AISI STANDARD

North American Standard for the Design of Profiled Steel Diaphragm Panels

2020 EDITION
DISCLAIMER

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

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

The materials set forth herein are for general information only. They are not a substitute for competent professional advice. Application of this information to a specific project should be reviewed by a registered professional engineer. Indeed, in most 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 resulting liability arising therefrom.

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PREFACE

The American Iron and Steel Institute Committee on Specifications has developed AISI S310-20, the 2020 Edition of the North American Standard for the Design of Profiled Steel Diaphragm Panels to provide design provisions for diaphragms consisting of profiled steel decks or panels which include fluted profiles and cellular deck profiles. This Standard is intended for adoption and use in the United States, Canada, and Mexico.

User Notes are non-mandatory portions of this Standard.

The major changes of this edition include:

- The contribution of screw connection through fiberglass is considered not only in the perimeters, but in the field of the diaphragm as well.
- Section A1.2.7 was added to clarify the governing standard in different situations.
- The diaphragm shear strength controlled by exterior support local buckling is added in Section D2.
- A simplified but improved prediction for the diaphragm shear strength of steel deck with concrete fill is provided in Section D4. Additionally, the upper limit on steel support thickness of concrete filled diaphragms is eliminated in Section D4.

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.

American Iron and Steel Institute
November 2020
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A. GENERAL PROVISIONS

A1 Scope, Applicability, and Definitions

A1.1 Scope

This Standard applies to diaphragms and wall diaphragms that contain profiled steel panels, which include fluted panels or deck, and cellular deck.

A1.2 Applicability

A1.2.1 Unless noted otherwise, the term, diaphragm, applies to level and sloped roof diaphragms and to wall diaphragms as defined in Section A1.3.

A1.2.2 This Standard determines the available strength [factored resistance] and stiffness of steel panels and their connections in a diaphragm system, but does not address determination of available strength [factored resistance] for the other components in the system.

A1.2.3 The design of other diaphragm components is governed by the applicable building code and other design standards.

A1.2.4 The in-plane nominal shear strength [resistance] per unit length and stiffness of steel diaphragm or wall diaphragm panels, deck, or cellular deck shall be determined in accordance with this Standard. When calculation is used, the nominal shear strength [resistance] per unit length and stiffness shall be determined in accordance with Chapter D. When testing is used, the nominal shear strength [resistance] per unit length and stiffness shall be determined in accordance with Chapter E.

A1.2.5 This Standard shall apply to roof or floor diaphragms, or wall diaphragms that are installed:

(a) With or without insulation between the panel and the support,

(b) Without insulation between the cellular deck and the support in accordance with Chapter D,

(c) With insulation between the cellular deck and the support in accordance with Chapter E,

(d) With or without concrete fill over the deck or cellular deck,

(e) With or without acoustic (perforated) panels or cellular acoustic deck, and

(f) With structural supports made of steel, wood, or concrete.

A1.2.6 If the lateral stability or diaphragm action to resist in-plane lateral loads is provided by cold-formed steel framing with diagonal bracing or covered with structural wood, gypsum board, fiberboard, flat steel sheet or other flat panel sheathing, the design and installation shall be in accordance with AISI S240, AISI S400, and the applicable building code.

User Note:
Walls (vertical diaphragms) often are part of the lateral force-resisting system and may be subject to additional requirements by the applicable building code, particularly when resisting and dissipating seismic energy.
A1.2.7 This *Standard* shall govern over other standards, including those referenced in this *Standard*, in matters pertaining to elements falling within the scope of this *Standard*, as defined in Section A1.1. Where conflicts between this *Standard* and the applicable building code occur, the requirements of the applicable building code shall govern. In areas without an applicable building code, this *Standard* defines the minimum acceptable standards for elements falling within the scope of this *Standard*, as defined in Section A1.1.

A1.2.8 This *Standard* does not preclude the use of other approved materials, assemblies, structures or designs of equivalent performance.

A1.2.9 This *Standard* consists of Chapters A through E, and Appendices 1 and 2.

A1.3 Definitions

Where terms appear in this *Standard* in *italics*, such terms shall have the meaning as defined in this section or as defined in AISI S100 if they are not defined in this section. Terms included in square brackets shall be specific to Limit States Design (LSD) terminology. Terms not italicized shall have the ordinary accepted meaning in the context for which they are intended.

**General Terms**

*Acoustic Panel.* Fluted *panel* or *deck* containing holes. Holes are in discrete locations or throughout the coil width. Insulation often is installed behind the holes to improve sound absorption.

*Approved.* Acceptable to the *authority having jurisdiction*.

*Authority Having Jurisdiction.* An organization, political subdivision, office, or individual charged with the responsibility of administering and enforcing the provisions of the applicable building code.

*Cellular Acoustic Deck.* Cellular *deck* with the bottom *deck* or flat sheet perforated to improve sound absorption. Holes are beneath the cavity formed with the top *deck* and fasteners are in either a perforated or non-perforated zone. Insulation often is installed in the cell cavity above the holes to improve sound absorption.

*Cellular Deck.* Composite or partially composite built-up *deck* formed by fastening either a flat steel sheet or a *panel* beneath and to another *panel*.

*Composite Deck.* Fluted *deck* or cellular *deck* that combines with structural concrete fill to form a slab with the *deck* as reinforcement. The fluted element has embossments, interlocking profile geometry, or other horizontal shear *connection* devices to develop mechanical bond between the *deck* and concrete so the slab compositely resists vertical and diaphragm shear loads.

*Configuration.* A specific arrangement of *panel* geometry, *thickness*, mechanical properties, span(s), and attachments that is unique to a test assembly.

*Connection Flexibility.* The property of a *connection* allowing local deflection caused by a unit *load*, and associated with *panel* distortion or slotting, and *connection* slip or strain.

*Deck.* A *panel* installed and covered by another membrane for weathertightness or by structural or insulating concrete.

*Diaphragm.* Roof, floor, or other horizontal or nearly horizontal membrane or bracing system

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that transfers in-plane forces to the lateral force-resisting system.

**Double-Skinned Panel.** A two-part built-up system that includes a bottom fluted panel fastened to supports. A sub-girt or other device is spaced periodically and fastened to the bottom fluted panel. A top fluted panel is both installed over and fastened to the sub-girts or other device. The top panel typically is not fastened directly to supports.

**Edge Panel.** Full or partial width panel that transfers in-plane forces to the lateral force-resisting system of the structure along a line that generally parallels the length of the panel.

**Exterior Flute.** A one pitch wide profile that is part of a multi-fluted panel and is located at a panel sidelap. The profile typically has one bottom flat, one top flat, and two webs.

**Exterior Support.** Support located at an end of an edge or interior panel.

**Flexibility.** The property of a diaphragm system that is the inverse of stiffness.

**Form Deck.** Fluted deck or cellular deck that chemically bonds with structural concrete or insulating concrete fill to form a slab that resists diaphragm shear loads. The deck resists the concrete dead load prior to concrete compressive strength being developed. Reinforcement is required in structural concrete to resist slab flexure.

**Insulating Concrete.** A mixture of Portland cement, cellular or expanded mineral concrete aggregate, and water forming a relatively lightweight concrete. When concrete is dried, the aggregate porosity and air content provide insulating characteristics to roofs.

**Interior Flute.** A one pitch wide profile that is part of a multi-fluted panel and is located away from a panel sidelap. The profile typically has one bottom flat, one top flat, and two webs.

**Interior Panel.** Full or partial width panel that transfers in-plane forces to other interior panels or edge panels.

**Interior Support.** Support located at an interior zone of an edge or interior panel.

**Interlocking Top Sidelap Connection.** A connection formed at a vertical sheet leg (edge stiffener of panel) inside an overlapping sheet hem, or at vertical legs back-to-back.

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

**Panel.** Product formed from steel coils into fluted profiles with top and bottom flanges connected by web members. Profile is connected to supports and can have a singular or a repeating pattern.

**Pitch.** Width of the repeating pattern of fluted panel measured from center-to-center.

**Prototype Diaphragm System.** A diaphragm system including a range of configurations that provide various combinations of profile, thickness, span, fastener type and pattern, support thickness, and edge detail.

**Power-Actuated Fastener (PAF).** Hardened steel fasteners driven through steel members into embedment material using either powder cartridges or compressed gas as the energy-driving source.

**Shear Wall.** Wall that provides resistance to lateral loads in the plane of the wall and provides Sidelap. Joint at which adjacent panels contact each other along a longitudinal edge.

**Sidelap Connection.** Also called a stitch connection. A connection with a fastener or weld located at a sidelap while not penetrating a support.

**Single Diaphragm System.** A diaphragm system having a specific configuration with one set of profile, thickness, mechanical properties, span, fastener type and pattern, support type and
_thickness_, fill type and thickness when applicable, and edge detail.

**Standing Seam Panel.** A roof panel having longitudinal (side) joints between the panels in a vertical position above the roof line. The roof panel system is secured to the roof substructure by means of concealed hold-down clips engaging the side joint.

**Stiffness.** The property of a diaphragm system resisting in-plane deflection.

**Structural Concrete.** A mixture of Portland or other hydraulic cement, fine aggregate, coarse aggregate, and water, used for structural purposes, including plain and reinforced concrete.

**Structural Connection.** Also called a support connection. A connection with a fastener or weld attaching one or more sheets to supporting members.

**Support Connection.** See Structural Connection.

**Top Arc Seam Sidelap Welds.** Arc seam welds applied at the top of an interlocking top sidelap connection.

**Top Overlapping Sidelap Connection.** Welded, screwed, or mechanically formed or crimped connection located at or near the top of an overlapping sidelap. These connections are often concealed when viewed from below.

**Wall Diaphragm.** A wall, load bearing or non-load bearing, designed to resist forces acting in the plane of the wall (commonly referred to as a “vertical diaphragm” or “shear wall”).

### A2 Materials

Profiled steel panels and cold-formed steel supports shall conform to the material requirements of AISI S100, Section A3.

Hot-rolled steel supports shall conform to the material requirements of ANSI/AISC 360.

Wood supports shall conform to the material requirements of ANSI/AWC NDS and shall be structural lumber.

Structural concrete shall conform to the material requirements of ACI 318.

Insulating concrete aggregate shall conform to ASTM C332.

### A3 Loads

The ASD or LRFD loads, load factors and load combinations shall be determined in accordance with the applicable building code. In the absence of an applicable building code, ASCE 7 shall apply.

Load factors and load combinations for LSD shall be as stipulated by AISI S100 Section B2.

### A4 Referenced Documents

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

1. American Concrete Institute (ACI), 38800 Country Club Dr., Farmington Hills, MI 48331:
   ACI 318-19, Building Code Requirements for Structural Concrete

2. American Iron and Steel Institute (AISI), 25 Massachusetts Avenue, NW, Suite 800, Washington, DC 20001:
   AISI S100-16 (2020) w/S2-20, North American Specification for the Design of Cold-Formed Steel Structural Members (Reaffirmed in 2020) With Supplement 2
   AISI S240-20, North American Standard for Cold-Formed Steel Structural Framing

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AISI S400-20, *North American Standard for Seismic Design of Cold-Formed Steel Structural Systems*
AISI S904-17, *Test Standard for Determining the Tensile and Shear Strengths of Steel Screws*
AISI S905-17, *Test Standard for Determining the Strength and Deformation Characteristics of Cold-Formed Steel Connections*
AISI S907-17, *Test Standard for Determining the Strength and Stiffness of Cold-Formed Steel Diaphragms by the Cantilever Test Method*

3. American Institute of Steel Construction (AISC), One East Wacker Drive, Suite 700, Chicago, IL 60601-1802:
   ANSI/AISC 360-16, *Specification for Structural Steel Buildings*

4. American Society of Civil Engineers (ASCE), 1801 Alexander Bell Drive, Reston, VA 20191-4400:

5. American Welding Society (AWS), 8669 NW 36 Street, #130, Miami, FL 33166-6672:

6. ASTM International (ASTM), 100 Barr Harbor Drive, PO Box C700, West Conshohocken, PA, 19428-2959:
   ASTM C33/C33M-18, *Standard Specifications for Concrete Aggregates*
   ASTM C330/C330M-17a, *Standard Specification for Lightweight Aggregates for Structural Concrete*
   ASTM C332-17, *Standard Specification for Lightweight Aggregates for Insulating Concrete*
   ASTM D1761-12, *Standard Test Methods for Mechanical Fasteners in Wood*

7. American Wood Council, 222 Catoctin Circle SE, Suite 201, Leesburg, VA 20175:

### A5 Units of Symbols and Terms

Any compatible system of measurement units is permitted to be used in the *Standard* except where explicitly stated otherwise. The unit systems considered shall include U.S. Customary units (force in kilopounds (kip) and length in inches (in.)), and SI units (force in Newtons (N) and length in millimeters (mm)).
B. SAFETY FACTORS AND RESISTANCE FACTORS

The safety and resistance factors for diaphragm systems shall be determined in accordance with Table B-1.

Table B-1

<table>
<thead>
<tr>
<th>Diaphragm System Conditions</th>
<th>Applicable AISI S310 Sections</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Diaphragm Strength</td>
</tr>
<tr>
<td></td>
<td>Determined by Calculation</td>
</tr>
<tr>
<td></td>
<td>Using Chapter D</td>
</tr>
<tr>
<td>Steel support and no concrete fill</td>
<td>Sections B1, D1.1.5</td>
</tr>
<tr>
<td>Wood supports</td>
<td>Sections D1.1.4.1, D1.1.5</td>
</tr>
<tr>
<td>Structural concrete supports</td>
<td>Section D1.1.5</td>
</tr>
<tr>
<td>Structural concrete fill</td>
<td>Section D4.1</td>
</tr>
<tr>
<td>Insulating concrete fill</td>
<td>Section D4.1</td>
</tr>
</tbody>
</table>

B1 Safety Factors and Resistance Factors of Diaphragms With Steel Supports

For diaphragms or wall diaphragms with steel support and no concrete fill, or with concrete fill with perimeter fasteners other than steel headed stud anchors, the safety and resistance factors shall be determined in accordance with Section B1.1. For diaphragms with concrete fill and steel headed stud anchors as perimeter fasteners, and where the limit state is diagonal tension cracking, the safety and resistance factors shall be determined in accordance with Section D4.1.

B1.1 Floor, Roof, or Wall Steel Diaphragm Construction

The in-plane diaphragm nominal shear strength [resistance], \( S_{nv} \), shall be established by calculation or test. The safety factors and resistance factors for diaphragms given in Table B1.1-1 shall apply to calculations or tests. The safety factors and resistance factors for tests shall be determined in accordance with Section E1.2.2 or Section E2.2 of this Standard, as applicable. However, the more severe factor from calibration and Table B1.1-1 shall be used unless noted otherwise in the Standard. If the nominal shear strength [resistance] is only established by test without defining all limit state thresholds, the safety factors and resistance factors shall also be limited by the values given in Table B1.1-1 for connection types and connection-related failure modes. The more severe factored limit state shall control the design. Where a combination of connection types is used within a diaphragm configuration, the more severe factor shall be used.

\[
\omega_d = \text{As specified in Table B1.1-1} \quad \text{(ASD)}
\]

\[
\phi_d = \text{As specified in Table B1.1-1} \quad \text{(LRFD and LSD)}
\]
Table B1.1-1
Safety Factors and Resistance Factors for Diaphragms

<table>
<thead>
<tr>
<th>Load Type or Combinations Including</th>
<th>Connection Type</th>
<th>Limit State</th>
<th>Connection-Related</th>
<th>Stability-Related</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Ωₖ (ASD)</td>
<td>φₖ (LRFD)</td>
</tr>
<tr>
<td>Wind</td>
<td>Welds</td>
<td></td>
<td>2.15</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>Screws</td>
<td></td>
<td>2.00</td>
<td>0.80</td>
</tr>
<tr>
<td>Earthquake and All Others</td>
<td>Welds</td>
<td></td>
<td>3.00</td>
<td>0.55</td>
</tr>
<tr>
<td></td>
<td>Screws</td>
<td></td>
<td>2.30</td>
<td>0.70</td>
</tr>
</tbody>
</table>

For mechanical fasteners other than screws:
(a) Ωₖ shall not be less than the Table B1.1-1 values for screws, and
(b) φₖ shall not be greater than the Table B1.1-1 values for screws.

In addition, the value of Ωₖ and φₖ using mechanical fasteners other than screws shall be limited by the Ω and φ values established through calibration of the individual fastener shear strength in accordance with Section D1.1.5, unless sufficient data exist to establish a diaphragm system effect in accordance with Section E1.2. Fastener shear strength calibration shall include the diaphragm material type.

If the nominal shear strength [resistance] per unit length of the diaphragm is established by test in accordance with AISI S907 or a connection strength of the diaphragm is established by test in accordance with AISI S905, the safety and resistance factors shall be determined in accordance with Section E1.2.2 or Section E2.2 of this Standard, as applicable. The test assembly shall be such that the tested failure mode is representative of the design. The impact of the thickness of the supporting material on the failure mode shall be included in the test, if applicable.

User Note:
Stability is discussed in the Commentary of Section D2.
Mechanical fasteners include screws, power-actuated fasteners, or other mechanical connections. Diaphragm system effect is established through tests in accordance with AISI S907.
C. DIAPHRAGM AND WALL DIAPHRAGM DESIGN

C1 General

The design of diaphragm and wall diaphragm systems shall be based on calculation or testing. Diaphragm or wall diaphragm system chords, ties, collectors, support framing, supplemental in-plane bracing systems, and the associated details and connections shall be designed in accordance with the applicable design standard for the material used. The in-plane shear strength per unit length and stiffness for panels or decks used as components of a diaphragm or wall diaphragm system shall be determined in accordance with this Standard.

Loads and load combinations shall be determined in accordance with Section A3.

The application of profiled steel panels or decks as a component of a diaphragm or wall diaphragm system shall meet the system limitations in the applicable building code.

User Note:
System limitations might include diaphragm span-to-depth ratio or flexibility limits.

C2 Strength Design

The available shear strength [factored resistance] per unit length of deck and panels shall satisfy the following equations:

For ASD,

$$R \leq \frac{S_n}{\Omega}$$  \hspace{1cm} (Eq. C2-1)

where

- $R$ = Required strength for ASD
- $S_n$ = Nominal shear strength [resistance] per unit length of diaphragm system as specified in Chapter D or E
- $\Omega$ = Safety factor for diaphragm strength determined in accordance with Table B-1

For LRFD,

$$R_u \leq \phi S_n$$  \hspace{1cm} (Eq. C2-2)

where

- $R_u$ = Required strength for LRFD
- $\phi$ = Resistance factor for diaphragm strength determined in accordance with Table B-1

For LSD,

$$\phi S_n \geq R_f$$  \hspace{1cm} (Eq. C2-3)

where

- $R_f$ = Effect of factored loads for LSD
- $\phi$ = Resistance factor for diaphragm resistance determined in accordance with Table B-1

C3 Deflection Requirements

Diaphragm deflection under load shall satisfy Eq. C3-1.

$$\delta_n \leq \delta_a$$  \hspace{1cm} (Eq. C3-1)

where

- $\delta_n$ = Calculated diaphragm deflection at the load determined in accordance with Section A3

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\( \delta_a = \) Allowable diaphragm deflection defined by the applicable building code and the structure’s service requirements

Deflection, \( \delta_n \), is determined using stiffness or flexibility analytical methods. Diaphragm stiffness, \( G' \), of the deck or panel shall be determined in accordance with Section D5. Diaphragm flexibility, \( F \), of the deck or panel shall be determined in accordance with Section D6.
D. DIAPHRAGM NOMINAL SHEAR STRENGTH PER UNIT LENGTH AND STIFFNESS DETERMINED BY CALCULATION

This section shall apply to fluted panels or deck within the following limits:

(a) 0.5 in. (12 mm) ≤ panel or deck depth ≤ 7.5 in. (191 mm),
(b) 0.014 in. (0.35 mm) ≤ base panel or deck thickness ≤ 0.075 in. (1.91 mm) for depth less than or equal to 3.0 in. (76.2 mm),
0.034 in. (0.85 mm) ≤ base panel or deck thickness ≤ 0.075 in. (1.91 mm) for depth greater than 3.0 in. (76 mm),
(c) 33 ksi (230 MPa) ≤ specified F_y of panel or deck ≤ 80 ksi (550 MPa),
45 ksi (310 MPa) ≤ specified F_u of panel or deck ≤ 82 ksi (565 MPa), and
(d) Panel or Deck pitch ≤ 12 in. (305 mm).

Additional requirements shall be satisfied for panels over insulation at supports as specified in Section D1.3, and cellular decks as specified in Section D1.5.

The available shear strength [factored resistance] per unit length of a diaphragm or wall diaphragm system shall be the lower value obtained from the limit states controlled by either connection strength or panel out-of-plane buckling strength.

\[
\frac{S_n}{\Omega} = \min \left( \frac{S_{nf}}{\Omega_{df}}, \frac{S_{nb}}{\Omega_{db}} \right) \quad \text{for ASD} \tag{Eq. D-1}
\]

\[
\phi S_n = \min (\phi_{df} S_{nf}, \phi_{db} S_{nb}) \quad \text{for LRFD and LSD} \tag{Eq. D-2}
\]

where

- \( S_n \) = Nominal shear strength [resistance] per unit length of diaphragm system
- \( S_{nf} \) = Nominal shear strength [resistance] per unit length of diaphragm system controlled by connections and in accordance with Section D1
- \( S_{nb} \) = Nominal shear strength [resistance] per unit length of diaphragm system controlled by panel out-of-plane buckling and in accordance with Section D2
- \( \phi \) = Resistance factor for diaphragm strength determined in accordance with Table B-1
- \( \phi_{df} \) = Resistance factor for diaphragm strength controlled by connections and in accordance with Table B-1
- \( \phi_{db} \) = Resistance factor for diaphragm strength controlled by panel out-of-plane buckling and in accordance with Table B-1
- \( \Omega \) = Safety factor for diaphragm strength determined in accordance with Table B-1
- \( \Omega_{df} \) = Safety factor for diaphragm strength controlled by connections and in accordance with Table B-1
- \( \Omega_{db} \) = Safety factor for diaphragm strength controlled by panel out-of-plane buckling and in accordance with Table B-1

The steel edge dimensions at sidelaps, end-laps, and end butt joints shall meet the requirements for connections specified in AISI S100.

**User Note:**

\( \phi_{df} \) and \( \phi_{db} \), or \( \Omega_{df} \) and \( \Omega_{db} \), are subsets of \( \phi \) or \( \Omega \) and indicate that two limit states must be investigated to determine the available shear strength [factored resistance] per unit length of panels.
**D1 Diaphragm Shear Strength per Unit Length Controlled by Connection Strength, S_{nf}**

The *nominal shear strength [resistance]* per unit length of a *diaphragm* or *wall diaphragm* controlled by *connection strength*, S_{nf}, shall be the smallest of S_{ni}, S_{nc}, S_{ne}, and S_{np}.

\[
S_{ni} = [2A(\lambda - 1) + \beta] \frac{P_{nf}}{L} \quad \text{(Eq. D1-1)}
\]

\[
S_{nc} = \left( \frac{N^2\beta^2}{L^2N^2 + \beta^2} \right)^{0.5} P_{nf} \quad \text{(Eq. D1-2)}
\]

\[
S_{ne} = \frac{(2\alpha_1 + n_p\alpha_2)P_{nf} + n_eP_{nf}}{L} \quad \text{(Eq. D1-3)}
\]

\[
S_{np} = \frac{n_dP_{nf}}{W_t} \quad \text{ (For Fluted Panels)} \quad \text{(Eq. D1-4a)}
\]

\[
= NP_{nf} \quad \text{ (For Cellular Deck)} \quad \text{(Eq. D1-4b)}
\]

where

- \( S_{ni} = \text{Nominal shear strength [resistance] per unit length of diaphragm or wall diaphragm controlled by connections at interior panels or edge panels} \)
- \( S_{nc} = \text{Nominal shear strength [resistance] per unit length of diaphragm or wall diaphragm controlled by support connections at the corners of interior panels or edge panels} \)
- \( S_{ne} = \text{Nominal shear strength [resistance] per unit length of diaphragm or wall diaphragm controlled by connections along the edge parallel to the panel span in an edge panel and located at a diaphragm reaction line} \)
- \( S_{np} = \text{Nominal shear strength [resistance] per unit length of diaphragm or wall diaphragm controlled by connections along the ends of interior or edge panels and into exterior supports} \)
- \( A = \text{Number of exterior support connections per flute located at the sidelap at an interior panel or edge panel end} \)
- \( \lambda = \text{Connection strength reduction factor at corner fastener, unitless} \)
  \[
  = \begin{cases} 
  1 - \frac{D_dL_v}{240\sqrt{t}} \geq 0.7 & \text{for U.S. Customary units} \\
  1 - \frac{D_dL_v}{369\sqrt{t}} \geq 0.7 & \text{for SI units} 
  \end{cases} \quad \text{(Eq. D1-5a)}
  \]

\[
= \begin{cases} 
  1 - \frac{D_dL_v}{240\sqrt{t}} \geq 0.7 & \text{for U.S. Customary units} \\
  1 - \frac{D_dL_v}{369\sqrt{t}} \geq 0.7 & \text{for SI units} 
  \end{cases} \quad \text{(Eq. D1-5b)}
\]

where

- \( D_d = \text{Depth of panel, in. (mm). See Figure D2-1} \)
- \( L_v = \text{Span of panel between supports with fasteners, ft (m)} \)
- \( t = \text{Base metal thickness of the panel, in. (mm)} \)

- \( \beta = \text{Factor defining connection contribution and interaction to diaphragm shear strength per unit length} \)
  \[
  = n_s\alpha_s + 2n_p\alpha_p^2 + 4\alpha_e^2 \quad \text{(Eq. D1-6)}
  \]

- \( n_s = \text{Number of sidelap connections along a total panel length, } L_v \text{, and not into supports} \)
- \( \alpha_s = \frac{P_{ns}}{P_{nf}} \quad \text{(Eq. D1-7)} \)
\[ P_{nf} = \text{Nominal shear strength [resistance] of a support connection per fastener} \]
\[ P_{ns} = \text{Nominal shear strength [resistance] of a sidelap connection per fastener} \]
\[ n_p = \text{Number of interior supports along a total panel length, } L \]
\[ \alpha_p^2 = \text{Analogous section modulus of panel interior support connection group in an interior or edge panel} \]
\[ = \left( \frac{1}{w^2} \right) \sum x_p^2 \quad (Eq. D1-8) \]
\[ w = \text{Panel cover width} \]
\[ x_p = \text{Distance from panel centerline to an interior support structural connection in a panel} \]
\[ \alpha_c^2 = \text{Analogous section modulus of panel exterior support fastener group in an interior or edge panel} \]
\[ = \left( \frac{1}{w^2} \right) \sum x_e^2 \quad (Eq. D1-9) \]
\[ x_e = \text{Distance from panel centerline to an exterior support structural connection in a panel} \]
\[ L = \text{Total panel length} \]
\[ = (n_p + 1)L_v \text{ for equal spans (Eq. D1-10)} \]
\[ N = \text{Number of support fasteners per unit width at an interior or edge panel's end} \]
\[ \alpha_1 = \text{Measure of exterior support fastener group distribution across a panel width, } w_e, \text{ at an edge panel} \]
\[ = \frac{\sum x_{ee}}{w_e} \quad (Eq. D1-11) \]
\[ x_{ee} = \text{Distance from panel centerline to an exterior support structural connection in an edge panel} \]
\[ w_e = \text{Panel cover width at the edge panel} \]
\[ \alpha_2 = \text{Measure of interior support fastener group distribution across a panel width, } w_e, \text{ at an edge panel} \]
\[ = \frac{\sum x_{pe}}{w_e} \quad (Eq. D1-12) \]
\[ x_{pe} = \text{Distance from panel centerline to an interior support structural connection in an edge panel} \]
\[ n_e = \text{Number of edge support connections between transverse supports and along an edge panel length, } L \]
\[ P_{nfs} = \text{Nominal shear strength [resistance] of an edge support connection installed parallel with an edge panel span and between transverse supports} \]
\[ n_d = \text{Number of support connections at any given flute bottom flat along the ends of interior or edge panels and into exterior supports} \]
\[ w_t = \text{Greatest tributary width to any given bottom flute with support connection(s) along the end perpendicular to the panel span and located at exterior support} \]

**User Note:**

Commentary Figure C-D2-1 provides examples on determination of \( n_d \) and \( w_t \).

See Figure D1-1 for an illustration of the parameters in Section D1.
Support fastener spacing shall not exceed 18 in. (460 mm).

For \( L_v > 5.00 \text{ ft (1.52 m)} \), the spacing of \textit{sidelap connections} between supports shall not exceed 3.00 ft (0.914 m), and the spacing of edge fasteners between supports shall not exceed 3.00 ft (0.914 m).

\( P_{nf} \) shall be determined in accordance with Section D1.1, and \( P_{ns} \) shall be determined in accordance with Section D1.2. If insulation is present between the \textit{panel} and support, \( P_{nf} \) shall be determined in accordance with Section D1.3. If the \textit{support connection} is subjected to combined shear and tension, \( P_{nf} \) shall be reduced in accordance with Section D3.

\( P_{nfs} \) used to determine \( S_{ne} \) in accordance with Eq. D1-3 shall be calculated as follows:

(a) \( P_{nfs} \) is determined in accordance with Section D1.1 where insulation is not present,

(b) \( P_{nfs} \) at steel supports is determined in accordance with Section D1.3.1.1 where the \textit{connection} is through the bottom flat of a \textit{panel}, insulation is present, and the gap caused by insulation between the \textit{panel} bottom and the \textit{edge support} is less than or equal to 3/8 in. (9.53 mm), and

(c) \( P_{nfs} = 0.0 \) for \textit{connections} under any of the following conditions:

1. through the top flat of a \textit{panel} with or without insulation beneath the \textit{panel},
2. through the bottom flat of a \textit{panel} where insulation is present and the gap caused by insulation between the \textit{panel} bottom and the \textit{edge support} is greater than 3/8 in. (9.53 mm), or
3. through the bottom flat of a \textit{panel} where insulation is present and the support is wood.

**User Note:**

Details are required at the edge support where \( P_{nfs} = 0.0 \) kips (0.00 kN) for the conditions listed in (c) above. The details are discussed in Section D1.3. The designer should provide a detail that is capable of transferring the \textit{diaphragm shear force} (reaction) from the \textit{edge panel} to the edge support at the \textit{lateral force-resisting system} line. If the \textit{diaphragm shear force} per unit length can flow across a potential \textit{lateral force-resisting system} to another \textit{lateral force-resisting system} without exceeding the \textit{available strength} [\textit{factored resistance}] of the \textit{diaphragm system}, the detail can be avoided.

The \textit{panel} width, \( w_{po} \), at an edge support is the distance from the adjacent \textit{interior panel sidelap} to the reaction line and is required to determine the \textit{nominal diaphragm shear strength} [\textit{resistance}] per unit length at an edge panel, \( S_{nf} \) (smallest of \( S_{ni} \), \( S_{nc} \), \( S_{ne} \), and \( S_{np} \)).

\( P_{nfs} \) and \( P_{nf} \) are required in Eq. D1-3 and the determination is consistent where insulation is present. See Section D1.3.1.1 for the determination of \( P_{nf} \) at \textit{interior flutes} of either \textit{interior} or \textit{edge panels} with screws through bottom flats and insulation over steel supports. \( P_{nf} \) at wood supports is the same as \( P_{nfs} \) and determined using (a) or (c), as applicable. Some \textit{connection} installations do not allow a gap where Section D1.1 is used. Consult the fastener manufacturer’s recommendations for acceptable installation tolerances or refer to AWS D1.3, as applicable.

Installations with insulation between the \textit{panel} and the edge support are discussed in Section D1.3 and are consistent with the Section D1 requirements.

\( S_{nx} \) and \( S_{ny} \), as shown in Figure D1-1, indicate a possible shear flow along the orthogonal axes \( x \) and \( y \) and clarify that the required \( S_n \) can be a variable along the \textit{diaphragm span}, \( L_d \), between \textit{lateral force-resisting systems}.

Appendix 2 presents a particular case of \( S_{nc} \) with \textit{loads} delivered through perimeter \textit{connections}.
The nominal diaphragm shear strengths [resistances] per unit length, $S_{ni}$, $S_{nc}$, $S_{ne}$, and $S_{np}$, are subsets of $S_{nf}$, and the safety and resistance factors controlled by connections apply to each subset for the applicable connections. See Table B1.1-1 in Section B1.

Eqs. D1-1, D1-2, or D1-4a or D1-4b can control nominal shear strength [resistance] per unit length at either an edge or interior panel. Both panel locations must be investigated when the fastener pattern or panel width varies between the interior and edge panels. Eq. D1-3 only applies at locations of load transfer along lateral force-resisting system lines or along load delivery members (struts).

When diaphragm shear per unit length is flowing from two sides into a lateral force-resisting system, the required strength [reaction] per unit length rather than the maximum shear per unit length in the panel is compared with the available shear strength [factored resistance] per unit length. Available shear strength [factored resistance] is $S_{ne}/\Omega$ for ASD and $\phi S_{ne}$ for LRFD or LSD, where $S_{ne}$ is determined in accordance with Eq. D1-3.

To develop edge support connection resistance at each of the $n_e$ fasteners between panel supports, the designer must require edge supports between the perpendicular supports. The edge supports are generally parallel with the panel span or the building edge and must be in the diaphragm support plane to allow attachment.
Figure D1-1  Schematic Illustration of Section D1 Parameters
D1.1 Support Connection Shear Strength in Fluted Deck or Panels, $P_{nf}$ and $P_{nfs}$

The *nominal shear strength* [resistance] of a *connection* per support fastener, $P_{nf}$, and per fastener at an edge, $P_{nfs}$, shall be calculated in accordance with (a) or determined by tests in accordance with (b).

(a) **Nominal Connection Shear Strength [Resistance] Determined by Calculation**

Connection strength shall be calculated in accordance with Sections D1.1.1 through D1.1.4, as applicable. Design values of $F_y$ and $F_u$ used in these sections shall be modified in accordance with AISI S100 Section A3.1.2 or A3.1.3 for steels not conforming to AISI S100 Section A3.1.1 unless noted otherwise.

(b) **Nominal Connection Shear Strength [Resistance] Determined by Test**

Tests shall be performed to determine the *nominal connection shear strength* [resistance] in accordance with Section D1.1.5.

**User Note:**

$P_{nf}$ is used to calculate $S_{ni}$ in Eq. D1-1 and $S_{nc}$ in Eq. D1-2, while $P_{nfs}$ is used to calculate $S_{ne}$ in Eq. D1-3. The *connection* detail and location in the *panel* can affect the ability to develop both $P_{nf}$ and $P_{nfs}$ so they are not always the same value. The impact of details on $P_{nfs}$ is discussed in Sections D1 and D1.3.

D1.1.1 Arc Spot Welds or Arc Seam Welds on Steel Supports

Arc spot welding and arc seam welding shall conform to AWS D1.3. Arc spot and arc seam welds shall be for welding steel sheet to thicker supporting members or sheet-to-sheet in the flat position. Arc spot welds (puddle welds) shall not be made on steel supports where the thinnest sheet exceeds 0.15 in. (3.81 mm) in *thickness*, nor through a combination of steel sheets having a total *thickness* over 0.15 in. (3.81 mm). The *nominal shear strength* [resistance] of arc spot welds and arc seam welds without washer shall be determined in accordance with AISI S100 Sections J2.2.2.1 and J2.3.2.1, and meet the edge and end distance requirements in accordance with AISI S100 Sections J2.2.1 and J2.3.1, respectively.

**Note:**

The following two revisions are made in the extracted AISI S100 Sections J2.2.1 and J2.3.1:

1. The *safety* and *resistance factors* and the design methods in AISI S100 Sections J2.2.2.1 and J2.3.2.1 do not apply. The *safety* and *resistance factors* are determined in accordance with Table B-1.

2. This *Standard’s* symbols, $P_{nf}$ or $P_{nfs}$, are inserted for consistent terminology.

[Beginning of Extraction]

**AISI S100 J2.2.1 Minimum Edge and End Distance**

The distance from the centerline of an arc spot weld to the end or edge of the connected member shall not be less than 1.5$d$. In no case shall the clear distance between welds and the end or edge of the member be less than 1.0$d$, where $d$ is the visible diameter of the outer surface of the arc spot weld. See AISI S100 Figures J2.2.1-1 and J2.2.1-2 for details.
AISI S100 J2.2.2.1 Shear Strength for Sheet(s) Welded to a Thicker Supporting Member

The nominal shear strength [resistance], $P_{nf}$ or $P_{nfs}$, of each arc spot weld between the sheet or sheets and a thicker supporting member shall be determined by using the smaller of either (a) or (b).

(a) $P_{nf} = \frac{\pi d_e^2}{4} 0.75 F_{xx}$  

(b) For $(d_a/t) \leq 0.815 \sqrt{E/F_u}$

$P_{nf} = 2.20 t d_a F_u$  

For $0.815 \sqrt{E/F_u} < (d_a/t) < 1.397 \sqrt{E/F_u}$

$P_{nf} = 0.280 \left[ 1 + 5.59 \sqrt{\frac{E}{F_u}} \frac{d_a}{t} \right] t d_a F_u$  

For $(d_a/t) \geq 1.397 \sqrt{E/F_u}$

$P_{nf} = 1.40 t d_a F_u$  

where

$P_{nf}$ = Nominal shear strength [resistance] of arc spot weld

$d_e$ = Effective diameter of fused area at plane of maximum shear transfer

$= 0.7d - 1.5t \leq 0.55d$
where
\( d \) = Visible diameter of outer surface of arc spot weld
\( t \) = Total combined base steel thickness (exclusive of coatings) of sheets involved in shear transfer above plane of maximum shear transfer
\( F_{xx} \) = Tensile strength of electrode classification
\( d_a \) = Average diameter of arc spot weld at mid-thickness of \( t \) where
\( d_a = (d - t) \) for single sheet or multiple sheets not more than four lapped sheets over a supporting member. See AISI S100 Figures J2.2.2.1-1 and J2.2.2.1-2 for diameter definitions.
\( E \) = Modulus of elasticity of steel
\( F_u \) = Tensile strength of sheet as determined in accordance with AISI S100 Section A3.1 or A3.2
**AISI S100 J2.3.1 Minimum Edge and End Distance**

The distance from the centerline of an arc seam weld to the end or edge of the connected member shall not be less than 1.5d. In no case shall the clear distance between welds and the end or edge of the member be less than 1.0d. See AISI S100 Figure J2.3.1-1 for details.

![AISI S100 Figure J2.3.1-1 End and Edge Distances for Arc Seam Welds](image)

**AISI S100 J2.3.2.1 Shear Strength for Sheet(s) Welded to a Thicker Supporting Member**

The nominal shear strength [resistance], $P_{nf}$ or $P_{nfs}$, of arc seam welds shall be determined by using the smaller of either AISI S100 Eq. J2.3.2.1-1 or Eq. J2.3.2.1-2.

\[
P_{nf} = \left[ \frac{\pi d_e^2}{4} + L d_e \right] 0.75F_{xx}
\]

(AISI S100 Eq. J2.3.2.1-1)

\[
P_{nf} = 2.5t F_u (0.25L + 0.96d_a)
\]

(AISI S100 Eq. J2.3.2.1-2)

where

- $P_{nf}$ = *Nominal shear strength [resistance]* of arc seam weld
- $d_e$ = Effective width of seam weld at fused surfaces
  - $= 0.7d - 1.5t$ (AISI S100 Eq. J2.3.2.1-3)

where

- $d$ = Visible width of arc seam weld
- $L$ = Length of seam weld not including circular ends
  - (For computation purposes, $L$ shall not exceed 3d)
- $d_a$ = Average width of seam weld
  - $= (d - t)$ for single or double sheets (AISI S100 Eq. J2.3.2.1-4)
- $F_u$, $F_{xx}$, and $t$ = Values as defined in AISI S100 Section J2.2.2.1

**[End of Extraction]**

For arc spot welds with washers, the nominal shear strength [resistance], $P_{nf}$ or $P_{nfs}$, shall be the lesser of AISI S100 Eq. J2.2.2.1-1 and Eq. D1.1.1-1. To determine $d_e$ in AISI S100 Eq. J2.2.2.1-5, $d$ shall be replaced by $d_o$ and $t$ shall be the thickness of the elements below the washer.
Eqs. D1.1.1-1a and D1.1.1-1b shall apply with the following limits:

(a) $d_o \geq 3/8$ in. (9.53 mm),
(b) $0.05$ in. (1.27 mm) < washer thickness < $0.08$ in. (2.03 mm), and
(c) Washer tensile strength, $F_{u, washer} \geq 45$ ksi (310 MPa), and is permitted to be less than the tensile strength of the element to be welded.

\[
P_{nf} = 99t(1.33d_o + 0.3F_{xx}t)
\]

in U.S. Customary units \( (Eq. \text{ D1.1.1-1a}) \)

\[
P_{nf} = 17.3t\left(\frac{d_o}{19.1} + \frac{F_{xx}t}{584}\right)
\]

in SI units \( (Eq. \text{ D1.1.1-1b}) \)

where

- $d_o$ = Hole diameter in washer, in. (mm)
- $t$ = Total combined base steel thickness (exclusive of coatings) of sheets beneath the washer and above the shear transfer plane, in. (mm)
- $F_{xx}$ = Tensile strength of electrode classification, ksi (MPa)
- $P_{nf}$ = Nominal shear strength [resistance] of arc spot weld with washer, kip (kN)

See AISI S100 Figure J2.2-2 for details.

**D1.1.2 Screws Into Steel Supports**

The minimum spacing, minimum edge and minimum end distances for screws shall satisfy the requirements as specified in AISI S100 Sections J 4.1 and J4.2. The connection nominal shear strength [resistance] per screw, $P_{nf}$ or $P_{nfs}$, shall be determined in accordance with AISI S100 Section J4.3.

**User Note:**
In AISI S100 Section J4.3:

\[
\begin{align*}
d &= \text{Nominal screw diameter} \\
\ t_1 &= \text{Thickness of member in contact with screw head or washer} \\
\ t_2 &= \text{Thickness of member not in contact with screw head or washer} \\
\ F_{u1} &= \text{Tensile strength of member in contact with screw head or washer} \\
\ F_{u2} &= \text{Tensile strength of member not in contact with screw head or washer}
\end{align*}
\]

Eqs. AISI S100 J4.3.1-1 through J4.3.1-5 provide $P_{nf}$ but the same equations also provide $P_{nfs}$ at supports unless noted otherwise.
Note:
The following two revisions are made in the extracted AISI S100 Section J4.3.1:
(1) The term, $P_{nf}$, is substituted for $P_{nv}$ and is consistent with support connection terminology in Sections D1 and D1.1.
(2) Section D1.2.5 refers to Section D1.1.2, and application of this section then provides $P_{ns}$ consistent with sidelap connection terminology in Sections D1 and D1.2.

[Beginning of Extraction]

AISI S100 J4.3.1 Shear Strength Limited by Tilting and Bearing

The nominal shear strength [resistance] of sheet per screw, $P_{nf}$, shall be determined in accordance with this section.

For $t_2/t_1 \leq 1.0$, $P_{nf}$ shall be taken as the smallest of

$$P_{nf} = 4.2 \left( t_2^3 d \right)^{1/2} F_{u2}$$

(AISI S100 Eq. J4.3.1-1)

$$P_{nf} = 2.7 t_1 d F_{u1}$$

(AISI S100 Eq. J4.3.1-2)

$$P_{nf} = 2.7 t_2 d F_{u2}$$

(AISI S100 Eq. J4.3.1-3)

For $t_2/t_1 \geq 2.5$, $P_{nf}$ shall be taken as the smaller of

$$P_{nf} = 2.7 t_1 d F_{u1}$$

(AISI S100 Eq. J4.3.1-4)

$$P_{nf} = 2.7 t_2 d F_{u2}$$

(AISI S100 Eq. J4.3.1-5)

For $1.0 < t_2/t_1 < 2.5$, $P_{nf}$ shall be calculated by linear interpolation between the above two cases.

[End of Extraction]

$P_{nf}$ or $P_{nfs}$ shall not exceed $P_{nss}$ where $P_{nss}$ is the nominal shear breaking strength [resistance] of the screw as reported by the manufacturer or determined by independent laboratory testing in accordance with AISI S904.

User Note:
Although $t_2$ at supports rarely controls the resistance, AISI S100 Eqs. J4.3.1-1 through J4.3.1-5 should be investigated, particularly for cold-formed steel supports. AISI S100 Section J4.3.1 is also applicable in Section D1.2.5. Each screw limit state should be checked at sidelap connections.

D1.1.3 Power-Actuated Fasteners Into Steel Supports

The connection nominal shear strength [resistance] per power-actuated fastener shall be established by tests in accordance with Section D1.1.5.

Nominal shear strength [resistance] of a support connection, $P_{nf}$ or $P_{nfs}$, shall not exceed $P_{npa}$ where $P_{npa}$ is the nominal shear breaking strength [resistance] of the power-actuated fastener as reported by the manufacturer or determined by independent laboratory testing.

User Note:
Within the thickness limits of Chapter D, nominal shear breaking strength [resistance], $P_{npa}$, is unlikely to control for fluted panels. However, $P_{npa}$ might control for cellular deck. Cellular deck is discussed in Section D1.5.
D1.1.4 Fasteners Into Wood Supports

D1.1.4.1 Safety Factors and Resistance Factors

The following safety and resistance factors shall be used to determine the available shear strength [factored resistance] per unit length of diaphragm systems with fasteners into wood supports in accordance with Section C2:

Ω = 3.00 ASD
φ = 0.55 LRFD
= 0.50 LSD

D1.1.4.2 Screw or Nail Connection Strength Through Bottom Flat and Into Support

Where a wood screw or nail is driven through the panel’s bottom flat and into a wood support, the nominal connection shear strength [resistance], \( P_{nf} \) and \( P_{nfs} \), shall be determined as follows:

(a) Wood screw connection is in accordance with Eq. D1.1.4.2-1 or Eq. D1.1.4.2-2, as applicable, and

(b) Nail connection is in accordance with Eq. D1.1.4.2-3 or Eq. D1.1.4.2-4, as applicable.

Wood screws shall have a minimum penetration of 4d into wood. Nails shall have a minimum penetration into wood of 1/3 of the required penetration for full strength as shown in Table D1.1.4.2-2. The spacing for full strength and minimum fastener spacing, end distance and edge distance shall be determined in accordance with AISI S100 Sections J4.1 and J4.2 for steel and AWC NDS for wood.

Screw Strength:

For \( 4d \leq h_s < 7d \)

\[
P_{nf} = \text{Minimum} \left( \frac{h_s}{7d} P_{nfw}, P_{nfs}, P_{nss} \right) \quad (\text{Eq. D1.1.4.2-1})
\]

For \( h_s \geq 7d \)

\[
P_{nf} = \text{Minimum}(P_{nfw}, P_{nfs}, P_{nss}) \quad (\text{Eq. D1.1.4.2-2})
\]

where

\( d \) = Nominal diameter of screw fastener

\( h_s \) = Threaded length of screw, including the tapered tip that is penetrated into the wood support

\( P_{nf} \) = Nominal shear strength [resistance] of connection limited by bearing of the screw or nail against either the wood support or panel, and modified in accordance with penetration

\( P_{nfw} \) = Nominal shear strength [resistance] of wood support connection for fully penetrated screw or nail controlled by bearing against the panel as defined in Table D1.1.4.2-1

\( P_{nfs} \) = Nominal shear strength [resistance] of fully penetrated wood support connection controlled by bearing against the wood as defined in Table D1.1.4.2-1

\( P_{nss} \) = Nominal shear breaking strength [resistance] of screw or nail, as applicable, as reported by the manufacturer or determined by independent laboratory testing.

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Nail Strength:

For \( \frac{h_{sn}}{3} \leq h_{sn} < h_{sf} \)

\[
P_{nf} = \text{Minimum}(\frac{h_{sn}}{h_{sf}}P_{nfw}, P_{nfws}, P_{nss})
\]  
\( (Eq. D1.1.4.2 -3) \)

For \( h_{sn} \geq h_{sf} \)

\[
P_{nf} = \text{Minimum}(P_{nfw}, P_{nfws}, P_{nss})
\]  
\( (Eq. D1.1.4.2-4) \)

where

- \( h_{sn} = \) Length of nail that is penetrated into the wood support
- \( h_{sf} = \) Nail penetration into the wood support as listed in Table D1.1.4.2 -2 to develop full nominal strength [resistance], \( P_{nfw} \).

User Note:

AWC NDS 2012 requires a minimum penetration of 4d for lag screws and 6d for wood screws and nails. These requirements should also be considered when determining the minimum length of nail or screw.

