1 through C3.3.3 inclusive. Elements resisting *shear wall* forces contributed by multiple stories shall be designed for the sum of forces contributed by each story.

C4 Diagonal Strap Bracing

Diagonal *strap bracing*, as part of a structural wall, is permitted to resist wind, seismic and other in-plane forces, and shall be designed in accordance with AISI S100 [CSA S136], AISI S200 and the requirements of this standard.

C4.1 Diagonal Strap Braced Wall Aspect Ratio

The aspect ratio (height/width) of a shear wall with diagonal *strap bracing*, as part of a structural wall, shall not exceed 2:1 unless a rational analysis is performed which includes joint flexibility and end moments in the design of the *chord studs*.

C5 Special Seismic Requirements

Where required by Section C1 of this standard, the requirements of this section shall apply in addition to the requirements of Sections C2, C3 and C4.

C5.1 Shear Walls

Where steel or wood sheathing is provided for lateral resistance, the requirements of this section shall apply.

C5.1.1 Connections

C5.1.1.1: The *available strength* [*factored resistance*] of connections for *boundary members* and *collectors* shall exceed the following, as applicable:

(a) ***In the United States and Mexico:*** *Nominal* tensile strength of the member, but need not exceed the *amplified seismic load*.

(b) ***In Canada:*** Loads the system can deliver (C5.1.5), but need not exceed the maximum anticipated seismic loads calculated with $R_dR_o = 1.0$.

C5.1.1.2: The pull-out resistance of screws shall not be used to resist seismic forces.

C5.1.2 Chord Studs and Anchorage

C5.1.2.1: *Studs* or other vertical *boundary members* at the ends of wall segments, that resist seismic loads, braced with sheathing, shall be anchored such that the bottom *track* is not required to resist uplift by bending of the *track web*.

C5.1.2.2: *Studs* or other vertical *boundary members* and uplift anchorage thereto shall have the *nominal strength* to resist the following, as applicable:

(a) ***In the United States and Mexico:*** Loads that the system can deliver, but need not exceed the *amplified seismic load*.

(b) ***In Canada:*** Loads the system can deliver (C5.1.5), but need not exceed the maximum anticipated seismic loads calculated with $R_dR_o = 1.0$.

C5.1.3 Foundations

In the United States and Mexico: Foundations need not be designed to resist loads resulting from the *amplified seismic load*s.

In Canada: Foundations shall be designed to resist the loads resulting from the lesser of the loads that the system can deliver (C5.1.5) and the maximum anticipated seismic loads calculated with $R_dR_o = 1.0$.

C5.1.4 Additional Requirements

C5.1.4.1: Wall *studs* and *track* shall be of ASTM A1003 Structural Grade 33 (Grade 230) Type H steel for members with a *designation thickness* of 33 and 43 mils, and A1003 Structural Grade 50 (Grade 340) Type H steel for members with a *designation thickness* equal to or greater than 54 mils.

C5.1.4.2: ***In the United States and Mexico:*** The *nominal* shear strength for light-framed wall systems for buildings, where the seismic response modification coefficient, R, used to determine the lateral forces is taken greater than 3, shall be based upon values from Table C2.1-3.

C5.1.5 Probable Shear Wall Force in Canada

The *seismic force resisting system* shall be assumed to deliver a load based on the probable shear capacity of the wall determined from the *nominal resistance* from Tables C2.1-4 and C2.1-5 accounting for overstrength and the appropriate principal and companion loads as required by the *NBCC* (Capacity Based Design). An overstrength factor of 1.33 shall be used with the *nominal resistance* values from Tables C2.1-4 and C2.1-5 for walls sheathed with DFP or OSB wood structural panels or gypsum. An overstrength of 1.45 shall be used with the *nominal resistance* values from Table C2.1-4 for walls sheathed with CSP wood structural panels.

C5.2 Diagonal Strap Bracing

Where diagonal *strap bracing* is provided for lateral resistance, the requirements of this section shall apply.

In the absence of verified physical properties measured in accordance with an *approved* test method, the R_y and R_t values in Table C5-1 shall be used. In either case, R_y shall not be less than 1.1.

Table C5-1
R_y and R_t Values for Diagonal Strap Bracing Members

Yield Strength	R_y	R_t
33 ksi [230 MPa]	1.5	1.2
37 ksi [255 MPa]	1.4	1.1
40 ksi [275 MPa]	1.3	1.1
50 ksi [340 MPa]	1.1	1.1

C5.2.1 Connections

C5.2.1.1: The *available strength* [*factored resistance*] of connections for diagonal *strap bracing* members, *boundary members* and *collectors* shall exceed the expected *yield strength* of the diagonal *strap bracing* member, $A_gR_yF_y$, except the *available strength* [*factored resistance*] need not exceed the following, as applicable:

(a) ***In the United States and Mexico:*** *Amplified seismic load.*
(b) ***In Canada:*** Maximum anticipated seismic loads calculated with R_dR_o = 1.0.

C5.2.1.2: The pull-out resistance of screws shall not be used to resist seismic forces.

C5.2.1.3: The connection of the diagonal *strap bracing* member shall be welded and designed to permit gross cross section yielding of the diagonal *strap bracing* member, unless one of the following criteria is satisfied for the alternate connection:

(a) It can be demonstrated that the alternate connection permits gross cross section yielding of the diagonal *strap bracing* member under cyclic loading in accordance with the loading protocol in ASTM E2126, or
(b) The diagonal *strap bracing* member has a ratio of $(R_tF_u)/(R_yF_y)$ greater than or equal to 1.2, and engineering calculations (capacity based design calculations) can demonstrate that the gross cross section yielding failure mode will occur prior to net section fracture based on the pattern and spacing of the fasteners.

C5.2.2 Chord Studs and Anchorage

C5.2.2.1: Studs or other vertical boundary members at the ends of wall segments, that resist seismic loads, braced with diagonal braces shall be anchored such that the bottom *track* is not required to resist uplift by bending of the *track web*. When the *track* is not designed to resist the horizontal shear force from the diagonal brace by compression or tension, the horizontal shear force shall be resisted by a device connected directly to the diagonal brace and anchored directly to the foundation or supporting structural element.

C5.2.2.2: All members in the load path and uplift and shear anchorage thereto from the diagonal *strap bracing* member to the foundation shall have the *nominal strength* to resist the expected yield strength, $A_gR_yF_y$, of the diagonal *strap bracing* member(s), except the *nominal strength* need not exceed the following, as applicable:

(a) ***In the United States and Mexico:*** *Amplified seismic load.*
(b) ***In Canada:*** Maximum anticipated seismic loads calculated with R_dR_o = 1.0.

C5.2.2.3: Eccentricity shall be considered in the design where single-sided diagonal *strap bracing* is provided.

C5.2.3 Foundations

In the United States and Mexico: Foundations need not be designed to resist loads resulting from the *amplified seismic loads*.

In Canada: Foundations shall be designed to resist the loads resulting from the lesser of the maximum anticipated seismic loads calculated with R_dR_o = 1.0, and the loads calculated using the expected yield strength, $A_gR_yF_y$, of the diagonal *strap bracing* member(s).

C5.2.4 Additional Requirements

The expected *yield strength*, $A_gR_yF_y$, of the diagonal *strap bracing* member shall not exceed the *expected* tensile strength of the member, $A_nR_tF_u$. Provisions shall be made for pretensioning, or other methods of installing tension-only diagonal *strap bracing* shall be used to guard against loose diagonal *straps*. The slenderness ratio of the diagonal *strap bracing* member shall be permitted to exceed 200.

D. DIAPHRAGMS

In the United States and Mexico: The design of *diaphragms* that resist wind, seismic or other in-plane lateral loads shall comply with the requirements of this section.

D1 General

The *diaphragm* sheathing shall consist of sheet steel, concrete, or wood structural panel sheathing or other *approved* materials.

D1.1 Seismic Requirements for Diaphragms

When the seismic response modification coefficient, R, (for steel systems) is taken equal to or less than 3, in accordance with the *applicable building code*, the design shall comply with these provisions exclusive of those in Section D3.

Where the seismic response modification coefficient, R, is taken greater than 3, in accordance with the *applicable building code*, the design shall comply with these provisions inclusive of those in Section D3.

D2 Diaphragm Design

D2.1 Available Shear Strength

The *available strength* of *diaphragms* shall be determined in accordance with Section B2. Alternatively for *diaphragms* sheathed with wood structural panels, the *available strength* is permitted to be determined by the Section D2.2.

D2.1.1 Design Deflection

The deflection of a blocked wood structural panel *diaphragm* shall be permitted to be calculated in accordance with the following:

$$\delta = \frac{0.052vL^3}{E_s A_c b} + \omega_1\omega_2\frac{vL}{\rho G t_{sheathing}} + \omega_1^{5/4}\omega_2(\alpha)\left(\frac{v}{2\beta}\right)^2 + \frac{\sum_{j=1}^{n}\Delta_{ci}X_i}{2b} \qquad (Eq.\ D2.1\text{-}1)$$

where

A_c = Gross cross-sectional area of *chord* member, in square inches (mm^2)

b = *Diaphragm* depth parallel to direction of load, in inches (mm)

E_s = Modulus of elasticity of steel = 29,500,000 psi (203,000 MPa)

G = Shear modulus of sheathing material, in pounds per square inch (MPa)

L = *Diaphragm* length perpendicular to direction of load, in inches (mm)

n = Number of *chord* splices in *diaphragm* (considering both *diaphragm chords*)

s = Maximum fastener spacing at panel edges, in inches (mm)

$t_{sheathing}$= Nominal panel thickness, in inches (mm)

t_{stud} = Nominal framing thickness, in inches (mm)

v = Shear demand (V/2b), in pounds per linear inch (N/mm)

V = Total lateral load applied to the *diaphragm*, in pounds (N)

X_i = Distance between the" ith" *chord*-splice and the nearest support (braced wall line), in inches (mm)

α = Ratio of the average load per fastener based on a non-uniform fastener pattern to the average load per fastener based on a uniform fastener pattern (= 1 for a uniformly fastened *diaphragm*)

β = 67.5 for plywood and 55 for OSB for U.S. Customary ($lb/in^{1.5}$)
= 2.35 for plywood and 1.91 for OSB for SI units ($N/mm^{1.5}$)

δ = Calculated deflection, in inches (mm)

Δ_{ci} = Deformation value associated with "ith" *chord* splice, in inches (mm)

ρ = 1.85 for plywood and 1.05 for OSB

ω_1 = s/6 (for s in inches) (*Eq*. D2.1-2a)
= s/152.4 (for s in mm) (*Eq*. D2.1-2b)

ω_2 = $0.033/t_{stud}$ (for t_{stud} in inches) (*Eq*. D2.1-3a)
= $0.838/t_{stud}$ (for t_{stud} in mm) (*Eq*. D2.1-3b)

For unblocked *diaphragms*, δ shall be multiplied by 2.50.

D2.2 Wood Diaphragms

The *nominal strength* of wood structural panel *diaphragms*, used to determine the *available strength*, is permitted to be taken from Table D2-1 subject to the requirements of this section. Sheathing material in wood *diaphragms* shall conform to DOC PS-1 and PS-2. Wood structural panel *diaphragms* shall be designed as either blocked or unblocked.

Where *allowable strength design* (*ASD*) is used, the *allowable strength* shall be determined by dividing the *nominal strength*, shown in Table D2-1, by a *safety factor* (Ω) of 2.5 for assemblies resisting seismic loads and 2.0 for assemblies resisting wind or other in-plane lateral loads.

Where *load and resistance factor design* (*LRFD*) is used, the *design strength* shall be determined by multiplying the *nominal strength*, shown in Table D2-1, by a *resistance factor* (ϕ) of 0.60 for assemblies resisting seismic loads and 0.65 for assemblies resisting wind or other in-plane lateral loads.

D2.2.1 Diaphragm Aspect Ratio

The aspect ratio (length/width) of wood *diaphragms* shall not exceed 4:1 for blocked *diaphragms* and 3:1 for unblocked *diaphragms*.

D2.2.3 Framing

The minimum *designation thickness* of framing members shall be 33 mils.

D2.2.4 Attachment of the Sheathing to Framing

Panel edges of sheathing shall be attached to framing as indicated in Table D2-1 with minimum No. 8 countersunk tapping screws in accordance with ASTM C1513. Screws in the field of the panel shall be attached to intermediate supports at a maximum 12-inch (305 mm) spacing along the framing.

D2.2.5 Blocking

Where *diaphragms* are designed as blocked, all panel edges shall be attached to framing members or *blocking*. Where used as *blocking*, flat strapping shall be a minimum thickness of 33 mils with a minimum width of 1½ inches (38.1 mm) and shall be either installed on top of or below sheathing. For other than steel sheathing, the screws shall be installed through the sheathing to the *blocking*.

