

Design Guide



**American
Iron and Steel
Institute**

AISI DESIGN GUIDE

Design Examples for the Design of

Profiled Steel Diaphragm Panels

Based on AISI S310-16

2017 Edition

Approved by
AISI Committee on Specifications
Diaphragm Design Task Group



Documents
Document Sharing Hub

**Design Examples for the Design of
Profiled Steel Diaphragm Panels
Based on AISI S310-16**

2017 EDITION

Design Guide D310-17

Approved by
AISI Committee on Specifications
Diaphragm Design Task Group

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

The material contained herein has been developed by the American Iron and Steel Institute Committee on Specifications for the Design of Cold-Formed Steel Structural Members. The Committee has made a diligent effort to present accurate, reliable, and useful information on this cold-formed steel application design guide. The Committee wishes to acknowledge and express gratitude to the author, John Mattingly, of this document.

With anticipated improvements in understanding of the behavior of cold-formed steel framing and the continuing development of new technology, this material will become dated. It is anticipated that AISI will publish updates of this material as new information becomes available, but this cannot be guaranteed.

The materials set forth herein are for general purposes only. They are not a substitute for competent professional advice. Application of this information to a specific project should be reviewed by a design professional. Indeed, in many jurisdictions, such review is required by law. Anyone making use of the information set forth herein does so at their own risk and assumes any and all liability arising therefrom.

1st Printing – June 2017

Copyright by American Iron and Steel Institute, 2017

Preface

This document is developed based on the 2016 edition of AISI S310, *North American Standard for the Design of Profiled Steel Diaphragm Panels*. This supporting document provides five design examples that illustrate the application of the design provisions in AISI S310. Users should not use this document without first reviewing the design provisions included in AISI S310, and should refer to AISI S310 for terminology and equations used in this document.

The material presented in this document has been prepared for the general information of the reader. While the material is believed to be technically correct and in accordance with recognized good practices at the time of publication, it should not be used without first securing content advice with respect to its suitability for any given application. Neither the American Iron and Steel Institute, its members, nor author John Mattingly warrant or assume liability for the suitability of the material for any general or particular use.

This page is intentionally left blank.

Table of Contents

Design Examples for the Design of Profiled Steel Diaphragm Panels Based on AISI Standard S310-16

Design Examples for the Design of Profiled Steel Diaphragm Panels Based on AISI S310-16	iii
2017 EDITION	iii
Design Guide D310-17	iii
Example 1a: Nominal Diaphragm Shear Strength of a Wide Rib Deck Welded to Supports in the Absence of Uplift	1
Example 1b: Edge Detail Parallel to the Edge Panel Span	10
Example 1c: Diaphragm Stiffness for Example 1a Configuration	13
Example 2a: Nominal and Available Diaphragm Shear Strength of a Perforated Panel Connected to a Thin Support in the Absence of Uplift	17
Example 2b: Nominal and Available Diaphragm Shear Strength of a Perforated Panel Connected to a Thin Support With Uplift	25
Example 2c: Required Edge Detail Parallel to the Edge Panel Span With Panel Connected to a Thin Support in the Absence of Uplift	33
Example 2d: Diaphragm Stiffness Calculation	39
Example 3a: Nominal and Available Diaphragm Shear Strength and Stiffness of a Panel on Wood Supports in the Absence of Uplift	46
Example 3b: Nominal Diaphragm Shear Strength of the Example 3a Configuration Subjected to an Uplift Load	55
Example 3c: Nominal Diaphragm Shear Strength per Unit Length, S_n , With Fiberglass Insulation	65
Example 3d: Edge Connections and Details Parallel to the Edge Panel Span	69
Example 4a: Multiple-Span Nominal Diaphragm Shear Strength Equivalent to That of a Single- Span Configuration With Large Side-Lap Spacing	79
Example 4b: Multiple-Span Nominal Diaphragm Shear Strength Equivalent to That of a Single- Span Configuration with Close Side-Lap Spacing	86
Example 5a: Available Diaphragm Shear Strength in the Absence of Uplift Where the Support Thickness Approaches the Panel Thickness	90
Example 5b: Stiffness of the Configuration in Example 5a	96

This page is intentionally left blank.

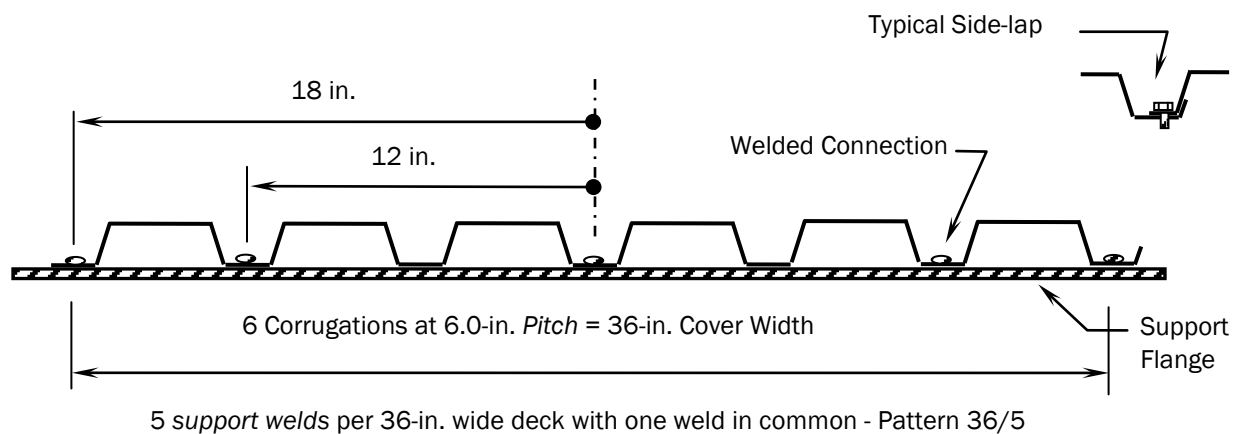
Example 1a: Nominal Diaphragm Shear Strength of a Wide Rib Deck Welded to Supports in the Absence of Uplift

Objective

Calculate the *nominal diaphragm shear strength* per unit length, S_n , and *available strength* of a Wide Rib Deck (WR) in the absence of uplift. WR is defined in AISI S310 Commentary Table C-1.1a.