Table D1.1.4.2-1

<table>
<thead>
<tr>
<th>Type</th>
<th>Diameter (d)</th>
<th>( P_{nfws} )</th>
<th>( P_{nfw} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nail</td>
<td>0.148 in.</td>
<td>0.148 in. (3.76 mm)</td>
<td>2.2t₁d F₁₁₁</td>
</tr>
<tr>
<td>Screw</td>
<td>No. 9</td>
<td>0.177 in. (4.50 mm)</td>
<td>2.2t₁d F₁₁₁</td>
</tr>
<tr>
<td></td>
<td>No. 10</td>
<td>0.190 in. (4.83 mm)</td>
<td>2.2t₁d F₁₁₁</td>
</tr>
<tr>
<td></td>
<td>No. 12</td>
<td>0.216 in. (5.49 mm)</td>
<td>2.7t₁d F₁₁₁</td>
</tr>
<tr>
<td></td>
<td>¼ in. (No. 14)</td>
<td>0.242 in. (6.30 mm)</td>
<td>2.7t₁d F₁₁₁</td>
</tr>
</tbody>
</table>

Note:
1. The \( P_{nfw} \) values are for dry and seasoned wood.
2. 0.148-in. (3.76mm) nails are of four types:
   a. 10d pennyweight that are 3 in. (76.2 mm) long and common nail,
   b. 12d pennyweight that are 3½ in. (82.6 mm) long and common nail,
   c. 16d pennyweight that are 3½ in. (82.6 mm) long and sinker nail, and
   d. 20d pennyweight that are 4 in. (102 mm) long and box nail.
3. Steel wire nail material requirements are in ASTM F1667.
4. It is permitted to use the strength of a 0.148 in. (3.76 mm) nail for nails of greater diameter.
5. \( G \) = Specific gravity of the wood as defined in AWC NDS.
6. \( t₁ = \) Thickness of member in contact with screw or nail head.
7. \( F₁₁₁ = \) Tensile strength of member in contact with screw or nail head or washer.
8. For ¼ in. (No 14) screw, it is permitted to use the nominal diameter of 0.25 in. (6.35 mm).
### Table D1.1.4.2-2
Nail Penetration Required for Full Shear Strength

<table>
<thead>
<tr>
<th>Wood Group</th>
<th>G¹</th>
<th>Partial Listing of Wood Species in Group (See AWC NDS for more species listings)</th>
<th>Penetration hsf</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>0.65</td>
<td>Ash, Beech, Birch, Hickory, Black and Sugar Maple, Pecan, Red Oak and White Oak</td>
<td>10d</td>
</tr>
<tr>
<td>II</td>
<td>0.55</td>
<td>Douglas Fir-Larch, Southern Pine, and Sweet Gum</td>
<td>11d</td>
</tr>
<tr>
<td>III</td>
<td>0.45</td>
<td>Douglas Fir-South, Hem-Fir, Eastern and Sitka Spruce, Yellow Poplar, and Pines (Lodgepole, Northern, Ponderosa, Red, and Sugar)</td>
<td>13d</td>
</tr>
<tr>
<td>IV</td>
<td>0.35</td>
<td>Northern White and Western Cedars, Balsam Fir, Eastern White Pine, Engelmann Spruce, and White Woods</td>
<td>14d</td>
</tr>
</tbody>
</table>

**Note:**
¹ The listed G is representative of the specific gravity within the group; more precise listing is contained in AWC NDS.

For fasteners that are not included in Table D1.1.4.2-1, the available shear strength [factored resistance] for connections shall be determined by tests in accordance with Section D1.1.5. In lieu of Section D1.1.5, it is permitted to use AWC NDS to determine the nominal connection shear strength [resistance] per fastener provided the connection safety factor is less than or equal to 3.50 in AWC NDS or the resistance factor is greater than or equal to 0.45.

### D1.1.4.3 Screw or Nail Connection Strength Through Top Flat and Into Support

Where the wood screw or nail in a sidelp connection is driven through the panel’s top flat and into a wood support at an interior panel, the connection nominal shear strength [resistance], \( P_{nf} \), shall be determined as follows:

(a) For a wood screw connection, \( P_{nf} \) is determined in accordance with Eq. D1.1.4.3-1 or Eq. D1.1.4.3-2, as applicable, and

(b) For a nail connection, \( P_{nf} \) is determined in accordance with Eq. D1.1.4.3-3 or Eq. D1.1.4.3-4, as applicable.

Where a wood screw or nail is driven through the panel’s top flat and into an edge, interior, or exterior wood support along the reaction line at an edge panel, the connection nominal shear strength [resistance], \( P_{nfs} \), shall be set equal to 0.0 or a detail shall be provided to allow shear transfer to the lateral force-resisting system’s edge support.  

**User Note:**
\( P_{nfs} \) applies over supports at a lateral force-resisting system line. See Section D1.3 for a discussion of required details and the determination of nominal diaphragm shear strength [resistance] per unit length, \( S_{ne} \). Where connections are through the top flat, the provisions in Section D1.3 are applicable with or without insulation.

Wood screws shall have a minimum penetration of 4d into the wood. Nails shall have a minimum penetration into the wood of 1/3 the required penetration for full strength as shown in Table D1.1.4.2-2. The spacing for full strength and minimum fastener spacing, end distance and edge distance shall be determined in accordance with AISI S100 Sections J4.1 and J4.2 for steel and AWC NDS for wood.
Screw Strength:
For \(4d \leq h_s < 7d\)
\[
P_{nf} = \text{Maximum}(\frac{h_s}{7d}P'_{nf}, P_{ns})
\]  
(Eq. D1.1.4.3-1)
For \(h_s \geq 7d\)
\[
P_{nf} = \text{Maximum}(P'_{nf}, P_{ns})
\]  
(Eq. D1.1.4.3-2)

Nail Strength:
For \(\frac{h_{sn}}{3} \leq h_{sn} < h_{sf}\)
\[
P_{nf} = \frac{h_{sn}}{h_{sf}}P'_{nf}
\]  
(Eq. D1.1.4.3-3)
For \(h_{sn} \geq h_{sf}\)
\[
P_{nf} = P'_{nf}
\]  
(Eq. D1.1.4.3-4)

where
\[d = \text{Nominal diameter of screw fastener}\]
\[P_{nf} = \text{Nominal shear strength [resistance] of connection through top flat of panel and at a sidelap}\]
\[P'_{nf} = \text{Nominal shear strength [resistance] of fully penetrated connection as defined in Table D1.1.4.3-1}\]
\[P_{ns} = \text{Nominal shear strength [resistance] of screw sidelap connection determined using Section D1.1.2, where } t_2 \text{ in AISI S100 Eqs. J4.3.1-2 and J4.3.1-3 is the panel thickness not in contact with screw head. It is permitted to exclude AISI S100 Eq. J4.3.1-1.}\]
\[h_s, h_{sn}, h_{sf} = \text{Values defined in Section D1.1.4.2}\]

\(P_{nf}\) shall not exceed \(P_{nss}\). \(P_{nss}\) shall be as reported by the manufacturer or determined by independent laboratory testing. The AISI S904 test standard shall be used to determine \(P_{nss}\) for screws.

User Note:
\(P_{nss}\) is the nominal shear breaking strength [resistance] of the screw or nail.
The screw or nail is fastened into the support, which inhibits tilting, and the impact of fixity and tilting resistance is in Table D1.1.4.3-1.

For fasteners through interior top flats and into supports, as illustrated in Figure D1.1.4.3-1, \(P_{nf} = 0.0\).
Table D1.1.4.3-1
Nominal Connection Shear Strength of Fastener
With Full Penetration

<table>
<thead>
<tr>
<th>Type</th>
<th>Diameter</th>
<th>$P'_{nf}$ kip (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$t_1$ in.</td>
</tr>
<tr>
<td>Nail</td>
<td>0.148 in. (3.76 mm)</td>
<td>17.3 $t_1$</td>
</tr>
<tr>
<td>Screw</td>
<td>No. 9, 0.177 in. (4.50 mm)</td>
<td>31.8 $t_1$</td>
</tr>
<tr>
<td></td>
<td>No. 10, 0.190 in. (4.83 mm)</td>
<td>33.5 $t_1$</td>
</tr>
</tbody>
</table>

Note:
$t_1$ = Design thickness of thinner element of panel at the sidelap, in. (mm)

For screws or nails that are not included in Table D1.1.4.3-1, the nominal connection shear strength [resistance] shall be determined by tests in accordance with Section D1.1.5. If screws having diameter greater than Table D1.1.4.3-1 are installed at a sidelap, it is permitted to use the $P'_{nf}$ value for the No. 10 screw in Table D1.1.4.3-1 provided the required penetration and spacing are based on the greater screw diameter and nominal connection strength [resistance] is determined in accordance with Eq. D1.1.4.3-1 or D1.1.4.3-2 with $P_{ns}$ based on the greater diameter screw. If nails having diameter greater than 0.148 in. (3.76 mm) are installed at a sidelap, it is permitted to use the $P'_{nf}$ value in Table D1.1.4.3-1 for the 0.148 in. (3.76 mm) nail provided the required penetration and spacing are based on the greater nail diameter and nominal connection strength [resistance] is determined in accordance with Eq. D1.1.4.3-3 or Eq. D1.1.4.3-4.

D1.1.5 Other Connections With Fasteners Into Steel, Wood or Concrete Support

For fasteners in connections that are not included in Sections D1.1.1 through D1.1.4 and otherwise conform to Chapter D limits (a) through (d), the equations for connection nominal fastener strength [resistance] per fastener, $P_{nf}$ and $P_{nfs}$, and safety and resistance factors shall be established by small-scale tests in accordance with Sections E1.1 and E1.2. It is permitted to test connections that are included in Section D1.1.1 through D1.1.4 in accordance with Section E1.1 and E1.2 and to use the tested values in design.

Where both lapped end joints and single thickness joints over interior supports exist, $P_{nf}$ connection strength shall be based on the single thickness shear test.
D1.1.6 Support Connection Strength Controlled by Edge Dimension and Rupture

Multiple lines of support fasteners in an interior flute or at sidelaps over supports shall conform to the shear rupture requirements of AISI S100 Section J6.1 and the block shear rupture requirements of AISI S100 Section J6.3, as applicable.

For any single support fastener or an exterior line of support fasteners with an edge dimension parallel with the force, the minimum edge dimension shall conform to Eq. D1.1.6-1 to develop the full connection nominal shear strength [resistance] per fastener, \( P_{nf} \).

\[
e_{min} = \frac{P_{nf}}{1.2F_u t} \quad \text{(Eq. D1.1.6-1)}
\]

where
- \( e_{min} \) = Clear distance between the end of the material and the edge of weld, fastener, or hole to develop full connection strength [resistance]
- \( = e - \frac{d}{2} \) for arc spot welds, arc seam welds, screws, or power-actuated fasteners

\[
e'_{min} = \frac{R \Omega}{S_n} e_{min} \quad \text{ASD} \quad \text{(Eq. D1.1.6-3a)}
\]
\[
e'_{min} = \frac{R_u}{\phi S_n} e_{min} \quad \text{LRFD} \quad \text{(Eq. D1.1.6-3b)}
\]
\[
e'_{min} = \frac{R_f}{\phi S_n} e_{min} \quad \text{LSD} \quad \text{(Eq. D1.1.6-3c)}
\]

where
- \( R, R_u, R_f \) = Required diaphragm shear strength [shear force due to factored loads] per unit length for ASD, LRFD and LSD, respectively.

See Section C2 for definitions of other variables.

User Note:
As shown in Eqs. D1-1 and D1-2, \( S_n \) is proportional to \( P_{nf} \). AISI S100 Section J6.1 indicates that the nominal shear strength [resistance] per connection controlled by edge dimension is proportional to \( e'_{min} \). Eqs. D1.1.6-1 and D1.1.6-3 are consistent with AISI S100 Eqs. J6.1-1 and J6.1-2.

D1.2 Sidelap Connection Shear Strength [Resistance] in Fluted Deck or Panel, \( P_{ns} \)

The nominal shear strength [resistance] of a sidelap connection per fastener, \( P_{ns} \), shall be calculated in accordance with (a) or determined by tests in accordance with (b).

(a) Connection Shear Strength [Resistance] Determined by Calculation

The nominal shear strength [resistance] of connections shall be determined in accordance with
Sections D1.2.1 through D1.2.6, as applicable. The design values of $F_y$ and $F_u$ used in these sections shall be modified in accordance with AISI S100 Sections A3.1.2 and A3.1.3 for steels not conforming to AISI S100 Section A3.1.1 unless noted otherwise.

(b) Connection Shear Strength [Resistance] Determined by Test
Tests shall be performed to determine the nominal connection shear strength [resistance] in accordance with Section D1.2.7.

**D1.2.1 Arc Spot Welds**

The *sidelap connection nominal shear strength [resistance]*, $P_{ns}$, of arc spot welds shall be calculated in accordance with AISI S100 Section J2.2.2.2. The minimum center-to-center arc spot weld spacing shall be 2.75d.

The following section is extracted from AISI S100 with two revisions:

1. The *safety and resistance factors* and the design methods in AISI S100 Sections J2.2.2.2 shall not apply.

2. The *safety and resistance factors* shall be determined in accordance with Table B-1 and the design methods shall be as listed in Section C2.

**AISI S100 J2.2.2.2 Shear Strength for Sheet-to-Sheet Connections**

The *nominal shear strength [resistance]*, $P_{nv}$, for each weld between two sheets of equal thickness shall be determined in accordance with AISI S100 Eq. J2.2.2.2-1.

$$P_{nv} = 1.65td_aF_u$$  \hspace{1cm} (AISI S100 Eq. J2.2.2.2-1)

where

- $P_{nv}$ = Nominal shear strength [resistance] of sheet-to-sheet connection
- $t$ = Base steel thickness (exclusive of coatings) of single welded sheet
- $d_a$ = Average diameter of arc spot weld at mid-thickness of t. See AISI S100 Figure J2.2.2.2-1 for diameter definitions

\[ d_a = (d - t) \]  \hspace{1cm} (AISI S100 Eq. J2.2.2.2-2)

where

- $d$ = Visible diameter of the outer surface of arc spot weld
- $F_u$ = Tensile strength of sheet as determined in accordance with AISI S100 Sections A3.1 or A3.2

\[ d_a = 0.7d - 1.6t \leq 0.55d \]

\[ d_a = d - t \]

**AISI S100 Figure J2.2.2.2-1 Arc Spot Weld – Sheet-to-Sheet**
In addition, the following limits shall apply:

1. \( F_u \leq 59 \text{ ksi} \) (407 MPa or 4150 kg/cm²)
2. \( F_{xx} > F_u \)
3. \( 0.028 \text{ in.} \) (0.711 mm) \( \leq t \leq 0.0635 \text{ in.} \) (1.61 mm)

See AISI Section J2.2.2.1 for definition of \( F_{xx} \).

[End of Extraction]

Weld washers shall not be used to join deck elements along sidelaps and between supports.

**User Note:**

\( t \) is the total of sheet thickness(es) above the plane of maximum shear transfer. Since the plane is normally between the two sheets, \( t \) equals the thickness of one sheet.

### D1.2.2 Fillet Welds Subject to Longitudinal Shear

The connection nominal shear strength [resistance] shall be determined in accordance with Eq. D1.2.2-1 or Eq. D1.2.2-2, as applicable. The lesser product, \( tF_u \), shall be used to determine \( P_{ns} \) if the sheets vary at the connection.

For \( L_w/t < 25 \),

\[
P_{ns} = \left( 1 - 0.01 \frac{L_w}{t} \right) L_w tF_u
\]  

(Eq. D1.2.2-1)

For \( L_w/t \geq 25 \),

\[
P_{ns} = 0.75 L_w tF_u
\]  

(Eq. D1.2.2-2)

where

- \( L_w \) = Length of fillet weld
- \( t \) = Base steel thickness of thinner steel element at the sidelap weld
- \( F_u \) = Tensile strength of sheet as determined in accordance with AISI S100 Section A3.1 or A3.2, for element corresponding to the thickness, \( t \)

The minimum center-to-center fillet weld spacing shall be 1.4 \( L_w \).

### D1.2.3 Flare Groove Welds Subject to Longitudinal Shear

The connection nominal shear strength [resistance] of flare groove welds shall be determined in accordance with Eq. D1.2.3-1. The lesser product, \( tF_u \), shall be used to determine \( P_{ns} \) if the sheets vary at the connection.

\[
P_{ns} = 0.75 L_w tF_u
\]  

(Eq. D1.2.3-1)

where

- \( L_w \) = Length of groove weld

Other parameters are defined in Section D1.2.2.

The minimum center-to-center flare groove weld spacing shall be 1.15 \( L_w \).
D1.2.4 Top Arc Seam Sidelap Welds Subject to Longitudinal Shear

Eqs. D1.2.4-1 and D1.2.4-2 are applicable within the following limits for steel conforming to Section D1.2 (a) and AISI S100 Section A3:

(a) \( F_{xx} \geq 60 \text{ ksi (415 MPa)}, \)
(b) \( h_{st} \leq 1.25 \text{ in. (31.8 mm)}, \)
(c) \( L_w = 1.00 \text{ in. (25.4 mm) through 2.50 in. (63.5 mm)}, \) and
(d) \( t = 0.028 \text{ in. (0.711 mm) through 0.064 in. (1.63 mm)}. \)

where

\( F_{xx} = \) Tensile strength of electrode classification
\( h_{st} = \) Nominal seam height measured to the top of the seam prior to welding. See Figure D1.2.4-1
\( L_w = \) Length of top arc seam sidelap weld. See Figure D1.2.4-1
\( t = \) Base steel thickness of thinner steel element at the sidelap weld

The nominal connection shear strength [resistance] of top arc seam sidelap welds shall be determined in accordance with Eq. D1.2.4-1. The length of weld, \( L_w \), shall be specified as the minimum length of fused weld along each contributing element’s thickness at the shear transfer plane of the weld.

\[
P_{ns} = \left( 4 \frac{F_u}{F_y} - 1.52 \right) L_w t F_y \left( \frac{t}{L_w} \right)^{0.33} \tag{Eq. D1.2.4-1}
\]

It is permitted to exclude the connection design reduction specified in AISI S100 Sections.
A3.1.2, A3.1.3(b) and A3.1.3(c) for top arc seam sidelap welds. The minimum top arc seam sidelap weld spacing, s, shall be determined in accordance with Eq. D1.2.4-2. The minimum top arc seam sidelap weld end distance between the end of the sheet and the center line of the weld shall be $s/2$.

$$s = \left( \frac{6.67 F_u}{F_y} - 2.53 \right) L_w \left( \frac{t}{L_w} \right)^{0.33}$$

(Eq. D1.2.4-2)

where

- $F_y$ = Yield stress of specified steel corresponding to the thickness, t
- $F_u$ = Tensile strength of sheet as determined in accordance with AISI S100 Section A3.1 or A3.2 corresponding to the thickness, t
- $s$ = Minimum center-to-center spacing of top arc seam sidelap weld

Vertical legs in either hem joints or vertical-to-vertical joints shall fit snugly. In hem joints, the hem shall be crimped onto the vertical leg and the crimp length shall be longer than the specified weld length, $L_w$. Burn through at either one or both ends of the hem is permissible.

**D1.2.5 Sidelap Screw Connections**

The sidelap connection nominal shear strength [resistance], $P_{ns}$, per screw shall be determined in accordance with Section D1.1.2.

**User Note:**

In AISI S100 Eq. J4.3.1-1 through AISI S100 Eq. J4.3.1-5, $t_2$ is the fluted deck or panel thickness not in contact with the screw head.

**D1.2.6 Non-Piercing Button Punch Sidelap Connections**

For fluted panel or deck less than or equal to 3 in. (76.2 mm) in depth, the nominal shear strength [resistance], $P_{ns}$, of a non-piercing button punch sidelap connection shall be:

$$P_{ns} = 0.10 \text{ kips} (0.45 \text{ kN})$$

For fluted panel or deck greater than 3 in. (76.2 mm) in depth or cellular deck as described in Section D1.5, the nominal shear strength [resistance], $P_{ns}$, of a non-piercing button punch sidelap connection shall be ignored, i.e.:

$$P_{ns} = 0.00 \text{ kips} (0.00 \text{ kN})$$

**D1.2.7 Other Sidelap Connections**

For sidelap connections that are not included in Sections D1.2.1 through D1.2.6 and for applications that conform to Chapter D limits (a) through (d), the equation for connection nominal fastener shear strength [resistance] per fastener, $P_{ns}$, and safety and resistance factors shall be established by small-scale tests in accordance with Section E1.1 and Section E1.2. It is permitted to test connections that are included in Section D1.2.1 through D1.2.6 in accordance with Sections E1.1 and E1.2.

**User Note:**

Proprietary crimped or mechanically formed connection shear strengths are determined in accordance with this section.
D1.3 Diaphragm Shear Strength per Unit Length Controlled by Support Connection Strength Through Insulation, $S_{nf}$

The following limits (a) through (f) shall be met for support connections through insulation:

(a) $0.50 \text{ in. (12 mm)} \leq \text{panel depth} \leq 4 \text{ in. (102 mm)}$,
(b) $0.014 \text{ in. (0.356 mm)} \leq \text{base steel thickness of panel} \leq 0.075 \text{ in. (1.91 mm)}$,
(c) $33 \text{ ksi (230 MPa)} \leq \text{specified } F_y \text{ of panel} \leq 80 \text{ ksi (550 MPa)}$,
   $45 \text{ ksi (310 MPa)} \leq \text{specified } F_u \text{ of panel} \leq 82 \text{ ksi (565 MPa)}$,
(d) Support types are steel or wood,
(e) Insulation types are fiberglass with a nominal thickness not exceeding 6 in. (15.2 mm) (R-19), or polyisocyanurate or polystyrene boards with a nominal thickness not exceeding $3 \frac{3}{4} \text{ in. (82.6 mm)}$, and
(f) Deck or panel pitch $\leq 12 \text{ in. (305 mm)}$.

The following additional limits shall be met for connections with screws through fiberglass insulation into steel supports, and where Section D1.3.1 applies:

(g) $0.164 \text{ in. (4.17 mm)} \leq \text{nominal screw diameter} \leq 0.25 \text{ in. (6.35 mm)}$, and
(h) Steel support thickness $\geq 0.043 \text{ in. (1.09 mm)}$.

For diaphragm systems outside the limits (a) through (h), the available strength [factored resistance] of the diaphragm system shall be determined in accordance with Chapter E.

User Note:
Support Connections over insulation are limited to screw connections in Sections D1.3.1 through D1.3.3. For nails and other support connections, see Sections D1.3.4 through D1.3.5.

The nominal diaphragm shear strength [resistance], $S_{nbv}$ per unit length controlled by panel buckling shall be determined in accordance with Section D2. The nominal diaphragm shear strength [resistance] per unit length controlled by nominal connection strength [resistance], $S_{nf}$, with connections at support through insulation shall be the minimum of the nominal diaphragm strengths [resistances], $S_{ni}, S_{nc},$ and $S_{ne}$, determined in accordance with Section D1 as modified below.

$S_{ni}$ and $S_{nc}$ shall be determined using Eqs. D1-1 and D1-2, respectively, where the connection nominal shear strength [resistance] per fastener, $P_{nf}$, for connection types shown in Figures D1.3-1, D1.3-2, D1.3-3, D1.3-4, and D1.3-5 shall be determined in accordance with Sections D1.3.1 through D1.3.5, as applicable.

The nominal diaphragm shear strength [resistance] per unit length at edge panels, $S_{nev}$, shall be determined as follows:

(a) $S_{ne}$ is determined using Eq. D1-3. The edge panel width, $w_e$, is the distance from the adjacent interior panel sidelap to the reaction line;
(b) For screws through bottom flats over compressed fiberglass insulation where the gap between the edge steel support and edge panel is less than or equal to $3/8 \text{ in. (9.53 mm)}$, screw strength, $P_{nfs}$, along the shear reaction transfer-line is determined in accordance with Section D1.3.1.1.

$P_{nf}$ for screws into panel supports at the edge panel is determined, as applicable, in accordance with Section D1.3.1 or D1.3.2 for steel supports and D1.3.2 or D1.3.3 for wood supports.
For other cases at the reaction line, $P_{nfs}$ is determined in accordance with (c).

(c) Where diaphragm shear per unit length flows from one or two sides into the lateral force-resisting system, a detail shall be provided to transfer shear directly to the edge support without going through insulation if any one of the following four conditions exists - see the exception given at (d):

1. Connections are through the bottom flat of panels and the gap caused by insulation between the edge steel support and panel exceeds 3/8 in. (9.53 mm),
2. Support is made of wood,
3. Polyisocyanurate or polystyrene boards are used, or
4. Connections are through the top flats of panels.

(d) If the diaphragm shear force per unit length can be transferred across a reaction line and be resisted by another lateral force-resisting system, then in lieu of providing a detail:

$$P_{nfs} = 0.00 \text{kips} (0.00 \text{kN})$$

**User Note:**

In normal applications, fiberglass insulation is compressed to a thickness between 1/4 in. (6.35 mm) and 3/8 in. (9.53 mm). Some details have thermal breaks over supports to overcome insulation compression at supports. The sum of a thermal break thickness and the compressed fiberglass thickness should be less than 3-1/4 in. (82.6 mm) to apply Section D1.3. Polyisocyanurate or polystyrene typically is not compressed significantly.

Where connections are through top flats, the opposing and stabilizing sidelap shear flow is not present at reaction lines. The stabilized condition is shown in Figure D1.3-4 at supports in the field of diaphragms. $P_{nfs}$ typically is neglected at reaction lines just as $P_{nf}$ is neglected at interior flutes.
Figure D1.3-1  Fasteners Through Bottom at Interior Flutes Over Insulation

Figure D1.3-2  Fasteners at Lap-Down Sidelap Over Insulation

Figure D1.3-3  Fasteners at Lap-Up Sidelap Over Insulation

Figure D1.3-4  Lap-Up Sidelap Over Insulation With Fastener Through Top and Into Support

Figure D1.3-5  Fasteners Through Top at Interior Flutes Over Insulation and Into Support
D1.3.1 Screws Through Bottom Flat of Panel Over Insulation and Into Steel Supports

D1.3.1.1 Screws at Interior Flutes

For screws through bottom flats into steel supports at interior flutes of panels over insulation, as shown in Figure D1.3-1, $P_{nf}$ as used in Eqs. D1-1 and D1-2 shall be reduced to $P'_{nf}$ determined in accordance with Eq. D1.3.1.1-1.

$$P'_{nf} = R P_{nf}$$  \((Eq. D1.3.1.1-1)\)

where

- $\textbf{R} =$ Reduction factor determined in accordance with Table D1.3.1.1-1
- $P_{nf} =$ Screw nominal shear strength [resistance] determined in accordance with Section D1.1.2
- $P'_{nf} =$ Reduced screw nominal shear strength [resistance]

### Table D1.3.1.1-1

Reduction Factor, $R$, for Screws Into Steel Supports, Through Bottom Flat of Panels Over Insulation, and at Interior Flutes

<table>
<thead>
<tr>
<th>Gap Caused by Insulation Between Panel and Support</th>
<th>Load Type or Combinations</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Wind</td>
</tr>
<tr>
<td></td>
<td>Earthquake and All Others</td>
</tr>
<tr>
<td>$0 &lt; \text{Gap} \leq 3/8 \text{ in. (9.53 mm)}$</td>
<td>0.95</td>
</tr>
<tr>
<td>Gap $&gt; 3/8 \text{ in. (9.53 mm)}$</td>
<td>0</td>
</tr>
</tbody>
</table>

**User Note:**
A gap of 3/8 in. (9.53 mm) is associated with a maximum thickness of 6-3/8 in. (160 mm) (R19) fiberglass insulation.

Where insulation is not present over a steel support, the support connection strength is determined using Section D1.1 and no reduction is required. Wood supports with insulation are covered in Section D1.3.2 and D1.3.3, as applicable.

D1.3.1.2 Screws at Exterior Flute With Lap-Down at Sidelap

For screws at lap-down sidelap at exterior flutes of panels over insulation, as shown in Figure D1.3-2, $P_{nf}$ as used in Eqs. D1-1 and D1-2 shall be reduced to $P'_{nf}$ in accordance with Eq. D1.3.1.1-1 where $P_{nf}$ is determined in accordance with Section D1.1.2 and the reduction factor, $R$, is determined in accordance with Table D1.3.1.2-1.

### Table D1.3.1.2-1

Reduction Factor, $R$, for Screws Into Steel Supports, Through Bottom Flat of Panels Over Insulation, and at Exterior Flutes

<table>
<thead>
<tr>
<th>Gap Caused by Insulation Between Panel and Support</th>
<th>Load Type or Combinations</th>
</tr>
</thead>
<tbody>
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</tr>
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<tr>
<td>$0 &lt; \text{Gap} \leq 3/8 \text{ in. (9.53 mm)}$</td>
<td>0.95</td>
</tr>
<tr>
<td>Gap $&gt; 3/8 \text{ in. (9.53 mm)}$</td>
<td>1</td>
</tr>
</tbody>
</table>
User Note:
A common support screw is a screw that penetrates the flats or edges of both panels along a sidelap and then engages or penetrates the support. This requires a minimum of two layers of steel panel. Table D1.3.1.2-1 neglects extra stability at sidelaps at a common screw connection due to opposing motion caused by diaphragm shear where the gap is less than 3/8 in. This allows one $P_{nf}$ value to be used at all flutes, which is the basis of the theory in Chapter D and common practice. $R = 1$ is used at gaps greater than 3/8 in. because the interior flute screw contribution is neglected and only one $P_{nf}$ is available. The maximum insulation thickness is limited by Section D1.3 (e). A screw that is not common at sidelaps does not increase stability relative to interior flutes and Table D1.3.1.1-1 should be used at the sidelap. An example of support screws that are not common is where Section D1.2.6 applies and support screws are at either side of sidelap. This example is shown in Figure C-D1-2.

Rationalization is required to use a reduced value at interior flutes and $R = 1$ at sidelaps where the gap is $\leq 3/8$ in. (9.53 mm) and the screw is common. Use a weighted value, $R$, at interior connections to determine $\alpha_p^2$ and $\alpha_e^2$ in Eqs. D1-8 and D1-9 while $\alpha_s$ in Eq. D1-7 is based on $P_{nf}$ at the sidelap. This is only a design issue where there are few support screws at interior flutes of a panel.

D1.3.1.3 Lap-Up Condition at Sidelap With Screws Not Into Support

For screws at lap-up connections at exterior flutes where the sidelap screws are not into the support as shown in Figure D1.3-3, $P'_{nf}$ as used in Eqs. D1-1 and D1-2 applies to interior flutes and is determined in accordance with Section D1.3.1.1.

Where $P'_{nf} = 0$ kips (0.0 kN) in Section D1.3.1.1, the nominal diaphragm shear strength [resistance] per unit length, $S_{nf}$, shall be determined in accordance with Eq. D1.3.1.3-1.

$$S_{nf} = \frac{n_s P_{ns}}{L} \quad (Eq. D1.3.1.3-1)$$

where

- $n_s =$ Number of sidelap connections along a total panel length, L, and not into supports
- $P_{ns} =$ Nominal shear strength [resistance] of a sidelap (stitch) connection per fastener. See Section D1.2
- $L =$ Total panel length

User Note:
Eq. D1.3.1.3-1 is a special case of Eq. D1-1 where only sidelap screws provide shear resistance. The support screws stabilize the panel and resist uplift.

D1.3.2 Screws Through Top Flat of Panel Over Insulation and Into Steel or Wood Supports

D1.3.2.1 Screws at Interior Flutes

For screws fastened through the top flat at interior flutes of panels over insulation as illustrated in Figure D1.3-5, the connection nominal shear strength [resistance], $P_{nf}$, per fastener shall be neglected:

$$P_{nf} = 0.00 \text{ kips} \ (0.00 \text{ kN})$$
**User Note:**

P_{nf} = 0.00 kips (0.0 kN) is consistent with Sections D1 and D1.1.4.3 and applies either with or without insulation for all support types. This is also consistent with R = 0 at large gaps in Section D1.3.1.1. The contribution of screws through top flats at sidelaps at exterior flutes varies with support type.

The screw limits in Section D1.1.4.2 apply at wood supports.

**D1.3.2.2 Lap-Up Condition With Sidelap Screws Into Support**

For *sidelap* screws at lap-up connections into the support at exterior flutes over insulation as shown in Figure D1.3-4, the *nominal diaphragm shear strength [resistance]* per unit length, S_{nfr} shall be determined in accordance with Section D1 except:

(a) β of Eq. D1-6 is simplified to Eq. D1.3.2.2-1, and

(b) P_{nf} of connection at *sidelap* is determined as follows:

1. \( P_{nf} = P_{ns} \) for supports other than wood, and
2. \( P_{nf} \) is determined in accordance with Section D1.1.4.3 for wood supports.

\[
\beta = n_s \alpha_s + n_f A_p + 2A \tag{Eq. D1.3.2.2-1}
\]

where

A = Number of exterior support connections located at the *sidelap* at an interior panel or edge panel’s end. See Figure D1-1

A_p = Number of interior support connections located at the *sidelap* at an interior panel or edge panel. See Figure D1-1

n_s = Number of *sidelap* connections along a total panel length, L, and not into supports

\( \alpha_s = 1 \) for support other than wood

\[
\alpha_s = \frac{P_{ns}}{P_{nf}} \text{ for wood support} \tag{Eq. D1.3.2.2-2}
\]

where

P_{ns} = Nominal shear strength [resistance] of a *sidelap* connection per fastener determined in accordance with Section D1.2

P_{nf} = Nominal shear strength [resistance] of a *support* connection per fastener at *sidelap* and into wood support in accordance with Section D1.1.4.3

**User Note:**

It is rational to limit the distance from the top of panel to the wood support to 3-¼ in. (82.6 mm) when using Eq. D1.3.2.2-2. This limit includes the contribution of insulation. Otherwise treat as support other than wood.

The screw limits of in Section D1.1.4.2 apply at wood supports.

If the screw at the *sidelap* is not into the support, Eq. D1.3.1.3-1 applies.

**D1.3.3 Screws Through Bottom Flat of Panel Over Insulation and Into Wood Supports**

For screws into wood supports listed in Section D1.1.4.2, and through the bottom flat of
panels over insulation, the nominal diaphragm shear strength [resistance] per unit length, \( S_{nf} \), shall be determined using \( P_{nf} \) for a screw through the top flat and into the support in accordance with Section D1.1.4.3 and the parameters, \( \beta \) and \( \alpha_{s} \), determined in accordance with Eq. D1.3.2.2-1 and Eq. D1.3.2.2-2.

**User Note:**
Screws through top flats into wood supports are controlled by Section D1.3.2 with or without insulation beneath the panel. Any gap associated with insulation is conservatively treated the same as the gap at a screw though the top flat.

### D1.3.4 Nails Through Bottom or Top Flat of Panel Over Insulation and Into Wood Supports

For nails listed in Section D1.1.4.2 and through the bottom or top flat of panels over insulation and into wood supports, the nominal shear strength [resistance] per fastener, \( P_{nf} \), shall be determined as follows:

At interior flutes: \( P_{nf} = 0.00 \) kips (0.00 kN)

At exterior flutes: \( P_{nf} \) is determined in accordance with Section D1.1.4.3

The parameters, \( \beta \) and \( \alpha_{s} \), as used in Eq. D1-1 and D1-2 are determined in accordance with Eq. D1.3.2.2-1 and Eq. D1.3.2.2-2.

**User Note:**
The treatment of nails through top flats into wood supports is consistent with that of screws through top flats in Section D1.3.2 where interior flutes are neglected and the strength, \( P_{nf} \), at exterior flutes is the same with or without insulation beneath the panel. For nails through bottom flats of panels over insulation and into wood supports, the strength, \( P_{nf} \), at exterior flutes is conservatively treated the same as that of a nail through the top flat where the gap is the panel’s depth.

### D1.3.5 Other Support Fasteners Through Insulation

For fasteners that are not listed within Section D1.1.2 or D1.1.4 while all other parameters of the diaphragm system conform to Section D1.3, the connection nominal shear strength [resistance] per fastener, \( P_{nf} \), shall be determined in accordance with Section D1.1.5. The tested support thickness contribution and insulation type shall be consistent with the intended use.

The nominal diaphragm shear strength [resistance] per unit length, \( S_{nf} \), shall be determined in accordance with Sections D1.3.1 through D1.3.4, as applicable, unless full-scale tests in accordance with Section E establish the strength.

### D1.4 Fluted Acoustic Panel With Perforated Elements

Nominal diaphragm shear strength [resistance] per unit length, \( S_{n} \), shall be determined using Section D1. Where acoustic panel connections are not installed at a perforated zone of the panel, \( P_{nf} \) and \( P_{ns} \) are permitted to be determined in accordance with Section D1.1 and Section D1.2, as applicable, using the nominal connection strength [resistance] at an unperforated element.

Where acoustic panel connections are installed at a perforated zone of the panel, the connection nominal shear strength [resistance] per fastener, \( P_{nf} \) or \( P_{ns} \), shall be determined in...
accordance with Section D1.1.5 or Section D1.2.7, as applicable.

Where lapped joints at panel ends and single steel thickness joints over interior supports exist along a panel length, \( L \), \( P_{nf} \) shall be the nominal shear strength [resistance] based on the single steel thickness as used in Eqs. D1-1, D1-2, and D1-3.

**D1.5 Cellular Deck**

Cellular deck nominal diaphragm shear strength [resistance] per unit length, \( S_{n} \), shall be determined using Section D1 provided the following limitations are met:

(a) 0.5 in. (12.7 mm) \( \leq \) cellular deck depth \( \leq \) 7.5 in. (191 mm),
(b) 0.034 in. (0.864 mm) \( \leq \) bottom plate base steel thickness \( \leq \) 0.064 in. (1.63 mm),
(c) 0.034 in. (0.864 mm) \( \leq \) top deck base steel thickness \( \leq \) 0.064 in. (1.63 mm),
(d) Support fastener types are welds, screws, or power-actuated fasteners,
(e) No insulation beneath the cellular deck at the support,
(f) Fastener edge dimensions satisfy requirements specified in AISI S100, and
(g) Deck pitch \( \leq \) 12 in. (305 mm).

**D1.5.1 Safety Factors and Resistance Factors for Cellular Deck**

The safety factors and resistance factors shall be in accordance with Table B-1.

**D1.5.2 Connection Strength and Design**

The following design provisions shall be applicable to combinations of top deck and bottom plate thickness that satisfy Section D1.5:

(a) The nominal shear strength [resistance] of a support connection per fastener, \( P_{nf} \), at an interior flute shall be determined in accordance with Section D1.1 using the total thickness of both top deck and bottom plate that are penetrated by the fastener above the plane of shear transfer at the support.

(b) Where a support fastener is installed at the sidelap, \( P_{nf} \) shall be determined in accordance with Section D1.1 using the thickness(es) of the elements above the plane of shear transfer. The weld effective diameter, \( d_{e} \), of the fused area at the plane of maximum shear transfer shall be based on the total thickness penetrated into the support and determined in accordance with AISI S100 Eq. J2.2.2.1-1 provided in Section D1.1.1.

(c) Where the design does not allow a support fastener to engage both sections of deck at the sidelap:

   (1) Fasteners shall be installed in each deck section at the sidelap, and
   (2) Fasteners shall conform to the required edge and end distances in AISI S100 Chapter J to develop the full nominal shear strength [resistance].

(d) The nominal shear strength [resistance] of a sidelap connection per fastener, \( P_{ns} \), shall be determined in accordance with Section D1.2 using the thickness of the thinner element containing the sidelap fastener. Where the sidelap is button punched:

   (1) \( P_{ns} = 0.00 \), or
   (2) \( P_{ns} \) shall be determined in accordance with Section D1.2.7.

**User Note:**

The contribution of a button punch is not neglected in the determination of \( G' \). See Section D5.2.5.
D1.6 Standing Seam Panels

For standing seam panels that do not conform to the limits (a) through (d) of Chapter D, or for support connections that are not defined in Section D1.1, the nominal diaphragm shear strength [resistance] per unit length, $S_n$, and the diaphragm stiffness, $G'$, of standing seam panels shall be ignored; i.e.:

(a) $S_n = 0.00$, and

(b) $G' = 0.00$.

Alternatively, the nominal diaphragm shear strength [resistance] per unit length and stiffness shall be determined by tests in accordance with Chapter E. The test support thickness and panel thickness shall be consistent with the intended use. If fixity is used at one end of the panel in design application, the test detail shall include fixity at one end. It is permitted to include backer plates or other stiffening details at the other end if they are part of the system design. The backer plates, other stiffening details, and the panel shall not be fastened to that support.

Applications of tests in accordance with Section E1 or E2 shall be limited by the test scope. Extrapolation is not permitted. The safety and resistance factors shall be determined in accordance with Section E1.2.2 or E2.2, as applicable. The safety factor shall be greater than or equal to and the resistance factor shall be less than or equal to those determined in accordance with Table B-1.

User Note:
AISI CF97-1, A Guide for Designing With Standing Seam Roof Panels, is a design guide for standing seam panels under various loading conditions. It defines test procedures to isolate the strength and stiffness of the diaphragm as required to determine the bracing capacity of the roof system.

D1.7 Double-Skinned Panels

The nominal diaphragm shear strength [resistance] per unit length, $S_n$, for double-skinned panels, as illustrated in Figure D1.7-1, shall be determined in accordance with Chapter D by neglecting the contribution of the top panel. The following conditions shall apply:

(a) A bottom panel is fastened directly to a structural support,

(b) Sub-girts or sub-purlins are fastened to the bottom panel at an elevated plane, and

(c) A top panel is fastened to the sub-girts or sub-purlins.

In addition, $\lambda = 1$ in Eq. D1-1, and the calculated available diaphragm shear strength [factored resistance] per unit length shall satisfy Eq. D1.7-1:

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\[
\frac{S_n}{\Omega} \leq \frac{S_{nw}}{\Omega_v} \quad \text{for ASD} \quad \text{(Eq. D1.7-1a)}
\]
\[
\phi S_n \leq \phi_v S_{nw} \quad \text{for LRFD and LSD} \quad \text{(Eq. D1.7-1b)}
\]
where
\[
\Omega = \text{Safety factor for diaphragm system determined in accordance with Table B-1}
\]
\[
\phi = \text{Resistance factor for diaphragm system determined in accordance with Table B-1}
\]
\[
S_{nw} = \text{Nominal shear strength [resistance] per unit length of the bottom panel acting as a web}
\]
\[
V_n \quad \text{(Eq. D1.7-2)}
\]
where
\[
V_n = \text{Nominal shear strength [resistance] per unit width between webs of the bottom panel determined in accordance with AISI S100, Section G2}
\]
\[
\Omega_v = 1.60 \text{ for ASD}
\]
\[
\phi_v = 0.95 \text{ for LRFD}
\]
\[
= 0.80 \text{ for LSD}
\]
If an end closure is detailed to transfer the shear from the top panel to the support at panel ends and over a lateral force-resisting system, the available shear strength [factored resistance] per unit length of the diaphragm is permitted to include the additive contribution of the top panel as determined by test in accordance with Chapter E.

**User Note:**
Eq. D1.7-1 applies to both perforated and solid bottom panels. Perforations can impact the nominal shear strength [resistance] per unit length determined using Eq. D1.7-2.

**D2 Diaphragm Shear Strength per Unit Length Controlled by Stability, S_{nb}**

The nominal shear strength [resistance] per unit length of a diaphragm or wall diaphragm controlled by panel stability, S_{nb}, for either acoustic or non-acoustic fluted panels shall be the smallest of S_{no} and S_{nf}, calculated in accordance with Eq. D2-1 and D2-2, respectively.

\[
S_{no} = \frac{7890}{\alpha L_v^2} \left( \frac{L_x g t^3 d}{s} \right)^{0.25} \quad \text{(Eq. D2-1)}
\]
\[
S_{nf} = P_{nw} \left( \frac{d-e}{D_d} \right) \left( \frac{1}{d} \right) \quad \text{(Eq. D2-2)}
\]
where
\[
S_{no} = \text{Nominal diaphragm shear strength [resistance] per unit length controlled by panel out-of-plane buckling, kip/ft (kN/m)}
\]
\[
S_{nf} = \text{Nominal diaphragm shear strength [resistance] per unit length controlled by local web buckling of panel over exterior support}
\]
\[
\alpha = \text{Conversion factor for units}
\]
\[
= 1 \quad \text{for U.S. Customary units}
\]
\[
= 1879 \text{ for SI units}
\]
\[
L_v = \text{Span of panel between supports with fasteners, ft (m)}
\]
\[ I_{xg} = \text{Moment of inertia of fully effective panel per unit width, in.}^4/\text{ft} \quad (\text{mm}^4/\text{mm}) \]
\[ t = \text{Base steel thickness of panel, in.} \quad (\text{mm}) \]
\[ d = \text{Panel corrugation pitch, in.} \quad (\text{mm}) \]
\[ e = \text{One-half the bottom flat width of panel measured between points of intercept as illustrated in Figure D2-1, in.} \quad (\text{mm}) \]
\[ D_d = \text{Depth of panel illustrated in Figure D2-1, in.} \quad (\text{mm}) \]
\[ P_{nw} = \text{Nominal web crippling strength [resistance] per web} \]

\[ = 4.36t^2 F_y \sin \theta \left( 1 - 0.04 \sqrt{\frac{R}{t}} \right) \left( 1 + 0.25 \sqrt{\frac{N_{ext}}{t}} \right) \left( 1 - 0.025 \sqrt{\frac{h}{q_s t}} \right) \quad \text{(Eq. D2-3)} \]

where

\[ F_y = \text{Design yield stress as determined in accordance with AISI S100 Section A3.3.1} \]
\[ \theta = \text{Angle between plane of web and plane of bearing surface, } 45^\circ \leq \theta \leq 90 \]
\[ R = \text{Inside bend radius} \]
\[ N_{ext} = \text{Bearing length at exterior support (3/4 in. (19 mm) minimum)} \]
\[ h = \text{Flat dimension of web measured in plane of web} \]
\[ q_s = \text{Perforated web adjustment factor} \]
\[ = 1 \text{ for panels with solid webs} \]

\[ = 1 - (1 - k) \left( \frac{w_p}{h} \right) \quad \text{(Eq. D2-4)} \]

where

\[ k = \text{Ratio of perforated element stiffness to that of a solid element of the same thickness, } t, \text{ determined in accordance with Appendix 1, Eq. 1.6-5} \]
\[ w_p = \text{Width of perforation band in the web flat of width, } w \]
\[ s = \text{Developed flute width per pitch, in.} \quad (\text{mm}) \]

\[ = 2(e + w) + f \quad \text{(Eq. D2-5)} \]

where

\[ w = \text{Web flat width of panel measured between points of intercept illustrated in Figure D2-1, in.} \quad (\text{mm}) \]
\[ f = \text{Top flat width of panel measured between points of intercept illustrated in Figure D2-1, in.} \quad (\text{mm}) \]

---

**Figure D2-1 Panel Configuration**
For fluted *acoustic panels*, the following shall apply:

(a) The developed flute width, \( s \), is determined in accordance with Eq. D2-5 using the modified element lengths in Appendix 1 Section 1.6 by setting \( e = e_p \), \( w = w_p \), and \( f = f_p \).

(b) The modified panel moment of inertia, \( I_{xg} \), is obtained from the manufacturer, and

(c) Other parameters in Eq. D2-1 are not modified.

**D2.1 Cellular Deck**

The *nominal diaphragm shear strength [resistance]* per unit length controlled by *panel out-of-plane buckling*, \( S_{nb} \), for either *cellular deck* or *cellular acoustic deck* shall be calculated using Eq. D2-1 for all span applications as modified below:

(a) \( I_{xg} = \) Moment of inertia of fully effective *cellular deck* per unit width, in.\(^4\)/ft (mm\(^4\)/mm), and

(b) \( t, s, d = \) Properties of the top fluted *deck* in *cellular deck*.

The moment of inertia, \( I_{xg} \), shall be modified for perforation in the top or bottom elements, as applicable. The modified \( I_{xg} \) is permitted to be obtained from the manufacturer. If the top *deck* is perforated, the top *deck* property, \( s \), shall be modified in accordance with Section D2 as specified for *acoustic panels*. Other parameters in Eq. D2-1 shall not be modified.

**D3 Shear and Uplift Interaction**

**D3.1 Support Connections**

Where a *support connection* is subjected to combined shear and tension, the *nominal shear strength [resistance]* of a *support connection* per fastener, \( P_{nf} \), at *edge and interior panels* shall be reduced to the *nominal shear strength [resistance]* of a *support connection* per fastener in the presence of a tensile *load*, \( P_{nft} \). The *nominal diaphragm strengths [resistances]* per unit length, \( S_{ni} \) and \( S_{nc} \), shall be calculated in accordance with Section D1 by setting \( P_{nf} \) equal to \( P_{nft} \).

Where applicable at *edge supports*, \( S_{ne} \) shall be calculated in accordance with Section D1 by setting \( P_{nf} \) equal to \( P_{nft} \) in Eq. D1-3. The *nominal shear strength [resistance]*, \( P_{nfs} \), of an edge *support connection* between transverse supports at an *edge panel* is permitted to not be reduced for combined shear and tension.

