Standard for Cold-Formed Steel Framing- Lateral Design, 2004 Edition

American Iron and Steel Institute

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AISI STANDARD

Standard for Cold-Formed Steel Framing – Lateral Design, 2004 Edition

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 may eventually become dated. It is anticipated that AISI will publish updates of this material as new information becomes available, but this cannot be guaranteed.

The materials set forth herein are for general purposes only. They are not a substitute for competent professional advice. Application of this information to a specific project should be reviewed by a 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 there from.
PREFACE

The American Iron and Steel Institute (AISI) Committee on Framing Standards (COFS) has developed this *Standard for Cold-Formed Steel Framing – Lateral Design* [Lateral Standard] 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.

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 their appreciation for the support and encouragement of the Steel Framing Alliance.

All terms in this Lateral Standard written in italics are defined in this Lateral Standard or the AISI *Standard for Cold-Formed Steel Framing – General Provisions*. Any listed definitions identified with [reference/year] are defined in the referenced document and listed to ease use of this Lateral Standard.

This 2nd Printing incorporates the Errata to the Standard for Cold-Formed Steel Framing – Lateral Design [Errata], dated April 21, 2005.
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STANDARD FOR COLD-FORMED STEEL FRAMING – LATERAL DESIGN

A. GENERAL

A1 Scope

This Standard for Cold-Formed Steel Framing—Lateral Design, hereinafter referred to as this Lateral Standard, contains design requirements for shear walls, diagonal strap bracing (that is part of a structural wall) and diaphragms. This Lateral Standard is intended for use with Load and Resistance Factor Design (LRFD) or Allowable Strength Design (ASD).

The design of light-framed walls of cold-formed carbon or low-alloy steel to resist wind and seismic loads shall be in accordance with the provisions of the Standard for Cold-Formed Steel Framing – General Provisions [General Provisions], the additional requirements of this Lateral Standard and the applicable building code. Where there is no applicable building code, the loads, load combinations, system limitations and general design requirements shall be taken from ASCE 7. This Lateral Standard does not intend to preclude the use of other materials, assemblies, structures or designs. Where there is a conflict between this Lateral Standard and other reference documents the requirements contained within this Lateral Standard shall govern. This Lateral Standard shall include Sections A through D inclusive.

A2 Definitions

Adjusted Shear Resistance. In Type II shear walls, the unadjusted shear resistance multiplied by the shear resistance adjustment factor.

ASD (Allowable Strength Design). Method of proportioning structural components such that the allowable strength equals or exceeds the required strength of the component under the action of the ASD load combinations. [AISC/AISI Terminology/2004]

ASD Load Combination. Load combination in the applicable building code intended for allowable stress design (allowable strength design). [AISC/AISI Terminology/2004]

Allowable Strength. Nominal strength divided by the safety factor $R_n/\Omega$. [AISC/AISI Terminology/2004]

Allowable Stress Design. Also known as allowable strength design, an out-dated term used in some reference documents.

Applicable Building Code. Building code under which the building is designed. [AISC/AISI Terminology/2004]

Amplified Seismic Load. Horizontal component of seismic load $E$ multiplied by $\Omega_o$, where $E$ and the horizontal component of $E$ are defined in the applicable building code.

Available Strength. Design Strength or allowable strength, as appropriate. [AISC/AISI Terminology/2004]

Blocking. C-shape, track, break shape, or flat strap material attached to structural members, flat strap or sheathing panels to transfer shear forces. [Prescriptive Method/2001]

Boundary Member. Diaphragm and shear wall boundary member to which the diaphragm transfers forces. Boundary members include chords and drag struts at diaphragm and shear wall perimeters, interior openings, discontinuities and re-entrant corners. (ASCE 7/1998)
Chord. Member of a shear wall or diaphragm that forms the perimeter, interior opening, discontinuity or re-entrant corner.

Collector Also known as a drag strut, member that serves to transfer forces between diaphragms and members of the lateral force resisting system.

Component. See structural component.

Connection. Combination of structural elements and joints used to transmit forces between two or more members. [AISC/AISI Terminology/2004]

Design Load. Applied load determined in accordance with ASD load combinations or LRFD load combinations, whichever is applicable. [AISC/AISI Terminology/2004]

Design Strength. Resistance Factor multiplied by the nominal strength, $\phi R_n$. [AISC/AISI Terminology/2004]

Diaphragm. Roof, floor or other membrane or bracing system that transfers in-plane forces to the lateral force resisting system. [AISC/AISI Terminology/2004]

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

Load. Force or other action that results from the weight of building materials, occupants and their possessions, environmental effects, differential movement, or restrained dimensional changes. [AISC/AISI Terminology/2004]

Load Effect. Forces, stresses, and deformations produced in a structural component by the applied loads. [AISC/AISI Terminology/2004]

Load Factor. Factor that accounts for deviations of the actual load from the nominal load, for uncertainties in the analysis that transforms the load into a load effect, and for the probability that more than one extreme load will occur simultaneously. [AISC/AISI Terminology/2004]

LRFD (Load and Resistance Factor Design). Method of proportioning structural components such that the design strength equals or exceeds the required strength of the component under the action of the LRFD load combinations. [AISC/AISI Terminology/2004]


Nominal Load. Magnitude of the load specified by the applicable building code. [AISC/AISI Terminology/2004]

Nominal Strength. Strength of a structure or component (without the resistance factor or safety factor) to resist the load effects, as determined in accordance with this Specification. [AISC/AISI Terminology/2004]

Resistance Factor ($\phi$). Factor that accounts for unavoidable deviations of the actual strength from the nominal strength and for the manner and consequences of failure. [AISC/AISI Terminology/2004]

Safety Factor ($\Omega$). Factor that accounts for the desired level of safety, including deviations of the actual load from the nominal load and uncertainties in the analysis that transforms the load into a load effect, in determining the nominal strength and for the manner and consequences of failure. [AISC/AISI Terminology/2004]

Seismic Design Category (SDC). Classification assigned to a building based upon its importance
and the severity of the design earthquake ground motion at the building site as given in the applicable building code.