Note: Other examples of deck, cellular deck, and concrete slab diaphragm designs are available in the Fourth Edition of Steel Deck Institute *Diaphragm Design Manual*.

Diaphragm Configuration



Deck Data

See Figure D2.1-1 for definitions of deck parameters.

Yield stress, F_y	= 40 ksi	Modulus of Elasticity, E	= 29500 ksi
Tensile strength, F_u	= 52 ksi	Panel length, L	= 18.0 ft
Depth, D_d	= 1.47 in.	Cover width, w	= 36.0 in.
Thickness, t	= 0.036 in.	Pitch, d	= 6.00 in.
Top flat width, f	= 3.56 in.	Web flat width, w	= 1.53 in.
Bottom flat width, $2e$	= 1.56 in.		
Moment of Inertia, I_{xg}	= 0.210 in. ⁴ /ft	This is the I_x value from manufacturer and conservatively used for I_{xg}	

Deck is end-lapped (strength of butt-joint will also be determined).

Note: Chapter D limits (a) through (d) are satisfied, and the deck can be designed per Section D1. Material is based on ASTM 1008 SS Grade 40; other steels conforming to AISI S100 Section A3 are acceptable.

Steel Support Data

Yield stress, F_y	= 50 ksi	Tensile strength, F_u	= 62 ksi
Thickness, t	= 0.25 in.	Spacing, L_v (shear span)	= 6.00 ft.

Connection Schedule

Support connection: Pattern = 36/5 – See Figure above
 3/4 in. arc spot weld d = 0.75 in. $F_{xx} = 70$ ksi

The same *support connection* type and spacing will be used at *interior* and *exterior supports*.

Side-lap connection: Spacing = 18 in. o.c. (between supports)
 #10 screw d = 0.190 in. $P_{nss} = 1.5$ k

P_{nss} is the breaking nominal shear strength of the side-lap screw.

Note: Ratio of support thickness to *panel* thickness = 6.94, so welding to the support shouldn't be an issue.

Total thickness of four (4) layers at the end-lap = $4(0.036 \text{ in.}) = 0.144 \text{ in.} < 0.15 \text{ in.}$ – OK per Section D1.1.1 at the end-lap, but 0.036 in. is at the upper limit unless the welding procedure qualifies a greater thickness.

Edge dimensions must be checked using AISI S100 Sections J2.2.1, J4.1 and J4.2 as applicable. This example assumes that the “as produced and installed” deck provides adequate edge dimensions. Consult panel manufacturer for dimensions, and check whether arc spot welds are permitted at end-laps and screws are permitted at *side-laps*.

Determine Available Strength per Eqs. D-1 and D-2

Safety and resistance factors are in Section B1

$$\frac{S_n}{\Omega} = \min\left(\frac{S_{nf}}{\Omega_{df}}, \frac{S_{nb}}{\Omega_{db}}\right) \quad \text{for ASD} \quad \phi S_n = \min(\phi_{df} S_{nf}, \phi_{db} S_{nb}) \quad \text{for LRFD and LSD}$$

$$S_{nf} = \min(S_{ni}, S_{nc}, S_{ne})$$

Note: Chapter D defines the *available strength* as the minimum based on S_{nf} and S_{nb} . S_{nf} is controlled by connection strength. Section D1 defines S_{nf} as the minimum of S_{ni} , S_{nc} , and S_{ne} . An edge detail is designed in Example 1b, so the strength controlled by an *edge panel*, S_{ne} , will not be considered in S_{nf} Example 1a. S_{nb} is defined in Section D2 and controlled by *panel buckling*. S_{nb} is calculated in Example 1a. *Stiffness*, G' , is calculated in Example 1c and is required for serviceability checks.

Calculate Nominal Diaphragm Shear Strength per Unit Length Controlled by Deck Welds, S_{nf} , Using Section D1

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

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

Note: S_{ni} is based on P_{nf} at an *interior support weld*, and S_{nc} is based on P_{nf} at an *exterior support weld*. In this example, the same P_{nf} will be used at all supports.

Calculate support connection strength, P_{nf} (Section D1.1.1 or AISI S100 Section J2.2.2.1):

Cases considered:

- (1) A single-thickness *support connection*, which applies at the top ply of end-laps and at interior flutes over *interior supports*;
- (2) A two-thickness *support connection* at end-laps or *interior support side-laps*;
- (3) A three-thickness *support connection* at end-laps along the *side-lap*; and
- (4) A four-thickness *support connection* at end-laps along the *side-lap*

$$P_{nf} = \frac{\pi d_e^2}{4} 0.75 F_{xx} \quad \text{AISI S100 Eq. J2.2.2.1-1}$$

$$P_{nf} = 2.20 t d_a F_u \quad \text{For } \frac{d_a}{t} \leq 0.815 \sqrt{\frac{E}{F_u}} \quad \text{AISI S100 Eq. J2.2.2.1-2}$$

$$P_{nf} = 0.280 \left[1 + 5.59 \frac{\sqrt{E/F_u}}{d_a/t} \right] t d_a F_u \quad \text{For } 0.815 \sqrt{\frac{E}{F_u}} < \frac{d_a}{t} \leq 1.397 \sqrt{\frac{E}{F_u}} \quad \text{AISI S100 Eq. J2.2.2.1-3}$$

$$0.815 \sqrt{\frac{E}{F_u}} = 0.815 \sqrt{\frac{29500 \text{ ksi}}{52 \text{ ksi}}} = 19.4$$

$$1.397 \sqrt{\frac{E}{F_u}} = 1.397 \sqrt{\frac{29500 \text{ ksi}}{52 \text{ ksi}}} = 33.3$$