\( P_{nft} \) shall be the smallest interaction value controlled by:

(a) Pull-over in the *connection*,

(b) Pull-out in the *connection*, and

(c) Breaking *strength* of the fastener.

The interaction equations to determine \( P_{nft} \) for prequalified *support connections* are listed in Sections D3.1.1 through D3.1.4. It is permitted to establish the interaction equations to determine \( P_{nft} \) for prequalified or alternative *connections* by small-scale tests in accordance with Sections E1.1 and E1.2. Where the *connection* strength is determined by tests in accordance with Section D1.1.5 for shear or AISI S100 Section K2.1 for tension, the interaction effect for any or all *limit states* shall be determined by:

(a) Test in accordance with AISI S905, or

(b) Defaulting to a linear interaction effect without additional tests.
User Note:
A linear interaction effect is consistent with providing sufficient fasteners to resist each required strength [effect due to factored loads] component separately. Eqs. D3.1.2.1-13 and -14, and D3.1.3-1a and variable b are linear interaction effects. The safety factor and resistance factor in linear interaction equations for a connection depend on the source of the nominal tension strength [resistance] for each limit state.

D3.1.1 Arc Spot Welds

P_{nft} shall be determined in accordance with Eq. D3.1.1-1 or Eq. D3.1.1-2, as applicable, for ASD; and in accordance with Eq. D3.1.1-3 or Eq. D3.1.1-4, as applicable, for LRFD and LSD.

For ASD

If \[ \left( \frac{\Omega_t T}{P_{nt}} \right)^{1.5} \leq 0.15 \]
\[ P_{nft} = P_{nf} \]  
(Eq. D3.1.1-1)

If \[ \left( \frac{\Omega_t T}{P_{nt}} \right)^{1.5} > 0.15 \]
\[ \left( \frac{P_{nft}}{P_{nf}} \right)^{1.5} + \left( \frac{\Omega_t T}{P_{nt}} \right)^{1.5} = 1 \]  
(Eq. D3.1.1-2)

where
- \( T \) = Required allowable tensile strength of a support connection per weld determined for loads and load combinations in accordance with Section A3
- \( P_{nf} \) = Nominal shear strength [resistance] of a support connection per weld in the absence of a tensile load and determined in accordance with Section D1.1.1
- \( P_{nft} \) = Nominal shear strength [resistance] of a support connection per weld in the presence of a tensile load
- \( P_{nt} \) = Nominal tension strength [resistance] of a support connection per weld determined using AISI S100 Section J2.2.3
- \( \Omega_t \) = Safety factor for a connection subjected to tension
  = 2.5 for a weld in deck applications

For LRFD and LSD

If \[ \left( \frac{T}{\phi_t P_{nt}} \right)^{1.5} \leq 0.15 , \]
\[ P_{nft} = P_{nf} \]  
(Eq. D3.1.1-3)

If \[ \left( \frac{T}{\phi_t P_{nt}} \right)^{1.5} > 0.15 , \]
\[ \left( \frac{P_{nft}}{P_{nf}} \right)^{1.5} + \left( \frac{T}{\phi_t P_{nt}} \right)^{1.5} = 1 \]  
(Eq. D3.1.1-4)
where
\[ \bar{T} = \text{Required tensile strength [tensile force due to factored loads] per weld} \]
determined for loads and load combinations in accordance with Section A3
\[ = T_u \text{ for LRFD} \]
\[ = T_f \text{ for LSD} \]
\[ \phi_t = \text{Resistance factor for a connection subjected to tension} \]
\[ = 0.60 \text{ for a weld in deck applications in LRFD} \]
\[ = 0.50 \text{ for a weld in deck applications in LSD} \]

**D3.1.2 Screws**

**D3.1.2.1 Screws Into Steel Supports**

\( P_{nf} \) shall be the smallest value controlled by cases (a), (b), and (c), as applicable:

(a) Interaction of Shear and Pull-Over

\( P_{nf} \) shall be the smaller value determined using Eq. D3.1.2.1-1 and Eq. D3.1.2.1-2 for ASD, or Eq. D3.1.2.1-3 and Eq. D3.1.2.1-4 for LRFD and LSD provided that the limits of AISI S100 Section J4.5.1 are met.

For ASD

\[ P_{nf} = P_{nf} \]  \hspace{1cm} (Eq. D3.1.2.1-1)

\[ \left( \frac{P_{nf}}{\Omega_d P_{nf}} \right) + \left( \frac{0.71 T}{P_{nov}} \right) = 1.1 \] \hspace{1cm} (Eq. D3.1.2.1-2)

where

\[ P_{nf} = \text{Nominal shear strength [resistance] of a support connection per screw in the presence of a tensile load} \]
\[ P_{nf} = \text{Nominal shear strength [resistance] of a support connection per screw determined in accordance with Section D1.1.2} \]
\[ T = \text{Required allowable tensile strength of a support connection per screw determined for loads and load combinations in accordance with Section A3} \]
\[ P_{nov} = \text{Nominal tension strength [resistance] of a support connection per screw controlled by pull-over and determined in accordance with AISI S100, Section J4.4.2} \]
\[ \Omega = \text{Safety factor for a screw connection subjected to combined shear and pull-over interaction} \]
\[ = 2.35 \]
\[ \Omega_d = \text{Safety factor for a diaphragm controlled by connections and determined in accordance with Table B1.1-1} \]
\[ = 2.00 \text{ for a screw connection subject to wind loads} \]

For LRFD and LSD

\[ P_{nf} = P_{nf} \]  \hspace{1cm} (Eq. D3.1.2.1-3)

\[ \left( \frac{\phi_d P_{nf}}{P_{nf}} \right) + \left( \frac{0.71 T}{P_{nov}} \right) = 1.1 \phi \] \hspace{1cm} (Eq. D3.1.2.1-4)
where

\[ \phi = \text{Resistance factor for a screw connection subjected to combined shear and pull-over interaction} \]
\[ = 0.65 \text{ for LRFD} \]
\[ = 0.55 \text{ for LSD} \]

\[ \phi_d = \text{Resistance factor for a diaphragm controlled by connections and determined in accordance with Table B1.1-1} \]
\[ = 0.80 \text{ for a screw connection subject to wind loads in LRFD} \]
\[ = 0.75 \text{ for a screw connection subject to wind loads in LSD} \]

\[ \overline{T} = \text{Required tensile strength [tensile force due to factored loads] per screw determined for loads and load combinations in accordance with Section A3} \]
\[ = T_u \text{ for LRFD} \]
\[ = T_f \text{ for LSD} \]

**User Note:**
Anomalies exist at Eq. D3.1.2.1-2 and Eq. D3.1.2.1-4 where \( \overline{T} \) approaches 0.00. Refer to the Commentary for more information.

AISI S100 Section J4.5.1 does not include pull-over strength for panels with insulation between the panel and a support, and tests or rational engineering analysis are required to determine pull-over strength. Many panel manufacturers have performed large-scale tests with insulation and may be able to provide the necessary information.

(b) Interaction of Shear and Pull-Out

\( P_{nft} \) shall be the smaller value using Eq. D3.1.2.1-5 and Eq. D3.1.2.1-6 for ASD, or Eq. D3.1.2.1-7 and Eq. D3.1.2.1-8 for LRFD and LSD provided that the limits of AISI S100 Section J4.5.2 are met. \( P_{nft} \) is determined in accordance with Section D1.1.2.

For ASD

\[ P_{nft} = P_{nf} \quad (Eq. \ D3.1.2.1-5) \]
\[ \left( \frac{P_{nft}}{\phi_d P_{nf}} \right) + \left( \frac{T}{P_{not}} \right) = \frac{1.15}{\Omega} \quad (Eq. \ D3.1.2.1-6) \]

where

\[ P_{not} = \text{Nominal tension strength [resistance] of a support connection per screw controlled by pull-out determined in accordance with AISI S100, Section J4.4.1} \]
\[ \Omega = \text{Safety factor for a screw connection subjected to combined shear and pull-out interaction} \]
\[ = 2.55 \]

Other parameters are defined in Section D3.1.2.1(a).

For LRFD and LSD

\[ P_{nft} = P_{nf} \quad (Eq. \ D3.1.2.1-7) \]
\[ \left( \phi_d P_{nft} \right) + \left( \frac{T}{P_{not}} \right) = 1.15\phi \quad (Eq. \ D3.1.2.1-8) \]
where
\[ \phi = \text{Resistance factor for a screw connection subjected to combined shear and pull-out interaction} \]
\[ = 0.60 \text{ for LRFD} \]
\[ = 0.50 \text{ for LSD} \]

Other parameters are defined in Section D3.1.2.1(a).

User Note:
Anomalies exist at Eq. D3.1.2.1-8 where \( T \) approaches 0.00. Refer to the Commentary for more information.

(c) Interaction of Shear and Tension in the Screw
Where \( P_{nss} \) controls \( P_{nf} \) and \( P_{nts} \) controls \( P_{nt} \), \( P_{nft} \) shall be the smaller value using Eq. D3.1.2.1-9 and Eq. D3.1.2.1-10 for ASD, or Eq. D3.1.2.1-11 and Eq. D3.1.2.1-12 for LRFD and LSD.

For ASD
\[ P_{nft} = P_{nf} \quad (Eq. \text{D3.1.2.1-9}) \]
\[ \frac{P_{nft}}{P_{nf}} + \frac{\Omega T}{P_{nts}} = 1.3 \quad (Eq. \text{D3.1.2.1-10}) \]

For LRFD and LSD
\[ P_{nft} = P_{nf} \quad (Eq. \text{D3.1.2.1-11}) \]
\[ \frac{P_{nft}}{P_{nf}} + \frac{T}{\phi P_{nts}} = 1.3 \quad (Eq. \text{D3.1.2.1-12}) \]

where
\[ P_{nss} = \text{Nominal shear breaking strength [resistance] of screw reported by the manufacturer or determined by independent laboratory testing in accordance with AISI S904} \]
\[ P_{nts} = \text{Nominal tensile breaking strength [resistance] of screw reported by the manufacturer or determined by independent laboratory testing in accordance with AISI S904} \]
\[ \Omega_t = \text{Safety factor for a connection subjected to tension} \]
\[ = 3.0 \text{ for a screw} \]
\[ \phi_t = \text{Resistance factor for a connection subjected to tension} \]
\[ = 0.50 \text{ for a screw in LRFD} \]
\[ = 0.40 \text{ for a screw in LSD} \]

Other parameters are defined in Section D3.1.2.1(a).

Tests shall be performed for conditions outside the limits of Section D3.1.2.1 (a), (b) or (c). Shear or tension tests are permitted for screws in accordance with Section D1.1.5. It is permitted to determine the nominal shear strength [resistance], \( P_{nfb} \) using Eqs. D3.1.2.1-13 to D3.1.2.1-15 for ASD or Eqs. D3.1.2.1-16 to D3.1.2.1-18 for LRFD or LSD, as applicable, for connection strengths based on small-scale tests for shear or tension with tension controlled by pull-over, pull-out, or breaking strength.
For ASD
\[
\frac{P_{nf}}{P_{nov}} + \frac{\Omega_T}{P_{nov}} = 1 \quad (Eq. D3.1.2.1-13)
\]
\[
\frac{P_{nf}}{P_{not}} + \frac{\Omega_T}{P_{not}} = 1 \quad (Eq. D3.1.2.1-14)
\]
\[
\frac{P_{nf}}{P_{nts}} + \frac{\Omega_T}{P_{nts}} = 1 \quad (Eq. D3.1.2.1-15)
\]
For LRFD and LSD
\[
\frac{P_{nf}}{P_{nov}} + \frac{\phi_T P_{nov}}{P_{nov}} = 1 \quad (Eq. D3.1.2.1-16)
\]
\[
\frac{P_{nf}}{P_{not}} + \frac{\phi_T P_{not}}{P_{not}} = 1 \quad (Eq. D3.1.2.1-17)
\]
\[
\frac{P_{nf}}{P_{nts}} + \frac{\phi_T P_{nts}}{P_{nts}} = 1 \quad (Eq. D3.1.2.1-18)
\]
where
\(\Omega_T\) = Safety factor for a connection subjected to tension in accordance with AISI S100 Eq. K2.1.2-2

\(\phi_T\) = Resistance factor for a connection subjected to tension and determined in accordance with Section E1.2.2(b)

It is permitted to use \(\Omega_T\) and \(\phi_T\) conforming to AISI Section J4.4.2 or J4.4.1 where \(P_{nov}\) or \(P_{not}\) are determined in accordance with AISI Section J4.4.2 or J4.4.1, respectively.

It is permitted to use \(\Omega_T\) and \(\phi_T\) determined in accordance with AISI Section J4 for nominal tensile breaking strength, \(P_{nts}\).

**D3.1.2.2 Screws Through Bottom Flats Into Wood Supports**

\(P_{nf}\) shall be the least strength determined in accordance with Sections D3.1.2.1(a) for shear and pull-over, D3.1.2.1(c) for shear and tension in the screw, and Eq. D3.1.2.2-1 for shear and tension controlled by wood. In Section D3.1.2.1(a), \(P_{nf}\) shall be determined in accordance with Section D1.1.4.2, where \(P_{nf}\) is limited by nominal bearing strength [resistance] against steel, \(P_{nfw}\), and \(P_{nss}\), but not by nominal bearing strength [resistance] against wood, \(P_{nfw}\).

\[
P_{nf} = \frac{\cos \theta}{\cos^2 \theta + \frac{P_{nfw}}{P_{nT} \sin^2 \theta}} \quad (Eq. D3.1.2.2-1a)
\]

\[
P_{nf} \leq P'_{nfw} \quad (Eq. D3.1.2.2-1b)
\]

where
\[
\theta = \tan^{-1}\left(\frac{T}{V}\right) \quad \text{for ASD} \quad (Eq. D3.1.2.2-2a)
\]

\[
\theta = \tan^{-1}\left(\frac{T}{V}\right) \quad \text{for LRFD and LSD} \quad (Eq. D3.1.2.2-2b)
\]
$V \leq \frac{P_{nft}}{\Omega}$ for ASD 

$V \leq \phi P_{nft}$ for LRFD and LSD

$\frac{P_{nft}}{P_{nfw}} \geq \left( \frac{\Omega S_{req}}{S_{nf}} \right)$ for ASD

$\frac{P_{nft}}{P_{nfw}} \geq \left( \frac{S_{req}}{\phi S_{nf}} \right)$ for LRFD and LSD

$P_{nf} = \text{Nominal shear strength [resistance] of a support connection per screw in the absence of a tensile load}$

$P_{nft} = \text{Nominal shear strength [resistance] of a support connection per screw in the presence of a tensile load}$

$P_{nfw} = \text{Nominal shear strength [resistance] of a support connection per screw controlled by bearing against the wood and modified for wood penetration}$

$= \frac{h_s}{7d} P_{nfw}$ for $4d \leq h_s < 7d$ 

$= P_{nfw}$ for $h_s \geq 7d$

$P_{nfw} = \text{Nominal shear strength [resistance] of fully penetrated wood support connection controlled by bearing against wood determined in accordance with Table D1.1.4.2-1}$

$P_{nT} = \text{Nominal pull-out strength [resistance] per wood support screw in the absence of a shear load, kips (kN)}$

$= 6.16 \alpha G^2 d h_s$ 

where

$G = \text{Specific gravity of wood defined in Section D1.1.4.2}$

$h_s = \text{Threaded length of screw, including the tapered tip that is penetrated into wood support, in. (mm)}$

$d = \text{Nominal diameter of screw, in. (mm)}$

$\alpha = \text{Conversion factor for units}$

$= 1$ for U.S. Customary unit  

$= 0.0069$ for SI unit

$S_{req} = \text{Required diaphragm shear strength [shear force due to factored load] per unit length determined for load and load combinations in accordance with Section A3}$

$T = \text{Required allowable tensile strength of a support connection per screw determined for ASD loads and load combinations in accordance with Section A3}$

$\bar{T} = \text{Required tensile strength [tensile force due to factored loads] of a support connection per screw determined for LRFD or LSD loads and load combinations in accordance with Section A3}$

$= T_u$ for LRFD  

$= T_f$ for LSD

$V = \text{Required allowable shear strength of a support connection per screw determined by ASD load and load combinations in accordance with Section A3}$
\[ \nabla \quad = \text{Required shear strength [shear force due to factored loads] of a support connection per screw determined for LRFD or LSD load and load combinations in accordance with Section A3}
\]
\[ = V_u \text{ for LRFD} \]
\[ = V_f \text{ for LSD} \]
\[ \Omega \quad = 3.00 \text{ for ASD} \]
\[ \phi \quad = 0.55 \text{ for LRFD} \]
\[ = 0.50 \text{ for LSD} \]

**User Note:**
In Eq. D3.1.2.2-7, \( h_s \) is not limited to \( 7d \).

### D3.1.3 Power-Actuated Fasteners

The shear and uplift (tension) connection interaction shall be established by small-scale tests. The safety factor and resistance factor of the interaction equation shall be determined in accordance with Section E1.2.2. In lieu of interaction testing, \( P_{nft} \) is permitted to be determined using Eq. D3.1.3-1.

For ASD
\[
\frac{P_{nft}}{P_{nf}} + \frac{T}{\min\left(\frac{P_{nov}}{\Omega_{tov}}, \frac{P_{not}}{\Omega_{tot}}\right)} = 1
\]
\[(Eq. \text{ D3.1.3-1a)}\]

For LRFD and LSD
\[
\frac{P_{nft}}{P_{nf}} + \frac{T}{\min(\phi_{tov} P_{nov}, \phi_{tot} P_{not})} = 1
\]
\[(Eq. \text{ D3.1.3-1b)}\]

where
\[
P_{nf} = \text{Nominal shear strength [resistance] of a support connection per fastener in the absence of a tensile load}
\]
\[
P_{nft} = \text{Nominal shear strength [resistance] of a support connection per fastener in the presence of a tensile load}
\]
\[
P_{nov} = \text{Nominal tension strength [resistance] of a support connection per power-actuated fastener controlled by pull-over}
\]
\[
P_{not} = \text{Nominal tension strength [resistance] of a support connection per power-actuated fastener controlled by pull-out}
\]
\[
T = \text{Required allowable tensile strength of a support connection per fastener determined for ASD loads and load combinations in accordance with Section A3}
\]
\[
\bar{T} = \text{Required tensile strength [tensile force due to factored loads] of a support connection per fastener determined for LRFD or LSD loads and load combinations in accordance with Section A3}
\]
\[ = T_u \text{ for LRFD} \]
\[ = T_f \text{ for LSD} \]
\[
\Omega_{tov} = \text{Safety factor for a power-actuated fastener controlled by pull-over} \]
\[ = 3.00 \text{ (ASD)} \]
\[
\Omega_{tot} = \text{Safety factor for a power-actuated fastener controlled by pull-out and determined} \]
D3.1.4 Nails Through Bottom Flats Into Wood Supports

\( P_{\text{nft}} \) shall be the least connection shear strength [resistance] per nail determined using Sections D3.1.2.1(a) for shear and pull-over, and D3.1.2.1(c) for shear and tension in the fastener where nail is substituted for screw, and Eq. D3.1.4-1 for shear and tension controlled by wood. In Section D3.1.2.1(a), \( P_n \) shall be determined in accordance with Section D1.1.4.2 where \( P_n \) is limited by nominal bearing strength [resistance] against steel, \( P_{nfw} \), and \( P_{nss} \), but not by nominal bearing strength [resistance] against wood, \( P_{nfw} \). \( P_{nov} \) is determined using the nail head or washer diameter in AISI S100 Eq. J4.4.2-1.

\[
\frac{P_{\text{nft}}}{P_{\text{nfw}}} + \frac{\Omega T}{P_{\text{nT}}} = 1.0 \quad \text{for ASD} \quad (\text{Eq. D3.1.4-1a})
\]

\[
\frac{P_{\text{nft}}}{P_{\text{nfw}}} + \frac{T}{\phi P_{\text{nT}}} = 1.0 \quad \text{for LRFD and LSD} \quad (\text{Eq. D3.1.4-1b})
\]

where

\( P_n = \text{Nominal shear strength [resistance] of a support connection per nail in the absence of a tensile load and determined in accordance with Section D1.1.4.2} \)

\( P_{nft} = \text{Nominal shear strength [resistance] of a support connection per nail in the presence of a tensile load} \)

\( P_{nfw} = \text{Nominal shear strength [resistance] of a support connection per nail controlled by bearing against wood and modified for wood penetration} \)

\[
= \frac{h_{sn}}{h_{sf}} P_{nfw} \quad \text{For } \frac{h_{sf}}{3} \leq h_{sn} < h_{sf} \quad (\text{Eq. D3.1.4-2})
\]

\[
= P_{nfw} \quad \text{For } h_{sn} \geq h_{sf} \quad (\text{Eq. D3.1.4-3})
\]

where

\( h_{sf} \) and \( P_{nfw} \) = Values defined in Section D1.1.4.2

\( P_{nT} = \text{Nominal pull-out strength [resistance] per nail in the absence of a shear load, kips (kN)} \)

\[
= 2.98\alpha G^{2.5} dh_{sn} \quad (\text{Eq. D3.1.4-4})
\]

where

\( G \) = Specific gravity of wood defined in Section D1.1.4.2

\( h_{sn} \) = Length of nail that is penetrated into the wood support, in. (mm) and not
limited to embedment depth, \( h_{sf} \), as given in Table D1.1.4.2-2

\[
\begin{align*}
d &= \text{Nominal diameter of nail, in. (mm)} \\
\alpha &= \text{Conversion factor for units} \\
&= 1 \text{ for U.S. Customary unit} \\
&= 0.0069 \text{ for SI unit} \\
T &= \text{Required allowable tensile strength of a support connection per nail determined for ASD loads and load combinations} \\
&\text{in accordance with Section A3} \\
\bar{T} &= \text{Required tensile strength [tensile force due to factored loads] of a support connection} \\
&\text{per nail determined for LRFD or LSD loads and load combinations} \\
&\text{in accordance with Section A3} \\
&= T_u \text{ for LRFD} \\
&= T_f \text{ for LSD} \\
\Omega &= 3.00 \text{ for ASD} \\
\phi &= 0.55 \text{ for LRFD} \\
&= 0.50 \text{ for LSD}
\end{align*}
\]

**D3.2 Sidelap Connections**

The *sidelap connection nominal shear strength [resistance]* per fastener, \( P_{ns} \), shall be determined in accordance with Section D1.2. It is permitted to not reduce \( P_{ns} \) for wind uplift.

**D4 Steel Deck Diaphragms With Structural Concrete or Insulating Concrete Fills**

The *available diaphragm shear strength [factored resistance]* per unit length with *insulating concrete* fill placed on *deck* or *form deck* on level or sloped roofs, or with *structural concrete* placed on *composite* or *form deck* in floor or roof *diaphragms*, shall be determined in accordance with Sections D4.1 through D4.4, as applicable, provided the following limitations are met:

(a) 0.5 in. (12.7 mm) \( \leq \) steel *deck depth* \( \leq \) 3 in. (76.2 mm),
(b) 0.014 in. (0.356 mm) \( \leq \) base steel *deck thickness* \( \leq \) 0.075 in. (1.91 mm),
(c) Fastener types include steel headed stud anchors, welds with or without washers, screws, and *power-actuated fasteners*,
(d) 33 ksi (230 MPa) \( \leq \) specified \( F_y \) of steel *deck* \( \leq \) 80 ksi (550 MPa),
\[
45 \text{ ksi (310 MPa)} \leq \text{specified } F_u \text{ of steel } \text{deck} \leq 82 \text{ ksi (565 MPa)},
\]
(e) *Structural concrete* fill has a minimum thickness of 2 in. (50.8 mm) over top of *form deck* and 2 in. (50.8 mm) over *composite deck*,
(f) The maximum design thickness of fill over the top of *deck* is 6 in. (152 mm),
(g) For lightweight *insulating concrete* without polystyrene inserts, the minimum thickness over the top of *form deck* is 2.5 in. (63.5 mm),
(h) *Structural concrete* has a specified compressive strength, \( f'_c \), not less than 2500 psi (17.2 MPa), and
(i) *Insulating concrete* aggregate conforms to ASTM C332.
D4.1 Safety Factors and Resistance Factors

The safety and resistance factors shall be applied for either structural concrete or insulating concrete-filled diaphragms to determine the available diaphragm strength [factored resistance].

For the limit state of diagonal tension cracking as computed using Section D4.2, the safety and resistance factors are as follows:

\[ \Omega = 2.00 \text{ for } ASD \]
\[ \phi = 0.80 \text{ for } LRFD \]
\[ = 0.75 \text{ for } LSD \]

For the limit state of fastener failure as computed using Section D4.3 and D4.4, where steel headed stud anchors are used for perimeter fasteners, the safety and resistance factors are as follows:

\[ \Omega = 3.00 \text{ for } ASD \]
\[ \phi = 0.55 \text{ for } LRFD \]
\[ = 0.50 \text{ for } LSD \]

For diaphragms where fasteners other than steel headed stud anchors are used on the perimeter, safety and resistance factors shall be determined in accordance with section B1.1.

D4.2 Structural Concrete-Filled Diaphragms

The nominal shear strength [resistance] per unit length of diaphragms with structural concrete fill shall be calculated using Eq. D4.2-1.

\[ S_n = k_c \lambda_{LW} b t_e \sqrt{f_c} \]  \hspace{1cm} (Eq. D4.2-1)

where

\[ S_n = \text{Nominal shear strength [resistance] per unit length of diaphragm system with structural concrete fill, kip/ft (kN/m)} \]
\[ k_c = \text{Factor for structural concrete strength} \]
\[ = 3.2/1000 \text{ For U.S. Customary units} \] \hspace{1cm} (Eq. D4.2-2a)
\[ = 0.266/1000 \text{ For SI units} \] \hspace{1cm} (Eq. D4.2-2b)
\[ \lambda_{LW} = \text{Factor for lightweight concrete} \]
\[ = 1.0 \text{ For normalweight concrete} \]
\[ = 0.75 \text{ For lightweight concrete} \]
\[ = 0.85 \text{ For sand-lightweight concrete} \]
\[ b = \text{Unit width of diaphragm with structural concrete fill, 12 in. for U.S. Customary units and 1000 mm for SI units} \]
\[ t_e = \text{Equivalent transformed concrete thickness, in. (mm)} \]
\[ = t_a + n_{sc} \frac{t_d}{s} \] \hspace{1cm} (Eq. D4.2-3)
\[ t_a = \text{Average thickness of structural concrete, calculated as the cross-sectional area of the structural concrete over one deck panel divided by the width of the deck panel, in. (mm)} \]
\[ n_{sc} = \text{Modular ratio of steel deck to concrete} \]
\[ = \frac{E}{E_c} \] \hspace{1cm} (Eq. D4.2-4)
\[ E = \text{Modulus of elasticity of steel} \]
E_c = Modulus of elasticity of concrete in accordance with ACI 318

Ec = Modulus of elasticity of concrete in accordance with ACI 318

D4.3 Lightweight Insulating Concrete-Filled Diaphragms

The nominal shear strength [resistance] per unit length of insulating concrete-filled diaphragms controlled by connections at interior panels or edge panels and fill shear strength shall be calculated using Eq. D4.3-1 or Eq. D4.3-2, as applicable. It is permitted to ignore the contribution of insulating concrete fill and to determine the nominal diaphragm shear strength [resistance] per unit length based on the deck alone and controlled by the smallest of Eqs. D1-1, D1-2 and D1-3.

(a) Insulating concrete without insulating board in fill:

\[
S_{ni} = \frac{\beta P_{nf}}{L} + 4 \frac{bd_c \sqrt{f_c'}}{3000} \quad \text{for U.S. Customary units} \quad (Eq. D4.3-1a)
\]

\[
S_{ni} = \frac{\beta P_{nf}}{L} + 1.11 (10)^{-4} bd_c \sqrt{f_c'} \quad \text{for SI units} \quad (Eq. D4.3-1b)
\]

where

\(d_c\) = Insulating concrete thickness above top of deck, in. (mm)

\(f_c'\) = Specified insulating concrete compressive strength, psi (MPa)

(b) Insulating concrete with insulating board in fill:

\[
S_{ni} = \frac{\beta P_{nf}}{L} + 0.064 \sqrt{f_c'} \quad \text{for U.S. Customary units} \quad (Eq. D4.3-2a)
\]

\[
S_{ni} = \frac{\beta P_{nf}}{L} + 11.2 \sqrt{f_c'} \quad \text{for SI units} \quad (Eq. D4.3-2b)
\]

where

\(f_c'\) = Specified insulating concrete compressive strength, psi (MPa)

Other parameters and required units are defined in Section D4.2.

Minimum insulating concrete thickness above insulating board shall be 2 in. (50.8 mm). Insulating board shall not be installed within 3 ft (0.915 m) of a lateral force-resisting system line if the insulating concrete fill contributes to the nominal diaphragm shear strength [resistance] per unit length.

D4.4 Perimeter Fasteners for Concrete-Filled Diaphragms

Where the contribution of structural or insulating concrete fill is included in diaphragm nominal shear strength [resistance] per unit length, \(S_n\) or \(S_{ni}\), the number of perimeter fasteners along the panel length, \(L_s\), to develop the full diaphragm shear strength [resistance] of the system shall be determined in accordance with Eq. D4.4-1, D4.4-2 or D4.4-3, as applicable.

On the perimeter edge parallel to an edge panel span:

For \(L_s \leq 5\) ft (1.52 m)
\[ n_e = \frac{S_n L}{P_{nfs}} \quad (Eq. \, D4.4-1) \]

For \( L_v > 5 \text{ ft (1.52 m)} \)
\[ n_e = \max \left( \frac{S_n L}{P_{nfs}}, \frac{L}{\alpha} \right) \quad (Eq. \, D4.4-2) \]

For the condition parallel to an edge panel span, fasteners shall be equally distributed along the total length that is connected to a lateral force-resisting system.

**User Note:**
Where a lateral force-resisting system line has sufficient stiffness at an edge panel to unload the concrete-filled diaphragm, and diaphragm shear per unit length flows from two sides, the required strength [effects due to factored loads] should be the reaction per unit length at that line.

On the perimeter edge perpendicular to an interior or edge panel span (i.e., along a longitudinal chord member):
\[ N = \frac{S_n}{P_{nf}} \quad (Eq. \, D4.4-3) \]

where
- \( L_v = \) Span of panel between supports with fasteners
- \( n_e = \) Number of edge support connections equally distributed along an edge panel length with concrete fill
- \( S_n = \) Nominal shear strength [resistance] per unit length of diaphragm system determined in accordance with Section D4.2 or D4.3, as applicable
  - \( = S_{ni} \) in Section D4.3
- \( L = \) Total length of panel
- \( P_{nf} = \) Nominal shear strength [resistance] of a support connection installed perpendicular to an interior or edge panel span with concrete fill and determined in accordance with Section D1.1 or Section D4.4.1, as applicable
- \( P_{nfs} = \) Nominal shear strength [resistance] of an edge support connection installed parallel with an edge panel span with concrete fill and determined in accordance with Section D1.1 or Section D4.4.1, as applicable
- \( N = \) Number of support connections per unit width at an interior or edge panel’s end
- \( \alpha = \) Conversion factor for units
  - \( = 3.0 \) for U.S. Customary units and \( L \) in (ft)
  - \( = 0.914 \) for SI units and \( L \) in (m)

Where the contribution of structural or insulating concrete fill is neglected, the nominal diaphragm shear strength [resistance] per unit length shall be determined in accordance with Section D1. The number of required perimeter fasteners, \( n_e \), shall conform to the spacing requirements of Section D1. It is permitted to use Eq. D4.4-1 or D4.4-2, as applicable, to determine \( n_e \).
User Note:
When fill is neglected to resist a given required strength [effect due to factored loads], the number of required edge support connections, \( n_e \), can be determined from Eq. D1-3, and the required support connections per unit width, \( N \), can be determined from Eq. D1-2 by setting the required strength [effect due to factored loads] equal to or less than the available strength [factored resistance]. The number of edge fasteners, \( n_e \), can include transverse support fasteners along the edge. \( P_{nf} \) may not equal \( P_{nfs} \) and \( n_e \) can be adjusted using Eq. D1-3 by setting \( \alpha_1 \) and \( \alpha_2 = 1 \).

When concrete fill is neglected, a steel headed stud anchor is equivalent to a large arc spot weld. AISI S100 Eq. J2.2.2.1-1 given in Section D1.1.1 can be neglected for a steel headed stud anchor. Required strength associated with Eq. D4.4-3 can vary along the diaphragm length, \( L_d \).

Where the full nominal diaphragm shear strength [resistance] per unit length of the system is not required, it is permitted to reduce the number of fasteners at an edge panel in accordance with Eq. D4.4-4. The maximum spacing limit shall apply for \( L_v > 5 \text{ ft (1.52 m)} \).

\[
\begin{align*}
    n_e &= \frac{\Omega_d R_L}{P_{nfs}} \quad \text{for ASD} \quad (Eq. \ D4.4-4a) \\
    n_e &= \frac{R_u L}{\phi_d P_{nfs}} \quad \text{for LRFD} \quad (Eq. \ D4.4-4b) \\
    n_e &= \frac{R_f L}{\phi_d P_{nfs}} \quad \text{for LSD} \quad (Eq. \ D4.4-4c)
\end{align*}
\]

Other parameters are defined in Section C2 and the safety and resistance factors are defined in Section D4.1 for perimeter fasteners consisting of steel headed stud anchors. If the diaphragm uses welds with or without washers, power-actuated fasteners or screws for perimeter connections, the safety and resistance factors in Section B1 shall be used.

**D4.4.1 Steel Headed Stud Anchors**

Welded steel headed stud anchors are permitted in structural concrete at edge panels or perimeters:
(a) To resist the required shear strength [shear force due to factored loads], and
(b) To replace other steel support fasteners with connection strength determined in accordance with Section D1.1.

Steel headed stud anchors shall conform to ANSI/AISC 360 material requirements and shall be welded in accordance with ANSI/AWS D1.1. ANSI/AISC 360 shall be followed to determine steel headed stud anchor available shear strength [factored resistance], maximum and minimum spacing, and edge dimension requirements.

Steel headed stud anchors are not permitted in lightweight insulating concrete-filled diaphragms as described in Section D4.3.

User Note:
Proprietary or other mechanical shear connectors are permitted provided the shear connector devices are qualified under the alternative method provisions of the applicable building codes or the connection strength is determined using Section D1.1.5.
D5 Diaphragm Stiffness

D5.1 Stiffness of Fluted Panels

D5.1.1 Fluted Panels Without Perforated Elements

For a diaphragm or wall diaphragm system with fluted deck or panels, the diaphragm stiffness, \( G' \), shall be calculated in accordance with Eq. D5.1.1-1:

\[
G' = \left( \frac{E_t}{2(1+\mu)} \left( \frac{s}{d} \right) + \gamma_c D_n + C \right) K \text{ kip/in. (kN/m)} \quad (Eq. D5.1.1-1)
\]

where

- \( E = \) Modulus of elasticity of steel  
  \( = 29,500 \text{ ksi, (203,000 MPa)} \)
- \( t = \) Base steel thickness of panel, in. (mm)
- \( K = \) Stiffness factor relating support and sidelap connection flexibilities  
  \( = 1 \) for steel panels with lap-down on steel supports  
  \( = \frac{S_f}{S_s} \) for steel panels with lap-up on steel supports  
  \( = 0.5 \) for steel panels on wood supports
- \( S_f = \) Structural support connection flexibility determined in accordance with Section D5.2, in./kip (mm/kN)
- \( S_s = \) Sidelap connection flexibility determined in accordance with Section D5.2, in./kip (mm/kN)

User Note:

Figure D1.3-2 shows lap-down with insulation but \( K = 1 \) with or without insulation. Figures D1.3-3 and D1.3-4 show lap-up with insulation but \( K = \frac{S_f}{S_s} \) on steel supports with or without insulation.

Ratio, \( S_f/S_s \), equals 0.433 for screws into steel supports and equals 0.5 for screws through bottom flats in wood supports. This can be confirmed using Sections D5.2.2 and D5.2.3. \( S_f/S_s \) equals 1 for screws through top flats and into wood supports but \( K = 0.5 \) is used for wood supports with a lap-down or lap-up condition.

- \( \mu = \) Poisson’s ratio for steel
  \( = 0.3 \)
- \( d = \) Panel corrugation pitch. See Figure D2-1
- \( s = \) Developed flute width per pitch. Defined in Section D2
- \( D_n = \) Warping factor considering distortion at panel ends determined in accordance with Appendix 1
- \( \gamma_c = \) Support factor for warping determined in accordance with Appendix 1, Table 1.3-1
- \( C = \) Slip constant considering slippage at sidelap connections and distortion at support connections

\[
C = \left( \frac{E_t}{w} \left( \frac{2L}{2\alpha_3 + n_p\alpha_4 + 2n_s \frac{S_f}{S_s}} \right) S_f \right) \quad (Eq. D5.1.1-2)
\]
where

\[
L = \text{Total panel length, in. (m)}
\]

\[
\alpha_3 = \text{Measure of exterior support fastener group distribution across a panel width, w, at an interior panel}
\]

\[
\alpha_3 = \sum \frac{x_e}{w}
\]  \hspace{1cm} (Eq. D5.1.1-3)

\[
x_e = \text{Distance from panel centerline to an exterior support structural connection in an interior panel}
\]

\[
w = \text{Panel cover width at the interior panel. See Figure D1-1}
\]

\[
\alpha_4 = \text{Measure of interior support fastener group distribution across a panel width, w, at an interior panel}
\]

\[
\alpha_4 = \sum \frac{x_p}{w}
\]  \hspace{1cm} (Eq. D5.1.1-4)

\[
x_p = \text{Distance from panel centerline to an interior support structural connection in an interior panel}
\]

\[
n_p, n_s = \text{Factors defined in Section D1}
\]

**User Note:**

Each of the three components in the denominator of Eq. D5.1.1-1 is unit-less. Where the prescribed units and the value of E that are listed in this section are used in Eq D5.1.1-2, C is unit-less. The SI units at Eq. D5.1.1-1 are correct using the prescribed units for the parameters, but a more common unit is kN/mm which can be obtained by dividing the calculated G’ by 1000.

S_s and n_s are permitted to be included in Eq. D5.1.1-2 whether the connection shear strength [resistance], P_{ns}, contribution of a sidelap connection is included or neglected in the determination of diaphragm nominal shear strength [resistance] per unit length, S_n.

Stiffness, G’, is permitted to be determined by tests in accordance with AISI S907.

Stiffness, G’, shall not be reduced due to shear and tension interaction caused by wind uplift.

### D5.1.2 Fluted Acoustic Panels With Perforated Elements

For diaphragm or wall diaphragm with fluted acoustic panels, the diaphragm stiffness, G’, shall be calculated in accordance with Eq. D5.1.1-1 modified for the perforation effect as follows:

(a) D_n is determined in accordance with Appendix 1 Section 1.6.

(b) C is determined using Eq. D5.1.1-2 with support connection flexibility, S_t, and sidelap connection flexibility, S_{so}, determined as follows:

1. In accordance with Sections D5.2.1 through D5.2.5, as applicable, for fasteners located in nonperforated zones of an element; or
2. In accordance with Section D5.2.6 for fasteners located in perforated zones of an element.

(c) s, the developed flute width per pitch modified for perforation, is determined using Eq. D5.1.2-1.
\[ s = 2e + 2w + f + \left( E_p + 2W_p + F_p \frac{1}{k} - 1 \right) \]  
\[ (Eq. \text{ D5.1.2-1}) \]

where

\[ E_p = \text{Width of perforation band in the bottom flat of width}, 2e, \text{ in. (mm)} \]

\[ W_p = \text{Width of perforation band in the web flat of width}, w, \text{ in. (mm)} \]

\[ F_p = \text{Width of perforation band in the top flat of width}, f, \text{ in. (mm)} \]

\[ k = \text{Ratio of perforated element stiffness to that of a solid element of the same thickness, } t, \text{ determined in accordance with Appendix 1, Eq. 1.6-5} \]

Other parameters are defined in Section D2.

### D5.2 Connection Flexibility

The structural support connection flexibility, \( S_f \), and sidelap connection flexibility, \( S_s \), shall be determined in accordance with Sections D5.2.1 through D5.2.5 or by tests in accordance with Section D5.2.6. It is permitted to determine connection flexibility by tests for connections listed in Sections D5.2.1 through D5.2.5.

The connection flexibility shall not be adjusted for an interaction effect due to the presence of wind uplift.

#### D5.2.1 Welds Into Steel

##### D5.2.1.1 Arc Spot or Arc Seam Welds

The connection flexibilities of arc spot or arc seam welds shall be determined in accordance with Eq. D5.2.1.1-1 and Eq. D5.2.1.1-2:

\[ S_f = \frac{1.15\alpha}{1000\sqrt{t}} \]  
\[ (Eq. \text{ D5.2.1.1-1}) \]

\[ S_s = \frac{1.25\alpha}{1000\sqrt{t}} \]  
\[ (Eq. \text{ D5.2.1.1-2}) \]

where

\[ S_f = \text{Structural support connection flexibility of arc spot or arc seam welds, in./kip (mm/kN)} \]

\[ S_s = \text{Sidelap connection flexibility of arc spot or arc seam welds, in./kip (mm/kN)} \]

\[ \alpha = \text{Conversion factor for units} \]

\[ = 1 \text{ for U.S. Customary units} \]

\[ = 28.8 \text{ for SI units} \]

\[ t = \text{Total combined base steel thickness of panel involved in shear transfer above the shear transfer plane, in. (mm)} \]

##### D5.2.1.2 Top Arc Seam Sidelap Welds

The sidelap connection flexibility, \( S_s \), of top arc seam sidelap welds formed between two sheets shall be determined in accordance with Eq. D5.2.1.2-1 for steel conforming to AISI S100 Section A3.

\[ S_s = \frac{1.12\alpha}{1000\sqrt{t}} \left( \frac{L_w}{\alpha_5} \right)^{0.25} \]  
\[ (Eq. \text{ D5.2.1.2-1}) \]
where

\[ \alpha_5 = \text{Conversion factor for units} \]
\[ = 1.5 \text{ for U.S. customary units} \]
\[ = 38 \text{ for SI units} \]
\[ L_w = \text{Length of top arc seam sidelap weld, in. (mm). See Figure D1.2.4-1 for details} \]
\[ \alpha \text{ and } t \text{ are defined in Section D5.2.1.1.} \]

**D5.2.2 Screws Into Steel**

The connection flexibility of screws into steel shall be determined in accordance with Eq. D5.2.2-1 and Eq. D5.2.2-2.

\[ S_f = \frac{1.3\alpha}{1000\sqrt{t}} \quad (Eq. \text{ D5.2.2-1}) \]
\[ S_s = \frac{3.0\alpha}{1000\sqrt{t}} \quad (Eq. \text{ D5.2.2-2}) \]

where

\[ S_f = \text{Structural support connection flexibility of screws, in./kip (mm/kN)} \]
\[ S_s = \text{Sidelap connection flexibility of screws, in./kip (mm/kN)} \]
\[ \alpha \text{ and } t \text{ are defined in Section D5.2.1.1.} \]

Eq. D5.2.2-1 shall be limited to screw size # 12 (nominal diameter = 0.216 in. (5.49 mm)) or #14 (nominal diameter = 0.25 in. (6.35 mm)).

The structural support connection flexibility, \( S_f \), shall be determined in accordance with Eq. D5.2.2-2 for screws through top flat and into supports and with or without insulation beneath the panel. The connection with insulation is illustrated in Figure D1.3-4.

**User Note:**

\( S_f \) at supports (Eq. D5.2.2-1) requires that the support is relatively thick and that bearing of the panel against the support screw controls connection strength.

#10 screws are commonly used at sidelaps and #8 screws can be, but are rarely used.

**D5.2.3 Wood Screws or Nails Into Wood Supports**

The connection flexibility, \( S_f \), of wood screws or nails fastened into wood supports with or without insulation beneath the panel shall be determined in accordance with Eq. D5.2.3-1 and Eq. D5.2.3-2, as applicable:

(a) For wood screws or nails fastened through bottom flat and into wood support, as illustrated in Figures D1.3-1 and D1.3-2,

\[ S_f = \frac{1.5\alpha}{1000\sqrt{t}} \quad (Eq. \text{ D5.2.3-1}) \]

(b) For wood screws or nails fastened through top flat and into wood support, as illustrated in Figures D1.1.4.3-1 and D1.3-4,

\[ S_f = \frac{3.0\alpha}{1000\sqrt{t}} \quad (Eq. \text{ D5.2.3-2}) \]

where

\[ S_f = \text{Structural support connection flexibility of fastener into wood supports, in./kip} \]
**D5.2.4 Power-Actuated Fasteners Into Supports**

Power-actuated fastener connection flexibilities shall be determined in accordance with Section D5.2.6.

**D5.2.5 Non-Piercing Button Punch Fasteners at Steel Panel Sidelaps**

The sidelap connection flexibility for non-piercing button punch fasteners in panels shall be determined in accordance with Eq. D5.2.5-1.

\[
S_s = \frac{30.0\alpha}{1000\sqrt{t}} \quad (Eq. \ D5.2.5-1)
\]

where

\( S_s \) = Sidelap connection flexibility of non-piercing button punch, in./kip (mm/kN)

\( \alpha \) and \( t \) are defined in Section D5.2.1.1.

**D5.2.6 Other Fasteners – Flexibility Determined by Tests**

Connection flexibilities that are not included in Sections D5.2.1 through D5.2.5 shall be determined by tests in accordance with Sections E1.1 and E1.2. For fasteners located in perforated zones of an element, the test specimen shall contain the perforation pattern.

**User Note:**

Proprietary crimped or mechanically formed sidelap connections are common and acceptable, and their connection flexibilities are determined in accordance with this section.

**D5.3 Stiffness of Cellular Deck**

**D5.3.1 Cellular Deck Without Perforations**

\( G' \) shall be calculated in accordance with Eq. D5.3.1-1:

\[
G' = \frac{Et}{A_a + C} \quad (Eq. \ D5.3.1-1)
\]

where

\( G' \) = Diaphragm stiffness of cellular deck without perforations, kip/in. (kN/m)

\( A_a \) = Material shear deformation component for cellular deck

\[
A_a = \frac{2.6 \frac{S}{d}}{1 + \frac{S}{w_d} \frac{t_b}{t}} \quad (Eq. \ D5.3.1-2)
\]
where
\( s \) = Developed flute width of top deck in \textit{cellular deck} in accordance with Eq. D2-5 in which the variables are defined as follows, in. (mm):
\( e \) = Distance from the cell top deck longitudinal fastener to the \textit{web}, in. (mm)
\( f \) and \( w \) are as defined in Section D2, in. (mm).
\( d \) = Panel \textit{corrugation pitch} of top fluted \textit{deck} in \textit{cellular deck}, in. (mm)
\( \text{wd} \) = Distance measured across the width and between longitudinal rows of fasteners connecting the top \textit{deck} to the bottom plate, in. (mm)

\( \text{wd} = d \) where top \textit{deck} to bottom plate fasteners are at the flute centerlines, in. (mm)

\textbf{User Note:}
The top \textit{deck} is attached to the bottom plate by fasteners along the panel's length. The base dimension for \textit{cellular deck} is \text{wd}, as shown in Figure D5.3.1-1, which can be less than \textit{pitch}, \text{d}. \textit{Pitch} \text{d} is used in the numerator of Eq. D5.3.1-2.

\( t_b \) = Base steel \textit{thickness} of bottom plate in \textit{cellular deck}, in. (mm)
\( t \) = Base steel \textit{thickness} of top \textit{deck} in \textit{cellular deck}, in. (mm)
\( C \) = Slip constant considering slippage at \textit{sidelap connections} and distortion at \textit{support connections}; defined by Eq. D5.1.1-2, in which:
(a) Structural \textit{support connection flexibility}, \text{Sf}, is based on the total thickness of elements above the shear transfer plane, in./kip (mm/kN),
(b) \textit{Sidelap connection flexibility}, \text{Ss}, is based on the thinner element containing the fastener, in./kip (mm/kN), and
(c) \( t \) is the top \textit{deck thickness}, in. (mm)

Other parameters are as defined in Section D5.1.1. See Figures D2-1 and D5.3.1-1 for details.