Table D2-1
United States and Mexico
Nominal Shear Strength (R_n) for Diaphragms with Wood Sheathing [1, 4]
(Pounds Per Foot)

Membrane Material	Screw Size	Thick-ness (in.)	Blocked				Unblocked	
			Screw spacing at diaphragm boundary edges and at all continuous panel edges (in.)				Screws spaced maximum of 6" on all supported edges	
			6	4	2.5	2	Load perpendicular to unblocked edges and continuous panel joints	All other configurations
			Screw spacing at all other panel edges (in.)					
			6	6	4	3		
Structural I	See note 2	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]	See note 2	3/8	690	920	1470	1840	615	460
		7/16	760	1015	1620	2030	680	505
		15/32	832	1110	1770	2215	740	555

1. For SI: 1" = 25.4 mm, 1 foot = 0.305 m, 1 lb = 4.45 N.
2. No. 8 screws (minimum) shall be used when framing members have a *designation thickness* of 54 mils or less and No. 10 screws (minimum) shall be used when framing members have a *designation thickness* greater than 54 mils.
3. Wood structural panels shall conform to DOC PS-1 and PS-2.
4. For wood structural panel sheathed diaphragms, tabulated R_n values shall be applicable for short-term load duration (wind or seismic loads). For other in-plane lateral loads of normal or permanent load duration as defined by the AF&PA NDS, the values in the table above for wood structural panel sheathed diaphragms shall be multiplied by 0.75 (normal) or 0.67 (permanent).

D3 Special Seismic Requirements

D3.1 General

Where the seismic response modification coefficient, R, used to determine the lateral forces is taken greater than 3, the requirements of this section shall apply in addition to the requirements of Sections D1 and D2.

Diaphragms shall be defined as flexible or rigid, in accordance with the *applicable building code*.

D3.2 Wood Diaphragms

The aspect ratio (length/width) of a *diaphragm* sheathed with wood structural sheathing shall be limited to 4:1 where all edges of the wood structural panel sheathing are attached to framing members or to intermittent *blocking*. Where there is no intermittent *blocking*, the aspect ratio shall be limited to 3:1. Wood structural panel sheathing shall be arranged so that the minimum panel width is not less than 24 inches (610 mm) unless further limited elsewhere in these provisions.

Open front structures with rigid wood *diaphragms* resulting in torsional force distribution shall be limited by the following:

1. The length of the *diaphragm* normal to the open side shall not exceed 25 feet (7.62 m), and the aspect ratio (length/width) shall be less than 1:1 for one-story structures or 2:3 for structures over one story in height, where the length dimension of the *diaphragm* is parallel to the opening.
2. Where calculations show that *diaphragm* deflections can be tolerated, the length normal to the opening shall be permitted to be increased to an aspect ratio (length/width) not greater than 3:2.

AISI S213-2007-C/
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American
Iron and Steel
Institute

AISI STANDARD

Commentary on North American Standard for Cold-Formed Steel Framing – Lateral Design 2007 Edition with Supplement No. 1

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Endorsed by:

Steel Framing Alliance™

DISCLAIMER

The material contained herein has been developed by the American Iron and Steel Institute Committee on Framing Standards. The Committee has made a diligent effort to present accurate, reliable, and useful information on cold-formed steel framing design and installation. 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 this *Commentary*.

With anticipated improvements in understanding of the behavior of cold-formed steel framing and the continuing development of new technology, this material will 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 design professional. Indeed, in many jurisdictions, such 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.

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PREFACE

This *Commentary* is intended to facilitate the use, and provide an understanding of the background, of AISI S213, the *North American Standard for Cold-Formed Steel Framing – Lateral Design*. The *Commentary* illustrates the substance and limitations of the various provisions of the standard.

Changes are made to the *Commentary*, which provide background information related to changes included in Supplement No. 1 to the standard.

In the *Commentary*, sections, equations, figures, and tables are identified by the same notation as used in the standard. Words that are italicized are defined in AISI S200. Terms included in square brackets are specific to LSD terminology.

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COMMENTARY ON THE NORTH AMERICAN STANDARD FOR COLD-FORMED STEEL FRAMING – LATERAL DESIGN

A. GENERAL

The provisions of AISI S213 (AISI, 2007) were initially based on the requirements in the *International Building Code* (ICC, 2003) and the NFPA 5000 *Building Construction and Safety Code* (NFPA, 2003). The provisions in those codes evolved since the early work of Tarpy (1976-80), APA-The Engineered Wood Association (1993), Serrette (1995a) and the *shear wall* provisions that were first introduced into the 1997 *Uniform Building Code* (ICBO, 1997). Research conducted by Serrette at Santa Clara University and Dolan at Virginia Polytechnic Institute and State University form the technical basis for the initial design values in the standard. Specific references to this research are cited in this *Commentary*. In 2007, provisions and design values related to *shear wall* and diagonal strap braced wall design, which are to be used with the 2005 *National Building Code of Canada* [*NBCC*] (NRCC, 2005), were added to AISI S213 (AISI, 2007) based largely on research carried out under the supervision of Rogers at McGill University (2005-7). At this time, AISI S213 does not address steel sheet *shear walls* or *diaphragms*. Studies are ongoing, and it is expected that these will be addressed in future editions of the standard.

A2 Definitions

The definition for *Amplified Seismic Load* was added to Section A2. The original S213 special seismic provisions were written when there were only load combinations for *Strength Design* (LRFD) with the system overstrength factor in the building code. Since then, ASCE 7 has added load combinations for *Allowable Stress Design* (ASD) with the system overstrength factor. If the load combinations for *Allowable Stress Design* (ASD) with the system overstrength factor are desired to be used and they are to be checked against the member nominal strength, then member nominal strength is required to be divided by 1.4.

A4 Loads and Load Combinations

Currently, *ASCE 7* (ASCE, 2006) has no geographical-based information on Mexico. Therefore, users with projects in Mexico should work with the appropriate authority having jurisdiction to determine appropriate loads and load combinations that are consistent with the assumptions and rationale used by *ASCE 7*.

A4.1 Modification Factors and Limitations in Canada

Building height limitations in Table A4-1 are listed as a function of short period ($S_a(0.2)$) and long period ($S_a(1.0)$) spectral acceleration adjusted for the site class and the earthquake importance factor. $S_a(0.2)$ and $S_a(1.0)$ are the 5% damped spectral response accelerations for a period of 0.2s and 1.0s, respectively, for the reference ground condition Site Class C, as defined in NBCC. $S_a(0.2)$ and $S_a(1.0)$ can be found in NBCC Table C-2 (Design Data for Selected Locations in Canada) for most towns and cities in Canada.

Ductility-related (R_d) and overstrength-related (R_o) seismic force modification factors are recommended for the design of *cold-formed steel* framed – wood structural panel *shear walls* using the 2005 *NBCC* (NRCC, 2005). These values have been selected by Boudreault et al. (2007) based on an approach documented by Mitchell et al. (2003) to conservatively represent the results of ductility based shear wall assembly tests (Branston et al., 2006b; Blais, 2006; Rokas, 2006; Hikita, 2007). The development of these force modification factors was directly dependent on the analysis of *shear wall* test data using the Equivalent Energy Elastic-Plastic (EEEP) approach (Park, 1989; ASTM E2126, 2005). Preliminary dynamic analyses of representative *cold-formed steel* framed buildings designed with the proposed R-values have shown that the inelastic shear deformations are within an acceptable range, as defined by test results (Blais, 2006). The proposed R-values should only be used in the case of a sheathing connection failure mode in the *shear wall*, as opposed to *chord stud* failure. The design method proposed by Branston et al. (2006a) must be implemented for the determination of *shear wall* resistance values for these R-values to be considered as valid. This method means that R_dR_o and the associated shear capacities are not independent values - they must be considered together. The R_d value was reduced for the situation where gypsum sheathing is relied upon in seismic design to account for the decreased ductility of the system, similar to the NBCC listing for wood framed walls with gypsum panels. Building height limits were set based on the current United States values listed in ASCE 7 (2006).

The designer should be aware that the *chord studs*, if not selected following a *Capacity Based Design* approach, may suffer from compression failure if gravity loads are present during seismic events. These *chord studs* should be selected following a *Capacity Based Design* approach such that the total expected compression force from gravity and lateral loads could be resisted. This is a prudent approach even when *Capacity Based Design* is not required by AISI S213. Hikita (2006) verified that *shear walls* subjected to combined gravity and lateral loads can perform at an adequate level to warrant the use of the tabulated R_d and R_o values, if the *chord studs* are designed to carry the anticipated compression force, related to sheathing connection failure during a seismic event, combined with the associated *NBCC* principal and companion gravity loads. The lateral overstrength of the *shear walls*, again related to sheathing connection failure, needs to be accounted for in the estimate of the expected compression force in the *chord studs*. The remaining elements in the *SFRS* also need to be designed following a *Capacity Based Design*, accounting for the effects of overstength.

Ductility-related (R_d) and overstrength-related (R_o) seismic force modification factors are also recommended for the design of *cold-formed steel* framed diagonal strap braced walls using the 2005 *NBCC* (NRCC, 2005). In the case where the braces of the wall are able to reach and maintain their yield strength in the inelastic range of behavior, i.e. yielding takes place along the length of the braces without failure of any other *SFRS* element, it is possible for the ductility and overstrength of the wall assembly to reach levels associated with those of a limited ductility (LD) concentrically braced frame (CBF) as described in CSA S16S1 (CSA, 2005) (Al-Kharat and Rogers, 2006). This requires the use of a *Capacity Based Design* approach whereby all *SFRS* elements are selected based on the probable yield capacity of the brace (A_g R_y F_y). If a *Capacity Based Design* approach is not implemented, then the ductility of the wall may be reduced due to failure of one or a number of other elements in the *SFRS*. Al-Kharat and Rogers (2005) showed through testing of double sided strap braced walls that the R values used for conventional construction (CC) in CSA S16S1 could also be applied for *cold-formed steel* systems that had not been designed with a capacity based approach. The use of conventional construction strap braced walls is limited to areas of low seismicity, and the height limit has been reduced. In the case of a wall designed using the CC R-values, it is also recommended that the wall be designed such that the *track* punching shear mode of failure does not occur. A *hold-down anchor* detail in which a flat plate is situated within the *track* section without a direct connection to the *chord stud* may result in the plate punching through the *track* on the uplift side of the wall. *Hold-down anchor* devices that are connected directly to the *chord studs*, which do not transfer force into the *track* sections, are recommended. See Al-Kharat and Rogers (2005, 2006 and 2007) for additional information on this failure mode.

In 2008, findings of a research project at McGill University on the inelastic performance of welded (Comeau and Rogers, 2008) and screw connected (Velchev and Rogers, 2008) *strap* braced walls demonstrated that the R_d, R_o and height limits values listed in Section A4.1 of AISI S213-07 were appropriate. The results of the research were forwarded to the Canadian National Committee on Earthquake Engineering (CANCEE) in a proposal for the inclusion of these *cold-formed steel strap* braced walls in the National Building Code of Canada (NBCC) seismic provisions. CANCEE approved of the inclusion of this seismic system in the 2010 NBCC, with two minor modifications. In 2009, the R_d values in Table A4-1 for diagonal *strap* braced (concentric) walls were adjusted to match the values approved by CANCEE.

Only the most common structural systems are identified and have assigned values of R_d and R_o in Table A4-1. If an SFRS not specifically identified in Table A4-1 is used, then R_d = R_o = 1.0 must be used for design. This requirement is based on the assumption that systems that are not described should be designed conservatively. R_d and R_o values of 1.0 have been assigned to systems that are not otherwise defined in Table A4-1 because their ductility and overstrength capacity, respectively, have not yet been demonstrated.

The limitations on the use of gypsum board in shear walls in Canada are based upon those used for the design of wood structures in CSA O86 (CSA, 2001).

B. GENERAL DESIGN REQUIREMENTS

B2 Shear Resistance Based on Principles of Mechanics

AISI S213 does not aim to limit *cold-formed steel* light-framed *shear walls*, diagonal *strap bracing* (that is part of a structural wall) and *diaphragms* to the configurations included in the standard. As such, the development of design values for other systems or configurations is permitted in accordance with rational engineering procedures and principles of mechanics. Design values based on calculations must, however, recognize the fundamental differences between the expected performance of structures under wind and seismic *loads*, and the performance of an individual lateral element. It must also recognize that the tabulated design values in the standard are based on test data for individual lateral elements. Recognition of these differences requires, where appropriate, that calculated values be scaled per existing design data.