Determine weld design parameters (AISI S100 Figure J2.2.2.1-1):Single-thickness *connections*: $d_a = (d - t) = (0.75 \text{ in.} - 0.036 \text{ in.}) = 0.714 \text{ in.}$

$$\frac{d_a}{t} = \frac{0.714 \text{ in.}}{0.036 \text{ in.}} = 19.8$$

$$d_e = 0.7d - 1.5t \leq 0.55d \quad \text{AISI S100 Eq. J2.2.2.1-5}$$

$$= 0.7(0.75 \text{ in.}) - 1.5(0.036 \text{ in.}) \leq 0.55(0.75 \text{ in.})$$

$$= 0.471 \text{ in.} > 0.413 \text{ in.}$$

$$d_e = 0.413 \text{ in.}$$

Two-thickness *connections*: $d_a = (0.75 \text{ in.} - 2(0.036 \text{ in.})) = 0.678 \text{ in.}$

$$\frac{d_a}{t} = \frac{0.678 \text{ in.}}{2(0.036 \text{ in.})} = 9.42$$

$$d_e = 0.7(0.75 \text{ in.}) - 1.5(2)(0.036 \text{ in.}) \leq 0.55(0.75 \text{ in.})$$

$$= 0.417 \text{ in.} > 0.413 \text{ in.}$$

$$d_e = 0.413 \text{ in.}$$

Three-thickness *connections*: $d_a = (0.75 \text{ in.} - 3(0.036 \text{ in.})) = 0.642 \text{ in.}$

$$\frac{d_a}{t} = \frac{0.642 \text{ in.}}{3(0.036 \text{ in.})} = 5.94$$

$$d_e = 0.7(0.75 \text{ in.}) - 1.5(3)(0.036 \text{ in.}) \leq 0.55(0.75 \text{ in.})$$

$$= 0.363 \text{ in.} < 0.413 \text{ in.}$$

$$d_e = 0.363 \text{ in.} \text{ See Note below.}$$

Four-thickness *connections*: $d_a = (0.75 \text{ in.} - 4(0.036 \text{ in.})) = 0.606 \text{ in.}$

$$\frac{d_a}{t} = \frac{0.606 \text{ in.}}{4(0.036) \text{ in.}} = 4.21$$

$$d_e = 0.7(0.75 \text{ in.}) - 1.5(4(0.036 \text{ in.})) \leq 0.55(0.75 \text{ in.})$$

$$= 0.309 \text{ in.} < 0.413 \text{ in.}$$

$$d_e = 0.309 \text{ in. See Note below.}$$

Note: $d_e \geq 3/8 \text{ in.}$ is required by AISI S100 Section J2.2. Therefore, a larger weld size is needed for three-thickness and four-thickness *connections*. For the connection strength calculation in this example, the size is not changed.

Calculate support connection strength at each thickness case:

Single-thickness *connections*: minimum of AISI S100 Eqs. J2.2.2.1-1 and J2.2.2.1-3:

$$P_{nf} = \frac{\pi(0.413 \text{ in.})^2}{4} 0.75 \left(70 \frac{\text{k}}{\text{in.}^2} \right) = 7.03 \text{ k}$$

$$19.4 < \frac{d_a}{t} = 19.8 < 33.3 \text{ Therefore,}$$

$$P_{nf} = 0.280 \left[1 + 5.59 \sqrt{\frac{29500 \text{ ksi}}{52 \text{ ksi}}} \right] (0.036 \text{ in.})(0.714 \text{ in.}) \left(52 \frac{\text{k}}{\text{in.}^2} \right) = 2.89 \text{ k}$$

Two-thickness *connections*: minimum of AISI S100 Eqs. J2.2.2.1-1 and J2.2.2.1-2:

$$P_{nf} = \text{Same value as single-thickness connections} = 7.03 \text{ k}$$

$$\frac{d_a}{t} = 9.42 \leq 19.4 \text{ Therefore,}$$

$$P_{nf} = 2.20(2(.036 \text{ in.}))(0.678 \text{ in.}) \left(52 \frac{\text{k}}{\text{in.}^2} \right) = 5.58 \text{ k}$$

Note: Two thicknesses are adjacent to four thicknesses at end-laps. The number of welds at double thickness is greater (three interior flutes vs. one at side-lap per sheet.)

Three-thickness *connections*: minimum of AISI S100 Eqs. J2.2.2.1-1 and J2.2.2.1-2:

$$P_{nf} = \frac{\pi(0.363 \text{ in.})^2}{4} 0.75 \left(70 \frac{\text{k}}{\text{in.}^2} \right) = 5.43 \text{ k}$$

Note: Consider that 0.363 in. is close enough to 0.375 in. to use the equation.

$$\frac{d_a}{t} = 5.94 \leq 19.4, \text{ Therefore,}$$

$$P_{nf} = 2.20(3(.036 \text{ in.}))(0.642 \text{ in.}) \left(52 \frac{\text{k}}{\text{in.}^2} \right) = 7.93 \text{ k}$$

Four-thickness *connections*: minimum of AISI S100 Eqs. J2.2.2.1-1 and J2.2.2.1-2:

$$P_{nf} = \frac{\pi(0.309 \text{ in.})^2}{4} 0.75 \left(70 \frac{\text{k}}{\text{in.}^2} \right) = 3.94 \text{ k}$$