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure_d5311.png}
\caption{Cellular Deck Types}
\end{figure}
D5.3.2 Cellular Deck With Perforations

Diaphragm stiffness, $G'$, of cellular deck with perforations shall be calculated in accordance with Eq. D5.3.1-1 with $A_a$ determined in accordance with Eq. D5.3.2-1 and $C$ defined in this section:

$$A_a = \frac{2.6 \frac{s'}{d}}{1 + \frac{s'}{d'} \frac{t_b}{t}}$$  \hspace{1cm} (Eq. D5.3.2-1)

where

$A_a$ = Material shear deformation component for cellular deck with perforations

$d'$ = Equivalent width of cellular acoustic deck bottom plate adjusted for perforations and measured between longitudinal rows of fasteners connecting the top deck to the bottom plate, in. (mm)

$$w_{dp} = \frac{1}{k_b} - 1$$  \hspace{1cm} (Eq. D5.3.2-2)

$k_b$ = Ratio of shear stiffness of perforated zone in the bottom plate of cellular acoustic deck to a solid zone of the same thickness, $t_b$, and determined in accordance with Appendix 1 Eq. 1.6-5

$s'$ = Developed flute width of top deck per width, $w_d$, in cellular deck in accordance with Eq. D5.1.2-1 and modified as follows if perforations are present in the top deck:

- $e$ = Distance from cell top deck longitudinal fastener to web, in. (mm)
- $E_p$ = Width of perforation band in the bottom flat of width, $2e$, in. (mm)
- $W_p$ = Width of perforation band in the web flat of width, $w$, in. (mm)
- $F_p$ = Width of perforation band in the top flat of width, $f$, in. (mm)
- $k$ = Ratio of perforated element stiffness to that of a solid element of the same thickness, $t$, determined in accordance with Appendix 1, Eq. 1.6-5

User Note:
By the above definitions, $E_p/2$ is the width of the perforation band in the width, $e$. $s'$ is $s$ in accordance with Section D5.3.1 when perforations are not present in the top deck.

$C$ = Slip constant considering slippage at sidelap connections and distortion at support connections determined in accordance with Eq. D5.1.1-2. $S_t$ and $S_s$ are determined in accordance with Section D5.2.6 for fasteners located in perforated zones of an element.

D5.4 Stiffness of Concrete-Filled Diaphragms

D5.4.1 Stiffness of Structural Concrete-Filled Diaphragms

The diaphragm stiffness, $G'$, shall be calculated in accordance with Eq. D5.4.1-1 or Eq. D5.4.1-2 for diaphragms with structural concrete fill over fluted deck or cellular deck and that satisfy the limits of applicability given in Section D4:

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\[
G' = \frac{Et}{2(1 + \mu) \frac{s}{d} + C} + K_3 \quad \text{for fluted deck} \tag{Eq. D5.4.1-1}
\]

\[
G' = \frac{Et}{A_a + C} + K_3 \quad \text{for cellular deck} \tag{Eq. D5.4.1-2}
\]

where

\[G' = \text{Diaphragm stiffness, kip/in. (kN/m)}\]

\[E = \text{Modulus of elasticity of steel, ksi (MPa)}\]

\[t = \text{Base steel thickness of fluted deck, or} \]

\[t = \text{Base steel thickness of top deck in cellular deck, in. (mm)}\]

\[K_3 = \text{Stiffness contribution of the structural concrete fill} \]

\[= 3.5d_c(f'_c)^{0.7}, \text{kip/in. for U.S. Customary units} \quad \text{(Eq. D5.4.1-3a)}\]

\[= 786d_c(f'_c)^{0.7}, \text{kN/m for SI units} \quad \text{(Eq. D5.4.1-3b)}\]

\[d_c = \text{Structural concrete thickness above top of deck, in. (mm)}\]

\[f'_c = \text{Specified structural concrete compressive strength, psi (MPa)}\]

\[A_a = \text{Material shear deformation component for cellular deck determined in accordance with Section D5.3.1} \]

\[A_a = \text{Material shear deformation component for cellular deck with perforations determined in accordance with Section D5.3.2} \]

Other parameters are defined in Sections D5.1 and D5.3, as applicable.

**User Note:**

Structural concrete is rarely used over perforated deck, but it may be used with perforated bottom plates in cellular acoustic deck.

### D5.4.2 Stiffness of Insulating Concrete-Filled Diaphragms

Diaphragm stiffness, \(G'\), shall be calculated in accordance with Eq. D5.4.2-1 or Eq. D5.4.2-2 for insulating concrete-filled diaphragms that are installed over fluted deck or cellular deck and that satisfy the limits of applicability given in Section D4:

\[
G' = \frac{Et}{2(1 + \mu) \frac{s}{d} + C} + K_3 \quad \text{for fluted deck} \tag{Eq. D5.4.2-1}
\]

\[
G' = \frac{Et}{A_a + C} + K_3 \quad \text{for cellular deck} \tag{Eq. D5.4.2-2}
\]

where

\[K_3 = \text{Stiffness contribution of the insulating concrete fill determined using Eq. D5.4.1-3:} \]

\[d_c = \text{Specified insulating concrete thickness above top of deck, in. (mm)}\]

\[f'_c = \text{Insulating concrete compressive strength, psi (MPa)}\]

Other parameters are defined in Section D5.4.1.
D6 Diaphragm Flexibility

The flexibility, \( F \), of the diaphragm system shall be calculated in accordance with Eq. D6-1 or determined by test in accordance with Chapter E.

\[
F = \frac{1}{G'} \tag{Eq. D6-1}
\]

Flexibility, \( F \), shall not be increased due to shear and tension interaction at connections.
E. DIAPHRAGM NOMINAL SHEAR STRENGTH PER UNIT LENGTH AND STIFFNESS DETERMINED BY TEST

The nominal diaphragm shear strength [resistance] per unit length, and the diaphragm stiffness or flexibility are permitted to be determined by tests in accordance with this chapter. Section E1 shall be applicable to any diaphragm system and Section E2 shall be applicable to a single diaphragm system.

E1 Strength and Stiffness of a Prototype Diaphragm System

Large-scale tests for a prototype diaphragm system and small-scale tests for a fastener or connection shall be performed at an independent testing laboratory or at a testing laboratory of a manufacturer.

User Note:
The requirements in this Standard are consistent with AISI S100 Section K2. Section 1703 of the 2018 Edition of the International Building Code requires that testing quality control, data, and test results must also be in conformance with the requirements of the local building official or approval agency. The International Building Code is published by International Code Council, Inc., 500 New Jersey Avenue, NW, Washington DC 20001.

E1.1 Test Protocol

Large-scale tests of a diaphragm system shall be performed in accordance with AISI S907. Small-scale tests for determining connection nominal shear strength [resistance], connection flexibility, connection nominal tensile strength [resistance], and shear and tension interaction of connections shall be performed in accordance with AISI S905. Screw shear and tensile breaking strength shall be determined in accordance with AISI S904.

Testing in accordance with AISI S907 and AISI S905 is permitted for diaphragm systems and connections that are connected to non-steel supports. In lieu of AISI S905, the following test methods are permitted to determine the diaphragm connection strength [resistance] of connections into non-steel supports:
(a) ASTM D1761 for wood supports, or
(b) ASTM E1190 or E488 for structural concrete supports.

Wood supports shall be seasoned and dry structural members.

E1.2 Design Using Test-Based Analytical Equations

This section shall be used to develop, modify, or verify test-based analytical equations and shall apply to each of the following five testing objectives:
(1) To determine the following nominal strengths [resistance] and flexibilities of connections in a diaphragm system that conforms to Chapter D:
   (i) Support connection nominal shear strength [resistance] per fastener, P_{nf} or P_{nfs}, that is not listed in Sections D1.1.1 through D1.1.4,
   (ii) Sidelap connection nominal shear strength [resistance] per fastener, P_{ns}, that is not listed in Sections D1.2.1 through D1.2.6, or
   (iii) Connection flexibility, S_f or S_s, per fastener that is not listed in Sections D5.2.1 through
D5.2.5;

(2) To refine the nominal connection strength [resistance] or flexibility that is listed in Chapter D;

(3) To establish analytical equations for fluted steel panels or cellular decks, or structural or insulating concrete-filled decks that are not within the limits listed in Chapter D for a diaphragm system that conforms to Chapter D otherwise;

(4) To establish analytical equations for strength and stiffness of diaphragm systems or components based on an existing test-based analytical model other than that of Chapter D; and

(5) To establish the contribution of an accessory or detail.

Application limits and safety and resistance factors shall be determined in accordance with the test constructions and results. Extrapolation beyond the established limits is not permitted. The established limits and safety and resistance factors of an existing diaphragm system analytical model shall apply in design as long as the theoretical nominal strength [resistance], Snth theory, from additional tests is based on the existing analytical model. Modifications to application limits and revisions to safety and resistance factors are permitted if enough tests are performed in accordance with the testing objective.

A basic diaphragm system does not include accessories or stiffening details. Accessories or stiffening details that increase strength or stiffness relative to the basic system shall be included in the test if increased strength or stiffness is the testing objective for design application. However, it is permitted to not include the tested accessories or details in design application regions where the basic system’s nominal strength [resistance] as determined by calculation or other tests provides the required strength or stiffness.

The analytical equation calibration shall be based on the base steel thickness, mechanical properties, fastener properties and fill material properties that are tested. The specified minimum material properties and steel minimum thickness shall be used in design applications of the developed analytical equations and shall be further modified as required by AISI S100 Section A3.1.

**User Note:**
Application limits of analytical systems commonly include material thickness and mechanical properties, connection types, profile types and dimensions, and rated accessories. See the Commentary for a discussion of existing analytical systems.

**E1.2.1 Test Assembly Requirements**

Small-scale tests to determine connection strength [resistance] or flexibility are permitted without additional large-scale diaphragm tests if the panel profile conforms to the limits (a) through (d) of Chapter D, and Sections D1 or D5 are used to establish diaphragm shear strength [resistance] per unit length or stiffness, respectively. Otherwise, large-scale tests shall be performed. Small-scale tests are permitted in conjunction with large-scale tests.

The essential test parameters for a given testing objective shall be as listed in Table E1.2-1. The number of required tests shall conform to the applicable test standards listed in Section E1.1.
Table E1.2-1
Essential Test Parameters

<table>
<thead>
<tr>
<th>Testing Objective</th>
<th>Essential Test Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck or panel profile</td>
<td>$t_{\text{deck}}, F_y \text{ deck}, F_u \text{ deck}$, profile geometry</td>
</tr>
<tr>
<td>Support fastener</td>
<td>$t_{\text{deck}}, F_u \text{ deck}, t_{\text{support}}, F_u \text{ support}$, fastener dimensions and mechanical properties</td>
</tr>
<tr>
<td>Sidelap fastener</td>
<td>$t_{\text{deck}}, F_u \text{ deck}$, fastener dimensions and mechanical properties</td>
</tr>
</tbody>
</table>

Note:

- $t_{\text{deck}}$ = Thickness of deck or panel
- $t_{\text{support}}$ = Thickness of support
- $F_y \text{ deck}$ = Yield stress of deck or panel
- $F_u \text{ deck}$ = Tensile strength of deck or panel
- $F_u \text{ support}$ = Tensile strength of support

Tests shall be required to establish the contribution of parameters that are not within the limits of a diaphragm analytical model while the model is used to establish nominal shear strength [resistance] per unit length, $S_n$, or stiffness, $G'$. It is permitted to use established nominal connection strength [resistance], $P_{nf}, P_{nfs}$, or $P_{ns}$, and connection flexibility, $S_f$ or $S_s$, from alternative analytical models in lieu of small-scale tests, provided the performance of the fasteners in the diaphragm system is confirmed by large-scale tests over the application range. It is permitted to use the nominal shear strength [resistance] per unit length due to out-of-plane buckling, $S_{nb}$, from an alternative analytical model within its acceptable limits, provided the alternative model’s safety or resistance factor for out-of-plane buckling, $\Omega_{db}$ or $\phi_{db}$, is used to determine available strength [factored resistance]. All safety or resistance factors shall be determined in accordance with Table B1 for tested assemblies.

Small-scale and large-scale tests shall include *end-laps* if *end-laps* are required by the testing objective.

The tested connection type, size, and spacing shall be that specified for the test.

In large-scale tests, arc spot welds shall be measured, and the tested parameters shall conform to and be applied in accordance with the following:

(a) Visible diameters of the outer surface of arc spot welds at panel supports are measured at all panel sidelaps and at the adjacent interior flutes, if applicable. It is permitted to measure all transverse support welds.

(b) Fused perimeters of all sidelap welds are measured.

(c) The average measured visible diameter of arc spot welds at supports, $d_{\text{test}}$, is used to calculate $P_{nf}$ and $S_{ni \text{ theory}}$, provided 90 percent of all measured welds at supports is within 25 percent of $d_{\text{test}}$. The visible diameter at each measured support weld is the average of two orthogonal measurements with one being the largest visible diameter at the weld.

where

- $P_{nf} = \text{Nominal weld shear strength [resistance] of a support connection}$ that is used to calculate $S_{ni \text{ theory}}$.
- $S_{ni \text{ theory}} = \text{Calculated diaphragm shear strength [resistance]}$ per unit length for Test i
(d) The average visible diameter of arc spot welds, $d_{s \text{ test}}$, at *sidelap connections* is used to calculate $P_{ns}$ and $S_{ni \text{ theory}}$, provided 90 percent of all measured diameters is within 25 percent of $d_{s \text{ test}}$.

where

$$P_{ns} = \text{Nominal weld shear strength [resistance] of a sidelap connection that is used to calculate } S_{ni \text{ theory}}.$$ 

The visible diameter at each *sidelap* weld is calculated as follows:

1. For continuous perimeter fusion at the *sidelap* weld, use the average of two orthogonal measurements with one being the largest visible diameter at the weld.
2. For discontinuous perimeter fusion at the *sidelap* weld, use the relationship:

$$\pi = \frac{\text{Perimeter Fused Measured Diameter}}{\text{Visible Measured Diameter}}$$  \hspace{1cm} (Eq. E1.2.1-1)

In large-scale tests, *sidelap* welds, e.g. fillet, groove, or *top arc seam sidelap welds*, shall be measured and the parameters shall conform to and be applied in accordance with the following:

(a) Fused lengths, $L_{w}$, at all *sidelap* welds are measured. For discontinuous fusion at a *sidelap* weld, $L_{w}$ is the total of fully fused zones. See Figure D1.2.4-1 for a *top arc seam sidelap weld*.

(b) The average fused length, $L_{w \text{ test}}$, of *sidelap welds* is the average of all measured fused lengths per weld. The average fused length is used to calculate $P_{ns}$ and $S_{ni \text{ theory}}$, provided 90 percent of all measured fused lengths is within 25 percent of the average fused length.

**User Note:**

Separate analytical equations for *connection strength* and *flexibility* can be developed at butt *joint* (no end-lap) and end-lap conditions in panels at *exterior supports*. In this case, the smaller value of support *connection strength [resistance]*, $P_{nf}$, and the greater support *connection flexibility*, $S_{f}$, can be used in design to calculate *nominal shear strength [resistance] per unit length* and *stiffness* for the *diaphragm system* when either butt *joints* or end-laps exist. If separate equations are not developed, industry practice often applies test results based on end-laps or combinations of end-laps and butt *joints* to design applications with butt *joints*. The converse is also true. However, some manufacturers use the potential benefit of end-lap end restraint from tests to increase *diaphragm system stiffness*.

The testing objectives are uniformity of welds and inclusion of the desired parameter range in the tested *configuration*. Welds will not match exactly the specified size. Some oversize welds may occur at touch-ups, which should not disallow a test. Weld prequalification procedures are recommended to control weld sizes.

**E1.2.2 Test Calibration**

Calibration shall be performed for small-scale and large-scale tests as described in Section E1.1. The *safety factor*, $\Omega$, and *resistance factor*, $\phi$, shall be determined in accordance with AISI S100 Eqs. K2.1.2-2 and K2.1.1-2, respectively. The calibrated *safety* and *resistance factors* shall be limited by those determined in accordance with Table B1.1-1. *Safety* and *resistance factors* shall be determined for the following test cases (a) through (c):
(a) Where certain *connections* are tested in accordance with Section D1.1.5, D1.2.7 or D3.1 and *nominal diaphragm strength [resistance]*, $S_{nt}$, and *stiffness*, $G'$, are determined analytically in accordance with Chapter D, the *connection safety factor*, $\Omega$, and *resistance factor*, $\phi$, based on small-scale tests, shall be determined in accordance with Section E1.2.2(b), provided the calibrated *safety* and *resistance factors* also conform to the following:

1. The *connection* and fastener *safety factor* shall be less than or equal to the *diaphragm system safety factor* required in Table B-1, and
2. The *connection* and fastener *resistance factor* shall be greater than or equal to the *diaphragm system resistance factor* required in Table B-1.

In addition to limits (a) to (d) of Chapter D, the *diaphragm system application* shall also conform to the limits of the *connection tests*.

(b) Small-scale tests to determine an analytical equation for *connection* strength, or large-scale tests to either determine an analytical equation for *connection* strength in an existing *diaphragm system model* or to verify a model's *nominal diaphragm shear strength [resistance]* per unit length shall conform to AISI S100 Section K2.1.1(b). Large-scale tests shall meet the additional requirements in Section E1.2.2(c). The calibration for small-scale and large-scale tests shall be in accordance with AISI S100 Section K2.1.1(c) as modified below:

\[
C_\phi = \text{Calibration coefficient} = \begin{cases} 
1.6 & \text{for LRFD} \\
1.5 & \text{for LSD} 
\end{cases}
\]

\[
P_m = \text{Mean value of professional factor, P, for tested component} \\
= \frac{\sum_{i=1}^{n} R_{Li}}{n}
\]

(AISI S100 Eq. K2.1.1-3)

where

- $i = \text{Index of tests} = 1$ to $n$
- $n = \text{Total number of tests}$
- $R_{Li} = \text{Tested connection strength [resistance] of Test i, or}$
  - $= \text{Tested nominal diaphragm shear strength [resistance] per unit length, } S_{ni\text{ test,} i}$ of Test i
- $R_{n,i} = \text{Calculated connection strength [resistance] of Test i per rational engineering analysis model, or}$
  - $= \text{Calculated nominal diaphragm shear strength [resistance] per unit length, } S_{ni\text{ theory,} i}$ of Test i per diaphragm system model

\[
V_Q = \text{Coefficient of variation of load effect} = \begin{cases} 
0.25 & \text{for LRFD and LSD} 
\end{cases}
\]

\[
V_P = \text{Coefficient of variation of test results determined in accordance with AISI S100 Eq. K2.1.1-6, but not less than 0.065}
\]

\[
C_P = \text{Correction factor, determined in accordance with AISI S100 Eq. K2.1.1-4}
\]

\[
\beta_o = \text{Target reliability index, determined in accordance with Table E1.2.2-1}
\]
F_m = Mean value of fabrication factor, F, determined in accordance with Table E1.2.2-1
M_m = Mean value of material factor, M, determined in accordance with Table E1.2.2-1
V_F = Coefficient of variation of fabrication factor determined in accordance with Table E1.2.2-1
V_M = Coefficient of variation of material factor determined in accordance with Table E1.2.2-1

<table>
<thead>
<tr>
<th>Diaphragm Conditions</th>
<th>β_o^2,3</th>
<th>F_m</th>
<th>M_m</th>
<th>V_F</th>
<th>V_M</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel support</td>
<td>3.5 for LRFD 4.0 for LSD</td>
<td>AISI S100 Section K2.1.1(b)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Structural concrete support or fill</td>
<td>3.5 for LRFD 4.0 for LSD</td>
<td>0.90</td>
<td>1.10</td>
<td>0.10</td>
<td>0.10</td>
</tr>
<tr>
<td>Insulating concrete fill</td>
<td>3.5 for LRFD 4.0 for LSD</td>
<td>AISI S100 Section K2.1.1(b)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wood support</td>
<td>4.0 for LRFD 4.5 for LSD</td>
<td>1.0</td>
<td>1.10</td>
<td>0.15</td>
<td>0.15</td>
</tr>
</tbody>
</table>

Note:
1. The most severe factors shall be used where fastener type or support varies in the diaphragm.
2. β_o = 2.5 is permitted in LRFD and by extension in ASD for wind load on diaphragms with steel supports and without structural or insulating concrete fill provided the limits of Table B1.1-1 are met.
3. β_o = 3.5 for all load effects in LRFD and by extension in ASD, and 4.0 for all load effects in LSD are permitted with wood supports provided bearing of the panel against the fastener controls the connection shear strength and the bearing strength controlled by wood is at least 25% greater than the steel bearing strength.

<table>
<thead>
<tr>
<th>Diaphragm Conditions</th>
<th>Sections for Additional Requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel support</td>
<td>Section B1</td>
</tr>
<tr>
<td>Structural concrete fill</td>
<td>ACI-318 for solid slab subjected to same load effect</td>
</tr>
<tr>
<td>Structural concrete support</td>
<td>Section D4.1</td>
</tr>
<tr>
<td>Insulating concrete fill</td>
<td>Section D4.1</td>
</tr>
<tr>
<td>Wood support</td>
<td>Section D1.1.4.1</td>
</tr>
</tbody>
</table>

Note:
1. Where diaphragm conditions are mixed, the most severe requirement applies.
User Note:
Although the contribution of structural concrete fill typically dominates diaphragm strength and stiffness, the safety and resistance factors are limited to the more severe case of strength controlled by the deck connection, or strength controlled by structural concrete. Where limited tests are performed and design is in accordance with Chapter D, the safety and resistance factors in Section D4.1 should apply. Otherwise, large-scale tests are performed and strength controlled by structural concrete is limited by the safety and resistance factors in ACI 318-19. The safety and resistance factors presented in ACI 318-19 are included below:

<table>
<thead>
<tr>
<th>Diaphragm With Structural Concrete Type</th>
<th>ACI 318-19 Section</th>
<th>( \phi )</th>
<th>( \Omega )</th>
</tr>
</thead>
<tbody>
<tr>
<td>With Supplemental Reinforcement</td>
<td>21.2.1</td>
<td>0.75</td>
<td>2.15</td>
</tr>
<tr>
<td>With Supplemental Reinforcement for Seismic Force-Resisting Systems Defined in ACI Section 9.3.4</td>
<td>21.2.4</td>
<td>See ACI Section 21.2.4</td>
<td></td>
</tr>
<tr>
<td>Without Reinforcement</td>
<td>21.2.1</td>
<td>0.60</td>
<td>2.65</td>
</tr>
</tbody>
</table>

Note: 1 For consistency with Chapter B, the safety factor, \( \Omega \), equals \( 1.6/\phi \) in the table above. Normally, the ACI safety factor equals \( 1.5/\phi \).

Table E1.2-1 defines the essential parameters when evaluating connections and diaphragms. AISI S905 and AISI S907 define the minimum number of tests and parameter distribution. All tests, including repeats of identical tests, are included in the total number of tests, \( n \).

When determining a screw nominal strength [resistance] through tests in accordance with AISI S904 for use in a diaphragm strength model, the safety factor, \( \Omega \), and resistance factor, \( \phi \), are determined in accordance with this section.

(c) Large-scale tests to develop, modify or verify a connection analytical equation in an existing diaphragm system model, or to extend the application limits of an existing system model shall conform to Section E1.2.2 (b) and the calibration shall be as modified below:

\[
\frac{R_{t,i}}{R_{n,i}} \geq 0.60 \quad (Eq. \ E1.2.2-1)
\]

\( R_{t,i} \) = Tested nominal diaphragm shear strength [resistance] per unit length, \( S_{ni \text{ test}} \), of Test i

\( R_{n,i} \) = Calculated nominal diaphragm shear strength [resistance] per unit length, \( S_{ni \text{ theory}} \), of Test i per diaphragm system model

\( C_P \) = Correction factor is determined as follows:

1. \( C_P \) is determined in accordance with AISI S100 Eq. K2.1.1-4, and
2. Where a tested system falls within or extends the limits of an existing analytical model, it is permitted to set \( C_P = 1 \)
Exception: Where a test (ith test) does not conform to Eq. E1.2.2-1, i.e., \( \frac{R_{t,i}}{R_{n,i}} < 0.60 \), additional tests shall be performed in accordance with the following:

1. Repeat the ith test that does not conform to Eq. E1.2.2-1. If the average of the two tests meets Eq. E1.2.2-1, both tests are used in the calibration.

2. If the average per item 1 does not meet Eq. E1.2.2-1 and there are no other test results in the tested range, the developed analytical equation for \( S_n \) excludes this tested range in the acceptable parameter limits.

3. If other tests in the same range of tested parameters as those in \( R_{t,i} \) bound the nonconforming \( R_{t,i} \) and the average over that range including the nonconforming \( R_{t,i} \) conforms to Eq. E1.2.2-1, the developed analytical equation for \( S_n \) is permitted to include that range.

In large-scale tests with welded connections or any fastener that is subject to variation in size or installation quality, \( R_{n,i} \) (i.e., \( S_{ni, \text{theory}} \)), shall be based on the average of connection sizes measured at the supports and the average of connection sizes measured at sidelaps. Weld sizes are determined in accordance with Section E1.2.1.

It is permitted to apply the safety and resistance factors of an existing diaphragm system model to applications based on new large-scale test data without further calibration, provided:

1. New test data conforms to Eq. E1.2.2-1,

2. \( P_m \) determined using AISI S100 Eq. K2.1.1-3 equals or exceeds 0.95 with \( n \) being the number of new tests, and

3. New test data is equally weighted over the applicable range.

It is permitted to apply an existing diaphragm system equation for stiffness to applications based on new large-scale test data without further calibration, provided:

1. New test data conforms to Eqs. E1.2.2-2 and E1.2.2-3,

\[
\frac{G'_{i, \text{test}}}{G'_{i, \text{theory}}} \geq 0.50 \quad (\text{Eq. E1.2.2-2})
\]

\[
\frac{1}{n} \sum_{i=1}^{n} \frac{G'_{i, \text{test}}}{G'_{i, \text{theory}}} \geq 0.70 \quad (\text{Eq. E1.2.2-3})
\]

where

- \( G'_{i, \text{test}} \) = Tested diaphragm stiffness for an individual test, \( i \)
- \( G'_{i, \text{theory}} \) = Theoretical diaphragm stiffness for an individual test, \( i \)
- \( n \) = Number of new tests

2. New test data is equally weighted over the applicable range.

If separate connection strength equations are developed at butt joint and end-laps for design, those strength equations shall be used to calculate the nominal diaphragm shear strength [resistance] per unit length, \( S_{ni, \text{theory}} \), and the ratio, \( \frac{R_{t,i}}{R_{n,i}} \), for large-scale tests.
E1.2.3 Laboratory Testing Reports

The laboratory testing report shall include the information specified in the applicable test standard.

E2 Single Diaphragm System

The requirements of Sections E1 and E1.1 shall apply to single diaphragm systems unless noted otherwise. The number of tests and test methods, including the testing of configuration parameters, shall be in accordance with the AISI S907 requirements for a single diaphragm system.

User Note:
A single diaphragm system is typically used to test a particular detail or design application for a project. AISI S907 evaluates single diaphragm system tests in accordance with AISI S100 Section K2.1.1(a), and defines the number of tests and repeatability requirements.

E2.1 Test System Requirements

The following conditions shall be satisfied:

(a) The test structural supports and edge conditions are representative of the specified structure. Where more than one edge condition exists, the theoretically weakest condition or that chosen by the authority having jurisdiction is tested.

(b) The specified thickness of the system panel is not less than 0.95 times the average tested base steel thickness. All tested base steel thicknesses are within five (5) percent of the average.

(c) All tested yield stresses and tested tensile strengths are within 10 percent of the average tested strengths, respectively.

$S_n$ and $G'$ determined in accordance with Eqs. E2.1-1 and E2.1-2, respectively, shall be used in design. $S_{ni\text{ test}}$ shall be adjusted in accordance with Section E2.4.

$$S_n = \frac{1}{n} \sum_{i=1}^{n} S_{ni\text{ adj test}}$$  \hspace{1cm} (Eq. E2.1-1)

$$G' = \frac{1}{n} \sum_{i=1}^{n} G'_{i\text{ test}}$$  \hspace{1cm} (Eq. E2.1-2)

where

- $S_n$ = Nominal shear strength [resistance] per unit length used in design and the average adjusted nominal shear strength [resistance] per unit length of all $n$ tests
- $n$ = Total number of tests for a single diaphragm system
- $S_{ni\text{ test}}$ = Tested shear strength [resistance] per unit length for an individual test, $i$
- $S_{ni\text{ adj test}}$ = Adjusted shear strength [resistance] per unit length for an individual test, $i$
- $G'$ = Diaphragm stiffness used in design and the average of all $n$ tests
diaphragm stiffness for an individual test, $i$

### E2.1.1 Fastener and Weld Requirements

In all cases, the tested fastener type, size, and spacing shall be that specified:

(a) Arc spot welds in *support connections* shall meet the following requirements:

1. The support welds shall be measured over the *deck* or *panel* supports at *panel sidelaps* and at the adjacent interior flutes, if applicable. It is permitted to measure support welds in all flutes. The measured visible diameter at each support weld is the average of two orthogonal measurements with one being the largest visible diameter at the weld.

2. The measured visible diameter of each arc spot weld shall not exceed the specified visible diameter, $d$, by more than 25 percent.

3. $d_{\text{test}}$ shall not exceed the specified visible diameter, $d$, by more than 15 percent in an individual test.

(b) *Sidelap* welds in *sidelap connections* shall meet the following requirements:

1. All *sidelap* welds shall be measured.

2. The measured visible diameter of each arc spot weld shall not exceed the specified visible diameter, $d$, by more than 25 percent. The measured visible diameter at each *sidelap* arc spot weld is calculated as follows:
   
   - For full perimeter fusion at the *sidelap* weld, use the average of two orthogonal measurements with one being the largest visible diameter at the weld.
   - For discontinuous perimeter fusion at the *sidelap* weld, use the relationship:

     $$
     \text{Measured Visible Diameter} = \frac{\text{Measured Fused Perimeter}}{\pi} \quad (\text{Eq. E2.1.1-1})
     $$

3. For all *sidelap* welds, the average measured visible diameter, $d_{\text{test}}$, or average fused length, $L_{\text{w test}}$, shall be within 15 percent of the specified visible diameter, $d$, or the specified *sidelap* weld length, $L_{\text{w}}$, as applicable. For discontinuous fusion at a *sidelap* weld, the measured $L_{\text{w}}$ is the total length of fully fused zones at each weld.

4. For *sidelap welds* along each *sidelap* seam, the average measured visible diameter of all arc spot welds at that seam, or the average measured fused length of all fillet, groove, or *top arc seam sidelap welds* at that seam shall be within 15 percent of the specified diameter, $d$, or specified length, $L_{\text{w}}$, respectively.

   where

   - $d$ = Visible diameter of outer surface of arc spot weld
   - $d_{\text{test}}$ = Average measured visible diameter of the smallest set of 10 arc spot welds
   - $L_{\text{w}}$ = Length of fillet, groove, or *top arc seam sidelap weld*
   - $L_{\text{w test}}$ = Average fused length for the smallest set of 10 *sidelap* welds

*Diaphragm* shear strength per unit length reductions for welds shall be in accordance with Table E2.4.1-1 for systems without *structural concrete* fill over *deck*. 

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Fused perimeter at arc spot welds or fused length is the indicator of connection strength since failure normally occurs at these perimeters. Connection strength equations are proportional to visible diameter or fused length. Adjustment to an equivalent perimeter or length is required at sidelap welds because “blow holes” or discontinuities might occur in tests at such welds, and “blow holes” do not contribute to strength.

E2.1.2 Concrete Requirements

Specified concrete compressive strength, $f'_{c}$, for structural concrete slabs (fill over deck) or structural concrete supports shall be greater than or equal to 2500 psi (17.2 MPa). The test curing time is permitted to be less than 28 days, but not less than 7 days, where test cylinders for slab or support indicate that $f'_{c\text{test}}$ is or will be greater than the specified $f'_{c}$. Test cylinders shall be cured and tested as required by AISI S907.

For structural or insulating concrete fill, the difference between $d_{c\text{test}}$ and specified $d_{c}$ shall be less than or equal to 7.5 percent of the specified $d_{c}$ for $d_{c}$ less than or equal to 2½ in. (64 mm), and shall be less than or equal to 3/16 in. (5.0 mm) for specified $d_{c}$ greater than 2½ in. (64 mm). Measurement shall be as specified by AISI S907.

where

\[ d_{c} = \text{ Structural or insulating concrete thickness above top of deck} \]

\[ d_{c\text{test}} = \text{ Average tested structural or insulating concrete thickness over the top of the deck measured at supports} \]

\[ f'_{c} = \text{ Specified concrete compressive strength} \]

\[ f'_{c\text{test}} = \text{ Average tested concrete compressive strength for an individual test, i} \]

Minimum acceptable specified concrete compressive strength, $f'_{c}$, can also be limited by other design considerations such as fire rating and composite deck slab strength. See the controlling design specifications.

E2.2 Test Calibration

Calibration for a single diaphragm system shall be in accordance with Section E1.2.2(b) as modified below:

- $n$ = Total number of tests in accordance with AISI S907 for a single diaphragm system as specified in AISI S100 Section K2.1.1(a)
- $P_{m}$ = Mean value of professional factor, $P$, for tested component
  \[ = 1.0 \]

It is permitted to apply the safety or resistance factors of an existing diaphragm system model to the nominal shear strength [resistance] per unit length determined through tests provided the configuration of the single diaphragm system meets the following conditions:

(a) The configuration conforms to the limits of the existing diaphragm system model;
(b) $S_{n\text{theory}}$ is determined using the existing diaphragm system model; and
(c) The ratio, $S_{n}/S_{n\text{theory}}$, is bounded by the existing diaphragm model test database.
where

\[ S_n = \text{Nominal shear strength [resistance] per unit length of single diaphragm system defined by Eq. E2.1-1} \]

\[ S_{n \text{ theory}} = \text{Calculated nominal diaphragm shear strength [resistance] per unit length for a configuration based on the specified parameters} \]

Where panel out-of-plane buckling controls the nominal diaphragm shear strength [resistance] per unit length, \( S_n \), of the tested configuration, and the test set does not define the connection-related diaphragm shear strength [resistance] per unit length, the following two options shall be used to determine the safety and resistance factors:

1. The safety and resistance factors are permitted to be determined by calibration of the test results using the calibration parameters listed in Table E1.2.2-1. The calibrated safety factor shall be greater than and the resistance factor shall be less than the values determined:
   (i) Using Section B Table B-1 where diaphragm strength is determined by calculation using Chapter D, and
   (ii) Using Table B1.1-1 for the connection-related limit state.

2. It is permitted to apply the panel out-of-plane buckling factors, \( \Omega_{db} \) or \( \phi_{db} \), of Section B1 to the nominal diaphragm shear strength [resistance] where:
   (i) The single diaphragm system test is within the limits (a) through (d) of Chapter D, and
   (ii) The available diaphragm shear strength [factored resistance] per unit length \( (S_{nf}/\Omega_{df} \text{ or } \phi_{df}S_{nf}) \) calculated in accordance with Section D1 does not control the available shear strength [factored resistance] of the tested single diaphragm system.

User Note:
The performance of a single diaphragm system is only applicable to that specific system; therefore, the calibration follows AISI S100 Section K2.1.1(a). \( V_P \), which indicates repeatability, is unique for the single diaphragm system. AISI S100 Section K2.1.1(a) does not require determination of a correlation coefficient, \( C_c \).

Sections E1.2 and E1.2.2 of the Commentary provide information on existing test-based systems and test data scatter.

E2.3 Laboratory Testing Reports

The laboratory testing report shall include the information specified in AISI S907 Section 13.

E2.4 Adjustment for Design

The nominal diaphragm shear strength [resistance] per unit length, \( S_{ny} \) of a single diaphragm system shall be determined in accordance with Eq. E2.1-1. Adjustment is required where any one specified \( d, t, F_u, d_c, \text{ or } t_c' \) is less than the corresponding \( d_{test}, t_{test}, F_{u\text{ test}}, d_{c\text{ test}}, \text{ or } t_{c'\text{ test}} \). The adjusted diaphragm shear strength [resistance] per unit length per test, \( S_{ni \text{ adj test}} \) shall be modified relative to the as-tested shear strength [resistance] per unit length, \( S_{ni \text{ test}} \), in accordance with Sections E2.4.1 for diaphragms without structural concrete fill and E2.4.2 for diaphragms with structural concrete fill.

\( S_{ni \text{ adj test}} \) shall not be increased for any parameter where the specified value is greater than or equal to the tested value for that parameter.

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where
\[
d_c = \text{Structural or insulating concrete thickness above top of deck}
\]
\[
d_{c\text{test}} = \text{Average tested structural or insulating concrete thickness above top of the deck measured at supports}
\]
\[
t = \text{Base steel thickness of panel}
\]
\[
t_{\text{test}} = \text{Average value of tested panel’s thickness for an individual test, i}
\]
\[
f_c = \text{Specified structural or insulating concrete compressive strength}
\]
\[
f_{c\text{test}} = \text{Average tested structural or insulating concrete compressive strength for an individual test, i}
\]
\[
F_u = \text{Specified tensile strength of sheet as determined in accordance with AISI S100 Sections A3.1 or A3.2}
\]
\[
F_{u\text{test}} = \text{Average value of tested panel’s tensile strength for an individual test, i}
\]
\[
S_{n\text{test}} = \text{Tested shear strength [resistance] per unit length for an individual test, i}
\]
Other parameters are defined in Section E2.1.

**E2.4.1 Adjustment to Strength of Diaphragms Without Structural Concrete Fill**

This section shall apply to diaphragms with panels only or with insulating concrete fill over deck, and shall include steel, wood or structural concrete supports. Reductions for each parameter listed in Section E2.4 or combination of parameters shall be applied to \(S_{n\text{test}}\). The adjusted diaphragm strength [resistance], \(S_{n\text{adj test}}\), shall be determined in accordance with Table E2.4.1-1:
### Table E2.4.1-1
Adjustment of Tested Nominal Diaphragm Strength, $S_{ni\text{ test}}$, Due to Variations in Deck, Panel, Concrete Support or Insulating Fill Material From Specified Values

<table>
<thead>
<tr>
<th>Condition</th>
<th>Modification$^5$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\frac{t}{t_{\text{test}}} &lt; 1$</td>
<td>$S_{ni\text{ adj test}} = \frac{t}{t_{\text{test}}} S_{ni\text{ test}}$</td>
</tr>
<tr>
<td>$\frac{F_u}{F_{u\text{ test}}} &lt; 1$</td>
<td>$S_{ni\text{ adj test}} = \min\left(\frac{1.1F_u}{F_{u\text{ test}}}, 1\right) S_{ni\text{ test}}$</td>
</tr>
<tr>
<td>$\frac{t}{t_{\text{test}}} &amp; \frac{F_u}{F_{u\text{ test}}} &lt; 1$</td>
<td>$S_{ni\text{ adj test}} = \frac{t}{t_{\text{test}}} \min\left(\frac{1.1F_u}{F_{u\text{ test}}}, 1\right) S_{ni\text{ test}}$</td>
</tr>
<tr>
<td>Weld$^1 \frac{d}{d_{\text{test}}} &lt; 1$</td>
<td>$S_{ni\text{ adj test}} = \frac{d}{d_{\text{test}}} S_{ni\text{ test}}$</td>
</tr>
<tr>
<td>$\frac{d}{d_{\text{test}}}, \frac{t}{t_{\text{test}}} &amp; \frac{F_u}{F_{u\text{ test}}} &lt; 1$</td>
<td>$S_{ni\text{ adj test}} = \frac{d}{d_{\text{test}}} \frac{t}{t_{\text{test}}} \min\left(\frac{1.1F_u}{F_{u\text{ test}}}, 1\right) S_{ni\text{ test}}$</td>
</tr>
<tr>
<td>Structural concrete $\frac{f_c'}{f_c'} &lt; 1$</td>
<td>$S_{ni\text{ adj test}} = \sqrt{\frac{f_c'}{f_{c\text{ test}}}} S_{ni\text{ test}}$</td>
</tr>
<tr>
<td>Wood Support</td>
<td>See Note$^4$</td>
</tr>
</tbody>
</table>

### Insulating Concrete Fill$^3$

<table>
<thead>
<tr>
<th>Condition</th>
<th>Modification</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\frac{d_c}{d_{c\text{ test}}} &amp; \frac{f_c'}{f_{c\text{ test}}} &lt; 1$</td>
<td>$S_{ni\text{ adj test}} = 0.5 \left(1 + \frac{d_c}{d_{c\text{ test}}} \sqrt{\frac{f_c'}{f_{c\text{ test}}}}\right) S_{ni\text{ test}}$</td>
</tr>
<tr>
<td>$\frac{d}{d_{\text{test}}}, \frac{t}{t_{\text{test}}} \frac{F_u}{F_{u\text{ test}}} \frac{f_c'}{f_{c\text{ test}}} &amp; \frac{d_c}{d_{c\text{ test}}} &lt; 1$</td>
<td>$S_{ni\text{ adj test}} = 0.5 \left(\frac{d}{d_{\text{test}}} \frac{t}{t_{\text{test}}} \min\left(\frac{1.1F_u}{F_{u\text{ test}}}, 1\right) + \frac{d_c}{d_{c\text{ test}}} \sqrt{\frac{f_c'}{f_{c\text{ test}}}}\right) S_{ni\text{ test}}$</td>
</tr>
</tbody>
</table>

**Note:**

1. Some variation is expected in $d_{\text{test}}$. See Section D1.1.1.
2. Reduction applies where *structural concrete bearing strength* controls the *support connection strength*.
3. At *concrete* supports, substitute $\sqrt{\frac{f_c'}{f_{c\text{ test}}}}$ for $\frac{d}{d_{\text{test}}}$ in combined modifications.
4. Wood support size, species, and fastener conform to Sections E2.1(a) and E2.1.1 and reduction is not required for the *support connection*.
5. For a specified value greater than or equal to the tested value for that parameter, insert 1 at that particular ratio in the reduction equation.

where

- $d$ = Visible diameter of outer surface of arc spot weld as specified and located over support
- $d_{\text{test}}$ = Average measured visible diameter of the smallest set of 10 support arc spot welds for an individual test, $i$
- $d_c$ = *Insulating concrete* thickness above top of deck as specified
- $d_{c\text{ test}}$ = Average tested *insulating concrete* thickness above top of the deck and at the

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supports for an individual test, \( i \)

\[
\begin{align*}
\bar{f}_c & = \text{Insulating concrete compressive strength as specified for fill} \\
& = \text{Specified structural concrete compressive strength as specified for structural concrete support} \\
\bar{f}_{c, \text{test}} & = \text{Average tested insulating concrete compressive strength for an individual test, } i \\
& = \text{Average tested structural concrete compressive strength for an individual test, } i, \text{ for structural concrete supports}
\end{align*}
\]

Other parameters are defined in Sections E2.1 and E2.4.

**User Note:**

The adjustment for lightweight insulating concrete depth or compressive strength variation from specified values should be made in accordance with Section E2.4.1. Diaphragm shear strength per unit length with lightweight insulating concrete fill without insulating board can be calculated in accordance with Eq. D4.3-1. Table E2.4.1-1 uses this relationship and rationalizes that both deck and lightweight insulating concrete fill provide significant contributions to insulating concrete-filled diaphragm strength. The Table E2.4.1-1 adjustment is applicable as long as the tested properties are close to specified values since it assigns relatively equal weight to each contribution.

Table E2.4.1-1 assumes that the support is sufficiently thick so its properties do not control connection strength. If this is not the case, an adjustment can be made by replacing panel thickness, \( t \), with \( t_{\text{support}} \) and replacing panel tensile strength, \( F_u \), with \( F_u \) support.

\( \bar{f}_c \) and \( d_{\text{test}} \) should be determined in accordance with AISI S907.

### E2.4.2 Adjustment to Strength of Diaphragms With Structural Concrete Fill

Tested strength [resistance], \( S_{ni, \text{test}} \), for diaphragms with concrete fill over deck shall be adjusted in accordance with this section. The adjusted diaphragm strength [resistance], \( S_{ni, \text{adj test}} \), shall be determined in accordance with Table E2.4.2-1:

<table>
<thead>
<tr>
<th>Condition</th>
<th>Modification</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \frac{f_c'}{f_{c, \text{test}}} &lt; 1 )</td>
<td>( S_{ni, \text{adj test}} = \frac{\sqrt{f_c'}}{f_{c, \text{test}}} S_{ni, \text{test}} )</td>
</tr>
<tr>
<td>( \frac{d_c}{d_{c, \text{test}}} &lt; 1 )</td>
<td>( S_{ni, \text{adj test}} = \frac{d_c}{d_{c, \text{test}}} S_{ni, \text{test}} )</td>
</tr>
<tr>
<td>( \frac{d_c}{d_{c, \text{test}}} &amp; \frac{f_c'}{f_{c, \text{test}}} &lt; 1 )</td>
<td>( S_{ni, \text{adj test}} = \frac{d_c}{d_{c, \text{test}}} \sqrt{\frac{f_c'}{f_{c, \text{test}}}} S_{ni, \text{test}} )</td>
</tr>
</tbody>
</table>

where

\[
\begin{align*}
\bar{d}_c & = \text{Structural concrete thickness above top of deck as specified} \\
\bar{d}_{c, \text{test}} & = \text{Average tested structural concrete thickness above top of the deck at the supports for an individual test, } i \\
\bar{f}_c' & = \text{Specified structural concrete compressive strength as specified for fill}
\end{align*}
\]

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= Structural concrete compressive strength as specified for structural concrete support

\[ f_{c_{\text{test}}} \]

= Average tested structural concrete compressive strength for fill in an individual test, i

\[ f_{c_{\text{test}}} \]

= Average tested structural concrete compressive strength for structural concrete supports in an individual test, i

\[ S_{n_{i \text{ test}}} \]

= Tested diaphragm shear strength [resistance] per unit length for an individual test, i

\[ S_{n_{i \text{ adj test}}} \]

= Adjusted diaphragm shear strength [resistance] per unit length for an individual test, i

User Note:

\[ f_{c_{\text{test}}} \] and \[ d_{c_{\text{test}}} \] should be determined in accordance with AISI S907. Table E2.4.2-1 assumes that structural concrete fill provides most of the nominal diaphragm shear strength [resistance] per unit length of the diaphragm system and that sufficient support connections are present to allow this.

Table E2.4.2-1 should be based on \[ f_{c} \] and \[ f_{c_{\text{test}}} \] for the structural concrete fill. If the support connection controls \[ S_{n_{i \text{ test}}} \] use the support ratio, \[ \sqrt{\frac{f_{c}}{f_{c_{\text{test}}}}} \], as the modifier. However, fill modifiers, \[ \frac{d_{c}}{d_{c_{\text{test}}}} \sqrt{\frac{f_{c}}{f_{c_{\text{test}}}}} \], need not be used in combination with the support modifier.

E2.5 Test Results Interpretation

The test results of a single diaphragm system shall be applied to applications as specified in the test’s objective. The test results of a single diaphragm system with a single span are permitted to be used in a design having multiple spans provided the following conditions are satisfied:

(a) The span of panel between supports with fasteners, \( L_v \), of the multiple span application is the same as that of the tested single span diaphragm system.

(b) The number of connections at exterior supports equals the number of connections in the tested single span diaphragm system, and the number of connections at interior supports is increased in accordance with the analytical method of Section D1. The increased number of connections at interior supports shall provide equivalent or greater calculated multiple span system diaphragm strength relative to the calculated diaphragm system strength of the tested single span system with calculations based on the specified design application parameters. All calculation parameters other than panel length, \( L \), and number of interior support connections are held constant including panel profile, thickness, yield stress, tensile strength, support connection type, sideload connection type and spacing, span of panel between supports with fasteners, \( L_v \), and where applicable, concrete fill type, compressive strength, and depth. As compared to the exterior supports, the additional interior support fasteners shall be located closest to sidelaps with one installed in each flute and progressing to the center of the deck or panel. The number of fasteners at end-laps over exterior supports shall be the same as that required at interior supports.
**User Note:**

See Figure D1-1 for clarification of exterior and interior supports. The difference in the required number of fasteners at interior and exterior supports is reflected in the variable, $\beta$, in Eqs. D1-1 and D1-2, where $\alpha_p^2 \neq \alpha_e^2$ in the multi-span case (say $L = 2L_v$ for two spans) and $n_p = 0.0$ in single spans.
Appendix 1: Determination of Factors, D_n and γ_c

1.1 General

1.1.1 Scope

This appendix addresses the determination of the warping factor, D_n, and the support factor, γ_c, that are required to analytically determine the stiffness, G’, in Section D5.

1.1.2 Applicability

This appendix applies to perforated and non-perforated profiled panels that conform to the limits (a) to (d) of Chapter D. It is permitted to set D_n = 0 for perforated or non-perforated cellular deck that conform to the limits (a) to (g) of Section D1.5.

1.2 Determination of Warping Factor, D_n

Where the analytical method of Chapter D is used, stiffness, G’, shall be determined in accordance with Eq. D5.1.1-1, in which D_n shall be determined in accordance with this appendix. Section 1.4 shall be used where insulation is not present beneath the panel. Section 1.5 shall be used where insulation is present beneath the panel and the diaphragm meets the limits (a) to (f) of Section D1.3. Section 1.6 shall be used where perforations are present in acoustic panels.

It is permitted to determine G’ by test in accordance with Chapter E. It is permitted to use existing diaphragm system theories to include the end warping effect in accordance with Section E1.2.

