



AISI S310-23



# AISI STANDARD

## **North American Standard for the Design of Profiled Steel Diaphragm Panels**

2023 Edition



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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.

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## PREFACE

The American Iron and Steel Institute Committee on Specifications has developed AISI S310-23, the 2023 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 standard sections have been reorganized to improve ease of use.
- The specific section for standing seam panels has been removed, but the design is covered under test-based design.
- The sections for double skin panels and the appendix for combined shear and axial loading case have been moved to design examples (not included).
- Connection strength of panels to steel supports is in accordance with AISI S100.
- Wood screw connection strength is in accordance with the ANSI/NDS, *National Design Standard for Wood Construction*.
- Test-based design follows the provisions of AISI S100.
- Flexibility of connections and connection strength not in included AISI S100 have been moved to Appendix 1.

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  
December 2023

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### SYMBOLS AND DEFINITIONS

$A$	= Number of exterior support <i>connections</i> per flute located at the <i>sidelap</i> at an <i>interior panel</i> or <i>edge panel end</i>	D1.1
$A_a$	= Material shear deformation component for <i>cellular panel</i> or <i>perforated cellular panel</i>	D3.2, D4.2, F2.2
$A_e$	= Ratio of bottom perforated width to the bottom width	D2.1
$A_f$	= Ratio of top perforated width to the top width	D2.1
$A_i$	= Ratio of perforated width to the full element width	D2.1
$A_v$	= Cross-sectional area per unit width, $b$ , of reinforcement oriented parallel to the direction of shear, in. <sup>2</sup> (mm <sup>2</sup> )	E1.1
$A_w$	= Ratio of web perforated width to the <i>web</i> width	D2.1
$b$	= Unit width of <i>diaphragm</i> with structural concrete fill, 12 in. for U. S. customary units and 1000 mm for SI units	E1.1
$C$	= Slip constant considering slippage at <i>sidelap connections</i> and distortion at support <i>connections</i>	D1.2, D3.2, D4.2, F1.2, F2.2
$C_f$	= Calibration coefficient	G2.2
$c_p$	= Hole center-to-center spacing	D2.1
$d$	= <i>Panel</i> corrugation pitch, in. (mm)	D1.1, D1.2, E1.1, E1.2, F1.2
$d$	= Top profiled <i>panel</i> corrugation pitch, in. (mm)	D3.2
$d'$	= Equivalent width of <i>cellular panel</i> perforated bottom plate adjusted for perforations and measured between longitudinal rows of fasteners connecting the top <i>profiled panel</i> to the bottom plate, in. (mm)	D4.2
$d_c$	= Structural concrete thickness above top of profiled <i>panel</i> , in. (mm)	E1.2, F1.1, F1.2
$D_d$	= Depth of <i>panel</i> , in. (mm)	D1.1, E1.2
$D_n$	= Unitless warping factor considering distortion at <i>panel ends</i> determined using the parameters as shown in Figure D1.1-1	D1.2
$d_o$	= Hole diameter in washer, in. (mm)	1.2.3.1
$d_p$	= Perforation hole diameter	D2.1
$e$	= One-half the bottom flat width of <i>panel</i> measured between points of intercept as illustrated in Figure D1.1-1, in. (mm)	D1.1

$E$	= Modulus of elasticity of steel, ksi (MPa)	D1.2, E1.1, E1.2, F1.2, F2.2
$e$	= Distance from the top <i>profiled panel</i> longitudinal fastener to the web, in. (mm)	D3.2
$E_c$	= Modulus of elasticity of concrete in accordance with ACI 318, ksi (MPa)	E1.1, E1.2
$E_p$	= Width of perforation band in the bottom flat of width, $2e$ , in. (mm)	D2.2
$f$	= Top flat width of panel measured between points of intercept illustrated in Figure D1.1-1, in. (mm)	D1.1, D2.1, D3.2
$f'_c$	= Specified concrete structural compressive strength, psi (MPa)	E1.1, E1.2, F1.1, F1.2
$f'_c$	= Material shear deformation component for <i>perforated cellular panel</i> determined in accordance with Section D4.2	F2.2
$F_m$	= Mean value of fabrication factor, $F$ , determined by statistical analysis or where applicable, as limited by AISI Table K2.1.1-1 for type of component involved	G2.2
$F_p$	= Width of perforation band in the top flat of width, $f$ , in. (mm)	D2.2
$F_{xx}$	= Tensile strength of electrode classification, ksi (MPa)	1.2.3.1
$F_y$	= Design <i>yield stress</i> as determined in accordance with AISI S100 Section A3.3.1	D1.1
$F_{ys}$	= Specified minimum <i>yield stress</i> of reinforcement, ksi (GPa)	E1.1
$G'$	= <i>Diaphragm shear stiffness</i> , kip/in. (kN/m)	F1.2, B3.2.4, B3.3, B3.7, D1.2, D2.2, D3.2, E1.2, E2.2, F1.2, G2.2
$h$	= Flat dimension of web measured in <i>plane of web</i>	D1.1, D2.1
$i$	= Index of perforated elements in a profile	D2.1
$I_{xg}$	= Moment of inertia of fully effective <i>panel</i> per unit width, in. <sup>4</sup> /ft (mm <sup>4</sup> /mm)	D1.1
$K$	= <i>Stiffness</i> factor relating support and <i>sidelap connection</i> flexibilities	D1.2
$k$	= Ratio of the perforated element stiffness relative to that of a solid element	D2.1, D2.2
$K_3$	= <i>Stiffness</i> contribution of the insulating concrete fill	F1.2, F2.2



$k_b$	= Ratio of perforated element <i>stiffness</i> relative to that of a solid element in the perforated bottom plate of <i>cellular panel</i> of the same thickness, $t_b$ , and determined in accordance with Eqs. D2.1-5 and D2.1-6	D4.2
$k_c$	= Factor for structural concrete strength	E1.1
$L$	= Total panel length	D1.1, D1.2
$L_v$	= Span of <i>panel</i> between supports with fasteners, ft (m)	D1.1
$L_w$	= Length of top arc seam <i>sidelap</i> weld, in. (mm). See AISI S100 Figure J2.4.1-1 for details.	1.2.4.1
$M_m$	= Mean value of material factor, $M$ , determined by statistical analysis or, where applicable, as limited by AISI S100 Table K2.1.1-1 for type of component involved	G2.2
$N$	= Number of support fasteners per unit width at an interior or edge panel's end	D1.1, D3.1
$N_c$	= Average number of support fasteners per unit width	E1.1
$n_d$	= Number of support <i>connections</i> at the flute bottom relative to the tributary width, $w_t$ , along the ends of interior or <i>edge panels</i> and into exterior supports	D1.1
$n_e$	= Number of edge support <i>connections</i> between transverse supports and along an <i>edge panel</i> length, $L$	D1.1
$N_{ext}$	= Bearing length at exterior support (3/4 in. (19 mm) minimum)	D1.1
$n_p$	= Number of interior supports along a total <i>panel</i> length, $L$	D1.1, D1.2
$n_s$	= Number of <i>sidelap connections</i> along a total <i>panel</i> length, $L$ , and not into supports	D1.1, D1.2
$n_{sc}$	= Modular ratio of steel <i>profiled panel</i> to concrete	E1.1
$P_n$	= <i>Nominal shear strength [resistance]</i> of arc spot weld with washer, kip (kN)	1.2.3.1
$P_{nf}$	= <i>Nominal shear strength [resistance]</i> of a support <i>connection</i> per fastener determined in accordance with Appendix 1	D1.1, D3.1
$P_{nf}$	= <i>Nominal shear strength [resistance]</i> of a support <i>connection</i> per fastener determined in accordance with AISC 360 for welded steel headed stud anchors or Appendix 1 as applicable	E1.1

$P_{nf}$	= Nominal shear strength [resistance] of a support connection per fastener in the absence of a tensile load	1.4.1
$P_{nfs}$	= Nominal shear strength [resistance] of an edge support connection installed parallel with an edge panel span and between transverse supports	D1.1
$P_{nft}$	= Nominal shear strength [resistance] of a support connection per fastener in the presence of a tensile load	1.4.1
$P_{not}$	= Nominal tension strength [resistance] of a support connection per power-actuated fastener controlled by pull-out	1.4.1
$P_{nov}$	= Nominal tension strength [resistance] of a support connection per power-actuated fastener controlled by pull-over	1.4.1
$P_{ns}$	= Nominal shear strength [resistance] of a sidelap connection per fastener determined in accordance with Appendix 1	D1.1
$p_o$	= Ratio of the area of perforations to the total area in the perforated band	D2.1
$q_s$	= Perforated web adjustment factor	D1.1
$R$	= Inside bend radius	D1.1
$s$	= Developed flute width per pitch, in. (mm)	D1.1, D1.2, E1.1, E1.2, F1.2
$s$	= Developed flute width of top profiled panel in a cellular panel in accordance with Eq. D1.1-15	D3.2
$s'$	= Developed flute width of top profiled panel per width, $w_d$ , in a cellular panel in accordance with Eq. D2.2-1	D4.2
$S_{ni}$	= Nominal shear strength [resistance] per unit length of diaphragm or wall diaphragm controlled by connections at interior panels or edge panels	D1.1
$S_{nc}$	= 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	D1.1
$S_{ne}$	= 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	D1.1
$S_{np}$	= Nominal shear strength [resistance] per unit length of diaphragm or wall diaphragm controlled by connections	D1.1

along the ends of *interior panels* or *edge panels* and into exterior supports

$S_{no}$	= Nominal shear strength [resistance] per unit length of diaphragm or wall diaphragm controlled by panel out-of-plane buckling, kip/ft (kN/m)	D1.1
$S_{nl}$	= Nominal shear strength [resistance] per unit length of diaphragm or wall diaphragm controlled by panel local web buckling over exterior support	D1.1
$S$	= Connection shear flexibility, in./kip (mm/kN)	1.2.1, 1.2.2, 1.2.4.1, 1.3.1, 1.5.2, 1.6.2
$S_f$	= Structural support connection shear flexibility determined in accordance with Appendix 1, in./kip (mm/kN)	D1.2
$S_s$	= Sidelap connection shear flexibility determined in accordance with Appendix 1, in./kip (mm/kN)	D1.2
$t$	= Base metal thickness of the panel, in. (mm)	D1.1, D1.2, F1.2
$t$	= Base steel thickness of top profiled panel in cellular panel, in. (mm)	D3.2, E1.1, E1.2, F1.2
$t$	= Total combined base steel thickness of panel involved in shear transfer above the shear transfer plane, in. (mm)	1.2.1, 1.2.2, 1.2.3.1, 1.5.2, 1.6.2
$t$	= Total combined base steel thickness (exclusive of coatings) of sheets beneath the washer and above the shear transfer plane, in. (mm)	1.2.3.1
$t$	= Base steel thickness of thinner connected sheet, in. (mm)	1.2.4.1
$T$	= Required allowable tensile strength of a support connection per fastener determined for ASD loads and load combinations in accordance with Section A3	1.4.1
	= 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	1.4.1
$t_1$	= Thickness of member in contact with screw head or washer, in. (mm)	1.3.1
$t_2$	= Thickness of member not in contact with screw head or washer, in. (mm)	1.3.1

$t_a$	= Average thickness of structural concrete, calculated as the cross-sectional area of the structural concrete over one <i>profiled panel</i> divided by the width of the <i>profiled panel</i> , in. (mm)	E1.1
$t_b$	= Base steel <i>thickness</i> of bottom plate in <i>cellular panel</i> , in. (mm)	D3.2
$t_e$	= Equivalent transformed concrete thickness, in. (mm)	E1.1
$U_1$	= Number of corrugations having fasteners in every valley across the <i>panel</i> width, $w$	D1.2
$U_2$	= Number of corrugations having fasteners in every second valley across the <i>panel</i> width, $w$	D1.2
$U_3$	= Number of corrugations having fasteners in every third valley across the <i>panel</i> width, $w$	D1.2
$U_4$	= Number of corrugations having fasteners in every fourth valley across the <i>panel</i> width, $w$	D1.2
$V_F$	= Coefficient of variation of fabrication factor listed in AISI S100 Table K2.1.1-1 for type of component involved	G2.2
$V_M$	= Coefficient of variation of material factor listed in AISI S100 Table K2.1.1-1 for type of component involved	G2.2
$V_Q$	= Coefficient of variation of load effect	G2.2
$w$	= <i>Panel</i> cover width	D1.1, D1.2
$w$	= Web flat width of <i>panel</i> measured between points of intercept illustrated in Figure D1.1-1, in. (mm)	D1.1
$w$	= <i>Web</i> flat width of top <i>profiled panel</i> measured between points of intercept illustrated in Figure D1.1-1, in. (mm)	D3.2
$w_d$	= Distance measured across the width and between longitudinal rows of fasteners connecting the top <i>profiled panel</i> to the bottom plate, in. (mm)	D3.2, D4.2
$w_{dp}$	= Total width of perforation bands in bottom plate width, $w_d$ , in. (mm)	D4.2
$w_e$	= <i>Panel</i> cover width at the <i>edge panel</i>	D1.1
$W_p$	= Width of perforation band in <i>web</i> flat of width, $w$	D2.1, D2.2

$w_t$	= Tributary width to any given bottom flute with support <i>connection(s)</i> along the end perpendicular to the <i>panel</i> span and located at exterior support	D1.1
$x_e$	= Distance from <i>panel</i> centerline to an exterior support <i>connection</i> in a <i>panel</i>	D1.1, D1.2
$x_{ee}$	= Distance from <i>panel</i> centerline to an exterior support <i>connection</i> in an <i>edge panel</i>	D1.1
$x_p$	= Distance from <i>panel</i> centerline to an interior support <i>connection</i> in a <i>panel</i>	D1.1, D1.2
$x_{pe}$	= Distance from <i>panel</i> centerline to an interior support <i>connection</i> in an <i>edge panel</i>	D1.1
$Z'$	= LRFD-adjusted lateral design value, $Z$ , in accordance with NDS	1.6.1
$\alpha$	= Conversion factor for units	D1.1, 1.2.1, 1.2.2, 1.2.4.1, 1.3.1, 1.5.2, 1.6.2
$\alpha_3$	= Measure of exterior support fastener group distribution across a <i>panel</i> width	D1.2
$\alpha_4$	= Measure of interior support fastener group distribution across a <i>panel</i> width, $w$ , at an <i>interior panel</i>	D1.2
$\alpha_5$	= Conversion factor for units	1.2.4.1
$\beta$	= Factor defining <i>connection</i> contribution and interaction to diaphragm shear strength per unit length	D1.1
$\beta_o$	= Target reliability index	G2.2
$\delta_{bi}$	= Lateral displacement indicator at bottom of corrugation for valley fastener cases, $i = 1$ to $4$ , in. <sup>2.5</sup> (mm <sup>2.5</sup> )	D1.2
$\delta_{ij}$	= Deflection indicator of profile racking per unit load per unit length required for $D$ , in. <sup>3</sup> (mm <sup>3</sup> )	D1.2
$\delta_{ti}$	= Lateral displacement indicator at top of corrugation for valley fastener cases, $i = 1$ to $4$ , in. <sup>2.5</sup> (mm <sup>2.5</sup> )	D1.2
$\phi$	= Lateral <i>resistance factor</i> in accordance with NDS	1.6.1
$\phi_{tot}$	= <i>Resistance factor</i> for a <i>power-actuated fastener</i> controlled by pull-out and determined by test using AISI S100 Chapter K	1.4.1

$\phi_{\text{tov}}$	= Resistance factor for a power-actuated fastener controlled by pull-over	1.4.1
$\gamma_c$	= Support factor for warping determined in accordance with Table D1.2-1	D1.2
$\gamma_i$	= Final displacement indicator at top of corrugation for valley fastener cases, $i = 1$ to 4, in. <sup>2.5</sup> (mm <sup>2.5</sup> )	D1.2
$\lambda$	= Connection strength reduction factor at corner fastener, unitless	D1.1
$\mu$	= Poisson's ratio for steel	D1.2, F1.2
$\Omega_{\text{tot}}$	= Safety factor for a power-actuated fastener controlled by pull-out and determined by test using AISI S100 Chapter K	1.4.1
$\Omega_{\text{tov}}$	= Safety factor for a power-actuated fastener controlled by pull-over	1.4.1
$\theta$	= Angle between plane of web and plane of bearing surface, $45^\circ \leq \theta \leq 90^\circ$	D1.1
$\lambda_{\text{LW}}$	= Factor for lightweight concrete	E1.1

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## NORTH AMERICAN STANDARD FOR THE DESIGN OF PROFILED STEEL DIAPHRAGM PANELS

### A. GENERAL PROVISIONS

This chapter addresses the scope and applicability of the *Standard*, lists the definitions of the terminology used, summarizes referenced specifications, codes, and standards, and provides requirements for materials.

The chapter is organized as follows:

A1 Scope, Applicability, and Definitions

A2 Reference Specifications, Codes, and Standards

A3 Materials

#### A1 Scope, Applicability, and Definitions

##### A1.1 Scope

This *Standard* applies to the design of in-plane shear strength and *shear stiffness* for *panels* and associated fasteners commonly used as the stressed skin of

- (a) *Diaphragms*, and
- (b) *Wall diaphragms* (shear walls).

This *Standard* does not address determination of *available strength* [*factored resistance*] for the other components in a *diaphragm* or *wall diaphragm* system.

This *Standard* shall apply to *panels* that are installed:

- (c) With or without insulation between the *panel* and the support,
- (d) With or without concrete fill over the *panel*,
- (e) With or without perforations in the *panel*, and
- (f) With structural supports made of steel, wood, or concrete.

This *Standard* does not preclude the use of other materials, assemblies, structures or designs if the other materials, assemblies, structures or designs demonstrate equivalent performance for the intended use of those specified in this *Standard*. Where there is a conflict between this *Standard* and other reference documents, the requirements contained within this *Standard* govern.

##### A1.2 Applicability

This *Standard* includes Symbols, Chapters A through G, and Appendix 1.

This *Standard* includes design provisions for *Allowable Strength Design (ASD)*, *Load and Resistance Factor Design (LRFD)*, and *Limit States Design (LSD)*. These design methods shall apply as follows:

- *ASD* and *LRFD* – the United States and Mexico, and
- *LSD* – Canada.

In this *Standard*, bracketed terms are equivalent terms that apply particularly to *LSD*.

The in-plane *nominal shear strength* [*resistance*] and *shear stiffness* of *diaphragm* or *wall diaphragm profiled panels* or *cellular panels* shall be determined in accordance with Chapters C through G and Appendix 1 of this *Standard*.

### 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.

Terms designated with + are common AISC-AISI terms that are coordinated between the two standards developers.

#### General Terms

*Cellular Panel*. Assemblage of one or more *profiled panels* or *profiled panel* and flat sheet interconnected to form a built-up section.

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

*Connection Shear 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.

*Diaphragm*. Roof, floor, or other membrane or bracing system acting to transfer in-plane forces to the *lateral force-resisting system*.

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

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

*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 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.

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

*Nominal Shear Strength*. The in-plane shear strength of the *profiled panel*, including the contribution of *connections* to support members and adjacent *panels*.

*Panel*. A product conforming to the definition of *profiled panel*, *cellular panel*, *perforated panel* or *perforated cellular panel* without or with concrete fill.

*Perforated Cellular Panel*. Assemblage of a *profiled panel* with another *profiled panel* or flat sheet including perforations.

*Perforated Profiled Panel*. *Profiled panel* with perforations.

*Profiled Panel*. Product formed from a single steel sheet into fluted profiles with top and bottom flanges connected by *web* members.

*Shear Stiffness*. The in-plane resistance to deformation of a panel, including the *connections*, measured by the ratio of the applied force to the corresponding displacement.

*Sidelap*. *Joint* at which adjacent *panels* contact each other along a longitudinal edge.

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

a support. Also called a *stitch connection*.

*Structural Analysis*+. Determination of *load effects* on members and *connections* based on principles of structural mechanics.

*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.

*Support Connection*. A *connection* with a fastener or weld attaching one or more sheets to supporting members.

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

*Wall Diaphragm*. A wall, bearing or nonbearing, designed to resist lateral forces acting in the plane of the wall. Commonly referred to as a “vertical *diaphragm*” or “*shear wall*”.

#### **A1.4 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)).

## **A2 Referenced Specifications, Codes and Standards**

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

1. American Iron and Steel Institute (AISI), 25 Massachusetts Avenue, NW, Suite 800, Washington, DC 20001:

AISI S100-16(R2020)w/S3-22, *North American Specification for the Design of Cold-Formed Steel Structural Members With Supplement 3*

AISI S240-20, *North American Standard for Cold-Formed Steel Structural Framing*

AISI S400-20, *North American Standard for Seismic Design of Cold-Formed Steel Structural Systems*

AISI S904-17, *Standard Test Methods for Determining the Tensile and Shear Strength of Screws*

AISI S905-17, *Test Standard for Cold-Formed Steel Connections*

AISI S907-17, *Test Standard for Cantilever Test Method for Cold-Formed Steel Diaphragms*

2. ASTM International (ASTM), 100 Barr Harbor Drive, P.O. Box C700, West Conshohocken, PA, 19428-2959:

ASTM C33/C33M-18, *Standard Specifications for Concrete Aggregates*

ASTM C330/C330M-23, *Standard Specification for Lightweight Aggregates for Structural Concrete*

ASTM C332-23, *Standard Specification for Lightweight Aggregates for Insulating Concrete*

ASTM C869-09, *Standard Specification for Foaming Agents Used in Making Preformed Foam for Cellular Concrete*

ASTM D1761-20, *Standard Test Methods for Mechanical Fasteners in Wood and Wood-Based Materials*

ASTM E488/E488M-22, *Standard Test Methods for Strength of Anchors in Concrete Elements*

ASTM E1190-21, *Standard Test Methods for Strength of Power-Actuated Fasteners Installed in Structural Members*

ASTM F1667-21a, *Standard Specification for Driven Fasteners: Nails, Spikes, and Staples*

ASTM A615/A615M-22, *Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement*

ASTM A706/A706M-22a, *Standard Specification for Deformed and Plain Low-Alloy Steel Bars for Concrete Reinforcement*

ASTM A1064/A1064M-22, *Standard Specification for Carbon-Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete*

ASTM A1022/A1022M-22a, *Standard Specification for Deformed and Plain Stainless Steel Wire and Welded Wire for Concrete Reinforcement*

### **A2.1 Referenced Specifications, Codes and Standards for United States and Mexico**

1. American Concrete Institute (ACI), 38800 Country Club Dr., Farmington Hills, MI 48331:  
ACI 318-19(22), *Building Code Requirements for Structural Concrete*
2. American Institute of Steel Construction (AISC), One East Wacker Drive, Suite 700, Chicago, IL 60601-1802:  
ANSI/AISC 360-22, *Specification for Structural Steel Buildings*
3. American Society of Civil Engineers (ASCE), 1801 Alexander Bell Drive, Reston, VA 20191-4400:  
ASCE/SEI 7-22, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*
4. American Welding Society (AWS), 550 N. W. LeJeune Road, Miami, FL 33135:  
ANSI/AWS D1.1/D1.1M-20, *Structural Welding Code – Steel*  
ANSI/AWS D1.3/D1.3M-18, *Structural Welding Code – Sheet Steel*
5. American Wood Council, 1111 Nineteenth Street, NW, Suite 800, Washington, DC 20036:  
ANSI/AWC NDS-2018, *National Design Specification (NDS) for Wood Construction*  
TR-12, *General Dowel Equations for Calculating Lateral Connection Values*, 2015

### **A2.2 Referenced Specifications, Codes and Standards for Canada**

1. CSA Group, 178 Rexdale Boulevard, Toronto, Ontario, Canada, M9W 1R3:  
CAN/CSA A23.3-04(R2010), *Design of Concrete Structures*  
CAN/CSA 086:19, *Engineering Design in Wood*  
CAN/CSA S16:19, *Design of Steel Structures*  
W47.1:19, *Certification of Companies for Fusion Welding of Steel*  
W55.3-08 (R2018), *Certification of Companies for Resistance Welding of Steel and Aluminum*  
W59-13, *Welded Steel Construction (Metal Arc Welding)*
2. National Research Council of Canada (NRC), 1200 Montreal Road, Bldg. M-58, Ottawa, Ontario, Canada, K1A 0R6:  
*National Building Code of Canada (NBCC)*, 2020

## **A3 Materials**

*Panels* and cold-formed steel supports shall conform to the material requirements of AISI

S100, Section A3.

1. *Insulating concrete* aggregate shall conform to ASTM C332.
2. Foaming agent for lightweight cellular *insulating concrete* shall comply with ASTM C869.

#### **A3.1 Materials for United States and Mexico**

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.

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

#### **A3.2 Materials for Canada**

Hot-rolled steel supports shall conform to the material requirements of CSA S16.

Wood supports shall conform to the material requirements of CSA 086.

*Structural concrete* shall conform to the material requirements of CSA A23.3.

## B. DESIGN REQUIREMENTS

This chapter addresses general requirements.

The chapter is organized as follows:

- B1 General Provisions
- B2 Loads and Load Combinations
- B3 Design Basis
- B4 Dimensional Limits and Considerations
- B5 Panel Properties
- B6 Fabrication and Erection
- B7 Quality Control and Quality Assurance
- B8 Evaluation of Existing Diaphragm Panels

### B1 General Provisions

The design of cold-formed steel *diaphragm panels* shall be consistent with the intended behavior of the *diaphragm* in the structure and the assumptions made in the structural analysis.

### B2 Loads and Load Combinations

Loads and load combinations shall be as stipulated by the *applicable building code*. Where no building code is stipulated, the loads, load combinations, and nominal loads [specified loads] shall be those stipulated as follows:

- (a) In the United States and Mexico, *ASCE/SEI 7, Minimum Design Loads and Associated Criteria for Buildings and Other Structures*; and
- (b) In Canada, *National Building Code of Canada*.

### B3 Design Basis

No applicable strength or serviceability *limit state* shall be exceeded when the structure is subjected to the applicable load combinations.

Design shall be in accordance with the following methods:

- (a) *ASD* or *LRFD* – the United States and Mexico; and
- (b) *LSD* – Canada.

#### B3.1 Required Strength [Effect Due to Factored Loads]

The *required strength* [effect due to *factored loads*] of *panels* and *connections* shall be determined by *structural analysis* for the appropriate load combinations as stipulated in Section B2.

The *required strength* [effect due to *factored loads*] shall be noted as follows:

$$\begin{aligned} \bar{R} &= \text{Required strength [effect due to factored loads]} \\ &= R \text{ in accordance with ASD load combinations} \\ &= R_u \text{ in accordance with LRFD load combinations} \\ &= R_f \text{ in accordance with LSD load combinations} \end{aligned}$$

## B3.2 Design for Strength

Panels and their connections shall be designed to have strength such that the *available strength* [*factored resistance*],  $R_a$ , equals or exceeds the *required strength* [effect due to *factored loads*].

Design for strength shall be in accordance with:

- (a) Section B3.2.1 for *Allowable Strength Design (ASD)*,
- (b) Section B3.2.2 for *Load and Resistance Factor Design (LRFD)*, or
- (c) Section B3.2.3 for *Limit States Design (LSD)*.