Note: Consider that 0.309 in. is close enough to 0.375 in. to use the equation.

$$\frac{d_a}{t} = 4.21 \leq 19.4, \text{ Therefore,}$$

$$P_{nf} = 2.20(4(0.036 \text{ in.}))(0.606 \text{ in.}) \left(52 \frac{\text{k}}{\text{in.}^2} \right) = 9.98 \text{ k}$$

In determining diaphragm strength, a minimum connection strength will be used:

$$P_{nf} = \min(7.03, 2.89, 7.03, 5.58, 5.43, 7.93, 3.94, 9.98) = 2.89 \text{ k}$$

Result: $P_{nf} = 2.89$ kips Applies at butt-joint and end-lap installations

Note: Bearing of the deck against the weld at single-thickness controls *connection strength*.

This connection strength is used:

1. As required for other fastener connections in Section D1.1.5, and
2. To calculate S_{ni} and S_{nc} in Section D1 per standard industry practice.

Calculate side-lap connection strength, P_{ns} (Section D1.2.5 or AISI S100 Section J4.3.1):

$$\frac{t_2}{t_1} = \frac{0.036 \text{ in.}}{0.036 \text{ in.}} = 1.0$$

$$P_{ns} = 4.2(t_2^3 d)^{1/2} F_{u2} \quad \text{AISI S100 Eq. J4.3.1-1}$$

$$= 4.2((0.036 \text{ in.})^3 (0.19 \text{ in.}))^{1/2} \left(52 \frac{\text{k}}{\text{in.}^2} \right) = 0.650 \text{ k}$$

$$P_{ns} = 2.7 t_1 d F_{u1} \quad \text{AISI S100 Eq. J4.3.1-2}$$

$$= 2.7(0.036 \text{ in.})(0.19 \text{ in.}) \left(52 \frac{\text{k}}{\text{in.}^2} \right) = 0.960 \text{ k}$$

$$P_{ns} = 2.7 t_2 d F_{u2} \text{ (same)} \quad \text{AISI S100 Eq. J4.3.1-3}$$

$$P_{nss} = 1.5 \text{ k fastener breaking nominal strength. See Connection Schedule.}$$

$$P_{ns} = \min(0.650, 0.960, 1.50) = 0.650 \text{ kips}$$

Result: $P_{ns} = 0.650$ kips **Tilting of screw in deck controls**

Calculate configuration parameters required for S_{nf} :

$$A = 1.0 \quad \text{Number of support welds at side-lap at deck ends}$$

$$\lambda = 1 - \frac{D_d L_v}{240 \sqrt{t}} \geq 0.7 \quad \text{Eq. D1-4a}$$

$$= 1 - \frac{(1.47 \text{ in.})(6.0 \text{ ft})}{240 \sqrt{0.036 \text{ in.}}} = 0.806 \quad \text{Input units are defined in Section D1}$$

$$= \max(0.806, 0.700) = 0.806 \quad \text{Unit-less}$$

$$N = \frac{4 \text{ welds}}{3 \text{ ft}} = 1.33 \frac{\text{welds}}{\text{ft}} \quad \text{Welds into support per ft along deck ends}$$

Note: Although five (5) welds appear in the sketch, one is common to each deck at side-lap. Therefore, $N = 4/3$.

$$\beta = n_s \alpha_s + 2 n_p \alpha_p^2 + 4 \alpha_e^2 \quad \text{Factor defining weld interaction} \quad \text{Eq. D1-5}$$

$$n_p = \frac{L}{L_v} - 1 = \frac{18 \text{ ft}}{6 \text{ ft}} - 1 = 2.0 \quad \text{Number of interior supports} \quad \text{Modified Eq. D1-9}$$

$$n_s = \left(\frac{(6 \text{ ft}) \left(12 \frac{\text{in.}}{\text{ft}} \right)}{18 \frac{\text{in.}}{\text{conn.}}} - 1 \right) \frac{18 \text{ ft}}{6 \text{ ft}} = 9 \quad \text{Number of side-lap connections along panel}$$

$$\alpha_s = \frac{P_{ns}}{P_{nf}} = \frac{0.650 \text{ k}}{2.89 \text{ k}} = 0.225 \quad \text{Connection strength ratio} \quad \text{Eq. D1-6}$$

$$\alpha_p^2 = \alpha_e^2 = \left(\frac{1}{w^2} \right) \sum x_e^2 \quad \text{See Figure D1-1} \quad \text{Eq. D1-8}$$

$$x_{e1} = 0 \text{ in.} \quad x_{e2} = 12 \text{ in.} \quad x_{e3} = 18 \text{ in.}$$

$$\alpha_p^2 = \alpha_e^2 = \left(\frac{1}{(36 \text{ in.})^2} \right) (0^2 + 2(12 \text{ in.})^2 + 2(18 \text{ in.})^2) = 0.722$$

$$\beta = 9(0.225) + 2(2)(0.722) + 4(0.722) = 7.80 \quad \text{Eq. D1-5}$$

Calculate S_{nf} :

$$S_{ni} = [2(1.0)(0.806 - 1) + 7.80] \frac{2.89 \text{ k}}{18 \text{ ft}} = 1.19 \text{ klf} \quad \text{Eq. D1-1}$$

$$S_{nc} = \left(\frac{(1.33 \frac{1}{\text{ft}})^2 (7.80)^2}{(18 \text{ ft})^2 (1.33 \frac{1}{\text{ft}})^2 + (7.80)^2} \right)^{0.5} 2.89 \text{ kips} = 1.19 \text{ klf} \quad \text{Eq. D1-2}$$

$$S_{nf} = \min(S_{ni}, S_{nc}) = \min(1.19, 1.19) = 1.19 \text{ klf} \quad S_{ne} \text{ is in Example 1b}$$

Note: Although not mandatory, S_{ni} and S_{nc} values often are close.