In seismic design, *loads* are modified to account for system/element/component ductility (inelastic behavior), redundancy and overstrength (ATC, 1995, NEHRP, 2000, SEAOC, 1999). As a result, the lateral resisting element (*diaphragm*, *shear wall* or diagonal *strap bracing*) must meet some minimum performance requirement(s). In wind design there is no modification in design loads per the lateral resisting system used. In light of this and the differences alluded to in the previous paragraph, where design values are determined by calculation, these values must be scaled to existing values (where available). For example, if AISI S213 provides design values for 27-mil, 33-ksi sheet steel on 43-mil framing and design values are sought (by calculation) for 33-mil, 33-ksi sheet steel on 43-mil framing, the following calculations should be undertaken:

i. The design value for 27-mil, 33-ksi sheet steel on 43-mil framing should be determined by calculation, with due consideration for code implied seismic performance, and the ratio of the design value in the standard to the computed design value determined. If the computed ratio exceeds unity, it should be taken as unity.

ii. The calculated value for 33-mil, 33ksi sheet steel on 43-mil framing should then be multiplied by the ratio (less than unity) determined in the previous step.

The intent of the scaled calculated value is to provide some limited assurance that inelastic dynamic performance characteristics are accounted for in the calculated value.

B3 Framing and Anchorage

In *diaphragms*, *shear walls* and other braced walls; the basic lateral resisting element is the attached sheathing or brace. The framing members, *collectors* and anchorage serve to transfer *load* from the point of origination to the lateral element and finally to the point of resistance (load path). In wind design, since design *loads* are not reduced, these components need only be designed for the design loads. However, because seismic loads are reduced, to develop the anticipated performance, it is desirable to focus damage (inelastic behavior) in the lateral element itself. As such, depending on the seismic risk level, the components transferring load to or from the lateral element should be capable of resisting the *nominal strength* of the element or some *amplified seismic load*. The *amplified seismic load* is essentially an estimate of the *nominal strength* that the lateral element is capable of developing.

C. WALLS

C1 General

The prescriptive requirements for setbacks in the previous edition of the Lateral Design Standard have been deleted. The floor diaphragm and framing members must be designed to transfer the overturning and shear forces from a shear wall or a diagonal strap braced wall where a setback occurs. There may be additional requirements for setbacks in the applicable building code.

C1.1 Seismic Requirements

In the United States and Mexico: When the seismic response modification coefficient, R, is greater than 3, AISI S213 requires that the design must follow the special seismic requirements of Section C5 and when R is less than or equal to 3, Section C5 is not required. In addition, AISI S213 is to be read in conjunction with the *applicable building code* documents. For ASCE 7 (ASCE, 2006), the design coefficients, factors and limitations assigned to light-framed *shear wall* systems in ASCE 7 are reproduced in Table C1-1, below. ASCE 7 also provides limitations based on the *Seismic Design Category*. For *Seismic Design Category* A through C, the designer has the option to use an R of 3 for systems with a higher assigned R to determine the seismic load and thereby avoid the special detailing in Section C5. For this case, the design coefficients and factors for "Steel Systems not Specifically Detailed for Seismic Resistance Excluding Cantilever Column Systems" of Table C1-1 would apply. In Seismic Design Category D through F, the designer does not have the option to choose an R of 3 for systems with a higher assigned R. The design coefficients and factors in Table C1-1 apply. Note that it is never permitted to choose an R of 3 for systems with a lower assigned R.

Table C1-1[d]
United States and Mexico
Design Coefficients and Factors for Basic Seismic Force-Resisting Systems

Basic Seismic Force-Resisting System [b]	Seismic Response Modification Coefficient, R	System Over-strength Factor, Ω_o[c]	Deflection Amplification Factor, C_d	Structural System Limitations and Building Height (ft) Limitations [a]				
				Seismic Design Category				
				A&B	C	D	E	F
A. BEARING WALL SYSTEMS								
Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	6 ½	3	4	NL	NL	65	65	65
Light-framed walls with shear panels of all other materials	2	2 ½	2	NL	NL	35	NP	NP
Light-framed wall systems using flat strap bracing	4	2	3 ½	NL	NL	65	65	65
B. BUILDING FRAME SYSTEMS								
Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	7	2 ½	4 ½	NL	NL	65	65	65
Light-framed walls with shear panels of all other materials	2 ½	2 ½	2 ½	NL	NL	35	NP	NP
H. STEEL SYSTEMS NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE, EXCLUDING CANTILEVER COLUMN SYSTEMS	3	3	3	NL	NL	NP	NP	NP

[a] NL = Not Limited and NP = Not Permitted.

[b] Per ASCE 7 (ASCE, 2006), a bearing wall system is defined as a structural system with bearing walls providing support for all or major portions of the vertical loads and a building frame system is defined as a structural system with an essentially complete space frame providing support for vertical loads. Per AISI S213, shear walls or braced frames are the basic seismic force resisting elements.

[c] The tabulated value of the overstrength factor, Ω_o, is permitted to be reduced by subtracting one-half for structures with flexible diaphragms, but shall not be taken as less than 2.0 for any structure.

[d] See ASCE/SEI 7 (ASCE, 2006) Table 12.2-1 for additional footnotes.

For SI: 1 foot = 0.305 m

In Canada: When R_dR_o is greater than 2 for sheathed shear walls and greater than 1.625 for diagonal strap braced walls, AISI S213 requires that the design must follow the special seismic requirements of Section C5. When R_dR_o is less than or equal to 2 for sheathed shear walls and 1.625 for diagonal strap braced walls, Section C5 is not required. For sheathed shear walls, a designer has the option to choose an R_dR_o of 2 for systems with a higher R_dR_o to determine the seismic load and thereby avoid the special detailing in Section C5. For this case, the height limitations for "Other Cold-Formed Steel SFRS(s) Not Listed Above" in

Table A4-1 of AISI S213 would apply. For diagonal strap braced walls, a designer has the option to choose an R_dR_o of 1.625 for systems with a higher R_dR_o to determine the seismic load and thereby avoid the special detailing in Section C5. For this case, the height limitations for "Conventional construction" in Table A4-1 of AISI S213 would apply. Note that the lower R_dR_o value of 1.625 associated with diagonal strap bracing was chosen to ensure that the system remains essentially elastic.

C1.2 Seismic Forces Contributed by Masonry and Concrete Walls

In 2007, requirements were added to AISI S213 for resisting seismic forces contributed by masonry and concrete walls. These requirements were patterned after provisions in the Special Design Provisions for Wind and Seismic (AFPA, 2005b).

The use of wood structural panel sheathed *diaphragms* with masonry or concrete walls is common practice. Story height and other limitations for *cold-formed steel* members and systems resisting seismic forces from concrete or masonry walls are given to address deformation compatibility. Due to significant differences in stiffness, wood structural panel sheathed *diaphragms* are not permitted where forces contributed by masonry or concrete walls result in torsional force distribution through the *diaphragm*. A torsional force distribution through the *diaphragm* would occur when the center of rigidity is not coincident with the center of mass, such as an open front structure. Where wood structural panel or steel sheet sheathed *shear walls* are used to provide resistance to seismic forces contributed by masonry and concrete walls, deflections are limited to 0.7% of the story height at *LRFD design load* [*factored load*] levels in accordance with deflection limits for masonry and concrete construction and Section 12.8.6 of ASCE 7 (ASCE, 2006). The intent is to limit failure of the masonry or concrete portions of the structure due to excessive deflection.

It should be noted that Section 12.10.2.1 of ASCE 7 (ASCE, 2006) requires that *collectors*, splices, and their connections to resisting elements be designed for the *amplified seismic load* when a structure is not braced entirely by light-frame shear walls. This imposes an additional requirement for *collectors* when *cold-formed steel* framing is used to resist seismic forces contributed by masonry and concrete walls.

C1.3 Seismic Forces from Other Concrete or Masonry Construction

Seismic forces from other concrete or masonry construction (i.e., other than walls) are permitted and should be accounted for in design. The provisions of this section specifically allow masonry veneers; i.e., a masonry facing attached to a wall for the purpose of providing ornamentation, protection or insulation, but not counted as adding strength to the wall. Likewise, the provisions of this section are not intended to restrict the use of concrete floors – including *cold-formed steel framed* floors with concrete toppings as well as reinforced concrete slabs – or similar such elements in floor construction. It is intended that where such elements are present in combination with a *cold-formed steel* framed system, the *cold-formed steel* framed system needs to be designed to account for the seismic forces generated by the additional mass of such elements. The design of *cold-formed steel* members to support the additional mass of concrete and masonry elements needs to be in accordance with AISI S100 [CSA S136] and required deflection limits as specified in concrete or masonry standards or the model building codes.

C2 Type I Shear Walls

A *shear wall* assembly using an approved adhesive to attach *shear wall* sheathing to the framing is not yet recognized by this standard or by ASCE 7. Sufficient test data to justify acceptance of *shear walls* that use adhesive alone or in combination with fasteners to attach sheathing to the framing members was not available at the time this standard was written. The limited existing test data indicates that *shear walls* using adhesives for sheathing attachment will generally not perform the same as *shear walls* with only fasteners attaching the sheathing to the framing.

While use of this type of system may be adequate for wind resistance or low seismic risk regions, these *shear walls* tend to have limited ductility and as a result, the R-value, Seismic Design Category limitations and height limitations required for systems resisting seismic forces for wood and steel sheet sheathed systems in ASCE 7 may not be generally applicable. In addition, the *shear wall* deflection equation provided in this standard would not be applicable, as adhesive-based *shear walls* tend to be stiffer than this equation would suggest.

Serrette (2006) conducted tests on *cold-formed steel* frame *shear walls* utilizing structural adhesives. For the walls with OSB attached by structural adhesive, the measured responses up to the maximum/peak wall resistances were relatively linear and the post-peak behavior was characterized by a somewhat sharp degradation, but not a complete loss of strength. The walls with sheet steel attached by a structural adhesive exhibited a more nonlinear behavior with a less severe reduction in strength after the maximum resistance; however, testing of such systems has been too limited to include specific provisions in the standard. Design deflections calculated in accordance with Section C2.1.1 would probably not be applicable for adhesive *shear wall* systems where *shear wall* sheathing is rigidly bonded to shear wall boundary members. Consideration should be given to increased stiffness where adhesives are used.

In 2009, Standard Equation C2.1-1 for determining the deflection of a blocked wood structural panel was consolidated for U.S. Customary and SI units.

C2.1 Available Shear Strength

In the United States and Mexico: The requirements for *Type I shear walls* in AISI S213 were based on studies by Serrette (1996, 1997 and 2002). This series of investigations included reverse cyclic and monotonic loading and led to the development of the design values and details for plywood, oriented strand board (OSB), and gypsum wall-board (GWB) *shear wall* assemblies that are included in the standard. Figures C2-1(a) and C2-1(b) show typical *Type I shear walls*, with and without detailing for force transfer around window openings, and with hold-down anchors at the ends of each wall segment. Where wall assemblies are engineered for force transfer around openings and engineering analysis shows that uplift restraint at openings is not required, the assembly may be treated as a *Type I shear wall* and hold-downs are required at the ends of the assembly only, as illustrated in Figure C2-1(b). The *nominal* values in Tables C2.1-1, C2.1-2 and C2.1-3 were based on tests with *studs* with 1.5-inch (38 mm) x 4-inch (100 mm) punchouts at a center-to-center spacing of 24 inches (600 mm), anchor bolts with standard cut washers and hold-down anchors on each end of the wall. As a result, the standard permits the use of *studs* with standard punchouts and anchor bolts with standard cut washers, and requires hold-down anchors even though calculations may demonstrate that hold-down anchors are not necessary.