### B3.2.1 Allowable Strength Design (ASD) Requirements

The *required strength* shall not exceed the allowable *diaphragm* shear in accordance with Equation B3.2.1-1 for ASD:

$$R \leq R_a \quad (\text{Eq. B3.2.1-1})$$

where

$$R_a = \frac{R_n}{\Omega} \quad (\text{Eq. B3.2.1-2})$$

$R$  = *Required strength* for ASD

$R_a$  = *Allowable strength*

$R_n$  = *Nominal shear strength* per unit length of *diaphragm* system as specified in Chapter D or E

$\Omega$  = *Safety factor* for *diaphragm strength* specified in Chapters C, D, E, F and G

### B3.2.2 Load and Resistance Factor Design (LRFD) Requirements

The *required strength* shall not exceed the factored *diaphragm* shear in accordance with Equation B3.2.2-1 for LRFD:

$$R_u \leq R_a \quad (\text{Eq. B3.2.2-1})$$

where

$$R_a = \phi R_n \quad (\text{Eq. B3.2.2-2})$$

$R_u$  = *Required strength* for LRFD

$R_a$  = *Design strength*

$R_n$  = *Nominal shear strength* per unit length of *diaphragm* system as specified in Chapter D or E

$\phi$  = *Resistance factor* for *diaphragm strength* specified in Chapters C, D, E, F and G

### B3.2.3 Limit States Design (LSD) Requirements

The effect of *factored loads* shall not exceed the factored *diaphragm* shear resistance in accordance with Equation B3.2.3-1 for LSD:

$$R_f \leq R_a \quad (\text{Eq. B3.2.3-1})$$

where

$$R_a = \phi R_n \quad (\text{Eq. B3.2.3-2})$$

$R_f$  = *Effect of factored loads* for LSD

$R_a$  = *Factored resistance*



- $R_n$  = Nominal *shear resistance* per unit length of *diaphragm* system as specified in Chapter D or E
- $\phi$  = *Resistance factor* for *diaphragm resistance* specified in Chapters C, D, E, F and G

### B3.2.4 Shear Deflection Requirements

The design shear deflection shall not exceed the permissible shear deflection in the *applicable building code* in accordance with Equation B3.2.4-1. In the absence of a building code, shear deflection requirements shall be permitted to be determined in accordance with:

- a) In the United States and Mexico, ASCE/SEI 7, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*; and
- b) In Canada, *National Building Code of Canada*.

$$\delta_n \leq \delta_a \quad (\text{Eq. B3.2.4-1})$$

where

$\delta_n$  = Predicted *diaphragm* deflection at the design load

$\delta_a$  = Maximum permissible *diaphragm* deflection

Deflection,  $\delta_n$ , is determined using *stiffness* analytical methods. *Diaphragm shear stiffness*,  $G'$ , of the *panel* shall be determined in accordance with Chapter D, E, F or G.

### B3.3 Design for Diaphragm Panels

The *nominal shear strength [resistance]*,  $S_n$ , and *diaphragm shear stiffness*,  $G'$ , shall be established by analysis (engineering mechanics) or test. The *nominal [resistance]* and *available strength [factored resistance]* of steel *diaphragm panels* that meet specified geometric and material limitations shall be permitted to be determined in accordance with Chapters D, E, F and G, as applicable.

### B3.4 Design of Connections

The strength of *connection* elements subject to shear or combined shear and tension shall be designed in accordance with the provisions of AISI S100 Chapter J. Connecting elements not covered by AISI S100 shall be designed in accordance with Appendix 1 of this *Standard*. The forces and deformations used in design shall be consistent with the intended performance of the *connection* and the assumptions used in structural analysis. Self-limiting inelastic deformations of the *connections* are permitted.

The elastic *stiffness* of *connection* elements shall be designed in accordance with the provisions of Appendix 1 of this *Standard*.

The strength and *stiffness* of *connections* outside the scope of AISI S100 Chapter J or Appendix 1 shall be determined by test in accordance with AISI S100 Chapter K.

#### User Note:

It is important to consider the combined effects that induce *diaphragm* shear in the plane of the panels in combination with out-of-plane wind uplift (outward wind effect) on the *panels*. The *connections* of the *panels* to supports will be subject to both shear effects from the *diaphragm* action and tension from the outward wind.

### **B3.5 Design for Stability**

Stability for *floor diaphragms*, *roof diaphragms* and *wall diaphragms (shear wall)* using *panels* as the stressed skin shall be in accordance with AISI S100 Chapter C.

### **B3.6 Design of Floor, Roof and Wall Diaphragm Assemblies**

The design of *floor diaphragm*, *roof diaphragm* and *wall diaphragm (shear wall)* assemblies shall be in accordance with this *Standard* and the applicable materials standards for the adjoining members or materials.

### **B3.7 Design for Serviceability**

The building structure shall be designed to perform its required functions during its expected life. Serviceability *limit states* shall be selected based on the intended function of the structure and shall be evaluated using realistic loads and load combinations. The in-plane shear deflection of *floor diaphragms*, *roof diaphragms*, or *wall diaphragm (shear wall)* assemblies using *panels* and associated support framing shall be determined with engineering mechanics using loads as specified by the *applicable building code* or project requirements and the *diaphragm shear stiffness,  $G'$* , provisions of Chapter D, E, F or G. The shear deflection for the *panels* is also permitted to be determined by analysis following the provisions of Chapter C for both linear elastic design or non-linear design.

### **B3.8 Design for Fatigue**

Fatigue need not be considered for seismic effects or for the effects of wind loading on typical building lateral force-resisting systems and building enclosure components. Fatigue shall be considered in accordance with AISI S100 Chapter M for cold-formed steel structural members and their *connections* subject to repeated loading within the elastic range of *stresses* of frequency and magnitude sufficient to initiate cracking and progressive failure.

### **B3.9 Design for Corrosion Effects**

Where corrosion may impair the strength or serviceability of a structure, structural components shall be protected against corrosion or shall be designed to tolerate corrosion.

## **B4 Dimensional Limits and Considerations**

Dimensional limits and considerations shall be as specified in Chapters D, E and F.

## **B5 Profiled Panel Properties**

Properties of strength, gross (full) cross-sections and reduced cross-sections (cross-sectional area, moment of inertia, section modulus, radius of gyration, etc.) shall be determined in accordance with AISI S100 with conventional methods of structural design.

For perforated *panels*, the section properties shall be based on analysis or testing.

**B6 Fabrication and Erection**

(Reserved)

**B7 Quality Control and Quality Assurance**

(Reserved)

**B7.1 Delivered Minimum Thickness**

The delivered minimum *thickness* shall be in accordance with AISI S100.

**B8 Evaluation of Existing Diaphragm Panels**

(Reserved)

**C. PROFILED PANEL DIAPHRAGM BY ANALYSIS**

The determination of *diaphragm shear strength* and *shear stiffness* of *profiled and cellular panels*, *profiled and cellular panels with structural concrete fill* and *profiled and cellular panels with insulating concrete fill* by rational engineering analysis is permitted.

## D. PROFILED PANEL DIAPHRAGM

This chapter addresses the *diaphragm shear strength* and *diaphragm shear stiffness* for *profiled panels* and *cellular panels* without concrete fill.

The chapter is organized as follows:

D1 Profiled Panels

D2 Perforated Profiled Panels

D3 Cellular Panels

D4 Perforated Cellular Panels

D5 Profiled Panels with Connections Through Insulation

### D1 Profiled Panels

This section shall apply to *profiled panels* without perforated elements and within the following limits:

- (a)  $0.5 \text{ in. (12 mm)} \leq \text{profiled panel depth} \leq 7.5 \text{ in. (191 mm)}$ ,
- (b)  $0.014 \text{ in. (0.35 mm)} \leq \text{base profiled panel thickness} \leq 0.075 \text{ in. (1.91 mm)}$  for depth less than or equal to 3.0 in. (76.2 mm),  
 $0.034 \text{ in. (0.85 mm)} \leq \text{base profiled panel thickness} \leq 0.075 \text{ in. (1.91 mm)}$  for depth greater than 3.0 in. (76 mm),
- (c)  $33 \text{ ksi (230 MPa)} \leq \text{specified } F_y \text{ of profiled panel} \leq 80 \text{ ksi (550 MPa)}$ ,  
 $45 \text{ ksi (310 MPa)} \leq \text{specified } F_u \text{ of profiled panel} \leq 82 \text{ ksi (565 MPa)}$ ,
- (d) *Profiled panel pitch*  $\leq 12 \text{ in. (305 mm)}$ , and
- (e) *Support fastener*  $\leq 18 \text{ in. (460 mm)}$ .

#### D1.1 Profiled Panel Diaphragm Shear Strength

The *available shear strength [factored resistance]* per unit length of a *profiled panel* shall be the least *available strength [factored resistance]* obtained from the *limit states* controlled by either *connection strength* or *panel stability* using the applicable *resistance factors* or *safety factors* in Table D1.1-1.

**Table D1.1-1**  
**Safety Factors and Resistance Factors**

Support Member Type	Load Type or Combinations Including	Connection Type	Limit State					
			Connection Strength, $S_{nf}$			Panel Stability, $S_{nb}$		
			$\Omega$ (ASD)	$\phi$ (LRFD)	$\phi$ (LSD)	$\Omega$ (ASD)	$\phi$ (LRFD)	$\phi$ (LSD)
Steel	Wind	Welds	2.15	0.75	0.60	2.00	0.80	0.75
		Screws	2.00	0.80	0.75			
		All Others	Chapter G					
	Earthquake and All Others	Welds	3.00	0.55	0.40			
		Screws	2.30	0.70	0.55			
		All Others	Chapter G					
Wood	All	Screws	3.00	0.55	0.50			
		Nails						
All Others	All	All Others	Chapter G					

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

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

$$S_{nc} = \left( \frac{N^2 \beta^2}{L^2 N^2 + \beta^2} \right)^{0.5} P_{nf} \quad (\text{Eq. D1.1-2})$$

$$S_{ne} = \frac{(2\alpha_1 + n_p \alpha_2) P_{nf} + n_e P_{nfs}}{L} \quad (\text{Eq. D1.1-3})$$

$$S_{np} = P_{nf} \frac{n_d}{w_t} \quad (\text{Eq. D1.1-4})$$

where the ratio of the number of support fasteners to the tributary width,  $n_d/w_t$ , shall be the resulting governing minimum combination for the *panel* under consideration.

The nominal shear strength [resistance] per unit length of a *profiled panel* controlled by panel stability shall be the smallest of  $S_{no}$  and  $S_{nl}$ .

$$S_{no} = \frac{7890}{\alpha L_v^2} \left( \frac{I_{xg}^3 t^3 d}{s} \right)^{0.25} \quad (\text{Eq. D1.1-5})$$

$$S_{nl} = P_{nw} \left( \frac{d-e}{D_d} \right) \left( \frac{1}{d} \right) \quad (\text{Eq. D1.1-6})$$

where

$S_{ni}$  = Nominal shear strength [resistance] per unit length of diaphragm or wall diaphragm controlled by connections at interior panels or edge panels

$S_{nc}$  = 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

$S_{ne}$  = 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

$S_{np}$  = Nominal shear strength [resistance] per unit length of diaphragm or wall diaphragm controlled by connections along the ends of interior panels or edge panels and into exterior supports

$S_{no}$  = Nominal shear strength [resistance] per unit length of diaphragm or wall diaphragm controlled by panel out-of-plane buckling, kip/ft (kN/m)

$S_{nl}$  = Nominal shear strength [resistance] per unit length of diaphragm or wall diaphragm controlled by panel local web buckling over exterior support

$A$  = Number of exterior support connections per flute located at the sidelap at an interior panel or edge panel end

$\lambda$  = Connection strength reduction factor at corner fastener, unitless

$$= 1 - \frac{D_d L_v}{240\sqrt{t}} \geq 0.7 \quad \text{for U.S. customary units} \quad (\text{Eq. D1.1-7})$$

$$= 1 - \frac{D_d L_v}{369\sqrt{t}} \geq 0.7 \quad \text{for SI units} \quad (\text{Eq. D1.1-8})$$

where

$D_d$  = Depth of panel, in. (mm). See Figure D1.1-1

$L_v$  = Span of panel between supports with fasteners, ft (m)

$t$  = Base metal thickness of the panel, in. (mm)

$\beta$  = Factor defining connection contribution and interaction to diaphragm shear strength per unit length

$$= n_s \alpha_s + 2n_p \alpha_p^2 + 4\alpha_e^2 \quad (\text{Eq. D1.1-9})$$

where

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

$$\alpha_s = \frac{P_{ns}}{P_{nf}} \quad (\text{Eq. D1.1-10})$$

where

$P_{ns}$  = Nominal shear strength [resistance] of a sidelap connection per fastener determined in accordance with Appendix 1

$P_{nf}$  = Nominal shear strength [resistance] of a support connection per fastener determined in accordance with Appendix 1

$n_p$  = Number of interior supports along a total panel length,  $L$

$\alpha_p^2$  = Analogous section modulus of panel interior support connection group in an interior or edge panel

$$= \left( \frac{1}{w^2} \right) \sum x_p^2 \quad (\text{Eq. D1.1-11})$$

where

$w$  = Panel cover width

$x_p$  = Distance from panel centerline to an interior support connection in a panel

$\alpha_e^2$  = Analogous section modulus of panel exterior support connection group in an interior or edge panel

$$= \left( \frac{1}{w^2} \right) \sum x_e^2 \quad (\text{Eq. D1.1-12})$$

where

$$\begin{aligned}
 x_e &= \text{Distance from } \textit{panel} \text{ centerline to an } \textit{exterior support connection} \text{ in a } \textit{panel} \\
 L &= \text{Total } \textit{panel} \text{ length} \\
 N &= \text{Number of support fasteners per unit width at an } \textit{interior} \text{ or } \textit{edge panel's} \text{ end} \\
 \alpha_1 &= \text{Measure of } \textit{exterior support} \text{ fastener group distribution across a } \textit{panel} \text{ width, } w_e, \text{ at} \\
 &\quad \text{an } \textit{edge panel} \\
 &= \frac{\sum x_{ee}}{w_e} \qquad \qquad \qquad (\text{Eq. D1.1-13})
 \end{aligned}$$

where

$$\begin{aligned}
 x_{ee} &= \text{Distance from } \textit{panel} \text{ centerline to an } \textit{exterior support connection} \text{ in an } \textit{edge panel} \\
 w_e &= \textit{Panel cover width at the edge panel} \\
 \alpha_2 &= \text{Measure of } \textit{interior support} \text{ fastener group distribution across a } \textit{panel} \text{ width, } w_e, \text{ at} \\
 &\quad \text{an } \textit{edge panel} \\
 &= \frac{\sum x_{pe}}{w_e} \qquad \qquad \qquad (\text{Eq. D1.1-14})
 \end{aligned}$$

where

$$\begin{aligned}
 x_{pe} &= \text{Distance from } \textit{panel} \text{ centerline to an } \textit{interior support connection} \text{ in an } \textit{edge panel} \\
 n_e &= \text{Number of } \textit{edge support connections} \text{ between transverse supports and along an } \textit{edge} \\
 &\quad \textit{panel length, L} \\
 P_{nfs} &= \text{Nominal } \textit{shear strength [resistance]} \text{ of an } \textit{edge support connection} \text{ installed parallel} \\
 &\quad \text{with an } \textit{edge panel} \text{ span and between transverse supports} \\
 n_d &= \text{Number of } \textit{support connections} \text{ at the flute bottom relative to the tributary width,} \\
 &\quad w_t, \text{ along the ends of } \textit{interior} \text{ or } \textit{edge panels} \text{ and into } \textit{exterior supports} \\
 w_t &= \text{Tributary width to any given bottom flute with } \textit{support connection(s)} \text{ along the end} \\
 &\quad \text{perpendicular to the } \textit{panel} \text{ span and located at } \textit{exterior support}
 \end{aligned}$$

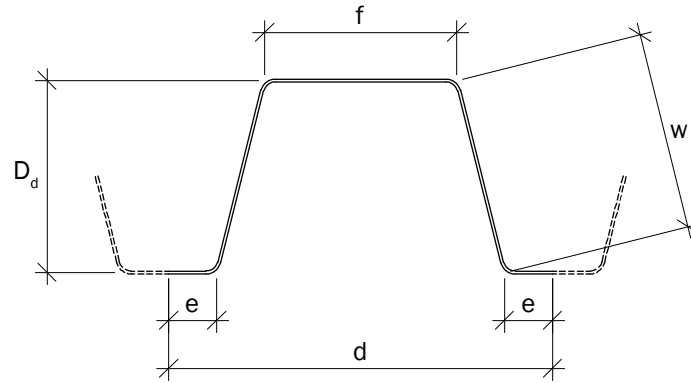
**User Note:**

Commentary Figure D1.1.5 provides examples on determination of  $n_d$  and  $w_t$  for a variety of *panel* configurations.

$$\begin{aligned}
 \alpha &= \text{Conversion factor for units} \\
 &= 1 \quad \text{for U.S. customary units} \\
 &= 1879 \text{ for SI units} \\
 I_{xg} &= \text{Moment of inertia of fully effective } \textit{panel} \text{ per unit width, in.}^4/\text{ft} \text{ (mm}^4/\text{mm)} \\
 d &= \textit{Panel corrugation pitch, in. (mm)} \\
 e &= \text{One-half the bottom flat width of } \textit{panel} \text{ measured between points of intercept as} \\
 &\quad \text{illustrated in Figure D1.1-1, in. (mm)} \\
 s &= \text{Developed flute width per pitch, in. (mm)} \\
 &= 2(e + w) + f \qquad \qquad \qquad (\text{Eq. D1.1-15})
 \end{aligned}$$

where

$$\begin{aligned}
 w &= \text{Web flat width of } \textit{panel} \text{ measured between points of intercept illustrated in} \\
 &\quad \text{Figure D1.1-1, in. (mm)} \\
 f &= \text{Top flat width of } \textit{panel} \text{ measured between points of intercept illustrated in} \\
 &\quad \text{Figure D1.1-1, in. (mm)}
 \end{aligned}$$



**Figure D1.1-1 Panel Configuration**

$$\begin{aligned}
 P_{nw} &= \text{Nominal web crippling strength [resistance] per web} \\
 &= 4.36t^2F_y \sin \theta \left( 1 - 0.04\sqrt{\frac{R}{t}} \right) \left( 1 + 0.25\sqrt{\frac{N_{\text{ext}}}{t}} \right) \left( 1 - 0.025\sqrt{\frac{h}{q_s t}} \right) \quad (\text{Eq. D1.1-16})
 \end{aligned}$$

where

$F_y$  = Design yield stress as determined in accordance with AISI S100 Section A3.3.1

$\theta$  = Angle between plane of web and plane of bearing surface,  $45^\circ \leq \theta \leq 90^\circ$

$R$  = Inside bend radius

$N_{\text{ext}}$  = Bearing length at exterior support (3/4 in. (19 mm) minimum)

$h$  = Flat dimension of web measured in plane of web

$q_s$  = Perforated web adjustment factor

= 1.00

## D1.2 Profiled Panel Diaphragm Shear Stiffness

The diaphragm shear stiffness,  $G'$ , for profiled panels shall be calculated in accordance with Eq. D1.2-1:

$$G' = \left( \frac{Et}{2(1+\mu)\frac{s}{d} + C + \gamma_c D_n} \right) K \quad (\text{Eq. D1.2-1})$$

where

$G'$  = Diaphragm shear stiffness, kip/in. (kN/m)

$E$  = Modulus of elasticity of steel

= 29,500 ksi, (203,000 MPa)

$t$  = Base steel thickness of panel, in. (mm)

$K$  = Stiffness factor relating support and sidelap connection flexibilities

= 1.00 for steel panels with lap-down on steel supports

=  $S_f/S_s$  for steel panels with lap-up on steel supports

= 0.50 for steel panels on wood supports

where

$S_f$  = Structural support connection shear flexibility determined in accordance with Appendix 1, in./kip (mm/kN)

$S_s$  = Sidelap connection shear flexibility determined in accordance with Appendix 1,



in./kip (mm/kN)

$\mu$  = Poisson's ratio for steel  
= 0.3

$d$  = Panel corrugation pitch. See Figure D1.1-1, in. (mm)

$s$  = Developed flute width per pitch. Defined in Section D1.1, in. (mm)

$C$  = Slip constant considering slippage at *sidelap connections* and distortion at *support connections*

$$= \left( \frac{Et}{w} \right) \left( \frac{2L}{2\alpha_3 + n_p\alpha_4 + 2n_s \frac{S_f}{S_s}} \right) S_f \quad (\text{Eq. D1.2-2})$$

where

$L$  = Total panel length

$\alpha_3$  = Measure of exterior support fastener group distribution across a panel width,  $w$ , at an interior panel

$$= \frac{\sum x_e}{w} \quad (\text{Eq. D1.2-3})$$

$w$  = Panel cover width at the interior panel

$\alpha_4$  = Measure of interior support fastener group distribution across a panel width,  $w$ , at an interior panel

$$= \frac{\sum x_p}{w} \quad (\text{Eq. D1.2-4})$$

$x_p$  = Distance from panel centerline to an interior support connection in an interior panel

$n_p$  = Number of interior supports along a total panel length,  $L$

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

$\gamma_c$  = Support factor for warping determined in accordance with Table D1.2-1

**Table D1.2-1**  
**Support Factor,  $\gamma_c$**

Spans	1	2	3	4	5	6	7
$\gamma_c$	1.00	1.00	0.90	0.80	0.71	0.64	0.58

$D_n$  = Unitless warping factor considering distortion at panel ends determined using the parameters as shown in Figure D1.1-1

$$= \frac{D}{L} \quad (\text{Eq. D1.2-5})$$

where

$D$  = Weighted average  $D_i$  value for warping across the panel width,  $w$ , in. (mm)

$$= \frac{U_1 D_1 + U_2 D_2 + U_3 D_3 + U_4 D_4}{U_1 + U_2 + U_3 + U_4} \quad (\text{Eq. D1.2-6})$$

where

$U_1$  = 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,  $w$

$D_1$  = Value for warping where bottom flange fastener is in every valley

$$= \frac{\gamma_1 f}{d(t)^{1.5}} \text{ in. (mm)} \quad (\text{Eq. D1.2-7})$$

$D_2$  = Value for warping where bottom flange fastener is in every second valley

$$= \frac{\gamma_2 f}{2d(t)^{1.5}} \text{ in. (mm)} \quad (\text{Eq. D1.2-8})$$

$D_3$  = Value for warping where bottom flange fastener is in every third valley

$$= \frac{\gamma_3 f}{3d(t)^{1.5}} \text{ in. (mm)} \quad (\text{Eq. D1.2-9})$$

$D_4$  = Value for warping where bottom flange fastener is in every fourth valley

$$= \frac{\gamma_4 f}{4d(t)^{1.5}} \text{ in. (mm)} \quad (\text{Eq. D1.2-10})$$

where

$\gamma_i$  = Final displacement indicator at top of corrugation for valley fastener cases,  $i = 1$  to  $4$ ,  $\text{in.}^{2.5} (\text{mm}^{2.5})$

$$\gamma_1 = \delta_{t1} \quad (\text{Eq. D1.2-11})$$

$$\gamma_3 = 2\delta_{t2} + \frac{2e}{f}\delta_{b2} \quad (\text{Eq. D1.2-12})$$

$$\gamma_3 = 2\delta_{t3} + \delta_{tc3} + 2\left(\frac{2e}{f}\right)\delta_{b3} \quad (\text{Eq. D1.2-13})$$

$$\gamma_4 = 2(\delta_{t4} + \delta_{tc4}) + \left(\frac{2e}{f}\right)(2\delta_{b4} + \delta_{bc4}) \quad (\text{Eq. D1.2-14})$$

where

$\delta_{ti}$  = Lateral displacement indicator at top of corrugation for valley fastener cases,  $i = 1$  to  $4$ ,  $\text{in.}^{2.5} (\text{mm}^{2.5})$

$\delta_{bi}$  = Lateral displacement indicator at bottom of corrugation for valley fastener cases,  $i = 1$  to  $4$ ,  $\text{in.}^{2.5} (\text{mm}^{2.5})$

$\delta_{b1} = 0$  for fasteners in the bottom flat of each flute.

$$\delta_{t1} = \frac{24f}{\kappa_{t1}} \left[ \frac{\kappa_{t1}}{4f^2(f+w)} \right]^{0.25} \quad (\text{Eq. D1.2-15})$$

$$\delta_{t2} = \frac{24f}{\kappa_{t2}} \left[ \frac{\kappa_{t2}}{4f^2(f+w)} \right]^{0.25} \quad (\text{Eq. D1.2-16})$$

$$\delta_{t3} = \frac{24f}{\kappa_{t3}} \left[ \frac{\kappa_{t3}}{4f^2(f+w)} \right]^{0.25} \quad (\text{Eq. D1.2-17})$$

$$\delta_{t4} = \frac{24f}{\kappa_{t4}} \left[ \frac{\kappa_{t4}}{4f^2(f+w)} \right]^{-0.25} \quad (\text{Eq. D1.2-18})$$

$$\delta_{b2} = \frac{48e}{\kappa_{b2}} \left[ \frac{\kappa_{b2}}{16e^2(2e+w)} \right]^{-0.25} \quad (\text{Eq. D1.2-19})$$

$$\delta_{b3} = \frac{48e}{\kappa_{b3}} \left[ \frac{\kappa_{b3}}{16e^2(2e+w)} \right]^{-0.25} \quad (\text{Eq. D1.2-20})$$

$$\delta_{b4} = \frac{48e}{\kappa_{b4}} \left[ \frac{\kappa_{b4}}{16e^2(2e+w)} \right]^{-0.25} \quad (\text{Eq. D1.2-21})$$

$$\delta_{tc3} = \frac{24f}{\kappa_{tc3}} \left[ \frac{\kappa_{tc3}}{4f^2(f+w)} \right]^{-0.25} \quad (\text{Eq. D1.2-22})$$

$$\delta_{tc4} = \frac{24f}{\kappa_{tc4}} \left[ \frac{\kappa_{tc4}}{4f^2(f+w)} \right]^{-0.25} \quad (\text{Eq. D1.2-23})$$

$$\delta_{bc4} = \frac{48e}{\kappa_{bc4}} \left[ \frac{\kappa_{bc4}}{16e^2(2e+w)} \right]^{-0.25} \quad (\text{Eq. D1.2-24})$$

where

$\kappa_{ij}$  = Spring constant indicator required for D, 1/in.<sup>3</sup> (1/mm<sup>3</sup>)

$$\kappa_{t1} = \frac{1}{\delta_{22} - \frac{\delta_{12}}{2}} \quad (\text{Eq. D1.2-25})$$

$$\kappa_{t2} = \frac{1}{\frac{2e}{f} \frac{\delta_{12}}{2} + \delta_{22}} \quad (\text{Eq. D1.2-26})$$

$$\kappa_{t3} = \frac{1}{\left(0.5 + \frac{2e}{f}\right) \delta_{12} + \delta_{22}} \quad (\text{Eq. D1.2-27})$$

$$\kappa_{t4} = \frac{1}{\left(1 + \frac{3e}{f}\right) \delta_{12} + \delta_{22}} \quad (\text{Eq. D1.2-28})$$

$$\kappa_{b2} = \frac{\frac{2e}{f}}{\frac{2e}{f} \frac{\delta_{11}}{2} + \delta_{12}} \quad (\text{Eq. D1.2-29})$$

$$\kappa_{b3} = \frac{\frac{2e}{f}}{\left(0.5 + \frac{2e}{f}\right) \delta_{11} + \delta_{12}} \quad (\text{Eq. D1.2-30})$$

$$\kappa_{b4} = \frac{\frac{2e}{f}}{\left(1 + \frac{3e}{f}\right) \delta_{11} + \delta_{12}} \quad (\text{Eq. D1.2-31})$$

$$\kappa_{tc3} = \frac{1}{\left(0.5 + \frac{2e}{f}\right) \delta_{11} + \delta_{22} + \frac{\delta_{12}}{2}} \quad (\text{Eq. D1.2-32})$$

$$\kappa_{tc4} = \frac{1}{\left(1 + \frac{3e}{f}\right) \delta_{11} + \delta_{22} + \left(1 + \frac{e}{f}\right) \delta_{12}} \quad (\text{Eq. D1.2-33})$$

$$\kappa_{bc4} = \frac{\frac{2e}{f}}{\left(1 + \frac{4e}{f}\right)\delta_{11} + 2\delta_{12}} \quad (\text{Eq. D1.2-34})$$

where

$\delta_{ij}$  = Deflection indicator of profile racking per unit *load* per unit length required for D, in.<sup>3</sup> (mm<sup>3</sup>)

$$\delta_{11} = \frac{D_d^2}{3}(2w + 3f) \quad (\text{Eq. D1.2-35})$$

$$\delta_{12} = \frac{\delta_{11}}{2} \quad (\text{Eq. D1.2-36})$$

$$\delta_{22} = \frac{1}{12} \left( \frac{D_d}{d} \right)^2 [s(4e^2 - 2ef + f^2) + d^2(3f + 2w)] \quad (\text{Eq. D1.2-37})$$

## D2 Perforated Profiled Panels

This section shall apply to *perforated profiled panels* within the limits specified in Section D1.