Result: $S_{nf} = 1.19 \text{ klf}$ Based on least weld strength, $P_{nf} = 2.89 \text{ k}$.
 S_{nf} applies at butt-joint or end-lap installations

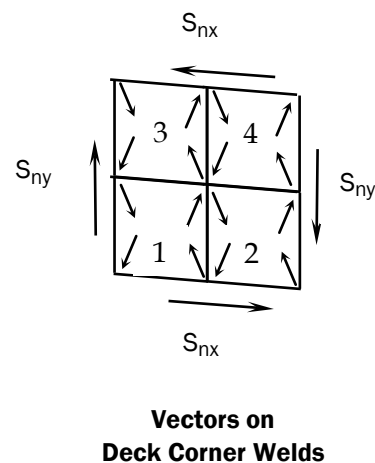
Comment:

Justification to use “one connection strength” by considering shear flow through weld support connections at the end-lap is:

The discussion assumes that proper quality control is maintained during welding and that prequalification procedures have been used to establish proper weld settings and times.

When shear flows from one sheet to the adjacent sheet at end-lap connections, “one support connection strength” is commonly used to determine the strengths S_{ni} and S_{nc} unless different type connections are used or a detail does not develop that strength at all connections.

Let's investigate the shear flow at end-laps in reference to the



Figure, Vectors on Deck Corner Welds. As shown in the figure, the shear in each interior *panel* can be divided into two components, horizontal (x-axis) and vertical (y-axis). The four vertical components are counterbalanced at the connection where four panels connect to the support with one common weld. The horizontal components are in the same direction in Panels 3 and 4 and in the same but opposite direction in Panels 1 and 2. Thus, the horizontal components are also counterbalanced, with one weld penetrating all four plies into the support. There is no force at the weld kernel in the support, but there are still two components bearing against the weld in each ply. The bearing strength in each ply decreases because of the slight decrease in d_a with depth. *Connections* at interior flutes at end-laps are through two plies (Panels 4 and 2, or 3 and 1) and have additive vertical components into the support while the horizontal components are counterbalanced.

If the *panels* are assembled with Panel 1 as the bottom ply followed by 2, 3, and 4, then the shear transfer planes may exist in the weld kernels between Plies 4 and 3, 3 and 2, 2 and 1, and 1 and the support. As discussed above, there is no shear at the last plane. The nominal shear strength in the top ply (Panel 4) is 2.89 k. The shear force in the kernel between Plies 4 and 3 is 2.89 k. The *nominal shear strength* in the top two plies is 5.58 k, average strength for each ply $\approx 5.58 \text{ k}/2 = 2.79 \text{ k}$ and very close to 2.89 k, and the single value used in the calculation is applicable. As shear flows across the plane between Panels 3 and 2, the force in the kernel is twice the horizontal component in the top ply, but less than the weld kernel strength at that plane, 7.03 k (the horizontal force must be less than twice 2.89 k). The *nominal shear strength* of the top three plies is 7.93 k, the average strength for each ply $\approx 7.93 \text{ k}/3 = 2.64 \text{ k}$ and close to 2.89 k. As shear flows across the plane between Panels 2 and 1, the force in the kernel is the same as that between Panels 4 and 3, but less than the weld kernel strength at that plane, 5.43 k (the force limit is 2.89 k). The *nominal shear strength* of the top four plies is 9.98 k, the average strength for each ply $\approx 9.98 \text{ k}/4 = 2.50 \text{ k}$, or 87% of 2.89 k. As shear flows across the plane between Panel 1 and the support, the force is 0.00 k. The most conservative evaluation of the bearing strength in Panel 1 might be based on (d-7t), and P_{nf} is then 2.05 k or 71% of 2.89 k. The center of the bottom ply is 3.5t below the top, and AISI S100 Section J2.2.2.1 uses a 45° decrease in weld size on each side. However, the strength at the adjacent interior flute with a weld in 2.79 k (average of two plies) and redistribution mitigates this reduction.

In summary, S_n could be calculated with $P_{nf} = 2.50 \text{ k}$ at end-laps and 2.89 k at *interior supports*. To exhaust the discussion, this will be done and then compared with S_n calculated with $P_{nf} = 2.89 \text{ k}$ at all *connections*. Common practice is to use one value of P_{nf} to determine S_{nv} , S_{nc} , and S_{ne} to simplify the process.

It should be noted that the vertical shear component in four panels can be in the same direction and resisted by a support, such as an edge panel over an interior lateral force-resisting system. In this case, the reaction shear plane is between the last ply and the support. The *nominal shear strength* of the weld is 3.94 k (based on four layers of thickness) to resist that reaction. However, the shear strength in each panel at either side of the lateral force-resisting system should be based on an average strength for each ply $\approx 9.98 \text{ k}/4 = 2.50 \text{ k}$.