User Note:

Section 1.4 can also be used for diaphragm systems with insulation between the panel and the support as long as the fluted panel meets the requirements (a) to (d) as specified in Chapter D. Chapter D does not consider increased stiffness caused by insulation above the panel with the exception of insulating concrete fill. Section 1.5 can be used for diaphragm systems without insulation between the panel and the support provided the fluted panel meets the requirements (a) to (f) specified in Section D1.3.

Where Chapter E is used, the test will include the end warping contribution.

1.3 Determination of Support Factor, γ_c

Support factor, γ_c, in Eq. D5.1.1-1 shall be determined in accordance with Table 1.3-1.

<table>
<thead>
<tr>
<th>Spans</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>≥7</th>
</tr>
</thead>
<tbody>
<tr>
<td>γ_c</td>
<td>1.00</td>
<td>1.00</td>
<td>0.90</td>
<td>0.80</td>
<td>0.71</td>
<td>0.64</td>
<td>0.58</td>
</tr>
</tbody>
</table>

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1.4 Determination of Warping Factor Where Insulation Is Not Present Beneath the Panel

\( D_n \) for profiled panels shall be determined using the dimensions defined in Section D2 and the parameters as shown in Figure 1.4-1.

![Figure 1.4-1 Panel Configuration](image)

The unitless warping factor, \( D_n \), shall be developed using equation, Eq. 1.4-1:

\[
D_n = \frac{D}{L}
\]

where

\( D = \) Weighted average \( D_i \) value for warping across the panel width, \( w \), in. (mm)

\[
D = \frac{U_1D_1 + U_2D_2 + U_3D_3 + U_4D_4}{U_1 + U_2 + U_3 + U_4}
\]

\( D_1 = \) Value for warping where bottom flange fastener is in every valley

\[
D_1 = \frac{\gamma_1 f}{d(t)^{1.5}} \text{ in. (mm)}
\]

\( D_2 = \) Value for warping where bottom flange fastener is in every second valley

\[
D_2 = \frac{\gamma_2 f}{2d(t)^{1.5}} \text{ in. (mm)}
\]

\( D_3 = \) Value for warping where bottom flange fastener is in every third valley

\[
D_3 = \frac{\gamma_3 f}{3d(t)^{1.5}} \text{ in. (mm)}
\]

\( D_4 = \) Value for warping where bottom flange fastener is in every fourth valley

\[
D_4 = \frac{\gamma_4 f}{4d(t)^{1.5}} \text{ in. (mm)}
\]

\( U_1 = \) Number of corrugations having fasteners in every valley across the panel width, \( w \)

\( U_2 = \) Number of corrugations having fasteners in every second valley across the panel width, \( w \)

\( U_3 = \) Number of corrugations having fasteners in every third valley across the panel width, \( w \)

\( U_4 = \) Number of corrugations having fasteners in every fourth valley across the panel width, \( w \)
width, \( w \)
\[ L = \text{Total panel length, in. (mm)} \]

**User Note:**
The warping factor, \( D_{yw} \), measures both the lateral racking and accordion distortion of panels and the arching of corrugations between support fasteners. This distortion is localized near the panel ends and the equation indicates that the impact is reduced at longer panel lengths.

The warping parameter, \( D \), considers the panel profile and connection spacing at panel ends.

The \( D \) value for warping shall be developed using Eq. 1.4-7 through Eq. 1.4-34.

where
\[ s = \text{Developed flute width per pitch} \]
\[ = f + 2c + 2w \]  
(Eq. 1.4-7)

\[ \delta_{ij} = \text{Deflection indicator of profile racking per unit load per unit length required for } D, \in./\text{in.}^3\text{ (mm}/\text{mm}^3\text{)} \]

\[ \kappa_{ij} = \text{Spring constant indicator required for } D, \frac{1}{\text{in.}^3}\left(\frac{1}{\text{mm}^3}\right) \]

**User Note:**
The relationship between the deflection, \( \Delta_{ij} \), at joint, \( i \), on a panel caused by a unit load per unit length at joint, \( j \), on the panel and the deflection indicator, \( \delta_{ij} \), is as follows:
\[ \Delta_{ij} = \frac{\delta_{ij}}{EI_y} \text{ in./kip/in (mm/kN/mm)} \]

See the Commentary on Appendix 1-4 and SDI DDM01, Appendix A, for an explanation of the subscripts and load point locations. For \( \Delta_{ij} \) and \( \delta_{ij} \) \( ij = 11, 12, 22 \).

The relationship between the spring constant, \( K_{ij} \), at joint, \( i \), on a panel associated with a bottom flat connection spacing or released restraint, \( j \), on the panel and the spring constant indicator, \( \kappa_{ij} \), is as follows:
\[ K_{ij} = EI_y \kappa_{ij} \text{ kip/in./in. (kN/mm/mm)} \]

\[ I_y = \frac{bt^3}{12} \text{ in.}^4/\text{in.} \text{ (mm}^4/\text{mm)} \]

where
\[ b = \text{unit length of the panel, 1 in. or 1 mm, as applicable} \]

For \( K_{ij} \) and \( \kappa_{ij} \) \( ij = t1, t2, t3, t4, b2, b3, b4, tc3, tc4, bc4 \). The subscripts, tc and bc, apply to spring constants at the top or bottom of central flutes where bottom flats are not restrained at cases \( j = 3 \) or 4. There is 1 central flute at \( j = 3 \) and there are 2 central flutes at \( j = 4 \).

where
\[ \delta_{11} = \frac{D^2_d}{3}(2w + 3f) \]  
(Eq. 1.4-8)

\[ \delta_{12} = \frac{\delta_{11}}{2} \]  
(Eq. 1.4-9)
\[ \delta_{22} = \frac{1}{12} \left( \frac{D_d}{d} \right)^2 \left[ \left( 4e^2 - 2ef + f^2 \right) + d^2(3f + 2w) \right] \]  
(Eq. 1.4-10)

\[ \kappa_{t1} = \frac{1}{\delta_{22} - \frac{\delta_{12}}{2}} \]  
(Eq. 1.4-11)

\[ \kappa_{t2} = \frac{1}{\left( \frac{2e}{f} \right) \left( \frac{\delta_{12}}{2} \right) + \delta_{22}} \]  
(Eq. 1.4-12)

\[ \kappa_{t3} = \frac{1}{\left( 0.5 + \frac{2e}{f} \right) \delta_{12} + \delta_{22}} \]  
(Eq. 1.4-13)

\[ \kappa_{t4} = \frac{1}{\left( 1 + \frac{3e}{f} \right) \delta_{12} + \delta_{22}} \]  
(Eq. 1.4-14)

\[ \kappa_{b2} = \frac{\frac{2e}{f} \delta_{11}}{\frac{2e}{f} + \frac{\delta_{12}}{2}} \]  
(Eq. 1.4-15)

\[ \kappa_{b3} = \frac{\frac{2e}{f}}{\left( 0.5 + \frac{2e}{f} \right) \delta_{11} + \delta_{12}} \]  
(Eq. 1.4-16)

\[ \kappa_{b4} = \frac{\frac{2e}{f}}{\left( 1 + \frac{3e}{f} \right) \delta_{11} + \delta_{12}} \]  
(Eq. 1.4-17)

\[ \kappa_{tc3} = \frac{1}{\left( 0.5 + \frac{2e}{f} \right) \delta_{11} + \delta_{22} + \frac{\delta_{12}}{2}} \]  
(Eq. 1.4-18)

\[ \kappa_{tc4} = \frac{1}{\left( 1.0 + \frac{3e}{f} \right) \delta_{11} + \delta_{22} + \left( 1.0 + \frac{e}{f} \right) \delta_{12}} \]  
(Eq. 1.4-19)

\[ \kappa_{bc4} = \frac{\frac{2e}{f}}{\left( 1 + \frac{4e}{f} \right) \delta_{11} + 2\delta_{12}} \]  
(Eq. 1.4-20)

\[ \delta_{ti} = \text{Lateral displacement indicator at top of corrugation for valley fastener cases, } i = 1 \text{ to } 4, \text{ in.}^{2.5} \left( \text{mm}^{2.5} \right) \]
\( \delta_{bi} \) = Lateral displacement indicator at bottom of corrugation for valley fastener cases, 
\( i = 1 \) to \( 4 \), in.\( ^{2.5}\) (mm\( ^{2.5} \))
\( \delta_{b1} = 0 \) for fasteners in the bottom flat of each flute.

\[
\delta_{t1} = \frac{24 f}{\kappa_{t1}} \left[ \frac{\kappa_{t1}}{4f^2 (f+w)} \right]^{-0.25} 
\text{(Eq. 1.4-21)}
\]

\[
\delta_{t2} = \frac{24 f}{\kappa_{t2}} \left[ \frac{\kappa_{t2}}{4f^2 (f+w)} \right]^{-0.25} 
\text{(Eq. 1.4-22)}
\]

\[
\delta_{t3} = \frac{24 f}{\kappa_{t3}} \left[ \frac{\kappa_{t3}}{4f^2 (f+w)} \right]^{-0.25} 
\text{(Eq. 1.4-23)}
\]

\[
\delta_{t4} = \frac{24 f}{\kappa_{t4}} \left[ \frac{\kappa_{t4}}{4f^2 (f+w)} \right]^{-0.25} 
\text{(Eq. 1.4-24)}
\]

\[
\delta_{b2} = \frac{48 e}{\kappa_{b2}} \left[ \frac{\kappa_{b2}}{16e^2 (2e+w)} \right]^{-0.25} 
\text{(Eq. 1.4-25)}
\]

\[
\delta_{b3} = \frac{48 e}{\kappa_{b3}} \left[ \frac{\kappa_{b3}}{16e^2 (2e+w)} \right]^{-0.25} 
\text{(Eq. 1.4-26)}
\]

\[
\delta_{b4} = \frac{48 e}{\kappa_{b4}} \left[ \frac{\kappa_{b4}}{16e^2 (2e+w)} \right]^{-0.25} 
\text{(Eq. 1.4-27)}
\]

\[
\delta_{tc3} = \frac{24 f}{\kappa_{tc3}} \left[ \frac{\kappa_{tc3}}{4f^2 (f+w)} \right]^{-0.25} 
\text{(Eq. 1.4-28)}
\]

\[
\delta_{tc4} = \frac{24 f}{\kappa_{tc4}} \left[ \frac{\kappa_{tc4}}{4f^2 (f+w)} \right]^{-0.25} 
\text{(Eq. 1.4-29)}
\]

\[
\delta_{bc4} = \frac{48 e}{\kappa_{bc4}} \left[ \frac{\kappa_{bc4}}{16e^2 (2e+w)} \right]^{0.25} 
\text{(Eq. 1.4-30)}
\]
\[ \gamma_i = \text{Final displacement indicator at top of corrugation for valley fastener cases, } i = 1 \text{ to } 4, \text{ in.}^{2.5}(\text{mm}^{2.5}) \]

\[ \gamma_1 = \delta_{1i} \quad (Eq. 1.4-31) \]

\[ \gamma_2 = 2 \delta_{2i} + \frac{2e}{f} \delta_{b2} \quad (Eq. 1.4-32) \]

\[ \gamma_3 = 2 \delta_{3i} + \delta_{tc3} + 2 \left( \frac{2e}{f} \right) \delta_{b3} \quad (Eq. 1.4-33) \]

\[ \gamma_4 = 2(\delta_{4i} + \delta_{tc4}) + \left( \frac{2e}{f} \right)(2\delta_{b4} + \delta_{bc4}) \quad (Eq. 1.4-34) \]

### 1.5 Determination of Warping Factor Where Insulation is Present Beneath the Panel

Where the limits of Chapter D are met and panel depth, \( D_d \), is less than or equal to 4 in. (102 mm), it is permitted to use the simplified Eq. 1.5-1 whether or not insulation is present. Section 1.5 shall not apply to perforated panels.

Parameters are defined in Section D2 and shown in Figure 1.4-1.

\[ D_n = \frac{1}{n} \sum_{i=1}^{n} D_{ni} \quad (Eq. 1.5-1) \]

where

\[ D_n = \text{Warping factor considering distortion at panel ends} \]

\[ D_{ni} = \text{Warping factor for each corrugation, } i \]

\[ D_n = \frac{D_d f^2 \left( \frac{1}{t} \right)^{1.5}}{25\alpha L} \quad \text{For } \psi = 1 \quad (Eq. 1.5-2) \]

\[ D_n = \frac{0.94d\psi^2}{f} \left( \frac{D_d f^2 \left( \frac{1}{t} \right)^{1.5}}{25\alpha L} \right) \quad \text{For } 1 < \psi \leq 3 \quad (Eq. 1.5-3) \]

where

\[ \alpha = \text{Conversion factor} \]

\[ \alpha = 1 \quad \text{for U.S. Customary units} \]

\[ \alpha = 420 \quad \text{for SI units} \]

\[ \psi = \text{Number of corrugations between support fasteners at the panel end for the set of corrugations containing the corrugation, } i \]

Unit of the parameters in Eqs. 1.5-2 and 1.5-3 are defined:

<table>
<thead>
<tr>
<th>Variables</th>
<th>U.S. Customary</th>
<th>SI</th>
</tr>
</thead>
<tbody>
<tr>
<td>( D_d, g, f, w, e, d, t )</td>
<td>in.</td>
<td>mm</td>
</tr>
<tr>
<td>L</td>
<td>ft</td>
<td>m</td>
</tr>
</tbody>
</table>

\[ n = \text{Number of corrugations in a total panel cover width, } w; \]

\[ n = \frac{w}{d} \quad (Eq. 1.5-4) \]
**User Note:**

\( \psi = 2 \) for alternate valley spacing. The Section 1.5 equations are based on a parametric study of the Section 1.4 method. The value, \( D_n \), will not always be exactly the same using the two methods. The Section 1.4 method is required for final design when \( \psi \) is greater than 3.

### 1.6 Determination of Warping Factor for Perforated Deck

The warping value, \( D \), in Eq. 1.4-1 shall be determined using the modified values, \( e_p \), \( f_p \), and \( w_p \) for the profile parameters, \( e \), \( f \), and \( w \) in Eq. 1.4-7 through Eq. 1.4-10. The modification shall be in accordance with Eq. 1.6-1 through Eq. 1.6-3.

\[
e_p = K_{E_e}^{1/3} e
\]
\[
f_p = K_{E_f}^{1/3} f
\]
\[
w_p = K_{E_w}^{1/3} w
\]

where

\( K_{E_i} \) = Indicator of relative flexural stiffness of an element without perforations to the stiffness of the element with perforations over part of its length

\[
K_{E_i} = 1 + A_i^3 \left( \frac{1}{k} - 1 \right)
\]

where

\( A_i \) = Ratio of perforated width to the full element width

\( i \) = Index of perforated elements in a profile

\( e \) = at bottom flat

\( w \) = at web

\( f \) = at top flat

\( A_e \) = Ratio of bottom perforated width to the bottom width

\( A_f \) = Ratio of top perforated width to the top width

\( A_w \) = Ratio of web perforated width to the web width

\( k \) = Ratio of the perforated element stiffness relative to that of a solid element

\[
k = 0.9 + p_o^2 - 1.875p_o \quad \text{for } 0.2 \leq p_o \leq 0.58
\]

where

\( p_o \) = Ratio of the area of perforations to the total area in the perforated band

**User Note:**

See the *Commentary* for a recommendation when \( p_o < 0.2 \).
Appendix 2: Strength at Perimeter Load Delivery Point

2.1 General

2.1.1 Scope

This appendix determines the following:
(a) Forces on panels and connections fastened to perimeter supports perpendicular to the panel span, and
(b) Available shear strength [factored resistance] per unit length of a diaphragm where perimeter connections are loaded.

2.1.2 Applicability

This appendix applies to perforated and non-perforated profiled panels and perforated and non-perforated cellular deck that are fastened to perimeter supports having limited weak axis bending stiffness when collection struts or wind trusses are not present to transfer load into the diaphragm.

2.2 Connection Design

The nominal diaphragm shear strength [resistance] per unit length, $S_{nf}$, shall be determined using Eq. 2.2-1. Parameters are defined in Section D1. See Figure 2.2-1.

For ASD

$$S_{nf} = \left( \frac{\beta N}{N^2 L^2 + \beta^2} \right) - \Omega_{df} w_a L + \sqrt{\frac{\Omega_{df}^2 w_a^2 L^2}{N^2 L^2 + \beta^2} + \left( P_{nf} - \Omega_{nf}^2 \frac{w_a^2}{N^2} \right) \frac{P_{nf}^2}{\Omega_{df}^2 N^2}}$$  \hspace{1cm} (Eq. 2.2-1a)

For LRFD and LSD

$$S_{nf} = \left( \frac{\beta N}{N^2 L^2 + \beta^2} \right) - \frac{w_u L}{\phi_{df}} + \sqrt{\frac{w_u^2 L^2}{\phi_{df}^2} + \left( N^2 L^2 + \beta^2 \right) \left( P_{nf} - \frac{w_u^2}{\phi_{df} N^2} \right) \frac{P_{nf}^2}{\phi_{df}^2 N^2}}$$  \hspace{1cm} (Eq. 2.2-1b)

where

- $w_a$ = External nominal load reaction, kip/ft (kN/m), requiring allowable diaphragm strength, $S_{nf} / \Omega_{df}$
- $w_u$ = Factored external nominal load reaction, kip/ft (kN/m), demanding the design diaphragm strength [factored resistance], $\phi_{df} S_{nf}$
- $L$ = Total panel length, ft (m) [at perimeter]
- $N$ = Number of support connections per unit width at an interior or edge panel’s end (perimeter panel end in this particular case), 1/ft (1/m)
- $P_{nf}$ = Nominal shear strength [resistance] of a support connection per fastener in accordance with Section D1.1, kip (kN) (at perimeter panel end)
- $S_{nf}$ = Nominal shear strength [resistance] per unit length of diaphragm system controlled by connections, kip/ft (kN/m)
\( \phi_{df} = \) Connection resistance factor for diaphragm strength controlled by connections and determined in accordance with Table B-1

\( \Omega_{df} = \) Safety factor for diaphragm strength controlled by connections and determined in accordance with Table B-1

**User Note:**

Eq. D1-2 is a special case of Eq. 2.2-1a where \( w_a = 0.0 \) or Eq. 2.2-1b where \( w_u = 0.0 \). Statics analysis indicates that all shear would flow from the chord (beam flange) into the diaphragm along this line. The force component per fastener at that perimeter is:

\[
\frac{S_n}{\Omega_{df} N} \text{ or } \frac{\phi_d S_n}{N}
\]

Eq. 2.2-1 is derived from the free body diagram in Figure 2.2-1. The statics requirement is covered by Eq. D1-2 when \( w_a \) or \( w_u \) is not present.

*Loads* causing compression in the panel are discussed in Section 2.3, and *loads* causing tension in the panel are discussed in Section 2.4. Both should be considered.

**2.3 Axial Compression Design in Panel**

**User Note:**

Design is an application of AISI S100 Sections E2 and E3 and Section H2.

\[
P_n = A_e F_n
\]

\( A_e = \text{Effective area per unit width of panel at stress } F_n, \text{ in}^2/\text{ft} (\text{m}^2/\text{m})\)

\( F_n = \text{Compressive stress at the nominal axial strength, kip/in}^2 (\text{kN/m}^2)\)

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\[ P_n = \text{Nominal compressive axial strength} [\text{resistance}] \text{ of panel per unit width, kip/ft (kN/m)} \]

For \( \lambda_c \leq 1.5 \)
\[ F_n = \left( 0.658 \lambda_c^2 \right) F_y \]  
(AISI S100 Eq. E2-2)

For \( \lambda_c > 1.5 \)
\[ F_n = \left( \frac{0.877}{\lambda_c^2} \right) F_y \]  
(AISI S100 Eq. E2-3)

\[ \lambda_c = \sqrt{\frac{F_y}{F_{cre}}} \]  
(AISI S100 Eq. E2-4)

\[ F_{cre} = \frac{\pi^2 E}{\left( \frac{KL_v}{r} \right)^2} \]  
(AISI S100 Eq. E2.1-1)

\[ r = \left( \frac{I_{xg}}{A_g} \right)^{1/2} \]  
(Eq. 2.3-1)

where
\( r \) = Radius of gyration of panel, in. (m)
\( A_g \) = Area of fully effective (unreduced) panel per unit width, in.\(^2\)/ft (m\(^2\)/m)
\( E \) = Modulus of elasticity of steel
\( F_{cre} \) = Elastic flexural buckling stress of panel, kip/in.\(^2\) (kN/m\(^2\))
\( F_y \) = Yield stress of specified steel, kip/in.\(^2\) (kN/m\(^2\))
\( I_{xg} \) = Moment of inertia of fully effective (unreduced) panel per unit width, in.\(^4\)/ft (m\(^4\)/m)
\( K \) = Effective length factor
\( L_v \) = Span of panel between supports with fasteners
\( \lambda_c \) = Slenderness factor

**User Note:**
Compression in a panel rarely controls. K can conservatively be set as 1. If a concern over shear lag exists, rational design might limit column resistance to the corrugations on either side of the end support connection – e.g., if connection spacing is three corrugations, base \( A_e \) on two corrugations and adjust \( P_n \) to per unit width consistent with the required compressive axial strength [compressive force due to factored loads].

### 2.3.1 Combined Compressive Axial Load and Bending in Panel

For ASD
\[ \frac{\Omega_c P}{P_n} + \frac{\Omega_b M_x}{M_n} \leq 1.0 \]  
(Particular application of AISI S100 Eq. H1.2-1)

For LRFD or LSD
\[ \frac{\bar{P}}{\phi_c P_n} + \frac{\bar{M}_x}{\phi_b M_n} \leq 1.0 \]  
(Particular application of AISI S100 Eq. H1.2-1)

where
\( P \) = Required compressive axial strength per unit width for ASD
\[ P = \text{Required compressive axial strength} \text{ [compression force due to factored loads]} \text{ per unit width for LRFD and LSD} \]
\[ = w_u, \text{kip/ft (kN/m)} \]
\[ M_n = \text{Nominal flexural strength} \text{ [moment resistance]} \text{ of deck or panel per unit width, in. kip/ft (kN m/m)} \]
\[ = S_x F_y \quad \text{(AISI S100 Eq. F3.1-1)} \]
\[ S_x = \text{Effective section modulus of panel at stress } F_y, \text{ in.}^3/\text{ft (m}^3/\text{m)} \]
\[ M_x = \text{Required flexural strength per unit width in ASD} \]
\[ = y_b P, \text{ in. kip/ft (kN m/m)} \quad \text{(Eq. 2.3.1-1)} \]
\[ y_b = \text{Distance from panel neutral axis to bottom flat connection, in. (m)} \]
\[ M_x = \text{Required flexural strength} \text{ [moment due to factored loads] per unit width in LRFD and LSD} \]
\[ = y_b P, \text{ in. kip/ft (kN m/m)} \quad \text{(Eq. 2.3.1-2)} \]
\[ \Omega_c = 1.80 \quad \text{ASD} \]
\[ \phi_c = 0.85 \quad \text{LRFD or LSD} \]
\[ = 0.80 \quad \text{LSD} \]
\[ \Omega_b = 1.67 \quad \text{ASD} \]
\[ \phi_b = 0.90 \quad \text{LRFD or LSD} \]

Other parameters are defined in Section 2.2.

**User Note:**
Other loads simultaneously can cause other bending moment, M_x or \( \overline{M_x} \), in the panel.

### 2.4 Axial Tension Design in Panel

#### 2.4.1 Combined Tensile Axial Load and Bending in Panel

**User Note:**
Where \( w_u \) or \( w_u \) can be significantly different than the compressive value, this limiting condition should be investigated for the panel. \( S_{nf} \) is determined using Eq. 2.2-1 and the greater value of \( w_u \) or \( w_u \). Other loads simultaneously can cause other bending moment in the panel.

Design is an application of AISI S100 Section D2 and Section H1.

For ASD
\[ \frac{\Omega_b M_x}{T_n} + \frac{\Omega M_n}{T_n} \leq 1.0 \quad \text{(Particular application of AISI Eq. H1.1-1)} \]

For LRFD or LSD
\[ \frac{T}{\phi T_n} + \frac{\overline{M_x}}{\phi_b M_n} \leq 1.0 \quad \text{(Particular application of AISI Eq. H1.1-1)} \]

where
\[ T_n = \text{Nominal tensile axial strength} \text{ [resistance]} \text{ of panel per unit width, kip/ft (kN/m)} \]
\[ = A_g F_y \quad \text{(Eq. 2.4-1)} \]
\[ T = \text{Required tensile axial strength} \text{ per unit width for ASD} \]

---

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\( w_{a}, \text{kip/ft (kN/m)} \)

\( \bar{T} = \text{Required tensile axial strength [tension force due to factored loads] per unit width for LRFD and LSD} \)

\( = w_u, \text{lb/ft (kN/m)} \)

\( M_x = \text{Required flexural strength per unit width in ASD} \)

\( = y_b T, \text{in. kip/ft (kN m/m)} \)  \hspace{1cm} (Eq. 2.4-2)

\( \bar{M}_x = \text{Required flexural strength [moment due to factored loads] per unit width in LRFD and LSD} \)

\( = y_b \bar{T}, \text{in. kip/ft (kN m/m)} \)  \hspace{1cm} (Eq. 2.4-3)

\( \Omega_t = 1.67 \text{ ASD} \)

\( \phi_t = 0.90 \text{ LRFD or LSD} \)

Other parameters are defined in Sections 2.3 and 2.3.1.
Commentary on the North American Standard for the Design of Profiled Steel Diaphragm Panels

2020 EDITION
The material contained herein has been developed by the American Iron and Steel Institute (AISI) Committee on Specifications. The Committee has made a diligent effort to present accurate, reliable, and useful information on cold-formed steel diaphragm design. The Committee acknowledges and is grateful for the contributions of the numerous researchers, engineers, and others who have contributed to the body of knowledge on the subject. Specific references are included in the Commentary on the Standard.

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

The materials set forth herein are for general information only. They are not a substitute for competent professional advice. Application of this information to a specific project should be reviewed by a registered professional engineer. Indeed, in most 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 resulting liability arising therefrom.
**PREFACE**


The purpose of the Commentary includes: (a) to provide a record of the reasoning behind, and justification for the various provisions of the Standard by cross-referencing the published supporting research data, and by discussing the current edition of the Standard; (b) to offer a brief but coherent presentation of the characteristics and performance of cold-formed steel diaphragms to structural engineers and other interested individuals; (c) to furnish the background material for a study of cold-formed steel diaphragm design methods to educators and students; and (d) to provide the needed information to those who will be responsible for future revisions of the Standard. Users are encouraged to refer to the original research publications for further information.

Consistent with the Standard, the Commentary contains a main document, Chapters A through E, and Appendices 1 and 2.

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.

American Iron and Steel Institute
November 2020
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INTRODUCTION

Cold-formed panels have been used successfully in diaphragms. These panels have fluted profiles and are cold-formed from steel sheet in roll-forming machines or by press brake or bending. Deck profiles may be connected to other deck profiles or flat bottom plates to form cellular decks in the manufacturing plant, and then the cellular decks are shipped as assembled units. The thickness of steel sheets used in fluted panels historically range between 0.014 in. (0.35 mm) and 0.105 in. (2.67 mm). Cellular decks are usually formed from thicker sheet steel because of fabrication requirements at longitudinal connections and web compactness requirements at deeper sections. The steel sheets can be perforated for acoustic, lighting, airflow or other serviceability purposes. The panels are generally in flat planes but may also be curved in the shop or the field to form arches or shell structures with bending along the panel length or across the width. This diaphragm Standard only addresses design and testing of plane diaphragm systems.

The use of steel panel diaphragms has several economic advantages and can reduce the required materials and labor. The diaphragm system is usually considered as a primary structural member that provides lateral resistance and stability to a building system while the panels simultaneously provide other serviceability functions. The functions include exposed weather-tight membranes (cladding); underlayment (decking) for other roofing membranes and insulating systems; concrete forms; permanent reinforcement in structural concrete slabs; secondary flexural structural members in floors, roofs, or walls; and bracing of primary structural members. The panels can also replace or supplement permanent diagonal bracing or other bracing systems (Luttrell, 1967).

Industry sponsored much of the original testing of diaphragms (Fenestra, Inc., Granco Steel Products Co., H. H. Robertson, R.C. Mahon, Inc., etc.). The testing was performed at or witnessed by independent laboratories, and the focus was to develop load tables to assist designers and market products. This work was proprietary and often empirical. Industry testing has continued in order to obtain product evaluation reports. The American Iron and Steel Institute (AISI) has sponsored research in this field since the 1950s. Some of the earliest work was at Cornell University (Nilson, A. H., 1956). AISI-sponsored work continued into the 1960s and 1970s under the direction of Dr. George Winter at Cornell University (Luttrell, 1967). Diaphragm applications have history and there is an established and extensive test database (SDI, DDM, 1981, etc.).

Two design manuals were developed for industry and end users, and these manuals have evolved into the primary design and analytical references for designers in North America. These manuals are: (1) Department of Army (1982¹), Seismic Design for Buildings (commonly called the Tri-Services Manual), based on the work of S. B. Barnes and Associates, John A. Blume and Associates, and Structural Engineers Association of California, first published in 1966; and (2) Steel Deck Institute Diaphragm Design Manual (SDI, 2004), based on the work of Dr. L. Luttrell and first published in 1981. Both manuals address flat planar diaphragm construction. The limits of design application are established by the tests.

Because these design manuals are not consensus documents, industry petitioned AISI to

¹ The 1982 edition was referenced due to errors in deck design that were found in the 1992 edition.
develop a consensus standard. The first edition of the North American Standard for the Design of Profiled Steel Diaphragm Panels (AISI S310-13) was prepared and issued in 2014. Whenever possible, this document is consistent with the edition of AISI S100 and AISI Test Standards referenced in Standard Section A4. Provisions outside of the scope of AISI S100 are based on the available research reports. AISI S310 establishes design analytical methods and minimum testing requirements. The first edition of the Commentary on the North American Standard for the Design of Profiled Steel Diaphragm Panels was prepared and issued in 2014.

The Standard and Commentary are intended for use by design professionals with demonstrated engineering competence in their fields.

A. GENERAL PROVISIONS

A1 Scope, Applicability, and Definitions

A1.1 Scope

Diaphragms are roof, floor or other membranes or bracing systems that transfer in-plane forces to the lateral force-resisting systems. A wall diaphragm can be a lateral force-resisting system. The Standard provides design provisions for components consisting of panels with fluted or cellular deck profiles. The diaphragm or wall diaphragm system includes other components and details that are not explicitly covered in the Standard. Wall diaphragm panels are in a vertical or nearly vertical plane and often are the lateral force-resisting system that transfers forces to foundations. Vertical and horizontal diaphragms have nuanced differences and similarities:

(a) The planar performance is the same but the span-to-depth ratios commonly function over different ranges. Both diaphragms resist the same load events.

(b) The wall dead load component is one more load that might cause diaphragm shear or interact with shear. (The wall diaphragm normally has a rigid base so wall dead load is compressive and not always additive to shear in the panel or support connections except as a seismic inertial force.)

(c) Column action in panels is treated the same for all diaphragms as in Appendix 2.

(d) The impact of other system components and details is a design issue that must be considered. Three examples are: (1) purlin or girt roll that inhibits transference of shear to frames, (2) perimeter details that transfer shear from panels to lateral force-resisting systems or across panel discontinuities, and (3) ties in structural members that are necessary to collect and transfer axial forces to lateral force-resisting systems.

(e) With proper connection details, the interaction of a wall diaphragm with a roof diaphragm is to unload a roof diaphragm’s in-plane shear at interior lateral force-resisting system lines, or to resist shear at end shear walls. Shear unloads proportionately to the relative stiffness of the diaphragm and lateral force-resisting system at those lines and is associated with deflection compatibility. This wall diaphragm action is similar to a moment frame’s flexural stiffness contribution to resisting a roof diaphragm’s in-plane loads.

(f) A wall diaphragm may be subject to additional requirements in some applicable building codes, particularly when the wall diaphragm is required for energy dissipation due to seismic load.

(g) AISI S907 is commonly used to evaluate all diaphragm systems.

(h) Standard Table B1.1-1 applies to walls, roofs, and floors when concrete fill is not present and supports are steel.
The *Standard* is not intended for cold-formed steel framing shear *diaphragms* covered with sheathing other than fluted *panels* or *cellular deck*. Such *diaphragms* should be designed in accordance with AISI S240.

Two design approaches are included in the *Standard*:

1. The analytical approach as provided in *Standard* Chapter D, and
2. The testing approach as provided in *Standard* Chapter E.

The analytical approach adopts the method presented in the *Diaphragm Design Manual* (Steel Deck Institute, 2004). The method is mechanics-based, confirmed by tests, and includes variations in steel *panel* properties and in *connection* types. Because the analytical method is confirmed by tests, the application limits are established by the tests. The *connection nominal strength* [resistance] is determined in accordance with AISI S100 wherever possible. The analytical method adopts the supplemental research sponsored by the Metal Construction Association (MCA) (Luttrell, 1999a) that included insulation between supports and *panels*. The MCA research also addressed wood supports. The testing approach is based on AISI S907, *Test Standard for Determining the Strength and Stiffness of Cold-Formed Steel Diaphragms by the Cantilever Test Method* and AISI S905, *Test Standard for Determining the Strength and Deformation Characteristics of Cold-Formed Steel Connections*. Both AISI test standards adopt the calibration methods presented in the *Standard* and AISI S100 Section K2.1.1 with modifications applicable to *diaphragms*.

The *Standard* limits itself to the determination of *diaphragm available shear strength* [factored resistance] and *stiffness*. However, performance also depends on adherence to design documents and quality control during installation. Guidance in this area is available in the following three references:

Steel Deck Institute (SDI), P.O. Box 426, Glenshaw PA 15116
3. *SDI Code of Standard Practice* (COSP), June 2017

### A1.2 Applicability

The *Standard* is applicable to *diaphragms*:

- (a) With or without insulation between the *panel* and the support,
- (b) Without insulation between the *cellular deck* and the support in *Standard* Chapter D,
- (c) With insulation between the *cellular deck* and the support in *Standard* Chapter E,
- (d) With or without *structural* or *insulating concrete* fill over the *deck* or *cellular deck*,
- (e) With or without *acoustic* (perforated) *panels* or *cellular deck*, and
- (f) With structural supports made of steel, wood, or *structural concrete*.

This does not preclude other support materials whose performance can be verified by tests or using material design specifications that provide *connection* reliability consistent with the *Standard*. Selection of alternate materials should consider other *serviceability limit state* requirements such as dissimilar materials (corrosion) and fire resistance.

### A1.3 Definitions

Definitions for certain commonly used terminologies in *diaphragm* systems are provided
The Standard also refers to the definitions provided in AISI S100 for strength-related terminologies. To apply the Standard, the definition of “diaphragm” should be in accordance with the Standard. Where possible, the definitions in the Standard are consistent with the test standards, AISI S905 and AISI S907.

A2 Materials

The materials of profiled steel panels should conform to the materials specified in the Standard. For low-ductility steels, the specified yield stress, \( F_y \), and specified tensile strength, \( F_u \), should be modified for design in accordance with AISI S100, Sections A3.1.2 or A3.1.3 for steels not conforming to Section A3.1.1, unless noted otherwise.

Other materials must conform to the national standards governing the design of that material.

A3 Loads

The required strength [effect of factored loads] in a diaphragm system should be calculated using the load combinations provided in the applicable building code. In the absence of an applicable building code, ASCE 7 should be used for ASD and LRFD. Refer to AISI S100 for appropriate building codes for LSD.

A4 Referenced Documents

In addition to the standards referenced in Standard Section A4, the following documents may be considered in a calculation-based design approach:

(a) American Iron and Steel Institute (AISI), 25 Massachusetts Avenue, NW, Suite 800, Washington, DC 20001:
   Design Examples for the Design of Profiled Steel Diaphragm Panels Based on AISI S310-17, 2017 Edition

In addition to the standards referenced in Standard Section A4, the following documents may be considered in a test-based design approach:

(a) Department of the Army Seismic Design For Buildings:

(b) Metal Construction Association (MCA), 4700 W. Lake Ave., Glenview, IL 60025
   A Primer on Diaphragm Design, 2004 Edition

(c) Steel Deck Institute (SDI), P.O. Box 426, Glenshaw PA 15116:
   Diaphragm Design Manual, First Edition (DDM01, 1981), and
   Deeper Steel Deck and Cellular Diaphragms, 2005 Edition with Supplement 2013

These references may be used in conjunction with Standard Section E1.2 in determining system effects and application limits of existing test-based methods.

Other references are listed at the end of this Commentary.

A5 Units of Symbols and Terms

The equations provided in the Standard are intended for design in any compatible system of
units (U.S. Customary, SI or metric and MKS systems). Equations for U. S. Customary and SI or metric unit systems plus the required unit(s) are provided if the Standard equation is not compatible with the unit systems. A conversion table between the unit systems is provided in Section A1.4 of the Commentary on AISI S100.
B SAFETY FACTORS AND RESISTANCEFACTORS

Table B-1 lists the Standard sections that provide the safety and resistance factors for both calculation and test-based design approaches. A dead to live load ratio of zero is used in the development of Table B-1 and in the calibration method of Standard Chapter E. Because of the nearly horizontal condition in floors and roofs, shear due to vertical load is perpendicular to the load effects causing diaphragm shear. In walls, the vertical load normally induces compression in the panels and little shear into the connections. As such, vertical load shear rarely adds to the in-plane shear stresses in the panels or the shear in the connections of diaphragms or wall diaphragms. Connections commonly control diaphragm resistance. The assumption of D/L = 0.0 is slightly more severe than the assumption of D/L = 0.2 for LRFD or D/L = 0.33 for LSD in AISI S100 Section K2.1.1. Since the dead load of the diaphragm was present in the tests for confirmation of the analytical method, inclusion of dead load is partly built into the calibration.

B1 Safety Factors and Resistance Factors of Diaphragm With Steel Supports

The safety and resistance factors provided in the 2013 edition of Standard Section B1 were extracted from the 2012 edition of AISI S100 Section D5. In 2016, those safety and resistance factors were moved to this Standard, updated and included in Standard Table B1.1-1. Refer to Section B1.1 for background information. The Standard Table B1.1-1 safety and resistance factors determine the diaphragm available strength [factored resistance] where the nominal strength [resistance] is determined in accordance with Standard Chapter D. The safety and resistance factors determined in accordance with Standard Chapter E are limited by those in Standard Table B1.1-1 unless noted otherwise in the Standard. The table distinguishes material-related limits and connection-related limits. The table also distinguishes different loads that cause the diaphragm shear.

B1.1 Floor, Roof, or Wall Steel Diaphragm Construction

The structural performance of a diaphragm can be evaluated by either calculations or tests. Several analytical procedures exist, and are summarized in the Commentary Sections A4 and E1.2. Analytical methods consider the limit states of the connections between the panels, and between the panels and structural supports, as well as the support thickness and its mechanical properties. As an example, the tilting of screws discussed in AISI S100 Section J4.3 is different from the bearing capacity controlled by panels. The analytical methods will then determine the capacity of the diaphragm based on the limit states. Yu and LaBoube (2010) provide a general discussion of structural diaphragm behavior.

AISI S907 provides the test procedures and commentary for cold-formed steel diaphragms. Historically, tested performance was measured using the procedures of ASTM E455, Standard Method for Static Load Testing of Framed Floor, Roof and Wall Diaphragm Construction for Buildings, and those results are valid. Future researchers using ASTM E455 should match the minimum number of panels required by AISI S907 over the shear field unless that number is not representative of the design application.

Table D5 in AISI S100-12 was based on a calibration of test data in DDM01 (SDI, 1981). The test data in DDM01 includes welds, screws, some proprietary power actuated fasteners and the interaction of all support and sidelap connections (i.e. system effects). The DDM01 statistical values are:
The SDI calibration was based on the same theory as the *Standard* but the test-based connection strengths differ from those in AISI S100 and in the *Standard*. The safety factors and resistance factors listed in *Standard* Table B1.1-1 are obtained through a recalibration of an expanded database of full-scale diaphragm tests. The weld test data now includes SDI (1981), Ellifritt (1970), and Bagwell (2008); and the screw test data includes: SDI (1981), Luttrell (1967), MCA (1999), and Bagwell (2008). The recalibration used the method of AISI S100-C Section B3.2.2 and AISI S100 Section K2.1.1 and the load factors in ASCE 7-10. AISI S100-C Section B3.2.2 only addresses the dead plus live load combination, which leads to the suggested factors of $C_\phi = 1.52$ and $V_Q = 0.21$ for members and individual connections – See the AISI S100-C Section B3.2.2 for background information. It is common for wind or seismic events to control the required diaphragm strength [effect of factored loads]. The recalibration is also consistent with the connection strength equations in *Standard* Section D1 and the provisions of *Standard* Section E1.2.2. The statistical factors in the recalibration are:

<table>
<thead>
<tr>
<th>Connection Type</th>
<th>$n$</th>
<th>$P_m$</th>
<th>$V_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welds</td>
<td>107</td>
<td>1.151</td>
<td>0.217</td>
</tr>
<tr>
<td>Screws</td>
<td>29</td>
<td>1.038</td>
<td>0.128</td>
</tr>
</tbody>
</table>

The most probable diaphragm D/L load ratio is zero. This results in the values given in *Standard* Section E1.2.2, $C_\phi = 1.60$ and $V_Q = 0.25$. The dominant diaphragm limit state is connection-related. Consistent with AISI S100 and explained in AISI S100-C Section B3.3.2(b), in the United States and Mexico, the calibration used $\beta_o = 3.5$ for all load effects except for wind load and allowed $\beta_o = 2.5$ for connections subjected to wind loads; in Canada, the recalibration used $\beta_o = 4.0$ for all load effects except wind load and used $\beta_o = 3.0$ for connections subjected to wind loads. These reliability indices are required by AISI S100 for individual connections with the exception of $\beta_o = 3.0$ in LSD, which is for a member. These individual reliability indices are applied in the diaphragm tests where all the connections are loaded. This approach is historical and conservative because of the repetition of connections and potential load redistribution in the diaphragm system. Based on the expanded test database and the connection strengths [resistances] given in this *Standard*, the safety factors and resistance factors listed in *Standard* Table B1.1-1 changed relative to those in the 2013 edition of AISI S310. The increased number of screw diaphragm tests shows greater reliability relative to welds in *Standard* Table B1.1-1. Using $\beta_o = 2.5$ for screw calibration suggests that the safety and resistance factors for connections are less severe than the safety and resistance factors for panel buckling, $\phi = 0.8$ and $\Omega = 2.0$. However, connection-controlled diaphragm safety and resistance factors are not permitted to be less severe than member-controlled diaphragms (panel buckling).
in Standard Table B1.1-1. $\phi_d = 0.70$ and $\Omega_d = 2.35$, that were included in Table D5 of AISI S100-12 for wind, are eliminated in Standard Table B1.1-1 for connection-controlled diaphragms. These safety and resistance factors were intended to keep the service load close to the test point, $0.4P_{\text{ultimate}}$, for stiffness determination. Since the stiffness is relatively constant in this service load zone, the recalibration allowed the method of Standard Section E1.2.2 to stand alone.

The calibration of resistance to seismic loads is based on a load factor of 1.6, which might be more conservative than required by ASCE 7. The impact of seismic events on $V_Q$ requires further study. No distinction is made between seismic load and other loads in the recalibration. This differs from earlier editions of the Standard and AISI S100 where safety and resistance factors for welding varied. However, with the exception of LSD, no change occurred for seismic load at welds relative to AISI S100-12 Table D5. When the load factor for earthquake loading is one, the 0.7 multiplier of ASCE 7 is allowed in ASD and the safety factors of Standard Table B1.1-1 should be applicable. If a local building code requires that a load factor of 1.6 be applied to the seismic load, the factors of Standard Table B1.1-1 should still be applicable.

The Steel Deck Institute (1987) and the Department of Army (1985) have consistently recommended a safety factor of 2.0 to limit “out-of-plane buckling” of diaphragms. Out-of-plane buckling is related to panel profile, while the other diaphragm limit state is connection-related. The remainder of the Standard and AISI S100 require different safety and resistance factors for the two limit states and larger safety factors for connection-controlled limit. The prescribed factors for out-of-plane panel buckling are constants for all loading types.

The Standard allows mechanical fasteners other than screws to be used in accordance with Standard Section D1.1.5 or D1.2.7, as applicable. The diaphragm shear value using any fastener must not be based on a safety factor less than the safety factor of individual fastener shear strength or a resistance factor greater than the resistance factor of individual fastener shear strength unless: 1) sufficient data exists to establish a system effect, 2) an analytical method is established from the tests, and 3) test limits are stated. The system effect is established through large-scale tests in accordance with AISI S907.
C. DIAPHRAGM AND WALL DIAPHRAGM DESIGN

*Standard* Chapter C is used to determine the available shear strength [factored resistance] per unit length and stiffness or flexibility of panels as diaphragm system components, and to compare these values with the required shear strength [shear force due to factored loads] and required stiffness. This *Standard* includes design of panel and support connections. Since the diaphragm resistance also includes the ability to transfer loads into the diaphragm and transfer loads out at either shear walls or at braced bents or rigid frames, this *Standard* includes those connections and details. The design of components and details not included in this *Standard* should be based on other standards. The following are the latest editions of the most commonly used standards:

(a) Hot Rolled Steel Structural Members: ANSI/AISC 360
(b) Cold-Formed Steel Structural Members: AISI S100
(c) Structural Concrete Members: ACI 318
(d) Wood Structural Members: ANSI/AWC NDS

Design must consider serviceability limits such as acceptable drift or the appearance of buckled shapes at service loads in some products. Local buckling or oil canning is common in cold-formed steel products and post-buckling strength is counted on in design. Such buckling or waves are not always a serviceability or cosmetic issue.

C1 General

The diaphragm and wall diaphragm systems transfer the forces through the chords and collectors to the panels that act as a stressed shear transfer skin. The stressed skin is attached to the panel’s supports in the field and at the edges of the diaphragm or wall diaphragm system. The *Standard* provides design equations that can be used to determine the in-plane shear resistance of the panels of the diaphragm and wall diaphragm, and the shear resistance of connections between panels and between panels and supports.

Wall diaphragm (shear wall) applications subject to seismic loading are dependent on the seismic system coefficients for the wall system. ASCE 7 does not provide specific requirements for the use of fluted sheet steel panels for wall diaphragm construction. Where the fluted sheet steel wall diaphragm’s seismic design coefficients are not otherwise recognized by the building official, the wall diaphragm may be classified as a light-framed wall with shear resisting panels under the all other materials provison in accordance with ASCE 7 Table 12.14-1 Item A15, which specifies a response modification factor, \( R = 2 \). This is lower than the response modification coefficient specified at Item A16 for light-framed (cold-formed steel) wall systems using flat strap bracing, \( R = 4 \), which is an inherently less ductile system than steel sheets in wall diaphragms (shear walls). Fluted sheet steel panels have characteristics similar to flat sheet panels because the nominal strength [resistance] and ductility are associated with the connections at the support framing and not the shape of the panel or the support framing, particularly when each flute is attached to the supports. Stojadinovic and Tipping (2008) demonstrated this through cyclic tests of corrugated sheet steel panels attached to cold-formed steel framing with self-drilling screws. The study found that following ATC 63, the seismic system coefficient, \( R \), would be in the range of 3 to 4. However, much higher coefficients are recommended based on the 90% draft of FEMA P-795. Stojadinovic and Tipping recommend the following seismic system coefficients: (a) Response Modification Factor, \( R = 5.5 \), (b) System Overstrength Factor, \( \Omega_o = 2.5 \), and (c) Deflection Amplification Factor, \( C_d = 3.25 \).
C2 Strength Design

A diaphragm system can be designed using ASD or LRFD in the U. S. and Mexico, and LSD in Canada. Information regarding these design methods can be found in Section B3.3 of the Commentary on AISI S100. Information regarding loads and load combinations can be found in Section B2 of the Commentary on AISI S100. For ASD, the Standard requires that the diaphragm panel system safety factor be applied to the diaphragm’s nominal shear strength [resistance] per unit length in Eq. C2-1. Since the diaphragm nominal shear strength [resistance] depends on the support and sidelap connection nominal shear strengths [resistance], most Standard sections only present the connection nominal shear strength [resistance]. The exception is the shear and uplift interaction that allows determination of the nominal connection shear strength [resistance] using either ASD, LRFD, or LSD since interaction is related to the required strength [force due to factored loads] in tension.

C3 Deflection Requirements

A diaphragm should be stiff enough so that \( \delta_n \) is less than or equal to \( \delta_a \). Deflection, \( \delta_{tr} \), is the total (shear plus flexural) deflection component of the diaphragm. Deflection can be determined by structural analysis, and several design examples are shown in the supplemental references listed in the Commentary in Section A4. For many diaphragm designs, the contribution of longitudinal strain in the perimeter members and the flexural deflection of the diaphragm are negligible when compared to the contribution of shear deflection, particularly when the accuracy of predicting diaphragm stiffness, \( G' \), is considered. This simplifies analysis but it is always permissible to consider both deflection components.

where
\[
\delta_n = \text{Theoretical diaphragm deflection at service load or nominal loads [specified loads]}
\]
\[
\delta_a = \text{Allowable diaphragm deflection defined by the applicable building code and the structure’s service requirements}
\]

The change in shear deflection between two points along the diaphragm span can be determined by calculating the area under the \[ \frac{V}{G'B} \] diagram between those points. However, this principle should be applied with caution when \( G' \) varies along the diaphragm span, \( L_d \) (\( L_d \) is shown in Figure C-C3-1). When \( G' \) varies, an energy method should be used to determine the in-plane deflection.