### D2.1 Perforated Profiled Panel Diaphragm Shear Strength

Nominal shear strength [resistance] per unit length,  $S_n$ , for *perforated profiled panels* shall be determined using Section D1.1 with the following modifications:

- The modified *panel* moment of inertia,  $I_{xg}$ , is obtained from the manufacturer and used in Eq. D1.1-5.
- In Eq. D1.1-5, the developed flute width per pitch,  $s$ , is determined in accordance with Eq. D1.1-15 using modified element lengths by setting  $e = e_p$ ,  $f = f_p$ , and  $w = w_p$  where:

$$e_p = K_{E_e}^{1/3} e \quad (\text{Eq. D2.1-1})$$

$$f_p = K_{E_f}^{1/3} f \quad (\text{Eq. D2.1-2})$$

$$w_p = K_{E_w}^{1/3} w \quad (\text{Eq. D2.1-3})$$

where

$$\begin{aligned} K_{E_i} &= \text{Indicator of relative flexural stiffness of an element without perforations to the} \\ &\quad \text{stiffness of the element with perforations over part of its length} \\ &= 1 + A_i^3 \left( \frac{1}{k} - 1 \right) \end{aligned} \quad (\text{Eq. D2.1-4})$$

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

$$\begin{aligned}
 k &= \text{Ratio of the perforated element stiffness relative to that of a solid element} \\
 &= 0.9 + p_o^2 - 1.875p_o \quad \text{for } 0.2 \leq p_o \leq 0.58 \quad (\text{Eq. D2.1-5}) \\
 &= 1 - 2.175p_o \quad \text{for } p_o < 0.2 \quad (\text{Eq. D2.1-6})
 \end{aligned}$$

where

$$\begin{aligned}
 p_o &= \text{Ratio of the area of perforations to the total area in the perforated band} \\
 &= 0.9069 \frac{d_p^2}{c_p^2} \quad (\text{Eq. D2.1-7})
 \end{aligned}$$

where

$d_p$  = Perforation hole diameter  
 $c_p$  = Hole center-to-center spacing

- (c) In Eq. D1.1-16 the perforated *web* adjustment factor,  $q_s$ , is calculated in accordance with Eq. D2.1-8.

$$q_s = 1 - (1 - k) \left( \frac{W_p}{h} \right) \quad (\text{Eq. D2.1-8})$$

where

$k$  = Ratio of the perforated element stiffness relative to that of a solid element calculated in accordance with Eqs. D2.1-5 and D2.1-6  
 $W_p$  = Width of perforation band in *web* flat of width,  $w$   
 $h$  = Flat dimension of *web* measured in plane of *web*

Where *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 Appendix 1, as applicable, using the *nominal connection strength [resistance]* at an unperforated element.

Where *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 Chapter G.

## D2.2 Perforated Profiled Panel Diaphragm Shear Stiffness

The *diaphragm shear stiffness, G'*, for *perforated profiled panels* shall be calculated in accordance with Section D1.2 with the following modifications:

- (a) In Eq. D1.2-1, the developed flute width per pitch,  $s$ , is determined in accordance with Eq. D2.2-1.

$$s = 2e + 2w + f + (E_p + 2w_p + F_p) \left( \frac{1}{k} - 1 \right) \quad (\text{Eq. D2.2-1})$$

where

$E_p$  = 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 the perforated element stiffness relative to that of a solid element calculated in accordance with Eqs. D2.1-5 and D2.1-6

- (b) The slip constant considering slippage at *sidelap connections* and distortion at *support connections, C*, is determined using Eq. D1.2-2 with *support connection shear flexibility, S<sub>f</sub>*, and *sidelap connection shear flexibility, S<sub>s</sub>*, determined as follows:

- (1) In accordance with Appendix 1, as applicable, for fasteners located in nonperforated zones of an element; or
- (2) In accordance with Section G for fasteners located in perforated zones of an element.
- (c) The unitless warping factor considering distortion at *panel* ends,  $D_{nv}$ , is determined in accordance with Eq. D1.2-5 with the following modifications:
  - (1) Modified element lengths are used by setting  $e = e_p$ ,  $f = f_p$ , and  $w = w_p$  in Eq. D1.1-15 and Eq. D1.2-35 through Eq. D1.2-37 where:
    - $e_p$  = Determined in accordance with Eq. D2.1-1
    - $f_p$  = Determined in accordance with Eq. D2.1-2
    - $w_p$  = Determined in accordance with Eq. D2.1-3

### D3 Cellular Panels

This section shall apply to *cellular panels* comprised of a *profiled panel* interconnected to a flat sheet without perforated elements within the following limits:

- (a)  $0.5 \text{ in. (12.7 mm)} \leq \text{cellular panels depth} \leq 7.5 \text{ in. (191 mm)}$ ,
- (b)  $0.034 \text{ in. (0.864 mm)} \leq \text{bottom plate base steel thickness} \leq 0.064 \text{ in. (1.63 mm)}$ ,
- (c)  $0.034 \text{ in. (0.864 mm)} \leq \text{top profiled panel base steel thickness} \leq 0.064 \text{ in. (1.63 mm)}$ ,
- (d) Support fastener types are welds, screws, or *power-actuated fasteners*,
- (e) No insulation between the *cellular panel* and the support,
- (f) Fastener edge dimensions satisfy requirements specified in AISI S100,
- (g) Top *profiled panel* pitch  $\leq 12 \text{ in. (305 mm)}$ , and
- (h) *Support fastener* spacing  $\leq 18 \text{ in. (460 mm)}$ .

#### D3.1 Cellular Panel Diaphragm Shear Strength

*Nominal shear strength [resistance]* per unit length,  $S_{nv}$  for *cellular panels* without perforations shall be determined using Section D1.1 with the following modifications:

- (a) In lieu of Eq. D1.1-4, the *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*,  $S_{np}$ , shall be calculated in accordance with Eq. D3.1-1.

$$S_{np} = NP_{nf} \quad (\text{Eq. D3.1-1})$$

where

$N$  = Number of support fasteners per unit width at an *interior* or *edge panel's* end

$P_{nf}$  = *Nominal shear strength [resistance]* of a *support connection* per fastener determined in accordance with Appendix 1

- (b) The *nominal shear strength [resistance]* per unit length of *cellular panels* controlled by *panel out-of-plane buckling*,  $S_{no}$ , is calculated in accordance with Eq. D1.1-5, with the following modifications:
  - (1)  $I_{xg}$  = Moment of inertia of fully effective *cellular panel* per unit width,  $\text{in.}^4/\text{ft}$  ( $\text{mm}^4/\text{mm}$ )
  - (2)  $t$  = Base metal *thickness* of the top *profiled panel* in *cellular panel*, in. (mm)
  - (3)  $d$  = *Panel* corrugation pitch of the top *profiled panel* in *cellular panel*, in. (mm)
  - (4)  $s$  = Developed flute width per pitch of the top *profiled panel* in *cellular panel* calculated

in accordance with Eq. D1.1-15, in. (mm)

- (c) The shear limit of local *web* crippling,  $S_{nl}$ , Eq. D1.1-6 which determines the shear limit of local *web* buckling, shall not be applied to *cellular panel diaphragm* shear strength.
- (d) The *nominal shear strength [resistance]* of a *support connection* per fastener,  $P_{nf}$ , shall be determined in accordance with Appendix 1 using the total *thickness* of both top *profiled panel* and bottom plate that are penetrated by the fastener above the plane of shear transfer at the support.
- (e) The *nominal shear strength [resistance]* of a *sidelap connection* per fastener,  $P_{ns}$ , shall be determined in accordance with Appendix 1 using the *thickness* of the 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 Chapter G.

### D3.2 Cellular Panel Diaphragm Shear Stiffness

The *diaphragm shear stiffness*,  $G'$ , for *cellular panels* without perforated elements shall be calculated in accordance with Eq. D3.2-1.

$$G' = \frac{Et}{A_a + C} \quad (\text{Eq. D3.2-1})$$

where

$G'$  = *Diaphragm shear stiffness*, kip/in. (kN/m)

$A_a$  = Material shear deformation component for *cellular panel*

$$= \frac{2.6 \frac{s}{d}}{1 + \frac{s t_b}{w_d t}} \quad (\text{Eq. D3.2-2})$$

where

$s$  = Developed flute width of top *profiled panel* in *cellular panel* in accordance with Eq. D1.1-15 in which the variables are defined as follows, in. (mm):

$e$  = Distance from the top *profiled panel* longitudinal fastener to the *web*, in. (mm)

$f$  = Top flat width of top *profiled panel* measured between points of intercept illustrated in Figure D1.1-1, in. (mm)

$w$  = *Web* flat width of top *profiled panel* measured between points of intercept illustrated in Figure D1.1-1, in. (mm)

$d$  = Top *profiled panel* corrugation pitch, in. (mm)

$w_d$  = Distance measured across the width and between longitudinal rows of fasteners connecting the top *profiled panel* to the bottom plate, in. (mm)

$t_b$  = Base steel *thickness* of bottom plate in *cellular panel*, in. (mm)

$t$  = Base steel *thickness* of top *profiled panel* in *cellular panel*, in. (mm)

$C$  = Slip constant considering slippage at *sidelap connections* and distortion at *support connections*; defined by Eq. D1.2-2, in which:

(a) Structural *support connection shear flexibility*,  $S_f$ , is based on the total thickness of elements above the shear transfer plane, in./kip (mm/kN),

(b) *Sidelap connection shear flexibility*,  $S_s$ , is based on the *thickness* of the element containing the *sidelap* fastener, in./kip (mm/kN), and

(c)  $t$  is the top *profiled panel thickness*, in. (mm)

#### D4 Perforated Cellular Panels

This section shall apply to *cellular panels* with perforated elements within the limits described in Section D3.

##### D4.1 Perforated Cellular Panel Diaphragm Shear Strength

Nominal shear strength [resistance] per unit length,  $S_n$ , for *cellular panels* with perforations shall be determined using Section D3.1 with the following modifications:

- 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 *profiled panel* is perforated, the developed flute width of top *profiled panel* in the *cellular panel*,  $s$ , shall be modified in accordance with Section D1.2 as specified for *perforated profiled panels*.

##### D4.2 Perforated Cellular Panel Diaphragm Shear Stiffness

The *diaphragm shear stiffness* for *perforated cellular panels* shall be calculated in accordance with Eq. D3.2-1 with  $A_a$  determined in accordance with Eq. D4.2-1 and  $C$  defined in this section:

$$A_a = \frac{2.6 \frac{s'}{d}}{1 + \frac{s'}{d'} \frac{t_b}{t}} \quad (\text{Eq. D4.2-1})$$

where

$C$  = Slip constant considering slippage at *sidelap connections* and distortion at *support connections* determined in accordance with Section D3.2.  $S_f$  and  $S_s$  are determined in accordance with Chapter G for fasteners located in perforated zones of an element.

$A_a$  = Material shear deformation component for *perforated cellular panel*

$s'$  = Developed flute width of top *profiled panel* per width,  $w_d$ , in *cellular panel* in accordance with Eq. D2.2-1

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

$$= w_d + w_{dp} \left( \frac{1}{k_b} - 1 \right) \quad (\text{Eq. D4.2-2})$$

where

$w_d$  = Distance measured across the width and between longitudinal rows of fasteners connecting the top *profiled panel* to the bottom plate, in. (mm)

$w_{dp}$  = Total width of perforation bands in bottom plate width,  $w_d$ , in. (mm)

$k_b$  = Ratio of perforated element *stiffness* relative to that of a solid element in the perforated bottom plate of *cellular panel* of the same *thickness*,  $t_b$ , and determined in accordance with Eqs. D2.1-5 and D2.1-6



## E. STRUCTURAL CONCRETE FILLED PROFILED PANELS

This chapter addresses the *diaphragm shear strength* and *diaphragm shear stiffness* for structural concrete filled *profiled panels* and *cellular panels*.

The chapter is organized as follows:

- E1 Structural Concrete Filled Profiled Panels
- E2 Structural Concrete Filled Cellular Panels

### E1 Structural Concrete Filled Profiled Panels

This section shall apply to *structural concrete filled profiled panels* within the following limits:

- (a)  $0.5 \text{ in. (12.7 mm)} \leq \text{profiled panel depth} \leq 3 \text{ in. (76.2 mm)}$ ,
- (b)  $0.014 \text{ in. (0.356 mm)} \leq \text{base steel panel thickness} \leq 0.075 \text{ in. (1.91 mm)}$ ,
- (c) Fastener types include steel headed stud anchors, welds with or without washers, bolts, screws, and *power-actuated fasteners*,
- (d)  $33 \text{ ksi (230 MPa)} \leq \text{specified } F_y \text{ of profiled panel} \leq 80 \text{ ksi (550 MPa)}$ ,  
 $45 \text{ ksi (310 MPa)} \leq \text{specified } F_u \text{ of profiled panel} \leq 82 \text{ ksi (565 MPa)}$ ,
- (e) *Structural Concrete* placed on steel deck shall conform to ACI 318 Chapter 19 for the United States and Mexico, or CSA A23.3 Chapter 4 for Canada. Admixtures containing chloride salts or other substances that are corrosive or otherwise deleterious to the *profiled panels* shall not be permitted,
- (f) *Structural concrete* fill has a minimum thickness of 2 in. (50.8 mm) of fill over top of *profiled panel*,
- (g) The maximum design thickness of *structural concrete* fill over the top of *profiled panel* is 6 in. (152 mm),
- (h) *Structural concrete* has a specified compressive strength,  $f'_c$ , not less than 2500 psi (17.2 MPa), and
- (i) Deformed bar reinforcement conforms to ASTM A615 or A706 and welded wire reinforcing conforms to ASTM A1064 or A1022. Minimum concrete cover over reinforcement and minimum bar spacing is in accordance with ACI 318. The maximum spacing of reinforcement is the lesser of five times  $t_a$  or 18 in. (457 mm). The minimum reinforcement ratio is 0.001 where reinforcement ratio is calculated as  $A_v/(t_a b)$ , where variables  $A_v$ ,  $t_a$  and  $b$  are defined in Section D4.1.1.

**Exception:** If reinforcement does not meet these requirements, the area of reinforcement,  $A_v$ , is taken as zero in the calculation of *available diaphragm shear strength [factored resistance]*.

#### E1.1 Structural Concrete Filled Profiled Panel Diaphragm Shear Strength

The *available shear strength [factored resistance]* per unit length of a *structural concrete filled profiled panel* shall be the least available strength *[factored resistance]* obtained from the *limit states* controlled by either *connection* strength or diagonal tension cracking using the applicable resistance factors or *safety factors* in Table E1.1-1.

**Table E1.1-1**  
**Safety Factors and Resistance Factors**

Support Member Type	Load Type or Combinations Including	Connection Type	Limit State					
			Connection Strength, $S_{nf}$			Diagonal Tension Cracking, $S_{nt}$		
			$\Omega$ (ASD)	$\phi$ (LRFD)	$\phi$ (LSD)	$\Omega$ (ASD)	$\phi$ (LRFD)	$\phi$ (LSD)
Steel	All	Welded Headed Shear Studs	3.00	0.55	0.50	2.00	0.80	0.75
		Welds	AISI S100 Section J2					
		Bolts	AISI S100 Section J3					
		Screws	AISI S100 Section J4					
		All Others	Chapter G					
Wood	All	Screws	NDS					
		Nails						
All Others	All	All Others	Chapter G					

The *nominal shear strength [resistance]* per unit length of a *structural concrete filled profiled panel* controlled by *connection strength*,  $S_{nf}$ , shall be calculated using Eq. E1.1-3:

$$S_{nf} = P_{nf} N_c \quad (\text{Eq. E1.1-3})$$

where

$P_{nf}$  = *Nominal shear strength [resistance]* of a *support connection* per fastener determined in accordance with AISC 360 for welded steel headed stud anchors or Appendix 1 as applicable

$N_c$  = Average number of support fasteners per unit width

The *nominal shear strength [resistance]* per unit length of a *structural concrete filled profiled panel* controlled by diagonal tension cracking shall be calculated using Eq. E1.1-4.

$$S_{nt} = k_c \lambda_{LW} b t_e \sqrt{f'_c} + A_v F_{ys} \leq 0.0075 b t_e \sqrt{f'_c} \quad (\text{Eq. E1.1-4})$$

where

$k_c$  = Factor for *structural concrete* strength

= 3.2/1000 For U.S. customary units (Eq. E1.1-5a)

= 0.266/1000 For SI units (Eq. E1.1-5b)

$\lambda_{LW}$  = Factor for *lightweight concrete*

= 1.0 For normal weight concrete

= 0.75 For lightweight concrete

= 0.85 For sand-lightweight concrete

$b$  = Unit width of *diaphragm* with *structural concrete* fill, 12 in. for U. S. customary units and 1000 mm for SI units

$t_e$  = Equivalent transformed concrete thickness, in. (mm)

$$= t_a + n_{sc} t \frac{d}{s} \quad (\text{Eq. E1.1-6})$$

$t_a$  = Average thickness of *structural concrete*, calculated as the cross-sectional area of the *structural concrete* over one *profiled panel* divided by the width of the *profiled*

$$n_{sc} = \text{Modular ratio of steel } \textit{profiled panel} \textit{ to concrete} = \frac{E}{E_c} \quad (\text{Eq. E1.1-7})$$

$E$  = Modulus of elasticity of steel  
 $E_c$  = Modulus of elasticity of concrete in accordance with ACI 318  
 $t$  = Base steel *thickness* of *panel*, in. (mm)  
 $d$  = *Panel* corrugation pitch. See Figure D1.1-1, in. (mm)  
 $s$  = Developed flute width per pitch. Defined in Eq. D1.1-15, in. (mm)  
 $f'_c$  = Specified concrete compressive strength, psi (MPa)  
 $A_v$  = Cross-sectional area per unit width,  $b$ , of reinforcement oriented parallel to the direction of shear, in.<sup>2</sup> (mm<sup>2</sup>)  
 $F_{ys}$  = Specified minimum yield stress of reinforcement, ksi (GPa)

**User Note:**

If reinforcement does not meet the requirements of D4, then the area of reinforcement,  $A_v$ , is taken as zero. It is permitted to use  $A_v$  equal to zero even if reinforcement meets the requirements of D4.

**E1.2 Structural Concrete Filled Profiled Panel Diaphragm Shear Stiffness**

The *diaphragm shear stiffness*,  $G'$ , for a *structural concrete filled profiled panel diaphragm* shall be calculated in accordance with Eq. E2.2-1.

$$G' = 4.8 \left[ \left( d_c + \frac{D_d}{2} \right) + t \left( \frac{E}{E_c} \right) \left( \frac{d}{s} \right) \right] \sqrt{f'_c} \quad (\text{Eq. E1.2-1})$$

where

$G'$  = *Diaphragm shear stiffness*, kip/in. (kN/m)  
 $d_c$  = *Structural concrete thickness* above top of *profiled panel*, in. (mm)  
 $D_d$  = *Depth of panel*, in. (mm)  
 $E$  = Modulus of elasticity of steel, ksi (MPa)  
 $E_c$  = Modulus of elasticity of concrete in accordance with ACI 318, ksi (MPa)  
 $t$  = Base steel *thickness* of the *profiled panel*, in. (mm)  
 $d$  = *Panel* corrugation pitch, in. (mm)  
 $s$  = Developed flute width per pitch, in. (mm)  
 $f'_c$  = Specified *structural concrete* compressive strength, psi (MPa)

**E2 Structural Concrete Filled Cellular Panels**

This section shall apply to *structural concrete filled cellular panels* with top *profiled panels* in *cellular panel* and properties that are within the limits described in Section E1.

**E2.1 Structural Concrete Filled Cellular Panel Diaphragm Shear Strength**

The *available shear strength [factored resistance]* per unit length of a *structural concrete filled cellular panel* shall be determined in accordance with Section E1.1.

**E2.2 Structural Concrete Filled Cellular Panel Diaphragm Shear Stiffness**

The *diaphragm shear stiffness,  $G'$* , for a *structural concrete filled cellular panel* shall be calculated in accordance with Section E1.2.

## F. INSULATING CONCRETE FILLED PANELS

This chapter addresses the *shear strength* and *diaphragm shear stiffness* for *insulating concrete filled profiled* and *cellular panels*.

The chapter is organized as follows:

- F1 Insulating Concrete Filled Profiled Panels
- F2 Insulating Concrete Filled Cellular Panels

### F1 Insulating Concrete Filled Profiled Panels

This section shall apply to insulating concrete filled *profiled panels* within the following limits:

- (a) 0.5 in. (12.7 mm)  $\leq$  *profiled panel depth*  $\leq$  3 in. (76.2 mm),
- (b) 0.014 in. (0.356 mm)  $\leq$  *base steel panel thickness*  $\leq$  0.075 in. (1.91 mm),
- (c) Fastener types include welds with or without washers, screws, and *power-actuated fasteners*,
- (d) 33 ksi (230 MPa)  $\leq$  specified  $F_y$  of *profiled panel*  $\leq$  80 ksi (550 MPa),  
45 ksi (310 MPa)  $\leq$  specified  $F_u$  of *profiled panel*  $\leq$  82 ksi (565 MPa),
- (e) The maximum design thickness of *insulating concrete fill* over the top of *profiled panels* is 6 in. (152 mm),
- (f) For lightweight *insulating concrete* without polystyrene inserts, the minimum thickness over the top of *profiled panels* is 2.5 in. (63.5 mm),
- (g) *Insulating concrete aggregate* conforms to ASTM C332,
- (h) Cellular insulating concrete foaming agent shall comply with ASTM C869,
- (i) Minimum *insulating concrete* thickness above insulating board shall be 2 in. (50.8 mm), and
- (j) 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.

#### F1.1 Insulating Concrete Filled Profiled Panel Shear Strength

The *available shear strength [factored resistance]* per unit length of *insulating concrete filled profiled panels* shall be obtained from the *limit state* controlled by *connection strength*.

$$\frac{S_n}{\Omega} = \frac{S_{ni}}{\Omega} \quad \text{for ASD} \quad (\text{Eq. F1.1-1})$$

$$\phi S_n = \phi S_{nf} \quad \text{for LRFD and LSD} \quad (\text{Eq. F1.1-2})$$

where

$S_n$  = *Nominal shear strength [resistance]* per unit length of *insulating concrete filled profiled panel diaphragm*

$S_{ni}$  = *Nominal shear strength [resistance]* per unit length of *insulating concrete filled profiled panel diaphragm* controlled by *connections*

$\Omega$  = *Safety factor* per Table F1.1-1

$\phi$  = *Resistance factor* per Table F1.1-1

**Table F1.1-1**  
**Safety Factors and Resistance Factors**

Support Member Type	Load Type or Combinations Including	Connection Type	Limit State		
			Connection Strength, $S_{ni}$		
			$\Omega$ (ASD)	$\phi$ (LRFD)	$\phi$ (LSD)
Steel	All	Welds or Screws	3.25	0.50	0.45
		All Others	Chapter G		
All Others	All	All Others	Chapter G		

The nominal shear strength [resistance] per unit length of insulating concrete filled profiled panels controlled by connections at interior panels or edge panels and fill shear strength shall be calculated using Eq. F1.1-3 or Eq. F1.1-4, as applicable.

(a) Insulating concrete without insulating board infill:

$$S_{ni} = \frac{\beta P_{nf}}{L} + \frac{4}{3000} b d_c \sqrt{f'_c} \quad \text{for U.S. customary units} \quad (\text{Eq. F1.1-3a})$$

$$S_{ni} = \frac{\beta P_{nf}}{L} + 1.11(10)^{-4} b d_c \sqrt{f'_c} \quad \text{for SI units} \quad (\text{Eq. F1.1-3b})$$

where

$d_c$  = Insulating concrete thickness above top of profiled panel, in. (mm)

$f'_c$  = Specified insulating concrete compressive strength, psi (MPa)

Other parameters and required units are defined in Section D1.1.

(b) Insulating concrete with insulating board infill:

$$S_{ni} = \frac{\beta P_{nf}}{L} + 0.064 \sqrt{f'_c} \quad \text{for U.S. customary units} \quad (\text{Eq. F1.1-4a})$$

$$S_{ni} = \frac{\beta P_{nf}}{L} + 1.12 \sqrt{f'_c} \quad \text{for SI units} \quad (\text{Eq. F1.1-4b})$$

where

$f'_c$  = Specified insulating concrete compressive strength, psi (MPa)

Other parameters and required units are defined in Section D1.1.

**User Note:**

See Commentary Figure C-F1.1-2 for insulation board detailing practices.

## F1.2 Insulating Concrete Filled Profiled Panel Diaphragm Shear Stiffness

Diaphragm shear stiffness,  $G'$ , shall be calculated in accordance with Eq. F1.2-1 for insulating concrete filled profiled panels.

$$G' = \frac{Et}{2(1+\mu)\frac{s}{d} + C} + K_3 \quad (\text{Eq. F1.2-1})$$

where

$G'$  = Diaphragm shear stiffness, kip/in. (kN/m)

$E$  = Modulus of elasticity of steel, ksi (MPa)

$t$  = Base steel thickness of profiled panel, in. (mm)

$$\begin{aligned}
 K_3 &= \text{Stiffness contribution of the insulating concrete fill} \\
 &= 3.5d_c(f'_c)^{0.7}, \text{ kip/in. for U.S. customary units} \quad (\text{Eq. F1.2-2a})
 \end{aligned}$$

$$= 786d_c(f'_c)^{0.7}, \text{ kN/m for SI units} \quad (\text{Eq. F1.2-2b})$$

where

$d_c$  = Insulating concrete thickness above top of profiled panel, in. (mm)

$f'_c$  = Specified insulating concrete compressive strength, psi (MPa)

$\mu$  = Poisson's ratio for steel

= 0.3

$s$  = Developed flute width per pitch. Defined in Section D1.1, in. (mm)

$d$  = Panel corrugation pitch. See Figure D1.1-1, in. (mm)

$C$  = Slip constant considering slippage at *sidelap connections* and distortion at *support connections*. Defined in Section D1.2

## F2 Insulating Concrete Filled Cellular Panels

This section shall apply to insulating concrete filled *cellular panels* with top *profiled panel* in *cellular panel* and properties that satisfy the limits of applicability given in Section F1.