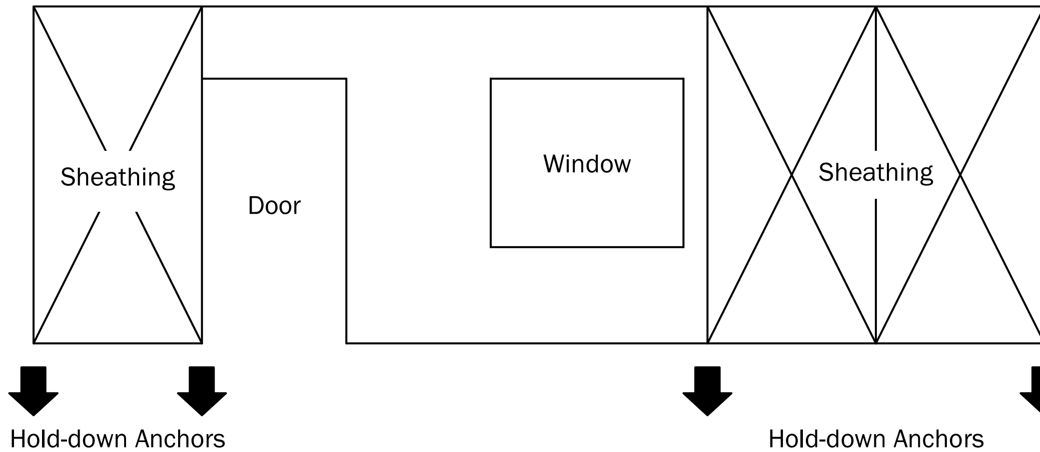


Figure C2-1(a) – Type I Shear Walls Without Detailing for Force Transfer Around Openings

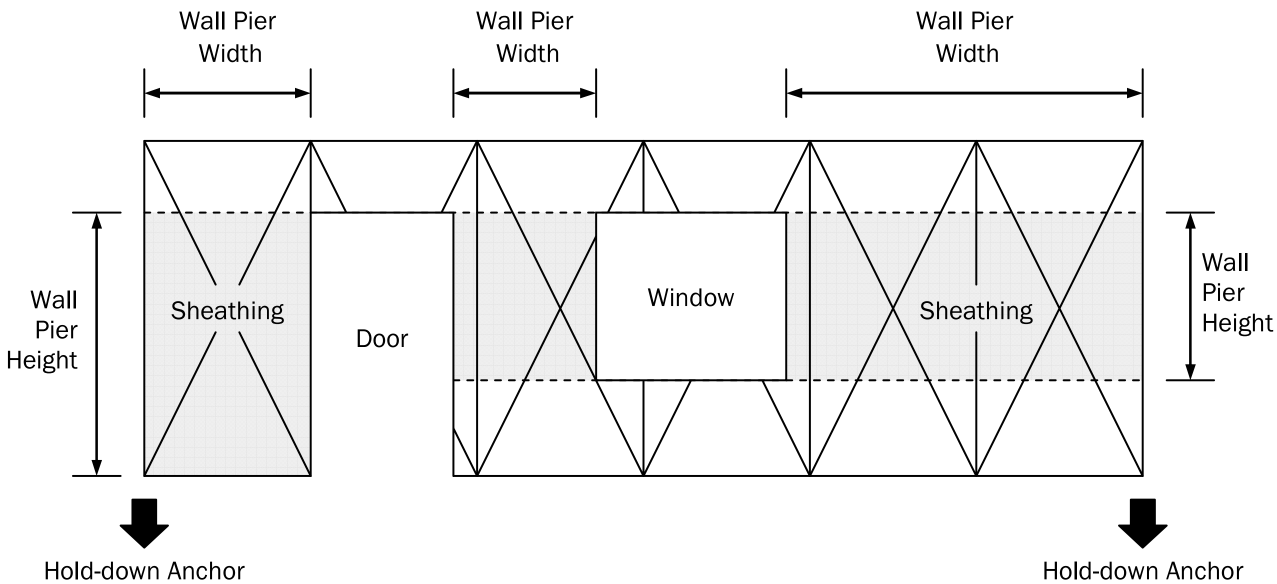


Figure C2-1(b) – Type I Shear Wall With Detailing for Force Transfer Around Openings

In the United States and Mexico: The *nominal strength* values in Tables C2.1-1 and C2.1-2 were based on monotonic tests data, and the values in Table C2.1-3 were based on reversed cyclic tests and degraded wall strength envelope responses. The basic reversed cyclic test protocol used in the tests is illustrated in Figure C2-2. Generally, the protocol frequency ranged between 0.2 and 1.0 Hz. The response (hysteretic) plot, and typical peak and degraded strength envelopes are illustrated in Figure C2-3. The degraded wall strength is the set of points describing the peak strength associated with the second cyclic of a target (repeated) input displacement (per Figure C2-2).

In the United States and Mexico: Table C2.1-3 prescribes a maximum *stud* thickness in order to preclude a change in failure mode of the screw fasteners. In 2007, the *nominal strength* values for 15/32” structural 1 sheathing (4-ply), one side and a maximum aspect ratio of 2:1 for 43- or 54-mil designation thickness with No. 8 screws were explicitly permitted to be used for 68-mil designation thickness with No. 10 screws, based on inspection of the tabulated values for 7/16" OSB, one side and analysis using the equations in AISI S100 [CSA S136] for screw tilting and screw shear.

In the United States and Mexico: Overdriving of the sheathing screws will result in lower strength, stiffness and ductility of a shear wall compared with the values obtained from testing (Rokas, 2006); hence sheathing screws should be firmly driven into framing members but not overdriven into sheathing.

In the United States and Mexico: In 2007, factors were included, based on load duration factors given in the 2005 NDS (AFPA, 2005a) as shown in Table C2-1, to account for the influence of the duration of the applied load on wood strength to allow the values in Tables C2.1-1, C2.1-3 and D2-1 to be used for other in-plane lateral loads. Since the *shear wall* tests used as the basis of AISI S213 were carried out over a short time span, the tabulated values are for short-term duration loads (i.e., wind or seismic). However, the tabulated values for *diaphragms* were calculated using a load duration factor of 1.33, rather than the factor of 1.6 given in the 2005 NDS.

In the United States and Mexico: In 2007, adjustments were made to Table C2.1-3 for 0.027" steel sheet, one side, based on testing at the University of North Texas (Yu, 2007). *Designation thickness* for *stud*, *track* and *blocking* associated with the existing 0.027" steel sheet tabulated values was increased from 33 mils (min.) to 43 mils (min.). New values were added for *designation thickness* for *stud*, *track* and *blocking* equal to 33 mils (min.).

Table C2-1
United States and Mexico
AFPA NDS Load Duration Factors

Load Duration	Factor	Typical Design Loads
Permanent	0.9	Dead Load
Ten years	1.0	Occupancy Live Load
Two months	1.15	Snow Load
Seven days	1.25	Construction Load
Ten minutes	1.6	Wind/Earthquake Load

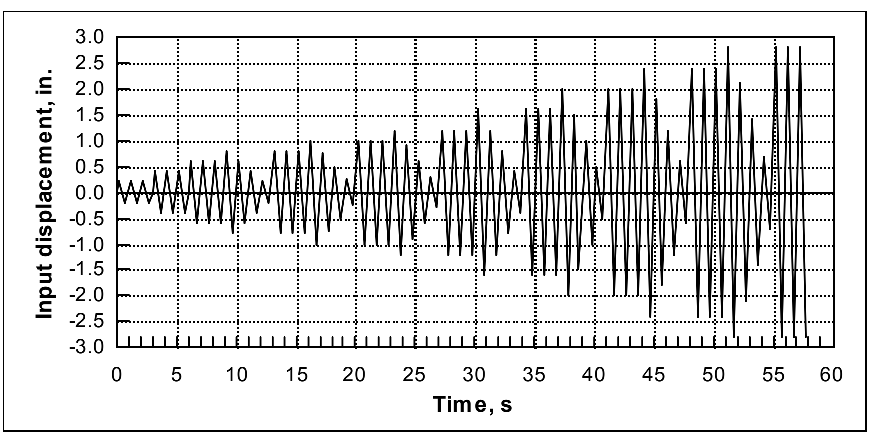


Figure C2-2 – Reverse Cyclic Test Protocol (1.0 Hz)

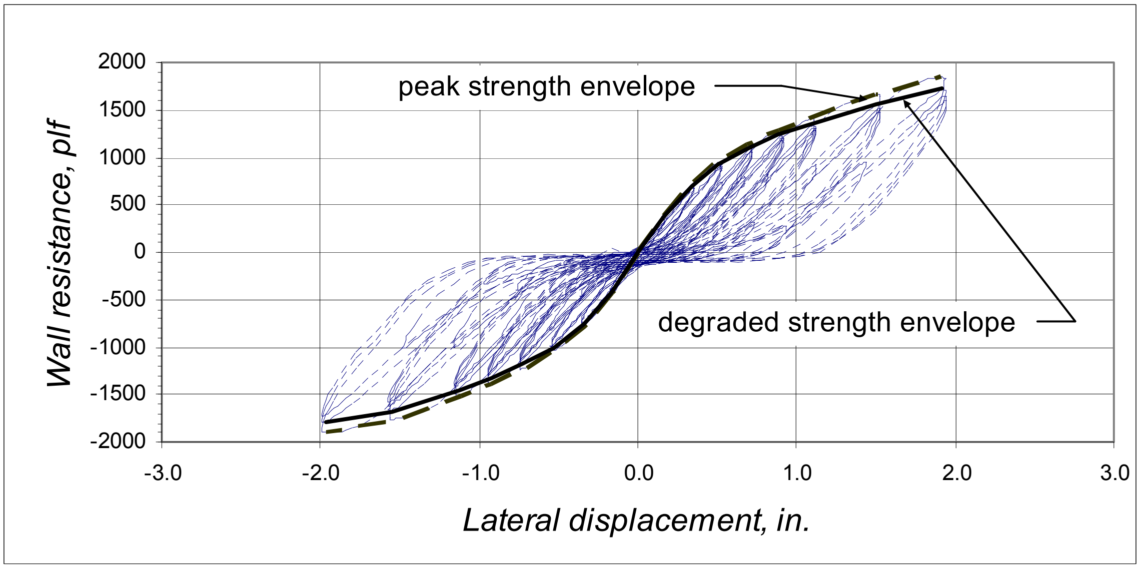


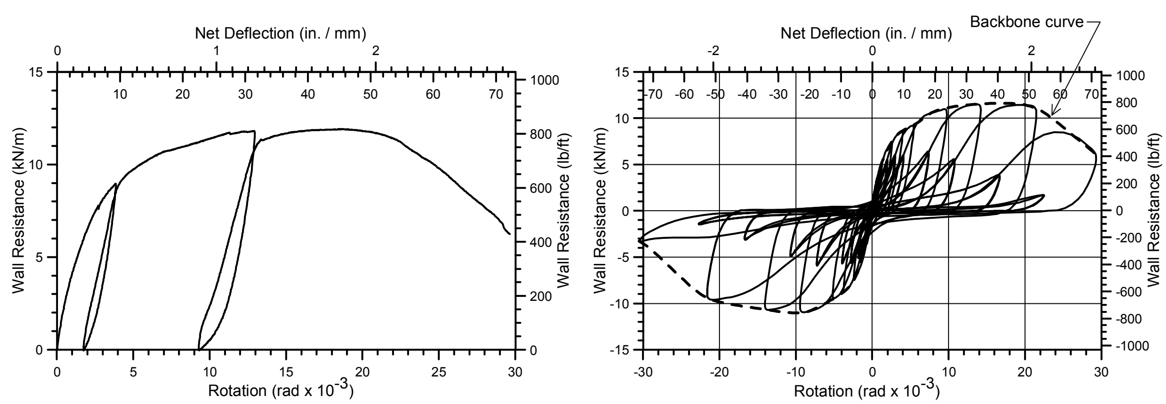
Figure C2-3 – Hysteretic Response Plot Showing Peak and Degraded Strength Envelopes

In the United States and Mexico: Recognizing that no standard method existed for interpreting reversed cyclic data from light frame wall tests and that lateral design values of other light frame lateral elements are based on monotonic tests, a simple procedure was developed to estimate *nominal strengths*. This procedure utilized the degraded strength envelope and defined the *nominal strength* of a particular wall configuration as the smaller of the maximum strength and 2.5 times the strength at 0.5 in. of lateral displacement. The 0.5-inch displacement was based on the allowable strength drift limit for an 8-ft. wall in accordance with the 1994 Uniform Building Code (ICBO, 1994), which was the code in effect at the time this information was first proposed for acceptance in a building code.

In the United States and Mexico: In 2006, requirements were added for *Type I shear walls* with *fiberboard* panel sheathing based on studies by the NAHB Research Center (NAHB, 2005) and by the American Fiberboard Association (PFS, 1996 and NAHB, 2006). The *nominal strength* values for *shear walls* faced with *fiberboard* in Table C2.1-2 were based on monotonic tests of *fiberboard* sheathed, *cold-formed steel* framed *shear walls* and were compared to the monotonic and cyclic tests that are the basis of the building code tabulated capacities for *fiberboard* sheathed, wood framed *shear walls*. For the 2-inch (50.8 mm) and 3-inch (76.2 mm) edge screw spacing, the *nominal strength* values in Table C2.1-2 were based on the average peak load from tests of two 8-foot (2.438 m) wide by 8-foot (2.428 m) tall wall specimens. These *nominal strength* values were found to be within 90 percent of the *nominal strength* values for similarly sheathed wood framed walls. The ratio of steel-to-wood *nominal strength* values increased as the edge (perimeter) fastener spacing increased and, therefore, extrapolating the 2/6 (92% ratio) and 3/6 (96% ratio) design values to 4/6 using a ratio of 90% was conservative. For the 4-inch (101.6 mm) edge screw spacing, the *nominal strength* values were calculated as 90 percent of the *nominal strength* value for a similarly sheathed wood framed wall. For seismic resistance design, shear walls with *fiberboard* panel sheathing are considered "light-framed walls with shear panels of all other materials" in ASCE 7 (ASCE, 2006). AISI S213 restricts use of shear walls with *fiberboard* panel sheathing to *seismic design categories* A, B and C. This same restriction exists for similarly sheathed wood framed walls (AFPA, 2005b).