Shear Wall. Wall that provides resistance to lateral loads in the plane of the wall and provides stability for the structural system. [AISC/AISI Terminology/2004]

Strap Bracing. Steel straps, applied diagonally to form a vertical truss that form part of the lateral force resisting system.

Strength Design. Also known as load and resistance factor design, an out-dated term used in some reference documents.

Structural Component. Member, connector, connecting element or assemblage. [AISC/AISI Terminology/2004]

Type I Shear Wall. Wall designed to resist in-plane lateral forces that is fully sheathed and that is provided with hold-down anchors at each end of the wall segment. Type I shear walls are only permitted to have openings where detailing for force transfer around the openings is provided.

Type II Shear Wall. Wall designed to resist in-plane lateral forces that is sheathed with wood structural panels or sheet steel that contains openings, but which has not been specifically designed and detailed for force transfer around wall openings. Hold-down anchors for Type II shear walls are only required at the ends of the wall.

Type II Shear Wall Segment. Section of shear wall (within a Type II shear wall) with full-height sheathing and which meets the aspect ratio limits of Section C3.2.3.

Unadjusted Shear Resistance. In Type II shear walls the unadjusted shear resistance is equal to the available shear strength for Type I shear walls as determined by the provisions in Section C2.1 provided the wall segments meet the aspect ratio limitations in Section C3.2.3.

A3 Symbols and Notations

C = Boundary chord force (tension/compression) (lbs, kN)
Ca = Shear resistance adjustment factor from Table C3.2-1
E = Effect of horizontal and vertical seismic forces as defined in the applicable building code
Li = Width of Type II shear wall segment (inches, mm)
R = Seismic response modification coefficient as defined by the applicable building code
Rn = Nominal strength
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
ΣLi = 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
Ω = Safety factor to be used in determining the allowable strength in ASD
Ωo = System overstrength factor as defined by the applicable building code
A4 Referenced Documents

The following documents are referenced in this Lateral Standard:


3. ASCE 7-02, Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, Reston, VA.

4. ASTM C954-00, 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.

5. ASTM A1003/A1003M-02a, Standard Specification for Steel Sheet, Carbon, Metallic- and Nonmetallic-Coated for Cold-Formed Framing Members, ASTM International, West Conshohocken, PA.

6. DOC PS 1-95, Construction and Industrial Plywood, United States Department of Commerce, National Institute of Standards and Technology, Gaithersburg, MD.

7. DOC PS 2-93, Performance Standard for Wood-Based Structural-Use Panels, United States Department of Commerce, National Institute of Standards and Technology, Gaithersburg, MD.
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 the North American Specification for the Design of Cold Formed Steel Structural Members [Specification] and the General Provisions and the referenced documents except as modified by the provisions of this Lateral Standard.

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

B2 Design Loads

The design loads and load combinations shall be determined in accordance with the applicable building code.

B3 Shear Resistance Based on Principles of Mechanics

The shear resistance of diaphragms, 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 so calculated defines the maximum resistance the diaphragm, shear wall, or strap bracing is capable of developing. Available strength shall be computed based on the wind and seismic force requirements in the applicable building code. Calculated values for systems defined in this Lateral Standard shall be scaled to the values in this Lateral Standard.

B4 Framing and Anchorage

Boundary members, chords, collectors and connections thereto shall be proportioned to transmit the induced forces and, where required, the amplified seismic loads.
C. WALLS

C1 General

The design of shear walls or systems using strap bracing that resist lateral forces from wind or seismic loads 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 Section C3.

C1.1 Seismic Requirements for Shear Walls

Where permitted by the applicable building code, when the seismic response modification coefficient, R, (for steel systems) is taken equal to or less than 3, the design shall comply with these provisions exclusive of Sections C5 and D3.

Where the seismic response modification coefficient, R, is taken greater than 3, the design shall comply with these provisions including Section C5 and D3.

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 are only permitted to have openings where detailing for force transfer around the openings is provided, as provided by this section.

The nominal shear strength for Type I shear walls, as shown in Table C2.1-1 for wind loads or Table C2.1-2 for wind and seismic loads or Table C2.1-3 for seismic loads, are permitted to establish available strength for such walls. The available strength shall be determined using the safety factor (Ω) or the resistance factor (φ) as set forth in Section C2.1.

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

The height to width aspect ratio (h/w) of a wall pier at the side of each opening shall be limited to a maximum of 2:1. The height of the wall pier (h) shall be defined as the clear height of the pier at the side of an opening. The width of a pier (w) shall be defined as the sheathed width of the pier. The width of wall piers shall not be less than 24 inches.

The aspect ratio (h/w) of the Type I shear wall shall not exceed the values in Tables C2.1-1, C2.1-2 and C2.1-3. Where the aspect ratio (h/w) exceeds 2:1 the available shear strength shall be adjusted as required by Section C2.1.