### F2.1 Insulating Concrete Filled Cellular Panel Diaphragm Shear Strength

The *available shear strength [factored resistance]* per unit length of an *insulating concrete filled cellular panel* shall be determined in accordance with Section F1.1.

### F2.2 Insulating Concrete Filled Cellular Panel Diaphragm Shear Stiffness

The *diaphragm shear stiffness,  $G'$* , shall be calculated in accordance with Eq. F2.2-1 for *insulating concrete-filled cellular panels*.

$$G' = \frac{Et}{A_a + C} + K_3 \quad (\text{Eq. F2.2-1})$$

where

$G'$  = Diaphragm shear stiffness, kip/in. (kN/m)

$E$  = Modulus of elasticity of steel, ksi (MPa)

$t$  = Base steel thickness of top profiled panel in cellular panel, in. (mm)

$K_3$  = Stiffness contribution of the structural concrete fill. Defined in Section E1.2

$A_a$  = Material shear deformation component for *cellular panel* determined in accordance with Section D3.2

= Material shear deformation component for perforated *cellular panel* determined in accordance with Section D4.2

$C$  = Slip constant considering slippage at *sidelap connections* and distortion at *support connections*. Defined in Section D1.2

## G. TEST-BASED DESIGN

This chapter addresses test-based determination of the *diaphragm shear strength* and *diaphragm shear stiffness* for steel *panels* and concrete filled steel *panels*.

The chapter is organized as follows:

- G1 Panel Connections
- G2 Diaphragm Assemblies

### G1 Panel Connections

This section shall apply to testing of *connections* for use with the design methods in sections C, D, E and F.

**User Note:**

*Connection* strengths and stiffnesses determined in this section may be used to aid in analysis of test results in Section G2.

#### G1.1 Test Standards

Panel *connections* shall be tested in accordance with AISI S100 Chapter K or an alternate published test standard for the material combination under consideration. The following test methods shall be considered acceptable published test standards:

- (a) AISI S905 for steel-to-steel *connections*,
- (b) AISI S923 for concrete-to-steel composite *connections*,
- (b) ASTM D1761 for steel-to-wood *connections*, or
- (c) ASTM E1190 or E488 for steel-to-concrete *connections*.

#### G1.2 Panel In-Plane Shear Strength and Stiffness Using Test-Based Connections

The analysis of *connection* test data shall be in accordance with AISI S100 Chapter K using the target reliability index and statistical data for *connections* in AISI Chapter K to determine the *resistance factor*. The *resistance factor* developed from *connection* testing shall not exceed the *resistance factor* to be applied to the *panel connection nominal design strength* [*resistance*] as specified in Chapter D, E, F or G. The basis of comparison for the test-based *connection resistance factor* shall be for the same type of *connection*. *Power-actuated fasteners* and other mechanical *connections* are permitted to be compared with screws.

For *allowable strength design*, the *safety factor* shall be determined from the governing *resistance factor*, using the conversion in AISI S100 Chapter K.

*Connection shear flexibility (stiffness)* of test-based *connections* shall be determined in accordance with AISI S905. For alternate published standards that do not provide a method for *Connection Shear Flexibility (stiffness)*, the provisions of AISI S905 shall be applied.

### G2 Diaphragm Assemblies

This section shall apply to the testing of *diaphragm* assemblies to determine the strength of a specific assembly, or to run a range of tests to establish a general *diaphragm* design model.



## G2.1 Test Standards

*Diaphragm* assembly test shall be in accordance with AISI S907 or an alternate published test standard for the material combination under consideration.

## G2.2 Test-Based Diaphragm Strength and Stiffness

Analysis of the test results of *diaphragm* assemblies shall be in accordance with AISI S100 Chapter K using statistical data for determination of the *resistance factor* as follows:

$$C_{\phi} = 1.6 \text{ for LRFD (United States and Mexico)}$$

$$= 1.5 \text{ for LSD (Canada)}$$

$$V_Q = 0.25$$

For LRFD:

$$\beta_o = 2.5 \text{ for all wind load governed } diaphragms \text{ when } connection \text{ strength governs.}$$

$$= 3.5 \text{ for seismic and all other load types (except wind) governed } diaphragms \text{ when } connection \text{ strength governs.}$$

$$= 2.0 \text{ for all load types where stability limit state governs.}$$

For LSD:

$$\beta_o = 3.0 \text{ for all wind load governed } diaphragms \text{ when } connection \text{ strength governs.}$$

$$= 4.0 \text{ for seismic and all other load types (except wind) governed } diaphragms \text{ when } connection \text{ strength governs.}$$

$$= 2.5 \text{ for all load types where stability limit state governs.}$$

For *diaphragm* assemblies, the statistical factors in Table G2.2-1 shall be used for the determination of the *resistance factor* for *diaphragm* assemblies. All other statistical factors shall be in accordance with AISI S100 Table K2.1.1-1.

**Table G2.2-1**  
**Statistical Data for the Determination of Resistance Factor**

Type of Component	$M_m$	$V_M$	$F_m$	$V_F$
Members				
<i>Panel stability (out-of-plane and web buckling)</i>	1.10	0.10	1.00	0.05
Connections				
Screws and <i>power-actuated fasteners</i> into steel supports	1.10	0.08	1.00	0.05

All other statistical factors for determination of the *resistance factor* and corresponding *safety factors* shall be in accordance with AISI S100 Chapter K.

*Diaphragm shear stiffness, G'*, shall be determined in accordance with AISI S907. For alternate published standards that do not provide a method for determining *diaphragm shear stiffness, G'*, the provisions of S907 shall be applied.

## Appendix 1: Connection Strength and Stiffness

### 1.1 Scope

This appendix addresses the strength and elastic *stiffness* (flexibility) of *connections* for profiled steel *diaphragm panels* not otherwise covered within AISI S100 Chapter J.

### 1.2 Welded Connections

#### 1.2.1 Arc Spot Weld for Sheet(s) Welded to a Thicker Supporting Member Connection Shear Flexibility

The *connection shear flexibility* of each arc spot weld between the sheet or sheets and a thicker supporting member shall be determined in accordance with Eq. 1.2.1-1:

$$S = \frac{1.15\alpha}{1000\sqrt{t}} \quad (\text{Eq. 1.2.1-1})$$

where

$S$  = *Connection shear flexibility* of an arc spot weld between the sheet or sheets and a thicker supporting member, in./kip (mm/kN)

$\alpha$  = Conversion factor for units

= 1 for U.S. customary units

= 28.8 for SI units

$t$  = Total combined base steel *thickness* of *panel* involved in shear transfer above the shear transfer plane, in. (mm)

#### 1.2.2 Arc Spot Weld for Sheet-to-Sheet Connection Shear Flexibility

The *connection shear flexibility* of each arc spot weld between sheets shall be determined in accordance with Eq. 1.2.2-1:

$$S = \frac{1.25\alpha}{1000\sqrt{t}} \quad (\text{Eq. 1.2.2-1})$$

where

$S$  = *Connection shear flexibility* of an arc spot weld between sheets, in./kip (mm/kN)

$\alpha$  = Conversion factor for units

= 1 for U.S. customary units

= 28.8 for SI units

$t$  = Base steel *thickness* (exclusive of coatings) of thinner connected sheet, in. (mm)

#### 1.2.3 Arc Spot Weld With Washer for Sheet(s) Welded to a Thicker Supporting Member

##### 1.2.3.1 Arc Spot Weld With Washer Shear Strength

For arc spot welds with washers, the *nominal shear strength [resistance]*,  $P_n$ , shall be the lesser of AISI S100 Eq. J2.2.2.1-1 and Eq. 1.2.2.1-1. To determine  $d_e$  in AISI S100 Eq. J2.2.2.1-5,  $d$  shall be replaced by  $d_o$  and  $t$  shall be as defined below.

Eqs. 1.2.3.1-1a and 1.2.3.1-1b 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*,  $F_{u \text{ washer}} \geq 45$  ksi (310 MPa), and is permitted to be less than the *tensile strength* of the element to be welded.

$$P_n = 99t(1.33d_o + 0.3F_{xx}t) \quad \text{in U.S. customary units} \quad (\text{Eq. 1.2.3.1-1a})$$

$$P_n = 17.3t \left( \frac{d_o}{19.1} + \frac{F_{xx}t}{584} \right) \quad \text{in SI units} \quad (\text{Eq. 1.2.3.1-1b})$$

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)

$P_n$  = *Nominal shear strength [resistance]* of arc spot weld with washer, kip (kN)

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

### 1.2.3.2 Arc Spot Weld With Washer Connection Shear Flexibility

The *connection shear flexibility* of each arc spot weld with washer shall be determined in accordance with Eq. 1.2.2-1.

### 1.2.3.3 Arc Spot Weld With Washer Tension Strength

The uplift *nominal tensile strength [resistance]*,  $P_{nt}$ , of each arc spot weld with washer shall be determined in accordance with AISI S100 Section J2.2.3 where the diameter of the hole in the weld washer,  $d_o$ , shall be used in lieu of the visible diameter,  $d$ , to determine  $d_e$  in AISI S100 Eq. J2.2.3-1 and  $d_a$  in AISI S100 Eq. J2.2.3-2.

### 1.2.3.4 Arc Spot Weld With Washer Combined Shear and Tension

For an arc spot weld connection with washer subjected to a combination of shear and tension, an interaction check shall be applied in accordance with AISI S100 Section J2.2.4.

## 1.2.4 Arc Seam Welds

### 1.2.4.1 Arc Seam Weld for Sheet(s) Welded to a Thicker Supporting Member Connection Shear Flexibility

The *connection shear flexibility* of each arc seam weld between the sheet or sheets and a thicker supporting member shall be determined in accordance with Eq. 1.2.1.2-1.

### 1.2.4.2 Arc Seam Weld for Sheet-to-Sheet Connection Shear Flexibility

The *connection shear flexibility* of each arc seam weld between sheets shall be determined in accordance with Eq. 1.2.1.4-1.

## 1.2.5 Top Arc Seam Welds

### 1.2.5.1 Top Arc Seam Weld Connection Shear Flexibility

The *connection shear flexibility* of each top arc seam weld between sheets shall be determined in accordance with Eq. 1.2.4.1-1:

$$S = \frac{1.12\alpha}{1000\sqrt{t}} \left( \frac{L_w}{\alpha_5} \right)^{0.25} \quad (\text{Eq. 1.2.4.1-1})$$

where

$S$  = *Connection shear flexibility* of a top arc seam weld formed between two sheets, in./kip (mm/kN)

$\alpha$  = Conversion factor for units  
= 1 for U.S. customary units  
= 28.8 for SI units

$\alpha_5$  = Conversion factor for units  
= 1.5 for U.S. customary units  
= 38 for SI units

$t$  = Base steel *thickness* (exclusive of coatings) of thinner connected sheet, in. (mm)

$L_w$  = Length of *top arc seam sidelap weld*, in. (mm). See AISI S100 Figure J2.4.1-1 for details.

## 1.3 Screw Connections

### 1.3.1 Screw Connection Shear Flexibility

The *connection shear flexibility* of each screw connection shall be determined in accordance with this section.

For  $t_2/t_1 \leq 1.0$ , Screw *connection* shear flexibility,  $S$ , shall be taken as the larger of Equations 1.3.1-1a and 1.3.1-1b:

$$S = \frac{3.0\alpha}{1000\sqrt{t_1}} \quad (\text{Eq. 1.3.1-1a})$$

$$S = \frac{3.0\alpha}{1000\sqrt{t_2}} \quad (\text{Eq. 1.3.1-1b})$$

For  $t_2/t_1 \geq 2.5$ ,

$$S = \frac{1.3\alpha}{1000\sqrt{t_2}} \quad (\text{Eq. 1.3.1-2})$$

For  $1.0 < t_2/t_1 < 2.5$ , Screw *connection* shear flexibility,  $S$ , shall be calculated by linear interpolation between the above two cases.

where

$S$  = *Connection shear flexibility* of a screw connection, in./kip (mm/kN)

$\alpha$  = Conversion factor for units  
= 1 for U.S. customary units  
= 28.8 for SI units

$t_1$  = *Thickness* of member in contact with screw head or washer, in. (mm)

$t_2$  = *Thickness* of member not in contact with screw head or washer, in. (mm)

Eq. 1.3.1-2 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)).

### 1.3.2 Screw Shear Strength and Stiffness Through Insulation

The *nominal shear strength [resistance]*,  $P_{nv}$  of a screw *connection* shall be determined in accordance with AISI S100 Section J4.3 with compressed insulation not exceeding 3/8 in. (9.5 mm).

The strength for screws with a maximum of 3/8 in. (9.5 mm) of compressed insulation,  $P_{nf}$  or  $P_{nfs}$  shall be multiplied by a factor of 0.95 for load combinations including wind or 0.80 for load combinations including earthquake and all others.

The screw *connection shear flexibility*,  $S$ , with up to 3/8 in. (9.5 mm) of compressed insulation shall be determined in accordance with Section 1.3.1.

## 1.4 Power-Actuated Fastener (PAF) Connections

### 1.4.1 Power-Actuated Fastener (PAF) Combined Shear and Tension

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. 1.4.1-1.

For ASD

$$\frac{P_{nft}}{P_{nf}} + \frac{T}{\min\left(\frac{P_{nov}}{\Omega_{tov}} + \frac{P_{not}}{\Omega_{tot}}\right)} \leq 1 \quad (\text{Eq. 1.4.1-1a})$$

For LRFD and LSD

$$\frac{P_{nft}}{P_{nf}} + \frac{\bar{T}}{\min(\phi_{tov}P_{nov} + \phi_{tot}P_{not})} \leq 1 \quad (\text{Eq. 1.4.1-1b})$$

where

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

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

$P_{nov}$  = *Nominal tension strength [resistance]* of a support connection per power-actuated fastener controlled by pull-over

$P_{not}$  = *Nominal tension strength [resistance]* of a support connection per power-actuated fastener controlled by pull-out

$T$  = *Required allowable tensile strength* of a support connection per fastener determined for ASD loads and load combinations in accordance with Section A3

$\bar{T}$  = *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

=  $T_u$  for LRFD

=  $T_f$  for *LSD*

$\Omega_{tov}$  = Safety factor for a *power-actuated fastener* controlled by pull-over  
= 3.00 (*ASD*)

$\Omega_{tot}$  = Safety factor for a *power-actuated fastener* controlled by pull-out and determined by test using AISI S100 Chapter K

$\phi_{tov}$  = Resistance factor for a *power-actuated fastener* controlled by pull-over  
= 0.50 (*LRFD*)  
= 0.40 (*LSD*)

$\phi_{tot}$  = Resistance factor for a *power-actuated fastener* controlled by pull-out and determined by test using AISI S100 Chapter K

**User Note:**

*Power-actuated connection* strength and flexibility may be determined by test or taken from another recognized source such as product research (evaluation) report.

## 1.5 Button Punch Connections

### 1.5.1 Non-Piercing Button Punch Shear Strength

For *profiled panels* 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 determined in accordance with Eq. 1.5.1-1.

$$P_{ns} = 0.10 \text{ kips (0.45 kN)} \quad (\text{Eq. 1.5.1-1})$$

### 1.5.2 Non-Piercing Button Punch Connection Shear Flexibility

The *connection shear flexibility* for non-piercing button punch fasteners in panels shall be determined in accordance with Eq. 1.5.2-1.

$$S = \frac{30.0\alpha}{1000\sqrt{t}} \quad (\text{Eq. 1.5.2-1})$$

where

$S$  = Connection shear flexibility of a non-piercing button punch, in./kip (mm/kN)

$\alpha$  = Conversion factor for units

= 1 for U.S. customary units

= 28.8 for SI units

$t$  = Base steel *thickness* (exclusive of coatings) of thinner connected sheet, in. (mm)

## 1.6 Fasteners Into Wood Supports

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 *connection shear flexibility*,  $S_f$ , shall be determined with this section.

### 1.6.1 Screw or Nail Connection Into Wood Support Shear Strength

Shear strength for *panel connections* to wood supports with screws or nails shall be the limiting strength of the following:

For *panels* directly bearing on the wood support, the shear strength of the fastener attaching

the *panel* to the wood support shall be determined using the reference lateral design value,  $Z$ , which is based on the yield limit equation methodology in the NDS. Alternately, it is permissible to determine the lateral reference design value in accordance with the provisions of TR-12.

For *panels* with compressed insulation between the panel and wood support, the reference lateral design value,  $Z$ , shall be based on the yield limit equation methodology in TR-12 with the insulation considered a gap,  $g$ , in accordance with TR-12.

The nominal design strength of the screw in a wood support,  $P_{nf}$ , shall be determined in accordance with Eq. 1.6.1-1.

$$P_{nf} = Z' / \phi \quad (\text{Eq. 1.6.1-1})$$

where

$Z'$  = LRFD-adjusted lateral design value,  $Z$ , in accordance with NDS

$\phi$  = Lateral resistance factor in accordance with NDS

The *nominal connection strength [resistance]* shall not exceed the bearing strength of the screw or nail bearing on the steel panel based on the bearing strength of a screw determined in accordance with AISI S100 without the consideration of tilting.

### 1.6.2 Screw or Nail Connection Into Wood Support Flexibility

The *connection shear flexibility*,  $S_f$ , of wood screws or nails fastened into wood supports with or without a maximum of 3/8 in. (9.53 mm) compressed insulation beneath the *panel* shall be determined in accordance with Eq. 1.6.2-1 or Eq. 1.6.2-2, as applicable:

(a) For wood screws or nails fastened through bottom flat and into wood support

$$S = \frac{1.5\alpha}{1000\sqrt{t}} \quad (\text{Eq. 1.6.2-1})$$

(b) For wood screws or nails fastened through top flat and into wood support

$$S = \frac{3.0\alpha}{1000\sqrt{t}} \quad (\text{Eq. 1.6.2-2})$$

where

$S$  = Connection shear flexibility, in./kip (mm/kN)

$\alpha$  = Conversion factor for units

= 1 for U.S. customary units

= 28.8 for SI units

$t$  = Total combined base steel *thickness* of *panel* involved in shear transfer above the shear transfer plane, in. (mm)

Eqs. 1.6.2-1 and 1.6.2-2 shall be limited to:

- (1) Wood screw sizes #9 (nominal diameter = 0.177 in. (4.50 mm)) through #14 (nominal diameter = 0.25 in. (6.35 mm)), and
- (2) Nail diameters greater than or equal to 0.148 in. (3.76 mm).

### 1.6.3 Screw or Nail Connection Into Wood Support Combined Shear and Tension

The combined shear and tension strength of the screw or nail *connections* to wood supports shall be the limiting strength of the following:

- (a) The combined shear and tension strength of the screw or nail *connections* in wood

determined in accordance with the NDS provisions for combined lateral and withdrawal loads for dowel-type fasteners.

- (b) The combined shear and pull-over strength for the screw or nail determined in accordance with AISI S100 provisions for screw *connections* subject to combined shear and pull-over.
- (c) The combined shear and tension strength in screws or nails determined in accordance with AISI S100 provisions for the screw *connection* subject to combined shear and tension.





# **AISI STANDARD**

## **Commentary on the North American Standard for the Design of Profiled Steel Diaphragm Panels**

2023 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.

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## PREFACE

This document provides a commentary on the 2023 edition of AISI S310, *North American Standard for the Design of Profiled Steel Diaphragm Panels*, herein referred to as “*Standard*.”

The purpose of the *Commentary* is: (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 G, and Appendix 1.

The Committee acknowledges and is grateful for the contributions of the numerous engineers, researchers, producers and others who have contributed to the body of knowledge on the subjects. The Committee particularly acknowledges the pioneering analytical and research work conducted by Dr. Larry Luttrell of West Virginia University and Clarkson Pinkham of S. B. Barnes Associates. Special thanks are given to the formal Chair of the Diaphragm Design Subcommittee, John Mattingly for his dedication and commitment. Special thanks are also given to Patrick Bodwell for his effort in reorganizing the content of this edition of the *Standard*. The Committee wishes to also express its appreciation for the support of the Steel Deck Institute.

American Iron and Steel Institute  
December 2023

## SYMBOLS AND DEFINITIONS

The following symbols appear in this *Commentary*. Refer to the list of Symbols and Definitions in the *Standard* for definitions of other symbols.

Symbol	Definition	Section
$C_{\phi}$	Calibration coefficient	D1.1
$C_p$	Correction factor	D1.1
$F_m$	Mean value of fabrication factor	D1.1
$k_{st}$	<i>Stiffness</i> of a single headed shear stud	E1.2
$L_d$	<i>Diaphragm</i> span between shear walls or reaction lines	E1.2
$M_m$	Mean value of material factor	D1.1
$n$	Total number of tests	D1.1
$n_s$	Number of headed shear studs in the direction of loading	E1.2
$P_m$	Mean value of professional factor	D1.1
$Q_{su}$	Strength of one headed shear stud in the direction of loading	E1.2
$V_F$	Coefficient of variation of fabrication factor	D1.1
$V_M$	Coefficient of variation of material factor	D1.1
$V_Q$	Coefficient of variation of load effect	D1.1
$w$	Uniformly distributed lateral load applied to the <i>diaphragm</i>	E1.2
$\Delta_z$	<i>Diaphragm</i> deflection due to <i>connection</i> deformation	E1.2

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## COMMENTARY ON THE NORTH AMERICAN STANDARD FOR THE DESIGN OF PROFILED STEEL DIAPHRAGM PANELS

In 2023, AISI S310 was reorganized with the goal of improving the flow of the *Standard*. To accomplish this, the *connection* strength extracts were removed from AISI S310 in favor of references to AISI S100. The *connection* strength provisions not included in AISI S100 and the *connection* stiffness provisions were moved to Appendix 1 of this *Standard*.

To simplify the scope of the *Standard*, the following contents have been removed from AISI S310 in 2023: (a) the section on shear strength for standing seam *panels* was removed because this section only specified the use of test-based design which is always an option for *panels* that exceed the limits of the prescriptive design methods in this *Standard*; and (b) provisions for double *profiled panel* assemblies and the combined *diaphragm* shear strength and axial transfer to the ends of *profiled panels* were removed in favor of including them in design examples. Both the double skin and combined shear and axial design provisions are applications of *profiled panels* that use methods in this *Standard* combined with engineering mechanics and the provisions of AISI S100.

Cold-formed steel *panels* are an integral part of the *diaphragm* in building structures. The *panels* provide the stressed skin that resists shear in the *diaphragm* assembly. These *panels* have fluted profiles and are cold-formed from steel sheets in roll-forming machines or by press brake or bending. Deck profiles may be connected to other deck profiles or flat bottom plates, which form *cellular decks*. The *cellular decks* are formed in the manufacturing plant and then shipped as assembled units. The *thickness* of steel sheets used in fluted *panels* historically ranges between 0.014 in. (0.35 mm) and 0.105 in. (2.67 mm). *Cellular panels* are usually formed from thicker sheet steel because of fabrication requirements at longitudinal *connections* and *web* compactness requirements for 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 *Standard* only addresses the design and testing of flat 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 a primary *structural member* that provides lateral resistance and stability to a building system while the *panels* simultaneously provide other *serviceability* functions. These 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, 1956). AISI-sponsored work continued into the 1960s and 1970s under the direction of Dr. George Winter at Cornell University (Luttrell, 1967). There is an established and extensive test database for *profiled steel panel diaphragms* (SDI, DDM, 1981, etc.).



Two design manuals were developed for industry and users, and these manuals have evolved into the primary design and analytical references in North America. These manuals are: (1) Department of Army, NAVFAC (1982<sup>1</sup>), *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 plane *diaphragm* construction. The limits of design application are established by the tests.

Because these design manuals are not consensus documents, industry petitioned AISI to 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 the scope of AISI S100 are based on the available research reports. AISI S310 establishes analytical design 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.

## A. GENERAL PROVISIONS

The following commentary is specific to cold-formed steel *profiled panels*. Users should reference the AISI S100 commentary for general cold-formed steel design concepts that are used within AISI S310.

### A1 Scope, Applicability, and Definitions

#### A1.1 Scope

This *Standard* is intended to provide analytical methods to predict the in-plane shear strength and *shear stiffness* of *panels* and concrete filled *panels*, including the contribution of the *connection* of the *panels* to supporting members. This is not intended to provide a complete solution to predict the overall behavior of a *diaphragm* system in a building structure that also includes the contribution of support framing, chords, collectors, and interconnection to other elements of the building structure such as the vertical system or vertical lateral force-resisting system.

This *Standard* covers a wide range of common *panel* usages including steel roof deck, non-composite steel deck, composite steel deck, *wall diaphragm panels* (shear wall *panels*), wall cladding *panels*, and roof cladding *panels*. The *panels* may be connected to support framing systems of hot-rolled steel, open *web* steel joists, cold-formed steel, wood, concrete and other materials based on the analytical provisions and test-based design methods in this *Standard*. The design of the supporting members or structural systems should follow the guidance of the appropriate material or system standards.

#### A1.2 Applicability

This *Standard* provides the basis to design a broad range of *panel* and concrete filled *panel* assemblies through advanced modeling, analytical methods, and test-based design approaches. These methods are applicable for use with the *Allowable Strength Design (ASD)*

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<sup>1</sup> The 1982 edition was referenced due to errors in deck design that were found in the 1992 edition.