Calculate Nominal Diaphragm Shear Strength per Unit Length Controlled by Panel Connections, S_{nf} , Based on Varying Support Connection Strength at the End-Lap and Interior Supports

Adjust the theory at Eq. D1-1 to cover variation of *support connection shear strength* along a *panel length*, L :

Note: *Diaphragm theory uses full nominal strength in one ply at all connections.*

Let P_{nfe} = Connection nominal strength at exterior supports in one ply (at end-lap)

P_{nfi} = Connection nominal strength at interior supports in one ply

$$S_{ni} = [2A(\lambda - 1)P_{nfe} + n_s P_{ns} + 2n_p \alpha_p^2 P_{nfi} + 4\alpha_e^2 P_{nfe}] \frac{1}{L} \quad \text{Modified Eq. D1-1}$$

Rewriting so closer in form to Eq. D1-1:

$$S_{ni} = [2A(\lambda - 1) \frac{P_{nfe}}{P_{nfi}} + n_s \frac{P_{ns}}{P_{nfi}} + 2n_p \alpha_p^2 + 4\alpha_e^2 \frac{P_{nfe}}{P_{nfi}}] \frac{P_{nfi}}{L}, \text{ or}$$

$$S_{ni} = [2A(\lambda - 1) + n_s \frac{P_{ns}}{P_{nfe}} + 2n_p \alpha_p^2 \frac{P_{nfi}}{P_{nfe}} + 4\alpha_e^2] \frac{P_{nfe}}{L}$$

See Example 1a above for parameters: A , n_s , α_e^2 , α_p^2 , and λ

$P_{nfe} = 2.50$ k Controlling strength at weld shear plane at *exterior supports*

Note: For the purpose of illustration, $P_{nfe} = 2.50$ k is used in this calculation.

$P_{nfi} = 2.89$ k Controlling strength at weld shear plane at *interior supports*

$$\beta = n_s \alpha_s + 2n_p \alpha_p^2 \frac{P_{nfi}}{P_{nfe}} + 4\alpha_e^2 \quad \text{Adjustment based on strength at exterior support, } P_{nfe}, \text{ allows } S_{ni} \text{ and } S_{nc} \text{ to be based on } P_{nfe}.$$

$$\alpha_s = \frac{P_{ns}}{P_{nfe}} = \frac{0.650 \text{ k}}{2.50 \text{ k}} = 0.260$$

$$\beta = 9(0.26) + 2(2)(0.722) \frac{2.89 \text{ k}}{2.50 \text{ k}} + 4(0.722) = 8.57$$

$$S_{ni} = [2(1.0)(0.806 - 1) + 8.57] \frac{2.50 \text{ k}}{18 \text{ ft}} = 1.14 \text{ klf} \quad \text{Eq. D1-1}$$

$$S_{nc} = \left(\frac{(1.33 \frac{1}{\text{ft}})^2 (8.57)^2}{(18 \text{ ft})^2 (1.33 \frac{1}{\text{ft}})^2 + (8.57)^2} \right)^{0.5} 2.50 \text{ kips} = 1.12 \text{ klf} \quad \text{Eq. D1-2}$$

Result: $S_{nf} = \min(S_{ni}, S_{nc})$

$$= \min(1.14, 1.12) = 1.12 \text{ klf} \quad (6\% \text{ reduction vs. } 1.19 \text{ klf at butt-joint})$$

Note: The difference will be reduced for multi-spans greater than 3 span where weighted contribution of interior supports increases. It is rational to use a simplification that is common practice, $P_{nf} = 2.89$ k and $S_{nf} = 1.19$ klf. Thickness, 0.036 in., is at the upper limit of a four-thickness end-lap.

Diaphragm strength with end-lap is controlled by *exterior support single-thickness connections*, but is still relatively balanced — 1.14 klf vs. 1.12 klf.

Calculate Nominal Diaphragm Shear Strength per Unit Length Controlled by Panel Buckling, S_{nb} , Using Section D2.1

$$S_{nb} = \frac{7890}{\alpha L_v^2} \left(\frac{I_{xg}^3 t^3 d}{s} \right)^{0.25} \quad \text{Eq. D2.1-1}$$

Note: See Deck Data for parameters. Required units are defined in Section D2.1.
Coefficient, 7890, includes necessary adjustments – See *Commentary* Section D2.1.

$$\begin{aligned} \alpha &= 1 && \text{Conversion factor for U.S. Customary Units} \\ s &= 2(e + w) + f = 2e + 2w + f && \text{Eq. D2.1-2} \\ &= 1.56 \text{ in.} + 2(1.53 \text{ in.}) + 3.56 \text{ in.} = 8.18 \text{ in.} \end{aligned}$$

$$S_{nb} = \frac{7890}{(1)(6 \text{ ft})^2} \left(\frac{(0.210 \text{ in.} / \text{ft})^3 (0.036 \text{ in.})^3 (6.0 \text{ in.})}{8.18 \text{ in.}} \right)^{0.25} = 5.20 \text{ klf} \quad \text{Eq. D2.1-1}$$

$$S_{nf} = 1.19 \text{ klf} < S_{nb} = 5.20 \text{ klf}$$

Result: $S_{nb} = 5.20 \text{ klf}$

Available Strength Results

Safety and resistance factors are in Section B1, Table B1.1:

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

$$\begin{aligned} \Omega_{df} > \Omega_{db}, S_{nf} < S_{nb}, \quad \text{and} \quad \frac{S_{nf}}{\Omega_{df}} < \frac{S_{nb}}{\Omega_{db}} \\ \phi S_n &= \min(\phi_{df} S_{nf}, \phi_{db} S_{nb}) \quad \text{for LRFD and LSD} \quad \text{Eq. D-2} \\ \phi_{df} < \phi_{db}, S_{nf} < S_{nb}, \quad \text{and} \quad \phi_{df} S_{nf} < \phi_{db} S_{nb} \end{aligned}$$

Note: For *LRFD* and *LSD*, the product of two smaller parameters is smaller. For *ASD*, the smaller number divided by the larger number is smaller. In this case with welds, S_{nb} will not control S_n for all *load combinations*.

Example 1a Result (No Uplift)

Nominal Strength: $S_n = 1.19 \text{ klf}$ Applies at butt-joint or end-lap installations

Nominal strength per unit length is controlled by the connection strength at one thickness, and both nominal strength limits (S_{ni} , S_{nc}) are balanced.