Setbacks of structural walls shall not exceed the web depth of the floor joist except when designed for the additional loads, but in no case shall the setback exceed four times the web depth of the member.

C2.1 Available Shear Strength.

Where allowable strength design (ASD) is used, the available strength shall be determined by dividing the nominal shear strength, shown in Table C2.1-1, C2.1-2 and C2.1-3, by a safety factor (Ω) of 2.5 for assemblies resisting seismic loads and 2.0 for assemblies resisting wind loads.

Where load and resistance factor design (LRFD) is used, the available strength shall be determined by multiplying the nominal shear strength, shown in Tables C2.1-1, C2.1-2 and
C2.1-3, by a resistance factor ($\phi$) of 0.60 for assemblies resisting seismic loads and 0.65 for assemblies resisting wind loads.

Where a height to width aspect ratio (h/w) of a shear wall segment is greater than 2:1, as permitted in Tables C2.1-1, C2.1-2 and C2.1-3, the available shear strength 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 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 applied to opposite faces of the same wall the available shear strength of material of the same capacity is cumulative. Where the material nominal shear strengths are not equal the available shear strength shall be either two times the available shear strength of the material with the smaller value or shall be taken as the value of the stronger side, whichever is greater. Summing the available shear strengths 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 is permitted to be calculated according to the following:

$$\delta = \frac{8vh^3}{E_s A_c b} + \omega_1 \omega_2 \frac{vh}{\rho G t_{\text{sheathing}}} + \omega_1^{5/4} \omega_2 \omega_3 \left( \frac{v}{\beta} \right)^2 + \delta_a$$

(Eq. C2.1-1)

For SI:$$\delta = \frac{2vh^3}{3E_s A_c b} + \omega_1 \omega_2 \frac{vh}{\rho G t_{\text{sheathing}}} + \omega_1^{5/4} \omega_2 \omega_3 \left( \frac{v}{0.00290\beta} \right)^2 + \delta_a$$

(Eq. C2.1-2)

where:

- $A_c$ = Gross cross-sectional area of chord member, in square inches (mm$^2$)
- $b$ = Width of the shear wall, in feet (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 feet (mm)
- $s$ = Maximum fastener spacing at panel edges, in inches (mm)
- $t_{\text{sheathing}}$ = Nominal panel thickness, in inches (mm)
- $t_{\text{stud}}$ = Framing designation thickness, in inches (mm)
- $v$ = Shear demand ($V/b$), in pounds per linear foot (N/mm)
- $V$ = Total lateral load applied to the shear wall, in pounds (N)
- $\beta$ = 810 for plywood and 660 for OSB
- $\delta$ = Calculated deflection, in inches (mm)
- $\delta_a$ = Deflection due to anchorage/attachment details, in inches (mm)
- $\rho$ = 1.85 for plywood and 1.05 for OSB
- $\omega_1$ = $s/6$ (for s in inches) and $s/152.4$ (for s in mm)
- $\omega_2$ = 0.033/ $t_{\text{stud}}$ (for $t_{\text{stud}}$ in inches) and 0.838/ $t_{\text{stud}}$ (for $t_{\text{stud}}$ in mm)
\[ \omega_3 = \sqrt{\frac{bh}{2}} \]
\[ \omega_4 = 1 \text{ for wood structural panels} \]
\[ = \sqrt{\frac{33}{F_y}} \text{ (for } F_y \text{ in ksi) and } = \sqrt{\frac{227.5}{F_y}} \text{ (for } F_y \text{ in MPa) for sheet steel} \]

### C2.2 Limitations for Systems in Tables C2.1-1, C2.1-2 and C2.1-3

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

1. **Studs** shall be C-shape members with a minimum thickness of 33-mils, flange width of 1-5/8 inches (41.3 mm), web depth of 3 ½ inches (89 mm) and an edge stiffener of 3/8 inches (9.5 mm).
2. **Track** shall be a minimum thickness of 33-mils with a flange width of 1-1/4 inches (31.8 mm) and a web depth of 3-1/2 inches (89 mm).
3. Fasteners along the edges in shear panels shall be placed not less than 3/8 inches (9.5 mm) in from panel edges.
4. Panel thicknesses shown are minimums.
5. Panels less than 12 inches (305 mm) wide shall not be used.
6. Maximum framing spacing shall be 24 inches (610 mm) on center.

### C2.2.1 Sheet Steel Sheathing

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

1. Steel sheets shall have a minimum base metal thickness as shown in Tables C2.1-1 or C2.1-3, and shall be of the following grade of structural quality steel: ASTM A1003 Grade 33 Type H.
2. Nominal shear strengths, used to establish the available shear strengths, are given in Tables C2.1-1 for wind loads and Table C2.1-3 for seismic loads. Table C2.1-3 shall also be permitted for calculating the nominal shear strength for wind loads.
3. Steel sheets are permitted to be applied either parallel to or perpendicular to framing.
4. In lieu of blocking, panel edges are 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 or seismic loads subject to the following:

1. Wood structural panels shall comply with DOC PS 1 or PS 2 and shall be manufactured using exterior glue.
2. Nominal shear strengths, used to establish the available shear strengths, are given in Tables C2.1-1, for wind loads and Table C2.1-3, for seismic loads. Table C2.1-3 shall also be permitted for calculating the nominal shear strength for
wind loads.