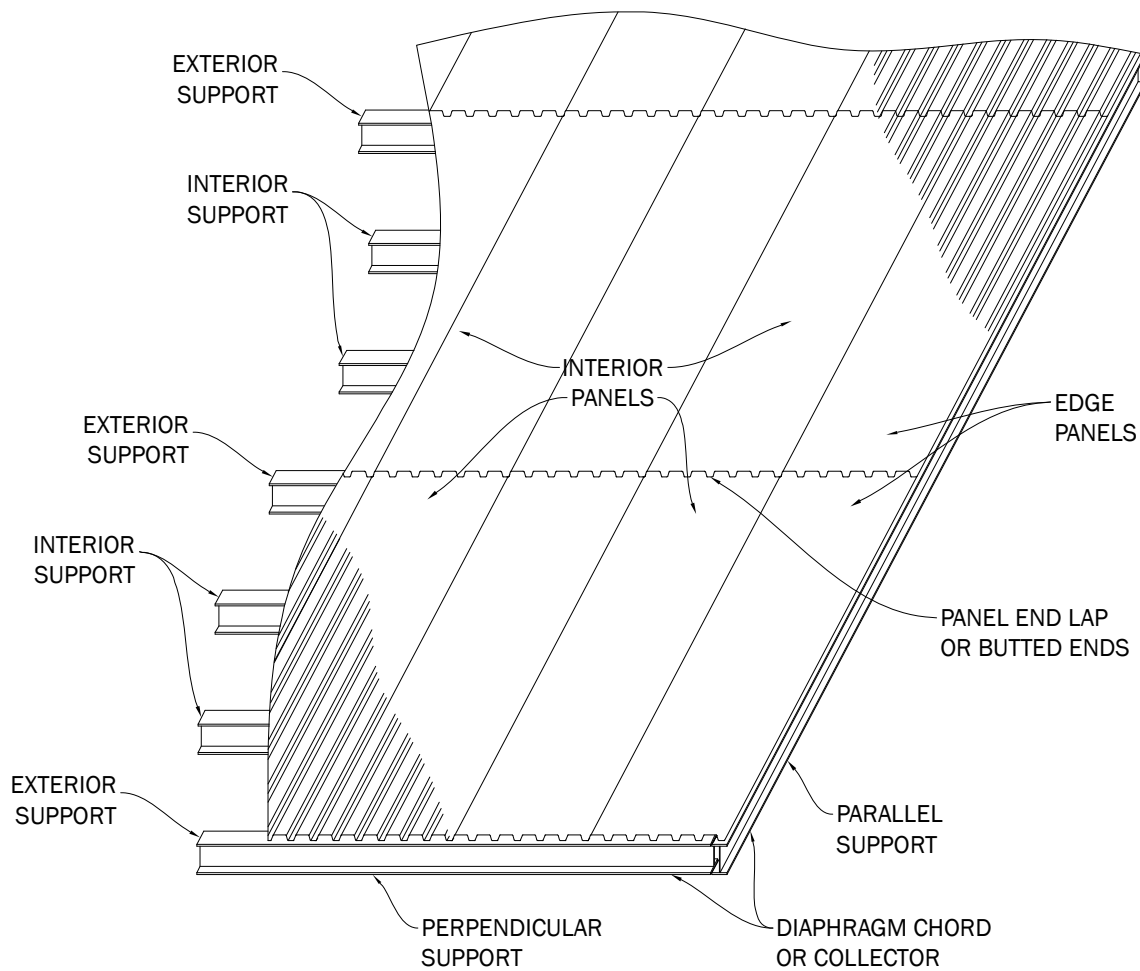
method or the *Load and Resistance Factor Design (LRFD)* method for the United States and Mexico, and the *Limit States Design (LSD)* method for Canada.

**A1.3 Definitions**

Many of the definitions in *Standard* Section A1.3 are self-explanatory. Only those which are not self-explanatory or may have unique usage in this *Standard* are briefly discussed below.

*Cellular Panel.* Commonly referred to as cellular steel deck.

*Diaphragm.* Profiled panels provide the stressed skin of a *diaphragm* system in a building structure. This assembly is comprised of the *profiled panels* resisting in-plane shear and providing *shear stiffness* in combination with framing, chords, and collectors that support the *profiled panels*. Figure C-A1.3-1 depicts the primary components of a *diaphragm* system.



**Figure C-A1.3-1 Diaphragm Assembly**

*Edge Panel.* An *edge panel* is a *panel* that transfers shear to a chord or collector that is parallel with the flutes. This may be at the edge of the *diaphragm*, or an interior collector within the

field of the *diaphragm*. Figure C-A1.3-1 depicts the location of *edge panels* relative to *interior panels* at the *diaphragm* edge.

*Exterior Support*. An *exterior support* relative to a *panel* is the support at each end of a *panel* perpendicular to the flutes. Figure C-A1.3-1 depicts the location of *exterior supports* relative to a *panel*. An *exterior support* is not relative to the *diaphragm* or *wall diaphragm* assembly.

*Interior Panel*. An *interior panel* is a typical panel in the field of the *diaphragm*. A *panel* at a chord or collector that is perpendicular to the flutes is also considered an *interior panel*. Figure C-A1.3-1 depicts the location of *interior panels* relative to *edge panels*.

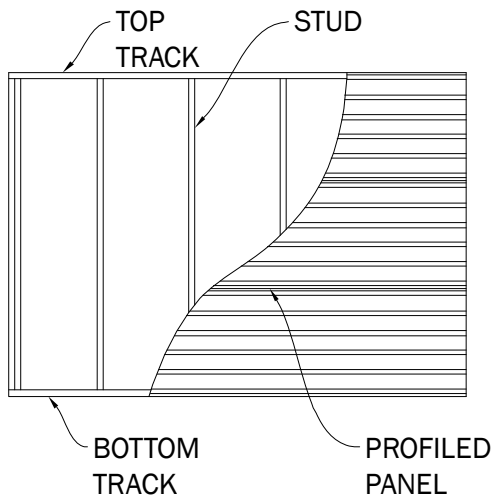
*Interior Support*. An *interior support* is a member that supports a *panel* perpendicular to the flutes, in between the *exterior supports* relative to a *panel*. Figure C-A1.3-1 depicts the location of *interior supports* relative to a *panel*.

*Perforated Cellular Panel*. Commonly referred to as acoustic cellular steel deck.

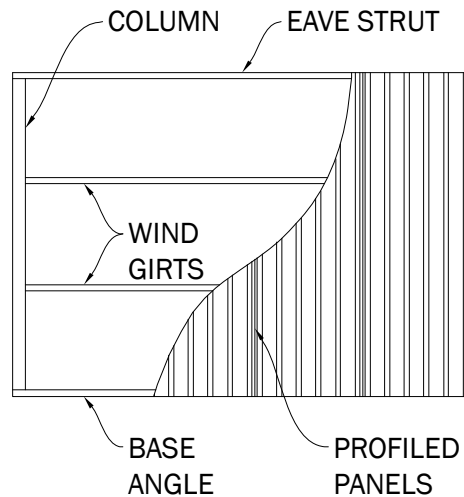
*Perforated Profiled Panel*. A *profiled panel* with perforations in the *webs*, bottom flange, top flange, the entire profile, or a combination of these. Acoustic steel roof deck is a common example of perforated *profiled panels*. Architectural wall or roof cladding *panels* are also often offered with perforations for mechanical screen wall and sunshade structure applications.

*Profiled Panel*. A *profiled panel* is formed from a single steel sheet without perforations. Examples of *profiled panels* are shown in Figure C-A1.3-1. Commonly referred to as steel deck, steel roof deck, non-composite steel deck, composite steel deck, wall cladding *panels* or roof *panels*.

*Wall Diaphragm*. A *wall diaphragm*, more commonly referred to as a shear wall, uses a *profiled panel* stressed skin and is common for light-framed structures, column and wind girt framed systems, and pole barn applications. A *wall diaphragm* system uses the in-plane shear strength and *shear stiffness* of *profiled panels* to resist the shear loads applied to a wall. The *profiled panels* are attached to vertical and/or horizontal members that resist axial and out-of-plane loads applied to a *wall diaphragm*. Figures C-A1.3-2a and C-A1.3-2b depict the primary components of a *wall diaphragm* system.



**Figure C-A1.3-2a Light-Framed Wall Diaphragm (Shear Wall)**



**Figure C-A1.3-2b Wind Girt Framed Wall Diaphragm (Shear Wall)**

#### **A1.4 Units of Symbols and Terms**

See the commentary in AISI S100 for units of symbols and terms.

#### **A2 Referenced Specifications, Codes and Standards**

Other referenced specifications and standards have been listed and updated in *Standard* Section A2 to provide their effective dates at the time of approval of this *Standard*. References for country-specific provisions are provided in *Standard* Section A2.1 for the United States and Mexico and A2.2 for Canada.

Additional references which the designer may use for related information are listed in the References section of the *Commentary*.

#### **A3 Materials**

See the commentary in AISI S100.

## **B. DESIGN REQUIREMENTS**

### **B1 General Provisions**

The following commentary addresses design requirements specific to cold-formed steel *panels*. Refer to the AISI S100 commentary for general cold-formed steel design requirements as applicable to cold-formed steel *panels* not addressed in this *Commentary*.

### **B2 Loads and Load Combinations**

See the commentary in AISI S100 for loads and load combinations.

### **B3 Design Basis**

As stated in *Standard* Section B3, design should be based on the principle that no applicable strength or serviceability *limit state* is exceeded when a structure is subjected to load effects corresponding to the applicable load combinations.

A *limit state* is the condition at which the structural usefulness of a load-carrying element or member is impaired to such an extent that it becomes unsafe for the occupants of a structure, or the element no longer performs its intended function. Typical *limit states* for cold-formed *profiled panel diaphragms* are controlled by the strength of the *connections* of the *panels* and *panel* or localized *buckling*.

Refer to the AISI S100 commentary for further discussion on design basis.

#### **B3.1 Required Strength [Effect Due to Factored Loads]**

The prescriptive design methods are intended for elastic level design. Design of *panels* by analysis using non-linear modeling or testing may be appropriate when inelastic design is necessary. The *required strength* [effect due to *factored loads*] is determined by the appropriate methods of structural analysis.

Reference the AISI S100 commentary for additional discussion on *required strength* [effect due to *factored loads*].

#### **B3.2 Design for Strength**

A *diaphragm* or *wall diaphragm* assembly using *panels* should be designed using *ASD* or *LRFD* in the United States and Mexico, and *LSD* in Canada. General discussion regarding these design methods can be found in the AISI S100 commentary.

#### **B3.3 Design for Diaphragm Panels**

The design of *panels* for in-plane *shear strength* and *shear stiffness* may be determined from prescriptive equations in Chapters D, E, and F using the equations therein derived from engineering mechanics and testing. Chapter C provides the option to use complex analytical modeling (finite element analysis) as an alternative to the derived equations in Chapters D, E, and F. The design of *panels* may also be developed from testing following the provisions of Chapter G.

For the design of cold-formed steel *panels* for in-plane shear strength and *shear stiffness*,

consideration should be given to several design *limit states* resulting from: (a) strength and flexibility of the *support connections*, (b) strength and flexibility of the *sidelap connections*, (c) plate-like *buckling* of the *panels* between supports, (d) local *web buckling* of the end of the *panels*, and (e) serviceability (*shear stiffness*) of the system.

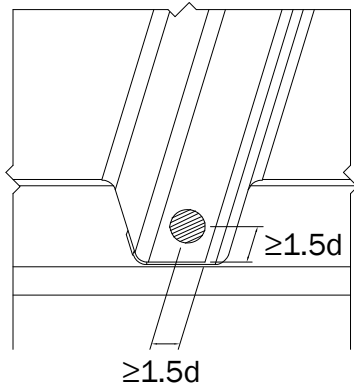
Example problems can be found in the SDI DDM04 (2015) and MCA Primer (2004).

### **B3.4 Design of Connections**

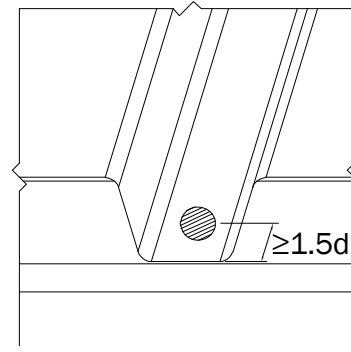
*Standard* Section B3.4 provides the charging language to use AISI S100 Chapter K and AISI S310 Appendix 1 for *connections* not covered in AISI S100 Chapter J. Appendix 1 is also the source for *connection* flexibility needed for the determination of *panel* in-plane *shear stiffness* to determine *diaphragm* deflection in a building structure.

It is common for *support connections* to experience simultaneous shear and tension (uplift) when the *diaphragm* resists a shear force caused by wind load with simultaneous wind uplift. The resulting reduced shear strength of the *connection* in the presence of tension (when applicable) should be used when determining the shear strength of *panels* in Chapters D, E, and F. This is consistent with common wind loading conditions in which the wind force develops net tension of the *profiled panels* and simultaneously induces shear on the *panels* acting as a *diaphragm* in a structure.

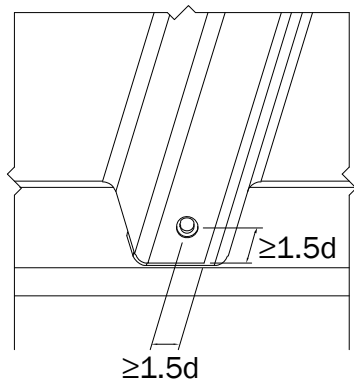
*Panels* have unique edge and end distance concerns because of the inherent proximity of the *support connections* and *sidelap connections* between adjacent sheets being especially close to the end or edge of the sheet. For the nestable deck, the top sheet of the rib with the lap needs to have adequate edge distance for *sidelap connections* along the length of the *panel*, and the *support connections* need adequate edge distance to the ends of the *panel*. The edge and end distances for common arc spot welds are depicted in Figures C-B3.4-1a and C-B3.4-1b. This is the same for screws as shown in Figure C-B3.4-1c. In addition to the minimum edge distance, rupture needs to be checked to ensure the edge or end distance is adequate to develop the full strength of the *connection*. If the horizontal lap is too short for an arc spot weld, the designer may consider ignoring the contribution of the lap *connection* on the top sheet or provide equivalent strength and stiffness *connections* that meet the edge distance with the short lap. *Power-actuated fasteners* or screws may be a solution because the minimum edge distance is generally less than 3/8 in. (9.53 mm) as shown in Figure C-B3.4-1c. A fillet weld along the edge of the short leg, qualified to weld through the bottom sheet to the support, of equivalent strength and stiffness may also be a solution to a short lap on the *panel* as shown in Figure C-B3.4-1d.



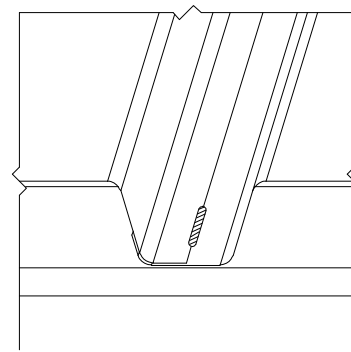
**Figure C-B3.4-1a Arc Spot Weld at Lap Edge and End Distance**



**Figure C-B3.4-1b Arc Spot Weld End Distance**

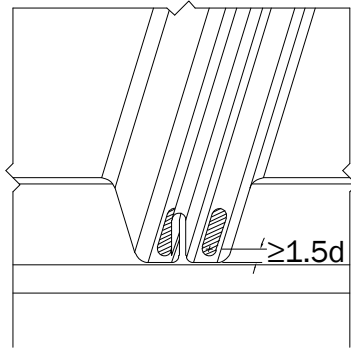


**Figure C-B3.4-1c Screw at Lap Edge and End Distance**

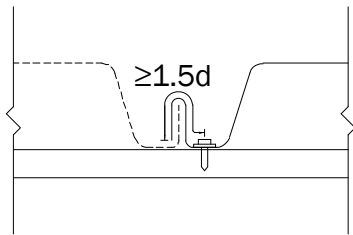


**Figure C-B3.4-1d Fillet Weld at Short Lap**

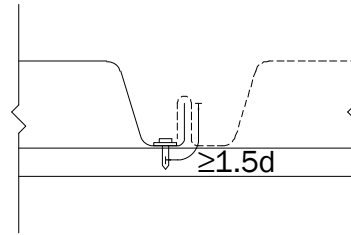
The end distance for an arc seam weld (elongated arc spot weld) is shown in Figure C-B3.4-2a. Arc seam welds are often substituted for arc spot welds in the narrow bottom flange of the interlock to prevent burn through of the *webs*. For interlock style *sidelaps*, the edge distance follows the centerline of the sheet steel as it wraps up to form the leg and hem as shown in Figures C-B3.4-2b and C-B3.4-2c. It is common for an interlock lap configuration to inherently have adequate edge distance.



**Figure C-B3.4-2a End Distance of Arc Seam Weld**



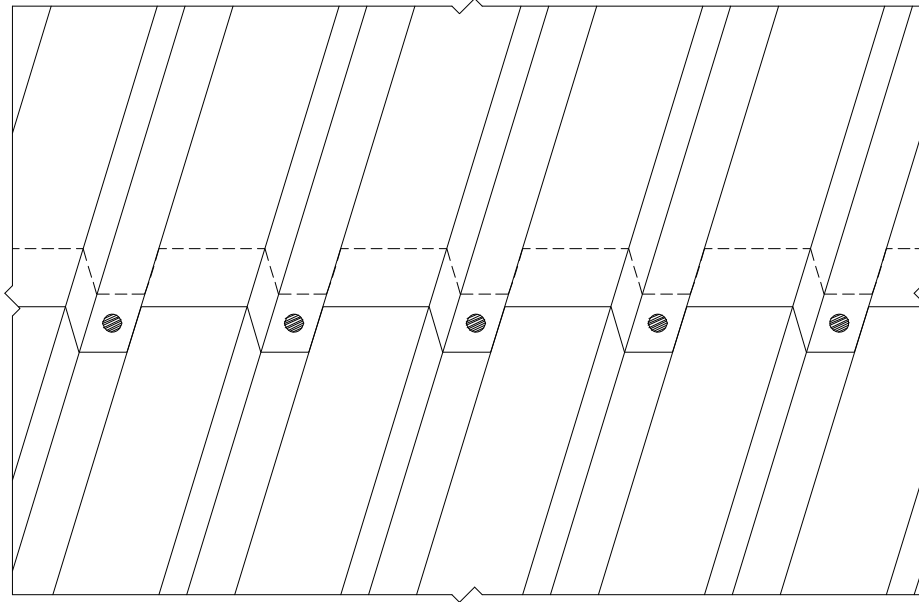
**Figure C-B3.4-2b Interlock Deck  
Hem Edge Distance**



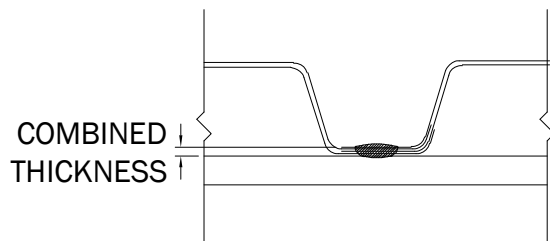
**Figure C-B3.4-2c Interlock Deck  
Leg Edge Distance**



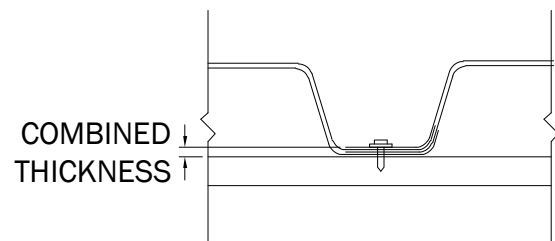
When nestable *panels* are installed with end laps as shown in Figure C-B3.4-3a, the corner *connection* of the *panels* goes through four layers of sheet steel. It is important that the combined *thickness* of the four layers of sheet steel is accounted for when selecting the *connections* to supports. Some *connection* options are illustrated in Figures C-B3.4-3b and C-B3.4-3c. For thick *panels*, the four layers may exceed the maximum *thickness* for arc spot welds. Self-drilling screws are generally long enough to accommodate this condition, or the length can be selected by the contractor to accommodate the combined *thickness*. *Power-actuated fastener* manufacturers should be consulted to determine the maximum appropriate combined *thickness* for the selected fastener.



**Figure C-B3.4-3a Nestable Panel  
With End Lap**

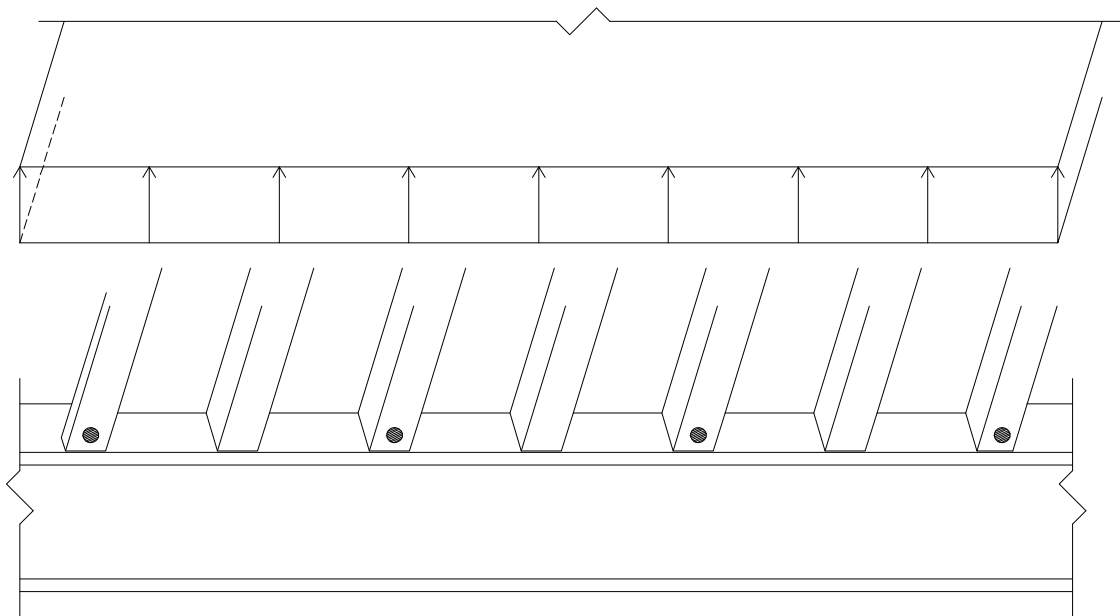


**Figure C-B3.4-3b Welded Connection**

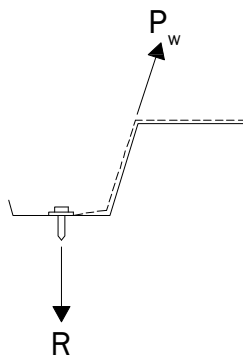


**Figure C-B3.4-3c Screw Connection**

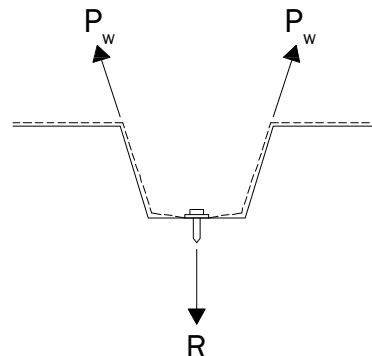
*Panels* subject to wind uplift generate tension on the *support connections* that lead to combined shear and tension interaction on *connections* when the *panels* also resist wind *diaphragm* shear loads as shown in Figure C-B3.4-4a. The resulting shear strength of the *connections* is the remaining strength after the effect of combined shear and tension interaction is considered. Fasteners attaching *panel sidelaps* to supports experience eccentric tension loading as shown in Figure C-B3.4-4b, while welds to supports in *panel* interior ribs experience concentric tension loading as shown in Figure C-B3.4-4c. See the AISI S100 commentary for arc spot welds in tension for further detail. The impact of eccentric loading on screws and *power-actuated fasteners* has not been demonstrated to be an issue and there are no specific additional requirements for these *connections*.



**Figure C-B3.4-4a Wind Uplift on Panel**



**Figure C-B3.4-4b Eccentric Loading on the Connection**



**Figure C-B3.4-4c Non-Eccentric Loading on the Connection**

### B3.5 Design for Stability

The primary function of *panels* used as the stressed skin of a floor *diaphragm*, roof *diaphragm*, or *wall diaphragm* (shear wall) is to resist in-plane shear deformation of the assembly. The in-plane deflection of a *diaphragm* or *wall diaphragm* will have a significant impact on the overall stability of the building structure. This *Standard* does not address the complexities of the design of a complete *diaphragm* or *wall diaphragm* assembly, or the interaction with an overall structure.

The *diaphragm shear stiffness*,  $G'$ , may be used to predict the shear deflection of a *diaphragm*. The change in shear deflection between two points along a *diaphragm* span of uniform stiffness may be determined by calculating the area under the diagram of shear divided by the product of the *diaphragm shear stiffness* and the depth,  $V/(G'b)$ . This does not address any other potential sources of deflection of a *diaphragm* or *wall diaphragm* assembly including but not limited to chords, collectors, support framing, hold-downs, and *connections* to an adjoining portion of a building structure.

### B3.6 Design of Floor, Roof and Wall Diaphragm Assemblies

The design of floor, roof or *wall diaphragms* with cold-formed steel *panels* is a combination of a stressed skin, support framing for the stressed skin, and the chords or collectors that transfer forces to the system. All *panels* together, whose resultant in-plane *shear strength* and *shear stiffness* are determined in accordance with this *Standard*, act as the stressed skin of a floor, roof, or *wall diaphragm* system. The support framing, chords, and collectors act to hold the *panels* in the plane of the *diaphragm* and distribute loads to the stressed skin. The most common support framing, chords, and collectors for floor and roof *diaphragms* are hot-rolled steel sections, open web steel joists, cold-formed steel sections, and wood members. The most common support framing, chords, and collectors for *wall diaphragms* (shear walls) are cold-formed steel studs, tracks, HSS steel, and wood members. The design of these systems involves a combination of materials from several material-specific standards and the applicable building code. This *Standard* does not provide complete guidance for all the members of these systems.

### B3.7 Design for Serviceability

The stiffness of the stressed skin of floor, roof, or wall (shear wall) *diaphragms* may have a significant impact on the deflection of a structure under lateral wind or seismic loading. This *Standard* does not address the complexities of the design of a complete floor, roof, or *wall diaphragm* subsystem, or the interaction of those systems in an overall structure. Further guidance is provided in AISI S100 for cold-formed steel structures, the applicable standard for other materials, and the applicable building code for the required deflection limits for an overall structure.

### B3.8 Design for Fatigue

The general *panel* in-plane shear strength equations are applicable to systems subject to *fatigue*. Reductions in strength should be accounted for in both *panels* and *connections* following the provisions of AISI S100 Chapter M.

### **B3.9 Design for Corrosion Effects**

In corrosive environments, appropriate metallic or other finishes should be provided to protect the steel from being compromised by excessive corrosion.

### **B4 Dimensional Limits and Considerations**

The dimensional limits provided in Chapters D, E, and F reflect the range of *panels* that were covered by testing used to validate the design methods in those chapters.

### **B5 Profiled Panel Properties**

Cold-formed steel *profiled panel* section properties are determined in accordance with AISI S100. Reference AISI S100 for discussion on member (*panel*) properties.

### **B6 Fabrication and Erection**

(Reserved)

The *Standard* is currently silent on fabrication and erection. For steel deck *panels*, SDI MOC3, *Manual of Construction with Steel Deck* (2016) and COSP-2023, *Code of Standard Practice for Steel Deck*, may be appropriate fabrication and erection standards.

### **B7 Quality Control and Quality Assurance**

(Reserved)

The *Standard* is currently silent on quality control and quality assurance. For steel deck *panels*, ANSI/SDI QA/QC-2017, *Quality Control and Quality Assurance for Installation of Steel Deck* (2017) may provide appropriate quality control and quality assurance standards.

#### **B7.1 Delivered Minimum Thickness**

The delivered minimum *thickness* for cold-formed steel *panels* is in accordance with AISI S100. See the AISI S100 commentary for further discussion.

### **B8 Evaluation of Existing Diaphragm Panels**

(Reserved)

This *Standard* does not currently provide specific guidance for the evaluation of existing structures. For structures with *profiled panels* that meet the requirements of this *Standard*, the methods herein may be used to determine the *shear strength* and *shear stiffness* of the existing *panels*.

### C. PROFILED PANEL DIAPHRAGM BY ANALYSIS

All *panels* are designed by analysis. The intent of this section is to provide a pathway to use high fidelity finite element modeling to determine the *nominal strength [resistance]* of *panels* and concrete filled *panels*. This follows the current trend in design by taking advantage of computational models that go beyond the simplified equations of engineering mechanics used to develop the provisions in Chapters D, E and F of this *Standard*. This *Standard* does not attempt to provide specific guidance on how to develop representative models and the software solutions to use them.