Figure C-C3-1 shows potential diaphragm deflection for a simple rectangular diaphragm under symmetric loading. This is idealized, but is a common event. Deflection can include longitudinal deformation of support members plus racking and twisting of end walls and interior frames. This example has braced end walls and sidewalls, so the shear walls are relatively rigid in plane when compared to the diaphragm stiffness. In this example, since the interior frames provide minimal deflection resistance, they do not unload the diaphragm and reduce diaphragm deflection and shear. A diaphragm can be considered as a deep beam in which shear deformation is dominant since the diaphragm has a small span-to-depth ratio, \( \frac{L_d}{B} \). Usually diaphragm stiffness, \( G' \), is significantly less than the flexural stiffness of the deep beam for the structure and the flexural component is often neglected in design. Some applicable building codes require both the diaphragm \( \frac{L_d}{B} \) ratio to be limited to a certain value, and lateral force-
resisting system stiffness to be considered when the diaphragm stiffness, $G'$, is within a certain range. The design engineer should satisfy these code requirements.

Where a diaphragm is not stiff relative to the lateral force-resisting system stiffness, the reaction line support approaches a rigid support and the diaphragm acts as a simple beam between rigid supports.

Where the interior frames are moment frames and the connection detail allows development of in-plane deflection resistance or the end wall is not relatively rigid, lateral force-resisting system deflections will affect load sharing and theoretical diaphragm deflection, $\delta_n$. Design is commonly based on deflection compatibility between shear walls, frames and diaphragms. A stiffness analysis might be required to determine the shear distribution to each frame or end wall.
The *diaphragm* deflection is a function of the primary parameters as shown below:

\[ \delta_n = \delta (V_m, B, L_d, \frac{1}{G'}, F, W_d) \]  

(C-C3-1)

where

- \( V_m \) = Maximum shear force, \( V \), delivered by the *diaphragm*
- \( B \) = *Diaphragm* depth (length parallel with shear force)
- \( L_d \) = *Diaphragm* span between shear walls or reaction lines
- \( G' \) = *Diaphragm* stiffness
- \( F \) = *Diaphragm* flexibility
- \( W_d \) = Load and load distribution causing \( V_m \)
- \( \delta \) = Deflection function symbol

Figure C-C3-1 provides one example of \( W_d \) where equal loads, \( P \), are being applied in line with the frames. This might happen where girts deliver wind loads to frame columns. The resultant shear per unit length in the roof *diaphragm panel* is: \( \frac{V_m}{B} = \frac{1.5P}{B} \) while the end wall resists \( \frac{2P}{B} \). \( \frac{V_m}{B} \) is the *required shear* per unit length in the *diaphragm edge panel*. 

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D. DIAPHRAGM NOMINAL SHEAR STRENGTH PER UNIT LENGTH AND STIFFNESS DETERMINED BY CALCULATION

The Standard section is applicable to fluted panels or deck with depth equal to or less than 7.5 in. (191 mm). The stated limits reflect the research by Luttrell (SDI, 1981) and Luttrell (1999a), and Bagwell and Easterling (2008). The thickness limit of 0.075 in. (1.91 mm) reflects industry practice and the total thickness tested by Bagwell and Easterling. This is a slight increase relative to the 0.064 in. (1.63 mm) limit reported in SDI DDM03 (2004).

Standard Chapter D provides design provisions to calculate the diaphragm nominal shear strength [resistance], stiffness and flexibility. The diaphragm nominal strength [resistance] calculated in accordance with Standard Chapter D is the minimum of the three following limit states of nominal shear strength [resistance] per unit length:

1. Connection nominal strengths [resistance] at the interior and exterior supports in each panel, and the corner connection nominal strength [resistance] in each panel (addressed in Standard Section D1);
2. Diaphragm out-of-plane buckling strength (addressed in Standard Section D2); and
3. Diaphragm support connection nominal strength [resistance] at edge panels over shear walls, reaction line frames, or collection struts (addressed in Standard Section D1).

Connection-controlled diaphragm nominal shear strength [resistance] is due to local failure at connections either by bearing of the panel against the fastener, by bearing or pull-out at supports, or by shear failure in the fastener body while the profile remains relatively intact. Redistribution is normally present in connection forces until system ultimate load occurs. The panel often performs elastically and reclaim its original shape upon unloading after connection failure.

The diaphragm nominal shear strength [resistance] due to shear buckling is controlled by:

(a) Total out-of-plane panel buckling with limited connection failure, or
(b) Significant development of panel waves and tension field action with redistribution of connection forces and subsequent connection or fastener failures.

The analytical approach outlined in Standard Chapter D is based on the fourth edition of the Diaphragm Design Manual (SDI DDM04, 2015) and is virtually the same as that of SDI DDM01 (1981). Design examples are provided in SDI DDM04 (2015) and AISI Design Guide (2017). The method includes the additive and independent contribution of support connections and sidelap connections as the system effect. The contribution of support connections varies linearly with distance from the center of the panel.

D1 Diaphragm Shear Strength per Unit Length Controlled by Connection Strength, S_{nf}

The diaphragm nominal shear strength [resistance] per unit length controlled by connection nominal strengths [resistance] in each panel includes fastener failure or connection failure in the panel. Standard Eq. D1-1 includes a ($\lambda$-1) relaxation term, which represents edge of panel corner buckling at support connections along sidelaps at panel ends. That corner connection cannot develop its full nominal strength [resistance]. This relaxation occurs at the compression corners as the panel racks in-plane but the reduction is applied in both directions (tension and compression) for simplicity. Standard Eq. D1-2 recognizes the orthogonal force components and greater demand at the corner connections in each panel as illustrated in Standard Appendix 2 Figure 2.2-1, which is showing a particular panel at the diaphragm edge and not the general case at interior panels (in the general case, $w_a$ or $w_u$ are not present).
The diaphragm nominal shear strength [resistance] controlled by connections at edge panels includes fastener failure or local panel failure at fasteners along lines where the shear is transmitted from the diaphragm to the lateral force-resisting system (shear walls, braced bents, or moment frames). These connections are spaced parallel to the panel, and typically edge supports are required in the shear transfer plane to allow installation of these edge connections. Standard Eq. D1-3 addresses this diaphragm strength limit, and the required strength [reaction due to factored loads] at frames is compared with this available shear strength [factored resistance]. It should be noted that the reaction at an interior support usually does not equal the shear in the diaphragm panel at the point of transfer, as beam reactions do not equal shear at interior reactions.

Standard Eq. D1-3, \( S_{\text{ne}} \), includes the contribution of all edge fasteners, \( n_e \), and the support connections between the panel centerline and the reaction line in the edge panel. Standard Eq. D1-3 is based on a symmetric connection pattern at each support in an edge panel, while the patterns can vary between interior and exterior supports. It is acceptable to only consider \( n_e \) and the support connections at the edge by letting \( \alpha_1 = 1 \) and \( \alpha_2 = 1 \) in Standard Eq. D1-3. Since adding connections at the edge parallel with the panel span normally does not significantly impact installation time, many designers will not let \( S_{\text{ne}} \) control diaphragm capacity. Common practice requires \( n_e \) to equal or exceed \( n_s \).

The diaphragm nominal shear strength [resistance] controlled by connections along the edge perpendicular to the panel span includes fastener failure or local panel failure at exterior supports. Attachment pattern has a significant influence on this diaphragm strength limit which is addressed by Standard Eq. D1-4. This theory was presented in a report by Nunna (2018a).

\( S_{\text{np}} \) is the smallest value of bottom flute fastener(s) shear strength per tributary width among all the bottom flute support connections. Some examples of the controlling combination for \( w_t \) and \( n_d \) used in Standard Eq. D1-4 are shown in Figure C-D1-1. For fastener patterns with a support fastener in every bottom flute, \( w_t \) is equal to the pitch of the deck. For cellular deck, the average number of support fasteners per unit width is used to calculate \( S_{\text{np}} \).
Figure C-D1-1 Tributary Width and Number of Fasteners for Various Attachment Patterns

Standard Figure D1-1 assumes that a common support connection occurs at the sidelap and that the panel ends are lapped. In Standard Figure D1-1, $N =$ the connections/unit width. For a cover width, $w = 3 \text{ ft} (0.914 \text{ m}), A = 2$ and $N = 5/3 \ (1/\text{ft}) \ (5.47 \ (1/\text{m}))$ at the top exterior support, while $A = 1$ and $N = 4/3 \ (1/\text{ft}) \ (4.38 \ (1/\text{m}))$ at the bottom exterior support. If a fastener is installed at either side of the sidelap (See Figure C-D1-2), then with everything else being equal, $N$ at the top is $7/3 \ (1/\text{ft}) \ (7.64 \ (1/\text{m}))$ and $N$ at the bottom is $5/3 \ (1/\text{ft}) \ (5.47 \ (1/\text{m})).$ In either case, when determining $\alpha_e$ and $\alpha_{e2},$ there are 7 values of $x_e$ at the top and 5 values of $x_e$ at the bottom.

In the Standard Figure D1-1, the number of sidelap connections, $n_s,$ is 6 and is distributed over

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the entire panel length, L. Note that \( n_s \) is not the number of sidelap connections per span, \( L_v \). The number of sidelap connections, \( n_e \), at the edge (lateral force-resisting system line in the schematic) is 9. The schematic in Standard Figure D1-1 conforms to common practice, \( n_e \geq n_s \).

The terms, exterior support and interior support, are relative to each panel and not descriptive of the location within the structure.

A support connection should be installed at either side of the sidelap when the panel design does not allow a single support connection to engage both sections of panel at the sidelap while developing full nominal strength [resistance]. Figure C-D1-2 shows examples of interlocking sidelap connection over an interior support; one option requires two support connections while the other may allow one. The critical edge dimension, \( d_e \), is shown for the second option. Figure C-D1-3 shows that \( d_e \) is also critical at nestable sidelaps. If the horizontal lip is too short, an arc spot weld might not be acceptable but an erector might use an equivalent strength fillet weld or a mechanical fastener to engage the support. The designer should determine equivalence.

It is possible to have multiple connections in each flute over any support and that effect is included in \( \alpha_1 \) and \( \alpha_2 \) in Eq. D1-3, and \( \alpha_p^2 \) and \( \alpha_e^2 \) in Eq. D1-6, respectively. The term “A” in Standard Eq. D1-1 accounts for the reduced corner connection resistance as limited by the compressive stiffness of the panel at the sidelap over the support. With multiple fasteners per flute, proper spacing and edge dimension must be maintained and the group effect might become a limit state at the fastener cluster. See Standard Section D1.1.6.

Standard Equations D1-1 through D1-3 and D1-5 through D1-12 were developed by Luttrell and first published in the SDI Diaphragm Design Manual, First Edition (SDI, 1981). They are also listed in SDI DDM0 4 (SDI, 2015). The Standard equations are based on fluted panels with the configuration illustrated in Standard Figure D2-1 and parameters shown in Standard Figure D1-1 and defined in Standard Section D1. The basic Standard equations and the mechanical model can be modified to be applicable to diaphragms with concrete fill over deck or with insulation between the panel and the support. The Standard applies such modifications in Sections D1.3 and D4. The modifications consider the potential for corner buckling and end warping, and the relative flexibilities of support connections.

### D1.1 Support Connection Shear Strength in Fluted Deck or Panels, \( P_{nf} \) and \( P_{nfs} \)

The Standard permits the nominal strength [resistance] of connections to be determined either by calculation or by tests.

Standard Sections D1.1.1 through D1.1.4 contain provisions to calculate support connection strength, \( P_{nf} \) or \( P_{nfs} \), at diaphragm connections whose strength is listed in AISI S100 or defined by the referenced research reports. Standard Section D1.1.5 contains provisions to test support connection strength, \( P_{nf} \) or \( P_{nfs} \), for unlisted fasteners into steel or wood supports. All fasteners into concrete supports should be tested in accordance with Section D1.1.5. Standard Section D1.1.6 addresses connection strength controlled by edge dimensions of panel for individual connections and at critical shear planes for connection groups.

The ductility provisions of AISI S100 Section A3.1 should be considered in Standard Chapter D. For example, to use ASTM A653 SS Grade 80 steel panels, the reduced yield stress, \( F_Y = 60 \text{ ksi} \) (415 MPa), and reduced tensile strength, \( F_U = 62 \text{ ksi} \) (430 MPa), should be used to calculate connection strength in accordance with AISI S100 Section A3.1, unless noted otherwise.

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D1.1.1 Arc Spot Welds or Arc Seam Welds on Steel Supports

The nominal strengths [resistance] of arc spot welds and arc seam welds on steel supports are extracted from AISI S100 Section J2.2.2.1 and Section J2.3.2.1. See the corresponding sections of the Commentary in AISI S100 for technical background information.

Weld washers typically are required at supports when the panel thickness is less than 0.028 in. (0.71 mm).

The thickness of supports can contribute to blowholes where panels are welded to supports. A rule of thumb is that the support should be at least 1/8 in. thick, but this does not always prevent blowholes. If blowholes caused by arc spot or arc seam welds into thin support material are a structural or cosmetic concern, fasteners should be considered as an alternative. A blowhole may become a structural concern when a significant amount of the support flange area is removed. Some blowholes should be expected at welded sidelap connections between supports.

Arc seam welds are often used in narrow flutes where it is difficult to achieve 1/2 in. (12.7 mm) diameter arc spot welds. The weld width must be sufficient to achieve adequate fusion at the support.

Welding through multiple thicknesses requires field quality control. It can be difficult to weld through multiple thicknesses of steel sheet, particularly through four layers of thickness (Snow and Easterling, 2008). However, it is possible to provide adequate welds through four thicknesses as long as the welder adheres to certain requirements. Quality depends on electrode choice (both size and type), weld settings, welding time, air gaps, ambient conditions, presence of moisture, support thickness (sometimes), and the skill of the welder (Guenfoud et al, 2010). Guenfoud reported that welding is possible if the support thickness exceeds 70% of the combined thickness(es) to be welded. Welding can be difficult when the support thickness is less than 50% of the combined thickness(es). The panel manufacturer might recommend that four-layer laps be avoided if the thickness of one element at the four-layer lap exceeds 0.06 in. (1.5 mm). In addition, nestability is a concern in thicker panels or panels with steep webs since air gaps contribute to welding difficulty.

A fastener connection should be considered if consistent welding quality is difficult to maintain. Figure C-D1.1.1-1 shows common four-thickness laps that might occur at sidelaps or in an end-lap. Minimum edge dimensions must be maintained at welds or mechanical fasteners.

D1.1.2 Screws Into Steel Supports

The design provisions of Standard Section D1.1.2 are extracted from AISI S100 Section
J4.3.1. The technical background information can be found in Section J4 of the Commentary to AISI S100. To ensure the required level of performance in structural applications, designers should specify screws that conform to ASTM C1513 or an equivalent standard.

D1.1.3 Power-Actuated Fasteners Into Steel Supports

The Standard requires that the nominal strengths [resistances] of power-actuated fasteners (PAF) be determined by tests. The tests should also set the PAF application limits. A designer can find the test-based strength and connection flexibility equations for specific power-actuated fasteners listed in SDI DDM04 (2015) and its appendices or can consult the fastener manufacturer for test data on these and other PAF fasteners. The PAF used in design should be specified and no substitution of other PAFs should be permitted unless the substituted fasteners are equivalent in strength and connection flexibility to the specified PAF. The designer should request data for replacement of proprietary PAF fasteners to substantiate the design values and conformance to the Standard.

Tilting should be considered in fastener selection and strength determination. The equations given in SDI DDM04 (2015) indicate the acceptable panel thickness and support thickness ranges. The application limits for each equation as listed in SDI DDM04 (2015) or provided by the manufacturer should be met for each particular fastener type.

D1.1.4 Fasteners Into Wood Supports

D1.1.4.1 Safety Factors and Resistance Factors

Because of uncertainty over the material factors in wood, a limiting safety and resistance factor is imposed even when the test-based calibration may indicate less severe factors. The limiting factors are consistent with the calibration in Luttrell and Mattingly (2004).

D1.1.4.2 Screw or Nail Connection Strength Through Bottom Flat and Into Support

The Standard equations for determining fastener nominal strength [resistance] into a wood support, P_{nfw}, are from AWC NDS (1986) as listed in the MCA research by Luttrell (1999a). Small-scale tests can be waived for other fasteners that are not listed in Standard Table D1.1.4.2-1 as long as their strengths are taken from AWC NDS (2012) and the safety or resistance factors corresponding to those strengths are less severe than those in Standard Section D1.1.4.1. Otherwise, testing is required in accordance with Standard Section D1.1.5. The principle for adopting new fasteners in the analytical method is that confidence in the fastener strength must be as good as the ability to predict system strength.

The interior support fastener shank is subject to single shear, while the end-lap fastener shank is subject to double shear. With the exception of the shank, the fasteners at the end-laps are subjected to lower shear stress at service load as compared to fasteners at interior support since the interior support usually has larger load tributary area than the end support. As shear flows from the panel to the fastener, each ply sees single thickness bearing against the fastener, so an end-lap ply sees one-half the force that an interior support sees. At nominal load, each support connection along a sidelap or sidelap connection resists as much as it can due to redistribution. Because of this, many manufacturers may...
use $P_{nf}$ that is based on the single panel thickness ($t_1$) for both interior and end-lap connections in both single- and multiple-span tables.

D1.1.4.3 Screw or Nail Connection Strength Through Top Flat and Into Support

Fastener nominal strength [resistance] through top flats at interior corrugations is neglected due to the connection flexibility caused by the cantilever action of the fastener where the fastener only bears on the panel at the top. Since the opposing shear motion at sidelaps limits this support fastener from tilting, the fastener contribution is included at sidelaps.

$P_{nf}$ at this condition is based on one thickness. Generally, the steel sheets at the sidelap have the same thickness. If the thickness of the two sheets is different, $P_{nf}$ should be based on the thickness of the thinner sheet since the thinner sheet controls the bearing.

The connection provisions provided in Standard Section D1.1.4.3 are based on two possible limit states: (1) the fastener bearing nominal strength [resistance] in the wood support, and (2) the fastener bearing nominal strength [resistance] against the steel panel. The nominal shear strengths [resistance] of fully penetrated fasteners, $P_{nf}$, are obtained from the National Design Specification (AWC, 1986) as reported by Luttrell (1999a), while the bearing strength against a steel panel, $P_{ns}$, is discussed in Commentary Section D1.2.5 and is based on AISI S100. Screw bearing against the panel equals the sidelap connection shear strength determined using AISI S100 Equation J4.3.1-2 and J4.3.1-3 in Standard Section D1.1.2. The connection is not stronger than the fastener breaking shear strength.

D1.1.5 Other Connections With Fasteners Into Steel, Wood, or Concrete Support

The nominal shear strength [resistance] of a diaphragm fastener into a concrete support must be tested. Similarly, a fastener into a wood support must be tested if the fastener is not listed in Standard Section D1.1.4. AISI S905 should be used to determine the fastener nominal strength [resistance] and the connection flexibility. Alternative ASTM test standards are acceptable in Standard Section E1.1 for nominal strength [resistance] determination of fasteners into non-steel supports. However, calibration should be in accordance with Standard Section E1.2.2.

If the diaphragm system satisfies the requirements defined in Standard Chapter D, the design provisions provided in Standard Sections D1 and D2 can be used to determine the diaphragm nominal shear strengths [resistance] per unit length controlled by diaphragm interior, corner, and edge connections as well as controlled by diaphragm out-of-plane buckling. Only small-scale tests are needed to determine diaphragm nominal shear strength [resistance] that is controlled by support connection strength and to determine diaphragm stiffness related to connection flexibility. However, the reliability of connection strength determined by small-scale tests must be consistent with the system requirements in Standard Table B-1.

Support connection performance depends on the thickness, tensile strength and hardness of the support. For two examples: (1) the tilting resistance of screws depends on the support thickness; and (2) power-actuated fastener nominal strength [resistance] and selection depend on support thickness, tensile strength and hardness. If support connection strength is
not controlled by the bearing strength of the panel against the fastener, the support material properties must be considered in the small-scale tests.

A rational approach has been to use a single value of $P_{nf}$ to calculate the diaphragm nominal shear strength [resistance], $S_n$, if one type of support fastener is used in the diaphragm. This disregards the thickness-related differences in connection nominal strength [resistance] that might occur at end-laps over exterior supports shown in Figure D1-1 and at single-thickness conditions over interior supports. This also disregards the difference that might occur at sidelaps relative to interior flutes. The Standard concludes that the single-thickness value based on small-scale tests will control, provided all required edge dimensions and Standard equation application limits are met. A detailed discussion of the differences between end-laps and butt-joints is provided in the examples in the AISI Design Guide (2017). The smallest single-steel sheet thickness value at interior flutes should be used in design. This approach can be confirmed for fasteners listed in Standard Sections D1.1.1 through D1.1.2. Where small-scale tests indicate otherwise, the smaller end-lap or sidelap connection value should be used. AISI S905 can be used to test single-thickness shear connections. The Commentary of AISI S905 discusses shear tests for multi-layer sheets. When large-scale tests are used to evaluate fasteners, at least one of the large-scale tests should include end-lap conditions to verify that such connections can be made for a particular panel and will not control diaphragm nominal shear strength [resistance].

D1.1.6 Support Connection Strength Controlled by Edge Dimension and Rupture

The analytical method determines the number of fasteners per flute required for diaphragm strength, and multiple connections are allowed per flute. However, minimum fastener spacing and edge dimensions must be maintained. In addition, individual fastener tear-out or the group effect might become a limit state at the fastener cluster – a cluster of fasteners starts to act as one large fastener with failure around the cluster. Group rupture is a concern when the spacing within the cluster is tight and the edge dimensions are minimal. The rupture cluster edge requirement is analogous to checking $e_{min}$ for one fastener. This is evaluated using the rupture provisions in AISI S100 Sections J6.1 and J6.3. Consult the Commentary of AISI S100 for technical background information.

The principle in determining connection strength of a fastener group is to determine the least sum value of fastener $P_{nf}$ controlled by failure planes, e.g. the sum of shear planes at each row of fasteners parallel with the force, or staggered planes, if applicable, or failure around the entire fastener group.

D1.2 Sidelap Connection Shear Strength [Resistance] in Fluted Deck or Panel, $P_{ns}$

Standard Sections D1.2.1 through D1.2.6 include provisions to determine the sidelap connection nominal strength [resistance], $P_{ns}$, for fasteners whose nominal strength [resistance] is listed in AISI S100 or defined by the referenced research reports.

The Standard permits the connection nominal strength [resistance] to be determined by tests in accordance with Standard Section D1.2.7 and the diaphragm nominal shear strength [resistance] to be determined using the analytical approach provided in Standard Section D1.

D1.2.1 Arc Spot Welds

Standard Section D1.2.1 is consistent with AISI S100 Section J2.2.2.2. See Section J2.2.2.2
in the Commentary of AISI S100 for the corresponding technical background information. The lesser product, \( t_{F_u} \), should be used in AISI S100 Eq. J2.2.2.2-1 in the unlikely event that thickness or tensile strength of the connected sheets varies at the sidetrap connection.

Weld washers are not used at sheet-to-sheet sidetrap connections between supports. Industry also recommends that sidetrap welds be avoided at a thickness less than 0.028 in. (0.71 mm).

The spacing limit, 2.75d, often is irrelevant since normal spacing will exceed this number to avoid multiple “burn-throughs” at sidetrips.

**D1.2.2 Fillet Welds Subject to Longitudinal Shear**

*Standard* Section D1.2.2 is consistent with AISI S100 Section J2.5. See Section J2.5 in the Commentary of AISI S100 for the corresponding technical background information. The lesser product, \( t_{F_u} \), should be considered in the unlikely event that thickness or tensile strength of the connected sheets varies at the sidetrap connection.

**D1.2.3 Flare Groove Welds Subject to Longitudinal Shear**

*Standard* Section D1.2.3 is consistent with AISI S100 Section J2.6. See Section J2.6 in the Commentary of AISI S100 for the corresponding technical background information. The more conservative single-valued AISI S100 equation (covering the range \( t \leq t_w \leq 2t \)) is chosen in the *Standard*.

**D1.2.4 Top Arc Seam Sidetrap Welds Subject to Longitudinal Shear**

The top arc seam sidetrap weld has long tenure in diaphragms as an interlocking top sidetrap connection. As with all sidetrap welds, field quality control by the erector is required. Greater panel depth and narrow flute gaps increase the difficulty of making this connection. As shown in *Standard* Figure D1.2.4-1, both vertical-to-vertical and hem-to-vertical connections are possible. Firm contact is required for fusion and shear transfer. The hem lap is pinched or button-punched to clamp the vertical leg and to establish contact between the three vertical legs. The hem lap must be burned through and fusion established at the top of at least the two adjacent vertical legs, with one being in each of the respective panels. At a hem lap, that leg must be closest to the center of the panel - see *Standard* Figure D1.2.4-1(a). With proper clamping, fusion at all three legs is common and preferred. Fusion must exist at both vertical legs in Figure D1.2.4-1(b). Blowholes at top arc seam sidetrap weld ends are to be expected and are not detrimental to the nominal strength [resistance], which is based on the fused length.

The design provisions are based on the Nunna (2012), S.B. Barnes Associates report. The calibration factors for the resistance equations in *Standard* Section D1.2.4 are compatible with the system factors of *Standard* Chapter B. The non-dimensional resistance includes the impact of ductility in the ratio, \( \frac{F_{u}}{F_{y}} \), and the ability to longitudinally distribute resistance along the weld in the ratio, \( \frac{t}{L_{w}} \). Further information is available in the Commentary of AISI S100 Section J2.4.1.
The acceptable weld length in design is critical because as lengths get too large, tearing in the vertical leg could become a dominant limit. Both the observed failure modes of tearing in the vertical leg perpendicular to the axis of the weld in thinner steel sheets and shearing of the steel parallel to the axis of the weld for thicker sheets is accounted for in the Standard’s resistance equation over the prescribed limits of $L_w$, $t$, and $h_{st}$. There is no lower limit on $h_{st}$ for nominal strength [resistance] of weld. However, $h_{st}$ must be sufficient to qualify as an edge stiffener when required and must be of sufficient length to allow proper button punching or crimping when those options are chosen.

A minimum spacing is included to avoid excessive shear in the sheets below the weld line while developing the weld capacity. The shear rupture provisions of AISI S100 are adopted to rationally control this concern.

**D1.2.5 Sidelap Screw Connections**

At the sidelap connection, tilting and bearing limit the screw connection nominal strength [resistance]. The provisions conform to AISI S100 Section J4.3.1. The technical background information can be found in Section J4.3 of the Commentary to AISI S100. To ensure the required level of performance in structural applications, designers might specify that screws conform to ASTM C1513 or an equivalent standard.

The typical application of this Standard section is that $t_1 = t_2$ and $F_{u1} = F_{u2}$. The tilting limit might control and must be checked. The system effect (multiple fasteners in a line) can mitigate but will not fully eliminate the tilting concern.

**D1.2.6 Non-Piercing Button Punch Sidelap Connections**

The performance of traditional (manual or mechanically actuated) non-piercing button punch interlocking top sidelap connections is dependent on panel sidelap dimensions delivered to the field, tool maintenance, and the care of the erector. Analytical equations defining connection nominal strength [resistance] also vary as discussed in Bagwell (2008). For these reasons, a lower bound value that is independent of thickness is included in Standard Section D1.2.6 for shallower panels. The contribution of button-punched sidelap connections is neglected at deeper panels. See Commentary Section D1.5.2 that justifies neglecting the nominal shear strength [resistance], $P_{ns}$, for button-punched cellular deck when determining the diaphragm nominal shear strength [resistance] per unit length, $S_{nv}$ in accordance with the analytical method of Standard Section D1. The same justification applies to deep panels. However, the contribution of a button punch is not neglected in the determination of stiffness, $G'$. See Commentary Section D5.2.5.

It is acceptable but not mandatory to neglect the contribution of button-punched sidelap connections in design where that contribution otherwise would be permitted.

**D1.2.7 Other Sidelap Connections**

If a diaphragm meets the requirements specified in Standard Chapter D, the analytical approach outlined in Standard Section D1 should be applicable to other connections. However, the connection strength and its relationship to thickness and mechanical properties of the connection materials must be established by small-scale tests as required in Standard Section E1.1 for sidelap connections not defined in Standard Sections D1.2.1 through D1.2.6. The reliability of the connection strength established through tests must be consistent with
the system factors determined in accordance with Standard Table B-1. The contribution of other parameters has been established by the tests leading to the analytical method of Standard Section D1 and the method has been shown to work over a range of sidelap connections.

Several manufacturers have developed proprietary crimping tools that sometimes pierce the vertical legs at the interlocking top sidelap connections. These connection strengths are determined by tests and differ from the non-piercing button punch sidelap connections discussed in Standard Section D1.2.6. Connection flexibilities for such proprietary connections are discussed in Commentary Section D5.2.6.

**D1.3 Diaphragm Shear Strength per Unit Length Controlled by Support Connection Strength Through Insulation, S_{nf}**

The gap caused by insulation between panels and supports creates cantilever action in the fastener and can reduce the support connection nominal strength [resistance]. The contribution of fasteners at interior flutes is presented in Sections D1.3.1.1, D1.3.2.1 and D1.3.4 and is neglected in the analytical method, i.e. P_{nf} = 0.0 kips (0.0 kN), where the gap caused by insulation between panel and support is larger than 3/8 in. (9.53 mm). The contribution of support connection strength for common screws at interior panel sidelaps is included due to the opposing action of shear at these support connections in diaphragms. A common screw at sidelap is defined in the User Note at Standard Section D1.3.1.2. This action stabilizes the connections and makes their contribution effective. The opposing shear action typically is not present at edge reactions even if the support connection is through a sidelap. Therefore, it can be difficult to develop P_{nf} or P_{nfs} at edge lines. Exceptions are listed at the determination of S_{ne} in Standard Section D1.3.

Design provisions given in Standard Section D1 are applicable to profiled panels with insulation, provided the additional requirements listed in Standard Section D1.3 are met.

All support connections stabilize the diaphragm against panel buckling and resist uplift. The Luttrell (1999a) research for MCA indicated that a positive path must be provided at shear walls and perimeters to get the shear into and out of a diaphragm system. Subsequent work by Lease and Easterling (2006) indicated that shear could be transferred in and out of the diaphragm through the end and edge fasteners provided the gap caused by insulation between the steel support and panel bottom flat is less than or equal to 3/8 in. (10 mm).

In 2018, the small-scale tests and full-scale confirmatory tests reported by Lease and Easterling (2006) and Luttrell (1999) were recalibrated using the method of Standard Section D1.1.5. Reduction factors for screws in steel supports were determined so the resistance and safety factors of Table B1.1-1 could be maintained. This eliminated an inconsistency where the contribution of support screws through bottom flats at interior flutes was neglected in Section D1.3 while Section D1 allowed the full strength at screws into steel supports at both perimeter and edge conditions. The requirements at wood supports and other connections are unchanged.

Many connections listed in Standard Section D1.1 are not practical over insulation or where weather tightness is critical. The best example is an arc spot weld. The typical applications over insulation are limited to either screws or nails with sealing washers. This does not preclude development of other proprietary connections that will provide the necessary service requirements. For other connections, the method of Section D1.3.5 is applicable.
D1.3.1 Screws Through Bottom Flat of Panel Over Insulation and Into Steel Supports

D1.3.1.1 Screws at Interior Flutes

In 2018, reduction factors were added and the contribution of screws into steel supports is permitted at all locations where the gap caused by insulation is less than or equal to 3/8 in. (9.53 mm). The diaphragm nominal shear strength [resistance] can be determined using Standard Section D1 with the factor, $\beta$, defined by Standard Eq. D1-6, and the reduced support strength, $R_{P_{nf}}$, is available at all interior and exterior flutes. Where $R = 0$, $\beta$ simplifies to Eq. D1.3.2.2-1 and is associated with $P_{nf}$ at the sidelap.

Commentary Section B1.1 discusses the calibration process for safety and resistance factors given in Standard Table B1.1-1 for without insulation between the panel and the support. The resistance factor for screws subjected to wind load was not allowed to be greater than the historical resistance factor for panel buckling, which is a member limit state. $\phi = 0.80$ for wind loads is conservative relative to the 0.95 from direct calibration. $P_{nf}$ is the nominal strength of screws through panels without insulation. For panels both with gaps over supports caused by insulation and with gap thickness $\leq$ 3/8 in., the reduction factor, $R$, which is applied to $P_{nf}$, were determined in accordance with Standard Section D1.1.5. $R_{P_{nf}}$ was compared with Lease’s (2006) screw test data for panels over insulation. The Lease’s data must provide the same resistance factors as Table B1.1-1 for screws and this resulted in Table D1.3.1.1-1. Since the $\phi P_{nf}$ for wind was already conservative using $\phi = 0.80$, Section D1.1.5 is satisfied by only requiring a further reduction of $R = 0.95$ for wind. The calibration results are tabulated below for LRFD:

<table>
<thead>
<tr>
<th>Load Type</th>
<th>All Other Loads</th>
<th>Wind</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Case 1$^1$</td>
<td>Case 2$^2$</td>
</tr>
<tr>
<td>$\beta$ in Calibration</td>
<td>3.50</td>
<td>2.50</td>
</tr>
<tr>
<td>Aim $\phi$ in Table B1.1-1</td>
<td>0.70</td>
<td>0.95</td>
</tr>
<tr>
<td>Required $R$</td>
<td>0.779</td>
<td>0.787</td>
</tr>
<tr>
<td>$R$ rounded to 0.05</td>
<td>0.80</td>
<td>0.80</td>
</tr>
</tbody>
</table>

Note:
Case 1 determines the resistance factor, $\phi$, without restriction in Table B1.1-1 based on the calibration method of S310 Section E1.2.2.

Case 2 imposes the additional restriction that the fastener resistance factor, $\phi$, cannot be larger than the member limit state for panel buckling, which is $\phi = 0.80$. Standard Table B1.1-1 is based on Scenario 2.

The available strength [factored resistance] at wind loads, $\phi R P_{nf}$, is identical: $(0.95)(0.80)P_{nf} = (0.80)(0.95)P_{nf}$. If the original $\phi$ had not been reduced to 0.80, the nominal strength [resistance] of screws over insulation at all load events, $R_{P_{nf}}$, would be the same: 0.80 $P_{nf}$.
D1.3.1.2 Screws at Exterior Flute With Lap-Down at Sidelap

Where screw strength contribution is neglected at interior flutes, \( R \) and \( P'_{nf} = 0 \) and the nominal strength [resistance] of the support connection, \( P_{nf} \), through the bottom flat at sidelaps equals the value determined in accordance with Standard Section D1.1 where insulation is not present. This uses stability due to opposing motion at a common screw at sidelaps. Where screw strength contribution is present at interior flutes, \( R \) and \( P'_{nf} \) exist. \( R \) and \( P'_{nf} \) at the sidelap are conservatively set equal to the values at interior flutes for gaps that are caused by insulation and less than 3/8 in. (9.53 mm). This is for convenience in design and consistency with the theory in Standard Section D1. It is permitted to use \( R = 1 \) at the sidelap if a more refined design considers the variation of support screw strength at interior flutes and exterior flutes. A clear example of this benefit is a 36/3 screw pattern (support screws at 18 in. (457 mm) o.c.) where a diaphragm strength design for gaps greater than 3/8 in. (9.53 mm) with gaps caused by insulation would otherwise be greater than that for gaps less than 3/8 in. (9.53 mm). With more support screws at interior flutes, the benefit is trivialized.

D1.3.1.3 Lap-Up Condition at Sidelap With Screws Not Into Support

Where \( R \) and \( P'_{nf} = 0.0 \) kips (0.0 kN) in Section D1.3.1.1, the strength contribution of support connections at interior flutes is neglected. Where sidelap connections over supports at exterior flutes are not fastened into the support, the diaphragm nominal shear strength [resistance] only is based on the sidelap connection nominal strength [resistance], \( P_{ns} \), as shown in Standard Eq. D1.3.1.3-1. The diaphragm shear flow is from sheet-to-sheet until a perimeter detail is reached.

Where \( R \) and \( P'_{nf} \) exist in Section D1.3.1.1, support screw strength equals \( P'_{nf} \), and diaphragm strength is determined in accordance with Standard Section D1, which considers the interaction of support and sidelap screws.

D1.3.2 Screws Through Top Flat of Panel Over Insulation and Into Steel or Wood Supports

D1.3.2.2 Lap-Up Condition With Sidelap Screws Into Support

If the sidelap connections are fastened into the support, the diaphragm nominal shear strength [resistance] can be determined using Standard Section D1. Standard Eq. D1-6 is modified for the insulation effect to determine the factor, \( \beta \), in accordance with Standard Eq. D1.3.2.2-1 by using \( \alpha_e = A \), \( \alpha_p = A_p \), \( \alpha_e^2 = 0.5A \), and \( \alpha_p^2 = 0.5A_p \).

In wood supports with support connection nominal strength [resistance] determined using Standard Section D1.1.4.3, \( P_{nf} \) does not always equal \( P_{ns} \), so \( \alpha_s \) is determined using Eq. D1-7 while \( \alpha_s = 1 \) for fasteners into steel supports. The nominal shear strength [resistance] of the support connection into wood is based on the Luttrell (1999a) report where connection strength increases relative to sidelap connection nominal strength [resistance] because of fixity in the thicker wood. In the case of steel, this increase is not allowed and the support connection strength defaults to the sidelap connection strength. The latter requirement can be conservative at thicker steel supports where the same degree of base fixity should occur.
D1.3.3 Screws Through Bottom Flat of Panel Over Insulation and Into Wood Supports

Strength of screws through panels over minimal insulation thickness should approach the no insulation value but there is no data to justify a transition as a function of thickness. A gap caused by insulation at any flute is considered the same as an offset to the top of a panel. \( P_{nf} \) is neglected at interior flutes.

D1.3.4 Nails Through Bottom or Top Flat of Panel Over Insulation and Into Wood Supports

Strength of nails through panels over minimal insulation thickness should approach the no insulation value where nails are through bottom flats. However, there is no data to justify a transition as a function of insulation thickness. A gap caused by insulation at any flute is considered the same as an offset to the top of a panel. \( P_{nf} \) is neglected at interior flutes and the contribution at exterior flutes is determined using Section D1.1.4.3 whether the nail is through the panel’s bottom or top flat.

D1.3.5 Other Support Fasteners Through Insulation

If the fluted panels meet the requirements specified in Standard Section D1.3, tests in accordance with Standard Section D1.1.5 should be used to determine the connection nominal strengths [resistances] for fasteners not listed in Standard Sections D1.1.2 or D1.1.4.

D1.4 Fluted Acoustic Panel With Perforated Elements

Perforations may be located in the bottom flats or other elements of the acoustic panels. A common design has perforations only in the fluted panel webs. These web perforations will not affect the support or sidelap connection nominal strength [resistance] since no perforations exist at the diaphragm supports and at panel sidelaps where connections are made. However, the perforations might reduce the stiffness of the diaphragm system, and the reduction can be calculated using Standard Section D5.1.2 and Appendix 1 that is based on Luttrell (SDI, 2011). Contact the panel manufacturer for the design parameters reduced for perforations.

Designs that require connections through perforated zones must be tested using Standard Section D1.1.5 and D1.2.7 as applicable.

D1.5 Cellular Deck

D1.5.1 Safety Factors and Resistance Factors for Cellular Deck

Bagwell (2008) reported that 35 tests of cellular deck using various combinations of cellular deck thickness and fastener types were performed. The mean ratio, \( P_{test}/P_{theory} \), was 0.98 based on the theory of Standard Section D1. This set of data fits well within the scatter of the total test data (SDI, 1981) that is the basis of Standard Table B1.1-1. Consequently, the safety and resistance factors given in Standard Table B1.1-1 are also applied to cellular deck.

D1.5.2 Connection Strength and Design

The Bagwell (2008) test data indicated that it was difficult to make button-punched sidelap connections (Standard Section D1.2.6) on some products. The cellular deck ratio,
$S_{n test}/S_{n theory}$ has less scatter when the button punch contribution is neglected. Because of that, the diaphragm nominal shear strength [resistance] contribution from button-punched sidelap connections is ignored in the Standard. This does not apply to proprietary button punches or other proprietary crimping tool connections with nominal strength [resistance] established by test.

Typically, cellular deck profiles cannot be end-lapped. However, the total steel thickness of cellular deck that fasteners must penetrate can be large at supports. Depending on cellular deck product design, support connections (fasteners) might not engage top and bottom elements of the cellular deck at interior flutes. For example, individual top hats of the cellular deck are fastened to a bottom plate, and the bottom flats of the hat are not large. This might create a significant gap; thus, only the bottom plate is continuous over the support. In such cases, only the bottom plate thickness is used to determine $P_{nf}$.

Examples of Standard design provisions D1.5.2 (b) and (c) are illustrated in Figure C-D1.5.2-1. Shear in the diaphragm flows from sheet to sheet through panel sidelap connections. Where a sidelap does not provide a sound path without going through the support, fasteners are required at either side of the sidelap over supports. In Figure C-D1.5.2-1(a), two fasteners are required and the plane of shear transfer is below the bottom plate of the cellular deck. Support connection nominal strength [resistance], $P_{nf}$, and the effective diameter, $d_e$, for a weld are determined using one bottom plate thickness plus one top deck thickness. In Figure C-D1.5.2-1(b), the path is from sheet to sheet and requires one fastener if edge conditions are satisfactory. The shear transfer plane is then below the bottom plate of the top cellular deck to the right of the figure, and $P_{nf}$ is determined using that bottom plate thickness. The effective diameter, $d_e$, for a weld is then determined using two bottom plate thicknesses plus one top deck thickness.

The weld configurations shown in Figures C-D1.5.2-1(a) and C-D1.5.2-1(b) are acceptable as long as required edge dimensions and combined thickness limitations are met. They are not necessarily equivalent, and their respective capacities can be calculated using Standard Section D1.1.1 as specified in Section D1.5.2 (b).

![Figure C-D1.5.2-1(a)](image1.png)  ![Figure C-D1.5.2-1(b)](image2.png)

**Figure C-D1.5.2-1 Cellular Deck Interlocking Sidelaps**

**D1.6 Standing Seam Panels**

Since standing seam roof system clips at sidelap connections typically permit the standing seam roof system panels to expand and contract (float) along the panel longitudinal direction, it is difficult to develop longitudinal shear. Additional support connections may not be present. In such a case, $P_{nf}$ does not exist and $P_{ns}$ may be small. The calculated diaphragm nominal shear strength [resistance] per unit length, $S_n$, determined using Standard Section D1 is negligible. The Standard codifies the historical approach where $S_n$ is set equal to zero. The system can be tested in accordance with Standard Chapter E to establish diaphragm nominal shear strength...
Some manufacturers have tested their products to define this contribution. Standard Section D1.6 enumerates some of the same testing principles that are contained in ASTM E1592. Standing seam roof system panels also can provide some lateral stability to supports. This resistance is similar to but not the same as diaphragm shear. Consult the panel manufacturer for guidance in both areas.

AISI CF97-1 suggests tests for a diaphragm system that is fixed at one end using the method of AISI S907 and two test configurations: (1) with no end restraint, and (2) with end restraint at both ends. AISI CF97-1 then provides a method to extend the large-scale tests to a larger building application. This is consistent with the intent of the AISI S310 Standard where lack of fixity must be addressed.

D1.7 Double-Skinned Panels

Double-skinned panels are illustrated in Standard Figure D1.7-1. If the top panels are not connected to supports but connected to sub-purlins or sub-girts at an elevated plane, the shear force in the top panels will not be efficiently transferred to the supports. This is because of roll at the sub-purlins or sub-girts and the flexural flexibility of the bottom panel webs (vertical elements). Therefore, the contribution of the top panels is ignored and only the bottom panels provide the diaphragm shear strength [resistance], which is determined in accordance with Standard Section D1.

\((\lambda - 1)\) in Standard Eq. D1-1 represents a reduction in the nominal strength [resistance], \(P_{nf}\), at the corner support connections in each panel due to local distortion in the panel profile at a sidelap. Since only the bottom panel’s contribution is considered, the design assumption of the double-skin system is conservative. Also, since the bottom panel side web is relatively stiff and prevents distortion at the corner fasteners, the Standard eliminates the reduction by setting \(\lambda = 1\).

The bottom panel’s flat is often very wide, and local waves caused by shear buckling across this flat are a major concern. Appearance at service load is often critical in these panels. To avoid shear buckling, an additional rational limit state is imposed in the Standard. If the bottom panel is fastened to the support, as illustrated in Standard Figure D1.7-1, the bottom panel vertical elements can be considered as beam flanges (where the depth, \(h\), of the beam is defined as the spacing between the vertical elements). The diaphragm supports can be considered as transverse stiffeners (where the distance, \(a\), between transverse stiffeners of reinforced beam webs is defined as the spacing of diaphragm supports). The nominal shear stress of the beam web (the bottom panel’s horizontal flat in the figure) is then determined in accordance with AISI S100 Section G1 based on the herein defined \(h/t\) and \(a/h\), where \(t\) is the thickness of the bottom panel. The beam web area is taken as the area between the vertical elements of the bottom panel (ht), and the unit area is taken as the web area divided by the spacing between the vertical elements of the bottom panel. The diaphragm nominal shear strength [resistance] controlled by shear buckling is calculated using Standard Eq. D1.7-2. Since \(S_n\) is the nominal strength [resistance] per unit length, the controlling diaphragm strength for design is based on the lowest available strength [factored resistance] considering all limit states.

D2 Diaphragm Shear Strength per Unit Length Controlled by Stability, \(S_{nb}\)

Standard Section D2 determines the diaphragm nominal shear strength [resistance] that is controlled by shear buckling (out-of-plane panel buckling) and the local web buckling of panel over
exterior support. This shear buckling might manifest as several relatively large diagonal waves across several panels or as general (column-like) buckling between supports. When several diagonal waves occur, post-buckling strength can be present until connection failure occurs, which further controls the diaphragm resistance. The load on these connections is redistributed by tension field action and may not follow the model of Standard Section D1. Buckling initially is a material limit, so the Standard Table B1.1-1 factors in Standard Section B1 vary from the connection-related limits.