3. Structural panels are 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, flat-head self-drilling tapping screws with a minimum head diameter of 0.285 inch (7.24 mm) or No. 10, flat-head self-drilling tapping screws with a minimum head diameter of 0.333 inch (8.46 mm), in accordance with Table C2.1-3.
5. 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).
6. Increases of the nominal loads 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. Nominal shear strengths, used to establish the available shear strengths, are given in Tables C2.1-2.
2. The available shear strengths determined from the values listed in Table C2.1-2 shall not be cumulative with the available shear strengths of other (dissimilar) materials applied to the same wall unless otherwise permitted herein.
3. Screws used to attach gypsum board shall be a minimum No. 6 in accordance with ASTM C954.

<table>
<thead>
<tr>
<th>Assembly Description</th>
<th>Maximum Aspect Ratio (h/w)</th>
<th>Fastener Spacing at Panel Edges&lt;sup&gt;2&lt;/sup&gt; (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>6</td>
</tr>
<tr>
<td>15/32” Structural 1 sheathing (4-ply), one side</td>
<td>2:1</td>
<td>1065</td>
</tr>
<tr>
<td>7/16” rated sheathing (OSB), one side</td>
<td>2:1</td>
<td>910</td>
</tr>
<tr>
<td>7/16” rated sheathing (OSB), one side oriented perpendicular to framing</td>
<td>2:1</td>
<td>1020</td>
</tr>
<tr>
<td>7/16” rated sheathing (OSB), one side</td>
<td>2:1</td>
<td>-</td>
</tr>
<tr>
<td>0.018” steel sheet, one side</td>
<td>2:1</td>
<td>485</td>
</tr>
<tr>
<td>0.027” steel sheet, one side</td>
<td>2:1</td>
<td>-</td>
</tr>
</tbody>
</table>

1) Nominal shear strengths shall be multiplied by the resistance factor (φ) to determine design strength or divided by the safety factor (Ω) to determine allowable shear 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, are permitted provided the nominal shear strength is multiplied by 2w/h. See Section C2.1.

6) Shear values 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 SI: 1" = 25.4 mm, 1 foot = 0.305 m, 1 lb = 4.45 N

### TABLE C2.1-2
**Nominal Shear Strength, \((R_n)\), for Wind and Seismic Loads for Shear Walls Faced with Gypsum Board**

(Pounds Per Foot)

<table>
<thead>
<tr>
<th>Wall Construction</th>
<th>Max. Aspect Ratio (h/w)</th>
<th>Orientation</th>
<th>Screw Spacing (inches o.c.)</th>
<th>Nominal Shear Strength (lb/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>½&quot; gypsum board on one side of wall; studs max. 24&quot; o/c</td>
<td>2:1</td>
<td>Gypsum board applied perpendicular to framing with strap blocking behind the horizontal joint and with solid blocking between the first two end studs or applied vertically with all edges attached to framing members</td>
<td>7 Edge 7 Field</td>
<td>290</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>4 Edge 4 Field</td>
<td>425</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>4 Edge 12 Field</td>
<td>295</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8 Edge 12 Field</td>
<td>230</td>
</tr>
</tbody>
</table>

1. *Nominal* shear strengths shall be multiplied by the resistance factor \((\phi)\) to determine design strength or divided by the safety factor \((\Omega)\) to determine allowable shear strengths 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 are permitted provided the nominal shear strength values above are multiplied by 0.35.
4. For SI: 1" = 25.4 mm, 1 foot = 0.305 m, 1 lb = 4.45 N

### TABLE C2.1-3
**Nominal Shear Strength, \((R_n)\), for Seismic Loads for Shear Walls**

(Pounds Per Foot)

<table>
<thead>
<tr>
<th>Assembly Description</th>
<th>Max. Aspect Ratio (h/w)</th>
<th>Fastener Spacing at Panel Edges (inches)</th>
<th>Designation Thickness of Stud and Track (mils)</th>
<th>Required Sheathing Screw Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>15/32&quot; Structural 1 sheathing (4-ply), one side</td>
<td>2:1&lt;sup&gt;3&lt;/sup&gt;</td>
<td>780 990 1377 2190</td>
<td>33 or 43</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>2:1</td>
<td>890 1330 1775</td>
<td>43 or 54</td>
<td>8</td>
</tr>
<tr>
<td>7/16&quot; OSB, one side</td>
<td>2:1&lt;sup&gt;3&lt;/sup&gt;</td>
<td>700 915 915 915</td>
<td>33</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>2:1&lt;sup&gt;3&lt;/sup&gt;</td>
<td>825 1000 1545 2060</td>
<td>43 or 54</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>2:1</td>
<td>940 1410 1760 2350</td>
<td>54</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>2:1</td>
<td>1232 1848 2310 3080</td>
<td>68</td>
<td>10</td>
</tr>
<tr>
<td>0.018&quot; steel sheet, one side</td>
<td>2:1</td>
<td>390 - - -</td>
<td>33 (min.)</td>
<td>8</td>
</tr>
<tr>
<td>0.027&quot; steel sheet, one side</td>
<td>4:1</td>
<td>- 1000 1085 1170</td>
<td>33 (min.)</td>
<td>8</td>
</tr>
</tbody>
</table>

1. *Nominal* shear strength shall be multiplied by the resistance factor \((\phi)\) to determine design strength or divided by the safety factor \((\Omega)\) to determine allowable shear 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, are permitted provided the nominal shear strength 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, per the General Provisions, is not permitted.
6. Wall studs and track shall be of ASTM A1003 Grade 33 Type H steel for members with a designation thickness of 33 and 43 mil, and A1003 Grade 50 Type H steel for members with a designation thickness equal to or greater than 54 mils.
7. For SI: 1" = 25.4 mm, 1 foot = 0.305 m, 1 lb = 4.45 N

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. In other than Seismic Design Category A the nominal shear strength shall be based upon edge screw spacing not less than 4 inches o.c.
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.