For steel roof deck *diaphragms* in warehouse/ industrial-type structures with rigid wall flexible *diaphragms*, the Applied Technology Council (2021) provides a good starting point to understand the depth of modeling that is possible. For common single to multi-story structures with structural concrete filled steel deck floor *diaphragms* in combination with bare steel roof deck, refer to Gengrui (2023).

## D. PROFILED PANEL DIAPHRAGM SHEAR STRENGTH AND STIFFNESS

### D1 Profiled Panels

The *profile panel* limits reflect the range of *panels* in the research by Luttrell (SDI, 1981), 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.

#### D1.1 Profiled Panel Diaphragm Shear Strength

The *safety* and *resistance* factors in Table D1.1-1 of this *Standard* are from AISI S310-20 Table B1.1-1 for *profiled panels* attached to steel supports, and *Standard* Section D1.1.4.1 for *profiled panels* attached to wood supports. The *resistance* and *safety* factors were recalibrated when the original AISI S310-14 *Standard* was developed. This included data from a database of full-scale *diaphragm* tests from Luttrell (1967), Ellifritt (1970), SDI (1981), MCA (1999), and Bagwell and Easterling (2008). Screw test data includes SDI (1981) and Bagwell (2008).

*Profiled panel nominal shear strength* [resistance] governed by *connection* strength is highly influenced by the concept of weak or strong *sidelap* conditions.

In a weak *sidelap* condition, the *sidelap connections* yield or have insufficient *stiffness* compared to the *support connections*; therefore, the *sidelap* slips relative to the adjacent sheet. The sheets in the weak *sidelap* condition rack as shown in Figure C-D1.1-1a. This creates a condition in which the corner *connections* of the *profiled panel* will have a resultant force that is resisting both the shear force across the end of the sheet and the shear force from the movement of the slipping *sidelap* as shown in Figure C-D1.1-3. *Nominal shear strength* [resistance] controlled by *connections* at *interior panels* ( $S_{ni}$ ) and *support connections* at the corners of *interior panels* or *edge panels* ( $S_{nc}$ ) are characteristic of the weak *sidelap* behavior.

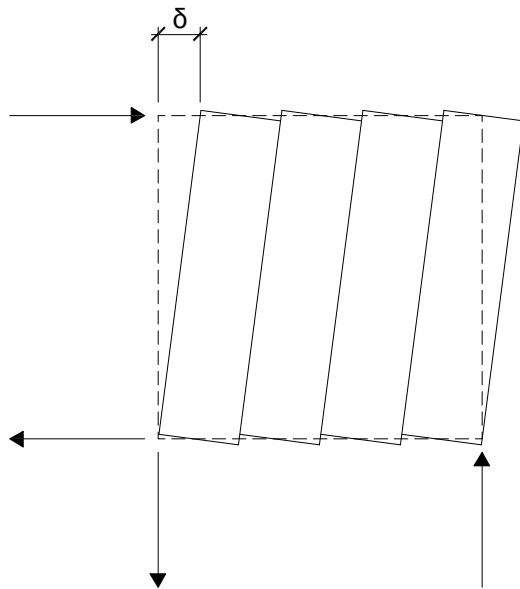


Figure C-D1.1-1a Weak Sidelap

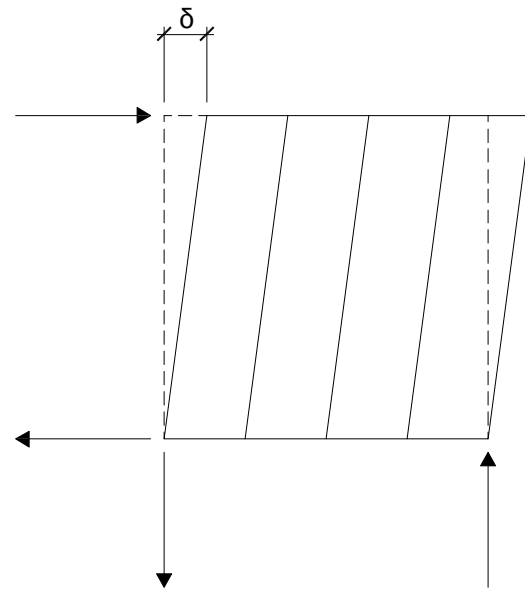
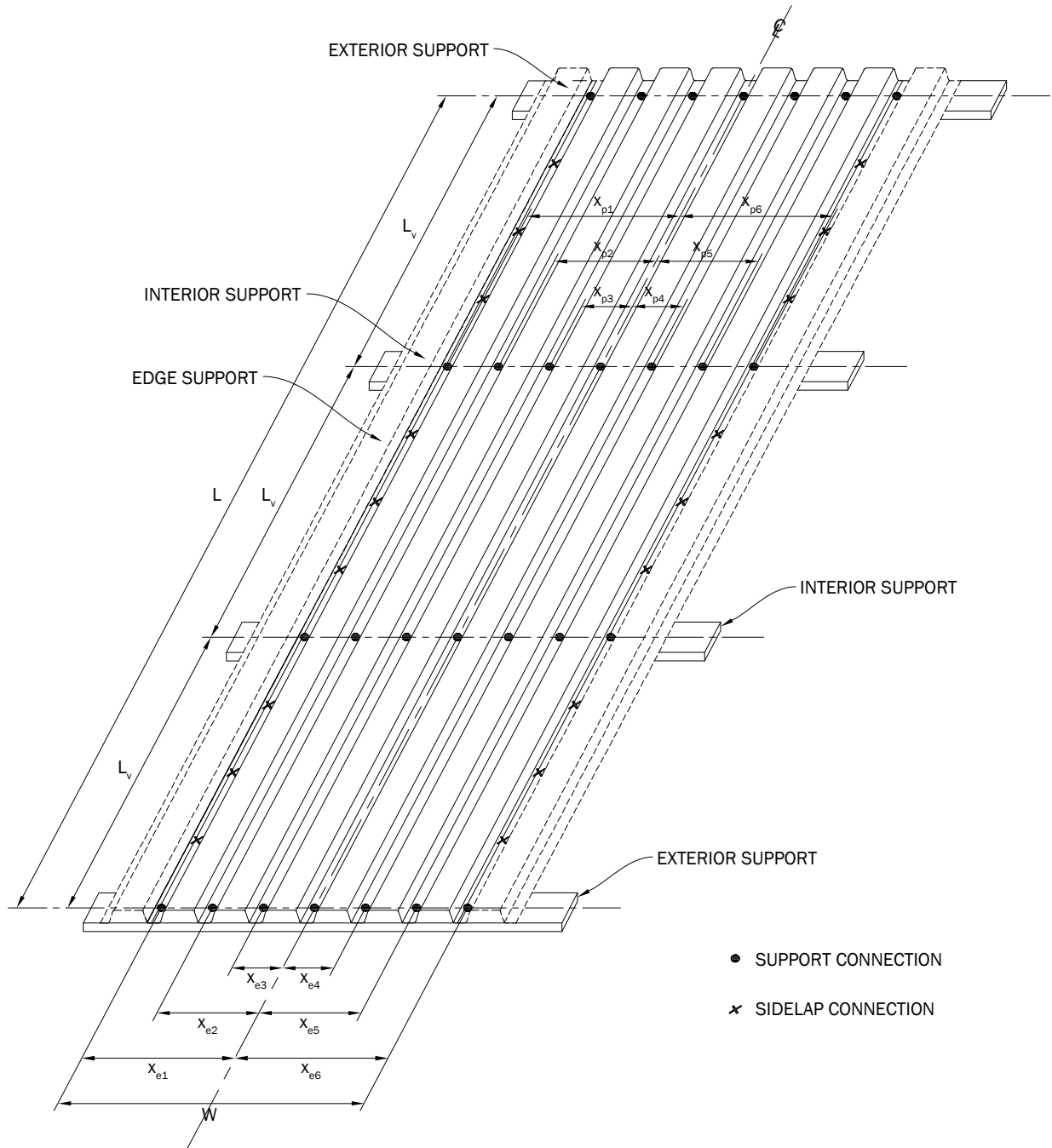


Figure C-D1.1-1b Strong Sidelap

When the *sidelap* is strong, it does not slip and this drives the governing shear of the *profiled*

*panel* to the ends of the *panel*. The *profiled panels* in a strong *sidelap* condition act more like one large continuous *profiled panel* as shown in Figure C-D.1-1b. Nominal shear strength [resistance] controlled by *connections* along the edge parallel to the *panel* span in an *edge panel* and located at a *diaphragm* reaction line ( $S_{ne}$ ), and *connections* along the ends of *interior panels* or *edge panels* and into *exterior supports* ( $S_{np}$ ), are characteristic of strong *sidelap* behavior. For stability modes of failure, nominal shear strength [resistance] controlled by *profiled panel* out-of-plane buckling ( $S_{no}$ ) and *profiled panel* local web buckling over *exterior supports* ( $S_{nl}$ ) occurs when the strong *sidelap* condition is exhibited and the *connections* do not yield.

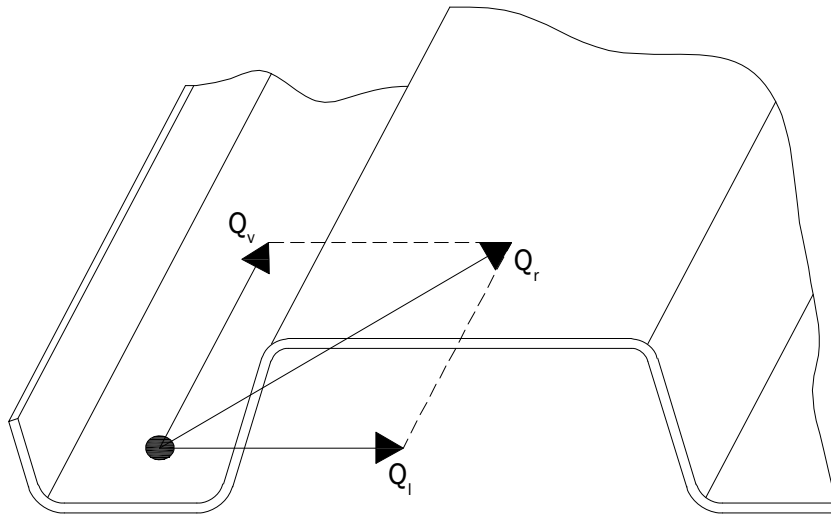
Nominal shear strength [resistance] controlled by connections at interior panels ( $S_{ni}$ ) is determined using Equation D1.1-1. Nominal shear strength [resistance] per unit length controlled by connections at interior panels or edge panels,  $S_{ni}$ , accounts for connection failure in the *profiled panel*. The relaxation term,  $(\lambda-1)$ , represents corner buckling at support connections at the edge and along *sidelaps* of *profiled panel* ends. This relaxation occurs at the compression corners as the *profiled panel* racks in-plane, but the reduction is applied in both directions (tension and compression) for simplicity. Figure C-D1.1-2 illustrates the geometry of connections for the interior panel equation.



**Figure C-D1.1-2 Geometry of Interior Panel Connection Pattern**

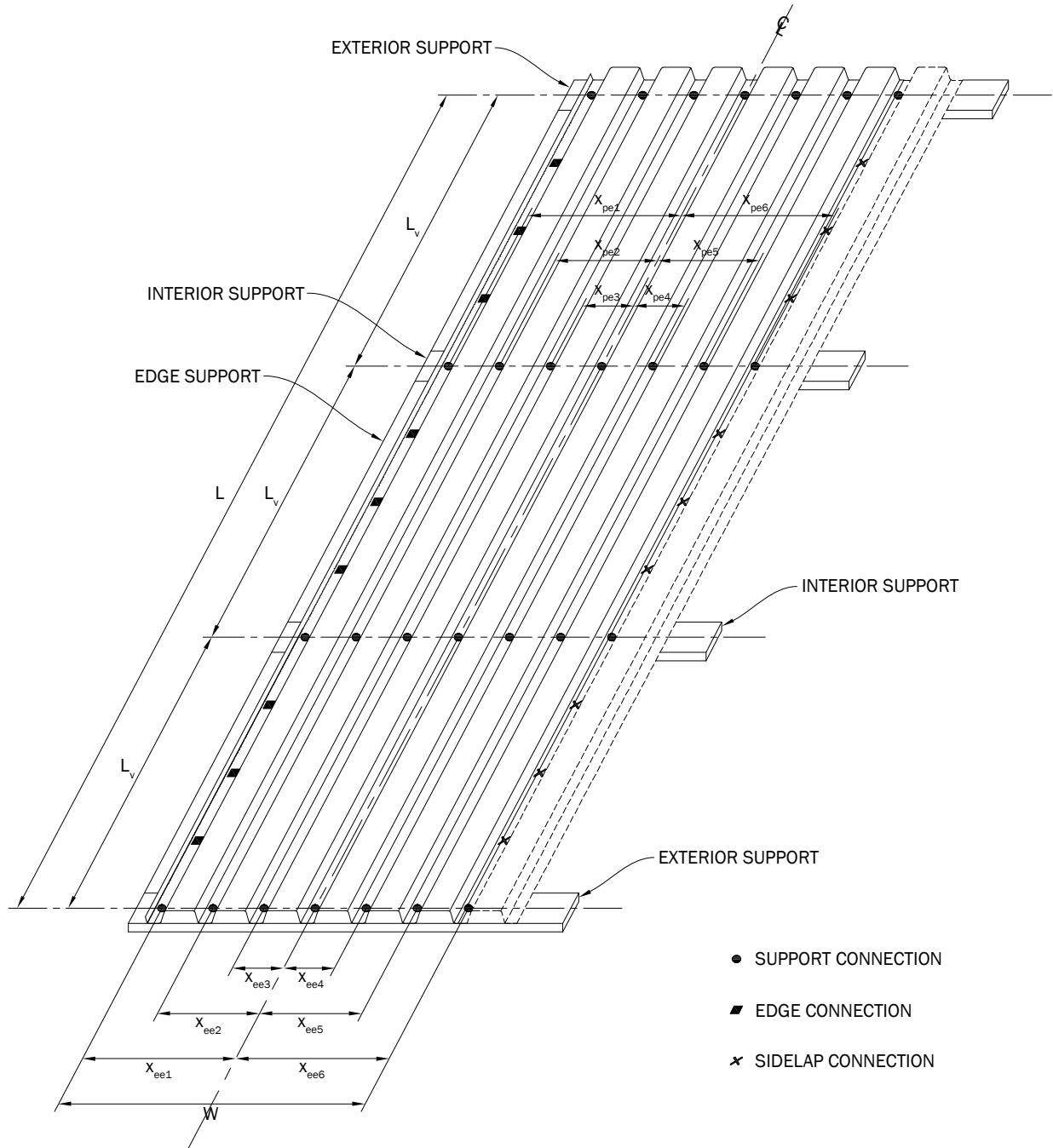


Nominal shear strength [resistance] controlled by *support connections* at the corners of *interior panels* or *edge panels* ( $S_{nc}$ ) is determined using *Standard Equation D1.1-2*. Nominal shear strength [resistance] per unit length controlled by *support connections* at the corners of *interior panels* or *edge panels*,  $S_{nc}$ , accounts for the orthogonal force components and greater demand at the corner connections in each *profiled panel*. Figure C-D1.1-3 depicts the concept of the combination of vectors in the perpendicular,  $Q_v$ , and parallel,  $Q_\ell$ , directions to the *profiled panel* and the resulting force,  $Q_r$ .



**Figure C-D1.1-3 Corner Shear Force and Its Components**

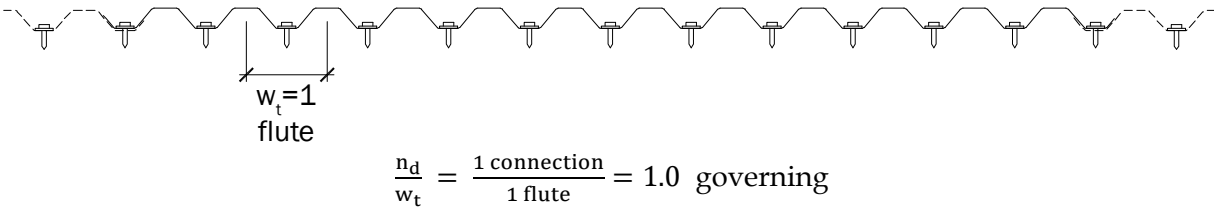
Nominal shear strength [resistance] controlled by *connections* along the edge parallel to the *profiled panel* span in an *edge panel* and located at a *diaphragm chord* or *collector reaction line* ( $S_{ne}$ ) is determined using *Equation D1.1-3*. Nominal shear strength [resistance] per unit length controlled by *connections* along the edge parallel to the *profiled panel* span in an *edge panel* and located at a *diaphragm chord* or *collector reaction line*,  $S_{ne}$  accounts for *connection failure* or *local profiled panel failure* at fasteners along lines where shear is transferred from the *diaphragm* to the lateral force-resisting system. The contribution of all *edge connections* and the *support connections* between the centerline of the *profiled panel* and the reaction line are included. It is acceptable to only consider the *edge connections* and ignore the *support connections* between the centerline of the *profiled panel* and the reaction line by letting  $\alpha_1 = 1$  and  $\alpha_2 = 1$ . In practice, designers will not typically let  $S_{ne}$  control *diaphragm capacity*. Figure C-D1.1-4 illustrates the geometry of *connections* for the *edge panel equation*.



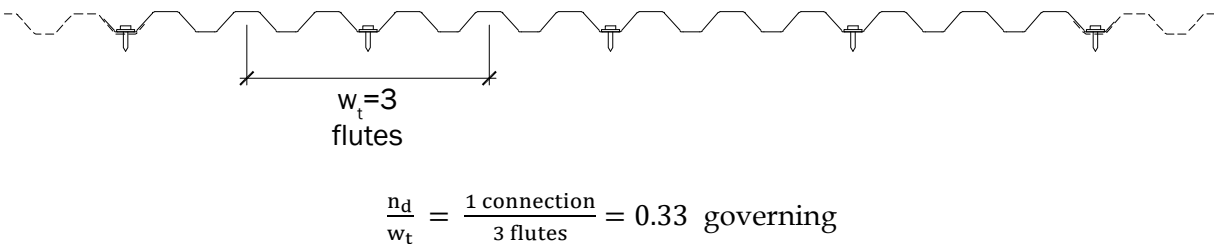
**Figure C-D1.1-4 Geometry of Connections for the Edge Panel**

Nominal shear strength [resistance] controlled by connections along the ends of interior panels or edge panels and into exterior supports ( $S_{np}$ ) is determined using Equation D1.1-4.  $S_{np}$  accounts for connection failure or local profiled panel failure at exterior supports. Attachment pattern has a significant influence on this strength limit (Nunna, 2018a).  $S_{np}$  is the smallest value of bottom flute connection(s) shear strength per tributary width among all the bottom flute support connections. Some examples of the controlling combination for tributary width,  $w_t$ , and number of support connections,  $n_d$ , are shown in Figures C-D1.1-5a, C-D1.1-5b, C-D1.1-5c, and C-D1.1-5d

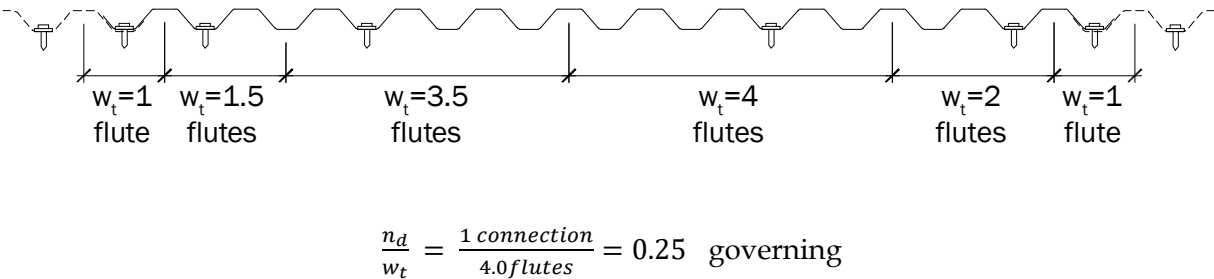
for a variety of representative patterns. For fastener patterns with a support fastener in every bottom flute,  $w_t$  is equal to the pitch of the *profiled panel*.



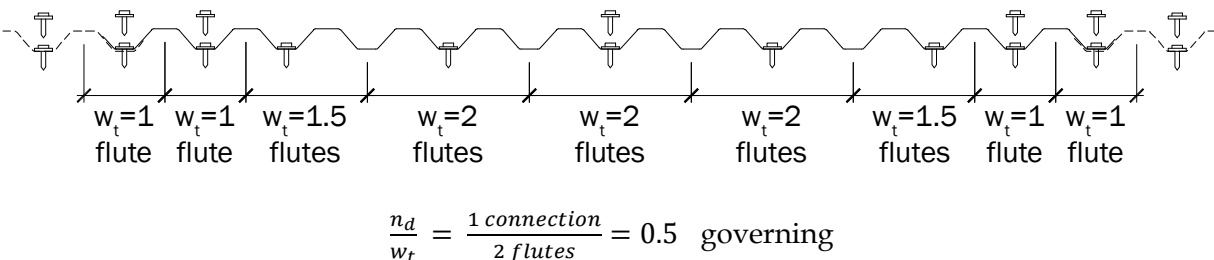
**Figure C-D1.1-5a Tributary Flute Widths  
With Single Connection Per Rib**



**Figure C-D1.1-5b Skip Pattern Tributary Flute Widths  
With Single Connection Per Rib**



**Figure C-D1.1-5c Multiple Tributary Flute Widths  
With Single Connection Per Rib**

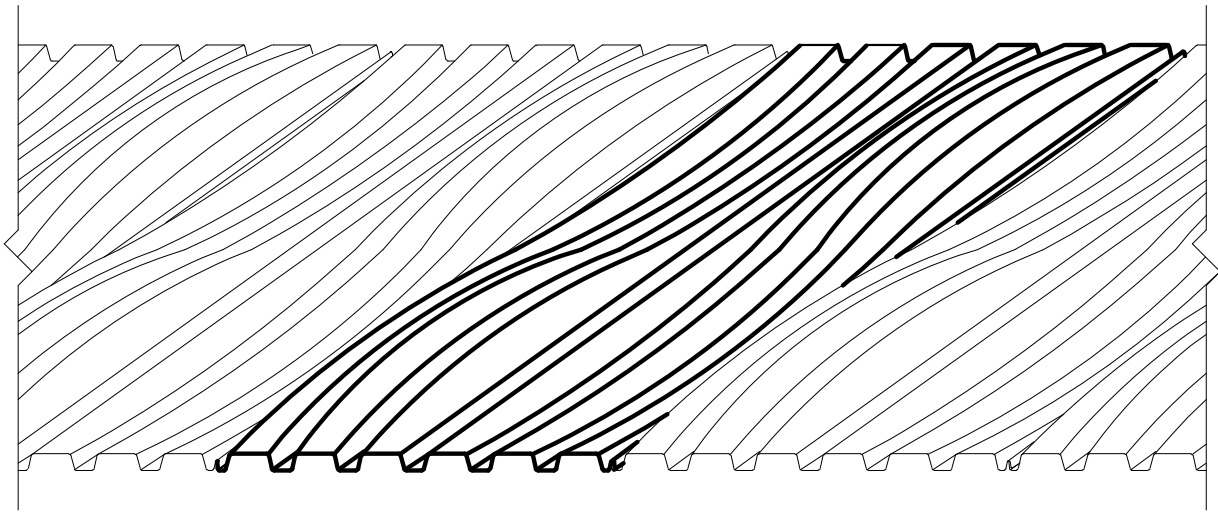


**Figure C-D1.1-5d Tributary Flute Width  
Multiple and Single Connection Per Rib**

Nominal shear strength [resistance] controlled by *profiled panel* out-of-plane buckling ( $S_{no}$ ) is determined using Equation D1.1-5. Figure C-D1.1-6 depicts out-of-plane buckling.  $S_{no}$  accounts for shear buckling which may include relatively large diagonal waves across several *profiled*

*panels* or as column-like *buckling* between supports. This theoretical limit includes the orthotropic nature of the *profiled panel* and represents the same theory used to design corrugated webs in girders. This theory was presented in SDI DDM04 (2015).

Nunna (2011) compared existing *diaphragm* test data with the equations in existing analytical models. The 28 tests exhibited *profiled 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 DDM03 (2004) 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 D1.1-1. The evaluation results were rationally extended to the entire acceptable range of *Standard* Section D1.1.



**Figure C-D1.1-6 Panel Shear Buckling**

Nominal shear strength [resistance] controlled by *profiled panel* local *web buckling* over exterior support ( $S_{nl}$ ) is determined using Equation D1.1-6.  $S_{nl}$  accounts for *local buckling* in the *web* caused by end warping. This theory was presented in a report by Nunna (2018b). Significant end warping is observed prior to *local web buckling*. As end warping behavior becomes more extreme, forces are transferred through the *profiled panel webs* in the form of tension and compression until the *web* fails in compression. This failure mode is similar to the failure mode observed in *web-crippling* tests on *profiled panels*. A modified *web-crippling* equation based on AISI S100 Equation G5-1 is used to calculate the *web buckling* strength of the *profiled panel web*. Slenderness of the *web* and bearing length influence *local web buckling* capacity. Figure C-D1.1-7 depicts *local web buckling*.

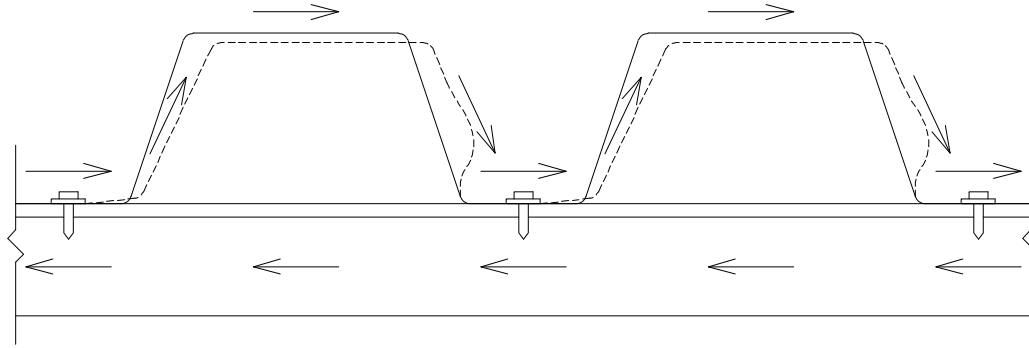


Figure C-D1.1-7 Local Web Buckling

## D1.2 Profiled Panel Diaphragm Shear Stiffness

The calculation of *shear stiffness* for a *diaphragm* or *wall diaphragm* system with *profiled panels* is based on SDI DDM04 (2015) and Luttrell (Luttrell, 1999a and 1999b; and MCA, 2004). It was developed based on the *profiled panel* as shown in *Standard Figure D1.1-1*.

For a given shear stress, more shear and warping displacements occur in longer elements of a profile with an open cross-section. Those impacts have been considered in the generalized stiffness equation where the shear deformation impact is considered in the first term of the denominator and warping deformation impact is considered in the warping factor,  $D_n$ . The support factors,  $\gamma_c$ , are based on tests and taken from SDI DDM01 (SDI, 1981).