Standard Eq. D2-1 is a theoretical limit that includes the orthotropic nature of the diaphragm fluted panel and represents the same theory used to design corrugated webs in girders. This theory was presented in SDI DDM04 (2015) and was initially evaluated by Easley (1975). The Easley research contained confirmatory tests limited to single spans. For practical cases, Easley and McFarland showed that the elastic buckling load for thin corrugated metal diaphragms is predicted using Eq. C-D2-1. Since the strong axis flexural stiffness is more commonly based on $I_x$ ($I_{xg}$ in the Standard), the axes presented in Eq. C-D2-1 are shifted for convenience relative to that presented in the Easley paper.

$$S_{no} = \frac{36\beta_E D_y^{1/4} D_x^{3/4}}{L_v^2}$$

(C-D2-1)

where

$\beta_E = \text{Buckling coefficient allowance for end restraint and determined by tests}$

$= 1.07$

$D_x = \text{Strong axis flexural stiffness per unit width, k-in. (kN-mm)}$

$$= \frac{E I_x}{d} = \frac{E I_x}{d}$$

(C-D2-2)

where

$E = \text{Modulus of elasticity of steel, 29500 ksi (203000 MPa)}$

$I_x' = \text{Moment of inertia of one corrugation, in.}^4/\text{pitch (mm}^4/\text{pitch)}$

$I_x = \text{Moment of inertia per unit width, in.}^4/\text{in. (mm}^4/\text{mm)}$

$d = \text{Panel corrugation pitch, in./pitch (mm/pitch). See Standard Figure D2-1}$

$I_x = \frac{I_x'}{d}$

$D_y = \text{Weak axis flexural stiffness per unit length, k-in. (kN-mm)}$

$$= \left(\frac{E t^3}{12}\right) \frac{d}{s}$$

(C-D2-3)

where

$s = \text{Developed flute width per pitch determined in accordance with Standard Eq. D2-5, in./pitch (mm/pitch)}$

$t = \text{Base steel thickness of panel, in. (mm)}$

$L_v = \text{Span of panel between supports with fasteners, ft (m)}$

$$S_{no} = \frac{36\beta_E E t^3 I_x^2 (d)}{L_v^2 \sqrt{12 \left(\frac{d}{s}\right)}}$$

(C-D2-4)

Eq. C-D2-4 is dimensionally admissible for any unit system, but dimensional analysis is required to adjust for product and material data as commonly presented. Examples are:
U.S. Customary Units

Where $I_{xg}$ has units, in.$^4$/ft, while other parameters and units are as shown in the definitions ($I_{xg}$ is substituted for $I_x$ to agree with the Standard):

$$S_{no} = \frac{36(1.07)(29500)}{L_v^2}\left(\frac{k}{\text{in.}^2\text{ft}^2}\right)\sqrt[3]{\frac{t^3}{12}}\left(\frac{\text{in.}^3\text{in.}^2\text{in.} \text{ft}^3\text{in.}}{1728\text{in.}^3}\right)$$

$$S_{no} = \frac{36(1.07)(29500)}{12L_v^2}\left(\frac{k}{\text{in.}^2\text{ft}^2}\right)\sqrt[3]{\frac{1}{12\text{in.}}}\left(\frac{t^3}{12}\frac{\text{in.}}{12\text{in.}}\right)\sqrt[3]{\frac{d}{s}}$$

$$S_{no} = \frac{7890}{L_v^2}\sqrt[3]{\frac{t^3}{12}}\left(\frac{d}{s}\right)\frac{k}{\text{ft}}$$

SI Units

Where $I_{xg}$ is substituted for $I_x$, while other parameters and units are as shown in the Definitions:

$$S_{no} = \frac{36(1.07)(203000)1000}{L_v^2}\left(\frac{kN}{m^2m^2}\right)\sqrt[3]{\frac{t^3}{12}}\left(\frac{d}{s}\right)\left(\frac{\text{mm}^3\text{mm}^2\text{mm}}{\text{mm}^3\text{mm}}\right)$$

$$S_{no} = \frac{36(1.07)(203000000)}{1.861L_v^2}\left(\frac{kN\text{mm}^3}{m^2m^2}\right)\sqrt[3]{\frac{1}{10^9\text{mm}^3}}\left(\frac{t^3}{12}\frac{\text{d}}{10^3\text{mm}}\right)\sqrt[3]{\frac{d}{s}}$$

$$S_{no} = \frac{4.20}{L_v^2}\sqrt[3]{\frac{t^3}{12}}\left(\frac{d}{s}\right)\frac{kN}{m}$$

Nunna (2011) compared existing diaphragm test data with the equations in existing analytical models. The 28 tests exhibited panel buckling and included five multiple-span tests plus one hybrid test mixing multiple- and single-spans. The equation in the Standard represents a best fit between theory and tests. The buckling coefficient increased relative to the previous SDI DDM04 (2015) value. The same buckling strength is attributed to single and multiple-span applications. The Nunna report indicates that the resistance factors are reasonable when determined in accordance with Standard Table B-1. The evaluation results were rationally extended to the entire acceptable range of Standard Section D1.1.

The gross section moment of inertia should be used in the stability analysis, and this might be more representative of the available buckling stiffness of the panel during tests. However, section properties are typically published at a stress level consistent with service loads and these values are commonly used in diaphragm design to determine diaphragm strength and develop load tables.

The nominal shear strength [resistance] of diaphragms formed by fluted panels is based on the typical fluted panel section illustrated in Standard Figure D2-1. Perforations can affect the moment of inertia and the d/s ratio. Luttrell (SDI, 2011) provided an analytical method to determine this effect. Contact the deck or panel manufacturer for these parameters.

Standard Eq. D2-2 determines the diaphragm nominal shear strength [resistance] that is controlled by exterior support local web buckling. This theory was presented in a report by Nunna (2018b). Exterior support local web buckling occurs at the exterior support of deck panels and may occur at both lapped and butted joints.

Significant end warping is observed prior to exterior support local web buckling. As end
warping behavior becomes more extreme, forces are transferred through the panel webs in the form of tension and compression until the web in compression eventually fails. This failure mode is very similar to the failure mode observed in web-crippling tests of steel deck panels. A modified web crippling equation which was based on AISI S100, Equation G5-1 is used to calculate the web-crippling strength of the panel web in the test analysis. Slenderness of the web as well as bearing length influence the diaphragm nominal shear strength [resistance] that is controlled by exterior support local web buckling. Standard Eq. D2-4 addresses panels with perforated webs and is based on methods developed by Luttrell, SDI (2011).

Testing is always allowed to verify buckling capacity.

D2.1 Cellular Deck

There is limited (if any) data on panel buckling of cellular deck. Cellular deck was not considered in the derivation of Standard Eq. D2-1, but rational design allows that provision to be applied using the moment of inertia of the cellular deck and the thickness, pitch, and developed width of the top deck. This is similar to using the top deck buckling strength but amplifying that strength using the full cellular deck moment of inertia. This is rational engineering that neglects some of the shear sharing between the top deck and bottom plate and the additional torsional restraint of the closed cell units.

Perforations can affect the moment of inertia of the cellular deck and the d/s ratio of the top element. The top element fluted deck is rarely perforated in cellular deck, but the bottom element is commonly perforated to provide acoustic treatment.

Testing is always allowed to verify buckling capacity.

D3 Shear and Uplift Interaction

It is common for connections to experience simultaneous shear and tension (uplift) when the diaphragm resists a shear force caused by wind load.

The connection nominal shear strength [resistance], \( P_{nft} \), associated with a tensile load should satisfy the interaction equations outlined in Standard Section D3, and \( P_{nft} \) should replace \( P_{nf} \) in the equations provided in Standard Section D1 to determine the diaphragm nominal shear strength [resistance] per unit length, \( S_{nft} \). Whenever possible, the Standard’s interaction equations are based on the AISI S100 provisions, but since \( P_{nft} \) is required, the Standard’s equations have been altered to make them more directly useful.

Where nominal shear strength [resistance] of a diaphragm is determined in accordance with Section D1 and is based on \( P_{nft} \), the system factor determined in accordance with Standard Table B-1 is applied to the controlling \( S_{nft} \) in accordance with Standard Eq. D-1 or Eq. D-2, as applicable. The available strength [factored resistance] should be greater than or equal to the required shear strength [shear per unit length due to factored loads] in accordance with Standard Section C2.

\( P_{nft} \) can be determined using ASD, LRFD, or LSD. The result can vary slightly among the design methods.

D3.1 Support Connections

Consistent with AISI S100, three limit states in tension must be considered. The Standard allows linear interaction in lieu of testing.
**D3.1.1 Arc Spot Welds**

The provisions given in *Standard* Section D3.1.1 are consistent with AISI S100 Section J2.2.4. See the corresponding section in the *Commentary* of AISI S100 for technical background information.

**D3.1.2 Screws**

**D3.1.2.1 Screws Into Steel Supports**

*Standard* Section D3.1.2.1 is consistent with AISI S100 Sections J4.5.1, J4.5.2 and J4.5.3. See the corresponding sections in the *Commentary* of AISI S100 for technical background information.

Three tensile limit states are possible in the connection: pull-out, pull-over, and fracture in the screw shank. The first is associated with the thickness of the support, the second is associated with the sheet steel thickness of the panel, and the third is a screw property that typically does not control. *Standard* equations are provided to investigate these three limit states.

It is rational to use the controlling value of $P_{nf}$ (bearing, tilting, or shank fracture) in *Standard* Sections D3.1.2.1 (a) and (b). Only breaking nominal strengths [resistance] should be used in *Standard* Section D3.1.2.1 (c).

$S_n$ is directly proportional to $P_{nh}$ in the presence of wind uplift and $P_{nf}$ in the absence of wind uplift. For pull-over, *Standard* Eqs. D3.1.2.1-1 and D3.1.2.1-3 have been adjusted for ease of application. $\frac{P_{nft}}{\Omega_d}$ is substituted for the fastener’s required allowable shear strength, $\bar{V}$, in AISI S100 Eq. J4.5.1-1. $\phi_dP_{nft}$ is substituted for the fastener’s required shear strength, $\bar{V}$, [shear force due to factored loads] in AISI S100 Eq. J4.5.1-1. A similar adjustment is used for pull-out.

Simple design suggests that each load effect be considered separately and a screw pattern chosen to resist the required diaphragm shear strength [shear force due to factored loads]. Additional support connections are then added to resist uplift. The final design should be checked for interaction, and adjustments should be made as needed.

An anomaly exists in pull-over *Standard* Eq. D3.1.2.1-2 for ASD and Eq. D3.1.2.1-4 for LRFD and LSD. A reduction of 6 to 19% exists in screw connection shear capacity when there is no uplift. This is because the diaphragm system resistance factor, $\phi_d$, for wind loads is more than 23% greater than the resistance factor, $\phi$, used in the *Standard’s* interaction equation. An anomaly also exists in pull-out *Standard* Eq. D3.1.2.1-6 for ASD and Eq. D3.1.2.1-8 for LRFD and LSD since a 10 to 23% reduction in screw shear capacity exists when there is no uplift. This is because the diaphragm system resistance factor, $\phi_d$, for wind loads is more than 33% greater than the resistance factor, $\phi$, used in the *Standard’s* interaction equation. (A similar anomaly exists in LRFD, but is negligible.) Rational design in LSD might allow no reduction of factored resistance in shear when the effect due to factored tension loads is less than 5% of the factored resistance in tension in the absence of shear.

Engineers often use linear interaction design when other information is not available.
The *Standard* permits this approach when design is outside the test limits of the existing pull-over, pull-out, or breaking *nominal strength [resistance]* equations.

**D3.1.2.2 Screws Through Bottom Flats Into Wood Supports**

Three *limit states* may exist: one failure controlled by wood properties, and two failures controlled by steel properties. Therefore, in addition to this section, both *Standard* Sections D3.1.2.1 and D3.1.2.2 must be investigated.

Where *bearing* of the steel *panel* against the screw controls *connection nominal shear strength [resistance],* $P_{nf}$, and *nominal tension strength [resistance]* is controlled by pull-over, the interaction equation of *Standard* Section D3.1.2.1(a) applies. Fracture in the screw is unlikely for most applications but should still be checked per *Standard* Section D3.1.2.1(c). Otherwise, where *bearing* of the screw against wood, $P_{nfw}$, or pull-out from wood, $P_{nT}$, controls, the interaction equations of *Standard* Section D3.1.2.2 apply.

The *Standard*’s shear and tension interaction provisions (controlled by wood bearing and pull-out) for screws fastened into wood supports are obtained from AWC NDS (2012), and the equation is an application of the Hankinson formula. $T$ or $\bar{T}$ is determined considering that *support connections* resist the total uplift. However, $V$ and $\bar{V}$ depend on the load sharing of both *sidelap connections* and *support connections*, and that load sharing is indicated in Eqs. D1-1 and D1-2. The design requires iteration as a *diaphragm configuration* is evaluated. To perform the iteration, an engineer can assume a $P_{nft}$ less than $P_{nfw}$, where $P_{nft}$ is the *nominal strength [resistance]*. When the *diaphragm required strength* [force due to factored loads] per unit length equals the *available strength [factored resistance]*, then $V = \frac{P_{nft}}{\Omega}$ for ASD or $\bar{V} = \phi P_{nft}$ for LRFD and LSD. Calculate $P_{nft}$ using Eq. D3.1.2.2-1 and the values of $V$ or $\bar{V}$ based on the assumption and the values of $T$ or $\bar{T}$ based on the design analysis, and compare the calculated $P_{nft}$ with the assumed $P_{nft}$. Depending on the difference, a new value of $P_{nft}$ is assumed for the *diaphragm configuration*. $S_{nf}$ is then calculated using Section D1 and the final $P_{nft}$. If Eq. D3.1.2.2-4 is satisfied, design is considered satisfactory for that *configuration*. Depending on the spread, alternate *diaphragm configurations* can be considered, and the process repeated.

Since sufficient data does not exist to provide an interaction equation for fasteners through the top flats, rational design or testing is required.

**D3.1.3 Power-Actuated Fasteners**

The *power-actuated fasteners* (PAFs) used in design should be specified and no substitutions should be allowed unless equivalence is substantiated by test data.

For PAFs, small-scale tests should be performed to determine the combined shear and tension effect in accordance with AISI S905. If a test-based interaction equation is not available, or development is not justified, the *Standard* allows a linear interaction equation. The *safety and resistance factors* for *available pull-over strength [factored resistance]* and the strength itself should then be determined in accordance with AISI S100 Section J4 for screws. When washers are present on *power-actuated fasteners*, it is rational to use the pull-over equation for screws in AISI S100 Section J4.4.2 to define $P_{nov}$. Pull-out must be
investigated as a limit state and these nominal strengths [resistances] can be obtained from the fastener manufacturer.

Luttrell (Section 4.10 of SDI 2004) has done the required small-scale tests to establish $P_{nf}$ and the interaction for particular (but not all) PAFs. When an interaction equation listed in SDI DDM04 (2015) is used to satisfy the Standard’s testing requirement, the resistance factor for shear, $\phi_d$, is the value listed in Standard Table B1.1-1 for screws, and the resistance factor for a power-actuated fastener subjected to tension, $\phi_t$, equals 0.5. The SDI listed equations assume pull-over will control tension.

Proprietary power-actuated fastener interaction equations may be available from manufacturers and could be similar to those listed in SDI DDM04 (2015), but the Standard requires test verification, and the resistance factors must be determined in accordance with Standard Section E1.2.2. The statistical parameters, $M_m$, $V_M$, $F_m$, and $V_F$, are listed in AISI S100 Table K2.1. To ensure that the calibrated interaction equation has the equivalent accuracy as provided by the Standard’s diaphragm system in Section D1, the calibrated interaction equation should have an equal or smaller safety factor, and an equal or greater resistance factor than the requirements of Standard Table B-1. Accuracy is dependent on the average ratio (test/theory) of the PAF interaction equation and the scatter which define the factors. Equivalent calibration factors could require a theoretical strength reduction.

If the support connection nominal shear strength [resistance], $P_{nf}$, can be determined in accordance with Standard Section D1.1.5 and the interaction effect is established by small-scale tests, large-scale shear diaphragm tests are not required. It should be noted that it is extremely difficult to conduct large-scale shear and uplift interaction tests using test facilities such as air bags or vacuum chambers. The large-scale tests can be used to determine the diaphragm nominal shear strength [resistance] per unit length, $S_{nv}$, and the support connection nominal shear strength [resistance], $P_{nfv}$, and to define the resistance factors without the uplift effect. The interaction equations will then be used to define $P_{nfft}$.

### D3.1.4 Nails Through Bottom Flats Into Wood Supports

The design provisions considering the interaction of shear and tension of nails into the wood supports are obtained from AWC NDS (2005). Three limit states are considered: 1. failure controlled by wood properties, 2. failure controlled by steel panel properties (combination of nail bearing against the panel in shear and nail pull-over), and 3. failure controlled by nail fracture properties. The pull-over nominal strength [resistance] equation and interaction equation for nails are the same as those for screws, which is given in Section D3.1.2.1. Washers may be required for weathertightness. Fracture in the nail is unlikely to occur for most applications but is checked in Standard Section D3.1.2.1(c).

An interaction equation for fasteners through the top flats is not included. Rational design or testing is required.

### D3.2 Sidetap Connections

The nominal shear strength [resistance] reduction due to wind uplift does not need to be considered for sidetap connections. The sidetap connections will move along with the steel panels under the wind uplift and there is negligible differential movement at the sidetap to cause strain in the sidetap connections. As a result, no tension force is introduced.
D4 Steel Deck Diaphragms With Structural Concrete or Insulating Concrete Fills

There are a limited number of publicly available experimental programs on steel deck diaphragms with structural concrete or insulating concrete fills. Porter led a series of 32 tests on steel deck diaphragms with structural concrete fill. Specimens were 15 ft (4.6 m) long by 15 ft (4.6 m) or 12 ft (3.7 m) across, used normalweight concrete (except one specimen that used lightweight), included arc spot welds, steel headed stud anchors, or a combination of the two for perimeter fasteners, used shored construction negating the need for intermediate support test frame members, and subjected the specimens to quasi-static cyclic loading (Porter and Greimann, 1980, Porter and Easterling, 1988, Easterling and Porter 1994a, Easterling and Porter 1994b). Luttrell (1971) conducted a series of eleven monotonic cantilever tests, nine with insulating concrete fill utilizing welded structural fasteners. Davies and Fisher (1979) tested four concrete on metal deck cantilever diaphragm specimens. Three of the four specimens incorporated re-entrant profile steel deck types, with the final specimen using the more traditional trapezoidal steel deck profile. ABK (1981) conducted one test on a concrete on metal deck diaphragm as part of a larger testing program. The specimen was tested in a three-span, simply supported condition with welded structural fasteners and subjected to dynamic loading, but was not loaded to failure. Fukuda et al. (1991) tested four concrete on metal deck diaphragm specimens with cellular deck and block-outs in the concrete for trench ducts. A substantial amount of private, proprietary testing on concrete on metal deck diaphragms was also performed in the 1960s through 1980s by S.B. Barnes and Associates, as identified by Porter and Easterling (1988).

Reinforcement conforming to ACI 318 Section 20.3 or fibers are used in practice for various purposes in structural concrete. This Standard does not currently address the use of fiber-reinforced concrete or conventional reinforcement in determining diaphragm shear strength. However, the use of fibers or conventional reinforcement in structural concrete is not precluded by this Standard. None of the tests reported by Porter and Greimann (1980) or Porter and Easterling (1988) included conventional reinforcement or fibers. If it is desired to consider conventional reinforcement in the calculation of diaphragm shear strength, the approaches in ACI 318 may be appropriate, including satisfying the related assumptions such as ductility of the reinforcement.

Slabs with cover greater than 6 in. (152 mm) are permitted but the analytical value, $S_n$, should be based on a maximum value of 6 in. (152 mm).

Lightweight insulating concrete is discussed in Standard Section D4.3.

Editions of this Standard prior to 2020 had a lower limit on the steel support thickness to be greater than or equal to 0.10 in. (2.54 mm). This was removed from the Standard recognizing that this limit did not exist in the Steel Deck Institute Diaphragm Design Manual, Second Edition (1987). A lower limit of 0.10 in. may be necessary for diaphragms that are welded to the supports in order to facilitate making the weld, but when diaphragms are supported on cold-formed steel framing with mechanical fastening, this lower limit is not necessary.

D4.1 Safety Factors and Resistance Factors

The factors for the calculation of available diaphragm shear strength [factored resistance] are limited to Standard Section D4.1. However, if the diaphragm nominal strength [resistance] is determined through large-scale testing, the safety and resistance factors should be determined.

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in accordance with Standard Section E1.2.2 for structural concrete and lightweight insulating concrete fill. For structural concrete, Standard Section E1.2.2 is not limited by the values provided in Standard Section D4.1.

Two distinct limit states are evaluated herein, namely, diagonal concrete tension cracking and perimeter fastener failure. A target reliability index of $\beta_o = 3.5$ is used for the fastener limit state and a target reliability index of $\beta_o = 2.5$ is used for the diagonal tension cracking limit state. The use of two separate target reliabilities is intended to parallel the basic target reliabilities in AISI S100 which employ $\beta_o = 2.5$ for members (thus assuming a lower consequence of failure) and $\beta_o = 3.5$ for connections (thus assuming a higher consequence of failure and hence a desire for a lower probability of failure in the connection). The same philosophy is applied here for the diaphragm, whereby diagonal tension cracking is the preferred, lower consequence, limit state and assigned the lower target reliability. Based on the selected target reliabilities the safety and resistance factors given in this section were calculated in accordance with AISI S100; see O’Brien et al. (2017) for details.

The use of two separate target reliabilities in the diaphragm design is a departure from past practice (i.e., AISI S310-16), as in the past the two limit states were mixed and the more conservative $\beta_o = 3.5$ was applied throughout. The test-based method of Chapter E continues to require a single $\beta_o$ of 3.5 for steel deck with structural concrete. In the Chapter E User Note and commentary, it is recommended that the target reliability for concrete limit states be aligned with that of structural concrete slabs, of which Tabsh (1997) suggests a $\beta_o = 3.0$ is appropriate. Examination of the reliability of structural concrete for beams in shear by Israel et al. (1987) using $\phi = 0.75$ (as is still used in ACI 318-19) suggests a realized $\beta_o$ in the range of 2.3 to 5.0 for concrete shear limit states, and more recently Rakoczy and Nowak (2014) examining both lightweight and normalweight concrete in shear indicate a realized $\beta_o$ in the range of 3.3 to 4.1 for unreinforced beams in shear and 2.1 to 2.6 for unreinforced slabs in shear. $\beta_o = 2.5$ has been selected for the diagonal tension cracking as it establishes clear preference for this limit state over loss of perimeter fastening, and aligns the diaphragm reliability in AISI S310 with basic member reliability in AISI S100. Further, the presence of the steel deck beneath the concrete fill, plus the higher reliability for the perimeter fastener limit states, provides a secondary load path that is not available in concrete slabs and thus the consequence of failure is further reduced and a higher $\beta_o$ is not deemed necessary.

### D4.2 Structural Concrete-Filled Diaphragms

The testing program at Iowa State University revealed three possible limit states for diaphragms with structural concrete fill (Porter and Greimann, 1980, Porter and Easterling, 1988, Easterling and Porter 1994a, Easterling and Porter 1994b): diagonal tension cracking, perimeter fastener failure, and shear transfer failure. The nominal shear strength [resistance] for diaphragms with structural concrete fill in this section is based on the limit state of diagonal tension cracking of the concrete. Section D4.4 addresses the limit state of perimeter fastener failure, and the commentary in Section D4.4 discusses the shear transfer failure limit state which is uncommon for diaphragms with typical perimeter fasteners.

The nominal shear strength [resistance] for diaphragms with structural concrete fill is calculated as based on the shear strength of the concrete slab including consideration of steel deck through a transformed section. This equation was validated against the Iowa State University tests; see Easterling and Porter (1994b) and O’Brien et al. (2017). The average
thickness of structural concrete, $t_c$, can be calculated as the cross-sectional area of the structural concrete over one deck panel divided by the width of the deck panel.

### D4.3 Lightweight Insulating Concrete-Filled Diaphragms

The equations provided in *Standard* Section D4.3 are adopted from SDI DDM04 (2015). The system in *Standard* D4.3(a) is based on lightweight insulating concrete fill with vermiculite aggregate or cellular foaming agent. The system in *Standard* D4.3(b) is based on a layer of lightweight insulating concrete fill placed to a level slightly above the corrugation top flats or crests. Rigid insulation boards of expanded cellular polystyrene, having about 2% of the board surface area containing holes, are then embedded into the concrete and the concrete slurry fills the hole openings. A 2-in. (50-mm) thick topping of insulating concrete is placed over the polystyrene to finish the diaphragm: e.g., if the polystyrene insert is 1 in. (25.4 mm) thick, the total cover over the form deck is approximately 3-¼ in. (82.6 mm) – i.e., 1/4 in. (6.35 mm) bonding slurry plus 1 in. (25.4 mm) insert plus 2 in. (50.8 mm) topping. Insulation boards are held 3 ft (1 m) back from the diaphragm shear-resisting reaction lines (shear walls or interior moment frames), so the insulating concrete fill is full depth (3-¼ in. (82.6 mm) in the example) in those zones. Full depth provides a path to transmit shear out of the fill and develop concrete bond at this critical reaction transfer zone. If a system differs significantly from these descriptions, diaphragm nominal shear strength [resistance] should be determined in accordance with *Standard* Chapter E. The insulation fill manufacturer may be able to provide this test information.

Type (b) is rarely used, and *Standard* Eq. D4.3-2 predicts a lower bound diaphragm nominal shear strength [resistance] for 2-in. (50.8-mm) cover over insulating board based on test data for various covers over deck and board thickness. In type (b), the minimum probable solid insulating concrete thickness over deck, $d_c$, near lateral force-resisting system lines is about 3 in. (76.2 mm) for 2-in. (50.8-mm) fill thickness over board. Using *Standard* Eq. D4.3-1, $d_c = 3$ in. (76.2 mm), and $f'_c = 125$ psi (0.862 MPa) lead to an insulating concrete contribution of 0.537 klf (7.84 kN/m). Eq. D4.3-2 leads to 0.716 klf (10.4 kN/m), so the results are not entirely out of line. When cover over board is 3 in. (76.2 mm), the insulating concrete fill thickness over deck at a lateral force-resisting system line is about 4 in. (102 mm) minimum and type (a) provides 0.716 klf (10.4 kN/m). Types (a) and (b) converge and for greater fill cover over board, the type (b) diaphragm nominal shear strength [resistance] equation predicts a lower value.

Differing amounts of Portland cement, water, aggregates (vermiculite and/or perlite) and/or preformed cellular foam are mixed together dependent on specific requirements (National Roof Deck Contractors Association, 2012). The insulating characteristics could be enhanced either by the entrapped air in the pores of the expanded aggregate, or by air injected under pressure into the concrete mix using the foaming agent to stabilize the mix. The latter creates closed cell air bubbles within the cellular insulating concrete mix. Cellular insulating concrete may contain no sand or other aggregate. Consult the insulating concrete fill manufacturer for specific product requirements and installation instructions. The cellular insulating concrete foaming agent should conform to ASTM C869.

Lightweight insulating concrete fill is typically placed over form deck other than composite deck. Depending on the roof membrane and fill manufacturer’s requirements, the deck may require venting.
D4.4 Perimeter Fasteners for Concrete-Filled Diaphragms

For structural and lightweight insulating concrete-filled diaphragms, sufficient connections must be provided along the perimeters so shear forces can be transferred into and out of the diaphragm at perimeter transverse supports, such as spandrel beams, and at edge panel longitudinal supports, such as shear walls, braced frames, or moment frames. The designer should include supports in the diaphragm’s bottom plane to allow fastener installation and shear transfer. However, this design requirement is sometimes wrongly overlooked at edges parallel with the deck span. See the Commentary on Section D1 since this concern also applies without fill.

Standard Eqs. D4.4-1 and D4.4-2 are based on the assumption that the nominal shear strength [resistance] per unit length, $S_{n\ell}$, is proportional to the number of edge fasteners, $n_e$, along a panel length, $L$, but $n_e$ should not be less than $L/\alpha$, where $\alpha$ is an industry serviceability limit for connection spacing at larger $L$. At perimeters perpendicular to the deck span, the number of fasteners per unit width, $N$, is determined based on the assumption that the diaphragm strength is proportional to the number of fasteners at perimeters. Where the required $S_n$ varies along the diaphragm span, $L_d$, the required $N$ can also vary along that length. Statics only requires the $N$ connections to resist the component perpendicular to the deck span, i.e., $S_n$. The limited flexural stiffness of most spandrel beams about the weak axis would not allow development of a component parallel with the deck’s span even where that component exists. SDI DDM04 (2015) addresses similar details and concerns that occur at slab perimeters or discontinuities at large holes in diaphragms.

Diagonal tension in structural or insulating concrete-filled diaphragms is associated with two perpendicular shear components. Since most of the shear is flowing through the fill in structural concrete, those components must be resisted by perimeter connections at building corners or perimeter points along reaction lines, for that is where shear gets out. For structural concrete-filled diaphragms, because of potential force redistribution and the large number of fasteners along longitudinal (reaction) and transverse (along $L_d$) perimeter lines, the Standard does not mandate an increase of connections at the corners.

It should be noted that $N$ should satisfy the industry maximum allowable spacing requirement in addition to developing the strength resistance requirement.

Standard Eq. D1-2 addresses the shear resistance needed along the perimeters and at the corners for diaphragms without concrete fill.

Standard Sections D4.2 through D4.4 address two limit states, namely diagonal tension cracking and perimeter fastener failure. A third, less common, limit state was identified by Easterling and Porter (1994a and 1994b) that consists of shear transfer failure between the deck and concrete. For this failure mode to occur, an unusually large amount of structural fasteners must be used without any structural fasteners that allow for a direct mechanical connection between steel beams and concrete fill (e.g., steel-headed stud anchors or standoff screws). In addition, diaphragms subjected to large gravity loads may be less likely to fail in the shear transfer limit state because normal forces impose additional friction resistance at the deck to concrete interface (Neilsen, 1984). Test specimens with perimeter fastener strength equivalent to one 3/4 in. (19 mm) arc spot weld per foot resulted in perimeter fastener failure instead of shear transfer failure. Some tests with more welds failed due to shear transfer. For diaphragm configurations with perimeter fasteners that do not make direct mechanical connection between the steel beams and concrete fill, but are stronger than one 3/4 in. (19
mm) arc spot weld per foot, consideration of the limit state of shear transfer failure may be warranted. See Easterling and Porter (1994a, 1994b).

### D4.4.1 Steel-Headed Stud Anchors

The structural concrete develops a significant portion of the total diaphragm nominal shear strength [resistance], which can be an order of magnitude greater than the strength without fill. The nominal shear strength [resistance] of other fasteners at edge panels may adequately provide the nominal strength [resistance], so steel-headed stud anchors are not required. At larger loads, steel-headed stud anchors may be required to transfer shear from the concrete slab to the lateral force-resisting system or to the transverse perimeter supports. The welded steel headed-stud anchors provide a direct path to collect shear from the concrete. This avoids having to count on the chemical bond between the deck and the concrete to transfer shear, and having to use an excessive number of other types of fasteners. The steel-headed stud anchors also can resist end “slip over” in structural concrete slabs on deck and provide composite beam resistance at the supports.

The required number of welded steel-headed stud anchors at edge panels, $n_e$, depends on the magnitude of the required diaphragm strength [shear due to factored loads] along the line of transfer, and $n_e$ should be determined in accordance with Standard Section D4.4. Steel-headed stud anchor nominal strength [resistance] is determined in accordance with ANSI/AISC 360. The maximum spacing required by ANSI/AISC 360 should be checked in addition to determining the number of steel-headed stud anchors required by Standard Eq. D4.4-1 or D4.4-2.

The thickness of the deck supports must be considered before selecting anchors to transfer shear. ANSI/AISC 360 provides guidance on support thickness and the impacts of galvanized thickness and deck thickness(es) on steel-headed stud anchor installation. ANSI/AISC 360 also provides guidance on spacing and edge dimensions. Where mechanical shear connections are allowed by the building code, they may be used in lieu of welded steel-headed stud anchors, but the designer should avoid mixing shear connection types unless the connection flexibilities are comparable.

The reliability of the connection nominal shear strength [resistance], $P_{nfs}$, must be consistent with that of the diaphragm system. The safety factor provided in ANSI/AISC 360 for steel-headed stud anchors is 2.31 and the resistance factor is 0.65. These factors are less severe than Standard Section D4.1, so the number of fasteners, $n_e$, should be determined in accordance with Standard Eq. D4.4-1 or D4.4-2 using the ANSI/AISC 360 nominal strength [resistance].

A mechanical shear connection may be used in structural concrete where the connection safety factor is greater than or the resistance factor is less than the factors in Standard Section D4.1. In such cases, the reliability of the connection nominal shear strength [resistance], $P_{nfs}$, is not consistent with that of the diaphragm system. The mechanical shear connection’s nominal strength [resistance], $P_{nfs}$, should be reduced proportionately to the respective factors when calculating $n_e$ in accordance with Standard Section D4.4. $P_{nfs}$ should not be increased if the connection’s factors obtained from tests are better than the diaphragm system factors. As an example, the safety factor for the shear connection in concrete is 4 (published by manufacturer or determined by push-off tests), but the diaphragm system’s safety factor is 3.25; therefore, a reduced $P_{nfs} = (3.25/4)P_{nfs}$ should be used in Standard Eqs. D4.4-1 and
D4.4-2. A similar reduction may be required in Standard Section D4.2 if the factors also are not consistent when based on tests with deck alone and in accordance with AISI S905.

This Standard provision does not preclude the use of a proprietary shear stud or a mechanical shear connection in lightweight insulating concrete or structural concrete-filled diaphragms with strength verified by large-scale test using Standard Chapter E.

D5 Diaphragm Stiffness

D5.1 Stiffness of Fluted Panels

D5.1.1 Fluted Panels Without Perforated Elements

Standard Eq. D5.1.1-1 used to calculate the diaphragm stiffness, \( G' \), is based on SDI DDM04 (2015) and Luttrell (Luttrell, 1999a and 1999b; and MCA, 2004). It was developed based on the fluted panel as shown in Standard Figure D2-1. The background information for this equation is provided in Commentary Appendix 1. In lieu of analytical Eq. D5.1.1-1, large-scale tests may be performed in accordance with Standard Chapter E.

Stiffness based on tests (AISI, S907) is determined at 0.4\( S_{ni \text{ test}} \) and is used to calculate in-plane deflection at the nominal load [specified load]. In Standard Eq. D5.1.1-1, the K factor measures the relationship between support connection and sidelap connection flexibilities. In the lap-down case over steel supports, \( K = 1 \) since this is the baseline for other cases. This indicates that support connection clamping to steel supports, particularly those at lap-down sidelap connections, restrains slippage at sidelap connections between supports. In the lap-up case, the diaphragm stiffness is significantly reduced and the ratio of support connection flexibility to sidelap connection flexibility is used to reduce the baseline stiffness. The wood support \( K \) is consistent with the ratio of connection flexibilities listed in Standard Section D5.2.3. For a screw-screw or nail-screw combination, \( \frac{S_f}{S_s} = \frac{1.5}{3} = 0.5 \). In the Standard, \( K = 0.5 \) is used even when the screw or nail is through the top flat and into the wood support where \( \frac{S_f}{S_s} = \frac{3}{3} = 1.0 \). Fasteners through the top flat and into the support are shown in Standard Figure D1.1.4.3-1.

Perimeter details should be designed to minimize purlin or structural joist roll and to provide a stable path for shear flow into the shear wall or moment frame. Figure C-D5.1.1-1 illustrates purlin roll. Figure C-D5.1.1-2 provides one possible detail to control purlin roll.
D5.1.2 Fluted Acoustic Panels With Perforated Elements

Perforations can affect all three items in the denominator of Standard Eq. D5.1.1-1. Luttrell (2011) provides a method to calculate this impact. The method addresses increased shear deflection in the panel elements and end warping. Shear deflection in a panel element is impacted by reduced shear stiffness across the perforated zone. End warping, $D_n$, discussed in the Standard Appendix 1 and its Commentary, is impacted by reduced flexural stiffness of the panel profile elements. The impact of perforations on $I_x$, $D_n$, and $G'$ can be calculated and the necessary parameters for $D_n$ and shear deflection in the panel elements can be obtained from the panel manufacturer.

The same fastener slippage constant, $C$, applies to acoustic panels and non-acoustic panels when fasteners are not in perforated zones. The effect of increasing $s$, $D_n$, and $C$ in the denominator of Standard Eq. D5.1.1-1 results in a decrease of stiffness, $G'$, and increased diaphragm deflection.

Standard Eq. D5.1.2-1 determines an equivalent width of a solid profile, $s$, for a perforated panel. The equivalent width provides the same shear deformation as a perforated element when subjected to the same diaphragm shear load. Note that this equivalent width, $s$, should not be used in the determination of $D_n$ (see Standard Appendix 1) since different deformations are being considered – shear deformation in elements vs. element flexural racking at $D_n$.

If perforations are localized in webs of panels with depths less than or equal to 3 in. (76.2 mm) and the total perforation area is limited, the impact on $I_x$, $s$, $D_n$, and $G'$ can be small if not negligible.

A common application might illustrate the impact of perforations on $G'$: Deck type is WR (see Commentary Appendix 1, Table C-1.1a), $s = 8.19$ in. (208 mm) with no perforation; however, holes only are in the webs with $W_p = 1$ in. (25.4 mm), $c_p = 3/8$ in. (9.53 mm), and $d_p = 1/8$ in. (3.18 mm). These parameters lead to $p_o = 0.10$ (per Eq. C-1.6-1) and $k_{web} = 0.78$ (per Eq. C-1.6-2 while extrapolating Standard Eq. 1.5-5 results in $k_{web} = 0.72$). The modified
s is 8.75 in. (222 mm) (per *Standard* Eq. D5.1.2-1) or 6.8% greater than s with no perforation. The modified first denominator term in Eq. D5.1.1-1 is 3.79 vs. 3.55 and is often of little consequence relative to the other denominator terms. In the example, Dn would also be modified (See *Standard* Appendix 1). \( \gamma_c D_n \) is normally much larger than 3.79. A conservative example to investigate Dn in the denominator is: \( L_v = 6 \) ft (2 m), \( L = 18 \) ft (6 m), \( t = 0.0358 \) in. (0.909 mm), and with a fastener in each flute, so the modified \( s \) is 8.26 in. (210 mm) (per *Standard* Eqs. 1.4-7 and 1.6-3) and \( \gamma_c D_n \) perforated is 0.9(4.30) = 3.87 vs. 0.9(4.28) = 3.85 unperforated – impact = 0.5%. The sum of the first two denominator terms is 7.66 vs. 7.40 or within 3.5% and this difference can be reduced by slippage, C, which could be about 7.5. Using this C, the denominator difference is about 2.0% (15.2 vs. 14.9). With a fastener in every other flute, \( \gamma_c D_n \) perforated is 0.9(35.9) = 32.3 vs. 0.9(35.8) = 32.2 unperforated. The sum of these three terms is 43.6 vs. 43.3 or within 1% and dominated by \( D_n \). Test data indicates \( \frac{G_{\text{test}}'}{G_{\text{theory}}'} \) scatter much greater than this. The example’s web perforation impact is negligible in both cases (< 2%).

The stiffness reduction due to perforations only in the webs and for panel depths less than 3 in. (76.2 mm) usually will not affect the diaphragm performance. This assumes the perforation pattern does not consume a large portion of the web area. Common web perforation patterns are less than 23% of the perforated zone.

### D5.2 Connection Flexibility

*Structural connection flexibility*, \( S_f \), and *sidelap connection flexibility*, \( S_s \), provide the values of connection flexibilities necessary to calculate diaphragm stiffness, \( G' \). These flexibilities are based on tests discussed in Luttrell (1981) and presented in SDI DDM04 (2015), Luttrell (1999a), and Nunna (2012). \( S_f \) and \( S_s \) indicate local distortion, strain, or slippage at fasteners in a connection.

#### D5.2.1 Welds Into Steel

**D5.2.1.1 Arc Spot or Arc Seam Welds**

The equations presented in *Standard* Section D5.2.1.1 for determining the connection flexibilities of arc spot or arc seam welds are adopted from SDI DDM04 (2015). Arc spot and arc seam welds are illustrated in *Standard* Sections D1.1.1 at supports and D1.2.2 at sidelaps. *Standard* Eqs. D5.2.1.1-1 and D5.2.1.1-2 indicate that size of weld has negligible impact on the tested connection flexibilities. Arc seam welds at sidelaps are not the same as top arc seam sid-lap welds. By way of comparison, they have the same flexibility at a 2.33-in. (59.2-mm) long top arc seam sid-lap weld, so there is relative consistency.

**D5.2.1.2 Top Arc Seam Sidelap Welds**

The equations presented in *Standard* Section D5.2.1.2 for determining the connection flexibilities of top arc seam sid-lap welds are based on the research sponsored by industry and reported by Nunna (2012).

*Thickness* affects the connection flexibility of a top arc seam sid-lap weld while the weld
length, $L_w$, may have less impact. This is consistent with arc spot welds (Standard Section D5.2.1.1) where weld size has no impact. The top arc seam sidelpad weld test data included ductile steels with a minimum $F_y = 31.9$ ksi (220 MPa) and a maximum $F_y = 54.2$ ksi (375 MPa) and lower ductility steel having a maximum tested value of $F_y = 105$ ksi (725 MPa). Therefore, the Standard’s connection flexibility equation applies over the acceptable range of Chapter D.

**D5.2.2 Screws Into Steel**

The Standard Eq. D5.2.2-1 is for thick supporting material and is adopted from SDI DDM04 (2015). The equation only considers the bearing deformation of the panel against screw. For thin supports, tilting can be considered rationally by linear interpolation between Standard Eqs. D5.2.2-1 and D5.2.2-2 that define the probable limits of $S_f$. This is a consideration in cold-formed steel framing. Since the original research determining $S_f$ did not include such supports, the following rational engineering judgments are provided.

For strength determined in accordance with Standard Section D1.1.2, the connection flexibility is determined as follows:

For $t_2 \geq t_3$, use Standard Eq. D5.2.2-1

For $t_2 < t_3$ and $t_2 \geq t_1$, linearly interpolate as shown in Figure C-D5.2.2-1

For $t_2 < t_1$, Standard use Eq. D5.2.2-2 based on $t = t_2$

where

$t_1 =$ panel thickness, in. (mm)

$t_2 =$ support thickness, in. (mm)

$t_3 =$ support thickness where $P_{nf}$ controlled by tilting or bearing of the screw against the support controls for the panel thickness, $t_1$ in. (mm)

$$S_f = \begin{cases} \frac{3\alpha}{1000t_1^{0.5}} & \text{if } t_2 \geq t_3 \\ \left(3 - 1.7 \frac{t_2 - t_1}{t_3 - t_1}\right) \left(\frac{\alpha}{1000t_1^{0.5}}\right) & \text{if } t_2 < t_3 \geq t_1 \\ \frac{1.3\alpha}{1000t_1^{0.5}} & \text{if } t_2 < t_1 \end{cases}$$

![Figure C-D5.2.2-1 Support Screw Flexibility](image)
The Standard’s sidelap connection flexibility Equation D5.2.2-2 is not limited to #12 screws and is commonly applied to #10 screws and #8 screws. SDI DDM04 (2015) includes nominal strength [resistance] research on #8 and #10 screws, and discusses that the slope of the load-slip curve was virtually constant for all diameters at lower loads. Choice of screw size, thread and point type depends on the application.

D5.2.3 Wood Screws or Nails Into Wood Supports

The Standard equations for determining the connection flexibility of screws or nails into a wood support are adopted from Luttrell (1999a).

Ss is not provided for nails, and nails typically are not used in sidelap connections that are not into the support.

D5.2.4 Power-Actuated Fasteners Into Supports

A designer can find the connection flexibility equations for specific power-actuated fasteners listed in SDI DDM04 (2015) and its appendices or can consult the fastener manufacturer for test data on these and other proprietary fasteners that conform to Standard Section D5.2.6.

D5.2.5 Non-Piercing Button Punch Fasteners at Steel Panel Sidelaps

The traditional non-piercing button punch requires a manual or automated crimping tool to draw a “dome-like” button into an interlocking top sidelap connection.

When determining $G'$, $n_s$ should not be neglected (even when it must be neglected for diaphragm nominal shear strength [resistance]) because the contribution of sidelap connections is greater at the service load level and can be depleted near ultimate load because of slip. Test data calibration for deep or cellular deck fits better when non-piercing button punch contribution is considered for diaphragm stiffness but neglected for diaphragm shear strength.

D5.2.6 Other Fasteners – Flexibility Determined by Tests

For fasteners not included in Standard Sections D5.2.1 to D5.2.5, the Standard permits connection flexibility to be determined through tests conducted in accordance with Standard Sections E1.1 and E1.2. AISI S905 includes connection flexibility test methods.

Support thickness dominates the support connection flexibility, $S_f$, even though other mechanical properties may affect connection flexibility as well. As support thickness approaches the sheet thickness, support connection flexibility, $S_f$, will approach $S_s$.

Several manufacturers have developed proprietary tools and sidelap connections to provide significant nominal strength [resistance] per connection, and some connections fully penetrate and fold the steel to form an interlock. These proprietary connection flexibilities must be tested in accordance with Standard Section D5.2.6.

D5.3 Stiffness of Cellular Deck

D5.3.1 Cellular Deck Without Perforations

The equations in Standard Section D5.3.1 are adopted from Luttrell (SDI, 2013). Bagwell (2008) evaluated the stiffness equation in an earlier edition, Luttrell (2005). Warping distortion in the bottom plate is negligible and tests indicate that warping in the top deck is
also negligible. The inherent torsional restraint of the closed cellular deck resists end warping and $D_n$ is not present in Standard Eq. D5.3.1-1. The bottom plate efficiently resists a significant part of the shear force. Standard Eq. D5.3.1-1 modifies Standard Eq. D5.1.1-1 and addresses both factors. The distribution of shear resistance between the bottom plate and top deck can be calculated based on shear deflection compatibility at the longitudinal lines of cellular deck connections. Standard Eq. D5.3.1-2 adjusts for load sharing and measures the shear flow and stress in the top deck. The numerator, $t$, of Standard Eq. D5.3.1-2 is based on the top deck thickness of the cellular deck.

Slippage at sidelap connections over and between supports can dominate deflection and depends on the connection flexibilities (and thicknesses) at the sidelap.

D5.3.2 Cellular Deck With Perforations

Perforations in either the bottom plate or top deck affect the shear distribution between the two elements. The more common condition is perforations in the bottom plate only. Luttrell (SDI, 2013) provided a method to calculate the shear distribution between top and bottom elements and the resultant $G'$. The method considers the increased shear strain due to perforations by calculating an equivalent increased element length for a non-perforated element. Standard Eq. D5.3.2-1 includes this method and reduces to Standard Eq. D5.3.1-2 when there are no perforations.

D5.4 Stiffness of Concrete-Filled Diaphragms

D5.4.1 Stiffness of Structural Concrete-Filled Diaphragms

The equations for alternative unit systems have been provided for structural concrete stiffness contribution, $K_3$, in the Standard. Standard Eq. D5.4.1-1 has additive components and is unit-sensitive. Therefore, a compatible unit system should be used. For example, if $f'_c = 3000$ psi and $d_c = 2.5$ in. are used in Standard Eq. D5.4.1-3a, $K_3 = 2377$ kip/in. Therefore, the U.S. Customary units that produce kip/in. ($E = \text{ksi and } t, s, d = \text{in.}$) should be used for the first term in Standard Eq. D5.4.1-1 or Standard Eq. D5.4.1-2. Similarly, if $f'_c = 21$ MPa and $d_c = 65$ mm are used in Standard Eq. D5.4.1-3b, $K_3 = 430,000$ kN/m, and the SI units that produce kN/m ($E = \text{MPa and } t, s, d = \text{mm}$) should be adopted for the first term. $C$ is unitless in Standard Eq. D5.1.1-2. In SI units, $G'$ often is adjusted and published as kN/mm.

Although the theories behind Standard Eqs. D5.4.1-1 and D5.4.1-2 apply to any support type that has a defined $S_f$ and allows calculation of $C$ per Standard Eq. D5.1.1-2, the research is based on tests of the structural concrete-filled diaphragms on hot-rolled shape supports. This includes both large-scale concrete slab tests and connection flexibility tests. The Standard limits applicability to shapes or steel joists whose top chord thicknesses are greater than or equal to 0.1 in. (2.5 mm). Testing is required for other cases.

The in-plane stiffness of concrete-filled deck diaphragms is high. However, an evaluation of the shear stiffness of the Porter test specimens shows that Standard Eq. D5.4.1-1 produces shear stiffness predictions that were 2.42 times larger on average than experimentally observed (O’Brien et al., 2017). A revised equation was proposed by O’Brien et al. (2017) that produces values that are 98% of the experimental stiffness on average. The equations

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are as follows:

\[ G' = \frac{E_t}{C_2} + K_4 \text{ for fluted deck} \]  

(C-D5.4.1-1)

\[ K_4 = \text{Stiffness contribution of the structural concrete fill} \]

\[ = 4.8t_e \sqrt{f'_c}, \text{ kip/in. for U.S. Customary units} \]  

(C-D5.4.1-2)

\[ = 0.40t_e \sqrt{f'_c}, \text{ kN/m for SI units} \]  

(C-D5.4.1-3)

\[ C_2 = \left( \frac{E_t}{w} \right) \left( \frac{2L}{2\alpha_3 + n_p\alpha_4} \right) S_t \]  

(C-D5.4.1-4)

The expressions provided above are recommended only if the structural response is sensitive to the potentially reduced stiffness.

D5.4.2 Stiffness of Insulating Concrete-Filled Diaphragms

The same analytical method is used for both structural and insulating concrete.

D6 Diaphragm Flexibility

Flexibility is the inverse of stiffness. Stiffness, \( G' \), or flexibility, \( F \), can be used to calculate in-plane deflection in accordance with Standard Section C3. The Commentary on Section E1.2 discusses the diaphragm system in the Tri-Service Manual (TM 5-809-10, 1985) as an acceptable method to determine diaphragm nominal strength [resistance] and stiffness by tests. The equations in the Tri-Service Manual provide diaphragm flexibility and are unit-sensitive.
E. DIAPHRAGM NOMINAL SHEAR STRENGTH [RESISTANCE] PER UNIT LENGTH AND STIFFNESS DETERMINED BY TEST

Standard Chapter E outlines the testing methods and test requirements. Testing objectives are discussed in Standard Section E1.2. The testing objective should establish the test matrix, and the test results should determine the applicable range of parameters, ultimate strength of diaphragm or diaphragm connections, and stiffness and flexibility of diaphragm or diaphragm connections, as required by the testing objective.

The available diaphragm shear strength [factored resistance] and stiffness can be based on tests in accordance with:

(a) Standard Section E1, which is used for a prototype diaphragm system based on an analytical method with considerations of parameters outlined in Commentary Section E1.2 and AISI S907; or

(b) Standard Section E2, which is for a single diaphragm system.