C3.2 Type II Shear Wall—Design Shear Resistance

The available strength 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) 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) 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 shear strength calculated in accordance with Section C2.1, based upon the nominal strengths (Rn) in Tables C2.1-1 and C2.1-3. The aspect ratio (h/w) of Type II shear wall segments used in calculations shall not exceed 2:1.

Exception: Where permitted by Tables C2.1-1 and C2.1-3, the aspect ratio (h/w) of Type II wall segments greater than 2:1 but in no case greater than 4:1, shall be permitted to be included in the calculation of the unadjusted shear resistance provided the shear values
in that segment are multiplied by \(2w/h\).

### 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

<table>
<thead>
<tr>
<th>Wall Height (h)</th>
<th>Maximum Opening Height Ratio (^1) and Height</th>
<th>(h/3)</th>
<th>(h/2)</th>
<th>(2h/3)</th>
<th>(5h/6)</th>
<th>(h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8'-0&quot; (244 mm)</td>
<td>2'-8&quot; (810 mm) 4'-0&quot; (1220 mm) 5'-4&quot; (1630 mm) 6'-8&quot; (2030 mm) 8'-0&quot; (2440 mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10'-0&quot; (3050 mm)</td>
<td>3'-4&quot; (1020 mm) 5'-0&quot; (1530 mm) 6'-8&quot; (2030 mm) 8'-4&quot; (2540 mm) 10'-0&quot; (3050 mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Percent Full-Height Sheathing (^2)</th>
<th>Shear Resistance Adjustment Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>10%</td>
<td>1.00</td>
</tr>
<tr>
<td>20%</td>
<td>1.00</td>
</tr>
<tr>
<td>30%</td>
<td>1.00</td>
</tr>
<tr>
<td>40%</td>
<td>1.00</td>
</tr>
<tr>
<td>50%</td>
<td>1.00</td>
</tr>
<tr>
<td>60%</td>
<td>1.00</td>
</tr>
<tr>
<td>70%</td>
<td>1.00</td>
</tr>
<tr>
<td>80%</td>
<td>1.00</td>
</tr>
<tr>
<td>90%</td>
<td>1.00</td>
</tr>
<tr>
<td>100%</td>
<td>1.00</td>
</tr>
</tbody>
</table>

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 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:

\[
v = \frac{V}{C_a \Sigma L_i}
\]

(Eq. C3.3-1)

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
- \(\Sigma L_i\) = 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. Where seismic loads govern, the uplift anchorage and boundary chords shall also comply with the requirements of Section C5.3.

\[
C = \frac{Vh}{C_a \sum L_i}
\]  

(Eq. 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
- \(\sum L_i\) = 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 Strap Bracing

Strap bracing, as part of a structural wall, is permitted to resist wind and seismic forces and shall be designed in accordance with the Specification and General Provisions.

C5 Special Seismic Requirements

C5.1 General

Where the seismic response modification coefficient, \(R\), used to determine the lateral forces is taken greater than three, the requirements of this section shall apply in addition to the requirements of Sections C2, C3 and C4.

C5.2 Connections

The required strength of connections for diagonal strap bracing members, top chord splices, boundary members and collectors shall be the lesser of the nominal tensile strength of the member or amplified seismic load. The pull-out resistance of screws shall not be used to resist seismic forces.

C5.3 Anchorage of Braced Wall Segments

Studs or other vertical boundary members at the ends of wall segments, that resist seismic loads, braced with either sheathing or diagonal braces, shall be anchored such that the bottom track is not required to resist uplift by bending of the track web. Both flanges of the studs shall be braced to prevent lateral torsional buckling. Studs or other vertical boundary members and anchorage thereto shall have the nominal strength to resist amplified seismic loads,
but need not be greater than the loads that the system can deliver. Foundations need not be designed to resist loads resulting from the amplified seismic loads.

**C5.4 Sheet Steel and Wood Sheathing**

Where steel or wood sheathing provide lateral resistance, the design and construction of such walls shall be in accordance with the additional requirements of this section. Perimeter members at openings shall be provided and shall be detailed to distribute the shearing stresses. Wall studs and track shall be of ASTM A1003 Grade 33 Type H steel for members with a designation thickness of 33 and 43 mil, and A1003 Grade 50 Type H steel for members with a designation thickness equal to or greater than 54 mils, and shall have a designation thickness, per the General Provisions, conforming to Table C2.1-3. 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.