End warping of the flutes of the *profiled panel* has a significant impact on the *shear stiffness* of the *profiled panel*. This is represented by the warping factor,  $D_{iv}$  and the support factor for warping,  $\gamma_c$ . The end warping addresses flute roll-over as shown in Figure C-D1.2-1 due to the *connection* to supports through the bottom flange of the *profiled panel* and the shear flow pushing the top flute over the bottom flute.

This is further magnified by the attachment pattern of the *profiled panel* to supports. The end warping involves a weighted average of the individual flute warping factors as shown in Figures C-D1.2-2a, C-D1.2-2b, C-D1.2-2c, and C-D1.2-2d for *profiled panels*. If every flute is attached to the support, the end warping involves just one flute. In addition, the figure demonstrates the determination of the warping factor  $D_n = D/L$ , using the weighted average for  $D$ .

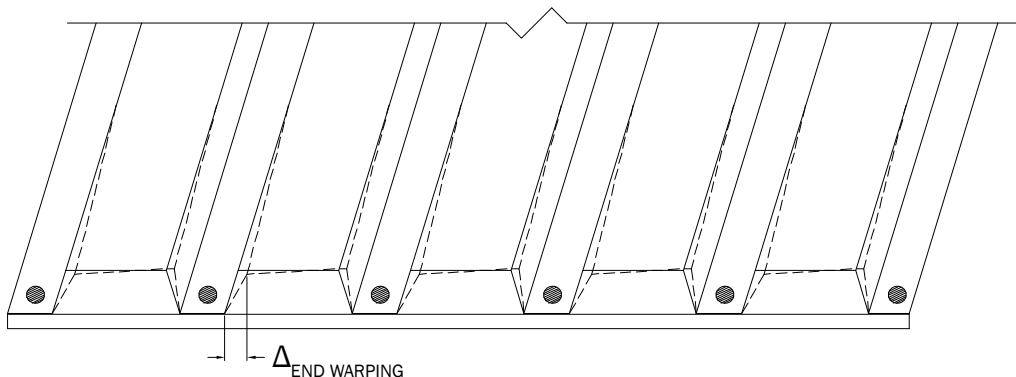


Figure C-D1.2-1 End Warping



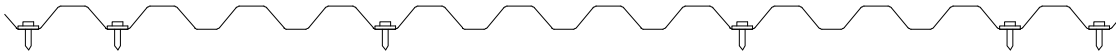
$$D = \frac{U_1 D_1}{U_1} = \frac{12 D_1}{12}$$

**Figure C-D1.2-2a Determination of D for Every Flute Attached**



$$D = \frac{U_2 D_2}{U_2} = \frac{12 D_2}{12}$$

**Figure C-D1.2-2b Determination of D for Every Other Flute**



$$D = \frac{U_1 D_1 + U_3 D_3}{U_1 + U_2 + U_3} = \frac{2U_1 + 6U_3 + 4D_4}{2 + 6 + 4}$$

**Figure C-D1.2-2c Determination of D for Non-Uniform Attachment Pattern**

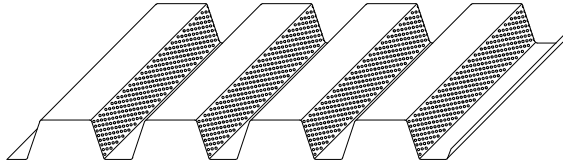


$$D = \frac{U_1 D_1 + U_2 D_2 + U_3 D_3 + U_4 D_4}{U_1 + U_2 + U_3 + U_4} = \frac{1U_1 + 4U_2 + 3U_3 + 4D_4}{1 + 4 + 3 + 4}$$

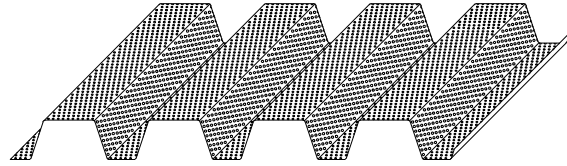
**Figure C-D1.2-2d Determination of D for Alternate Non-Uniform Attachment Pattern**

## D2 Perforated Profiled Panels

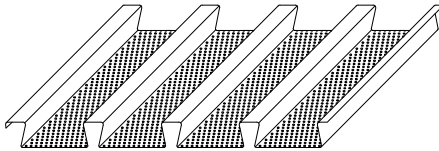
*Perforated profiled panels* are used for a variety of applications. Acoustic steel roof deck *panels* have perforated bands that attenuate reflected sound in a structure in combination with acoustic media, commonly fiberglass batts. Architectural use of *perforated profiled panels* includes sunshade *panels*, roof screen *panels*, and soffit *panels*. *Perforated profiled panels* may have perforations in specific locations, such as *webs*, top flanges, or bottom flanges, or the entire *profiled panel* may be perforated. Figures C-D2.1-1a, C-D2.1-1b, C-D2.1-1c and C-D2.1-1d are common examples of web-perforated, total perforated, bottom flange perforated and total *perforated panels* respectively.



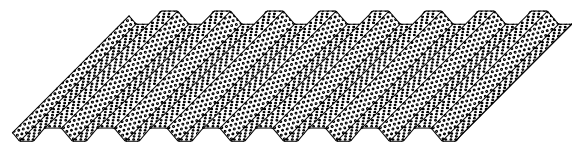
**Figure C-D2.1-1a Web  
Perforated Steel Roof Deck**



**Figure C-D2.1-1b Total  
Perforation Steel Roof Deck**



**Figure C-D2.1-1c Bottom  
Flange Perforated Roof Deck**



**Figure C-D2.1-1d Total  
Perforation Screen Panel**

The perforations impact the strength and potentially the flexibility of *connections* in the *perforated profiled panels*. The strength and flexibility of support *connections* or *sidelap connections* in perforated areas of the *panel* require testing. Design strengths determined using the equations in AISI S100 and Appendix 1 of this *Standard* all assume that the *connections* are in solid, non-perforated sheet. Refer to Chapter G of this *Standard* for *connection* testing provisions.

## D2.1 Perforated Profiled Panel Diaphragm Shear Strength

For *profiled panels* with perforations, the reduced moment of inertia and element *stiffness* is considered in the calculation of out-of-plane *buckling* strength of the panel. The influence of reduced element *stiffness* due to perforations is accounted for in the developed flute term. The reduced *stiffness* of the elements is based on Luttrell (SDI, 2011). The *panel* manufacturer may provide the reduced moment of inertia for the *perforated panels* produced.

The influence of perforated *web* elements is considered in the calculation of *perforated profiled panel* local *web buckling* over *exterior support*. The perforated *web* adjustment factor,  $q_s$ , accounts for the reduced effective area of the perforated *web* element and is based on Luttrell (SDI, 2011).

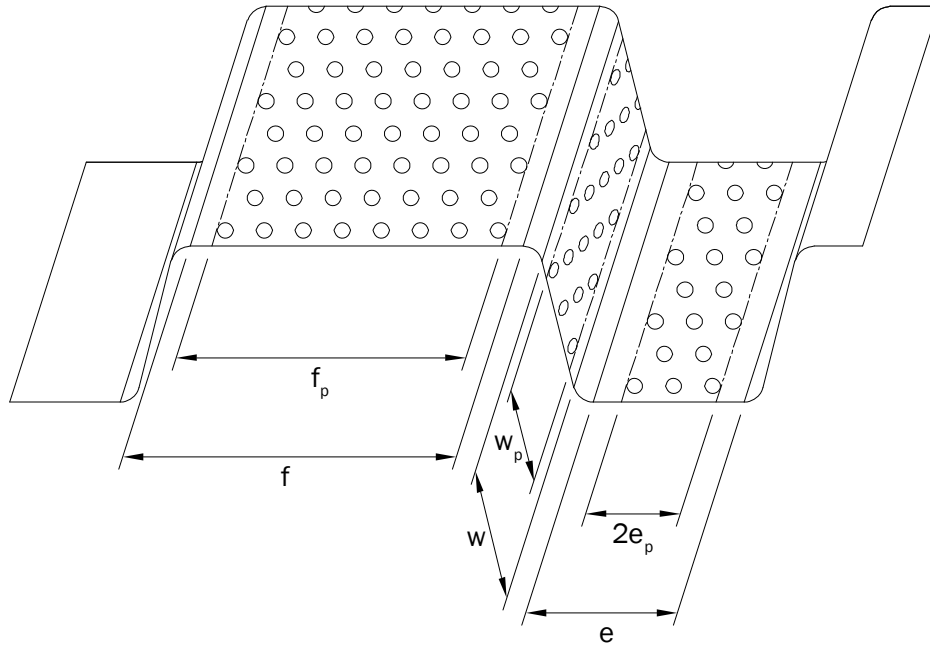
Panel *connection* shear strength in perforated elements must be accounted for in the calculation of the *perforated profiled panel* nominal shear strength [*resistance*] controlled by *connection* strength. The shear strength of *panel connections* in perforated elements is determined by testing and is supplied by the manufacturer.

## D2.2 Perforated Profiled Panel Diaphragm Shear Stiffness

*Perforated profiled panel* in-plane shear *stiffness* is influenced by the *stiffness* of the perforated elements. This influence is accounted for in the end warping factor,  $D_n$ , and the developed flute width,  $s$ , and is based on Luttrell (SDI, 2011).

*Perforated profiled panel connections* in perforated elements are accounted for in the calculation of the slip constant,  $C$ . Stiffness of *perforated profiled panel connections* in perforated elements is determined by testing.

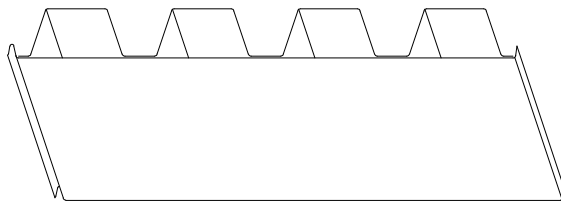
Figure C-D2.2-1 depicts the width of the perforation band and the flat width of the top flute, bottom flute, or *web* of the *perforated profiled panels* for use in determining the development width of *perforated profiled panels* per Equation D2.2-1.



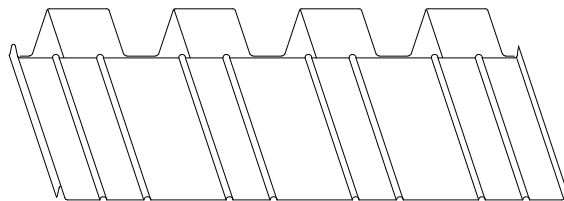
**Figure C-D2.2-1 Perforation and Flat Width of Perforated Profiled Panels**

**D3 Cellular Panels**

The *profiled cellular panels*, most commonly cellular steel deck, covered by this section of the *Standard* are the combination of a top profiled section and a bottom flat pan section that are typically connected with welds or rivets, or clinch *connections*. Figure C-D3-1 depicts this common assembly. The pan section is flat to ensure that the connections to supports engage both the profiled section and the flat pan section. It is common for the pan section to have upward stiffening ribs to improve serviceability without standing the cellular deck off the support members.



**Figure C-D3-1a Profiled/Pan Cellular Deck**

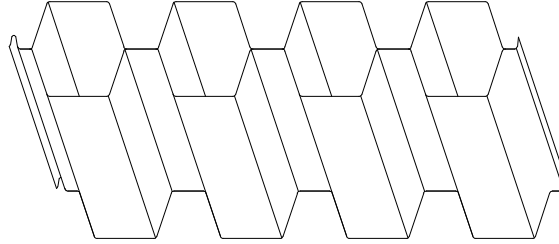


**Figure C-D3-1b Profiled/Pan Cellular Deck With Stiffeners**

Double profiled cellular deck as shown in Figure C-D3-2 exceeds the scope of Sections D3 and



D4 of this *Standard* because the bottom profiled section prevents the upper profiled section from being connected directly to the support. Double profiled cellular deck and other cellular decks may be designed in accordance with Chapter C, Advanced Engineering Mechanics, or Chapter G, Test-Based Design, of this *Standard*.



**Figure C-D3-2 Double Profiled Cellular Deck**

### D3.1 Cellular Panel Diaphragm Shear Strength

The *nominal shear strength [resistance]* determination for *cellular panels* is developed from research performed at Virginia Tech by Bagwell and Easterling (2008). This research demonstrated that the prediction of *connection strength-based nominal shear strength [resistance]* for non-cellular *profiled panels* is applicable to *cellular panels*, with limitations on button punch *sidelap connection strength*. The combined *thickness* of the top *profiled panel* and the bottom panel for the design of *connections* to supports needs to be considered.

For *cellular panels*, the average number of support fasteners per unit width is used to calculate  $S_{np}$ .

The *local web buckling limit state* developed for non-cellular *profiled panels* does not show up as an applicable mode of failure in the Bagwell and Easterling (2008) research for *cellular panels*. This is likely because the bottom panel braces the warping and the sheet ends, limiting the *local web buckling* mode of failure.

### D3.2 Cellular Panel Diaphragm Shear Stiffness

The in-plane *shear stiffness* determination for *cellular panels* is developed from research performed at Virginia Tech by Bagwell and Easterling (2008).

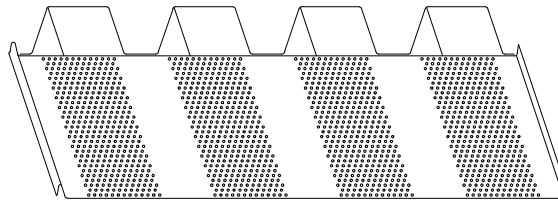
The equations in *Standard* Section D3.2 are adopted from Luttrell (2013). Bagwell and Easterling (2008) evaluated the *shear stiffness* equation in an earlier edition (Luttrell, 2005). Warping distortion in the bottom plate is negligible and tests indicate that warping in the top profiled section is also negligible. The inherent torsional restraint of the closed *profiled cellular panel* resists end warping and  $D_n$  is not present in *Standard* Equation D3.2-1. The bottom plate efficiently resists a significant part of the shear force. *Standard* Equation D3.2-1 modifies *Standard* Equation D1.2-1 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 *profiled cellular panel connections*. *Standard* Equation D3.2-2 adjusts for load sharing and measures the shear flow and stress in the top profiled section. The numerator,  $t$ , of *Standard* Equation D3.2-1 is based on the top profiled section

*thickness of the cellular panel.*

Slippage at *sidelap connections* over and between supports can dominate deflection and depends on the *connection shear flexibilities* (and *thicknesses*) at the *sidelap*.

#### D4 Perforated Cellular Panels

*Perforated cellular panels* have the perforations in either the bottom plate, as shown in Figure C-D4.1-1, or less commonly, in the top profiled section. This product is commonly referred to as cellular acoustic steel deck where the perforations in combination with acoustic batts, typically made of fiberglass, in the cells attenuate reflected sound inside a structure.



**Figure C-D4.1-1 Perforated Cellular Deck**

##### D4.1 Perforated Cellular Panel Diaphragm Shear Strength

*Perforated cellular panels* may be designed using the same methods as non-perforated for *nominal shear strength [resistance]* provided that the *connections* are through the non-perforated portions of the cellular deck. When the *connections* are made through the perforated areas, testing of the *connections* is likely required to determine the strength of the *connections*. The reduced full moment of inertia, considering the perforations, is suitable for the out-of-plane buckling of the *perforated cellular panels*.

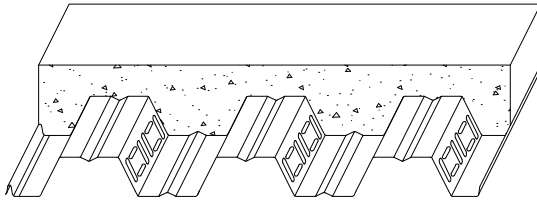
##### D4.2 Perforated Cellular Panel Diaphragm Shear Stiffness

Luttrell (SDI, 2013) provided a method to calculate the shear distribution between the top profiled section and bottom pan element to develop the resultant *diaphragm shear stiffness,  $G'$* . The method considers the increased shear strain due to perforations by calculating an equivalent increased element length for a non-perforated element. *Standard Equation D4.2-1* includes this method and reduces to *Standard Equation D4.2-2* when there are no perforations.

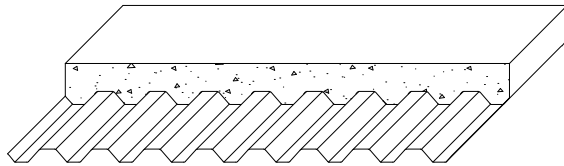
## E. STRUCTURAL CONCRETE FILLED PROFILED PANELS

### E1 Structural Concrete Filled Profiled Panels

*Structural concrete filled profiled panels*, more commonly described as composite steel deck-slabs or non-composite steel deck with concrete fill as shown in Figures C-E-1a and C-E-1b, respectively, are the common applications used for *diaphragm* shear resistance. The *structural concrete* may be normalweight, sand-light weight, or lightweight *structural concrete* following the provisions of ACI 318. Additional design information for composite steel deck-slabs and non-composite steel deck may be found in ANSI/SDI SD2022.



**Figure C-E-1a Composite Steel Deck-Slab**



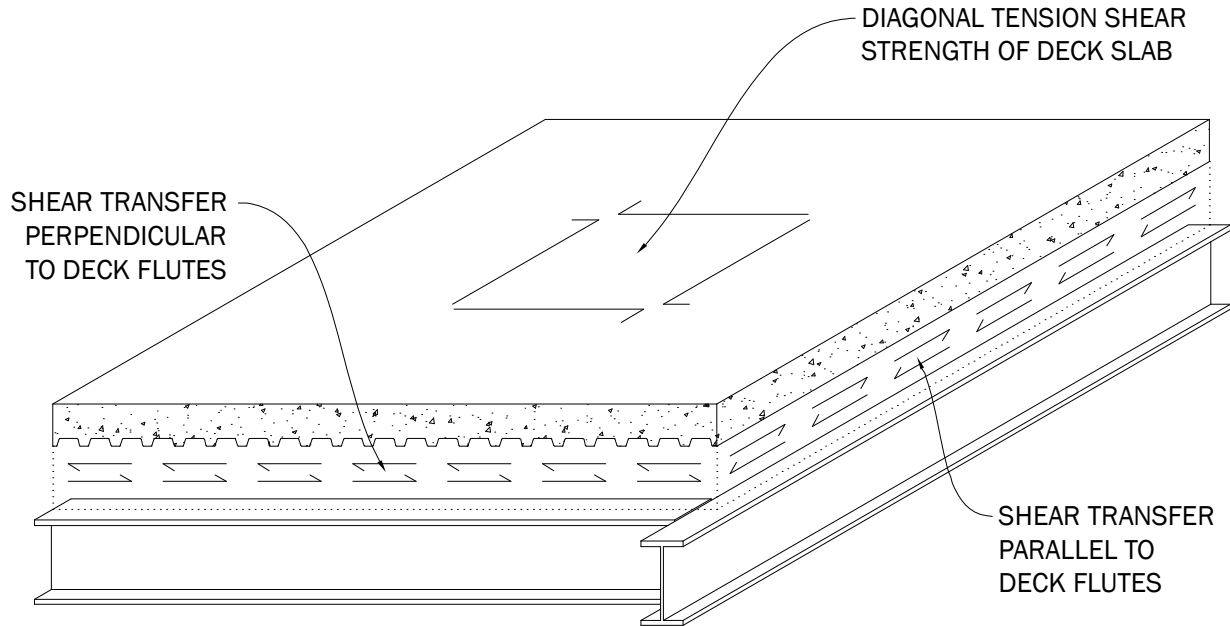
**Figure C-E-1b Non-Composite Steel Deck with Concrete Fill**

The structural concrete filled *profiled panel* limitations reflect the range of test assemblies used to develop the design methods in this *Standard*. 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). Requirements for reinforcement steel that is to be included in the calculation of *diaphragm* shear strength are described in *Standard* Section E1. These requirements do not represent the limits for reinforcement used for flexure or reinforcement used to resist temperature and shrinkage. Reinforcement with smaller reinforcement ratios can be used to resist temperature and shrinkage and the area of reinforcement used to calculate *diaphragm* shear strength is taken as zero.

This section of the *Standard* does not address insulating lightweight concrete covered in Section F.

#### E1.1 Structural Concrete Filled Profiled Panel Diaphragm Shear Strength

The *nominal shear strength [resistance]* of *structural concrete filled profiled panels*, commonly referred to as composite steel deck-slabs or non-composite steel deck with concrete fill, used in *diaphragm* assemblies is based on two distinct *limit states*. The *limit states* are the *nominal shear strength [resistance]* of the *connections* between the concrete filled *profiled panels* to the *diaphragm* chord or collector and the diagonal tension strength of the concrete, reinforcement, and profiled *panel* in the field of the *diaphragm*. Figure C-E1.1-1 depicts the concept of diagonal shear and the *connection* shear transfer to the chord or collectors.



**Figure C-E1.1-1 Diagonal Tension Shear Strength and Perimeter Fastener Shear Transfer**

The available design strength [factored resistance] for concrete filled *profiled panels* is the lesser of the available strength [factored resistance] for the connections to chords and collectors, and the available diagonal tension strength [factored resistance]. The resistance and safety factors for welded headed shear stud anchors were determined as a result of full-scale *diaphragm* tests (O'Brien et al., 2017). The use of the connection resistance and safety factors for welds, bolts, screws, and other connections are based on AISI S100 to ensure that the design of connections based on a resistance factor derived with the target reliability index for connections,  $\beta_o = 3.5$ , would be more conservative than the resistance factor for diagonal tension derived with the target reliability index,  $\beta_o = 2.5$  for members.  $\beta_o = 2.5$  for diagonal tension cracking establishes a clear preference for this *limit state* over the loss of perimeter connections and aligns the *diaphragm* reliability in AISI S310 with the basic member reliability in AISI S100. Further, the presence of the *profiled panel* (steel deck) beneath the concrete fill, in addition to 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.

The nominal shear strength [resistance] per unit length of a structural concrete filled *profiled panel* controlled by connection strength,  $S_{nt}$ , is calculated using Equation E1.1-3. This *limit state* is simply the shear transfer per unit length developed by the connection between the concrete filled *profiled panel* and the chord or collector of the *diaphragm*. This was validated through tests by Easterling and Porter (1994b) and O'Brien et al. (2017).

The nominal shear strength [resistance] per unit length of a structural concrete filled *profiled panel* controlled by diagonal tension cracking is calculated using Equation E1.1-4. The nominal shear strength [resistance] for *diaphragms* with structural concrete fill is calculated based on the shear strength of the concrete slab and considers a *profiled panel* (steel deck) through a transformed cross-section and reinforcement. Standard Equation E1.1-4 was validated for unreinforced *diaphragms* against the Iowa State University tests; see Easterling and Porter

(1994b) and O'Brien et al. (2017). The average thickness of *structural concrete*,  $t_a$ , can be calculated as the cross-sectional area of the *structural concrete* over one *profiled panel* divided by the width of the *profiled panel*. This equation was validated for *diaphragms* with reinforcement against the tests conducted by Avellaneda-Ramirez et al. (2021), which included three specimens incorporating reinforcement. The experiment-to-prediction ratios obtained for the specimens which failed exclusively through diagonal tension cracking were 0.97 and 1.08 while the third specimen with reinforcement experienced a mixed failure mode of diagonal tension cracking and perimeter fastener failure.

Reinforcement designed to resist *diaphragm* in-plane forces is in addition to reinforcement designed to resist other load effects, except that reinforcement designed to resist shrinkage and temperature load effects may also resist *diaphragm* in-plane forces.

## E1.2 Structural Concrete Filled Profiled Panel Diaphragm Shear Stiffness

The *shear stiffness* of structural concrete filled *profiled panel* (steel deck) and *diaphragms* were studied by Avellaneda-Ramirez et al. (2021). It was found that deflections of the cantilever *diaphragm* test specimens were due to four components of deformation: shear deformations, flexural deformations, deformations at the *connections* to the support (e.g., headed shear studs), and deflections of the supports, if applicable.

Due to the relatively small size of the cantilever test specimens, the deformations at the support *connections* contributed approximately 40% of the *diaphragm* deflection on average, but for a typical *diaphragm* span, the support *connections* are shown to contribute less than 1% of the *diaphragm* deflection. If it is desired to consider shear stud deformations along the collector line in the calculation of *diaphragm* deflection, one approach, as described in Avellaneda-Ramirez et al. (2021), is based on the work of Porter and Easterling (1988) and Dodd (1986), which results in Equation C-E1.2-1 for a simple span *diaphragm*. For typical *diaphragm* configurations, spans, typical headed shear stud anchor size, and shear stud spacing, the deformation of the shear studs is expected to be negligible in determination of the concrete filled *panel diaphragm shear stiffness*.

$$\Delta_z = \frac{wL_d}{k_{st}Q_{su}n_s} \quad \text{Eq. C-E1.2-1}$$

where

$\Delta_z$  = *Diaphragm* deflection due to *connection* deformation, in. (mm)

$w$  = Uniformly distributed lateral load applied to the *diaphragm*, kip/ft (kN/m)

$L_d$  = *Diaphragm* span between shear walls or reaction lines, ft (m)

$K_{st}$  = *Stiffness* of a single headed shear stud

= 145.3 (kip/in)/kip for U.S. Customary units

= 5.72 (kN/mm)/kN for SI units

$Q_{su}$  = *Strength* of one headed shear stud in the direction of loading, kip (kN)

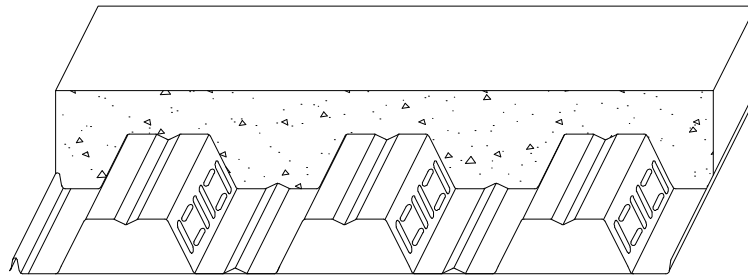
$n_s$  = *Number* of headed shear studs in the direction of loading

The equation for *diaphragm shear stiffness*,  $G'$ , given by *Standard Equation E1.2-1* was derived by Avellaneda-Ramirez et al. (2021) and is based on the mechanics of shear deformation. The accuracy of the equation was evaluated in the same report against the experimentally observed *stiffnesses* of 25 cantilever *diaphragm* test specimens. Predicted *diaphragm shear stiffness* was calculated using the proposed *shear stiffness*, *flexural stiffness*, and *connection stiffness* using the equation in the previous section, and an allowance for

reaction deflection where applicable. For the 25 test specimens examined, the ratio of experimentally observed *stiffness* to predicted *stiffness* had a mean of 0.98 and standard deviation of 0.21 (Avellaneda-Ramirez et al., 2021).

## E2 Structural Concrete Filled Cellular Panels

The *structural concrete* filled *profiled panels* (steel deck) provisions may be extended to *cellular panels* (cellular steel deck). This is an extension of the theory based on the engineering mechanics of the system for non-cellular *panels*. The *structural concrete* filled *cellular panel* is comprised of the concrete, top *profiled panel*, and the bottom pan forming the *cellular panel* as shown in Figure C-E2-1.