Safety Factors and Resistance Factors are in Section B1 for different loading events.

$$\begin{aligned} \text{Available Strength: } & \frac{S_n}{\Omega_d} \text{ for ASD} \\ & \phi_d S_n \text{ for LRFD and LSD} \end{aligned}$$

Example 1b: Edge Detail Parallel to the Edge Panel Span

Objective

Using Section D1, determine a required edge detail parallel to the *edge panel* span so S_{ne} exceeds S_n from Example 1a (No Uplift) - 1.19 klf.

Note: Interior *lateral force-resisting systems* might require a reaction $> S_n$.

Edge Detail Configuration

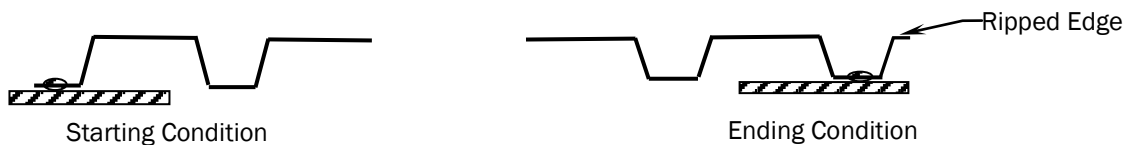
Consider two cases:

- (1) Starting condition allows standard installation of full width sheet, $w_e = 36$ in.
- (2) Ending condition requires a partial width sheet, $w_e = 24$ in.

Both cases allow a bottom flat on edge support.

Try to match *side-lap* spacing = 18 in. o.c. so $n_e = n_s$.

The same *support connection* type as Example 1a (3/4 in. arc spot weld) will be used along *edge panel* parallel with deck to determine S_{ne} .



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

$$P_{nf} = P_{nfs} = 2.89 \text{ k} \quad \text{(From Example 1a)}$$

Case 1: Full Width Sheet at Starting Condition

Based on Example 1a, $S_n = 1.19$ k. The starting connection should be designed so that $S_{ne} \geq S_n$.

Try the same 36/5 pattern as at interior *panels* in Example 1a.

$$\alpha_1 = \alpha_2 = \frac{\sum x_{ee}}{w_e} \quad \text{Eqs. D1-10 and D1-11}$$

$$x_{ee1} = 0 \text{ in.} \quad x_{ee2} = 12 \text{ in.} \quad x_{ee3} = 18 \text{ in.} \quad \text{Figure D1-1}$$

$$\alpha_1 = \alpha_2 = \left(\frac{1}{36 \text{ in.}} \right) (0 + 2(12 \text{ in.}) + 2(18 \text{ in.})) = 1.67$$

From Example 1a, $n_p = 2$, $n_s = 9$ and $L = 18$ ft. Try $n_e = n_s$

$$S_{ne} = \frac{(2(1.67) + 2(1.67))2.89 \text{ k} + 9(2.89 \text{ k})}{18 \text{ ft}} = 2.52 \text{ klf} \quad \text{Eq. D1-3}$$

$$S_{ne} = 2.52 \text{ klf} > S_n = 1.19 \text{ klf} \text{ OK} \quad S_{ne} \text{ will not control } S_n$$

If simplification is used, $\alpha_1 = \alpha_2 = 1.0$ (only considers welds along the edge):

$$\alpha_1 = \alpha_2 = 1.00$$

$$S_{ne} = \frac{(2(1.0) + 2(1.0))2.89 \text{ k} + 9(2.89 \text{ k})}{18 \text{ ft}} = 2.09 \text{ klf} \quad \text{Eq. D1-3}$$

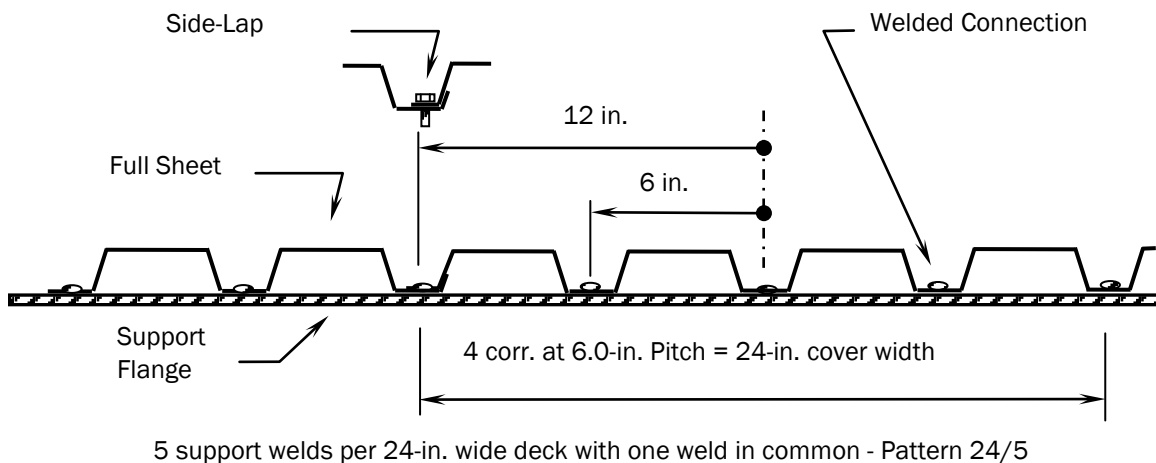
Note: End-lap weld acts as two welds along the edge – each ply develops its bearing strength, 2.89 k, and kernel strength $7.03 \text{ k} > 2(2.89 \text{ k})$. This justifies the coefficient, 2, at α_1 . See Comments at Example 1a for two thicknesses - $2.79 \text{ k} \approx 2.89 \text{ k}$. Simplification (2.09 klf) is 83% of the more precise value (2.52 klf).

$$S_{ne} = 2.09 \text{ klf} > S_n = 1.19 \text{ klf} \quad \text{OK} \quad S_{ne} \text{ will not control } S_n$$

Case 1 Result: Use 18 in. o.c. spacing at *edge panel over edge support*.