**C5.5 Diagonal Strap Bracing**

Where diagonal strap bracing is provided for lateral resistance, provisions shall be made for pretensioning, or other methods of installing tension-only bracing shall be used to guard against loose diagonal straps. The slenderness ratio of the brace shall be permitted to exceed 200.
D. DIAPHRAGMS

D1 General

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

D2 Diaphragm Design

D2.1 Available Shear Strength

The available shear strength of diaphragms shall be determined in accordance with Section B3. Alternatively for diaphragms sheathed with wood structural panels the available shear 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 is permitted to be calculated by the use of the following formulas:

\[
\delta = \frac{5vL^3}{8E_cA_c b} + \frac{vL}{\rho G t_{\text{sheathing}}} + \frac{v}{G t_{\text{sheathing}}} \sum_{j=1}^{n} \Delta_{ci} X_i \left( \alpha \left( \frac{v}{2\beta} \right)^2 + \frac{\sum_{j=1}^{n} \Delta_{ci} X_i}{2b} \right) \quad (\text{Eq. D2.1-1})
\]

For SI:

\[
\delta = \frac{0.052vL^3}{E_c A_c b} + \frac{vL}{\rho G t_{\text{sheathing}}} + \frac{v}{G t_{\text{sheathing}}} \sum_{j=1}^{n} \Delta_{ci} X_i \left( \frac{v}{0.00579\beta} \right)^2 + \frac{\sum_{j=1}^{n} \Delta_{ci} X_i}{2b} \quad (\text{Eq. D2.1-2})
\]

where:

- \( A_c \) = Gross cross-sectional area of chord member, in square inches (mm²)
- \( b \) = Diaphragm depth parallel to direction of load, in feet (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 feet (mm)
- \( n \) = Number of chord splices in the diaphragm (considering both diaphragm chords)
- \( s \) = Maximum fastener spacing at panel edges, in inches (mm)
- \( t_{\text{sheathing}} \) = Nominal panel thickness, in inches (mm)
- \( t_{\text{stud}} \) = Nominal framing thickness, in inches (mm)
- \( v \) = Shear demand \((V/2b)\), in pounds per linear foot (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 feet (mm)
- \( \alpha \) = Ratio of the average load per nail based on a non-uniform nail pattern to the average load per nail based on a uniform nail pattern (= 1 for a uniformly fastened diaphragm)
- \( \beta \) = 810 for plywood and 660 for OSB
- \( \delta \) = Calculated deflection, in inches (mm)
- \( \Delta_{ci} \) = Deformation value associated with “ith” chord splice, in inches (mm)
\[ \rho = 1.85 \text{ for plywood and 1.05 for OSB} \]

\[ \omega_1 = \frac{s}{6} \text{ (for } s \text{ in inches)} \]
\[ \omega_2 = \frac{s}{152.4} \text{ (for } s \text{ in mm)} \]

\[ \omega_2 = \frac{0.033}{t_{\text{stud}}} \text{ (for } t_{\text{stud}} \text{ in inches)} \]
\[ \omega_2 = \frac{0.838}{t_{\text{stud}}} \text{ (for } t_{\text{stud}} \text{ in mm)} \]

For unblocked diaphragms, \( \delta \) shall be multiplied by 2.50.

### D2.2 Wood Diaphragms

The *nominal* shear strength of wood structural panel *diaphragms*, used to determine the *available* shear 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* shear strength shall be determined by dividing the *nominal* shear strength, shown in Table D2-1, by a *safety factor* (\( \Omega \)) of 2.5 for assemblies resisting seismic loads and 2.0 for assemblies resisting wind loads.

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

#### Table D2-1

**NOMINAL SHEAR STRENGTH FOR DIAPHRAGMS WITH WOOD SHEATHING**

(Pounds Per Foot)

<table>
<thead>
<tr>
<th>Membrane Material</th>
<th>Screw Size</th>
<th>Thickness (in)</th>
<th>Screw spacing at diaphragm boundary edges and at all continuous panel edges</th>
<th>Screws spaced maximum of 6” on all supported edges</th>
<th>Load perpendicular to unblocked edges and continuous panel joints</th>
<th>All other configurations</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>6</td>
<td>4</td>
<td>2.5</td>
<td>2</td>
</tr>
<tr>
<td>Structural I</td>
<td>See note 2</td>
<td>3/8</td>
<td>768</td>
<td>1022</td>
<td>1660</td>
<td>2045</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7/16</td>
<td>768</td>
<td>1127</td>
<td>1800</td>
<td>2255</td>
</tr>
<tr>
<td></td>
<td></td>
<td>15/32</td>
<td>925</td>
<td>1232</td>
<td>1970</td>
<td>2465</td>
</tr>
<tr>
<td>C-D, C-C</td>
<td>See note 2</td>
<td>3/8</td>
<td>690</td>
<td>920</td>
<td>1470</td>
<td>1840</td>
</tr>
<tr>
<td>and other graded</td>
<td></td>
<td>7/16</td>
<td>760</td>
<td>1015</td>
<td>1620</td>
<td>2030</td>
</tr>
<tr>
<td>wood structural</td>
<td></td>
<td>15/32</td>
<td>832</td>
<td>1110</td>
<td>1770</td>
<td>2215</td>
</tr>
<tr>
<td>panels in DOC PS-1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>and PS-2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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.

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 thickness of framing members shall be 33 mil.

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 #8 flat-head screws. 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 shall be attached to flat strapping. Flat strapping shall be either installed on top of or below sheathing. Sheathing shall be attached to strapping as required for panel edges. Strapping shall, as a minimum, be 1 ½” (38.1 mm) wide by 33 mil thick strap material.

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 (l/w) 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 (l/w) not greater than 3:2.
AISI STANDARD

Commentary on the Standard for Cold-Formed Steel Framing – Lateral Design, 2004 Edition

Endorsed by:

Steel Framing Alliance™
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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 may eventually become dated. It is anticipated that AISI will publish updates of this material as new information becomes available, but this cannot be guaranteed.