Standard Chapter E does not preclude acceptance of any products (or fasteners) and research or tests that preceded the Standard and thus might not conform to the Standard's requirements, where the research or tests were performed under the direction of an engineer in accordance with acceptable practices.

E1 Strength and Stiffness of a Prototype Diaphragm System

E1.1 Test Protocol

The tests should be based on existing AISI Test Standards wherever possible. Some ASTM Standards are permitted for small-scale tests for particular support connections into non-steel supports.

Seasoned and dry wood is required to eliminate the greater variation associated with wood to establish a baseline and to help isolate the contribution of other parameters. Design should consider reductions in nominal strength [resistance] and increases in connection flexibility for structural connections in wood supports due to less seasoning and greater moisture. Various reduction factors are provided in AWC NDS.

E1.2 Design Using Test-Based Analytical Equations

Existing diaphragm system methods (SDI, 2004; MCA, 2004; TM 5-809-10, 1982) are test-based and may be used to establish analytical equations or as starting points to extend the stated limits of these methods.

Since the existing analytical methods already have defined the contributions of many parameters and the controlling limit states, those parameters and limit states do not have to be considered in the development of test matrices unless the desired application range is outside the established limits. The number of tests to extend these limits typically is minimal, and calibration could include the entire testing database verifying the existing method plus any extension tests. AISI S100 Section J4 allows tests in lieu of analytical equations for screws. Standard Section E1.2 similarly allows testing of any fastener or connection even when the analytical method of Standard Chapter D applies. For some proprietary fasteners, the provisions in Standard Chapter D may be applicable. However, the manufacturer has the option to refine the nominal strength [resistance] or connection flexibility of the fastener by tests.
The *Standard* does not address the development of a new analytical method, but it does not exclude that option. Alternative analytical methods to establish *diaphragm nominal shear strength [resistance]* and *stiffness* equations should be developed under the supervision of an experienced professional and should be confirmed by sufficient tests. The method should define the controlling *limit states*, application limits of parameters, and *safety* and *resistance factors*. Since the number of required tests depends on testing objectives, the number of required tests to develop a new analytical method typically requires more tests than that required to extend an existing method. The *safety* and *resistance factors* for new analytical methods are determined in accordance with *Standard* Section E1.2.2. Alternative analytical methods and new tests should also meet building code requirements and be acceptable to the design professional and other *authorities having jurisdiction*.

In any analytical method, theoretically, all parameters must be considered. However, since some of the parameters may not be pertinent to the method or test scope, those parameters can be eliminated. A list of probable parameters is contained in *AISI S907*. Some parameters are included in each test assembly, such as span, profile, *thickness*, and mechanical properties. The contributions of these essential parameters are considered in tests that are constructed to evaluate other parameters. More than two parameters can be included in one test where their contributions are defined in an existing system method equation and the test combination for these parameters is within the method’s established limits. The contribution of parameters being purposely evaluated must be significant in the test or the result may be trivial. *Commentary* Table C-E1.2-1 summarizes the parameters whose contributions are defined in each of the listed method or model’s analytical equations.

Although other effects are possible, profile geometry can affect the following:

(a) The number, type and size of acceptable fasteners,
(b) The ability to end-lap,
(c) *Buckling resistance*, and
(d) The response to warping shear.

Fastener dimensions and mechanical properties might include but are not limited to shank dimensions, head or washer type, shank hardness, and *tensile strength*. The number of required tests will depend on the test objective; e.g., the desired application limits of the fastener in the analytical model.

Testing can establish either a constant, trivial or asymptotic value for certain parameters and the results can use those values in the *nominal strength [resistance]* or *flexibility* analytical equations, which define the *panel*, *support*, and *connection* interaction in the *diaphragm* system. It is acceptable to set a singular, i.e. minimum or maximum, value for the contribution of a parameter. Examples include:

(a) Constant *sidelap connection nominal shear strength [resistance]*, $P_{ns}$, for all values of $t$ and $F_{u}$, and
(b) The benefit of increasing $F_{u}$ for a particular fastener is determined to be negligible after some value of $F_{u}$.

A particular example of (a) is selecting a single value for $P_{ns}$ in *Standard* Section D1.2.6 for a non-piercing button punch.

Where the method of *Standard* Chapter D is used to calculate *diaphragm* shear strength and *stiffness* and all parameters (other than those *connections* to be tested) conform to *Standard* Chapter D, small-scale tests can be performed to define the *nominal strength [resistance]* and
flexibility of the connections that are not already defined. The connection strength and connection flexibility equations will include the contributions of the essential parameters listed in Standard Table E1.2-1, as applicable to the research scope. Although large-scale tests in accordance with AISI S907 and no small-scale tests are the acceptable option, the more common approach is to only use small-scale tests in accordance with Standard Sections D1.1.5 and D1.2.7. Standard Section E1.2.2(a) sets additional restrictions on the connection nominal strength [resistance] equation, so the confidence in connection strength equals or exceeds that of the Standard Chapter D analytical system. Application of Standard Section E1.2.2(a) requires reduction of the nominal strength [resistance] equation and recalibration where an initial calibration generates a resistance factor less than that of the diaphragm system for the same load effect and construction type. To use the limits of an existing method for a diaphragm system, the new connection test matrix must encompass those same limits.

To use the theory of an existing method with limited new large-scale tests, the mean, \( P_m \), of all large-scale test data leading to the original calibration of the theory plus any new test data should not shift significantly. This indicates that the accuracy of a newly developed support or sidelap connection nominal strength [resistance] equation is in line with other fastener equations provided in the existing method. This also indicates that:

(a) Inclusion of the new connection equation would not significantly affect the calibration leading to Standard Table B1.1-1 or other test-based theories, and

(b) The existing system safety and resistance factors are valid for the new data.

An analytical method includes the interaction of support and sidelap connections and the impact of the panel to define a diaphragm system's nominal shear strength [resistance] per unit length controlled by connections, \( S_{nf} \). The method also defines a separate limit state, the nominal shear strength [resistance] per unit length controlled by panel buckling, \( S_{nb} \). Existing analytical methods should not be mixed to calculate either limit state, \( S_{nf} \) or \( S_{nb} \), unless the combination is considered a new analytical method. However, when determining \( S_{nf} \), insertion of a test-based analytical equation for connection nominal strength [resistance] from one existing analytical method into another analytical method provides the same result as small-scale tests and is acceptable as long as the reliability of the borrowed connection equation is consistent with that of the primary analytical method and all other provisions of Section E1 are met. The stability analytical method leading to \( S_{nb} \) defines a separate limit state and does not mix methods if it is unchanged. Although it is rarely done, it should be acceptable to use \( S_{nf} \) from one existing model and \( S_{nb} \) from an alternate model as long as the correct safety or resistance factor is used for each limit state. Large-scale testing may still be required to confirm the system application in accordance with Section E1.

To minimize the number of tests or load tables for design applications, it is conservative to use lightweight structural concrete test results for normalweight structural concrete available strength [factored resistance] in design, or to use lesser specified concrete compressive strength, \( f'_{c} \), test results for greater compressive strength in design.
### Table C-E1.2-1
List of Parameters Defined in Common Analytical Methods

<table>
<thead>
<tr>
<th>Parameter</th>
<th>SDI &amp; MCA Method&lt;sup&gt;2&lt;/sup&gt;</th>
<th>Tri-Service Method&lt;sup&gt;1, 3&lt;/sup&gt;</th>
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<td>Equations for pre-qualified fasteners define limits</td>
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**Note:**

1. The Tri-Service method is empirical and provides the *allowable strength* [resistance]. The *safety factor* is 3 for *panels* with and without fill. The support weld requires an effective diameter, $d_e$ in D1.1.1, of ½ in. (13 mm) for arc spot welds or 3/8 × 1 in. (9.50× 25.4 mm) for arc seam welds. The empirical equation does not include *panel* $F_y$ or $F_u$ as parameters, and a minimum $F_y = 40$ ksi (275 MPa), and a minimum $F_u = 55$ ksi (380 MPa) are required. That material contribution is attributed to all steel in the equation.

2. Some end-laps were included in the large-scale tests, and this is partly covered in the value of $P_{nf}$. If the value $P_{nf}$ at end-laps is determined, then the method addresses the impact.

3. The empirical formulas are limited to particular *connections* and sizes. However, the methodology can be applied after the contributions for other *connections* are defined and verified by tests. This contribution is permitted to include fastener size and type, *panel thickness* and mechanical properties. With sufficient tests, lesser *safety factors* can be justified. However, the more common approach adopts the existing *factor* as discussed in the *Commentary* of this Section.

4. 3/16 in. (4.76 mm) minimum support thickness is required. Lesser thickness requires tests.
The *Tri-Service Manual* (Department of Army, 1985) provides design equations for determining allowable strength and flexibility of diaphragm configurations, and includes a system relationship between support and sidelap connection strengths. For diaphragm configurations whose parameters other than connection(s) conform to the limits of the analytical method given in the *Tri-Service Manual* (Department of Army, 1985), the following procedures can be followed to determine the connection strength and flexibility of a new connection to be used in the analytical method:

(a) Perform small-scale tests using AISI S905 to determine the nominal strength [resistance] and connection flexibility of all connections that are not already defined in the method. The connection strength and flexibility equations will include the contributions of the essential parameters as applicable to the research scope, including those that affect constant, K (in Section 5-6 of the *Tri-Service Manual* (TM 5-809-10, Eq. 5-12)) for support connections. K requires large-scale tests (see item (b) below) which determine constants C2 and C3 (in Section 5-6 of the *Tri-Service Manual* (TM 5-809-10, Eq. 5-13 and Eq. 5-14)) that are required for sidelap connections.

(b) Perform large-scale tests per AISI S907 to complete the test matrix, and analyze the results to determine the constants mentioned in (a).

(c) Modify the existing equations in the *Tri-Service Manual* for allowable diaphragm shears and flexibility factors so the test results conform to the Standard’s Sections E1.2.1 and E1.2.2(c). Units should be consistent in this analysis.

(d) Develop load tables and diaphragm flexibilities using the safety and resistance factors determined using Standard Section E1.2.2. Tables must state any application limits.

**E1.2.1 Test Assembly Requirements**

If diaphragm nominal shear strength [resistance] is controlled by a tested edge connection or detail, the interior panel diaphragm shear strength would not be established to evaluate analytical equations for the panel. This implies that test assembly details should isolate the system contribution of the diaphragm’s panels and connections by eliminating nonessential parameters or details that could otherwise control diaphragm nominal shear strength [resistance]. Partial-width panels should not be used to complete a test assembly unless the partial-width panel is fastened to supports and stitch-connected to the full-width panel to develop the same strength as a full-width panel and its connections. Standard Section D1 shows that the cover width impacts strength and stiffness. As an example, more side seams are present for slip to occur in partial-width or narrow panels, and the section modulus of the support connection cluster is smaller in narrow panels. The number of full-width panels should conform to the requirements of AISI S907.

A diaphragm test should consider that total shear (100%) has to flow into or out of the diaphragm through the longitudinal and transverse perimeter support connections. The analytical method in Standard Chapter D addresses this requirement using Eqs. D1-2 and D1-3 by checking the resistance to required shear flow at panel ends and edges. Eqs. D1-1 and D1-2 (where applicable) cover the field diaphragm panel connection resistance. When these limits plus out-of-plane buckling and local web buckling concerns are met, the panel design for tests is considered satisfactory.

Where the test objective is to establish the diaphragm strength per unit length of the edge detail, the edge condition should control the diaphragm failure. Otherwise, sufficient
connections should be provided parallel to the panel span to direct the strength limit to the diaphragm field. In lieu of edge detail testing, the perimeter or edge connection parallel to the panel span can be designed to transfer forces without limiting diaphragm nominal shear strength [resistance] or stiffness. Many engineers consider that the addition of more fasteners and thicker accessories at the edge does not significantly impact the total project installation cost, and this approach can avoid the edge detail’s control of the system resistance.

Where the tested nominal strength [resistance] is limited by a parallel edge connection or detail, the designer is cautioned to not apply the tested stiffness or strength to the entire diaphragm design. Local warping in flashings or accessories, support distortion or roll, and fastener slippage at perimeter details can provide greater proportionate impact relative to a few panels in a test frame than they will in a larger structure.

Each configuration has one set of parameters. Absolute repeatability of some parameters within different test specimens of similar configuration is not possible, but such parameters should be held reasonably close to study the contribution of other tested parameters. An example is specified concrete compressive strength, \( f'_c \), when determining the contribution of fill thickness while the impact of \( f'_c \) is not already defined by the method.

Where a new panel or fastener is being developed and end-laps are a feature of both the panel used in the test and the testing objective, the test should demonstrate that end-laps can be made and that connections can be installed. The connection nominal strength [resistance] at panel ends can be determined using an existing analytical method provided the connection strength equation is applicable for both end-laps and butt joints. Since designers may not know whether end-laps or butt joints will be installed on projects, it is rational to select the lesser nominal strength [resistance]. Manufacturers commonly use only one connection strength in load tables including interior support connections, and butt joint or end-lap support connections by using the least of all these support connection nominal strengths [resistance], \( P_{nf} \).

Parameters that are not defined by existing method equations require the minimum number of tests specified in AISI S907 or S905, as applicable, to establish each parameter’s contribution over the desired range. Historically, a minimum of three tests is required to establish linearity or non-linearity of contribution for each parameter, but this depends on the test objective and the desired application range. As long as the contribution of each is not trivial, more than one undefined parameter can be isolated and included in a configuration.

A large-scale test evaluates the system effect of connections and can detect the weakest link of the diaphragm system. Analytical methods include the system contribution of support and sidetap connections. The Commentary of AISI S907 provides minimum relative contribution requirements for connections to ensure that interaction is present and the contribution of each connection is measurable while nominal strength [resistance] equations are tested. The tested connections should be as specified for the test assembly. Some variance is acceptable at welds as long as the overall uniformity of connections remains. The Standard provides rules to establish reasonable uniformity.

If panel application is limited to single-span, single-span tests should be performed. Whenever possible, three or more test spans are preferred for confirmation of multiple-span diaphragms. Two-span test results could be permitted for multiple-span diaphragms
where available test frame size is limited. Many manufacturers apply three-span tables to a
greater number of spans for both diaphragm shear strength and stiffness. Most end warping
occurs at panel ends. Generally, the impact of end warping is greater in single- or double-
span tests than in three span tests when all the variables are kept the same other than panel
length, L. For this reason, direct use of the tested G' from a single- or double-span test is
conservative. However, if the existing model considers the variation of span number, G'
can be calculated when applying single-span test confirmations to multiple-span applications.

If a new analytical method is being developed and the impact of continuity is not
defined by the tests, the design engineer should consider the continuity impact using an
existing analytical method before applying single-span test results to multiple-span
applications or multiple-span test results to single-span applications.

**E1.2.2 Test Calibration**

Calibration of an analytical equation for a diaphragm system should be based on AISI
S100 Section K2.1.1(b). However, the number of tests required by the test standards listed
in Standard Section E1.1 for particular testing objectives could vary from the requirement in
AISI S100 Section K2.1.1(b) and Standard Section E1.1 controls. The calibration method
provided in Standard Section E1.2.2 is an application of AISI S100 Section K2.1.1(c) that is
specific to diaphragm systems. A detailed discussion of diaphragm-specific calibration is
included in Section B1.1. See AISI S100 Commentary Section B3.3 for a discussion of
probability analysis concepts and calibration of resistance factors.

AISI S100 Section K2.1.1(a) requires that deviation of any individual test result from the
average value obtained from all tests should not exceed 15 percent. When this is not
satisfied, additional testing is required. However, AISI S100 Section K2.1.1(a) only applies
in Standard Section E2. To develop analytical equations through tests, the Standard adopts
AISI S100 Section K2.1.1(b) with these modifications:

(a) Relax the scatter criterion due to repetition of components and contributing
parameters in each diaphragm test while recognizing the probable variance in the
large-scale test due to installation quality at these repeating components (weakest
link and redistribution potential), and

(b) Allow the calibration process to provide the necessary safety and resistance factors

with restrictions on $\frac{R_{t,i}}{R_{n,i}}$ while retaining the $C_c$ requirement of AISI S100 Section
K2.1.1(b).

Reasonable resistance factors have been obtained using this approach to verify that a
theory adequately predicts tested performance. Each large-scale test evaluates the same
analytical equation, so each test is a repeat verification. The AISI S100 Section K2.1.1(a)
requirement for three identical specimens does not apply in AISI S907 since there can be
many connections and panels in each test. Note that in AISI S100 Section K2.1.1(c), n is the
total number of large-scale tests in the test program verifying the analytical method. n is
not the total number of connections in all tests.

If large-scale tests are selected to verify an analytical method, $S_{n_{test}} / S_{n_{theory}}$ for each
test should be consistent with the tests used to develop Standard Table B1.1-1. Analysis of
the test data in DDM01 (SDI, 1981) reports using the theory of the Standard creates the
scatter of 0.58 to 2.03 while Bagwell (2008) reports the scatter of 0.81 to 1.39 for cellular deck. The ratio limit of 0.6 in Standard Eq. E1.2.2-1 ensures that most of the tests constructed in a laboratory provide more resistance than the theoretical factored strength, $\phi S_n$, that might be used in design, and avoids resistance equations significantly over predicting tested performance. This value also takes into consideration the historical scatter in diaphragm tests. The engineer in charge of testing must determine whether a testing anomaly exists to discount a value lower than 0.6. If an anomaly does not exist, the test should be repeated to determine if a flaw exists in the resistance equation and if the equation reasonably predicts all regions of the proposed parameter range. The engineer should determine if other tests that support the equation in the same range of parameters can offset a low ratio. By following the restrictions requiring a ratio greater than 0.6 and n conforming to AISI S907, the calibration process outlined in Standard Section E1.2.2 is consistent with the intent of AISI S100 Section K2.1.1(b).

The Standard requires that connections be as specified but recognizes that size variance will exist at welds. Therefore, $S_n$ theory is based on the average measured weld sizes at support and sidelaps, but the assembly weld size scatter must conform to the range limits of Standard Section E1.2.1. Use of the average is consistent with the diaphragm system effect and redistribution potential of that system.

The method of measuring deflection within a test will affect the calculated value of $G'_{i\text{ test}}$. $G'_{i\text{ test}}$ often is the secant value of a load-deflection curve in the lower range of loads. Since the difference in deflection values at defined loads is in the denominator and can be a very small number, very slight errors in deflection readings can have a large impact on $G'_{i\text{ test}}$. If deflections (leading to $G'_{i\text{ test}}$) or panel parameters (leading to $G'_{i\text{ theory}}$) are not measured accurately, an individual ratio, $\frac{G'_{i\text{ test}}}{G'_{i\text{ theory}}}$, could be less than 0.50. However, the Standard sets 0.50 as the lowest acceptable ratio to ensure a consistency between measured and calculated stiffness values, and most existing data conform to this requirement. Generally, tests should result in an average value of all $\frac{G'_{i\text{ test}}}{G'_{i\text{ theory}}}$ greater than 0.7 or the analytical method should be revised. Because of the potential scatter, a significant number of tests may be required to bring the desired balance, and it is reasonable to consider new tests verifying an existing theory as extensions of the previous tests. The new test results can be added to the published existing database. Tested or calculated $G'$ is, at best, a good approximation of stiffness in an actual structure’s diaphragm.

The target reliability index, $\beta_o = 3.5$, is used in LRFD and $\beta_o = 4.0$ is used in LSD because AISI S100 Section K2.1.1 requires this for connections. $\beta_o = 2.5$ is permitted when wind plus dead load causes diaphragm shear in LRFD and by extension in ASD. These $\beta_o$ options might not apply to steel deck with concrete fill or panels on wood support, and this is shown in Standard Table E1.2.2-1. For diaphragms with structural concrete fill, $\beta_o$ should not be less than the $\beta_o$ allowed for concrete shear in ACI 318.

The research by Nowak and Szerszen (2003) reported that for structural concrete slabs, $F_m = 0.92; V_F = 0.12; P_m = 1.02; V_p = 0.06; M_m = 1.35$ to 1.12; and $V_M = 0.102$ to 0.042 for ready mix concrete 28-day cylinders with $f'_c = 3$ ksi and $f'_c = 6$ ksi, respectively. However,
in a separate study relating in-situ $f'_c$ to 28-day cylinder $f'_c$, Petersons (1968) suggests that the in-situ strength is approximately 90% of the cylinder $f'_c$. Tabsh (1997) reports $\beta_o = 3$ for structural concrete slabs in shear. Statistical data, $F_m = 0.9; V_F = 0.10; M_m = 1.1$; and $V_M = 0.10$, are selected based on these reports recognizing that the diaphragm consists of steel deck and connections in addition to concrete fill. The other calibration values, $P_m$ and $V_p$, are calculated from the test data.

Structural concrete fill dominates diaphragm nominal shear strength [resistance]. However, the system $\beta_o$ is conservatively chosen to be greater than or equal to that allowed for concrete slabs and that allowed for steel connections in AISI S100.

Standard Eqs. D4.3-1 and D4.3-2 indicate that both deck and lightweight insulating concrete fill contribute to diaphragm shear strength. The contribution of each component to the total strength can be significant. AISI S100 Table K2.1.1-1 lists statistical data for determination of resistance factors using AISI S100 Eq. K2.1.1-2 for various deck connections. The statistical parameters for insulating concrete fill should not deviate greatly from those of deck connections. For simplicity, the deck connections statistical data in AISI S100 Table K2.1.1-1 are used to calibrate the combination of deck and lightweight insulating concrete. Consistent with AISI S100 Section K2.1.1, statistical data can otherwise be determined by statistical analysis.

The statistical data in Standard Table E1.2.2-1 for wood-supported diaphragms are determined based on rational engineering after a review of Rosowsky (2005) and Bulleit (2007). The data in Standard Table E1.2.2-1 provide reasonable agreement with Standard Section D1.1.4.1.

In large-scale tests for extending or verifying existing theories, $C_p$ may be taken as 1.0 because the entire theory database can be used to define $n$ in AISI S100 Eq. K2.1.1-4. The Commentary of AISI S907 includes a discussion of the historical and extensive testing performed on existing analytical methods.

**E1.2.3 Laboratory Testing Reports**

When developing analytical equations, two separate reports may be produced:

1. Laboratory Testing Report

   This report provides the required information defined in AISI S905 and AISI S907. The size and nominal strength [resistance] requirements for connections include the visible diameter for welds and the nominal tensile strength [resistance] and hardness for screws and power-actuated fasteners. When welds are used, report electrode size, tensile strength and type, weld settings, and welding time. Note air gaps, ambient conditions, support thickness, and whether qualified welders were used. Also, note whether large-scale tests conform to the prequalification procedures used in small-scale tests if applicable.

   Weld procedures in tests do not define the exact procedure required for construction site installations because those conditions can be entirely different from laboratory conditions. Often, welder preferences and prequalification procedures required by the applicable building code will establish what is required for suitable welds on construction sites.
(2) Engineering Report

This report justifies the analytical equation(s), responds to the requirements of Standard Section E1, and provides information for future researchers, designers and building officials. This report might include:

(a) Developed analytical equations,
(b) Calculated nominal shear strength [resistance] and stiffness for each test using the analytical equation,
(c) Table of \( \frac{S_{ni,\text{test}}}{S_{ni,\text{theory}}} \) and \( \frac{G'_{i,\text{test}}}{G'_{i,\text{theory}}} \) for each test,
(d) Calculated safety and resistance factors or statement that established system factors apply,
(e) Applicable range limits of the developed equation(s), and
(f) Certification that the equation development and calibration conform to the Standard and were performed under the direction of a professional engineer.

**E2 Single Diaphragm System**

A single diaphragm system is defined in Standard Section A1.3. A single diaphragm system has one configuration with no variation in construction parameters.

**E2.1 Test System Requirements**

The specified diaphragm system must be tested whether isolating a detail or verifying the field of diaphragm construction. The limits in Standard Sections E2.1(b) and E2.1(c) recognize that tested mechanical properties typically vary from specified values. However, tested values should be reasonably close to those specified, and repeated tests should have consistent materials and construction. Adjustments are required in Standard Section E2.4 to account for variance of tested material properties from specified properties. Material controls and design adjustments are consistent with AISI S100 Section A3. Because the tests repeat the same construction and installation techniques, and the material is taken from the same batches where applicable, repeatability should be more readily achievable than testing in Section E1. Therefore, the test deviation requirements of AISI S100 Section K2.1.1(a) are imposed.

A “support representative of the design” does not have to be the same member size that will be used on a particular project, but the tested dimension should be adequate so that the same type of failure will occur at the connection; e.g., bearing of panel against and slotting around the fastener. The requirement to test dry and seasoned wood supports does not apply in Standard Section E2. The wood support should be representative of the design and application.

**E2.1.1 Fastener and Weld Requirements**

Size variation should be expected in the installation of specified welded connections. However, care should be taken to closely achieve the desired weld sizes in test construction. A prequalification procedure is suggested to achieve the desired sizes. The Standard provides acceptable limits and requires adjustment based on the smallest
measured dimensions. Design based on a set of smallest values recognizes distribution potential and that the smallest connections should be the weakest link. The number of smallest welds, 10, is a rational judgment. This approach slightly differs from the average value that is used to calculate $S_{ni,\text{theory}}$ in Standard Section E1.2.1. Standard Section E1.2.1 is testing a theory, while Standard Section E2.1.1 is defining the nominal shear strength [resistance] and stiffness of a particular diaphragm construction. Slightly greater quality control is also required in single diaphragm system tests.

**E2.1.2 Concrete Requirements**

Depth and compressive strength variation relative to specified values should be expected at structural or insulating concrete fill (slabs) over deck in individual or repeat tests. The Standard provides acceptable variations. The structural concrete $f'_{c}$ lower limit is consistent with ACI 318.

Perimeter connections that transfer shear are critical and typically collect shear from the concrete through chemical bond or steel-headed stud anchor shear resistance. Chemical bond is critical at diaphragm edge panels and in the diaphragm field. The test curing time of seven days was chosen to provide a rational minimum time for undisturbed bonding.

**E2.2 Test Calibration**

Testing of a single diaphragm system involves limited tests and does not establish all parameter contributions (as variables) or limit state thresholds. Since safety and resistance factors associated with connection failures are typically more severe than those controlled by panel out-of-plane buckling, it is possible to overstate the available shear strength [factored resistance] for a tested configuration that is controlled by panel buckling and not controlled by connection failure. The question is: when would connection failure have occurred if panel buckling had not occurred? This requires that the diaphragm system safety and resistance factors controlled by connections be applied to a tested shear strength controlled by buckling. As an example, a 9-foot span is tested and panel out-of-plane buckling controls. However, connection-controlled failure also might be imminent. If a lower safety factor is applied because buckling controls, this might provide a greater available strength [factored resistance] value than if an 8-foot span is tested and connections control failure, which might require a greater safety factor. This is consistent with Standard Section B1.1. However, if structural analysis using an established method indicates that connection failure is not imminent and that the available strength [factored resistance] controlled by connections will exceed the available strength [factored resistance] controlled by buckling and the buckling limit is reasonably confirmed by test, it should be acceptable to apply the buckling safety and resistance factor to the test results.

If the tested configuration falls within the acceptable limits of an existing system model, it is rational to accept the existing safety and resistance factors in lieu of the factors based on three to six tests as long as the test results fit the normal scatter of tests based on the existing model. In the absence of such data, the scatter discussed in the Commentary of Section E1.2.2 can be used for the models listed in Commentary Sections E1.2 and A4.

**E2.3 Laboratory Testing Reports**

Standard Section E2.3 adopts the test report requirements in AISI S907. See the
Commentary on Section E1.2.3.

**E2.4 Adjustment for Design**

When an analytical method, such as Chapter D, that includes the impact of specified design parameters is not used in design but a single diaphragm system test is used for design, the Standard requires that the test results be adjusted for the specified design parameters.

**E2.4.1 Diaphragms Without Structural Concrete Fill**

Standard Section E2.4.1 extends the concept that design is based on the design thickness and the minimum specified mechanical properties of steel. Standard Section E2.1 requires that the specified design values be tested while recognizing the normal variance of ordered material properties. The calibration process uses the normal increase of material properties. Material factors in AISI S100 Table K2.1.1-1 already assume that $F_y$ and $F_u$ will be at least 10% greater than the specified minimum $F_y$ and $F_u$. The reduction multiplier does not penalize tested values within the norm. The reduction is consistent with the design equations in AISI S100 that relate connection resistance to $F_u$ rather than $F_y$. Tested properties should be relatively close to the design values so linear reductions are applied to most parameters to establish nominal strength [resistance]. The exception is concrete-filled diaphragms where the concrete fill shear contribution is historically proportional to the square root of $f_c$. The contribution of insulating concrete is additive to the contribution of steel, both components are significant, and $f_c$ should be reasonably close to design values, even though limits are not set on the $f_c$ deviation. Adjustments for lightweight insulating concrete fill rationally address both contributions, but theoretically might not be precise since the adjustment assumes that the contribution of the deck equals the contribution of the insulating concrete fill (or one-half the total strength).

**E2.4.2 Diaphragms With Structural Concrete Fill**

Structural concrete often dominates diaphragm shear strength and deck connections contribute strength, to a lesser extent, in the field of diaphragms. The concrete over the top of deck, $d_c$, and compressive strength, $f_c$, are most critical to diaphragm shear strength. Consequently, reductions are required for the concrete contribution without adjustments for other tested properties when the other properties are reasonably close to the specified properties. No limits are imposed on $f_{c_{\text{test}}}$ The Standard imposes limits on concrete screed quality control and implicitly directs the testing engineer to aim at $d_{c_{\text{test}}}$ greater than $d_c$ to avoid penalty. When a parameter is within the Standard’s limits but less than the specified value, an increase in nominal shear strength [resistance] is not allowed for that parameter. The reduction functions are consistent with the strength contribution in Standard Section D4.2.

**E2.5 Test Results Interpretation**

Test frame size can limit tests to a single (simple)-span condition. Commentary Section E1.2 outlines existing analytical models that are acceptable in tests such as the analytical model
presented in *Standard* Section D1. Such models indicate that with everything else being equal, single-span nominal shear strength [resistance] is greater than multiple-span strength. A 50 percent increase in the number of fasteners at *interior supports* can be determined as the maximum required increase to balance multiple-span and single-span shear strengths based on *Standard* Section D1. When *sidelap connection* contribution is significant, the required increase in *support connection* quantity can be negligible.

A weld kernel or the shank of a fastener at end-laps has the same tributary length as that of an *interior support connection*, while at service load, butt joint *connections* have the same tributary length as *connections* in simple spans. Near nominal strength redistribution occurs and each connection contributes as much as it can so tributary length has less meaning. To simplify field installation and to overcome quality control concerns at end-laps, *Standard* Section E2.5 extends the *interior support* increase to *exterior supports* with lapped ends. At *exterior supports* (see Figure D1-1) where panels have butt joints or are at perimeter ends, design applications could require the same number of *connections* as the single-span test.

The design provisions in accordance with *Standard* Section D1 indicate the theoretical difference in resistance between single- and multiple-span *diaphragms* and can, therefore, be used to determine the additional required number of *interior support connections* so the multiple-span *diaphragms* provide the same nominal strength [resistance] as the single-span *diaphragm* system. If the single-span test results reasonably confirm the single-span theoretical result, the designer might increase the number of *interior support connections* and apply the test results to multiple-span applications.

The required number of *support connections* depends on the number and type of *sidelap connections*. A detailed discussion is presented in Example 4 of the AISI Design Guide (2017). See Figure C-E2.5-1 for an illustration of how to fulfill an analysis requirement to increase fasteners at *interior supports* by approximately 50% to justify the use of the single-span tested strength.

The required location of additional fasteners at *interior supports* is consistent with *Standard* Section D1 since those *support connections* furthest from the *panel* centerline provide the greatest benefit in the determination of $\alpha_p^2$ and $S_n$.

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**Figure C-E2.5-1** Fasteners Required in Multiple-Span Application Based on Single-Span Test of a Single Diaphragm System

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APPENDIX 1: DETERMINATION OF FACTORS, $D_n$ AND $\gamma_c$

1.1 General

1.1.1 Scope

The factors, $D_n$ and $\gamma_c$, are necessary to calculate the stiffness, $G'$.

1.1.2 Applicability

This appendix applies to perforated and non-perforated fluted panels. Warping, associated with $D_n$, is negligible in cellular deck and diaphragms with fill.

1.2 Determination of Warping Factor, $D_n$

For a given shear stress, more shear and warping displacements will occur in longer elements of a profile with open cross-section. Those impacts have been considered in the generalized stiffness equation presented in Standard Eq. D5.1.1-1 where the shear deformation impact is considered in the first term of the denominator and warping deformation impact is considered in warping factor, $D_n$.

Since end warping is restrained by structural or insulating concrete fill, $D_n$ does not appear in Standard Eqs. D5.4.1-1 or D5.4.2-1. In Standard Eq. D5.3.1-1, cellular deck requires a modifier for material shear displacement due to load sharing between the bottom plate and top deck, and torsional restraint in profiles with closed cross-section makes $D_n$ negligible. A condensed text is presented in Commentary Appendix 1 Section 1.4 and summarizes the development of $D_n$.

Perforations in any profile element reduce the shear and flexural stiffness of that element. A shear stiffness change impacts the shear displacement of the element (indicated by the first term in the denominator of Standard Eq. D5.1.1-1), while a flexural stiffness change (resisting transverse racking) impacts the profile warping factor, $D_n$, i.e., the second term in the denominator. Standard Section D5.1.1 allows calculation of these impacts by determining the equivalent lengths of an unperforated element for shear displacement and end warping respectively. These modified lengths are substituted in Standard Eq. D5.1.1-1. The warping adjustment is discussed in Commentary Appendix 1 Section 1.6.

Table C-1.1 presents dimensions and Table C-1.2 presents the warping parameter, $D$, for generic deck profiles. $D_n$ is related to $D$ as shown in Standard Eq. 1.4-1. $D_n$ is dimensionless but $D$ has units as shown in Standard Appendix 1 Section 1.4, so $L$ must be in the same units as $D$. The panel manufacturers can use product-specific dimensions and Standard Appendix 1 Section 1.4 to determine $D_n$ for other profiles.

Note in the Tables C-1.1 and C-1.2:

WR = Wide rib and commonly called B deck
IR = Intermediate rib and commonly called F deck
NR = Narrow rib and commonly called A deck
DR = Deep rib and commonly called N deck

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### Table C-1.1a
Profile Dimensions (Customary Units in.)

<table>
<thead>
<tr>
<th>Type</th>
<th>D_d</th>
<th>w</th>
<th>d</th>
<th>2e</th>
<th>f</th>
<th>s</th>
</tr>
</thead>
<tbody>
<tr>
<td>WR</td>
<td>1.47</td>
<td>1.53</td>
<td>6.00</td>
<td>1.56</td>
<td>3.56</td>
<td>8.19</td>
</tr>
<tr>
<td>IR</td>
<td>1.47</td>
<td>1.59</td>
<td>6.00</td>
<td>0.53</td>
<td>4.24</td>
<td>7.95</td>
</tr>
<tr>
<td>NR</td>
<td>1.47</td>
<td>1.51</td>
<td>6.00</td>
<td>0.36</td>
<td>4.99</td>
<td>8.36</td>
</tr>
<tr>
<td>DR</td>
<td>3.00</td>
<td>3.07</td>
<td>8.00</td>
<td>1.49</td>
<td>5.24</td>
<td>12.86</td>
</tr>
</tbody>
</table>

Note: Table C-1.1 column headers are shown in *Standard* Appendix 1, Figure 1.4-1, and *Standard* Section D2.

### Table C-1.1b
Profile Dimensions (SI Units mm)

<table>
<thead>
<tr>
<th>Type</th>
<th>D_d</th>
<th>w</th>
<th>d</th>
<th>2e</th>
<th>f</th>
<th>s</th>
</tr>
</thead>
<tbody>
<tr>
<td>WR</td>
<td>37.3</td>
<td>39.0</td>
<td>152</td>
<td>39.7</td>
<td>90.5</td>
<td>208</td>
</tr>
<tr>
<td>IR</td>
<td>37.3</td>
<td>40.5</td>
<td>152</td>
<td>13.4</td>
<td>108</td>
<td>202</td>
</tr>
<tr>
<td>NR</td>
<td>37.3</td>
<td>38.3</td>
<td>152</td>
<td>9.10</td>
<td>127</td>
<td>212</td>
</tr>
<tr>
<td>DR</td>
<td>76.2</td>
<td>77.9</td>
<td>203</td>
<td>37.8</td>
<td>133</td>
<td>327</td>
</tr>
</tbody>
</table>

### Table C-1.2
D Values

<table>
<thead>
<tr>
<th>Roof Deck Type</th>
<th>t</th>
<th>Valley Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Each</td>
</tr>
<tr>
<td></td>
<td>in.</td>
<td>in.</td>
</tr>
<tr>
<td>WR</td>
<td>0.0295</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>0.0358</td>
<td>0.91</td>
</tr>
<tr>
<td></td>
<td>0.0474</td>
<td>1.20</td>
</tr>
<tr>
<td></td>
<td>0.0598</td>
<td>1.52</td>
</tr>
<tr>
<td>IR</td>
<td>0.0295</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>0.0358</td>
<td>0.91</td>
</tr>
<tr>
<td></td>
<td>0.0474</td>
<td>1.20</td>
</tr>
<tr>
<td></td>
<td>0.0598</td>
<td>1.52</td>
</tr>
<tr>
<td>NR</td>
<td>0.0295</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>0.0358</td>
<td>0.91</td>
</tr>
<tr>
<td></td>
<td>0.0474</td>
<td>1.20</td>
</tr>
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<td>1.52</td>
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<tr>
<td>DR</td>
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<td>0.75</td>
</tr>
<tr>
<td></td>
<td>0.0358</td>
<td>0.91</td>
</tr>
<tr>
<td></td>
<td>0.0474</td>
<td>1.20</td>
</tr>
<tr>
<td></td>
<td>0.0598</td>
<td>1.52</td>
</tr>
</tbody>
</table>
Standard Section D2 and Appendix 1 Section 1.4 show the required units. Where those units are used and the method of Appendix 1 is applied, the SI value of D is mm. For convenience of size, D is converted to m in Table C-1.2. Some valley spacing in Table C-1.2 is not recommended for serviceability in roofs and exceeds industry standards, while other spacing physically is not possible because of product pitch and available cover widths. Typical roof fastener spacing is limited to each or alternate corrugations, with three being a maximum. Table C-1.2 values are included to illustrate that numbers can be determined and can be used as a computer program check. Fasteners spaced at every fourth corrugation are quite possible in non-composite form decks. Table C-1.2 does not limit thickness; other thicknesses are permitted.

1.3 Determination of Support Factor, $\gamma_c$

The support factors are based on tests and taken from SDI DDM01 (SDI, 1981).

1.4 Determination of Warping Factor Where Insulation Is Not Present Beneath the Panel

Commentary on Diaphragm Panel End Warping by Luttrell:

Open corrugated or fluted steel panels are produced with individual panel widths containing flutes as shown in Standard Figure 1.4-1. Panels may be several feet (m) long and installed over supports using fasteners through the panel’s bottom flanges. Under in-plane shear loading, the connected diaphragm lower flanges receive direct loads from the frame while top elements are loaded by shears moving through supporting webs, w. The top flange typically is not connected to supports at panel ends and the section can roll over in torsion from the uneven loading. This allows the top flange at the panel end to move laterally, i.e., perpendicular to the panel span.

Diaphragm stiffness is defined as $G' = \frac{S a}{A}$

where:
- $S$ = specified average shear level,
- $a$ = system width, and
- $A = A_S + A_D + A_C$.

In order, these deflection components are material shear displacement, shear relaxation from warping, and slip at fasteners.

The $A_D$ expression was first developed prior to publishing the Steel Deck Institute Diaphragm Design Manual, First Ed., in 1981. This warping expression involves a fourth order differential equation and up to five interconnected horizontal panel elements acting as beams on elastic foundations. The materials in the successive groups of equations presented in Standard Appendix 1.4 represent the general warping solution. Specific warping D values from this solution are listed in Table C-1.2 for certain standard deck profiles.

Referring to Standard Appendix 1 Figure 1.4-1, s is the developed width per pitch, d. A unit length of the panel of Standard Appendix 1 Figure 1.4-1 is considered as a frame under horizontal unit load applications. For any pair of designated points, i and j, deflections can be established as $\delta_{ij}$ and read, for example, as “deflection at point 1 due to a unit load at point 2.” Here, 2 represents a point at the right edge of the f element and 1, a point at middle of the lower flange, of width 2e, to the right of the figure. For a single flute with a fastener at the bottom left
and a roller support at the right, unit loadings lead to $\delta_{11}$, $\delta_{12}$, and $\delta_{22}$ as defined in Standard Eq. 1.4-8 through Eq. 1.4-10.

The combined $\delta_{ij}$ terms can be used to describe spring constants that indicate the resistance of horizontal panel elements to lateral movement under load. With $n = 2$ marking the case of fasteners in alternate valleys, the top spring constants are in the form $K_{12} = \kappa_{12} (EI)$ for a panel thickness, $t$, where $EI = \frac{E(b)^3}{12}$ and $b$ is a unit length of panel, 1 in. or 1 mm as applicable. Subscript, $b_2$, marks a bottom flange condition where $n = 2$. This model leads to Standard Eq. 1.4-11 through Eq. 1.4-20.

The released end restraints lead to top displacements, $\delta_{tn}$, and bottom displacements, $\delta_{bn}$, where $n$ is 1, 2, 3, or 4 for fasteners in each valley, every second valley, every third valley, or every fourth valley. These displacements are used to measure the energy associated with the transverse restraints. Combining forms for $n = 1, 2, 3, 4$ valley spacing, this model leads to Standard Eq. 1.4-21 through Eq. 1.4-30.

For applications where the panel has mixed end fastener conditions, $U_1$ of the total panel width has $D_1$ warping; $U_2$ of the width has $D_2$ warping; $U_3$ has $D_3$ warping; and $U_4$ has $D_4$ warping, the mixed warping effect $D$ then is the weighted average in accordance with Standard Eq. 1.4-2.

For example, a 36 in. (915 mm)-wide panel with 6 in. (152 mm)-wide flutes (6 corrugations) has a leading edge fastener, with the next fastener being 6 in. (152 mm) (1 corrugation) from the edge; the next fastener is at 12 in. (305 mm) (2 corrugations) from the last one; and the far edge fastener 18” (457 mm) (3 corrugations) beyond the previous. In this example: $U_1 = 1$, $U_2 = 2$, $U_3 = 3$, $U_4 = 0$ and the sum $U_1 + U_2 + U_3 = 6$ or 6 corrugations per panel width. Then apply Standard Eq. 1.4-2:

$$D = \frac{6D_1 + 12D_2 + 18D_3}{36} = \frac{D_1 + 2D_2 + 3D_3}{6}$$

The approach presented here is adaptable to a spreadsheet application from which warping $D$ values may be easily established.

1.5 Determination of Warping Factor Where Insulation is Present Beneath the Panel

The method of Standard Appendix 1.5 is an approximation of Standard Appendix 1.4 and is based on a parametric study by Luttrell (MCA, 2001), which is published in A Primer on Diaphragm Design (MCA, 2004). There is relatively good agreement, particularly at $\psi = 2$ and 3. Some agreement accuracy is lost at $\psi = 4$ and this case is excluded. Fasteners at every fourth corrugation are rare but can occur in shallow product with relative small pitch such as concrete form deck. Even here, concrete fill is normally installed and the end warping concern only applies without fill.
The greatest difference occurs in the $D_1$ term (Standard Eq. 1.4-3/L vs. Standard Eq. 1.5-2), but $D_1$ is normally small and not dominant in the calculation of $G'$. There are three terms in the denominator of $G'$ and as $D_n$ decreases, the impact on $G'$ is less.

The parametric study did not include perforated panels. When perforated panels are present, use Standard Appendix 1.6. However, when the manufacturer says the impact of perforations is negligible, it is reasonable to use Standard Appendix 1.5.

### 1.6 Determination of Warping Factor for Perforated Deck

Developed by Luttrell, SDI (2011) presents a method to determine the impact of perforations on shear displacement and end warping. This method was adopted in the Standard to calculate $G'$.

The reduced stiffness of perforated elements can affect the three components in the denominator of Standard Eq. D5.1.1-1 and the two components in Standard Eq. D5.3.1-1. The profile parameters, $D_d$ and $d$, in Standard Figure 1.4-1 are not modified in the Standard’s equations. The modified values of $e_p$, $f_p$, $w_p$ are not used in Standard Eq. 1.4-12 through Eq. 1.4-34. The equations assume that the perforation pattern, and thus $k$, will be a constant in all elements of the profile.

$\rho_o$ can be obtained from the panel manufacturer or by using tables published by the Industrial Perforators Association (IPA) for the profile’s perforation pattern. For the common 60-degree staggered pattern:

$$\rho_o = 0.9069 \frac{d_p^2}{c_p^2}$$  \hspace{1cm} (C-1.6-1)

where

- $d_p =$ Perforation hole diameter
- $c_p =$ Hole center-to-center spacing

An example is: $d_p = 0.188$ in. (4.78 mm) and $c_p = 0.375$ in. (9.53 mm) leading to $\rho_o = 0.228$ and $k = 0.524$. The IPA test-based limits on $\rho_o$ are imposed in the Standard. Common products use $\rho_o$ close to 0.1. Rational engineering suggests that for $\rho_o$ less than 0.2:

$$k = 1 - 2.175\rho_o$$ \hspace{1cm} (C-1.6-2)
For the *web* shown in Figure C-1.6-1:

\[ A_w = \frac{W_p}{w} \]  \hspace{1cm} (C-1.6-3)

where

\[ A_w = \text{Ratio to perforated width (in web) to the full element (web) width.} \]

See *Standard* Eq. 1.6-4. \((i = w \text{ at } A_i)\)

\[ W_p = \text{Out-to-out perforation band width in web} \]
\[ w = \text{Point-of-intercept to point-of-intercept web width} \]

Similarly in *Standard* Appendix 1.6:

\[ A_e = \frac{F_p}{2e} \quad (i = e \text{ at } A_i) \quad \& \quad A_f = \frac{F_p}{f} \quad (i = f \text{ at } A_i) \]

The impact of perforations on the other components in the denominator of *Standard* Eq. D5.1.1-1 is discussed in *Commentary* Section D5.1.2.

The *Standard’s* method to calculate the impact of perforations on \(D_n\) and \(G’\) is consistent with SDI (2011). If perforations are not at fastener locations and only located in the bottom plates of cellular deck, the effect of perforations on \(G’\) is negligible. In this case, \(C\) (*Standard* Eq. D5.1.1-2) and \(s\) (in *Standard* Eq. D5.3.1-2) are not affected. Perforations that are only in the webs of deck (non-cellular) can have negligible effect on \(D_n\) and \(G’\) where \(p_o\) and \(W_p\) are small.
APPENDIX 2: STRENGTH AT PERIMETER LOAD DELIVERY POINT

The required load must be transmitted to a diaphragm to develop the available shear strength \[\text{factored resistance}\]. The Standard appendix considers the development of additional shear in connections at a load delivery point where the perimeter edge is perpendicular to the panel length. Sometimes drag struts are provided to transmit load, and this is analogous to stiffeners below concentrated loads in plate girders. When the perimeter supports along the diaphragm length are not sufficiently stiff and struts are not provided to relieve bending in the supports, compression can be developed in the panel to transmit the load and develop shear.

The design should consider the combined bending and axial stress interaction in the panel. The Standard appendix provides a rational design method based on AISI S100 and considers both axial compression and tension. The compression or tension consideration is analogous to checking local bearing or pull on the web of a beam or plate girder due to a large concentrated load. This sometimes requires transverse stiffeners in beams. The condition in diaphragms sometimes requires extra perimeter fasteners and requires a check of the panel as a column strut and a consideration of eccentric end moment in the panel.

The definition of limited weak axis bending is not precise and requires judgment. Simplistically, use the theoretical deflection of the support at the upper bound shear transfer, \(w_a\) (or per connection, \(w_a/\sqrt{N}\)), and the connection flexibility response as a first iteration to estimate the part transmitted to the panel in compression. The panel normally has sufficient capacity to transfer load, and additional fasteners at the perimeter do not significantly impact the overall installation. If there is doubt, check the effect by neglecting any transverse load transfer to the spandrel beam in bending and use Standard Eq. 2.2-1 as applicable.

Obtain the required profile properties from the manufacturer. For common profiles, visit SDI DDM04 (2015) Appendix V for a reasonable approximation of \(I_{xg}\) and Commentary Table C-1.1 for \(s\), as defined in Standard Eq. D2-5, so that \(A_g\) can readily be determined. Since manufacturers commonly publish \(I_x\) to calculate deflection based on the effective widths at the stress caused by service loads, many designers substitute this value for \(I_{xg}\).
REFERENCES


Luttrell, L.D. (1971), “Shear Diaphragms with Lightweight Concrete Fill,” International Specialty Conference on Cold-Formed Steel Structures, Rolla, MO.