In Canada: A research program on steel-frame/wood panel *shear walls* was undertaken in 2001 to develop a *shear wall* design method that could be used in conjunction with the provisions of the 2005 *National Building Code of Canada* [*NBCC*] (NRCC 2005). An extensive test program of single-story laterally loaded *shear walls* constructed of Canadian products was first carried out (Branston et al. 2006b). Based on the data obtained from this test program, as well as the wall behavior/performance that was observed (Chen et al. 2006), a design method was developed (Branston 2006a). Shear resistance values for additional wall configurations have been provided by Boudreault (2005), Blais (2006), Rokas (2006) and Hikita (2006). Recommended *nominal resistances* are for walls sheathed with CSP plywood per CSA O151 (CSA, 2003a) or DFP plywood per CSA O121 (CSA, 2003b), or OSB sheathing per CSA O325.0 (CSA, 2003c). Panel edges were fully blocked and sheathing screws were installed such that their heads were flush with the surface of the wood panel. Overdriving of the sheathing screws will result in lower strength, stiffness and ductility of a shear wall compared with the values obtained from testing (Rokas, 2006); hence, sheathing screws are to be firmly driven into framing members but not overdriven into sheathing.

In Canada: *Hold-down anchors* were used to connect the *chord studs* to the test frame in all cases. Built-up *chord members* were incorporated in the construction of test walls in order to prevent failure of the end *stud* due to compression forces exerted by the lateral loading. Monotonic testing (Figure C2-4) was carried out along with reversed cyclic testing, in which the CUREE protocol for ordinary ground motions (Figure C2-5) (Krawinkler et al. 2000; ASTM E2126 2005) was used for the majority of wall specimens (Boudreault 2005). In most cases, six specimens (3 monotonic and 3 reversed cyclic) were tested per wall configuration. A typical shear resistance vs. displacement hysteresis for a reversed cyclic test is provided in Figure C2-4. *Nominal resistance* values for wood sheathed *shear walls* were obtained from the test data using the equivalent energy elastic–plastic (EEEP) analysis approach (Figure C2-6). The concept of equivalent energy was first proposed by Park (1989) and then presented in a modified form by Foliente (1996). A codified version of the equivalent energy elastic–plastic (EEEP) approach to calculating the design parameters of light framed shear walls can also be found in ASTM E2126 (2005).



(a) Monotonic test curve

(b) Cyclic test and backbone curves

Figure C2-4 – Force – Deformation Response of Typical Monotonic and Reversed Cyclic Tests

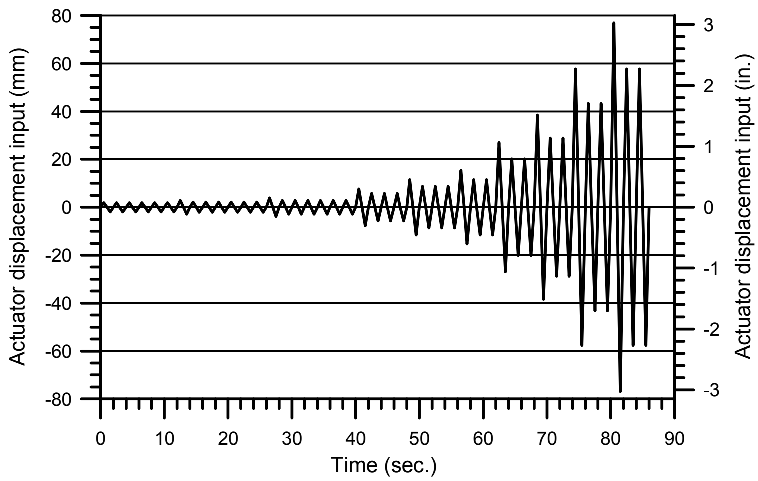


Figure C2-5 – CUREE Reversed Cyclic Test Protocol (0.5 Hz)

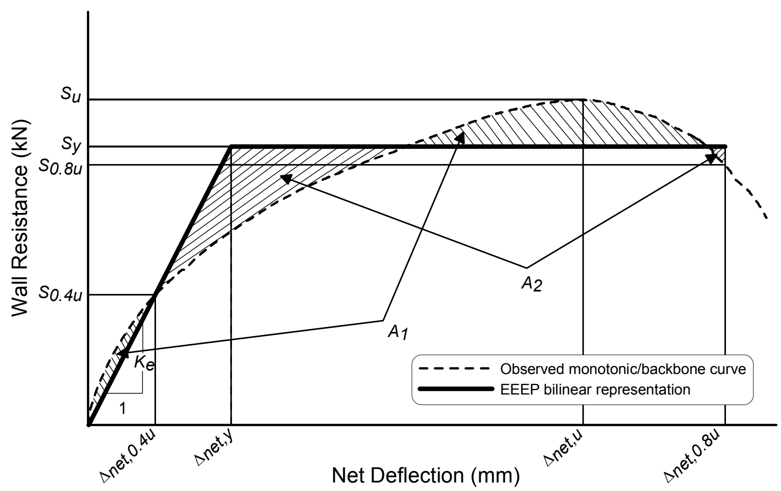


Figure C2-6 – Equivalent Energy Elastic-Plastic (EEEP) Analysis Model

In Canada: The equivalent energy elastic-plastic model is based on the notion that the energy dissipated by the wall specimen during a monotonic or reversed cyclic test is equivalent to the energy represented by a bilinear curve. For simplicity, the model's curve is chosen to be bilinear, which depicts linear elastic behavior of the system until the yield point and perfectly plastic behavior until failure. Wood sheathed shear wall systems tend to fail in a gradual manner, exhibiting an ability to maintain load carrying capacity in the inelastic range of deformation. As well, these shear walls are able to perform reasonably well beyond the peak wall resistance; that is, they do not exhibit a significant or sudden reduction in strength. The failure limit state was defined as the 80% post ultimate load for these reasons, and because of the recommendations found in ASTM E2126 (2005).

In Canada: In the case of each reversed cyclic test, a backbone curve was first constructed for both the positive and negative displacement ranges of the resistance vs. deflection hysteresis. This backbone curve represents the outer envelope of the first loading cycles in the CUREE protocol. The resistance vs. deflection curve for monotonic specimens and the backbone curves for cyclic tests were used to create EEEP curves based on the equivalent energy approach, as illustrated in Figure C2-7. The resulting plastic portion of the bilinear curve was defined as the *nominal resistance*. The 2005 *NBCC* also requires that for seismic design, lateral inelastic deflections be limited to 2.5% of the story height for buildings of normal importance. A limit of 2.5% drift was also used in the energy balance (Branston et al. 2006b). When this inelastic drift limit was incorporated, it had the effect of lowering the recommended *nominal resistance*. *Nominal resistances* were not modified based on a deflection controlled service limit state, such as the h/500 drift limit associated with in-plane wind loading. A typical series of tests (monotonic and backbone) and EEEP curves for a wall configuration is shown in Figure C2-8. Since the CUREE reversed cyclic protocol for ordinary ground motions produces results that are very similar to those revealed by a monotonic test for an identical wall configuration (Chen 2004; Chen et al., 2006), it was decided that the results for the monotonic tests and the reversed cyclic tests would be combined to produce a minimum of six nominal shear values for each wall configuration.

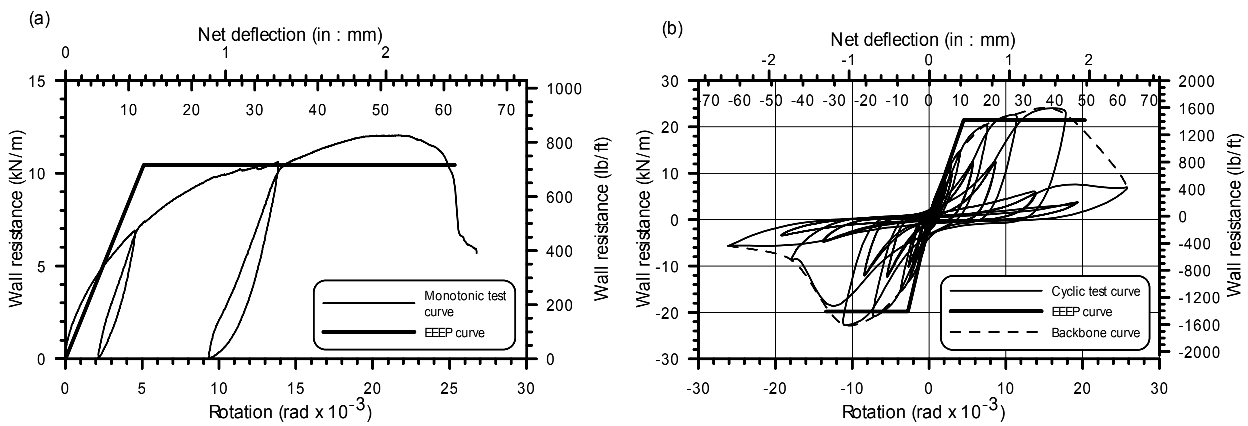
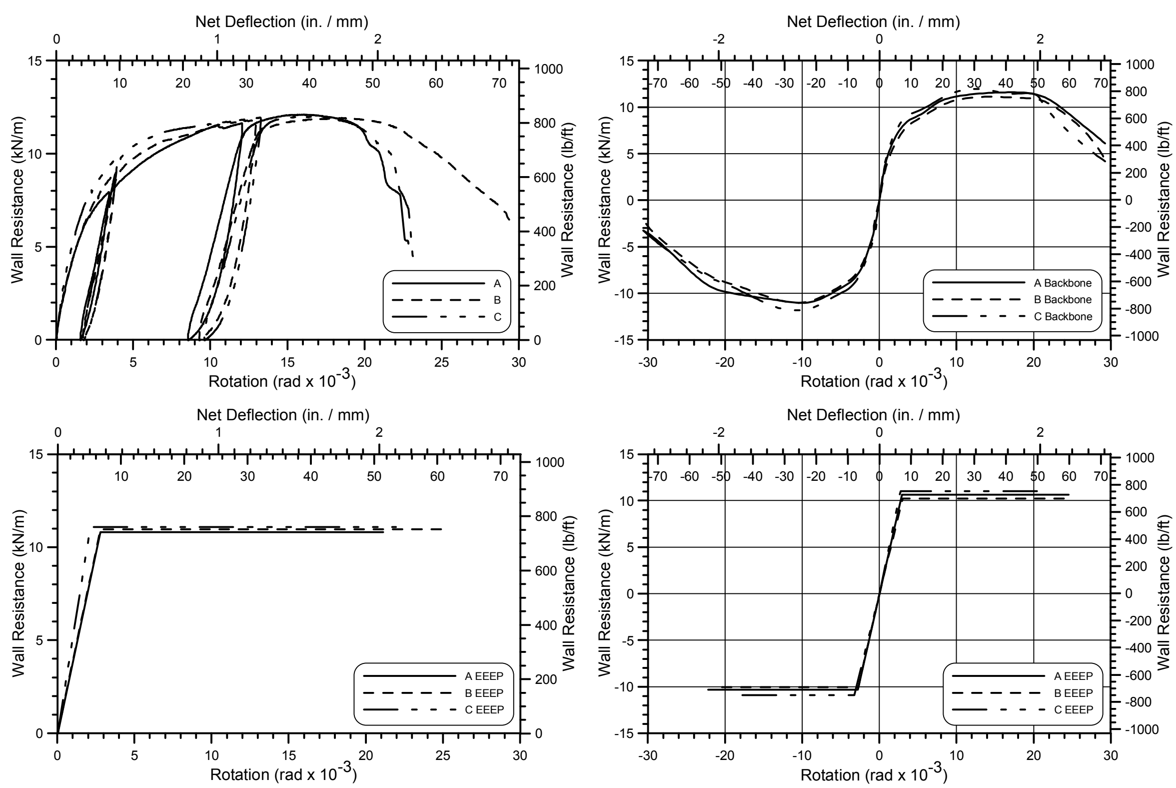


Figure C2-7 – Typical Test and EEEP Curves: (a) Monotonic; (b) Reversed Cyclic



(a) Monotonic test and EEEP curves (b) Cyclic backbone and EEEP curves

Figure C2-8 – Typical Series of Test and EEEP Curves for Monotonic and Reversed Cyclic Tests

In Canada: The recommended *nominal resistance* of the steel-frame/wood panel shear walls were initially developed based on the mean value of the monotonic and reversed cyclic test data for a particular wall configuration. A reduction factor was then determined from the assumed normal statistical distribution of test-to-predicted (mean) results, which made it possible to recommend the fifth percentile results that are tabulated in AISI S213. Use of the fifth percentile approach to determine nominal shear strengths resulted in an average ASD factor of safety of 2.67 (Branston et al., 2006a).