The materials set forth herein are for general purposes only. They are not a substitute for competent professional advice. Application of this information to a specific project should be reviewed by a 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 there from.
PREFACE

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

In the Commentary, sections, equations, figures, and tables are identified by the same notation as used in the Lateral Standard.
## AISI COMMITTEE ON FRAMING STANDARDS

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<td>Rahim Zadeh</td>
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AISI also acknowledges the following corresponding members who helped develop this document.

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Dan Dolan  Washington State University
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COMMENTARY ON THE
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A. GENERAL

The provisions of the Lateral Standard were based on the requirements in the International Building Code (ICC, 2003) and the NFPA 5000 Construction and Safety Code (NFPA, 2003). The provisions in those codes have evolved since the early work of Tarpy (1976-80), APA-The Engineered Wood Association (1993), Serrette (1995) 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 design values in the Lateral Standard. Specific references to this research are cited in this Commentary.

The Lateral Standard covers the levels of performance for wind and seismic resistance as expressed in the current model codes. It is further intended that the Lateral Standard will be introduced for adoption in future editions of the model codes, including the ICC International Building Code (2006) and NFPA 5000 Construction and Safety Code (2006), and other design standards and resource documents such as ASCE 7 Minimum Design Loads for Buildings and Other Structures (2005 Supplement No. 1) and the NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (Next edition expected in 2008).
B. GENERAL DESIGN REQUIREMENTS

B3 Shear Resistance Based on Principles of Mechanics

The Lateral Standard does not aim to limit cold-formed steel frame diaphragms, shear walls and braced walls to the configurations included in the Lateral 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 Lateral 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 brace) 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 the Lateral Standard 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 Lateral 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.

B4 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 the lateral element is capable of developing.
C. WALLS

C1  General

C1.1  Seismic Requirements for Shear Walls

The Lateral Standard imposes the special seismic requirements of Section C5 and D3 when the seismic response modification factor, R, (for steel systems) is greater than 3. This is consistent with ASCE 7 (ASCE, 2002), which is adopted by reference by the IBC (ICC, 2003) and NFPA 5000 (NFPA, 2003) building codes and permits an R factor of 3 for steel systems in Seismic Design Category A-C. In these low Seismic Design Categories a designer may choose to use an R factor of 3 to determine the seismic load and not use the special detailing in Sections C5 and D3 or may use the R factor assigned to the specific system and use the detailing in Sections C5 and D3. In Seismic Design Category D–F, the option of using an R of 3 is not permitted and the designer must use the special seismic requirements in Sections C5 and D3.

The design coefficients, factors and limitations assigned to light-framed shear wall systems in ASCE 7 are reproduced in Table C1-1, below.

### Table C1-1
Design Coefficients and Factors for Basic Seismic Force-Resisting Systems

<table>
<thead>
<tr>
<th>Basic Seismic Force-Resisting System</th>
<th>Seismic Response Modification Coefficient, R</th>
<th>System Over-strength Factor, ( \Omega_o )</th>
<th>Deflection Amplification Factor, ( C_d )</th>
<th>Structural System Limitations and Building Height (ft) Limitations a</th>
<th>Seismic Design Category</th>
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<td>Light-framed walls</td>
<td>6</td>
<td>3</td>
<td>4</td>
<td>NL</td>
<td>A&amp;B C D E F</td>
</tr>
<tr>
<td>sheathed with wood</td>
<td></td>
<td></td>
<td></td>
<td>NL</td>
<td></td>
</tr>
<tr>
<td>structural panels rated</td>
<td></td>
<td></td>
<td></td>
<td>65</td>
<td></td>
</tr>
<tr>
<td>for shear resistance or</td>
<td></td>
<td></td>
<td></td>
<td>65</td>
<td></td>
</tr>
<tr>
<td>steel sheets</td>
<td></td>
<td></td>
<td></td>
<td>65</td>
<td></td>
</tr>
<tr>
<td>Light-framed walls with</td>
<td>2</td>
<td>2 ½</td>
<td>2</td>
<td>NL</td>
<td>A&amp;B C D E F</td>
</tr>
<tr>
<td>shear panels of all other materials</td>
<td></td>
<td></td>
<td></td>
<td>NL</td>
<td></td>
</tr>
<tr>
<td>Light-framed wall</td>
<td>4</td>
<td>2</td>
<td>3 ½</td>
<td>NL</td>
<td>A&amp;B C D E F</td>
</tr>
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<td>systems using flat strap</td>
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<td></td>
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</tr>
<tr>
<td>Light-framed walls</td>
<td>6 ½</td>
<td>2 ½</td>
<td>4 ½</td>
<td>NL</td>
<td>A&amp;B C D E F</td>
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<tr>
<td>sheathed with wood</td>
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<td></td>
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<td>NL</td>
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<td>structural panels rated</td>
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<tr>
<td>Light-framed walls with</td>
<td>2 ½</td>
<td>2 ½</td>
<td>2 ½</td>
<td>NL</td>
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<td></td>
<td>NP</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>NP</td>
<td></td>
</tr>
</tbody>
</table>

a NL = Not Limited and NP = Not Permitted.
For SI: 1 foot = 0.305 m
Per ASCE 7 (ASCE, 2002), 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 the Lateral Standard, shear walls or braced frames are the basic seismic force resisting elements.

C2 Type I Shear Walls

C2.1 Available Shear Strength.