**Figure C-E2-1 Structural Concrete Filled Cellular Steel Deck**

### E2.1 Structural Concrete Filled Cellular Panel Diaphragm Shear Strength

The diagonal tension strength of the *structural concrete* filled *cellular panels* (cellular steel deck) is a combination of the concrete strength and the strength of the top *profiled panel* of the *cellular panel*. The contribution of the flat bottom pan to the strength of the *structural concrete* filled *cellular panel diaphragm* shear strength is unknown. The bottom pan of the *cellular panel* that is not in contact with the concrete may not be stiff enough to prevent buckling compared to the top profiled section in contact with the concrete. It is therefore considered conservative to ignore the potential added strength of the bottom pan of the *cellular panel* unless testing or *stiffness* modeling demonstrate otherwise.

### E2.2 Structural Concrete Filled Cellular Panel Diaphragm Shear Stiffness

The in-plane *shear stiffness* of the *structural concrete* filled *cellular panel*, cellular steel decks, is a combination of the concrete stiffness and the stiffness of the top *profiled panel* of the *cellular panel*. The contribution of the flat bottom pan to the stiffness of the *structural concrete* filled *cellular panel diaphragm shear stiffness* is unknown. The bottom pan of the *cellular panel* that is not in contact with the concrete may not be stiff enough to prevent buckling compared to the top profiled section in contact with the concrete. It is therefore considered conservative to ignore the potential added stiffness of the bottom pan of the *cellular panel* unless testing or stiffness modeling demonstrate otherwise.

## F. INSULATING CONCRETE FILLED PANELS

### F1 Insulating Concrete Filled Profiled Panels

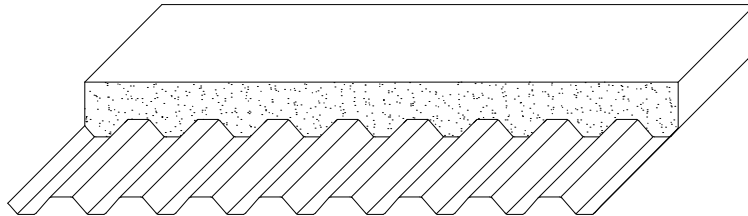
*Insulating concrete* is installed over *profiled panels* (non-composite or roof deck) to provide wind uplift resistance, fire resistance, thermal insulation, and a base for roofing system attachment. The *insulating concrete* may contribute to the *diaphragm* system performance compared to that of the *profiled panels* alone. *Insulating concrete* consists of differing combinations of Portland cement, water, aggregates (vermiculite or perlite) or cellular foaming agent, dependent on specific requirements. The provisions in the *Standard* reflect historic design methods and are suitable for both new construction and analyzing existing structures, which meet the requirements of this section.

A rational alternative to using the *profiled panel nominal shear strength* with *insulating concrete* is to ignore the strength contribution of the *insulating concrete* and use the *profiled panel* provisions of Section D1.1 to determine the *insulating concrete filled profiled panel nominal shear strength*. It may not be appropriate to ignore the *shear stiffness* contribution of the *insulating concrete* fill because the stiffer *diaphragm* may develop a different load distribution to the vertical lateral force-resisting system than that of a flexible *profiled panel diaphragm* without insulating concrete fill.

#### F1.1 Insulating Concrete Filled Profiled Panel Diaphragm Shear Strength

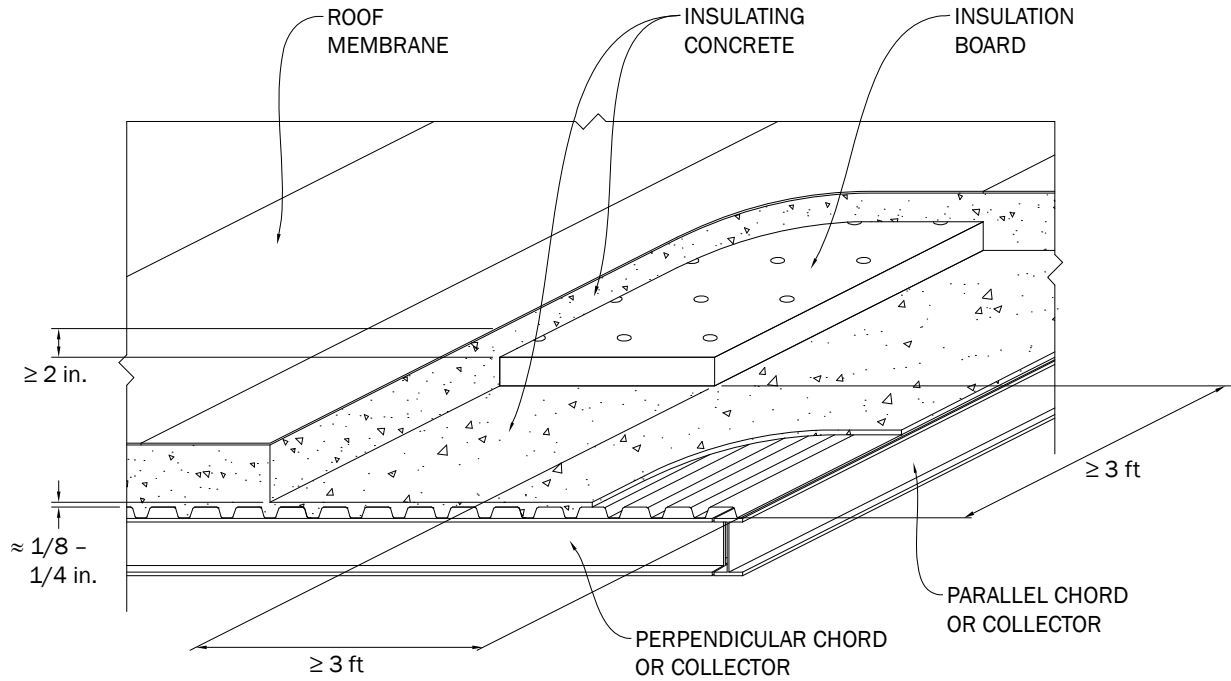
The equations provided in *Standard* Section F1.1 are adopted from SDI DDM03 (2004).

*Nominal shear strength [resistance]* controlled by *connections* at *interior panels* or *edge panels* for *insulating concrete filled profiled panels* without insulating board in the fill ( $S_{ni}$ ) is determined using Equation F1.1-1. This is for *profiled panels* in lightweight insulating concrete fill without embedded insulation board as shown in Figure C-F1.1-1.



**Figure C-F1.1-1 Lightweight Insulating Concrete Filled Profile Panel**

*Nominal shear strength [resistance]* controlled by *connections* at *interior panels* or *edge panels* for *insulating concrete filled profiled panels* with insulating board in the fill ( $S_{ni}$ ) is determined using Equation F1.1-2. Rigid insulation boards of expanded cellular polystyrene, having holes in about 2% of the board surface area, are embedded into *insulating concrete* which covers the top flutes of the deck, before a 2-in. (50-mm)-thick topping of *insulating concrete* is placed over the polystyrene to support the roof membrane. The insulation boards are held 3 ft (1 m) back from the *diaphragm* chords or collectors, so the *insulating concrete* is full depth in those zones as shown in Figure C-F1.1-2.



**Figure C-F1.1-2 Typical Layers of Insulating Concrete Filled Profiled Panel**

**F1.2 Insulating Concrete Filled Profiled Panel Diaphragm Shear Stiffness**

The equations provided in *Standard* Section F1.2 are adopted from SDI DDM03 (2004).

**F2 Insulating Concrete Filled Cellular Panels**

**F2.1 Insulating Concrete Filled Cellular Panel Diaphragm Shear Strength**

The equations provided in *Standard* Section F2.1 are adopted from SDI DDM03 (2004).

**F2.2 Insulating Concrete Filled Cellular Panel Diaphragm Shear Stiffness**

The equations provided in *Standard* Section F2.2 are adopted from SDI DDM03 (2004).



## G. TEST-BASED DESIGN

This edition of AISI S310 primarily points to AISI S100 Chapter K for the provisions of testing cold-formed steel *connections*, members, and assemblies. This is a significant shift from AISI S310-20 that contained test-based provisions that were substantially independent of AISI S100. When AISI S310-14 was originally drafted, the provisions of AISI S100-07 Chapter F, Test for Special Cases, specifically excluded *diaphragms* and did not have provisions to compare a range of test configurations to an analytical theory. The updated test-based design provisions in AISI S100-12 Chapter G that carry through to the current AISI S100 Chapter K provisions eliminate the exclusion of *diaphragms* and provide methods to compare a range of test configurations to theory. This allows AISI S310 to reference AISI S100 for test-based design that applies to all cold-formed steel, including *panels*, rather than detailing *panel*-specific test requirements in AISI S310. This is intended to streamline the *Standard* and ensure that all cold-formed steel test-based designs meet the same standards.

### G1 Panel Connections

Test-based strength for *panel connections* is an option for the design of *profiled panels*, including *structural concrete filled panels*, and *insulating concrete filled panels*. Test-based strength of *panel connections* opens the door to a wide array of *connections* that exceed the scope of this *Standard* and AISI S100. These include *power-actuated fasteners* (also referred to as power-actuated nails or points), specialty screws, self-drilling screw stud anchors, and clinch *connections* for *sidelaps*. This provides the industry with the ability to offer code-compliant proprietary solutions through the prescriptive test methods for connectors.

#### G1.1 Test Standards

AISI S905 is a test standard for cold-formed steel *connections* or cold-formed steel *connections* to hot-rolled steel. This standard provides a variety of test configurations for shear, flexibility, and tension loading of *connections*. The methods included are suitable for both *panel* to steel support and *panel* to *panel* (*sidelap*) *connections*.

AISI S923 is a test standard for shear connectors for concrete filled steel deck or cast-in-place concrete connected to steel support members. This is commonly used for composite cold-formed or hot-rolled steel beams. The method is also suitable for determining the *nominal shear strength* [*resistance*] and *shear stiffness* for the transfer of *diaphragm* forces to chords and collectors.

ASTM D1761 is a test standard for wood *connections* and *connections* of steel side plate members to wood support members. This method is suitable for testing the strength of dowel-type fasteners (screws, nails, bolts, or spikes) connecting steel *panels* or other steel members to wood supports. When using this method, the ultimate strength needs to be identified and the *connection* flexibility needs to be captured following the guidance of AISI S905 at  $0.4P_u$ .

ASTM E1190 and ASTM E488 are test methods for anchors in concrete. These methods would be suitable for developing the strength of *connections* of profiled steel *panels* or other steel members to concrete supports. Examples of this would be *power-actuated fasteners*, concrete screws, and wedge bolts that may be used to attach steel *panels* directly to concrete support members. When using these methods, the *connection* flexibility needs to be captured following the guidance of AISI S905 at  $0.4P_u$ .

## G1.2 Panel In-Plane Shear Strength and Stiffness Using Test-Based Connections

When using test-based *connection* strength in combination with an analytical *nominal shear strength* [*resistance*] and *shear stiffness* theory in Chapter C, D E, or F, the determination of the appropriate governing *resistance factor*, or corresponding *safety factor*, is critical. The *resistance factor* for these sections ties back to system-level behavior that encompasses the variability of the analytical method to full-scale *diaphragm* tests, the variability of the *connections* in the tests, tolerances of the assembly, and material variation. The variability of the *connection* is one element of this overall variability. To ensure that the *diaphragm* design is safe when using test-based *connections* without full-scale *diaphragm* testing, the *resistance factor* is limited to that of the *connection* testing and cannot exceed that of the *diaphragm* system in Chapter C, D, E, or F. The basis for this is that if the *resistance factor* for *connections* exceeds that of the *diaphragm* system, then the *connection* is inherently less variable than that used to develop the *diaphragm* system *resistance factor*; therefore, if this *connection* were used in a full-scale *diaphragm* test, the resulting *resistance factor* would be as good or better than the *diaphragm* system. When the *resistance factor* for *connection* level testing is less than the *diaphragm* level *resistance factor*, then the more severe *connection*-level *resistance factor* is applied to the *diaphragm*. At face value, there may be concern that even though this *resistance factor* is more severe, it does not include the additional variability within a full-scale *diaphragm* test. This is true; however, the statistical factors used for individual *connections* from AISI S100 Chapter K are more severe than those for the *diaphragm* system. Therefore, the potential to be unconservative is reduced significantly.

## G2 Diaphragm Assemblies

Testing of *diaphragm* assemblies may be used to extend the use of the prescriptive methods in Chapters D, E, and F beyond the limitations of those sections or to develop new analytical methods to predict the *panel nominal shear strength* and *shear stiffness* derived from the *diaphragm* test. Currently, the development of system-level *resistance factors* for *diaphragms* is through full-scale testing and analysis following the provisions of AISI S100 Chapter K.

### G2.1 Test Standards

The primary test standard for panels is AISI S907, *Test Standard for Determining the Strength and Stiffness of Cold-Formed Steel Diaphragms by Cantilever Test Method*. This standard references the cantilever test method in ASTM E455, *Standard Test Method for Static Load Testing of Framed Floor or Roof Diaphragm Constructions for Buildings*. AISI S907 has specific provisions that relate to the historic methods used for *panels* and incorporates provisions for reverse cyclic testing in addition to the quasi-static monotonic method of loading. AISI S907 is suitable for both horizontal floor and roof *diaphragms* and is also suitable for *wall diaphragms* (shear walls) because it specifically captures the shear strength and *shear stiffness* of the *panels* and *connections* with the *stiffness* of the support framing being deducted. In this way, the resulting shear strength and *stiffness* of the tested *panels* may be used for a wide range of applications with a variety of support framing systems. Alternative test standards may be used with consideration of the effect of the shear resistance and *stiffness* of the support framing.

### G2.2 Test-Based Diaphragm Strength and Stiffness

The development of design shear strength and *shear stiffness* for *panels* from testing involves

the development of a *resistance factor* from the statistical provisions of AISI S100 Chapter K and the supplemental *diaphragm*-specific provisions in AISI S310 Section G2.2.

The calibration coefficient,  $C_\phi$ , and the coefficient of variation of load effect,  $V_Q$ , are different from those in AISI S100 because the most probable dead to live load (D/L) ratio is zero. This results in the values given in *Standard*,  $C_\phi = 1.60$  and  $V_Q = 0.25$ .

For *profiled panel* strength, where *connections* govern the failure mode, the target reliability index and statistical factors are generally set to match those in AISI S100 Chapter K except for *diaphragms* with screw *connections* subjected to wind loading. The target reliability index for *diaphragms* controlled by *connections* subjected to seismic loading is set to  $\beta_o = 3.5$  for *LRFD* and  $\beta_o = 4.0$  for *LSD*. To maintain consistency with the traditional *safety factors* used in SDI DDM03, *Diaphragm Design Manual, Third Edition, Steel Deck Institute* (2004), the target reliability index for *diaphragms* that are *connection* controlled and subjected to wind loading is set to  $\beta_o = 2.5$  for *LRFD* and  $\beta_o = 3.0$  for *LSD*.

The statistical factors for *profiled panels* governed by strength: mean value of material factor ( $M_m$ ), coefficient of variation of material factor ( $V_M$ ), mean value of fabrication factor ( $F_m$ ), and coefficient of variation of fabrication factor ( $V_F$ ) are determined by the members in AISI S100 Table K2.1.1-1. The exception is the statistical factors for screws used to calibrate the *resistance factors* for *profiled panels*, which differs from AISI S100 as presented in Table G2.2-1. This has been extended to include *power-actuated fasteners* based on the historic practice.

For *profiled panel* stability, out-of-plane buckling, and web buckling, the target reliability index and statistical data for the determination of *resistance factor* are for members. Plate-like buckling in the field of the *profiled panel* and local web buckling fall within the member category of AISI S100 Chapter K provisions. The target reliability index is set to  $\beta_o = 2.0$  for *LRFD* and  $\beta_o = 2.5$  for *LSD* design basis. The statistical factors: mean value of material factor ( $M_m$ ), coefficient of variation of material factor ( $V_M$ ), mean value of fabrication factor ( $F_m$ ) and coefficient of variation of fabrication factor ( $V_F$ ) are determined by the members as summarized in *Standard* Table G2.2-1, the same as those for members in AISI S100 Table K2.1.1-1. These modes of failure are not *connection*-related; therefore, the target reliability index and statistical factors for members are appropriate.

The *shear stiffness* is taken at 40% of ultimate,  $0.4P_u$ , of the *diaphragm* test. This has been the historic practice from AISI S905 and represents the deflection corresponding to a *safety factor* of 2.5, which corresponds to the approximate maximum allowable design load for *panels* when considering *safety factors* in the range from 2.0 to 3.0. This is still within the elastic range of the *diaphragm* for most assemblies with *panels*. For *diaphragm* tests that are especially non-linear, deriving a linear expression for the *shear stiffness* at  $0.4P_u$  may not be appropriate and may require a more complex expression.

## APPENDIX 1: CONNECTION STRENGTH AND STIFFNESS

### 1.1 Scope

Appendix 1 primarily addresses predictive *connection* shear flexibility methodology that is not addressed in AISI S100. There are also additional *connection* types covered in this Appendix that are not covered in AISI S100, such as arc spot welds with weld washers and specific button punch *connections* primarily used for interlocking steel decks.

### 1.2 Welded Connections

#### 1.2.1 Arc Spot Weld for Sheet(s) Welded to a Thicker Supporting Member Connection Shear Flexibility

The shear flexibility for arc spot welds is subject to the relative *thickness* of the *panel* and support member. Thicker support is intended for a hot-rolled steel beam, HSS, open web steel joist, etc. There is no defined limit; however, thicker has been understood to be in the range of 1/8 in. or greater. These are solutions in which the support will react in an ideally rigid manner in comparison with the high local distortion of the sheet steel in the region of the connection. Equation 1.2.1-1 is adopted from SDI DDM03 (2004).

#### 1.2.2 Arc Spot Weld for Sheet-to-Sheet Connections Shear Flexibility

This section is intended for *sidelap* welds between sheets. For these *connections*, both the top sheet and the bottom sheet will experience localized distortion as shear is applied, leading to higher *shear flexibility* than those *connections* to a support that is relatively rigid. Equation 1.2.2-1 is adopted from SDI DDM03 (2004).

#### 1.2.3 Arc Spot Weld With Washer for Sheet(s) Welded to a Thicker Supporting Member

Weld washers are required for *panels* under 0.028-in. (0.711-mm) thick to reduce burnout of the sheet steel around the arc spot weld as specified in AISI S100 Section J2.2.

##### 1.2.3.1 Arc Spot Weld With Washer Shear Strength

*Standard* Equations 1.2.3.1-1a and 1.2.3.1-1b are adopted from SDI DDM03 (2004).

##### 1.2.3.2 Arc Spot Weld With Washer Connection Shear Flexibility

The shear flexibility for an arc spot weld with a weld washer is set to be the same as that for an arc spot weld to a thicker support per *Standard* Section 1.2.1.

##### 1.2.3.3 Arc Spot Weld With Washer Tension Strength

The effective diameter for an arc spot weld with a weld washer has been set to be the same as that for *shear strength* in *Standard* Section 1.2.3.1.

##### 1.2.3.4 Arc Spot Weld With a Washer Combined Shear and Tension

The shear and tension interaction for an arc spot weld with a washer has been set to be

the same as that for arc spot welds without weld washers.

## 1.2.4 Arc Seam Welds

### 1.2.4.1 Arc Seam Weld for Sheet(s) Welded to a Thicker Supporting Member Connection Shear Flexibility

The *shear flexibility* for an arc seam weld to a thicker support is set to be the same as that for an arc spot weld to a thicker support per *Standard* Section 1.2.1.

### 1.2.4.2 Arc Seam Weld for Sheet-to-Sheet Connections Shear Flexibility

The shear flexibility for an arc seam weld for sheet-to-sheet *connections* is set to be the same as that for an arc spot weld for a sheet-to-sheet *connection* per *Standard* Section 1.2.2.

## 1.2.5 Top Arc Seam Welds

### 1.2.5.1 Top Arc Seam Weld Connection Shear Flexibility

The equations presented in *Standard* Appendix Section 1.2.5 for determining the *connection* flexibilities of top arc seam *sidelap* welds are based on research sponsored by industry and reported by Nunna (2012). *Thickness* affects the *connection* flexibility of a top arc seam *sidelap* weld, while the weld length,  $L_w$ , may have less impact. The *connection* flexibility equation applies over the specified range of Chapter D.

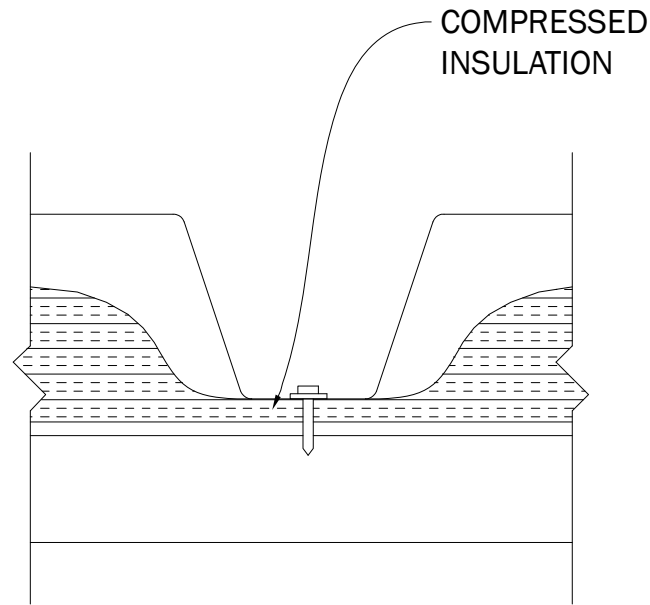
## 1.3 Screw Connections

### 1.3.1 Screw Connection Shear Flexibility

The screw shear flexibility application has been updated for simplicity to reflect the transition between bearing mode of failure when  $t_2/t_1 \leq 1.0$  and tilting mode of failure when  $t_2/t_1 > 2.5$ . Interpolation is accepted when the condition is between those two limiting modes. The traditional flexibility for support fasteners from SDI DDM03 (2004) is used for bearing mode and the *sidelap* flexibility is used for the tilting mode range. This reflects the combination of *thicknesses* that were typical for steel deck when the flexibilities were developed. These requirements parallel the *thickness* ratios for strength provisions, simplifying the design process.

### 1.3.2 Screw Shear Strength and Stiffness Through Insulation

The screw strength, through insulation, is based on the *Primer for Diaphragm Design* (Metal Construction Association, 2004). The strength, with insulation present, can be combined with *diaphragm* system predictive equations in Chapter D for conditions where insulation is sandwiched between the *profiled panel* and the steel support member as shown in Figure C-1.3.2-1.



**Figure C-1.3.2-1 Screw Connection Through Insulation**

#### **1.4 Power-Actuated Fastener (PAF) Connections**

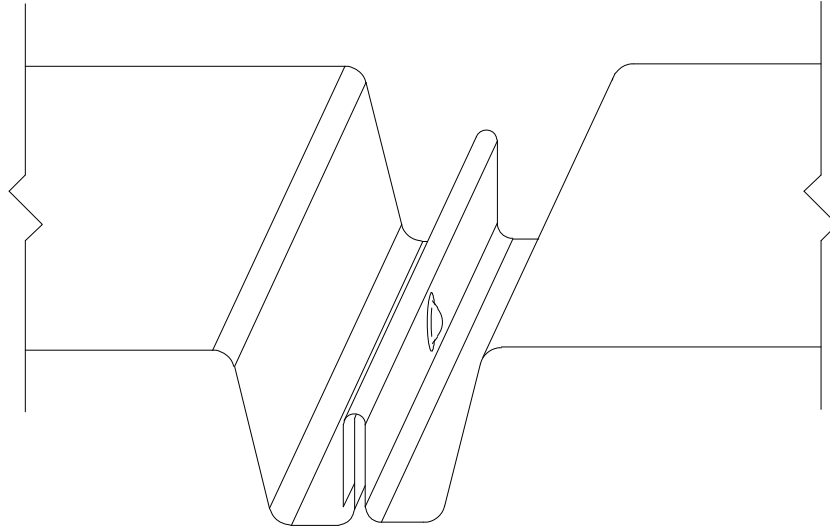
For *panel* assemblies, predictive shear strength, *shear stiffness*, and pull-out design methods for *power-actuated fasteners* fall outside the scope of AISI S100 and AISI S310 Appendix 1. Testing following the provisions of Chapter G is therefore necessary to determine the *power-actuated fastener's* shear strength and *shear stiffness*.

##### **1.4.1 Power-Actuated Fastener (PAF) Combined Shear and Tension**

General combined shear and tension interaction *Standard* Equations 1.4.1-1a and 1.4.1-1b are provided to permit the design of interaction without the requirement for a test program that addresses shear and tension testing interaction.

#### **1.5 Button Punch Connections**

Button punch *connections* are a traditional low cost and low strength method for attaching two sheets of steel roof or floor deck together. The connection strength and *stiffness* present in the *Standard* is a lower bound for all interlock *sidelap* types. The quality of this *connection* is subject to the installation tool, installer, and the *sidelap* of the steel deck when using hand crimp tools. Pneumatic or other power actuated tools produce more consistent connections. See Figure C-1.5-1.



**Figure C-1.5-1 Button Punch**

### **1.5.1 Non-Piercing Button Punch Shear Strength**

The shear strength Equation 1.5.1-1 is adopted from SDI DDM03 (2004).

See *Standard* Section D3.1 for discussion regarding the use of non-piercing button punch with cellular panels.

### **1.5.2 Non-Piercing Button Punch Connection Shear Flexibility**

The shear flexibility Equation 1.5.2-1 is adopted from SDI DDM03 (2004).

## **1.6 Fasteners Into Wood Supports**

### **1.6.1 Screw or Nail Connection Into Wood Support Shear Strength**

This *Standard* specifies the use of ANSI/AWS NDS-2018 and TR-12 for the shear strength of screws or nails for metal *panels* attached to wood supports. ANSI/AWS NDS and TR-12 contain provisions for the design of screw *connections* with steel side plates (the *profiled panel*) to wood support members. Prior to this edition, AISI S310 used the shear strength for wood supports from the *Primer on Diaphragm Design* (Metal Construction Association, 2004) based on the 1986 NDS provisions.

An advantage of the use of the TR-12 provisions is the ability to determine the shear strength of a screw *connection* with an air gap between the panel and the wood support member. This is suitable for installations that have fiberglass batt insulation with little shear resistance sandwiched between the panel and wood support. The insulation may be treated as the air gap for the shear strength of the screw connection.

### **1.6.2 Screw or Nail Connection Into Wood Support Flexibility**

The screw or nail *shear flexibility* Equations 1.6.2-1 and 1.6.2-2 for metal panels attached to wood supports is adopted from the Metal Construction Association (2004).

### **1.6.3 Screw or Nail Connection into Wood Support Combined Shear and Tension**

The interaction of shear and tension for screw or nail *connections* between *profiled panels* to wood supports is adopted from the 2018 NDS with the additional provisions from AISI S100 for combined shear and pull-over, and shear and tension. This follows the direction in the NDS to use its provisions for shear and pullout, and the corresponding standard for the other material attached to the wood member. The AISI S100 provisions for screw pull-over and shear, and tension and shear are therefore adopted following the direction in the NDS for metal side plate (*profiled panel*) connections to wood support members.



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