In Canada: Further to this design approach, it is recommended that a factor be included to account for the influence of the duration of the applied load on wood strength. This recommendation is due to the dependence of the shear wall resistance on the sheathing connections, more specifically their capacity in terms of wood bearing and plug shear strength. Since the *shear wall* tests were carried out over a short time span, the tabulated values are for short-term duration loads, including wind and earthquake. In general, wood products exhibit a decreased resistance to long-term loads, and hence the shear resistance should be decreased accordingly for standard and permanent loads. The recommended reduction factors are based upon those used for the design of wood structures in CSA O86 (CSA, 2001), although they have been normalized to the short-term loading case. Justification of the CSA O86 load duration factors can be found in the work of Foschi et al. (1989) and Wood (1960).

In Canada: A *resistance factor* (ϕ) was calibrated according to the *limit state design* procedures prescribed in the 2005 *NBCC*. The calibration equation from AISI S100 [CSA S136] was used, which was based on the work of Ravindra and Galambos (1978). The CSA S408 *Guideline for the Development of Limit States Design* (CSA, 1981) also presents the derivation of a similar equation. A reliability/safety index β_o of 2.5 was used because the recommended nominal design resistances are not the ultimate capacity of the test walls (Fig. C2-7). A ϕ value of 0.7 was obtained for use in the design of *cold-formed steel* framed/wood panel shear walls laterally loaded by 2005 *NBCC* wind forces. It is recommended that the *resistance factor* calculated for the 2005 *NBCC* wind loads also be used in seismic design. This approach is warranted because the *resistance factor* (ϕ) exists in the equations for both the equivalent static earthquake base shear (V) and the factored wall resistance. Where R_o, the overstrength-related force modification factor is a function of R_ϕ, which is equal to $1/\phi$ R_ϕ is included in the definition of R_o because seismic resistant design is based on a return period of 2500 years for the design level earthquake (probability of exceedance of 2% in 50 years) (Mitchell et al. 2003). This represents a rare loading event for which a *nominal resistance*, in place of a *factored resistance*, is considered to be adequate for design. A *resistance factor* of ϕ = 0.7 is therefore recommended for seismic design; first of all to be consistent with the factor calibrated for wind loads, and secondly because this value was used by Boudreault et al. (2007) in the calculation of R_o.

In Canada: The *factor of safety* at the design level of shear resistance was determined using an *allowable stress design* approach. Although not explicitly relied on for *limit states design*, the *factor of safety* was nonetheless determined for each test configuration when a *resistance factor* of 0.7 is used in design. Figure C2-9 illustrates the *factor of safety* in terms of an *LSD* approach. It should be noted that these *factors of safety* apply to the case of lateral loading alone, and hence do not include the effects of gravity loads in combination with lateral loads. The *LSD factor of safety* can be multiplied by the *load factor* for wind loads (1.4) (NRCC 2005) to obtain an equivalent *factor of safety* in *allowable stress design*. When amplified by the *load factor* of 1.4, the *factor of safety* comparable to *allowable stress design* has a mean value of 2.4. Use of the fifth percentile approach to determine nominal shear strengths resulted in an increase of the *ASD factor of safety* from 2.4 to 2.67.

In Canada: *Nominal resistance* values for gypsum sheathed walls were set at 80% of the values currently found in Table C2.1-2. This reduction in resistance level is similar to what is found for the wood sheathed walls of similar construction, i.e. Table C2.1-4 vs. Table C2.1-3.

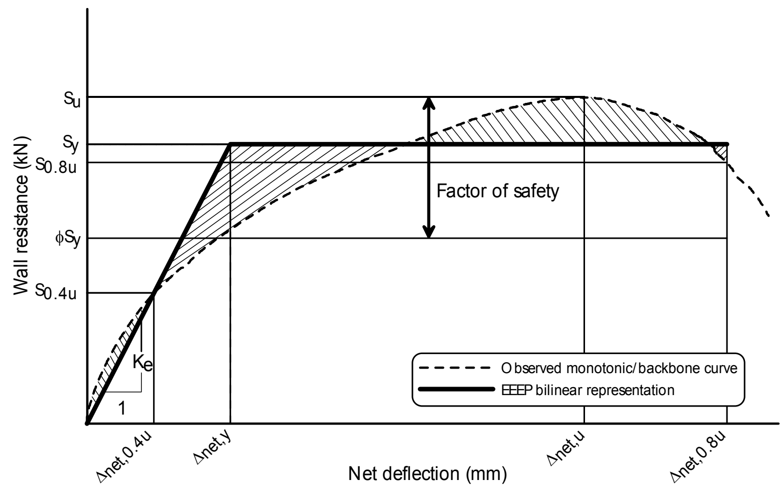


Figure C2-9 – Factor of Safety for Limit States Design

C2.1.1 Design Deflection

The deflection provisions are based on work performed by Serrette and Chau (2003). Equation C2.1-1 may be used to estimate the drift deflection of *cold-formed steel* light-framed *shear walls* recognized in the building codes. The equation should not be used beyond the *nominal strength* values given in AISI S213. The method is based on a simple model for the behavior of *shear walls* and incorporates empirical factors to account for inelastic behavior and effective shear in the sheathing material. Specifically, the model assumes that the lateral deflection (drift) of a wall results from four basic contributions: linear elastic cantilever bending (*boundary member* contribution), linear elastic sheathing shear, a contribution for overall nonlinear effects and a lateral contribution from anchorage/hold-down deformation. These four contributions are additive.

$$\delta = \frac{2vh^3}{3E_sA_cb} + \omega_1\omega_2\frac{vh}{\rho Gt_{sheathing}} + \omega_1^{5/4}\omega_2\omega_3\omega_4\left(\frac{v}{\beta}\right)^2 + \frac{h}{b}\delta_v$$

Linear elastic cantilever bending: $\frac{2vh^3}{3E_sA_cb}$

Linear elastic sheathing shear: $\omega_1\omega_2\frac{vh}{\rho Gt_{sheathing}}$

Overall nonlinear effects: $\omega_1^{5/4}\omega_2\omega_3\omega_4\left(\frac{v}{\beta}\right)^2$

Lateral contribution from anchorage/hold-down deformation: $\frac{h}{b}\delta_v$

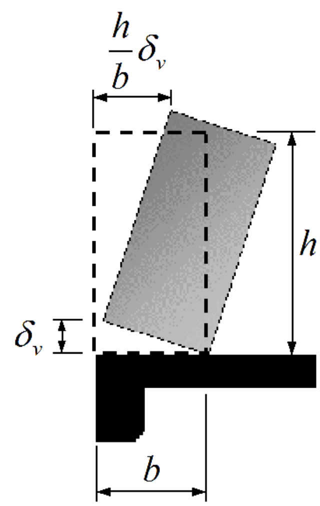


Figure C2-10 – Lateral Contribution from Anchorage/Hold-down Deformation

The lateral contribution from anchorage/hold-down deformation is dependent on the aspect ratio of the wall, as illustrated in Figure C2-4. The empirical factors used in the equation are based on regression and interpolation analyses of the reversed cyclic test data used in development of the *cold-formed steel shear wall* design values. The ρ term in the linear elastic sheathing shear expression attempts to account for observed differences in the response of walls with similar framing, fasteners and fastener schedules, but different sheathing material. Low values of ρ for sheet steel are a result of shear buckling in the sheet. The equations were based on *Type I shear walls* without openings, and the user should use caution if applying them to *Type I shear walls* with openings or to *Type II shear walls*. The *shear wall* deflection equations do not account for additional deflections that may result for other components in a structure (for example, wood sills and raised floors).

For wood structural panels, the shear modulus, G, is not a readily available value, except for Structural I plywood panels in the IBC (ICC, 2003) and UBC (ICBO, 1997) codes. However, the shear modulus may be approximated from the through thickness shear rigidity (G_vt_v), the nominal panel thickness (t) and through thickness panel grade and construction adjustment factor (C_G) provided in the AFPA Manual (AFPA, 2001). For example, G for 7/16-in. 24/16 OSB rated sheathing can be approximated as follows:

G_vt_v (24/16 span rating) = 25,000 lb/inch (strength axis parallel to framing)

t = 0.437 inch (as an approximation for t_v)

C_G = 3.1

G (approximate) = 3.1 x 25,000 / 0.437 = 177,300 psi

Thus, $C_GG_vt_v$ = 77,500 lb/inch and Gt = 77,500 lb/inch

A comparison of the $C_GG_vt_v$ and Gt values suggests that using the nominal panel thickness as an approximation to t_v is reasonable given that the deflection equation provides an estimate of drift.

Currently, the *shear wall* deflection equations do not include provisions for gypsum board or *fiberboard shear walls*. However, the reader is reminded that given the low seismic response modification coefficient, R, assigned by the building codes to gypsum board *shear walls*, it is expected that these systems will perform in the elastic range of behavior and deflections will be less likely to control the design.

In 2009, Standard Equation C2.1-1 for determining the deflection of a blocked wood structural panel was consolidated for U.S. Customary and SI units.

C2.2 Limitations for Tabulated Systems

AISI S213 provides a section on limitations for *shear wall* systems using the *nominal* values in Tables C2.1-1, C2.1-2, C2.1-3, C2.1-4 and C2.1-5. Since the values in these tables are based on test data, it was the intent to provide the user with the limiting values of the tested systems. The intent is not to prevent an engineer from using judgment, the principles of mechanics and supplemental data to develop alternate shear values from those shown in the standard, as discussed in Section B3 above.

For both wood structural panels and sheet steel, aspect ratios up to 4:1 are permitted with reductions in *nominal strength*. The reduced strength values are conservative based on 4:1 aspect ratio tests conducted by Serrette (1997).

It should be noted that flat strapping used as *blocking* to transfer shear forces between sheathing panels is permitted, but is not required to be attached to framing members.

It should also be noted that for wood structural panel, gypsum board and *fiberboard* sheathing, the screws must be installed through the sheathing to the *blocking*. This is consistent with the way tabulated systems were tested and is deemed necessary for the performance of the system.

In addition, sheathing screws should be driven to the proper depth appropriate for the head style used. Bugle, wafer and flat head screws should be driven flush with the surface of the sheathing; pan head, round head, and hex-washer head screws should be driven with the bottom of the head flush with the sheathing. See *Commentary* Section C2.1 for more discussion on overdriving sheathing screws.

C3 Type II Shear Walls

The requirements for *Type II shear walls,* also known as perforated *shear walls*, in AISI S213 were based on provisions in NEHRP (2000) for wood systems. In this method, the shear capacity ratio, F, or the ratio of the strength of a *shear wall* segment with openings to the strength of a fully sheathed wall segment without openings, is determined as follows:

$$F = \frac{r}{3 - 2r}$$

where

$$r = \frac{1}{1 + \frac{A_0}{h \sum L_i}}$$

A_0 = total area of openings

h = height of wall

$\sum L_i$ = sum of the length of full-height sheathing

Research by Dolan (1999, 2000a, 2000b) demonstrated that this design procedure is as valid for steel framed systems as for all wood systems, and the IBC (ICC, 2003) and NFPA 5000 (NFPA, 2003) building codes both permit the use of *Type II shear walls* for steel systems. Test results revealed the conservative nature of predictions of capacity at all levels of monotonic and cyclic loading. AISI S213 does not provide a method or adjustment factor for estimating the lateral displacement of *Type II shear walls*. As such, the user should be cautious if a *Type II shear wall* is used in a deflection sensitive design.