The requirements for Type I shear walls in the Lateral Standard 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 Lateral Standard. Figure C2-1 shows typical Type I shear walls with hold-down anchors at the ends of each wall segment. The nominal values in Tables C2.1-1, C2.1-2 and C2.1-3 were based on tests with hold-down anchors on each end of the wall. As a result, the Lateral Standard requires hold-down anchors even though calculations may demonstrate that hold-down anchors are not necessary.

![Figure C2-1 – Type I Shear Walls](image)

The nominal 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).
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 wall 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 addition, Type I shear walls sheathed with wood structural or sheet steel panels are permitted to have window openings, between hold-down anchors at each end of a wall segment, where details are provided to account for force transfer around openings.

C2.1.1 Design Deflection

The deflection provisions are new to the Lateral Standard and are based on work performed by Serrette (2003). Equations C2.1-1 and C2.1-2 may be used to estimate the
drift deflection of cold-formed steel frame shear walls recognized in the IBC (ICC, 2003) and NFPA 5000 (NFPA, 2003) building codes. The equations should not be used beyond the nominal shear values given in the Lateral Standard. 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.

\[
\delta = \frac{8v h^3}{E_s A_c b} + \omega_1 \omega_2 \frac{v h}{\rho G t_{sheathing}} + \omega_1^{5/4} \omega_2 \omega_3 \omega_4 \left( \frac{v}{\beta} \right)^2 + \delta_a
\]

Linear elastic cantilever bending: \( \frac{8v h^3}{E_s A_c b} \)

Linear elastic sheathing shear: \( \omega_1 \omega_2 \frac{v h}{\rho G t_{sheathing}} \)

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: \( \delta_a \)

![Figure C2-4 – Lateral Contribution from Anchorage/Hold-down Deformation](image)

The \( \delta_a \) term 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 IBC and NFPA 5000 cold-formed steel shear wall design values. The \( \rho \) 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 \( \rho \) 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 \( G_{tv} \), the nominal panel thickness \( t \) and through thickness panel grade and construction adjustment factor \( C_G \) provided in the AFPA Manual for Engineered Wood Construction Supplement: Wood Structural Panels (AFPA, 2001). For example, G for 7/16-in. 24/16 OSB rated sheathing can be approximated as follows:

\[
G_{tv} \text{ (24/16 span rating)} = 25,000 \text{ lb/inch (strength axis parallel to framing)}
\]

\[
t = 0.437 \text{ inch (as an approximation for } t_v \text{)}
\]

\[
C_G = 3.1
\]

\[
G \text{ (approximate)} = 3.1 \times 25,000 / 0.437 = 177,300 \text{ psi}
\]

Thus, \( C_G G_{tv} = 77,500 \text{ lb/inch and } G_t = 77,500 \text{ lb/inch} \)

A comparison of the \( C_G G_{tv} \) and \( G_t \) 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 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.

**C2.2 Limitations for Systems in Tables C2.1-1, C2.1-2 and C2.1-3**

The Lateral Standard provides a section on limitations for shear wall systems using the nominal values in Tables C2.1-1, C2.1-2 and C2.1-3. 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 Lateral 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).

**C3 Type II Shear Walls**

The requirements for Type II shear walls, also known as perforated shear walls, in the Lateral Standard 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. The Lateral Standard 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](image)

Table C3.2-1 in the Lateral 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.
C4 Strap Bracing

Where braced walls utilize strap bracing, it is acceptable to compute the deflection of these walls using standard engineering analysis for braced walls. Because loose straps permit lateral displacement without resistance, the Lateral Standard requires that straps be installed taut.

C5 Special Seismic Requirements

The special seismic requirements were based on engineering judgment, industry practice, building code provisions and appropriate limitations to replicate the conditions of the tested assemblies.

C5.3 Anchorage of Braced Wall Segments

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. To develop a desirable response, the Lateral Standard requires that components transferring load to and from the lateral element be capable of developing the nominal strength of the element or the expected over strength ($\Omega_o$ times the design seismic load) of the lateral resisting element.

C5.5 Diagonal Strap Bracing

The slenderness of tension-only 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.
D. DIAPHRAGMS

D1 General

The Lateral Standard permits the use of sheet steel, concrete or wood structural panel sheathing or other approved materials to serve as the diaphragm sheathing.

D2 Diaphragm Design

D2.1 Available Shear Strength

The available shear strength of diaphragms is to be based upon principles of mechanics, per section B3. Alternatively for diaphragms sheathed with wood structural panels the available shear 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 Lau (LGSEA, 1998). Lau developed ASD design tables using an analytical method outlined by Tissell (1992) for wood framed diaphragms 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 designated steel thicknesses greater than 33-mil resulted in higher strength values, no increase in strength was included for these greater thicknesses.

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 frame construction. The current equation for wood frame construction (ICC, 2003) is as follows:

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

For SI:

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

where:

- \( A \) = Area of chord cross section, in square inches (mm²)
- \( b \) = Diaphragm width, in feet (mm)
- \( E \) = Elastic modulus of chords, in pounds per square inch (N/mm²)
- \( e_n \) = Nail deformation, in inches (mm)
- \( G \) = Modulus of rigidity of wood structural panel, in pounds per square inch (N/mm²)
- \( L \) = 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)
\[ \Delta = \text{The calculated deflection, in inches (mm)} \]
\[ \Sigma(\Delta c X) = \text{Sum of individual chord-splice values on both sides of the diaphragm, each multiplied by its distance from the nearest support} \]

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.

**D3 Special Seismic Requirements**

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