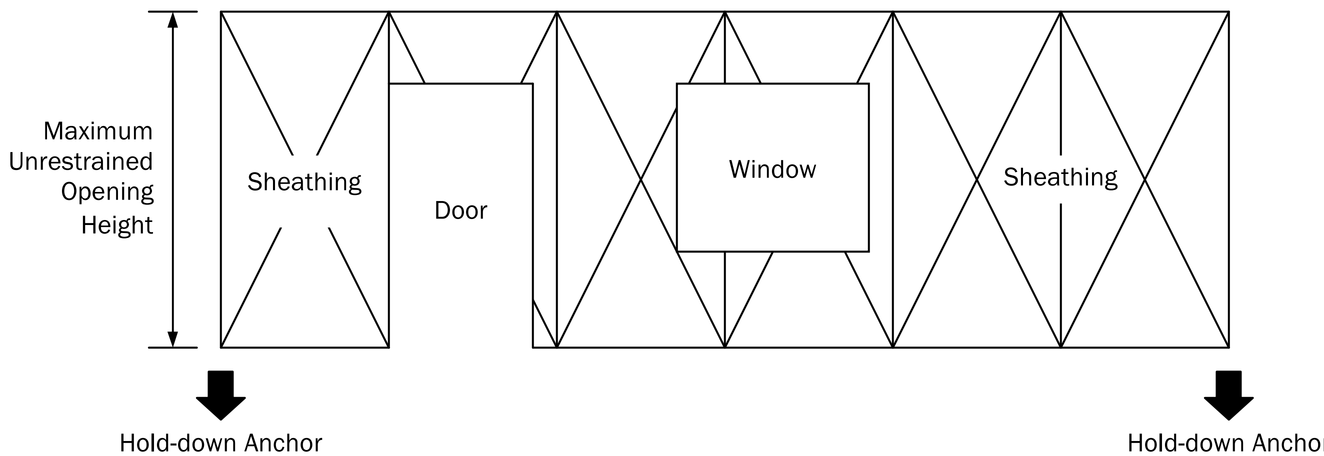


Figure C3-1 – Typical Type II Shear Wall

Table C3.2-1 in the standard, which establishes an adjustment factor for the shear resistance, is based on the methodology described in this section and exists in essentially the same form in both the wood and steel chapters of the IBC (ICC, 2003) building code. There is also a similar table in the AISI *Standard for Cold-Formed Steel Framing - Prescriptive Method*; however, the *Prescriptive Method* establishes an adjustment factor for the *shear wall* length rather than the *shear wall* resistance.

Although the Dolan work was based on structural sheathing, the Committee felt it was appropriate to extend this methodology to *shear walls* with sheet steel panels due to the similar performance of structural sheathing and steel sheet panels in monotonic and cyclic tests (Serrette, 1997) of *Type I shear walls*.

In accordance with standard Section C3.2.3, it is required to check the height/width ratio of each *Type II shear wall segment* and reduce the strength of each segment that has an aspect ratio greater than 2:1, but less than or equal to 4:1 by the factor of 2w/h. It is not required to apply the reduction to all Type II shear wall segments that have an aspect ratio equal to or greater than 2:1. This aspect ratio reduction factor is cumulative with the shear resistance adjustment factor, C_a.

C4 Diagonal Strap Bracing

Where braced walls utilize diagonal *strap bracing*, it is acceptable to compute the deflection of these walls using standard engineering analysis for braced walls. Deflection calculations should consider all elements that contribute to the horizontal top of wall displacement, including axial deformation of the *studs*, elongation of the *straps*, and a lateral contribution from anchorage/hold-down deformation, as well as additional deflections that may result for other components in a structure (for example, wood sills and raised floors). Because loose *straps* permit lateral displacement without resistance, AISI S213 requires that *straps* be installed taut. Also see Section C5.2.4, Additional Requirements.

C5 Special Seismic Requirements

In 2007, Section C5 was reorganized to provide separate sections for *shear walls* and diagonal *strap bracing* to allow a clearer presentation of the special seismic requirements for these unique systems. The special seismic requirements for *shear walls* and diagonal *strap bracing* were based on available data, engineering judgment, industry practice, building code provisions and appropriate limitations to replicate the conditions of the tested assemblies.

As discussed in Section B4, in areas where expected demand from seismic event is high, it is desirable that the lateral resisting elements develop its full range of behavior before failure. As such, the performance of all components related to the overall response of the lateral system become significant.

C5.1 Shear Walls

Section 12.10.2.1 of ASCE 7 (ASCE, 2006) exempts structures or portions thereof that are braced entirely by light-frame shear walls from the requirement to have *collectors*, splices, and connections to resisting elements designed to resist *amplified seismic loads*. Nevertheless, to develop a desirable response, AISI S213 requires that connections for *boundary members* and *collectors* transferring load to and from the *shear wall* be capable of developing the *nominal strength* of the *shear wall* or the expected over-strength of the *shear wall*. This requirement is applicable to splices in *track* that serves as a *boundary member* or *collector*.

It should be noted that the *nominal strengths* shown in Table C2.1-3 are based on a degraded backbone curve determined using the SPD cyclic protocol. It has been noted that the CUREE cyclic protocol may be a better representation of the strength of a light-framed *shear wall,* and it has been observed that strength of steel frame/wood panel shear walls may be 20 percent higher using the CUREE cyclic protocol (Boudreault, 2005). The CUREE protocol has been observed to be fairly close, but less than the monotonic test values. Also, the use of the degraded backbone curve (stabilized curve) compared to first cycle backbone curve yields lower strengths. This would mean that the maximum loads that the system can deliver, for steel frame/wood panel shear walls, could be 20 to 30 percent higher than the *nominal strengths* shown in Table C2.1-3 due to the use of CUREE, rather than SPD combined with the difference between strengths based on first cycle compared to degraded backbone curve.

C5.1.5 Probable Shear Wall Force in Canada

Current Canadian earthquake resistant design requirements incorporate a capacity based approach in which an element (fuse) of the *seismic force resisting system* (*SFRS*) of a structure is designed to dissipate energy (NRCC 2005). That is, at the “Life Safety” level of design, the fuse element must be able to carry seismic loads over extensive inelastic displacements without sudden failure. It is expected that the fuse element will fail in a ductile, stable and predictable manner, at which time it will reach and maintain its maximum load carrying resistance. In a structure that makes use of *cold-formed steel* framed/wood panel shear walls as lateral force resisting elements, the *shear walls* themselves can initially be thought of as the fuse elements which fail in a conventional sense. More specifically, it is the wood sheathing-to-steel framing connections of the *shear* *wall* that have been shown to fail in a ductile fashion, and hence, it is these connections that should be designated as the fuse element because they are able to dissipate energy due to seismic excitation.

The *capacity based design* approach also stipulates that all other elements in the lateral load carrying path must be designed to withstand the probable capacity of the fuse element, which takes into account any overstrength that may exist. In the case of a *cold-formed steel* framed *shear wall*, the *SFRS* elements include the *chord studs*, intermediate *studs*, *hold-down anchors*, *track*, etc.; these elements are designed to carry the probable ultimate capacity of the *shear wal*l while the sheathing-to-framing connections fail in a ductile manner. In order to design the *chord studs* and other elements of the *SFRS,* it is necessary to estimate the probable capacity of the *shear wall* based on a sheathing connection failure mode. This can be achieved by applying an overstrength factor to the *nominal resistance* (Figure C5-1). Comparison of the ultimate test shear resistance with the recommended 5th percentile nominal design resistance provided justification for an overstrength factor of 1.33 for walls sheathed with DFP and OSB, and 1.45 for walls sheathed with CSP panels. Initial selection of the *shear wall* to resist the expected *NBCC* seismic base shear should be based on a *factored resistance*, i.e., the overstrength factor should not be included during wall selection. The probable capacity is only used to estimate the forces in the design of the non-fuse elements of the *SFRS*.

Investigations into the effect of combined gravity and lateral loads on *shear wall* performance by Hikita (2006) have shown that the addition of gravity loads does not change the lateral performance characteristics of a steel-frame/wood panel shear wall if the selection of the *chord studs* is appropriate, i.e., the *chord studs* are designed to resist the compression forces due to gravity loads in combination with the forces associated with the probable ultimate shear capacity of the wall as controlled by sheathing connection failure.

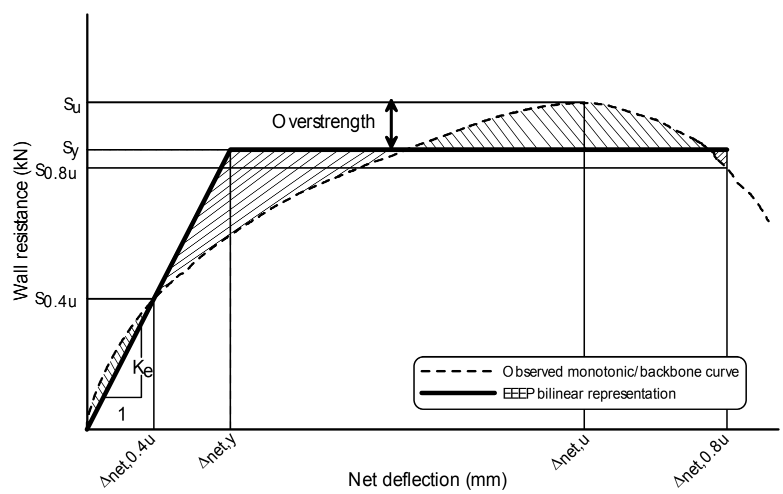


Figure C5-1 – Overstrength in Design

C5.2 Diagonal Strap Bracing

In 2007, additional special seismic requirements for diagonal *strap bracing* were introduced based largely on the research of Rogers at McGill University (Al-Kharat and Rogers, 2005, 2006, 2007), testing by Jim Wilcoski of the U.S. Army Corps of Engineers, and engineering judgment. Pending further research, these provisions were deemed appropriately conservative and necessary to achieve the performance expectations of the building codes.

The factors for expected *yield strength* and tensile strength of the diagonal *strap bracing* member, R_y and R_t, were based on similar values published for hot-rolled structural steel materials (AISC, 2005), results of an in-house study on galvanized sheet steel by a sheet steel producer (see Table C5-1) and engineering judgment.

Table C5-1
Expected Material Properties versus Minimum Specified [1]

Material Property	Grade 33 [230] [2]	Grade 50 [340]
Yield Strength:		
- Minimum Specified	33 ksi [230 MPa]	50 ksi [340 MPa]
- Range	1 to 2	1 to 1 ½
- Typical	1 ½	1 1/8
Tensile Strength:		
- Minimum Specified	45 ksi [310 MPa]	65 ksi [450 MPa]
- Range	1 to 1 ½	1 to 1 ¼
- Typical	1 ¼	1 1/16
Elongation:		
- Minimum Specified	20 percent	12 percent
- Range	1 to 2	1 to 3
- Typical	1 ½	2 ¼

1) Results are based on a 1995 in-house study conducted by Bethlehem Steel for the U.S. Army Corps of Engineers on ASTM A653 (ASTM, 2002) material. In this study, data was gathered from two galvanized coating lines, where the conditions of the lines varied significantly so as to provide a good range of tests 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.
2) Grade 33 data also included some material specified as Grade 40.

AISI S213 allows R_y and R_t to be determined in accordance with an *approved* test method. Such a test method should prescribe a minimum of one tensile test per coil and not permit use of mill test reports. If a test value for R_y is available, the use of the test value is optional if less than the value in Table C5-1; however, the test value must be used if greater than the value in Table C5-1. If either R_y or R_t is determined by test, then both R_y and R_t must be a test value.

C5.2.1 Connections

To develop a desirable response, AISI S213 requires that connections for diagonal *strap bracing* members, top chord splices, boundary members and collectors be capable of developing the expected yield strength of the diagonal *strap bracing* member or, if lower, the expected overstrength (Ω_o times the design seismic load [United States and Mexico] or seismic loads calculated with R_dR_o = 1.0 [Canada]) of the diagonal *strap bracing* member.

The requirements for the alternate connection; i.e., other than welded, were based on engineering judgment and available data. In determining the adequacy of the alternate connection, one must perform capacity based design calculations to determine if cross section yielding of the strap occurs prior to fracture at the net section. This would be the case if $A_gR_yF_y$ were less than $A_nR_tF_u$. Such calculations should use the capacity based design philosophy, and should be in accordance with the applicable building code.

In 2008, findings of a research project at McGill University on the inelastic performance of screw connected *strap* braced walls (Velchev and Rogers, 2008) demonstrated that screw connected walls designed following the capacity design method described in AISI 213 and using steel with at least an F_u/F_y ratio of 1.2 can reach similar inelastic drifts to the weld connected walls.

This 2008 study (Velchev and Rogers, 2008) also demonstrated that the use of reduced width fuse braces makes the brace end connection requirements easier to satisfy; however, the research report outlines some key design aspects to using these braces that need to be considered.

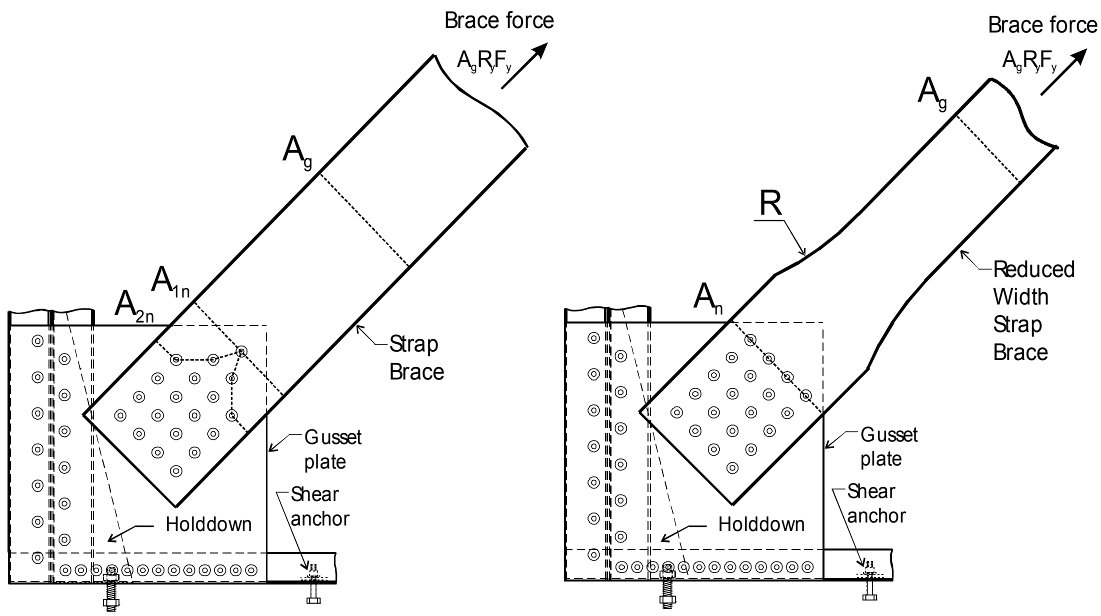


Figure C5-2 – Regular Brace versus Reduced Width Fuse Brace (Velchev and Rogers, 2008)

C5.2.2 Chord Studs and Anchorage of Braced Wall Segments

To develop a desirable response, AISI S213 requires that components transferring *load* to and from the diagonal *strap bracing* member be capable of developing the expected yield strength of the diagonal *strap bracing* member or, if lower, the expected over-strength (Ω_o times the design seismic load [United States and Mexico] or seismic loads calculated with R_dR_o = 1.0 [Canada]) of the diagonal *strap bracing* member.

The standard does not require that the horizontal shear force from the diagonal brace be resisted by a device connected directly to the diagonal brace and anchored directly to the foundation or supporting structural element when the *track* is designed to resist the horizontal shear force by compression or tension because testing (Al-Kharat and Rogers, 2005, 2006, 2007) has shown satisfactory performance of such assemblies.

In 2008, a research project at McGill University on the inelastic performance of screw connected *strap* braced walls (Velchev and Rogers, 2008) investigated various methods of increasing the *track* capacity such that the expected yield strength of the brace can be carried. This study concluded that it was most efficient to use thicker *track*. *Track* that is reinforced requires significant effort in terms of labor, and it is not clear as to the length of *track* that needs to be reinforced, nor the number of connections. Extending the *track* (i.e., using the *track* in tension) may also be a viable solution.

The standard requires that eccentricity be considered in the design where single-sided diagonal *strap bracing* is provided. Single-sided diagonal *strap bracing* causes an eccentric compression force to be applied to the *chord studs*, which results in a strong axis moment in addition to the axial force. The eccentricity is half of the *stud* depth.

C5.2.4 Additional Requirements

To ensure gross cross section yielding of the diagonal *strap bracing* member, AISI S213 requires that the expected yield strength not exceed the expected tensile strength of the diagonal *strap bracing* member. When $A_gR_yF_y$ exceeds $A_nR_tF_u$, a material with a larger ratio of F_u to F_y could be selected or the diagonal strap member could be modified to reduce the ratio of A_g to A_n. It is not considered acceptable to just assume a lower F_y in the calculations.

The slenderness of tension-only diagonal *strap bracing* is not limited because straps are expected to be installed taut and are typically not used in an exposed condition where vibration of the strap may be an issue.

In 2008, findings of a research project at McGill University on the inelastic performance of welded *strap* braced walls (Comeau and Rogers, 2008) demonstrated that the AISI S213 capacity design procedure and material requirements allowed for the desired ductile wall performance (yielding of the braces) to develop in 1:1 and 2:1 aspect ratio walls. However, walls with aspect ratios of 4:1 were observed to be significantly more flexible than the longer walls. Furthermore, they were not able to maintain their yield capacity, and in some cases did not even reach their predicted yield capacity as determined using the brace strength, under lateral loading due to compression/flexure failure of the *chord studs*. At this stage, the use of *strap* braced walls with aspect ratios of 4:1 is not recommended unless a rational analysis is performed to define joint flexibility; end moments in combination with the axial compression force are to be considered in the design of the chord studs.

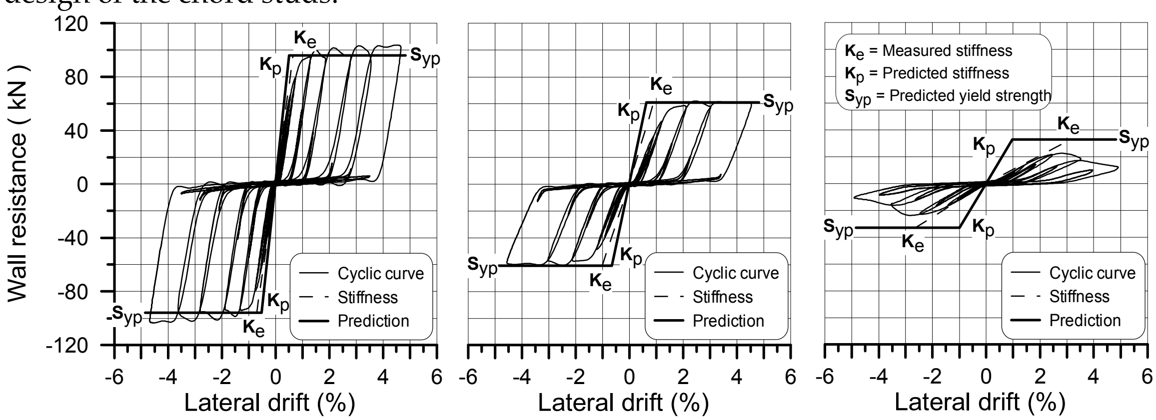


Figure C5-3 – Resistance versus Drift Hystereses for 1:1, 2:1 and 3:1 Aspect Ratio Walls with Same Brace Size (Comeau and Rogers, 2008)

This 2008 study (Comeau and Rogers, 2008) also demonstrated that allowing for supplementary holes in regular braces due to attaching the *straps* with screws to the interior *studs* does not have an adverse impact on the overall ductility. However, strict control was used in the size of the screws (No. 8) and number of screws (1 per brace to interior *stud* connection). The use of multiple screws or screws close to the edge of a brace may reduce the lateral ductility. It is assumed that penetrations in the braces by the use of No. 6 screws for the application of drywall or similar products would not be detrimental given the observed performance of the walls with No. 8 screws installed in the braces. The one exception to this would be the use of screws in the fuse section of a reduced width brace (short fuse section).

D. DIAPHRAGMS

AISI S213 does not currently address the design of *diaphragms* in Canada; however, pending the completion of research that is currently underway, it is expected that the design of *diaphragms* in Canada will be addressed in a future edition of the standard.

D1 General

AISI S213 permits the use of sheet steel, concrete or wood structural panel sheathing or other *approved* materials to serve as the *diaphragm* sheathing.

D1.1 Seismic Requirements for Diaphragms

***In the United States and Mexico*:** When the seismic response modification coefficient, R, is greater than 3, AISI S213 requires that the design must follow the special seismic requirements of Section D3. When R is less than or equal to 3, Section D3 is not required. In addition, AISI S213 is to be read in conjunction with the *applicable building code* documents. Refer to the discussion in *Commentary* Section C1.1.

D2 Diaphragm Design

D2.1 Available Shear Strength

The *available strength* of *diaphragms* is to be based upon principles of mechanics, per section B2. Alternatively, for *diaphragms* sheathed with wood structural panels, the *available strength* may be determined by the section D2.2. The design values for *diaphragms* with wood sheathing in Table D2-1 were based on work by Lum (LGSEA, 1998). Lum developed ASD design tables using an analytical method outlined by Tissell (APA, 1993; APA 2000) for wood framing and the provisions of the 1991 NDS (AFPA, 1991). Because steel is not affected by splitting or tearing when fasteners are closely spaced, no reduction in the calculated strength was taken for closely spaced fasteners. In addition, although steel with *designation thicknesses* greater than 33 mil resulted in higher strength values, no increase in strength was included for these greater thicknesses.

It should be noted that flat strapping used as *blocking* to transfer shear forces between sheathing panels is permitted, but is not required to be attached to framing members.

It should be noted that the *diaphragm* design values by Lum were based on the *nominal strength* of a No. 8 screw attaching wood *structural sheathing* to 33-mil *cold-formed steel* framing members. The 1991 NDS calculation methodology, which was used by Lum, yielded a *nominal strength* of 372 lb and a *safety factor* of 3.3. However, the NDS methodology was revised in 2001, and the revision greatly reduced the calculated strength of screw connections. Until Lum's work is updated, justification for maintaining the current *diaphragm* design values in AISI S213 is based, in part, on tests performed by APA (APA, 2005). Test results for single lap shear tests for a No. 8 screw attaching ½ in. plywood to 68-mil sheet steel indicated that the *nominal strength* of the connection was governed by the strength of the screw in the sheet steel; i.e., the wood *structural sheathing* did not govern the capacity. Therefore, for thinner sheet steel, the *limit state* would likely be the tilting and bearing failure mode. For a No. 8 screw installed in 33-mil sheet steel, computations of connection capacity in accordance with AISI S100 [CSA S136] would yield a *nominal strength* of 492 lbs and a *safety factor* of 3.0. Additionally, connection tests for plywood attached to 33-mil *cold-formed steel* framing members were performed by Serrette (1995b) and produced an average ultimate connection capacity of 1177 lbs, and Serrette suggested the use of a *safety factor* of 6, as given by APA E380D. A review of the *allowable strengths*, as summarized in Table D2-1 below, indicates that although Lum's design values are based on an earlier edition of the NDS, the value is conservative when compared to both AISI and Serrette.

Table D2-1
No. 8 Screw Shear Strength (lbs) for 33-mil Cold-Formed Steel Member

Lum		AISI 2001		Serrette	
Nominal	Allowable	Nominal	Allowable	Nominal	Allowable
372	112	492	164	1177	196

D2.1.1 Design Deflection

The methodology for determining the design deflection of *diaphragms* was based on a comparison of the equations used for estimating the deflection of wood frame *shear walls* and *diaphragms*, coupled with similarities in the performance of *cold-formed steel* and wood frame *shear walls*. Collectively, these comparisons suggested that the wood frame *diaphragm* equation could be adopted, with modifications to account for the difference in fastener performance, for application to *cold-formed steel* light-frame construction. The current equation for wood frame construction (ICC, 2003) is as follows:

$$\Delta = \frac{5vL^3}{8EAb} + \frac{vL}{4Gt} + 0.188Le_n + \frac{\sum(\Delta_c X)}{2b}$$

For SI: $$\Delta = \frac{0.052vL^3}{EAb} + \frac{vL}{4Gt} + \frac{Le_n}{1627} + \frac{\sum(\Delta_c X)}{2b}$$

where

A = Area of *chord* cross section, in square inches (mm^2)

b = *Diaphragm* width, in feet (mm)

E = Elastic modulus of *chords*, in pounds per square inch (N/mm^2)

e_n = Nail deformation, in inches (mm)

G = Modulus of rigidity of wood structural panel, in pounds per square inch (N/mm^2)

L = *Diaphragm* length, in feet (mm)

t = Effective thickness of wood structural panel for shear, in inches (mm)

v = Maximum shear due to design *loads* in the direction under consideration, in pounds per linear foot (N/mm)

Δ = The calculated deflection, in inches (mm)

$\Sigma(\Delta_c X)$ = Sum of individual *chord*-splice values on both sides of the *diaphragm*, each multiplied by its distance from the nearest support

The above equation applies to uniformly nailed, blocked *diaphragms* with a maximum framing spacing of 24 inches (610 mm) on center. For unblocked *diaphragms*, the deflection must be multiplied by 2.50 (APA, 2001). If not uniformly nailed, the constant 0.188 (For SI: 1/1627) in the third term must be modified accordingly.

In 2009, Standard Equation D2.1-1 for determining the deflection of *diaphragms* was consolidated for U.S. Customary and SI units.

D3 Special Seismic Requirements

The special seismic requirements for *diaphragms* were based on engineering judgment, industry practice and code provisions.

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Advisory Note: The Light Gauge Steel Engineers Association (LGSEA) in 2006 changed its name to Cold-Formed Steel Engineers Institute (CFSEI).

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