Case 2: Partial Width Sheet at Ending Condition

Considering that S_n cannot be reduced, try 24/5 pattern or 3/4 ϕ arc spot weld at 6 in. o. c. over supports, and #10 screw *side-lap connections* at 12 in. o.c. at *side-lap* between partial and last full-width sheet.



$$x_{ee1} = 0 \text{ in.} \quad x_{ee2} = x_{ee4} = 6 \text{ in.} \quad x_{ee3} = x_{ee5} = 12 \text{ in.}$$

Figure D1-1

$$\alpha_p^2 = \alpha_e^2 = \left(\frac{1}{(24 \text{ in.})^2} \right) (0^2 + 2(6 \text{ in.})^2 + 2(12 \text{ in.})^2) = 0.625 \quad \text{Eq. D1-8}$$

$$n_s = \left(\frac{(6 \text{ ft}) \left(\frac{12 \text{ in.}}{\text{ft}} \right)}{12 \frac{\text{in.}}{\text{conn.}}} - 1 \right) \frac{18 \text{ ft}}{6 \text{ ft}} = 15$$

$$N = \frac{4 \text{ welds}}{2 \text{ ft}} = 2.0 \frac{\text{welds}}{\text{ft}}$$

n_p , L , A , α_s and λ are from Example 1a.

$$\beta = 15 \times 0.225 + 2 \times 2 \times 0.625 + 4 \times 0.625 = 8.38 \quad \text{Eq. D1-5}$$

Note: N and β are greater than those at full-width sheet (36 in.).

$$S_{ni} = [2(1.0)(0.806 - 1) + 8.38] \frac{2.89 \text{ k}}{18 \text{ ft}} = 1.28 \text{ klf} \quad \text{Eq. D1-1}$$

$$S_{nc} = \left(\frac{(2 \frac{1}{\text{ft}})^2 (8.38)^2}{(18 \text{ ft})^2 (2 \frac{1}{\text{ft}})^2 + (8.38)^2} \right)^{0.5} 2.89 \text{ kips} = 1.31 \text{ klf} \quad \text{Eq. D1-2}$$

$$S_{nf} = \min(S_{ni}, S_{nc}) \\ = \min(1.28, 1.31) = 1.28 \text{ klf} > S_n = 1.19 \text{ klf} \quad \text{OK at partial-width panel}$$

Note: Simplified S_{ne} (2.09 klf) at full-width *panel* is used to verify partial width since that value is independent of cover width.

$$S_{ne} = 2.09 \text{ klf} > S_n = 1.19 \text{ klf} \quad \text{OK at ending condition} \quad \text{Eq. D1-3}$$

Example 1b Edge Detail Result:

Connection Schedule:

Support connections at panels — 3/4 in. arc spot weld, $F_{xx} = 70 \text{ ksi}$

Starting Condition: Pattern = 36/5 — See Figure at Example 1a

Ending Condition: Pattern = 24/5 — See Figure at Example 1b

Support connections at edge panel over lateral force-resisting system

Starting Condition: 3/4 in. arc spot weld at 18 in. o.c. at and between supports

Ending Condition: 3/4 in. arc spot weld at 18 in. o.c. at and between supports

Side-lap connections at first and last panel — #10 screw

Starting Condition: Spacing at 18 in. o.c. between supports

Ending Condition: Spacing at 12 in. o.c. between supports

Note: Example 1b illustrates the need to require extra connections at partial width *panels* and the rationality of the rule of thumb to let $n_e \geq n_s$.

Example 1c: Diaphragm Stiffness for Example 1a Configuration

Objective

Use the *configuration* of Example 1a to calculate the *diaphragm stiffness*, G' .

Note: Sections D5.1.1 and D5.2 will be used. Appendix 1.4 will be used for D_n and then Appendix 1.5 will be used as a check. G' is based on *interior panels*.

$$G' = \left(\frac{Et}{2(1+\mu)\frac{s}{d} + \gamma_c D_n + C} \right) K \quad \text{Eq. D5.1.1-1}$$

$K = 1.0$ for lap down with *connections* into steel supports

$$C = \left(\frac{Et}{w} \right) \left(\frac{2L}{2\alpha_3 + n_p \alpha_4 + 2n_s \frac{S_f}{S_s}} \right) S_f \quad \text{Eq. D5.1.1-2}$$

$$x_{e1} = 0 \text{ in.} \quad x_{e2} = 12 \text{ in.} \quad x_{e3} = 18 \text{ in.} \quad \text{Figure D1-1}$$

$$\alpha_3 = \alpha_4 = \frac{\sum x_e}{w} = \left(\frac{1}{36 \text{ in.}} \right) (0 + 2(12 \text{ in.}) + 2(18 \text{ in.})) = 1.67 \quad \text{Eq. D5.1.1-3}$$

$$S_f = \frac{1.15\alpha}{1000\sqrt{t}} = \frac{1.15(1)}{1000\sqrt{.036 \text{ in.}}} = 0.0061 \frac{\text{in.}}{\text{kip}} \quad \text{Eq. D5.2.1.1-1}$$

$$S_s = \frac{3.0\alpha}{1000\sqrt{t}} = \frac{3.0(1)}{1000\sqrt{.036 \text{ in.}}} = 0.0158 \frac{\text{in.}}{\text{kip}} \quad \text{Eq. D5.2.2-2}$$

E, t, w, L, n_s from Deck Data and Example 1a at calculation of S_{nf}

$$C = \left(\frac{29500 \frac{\text{k}}{\text{in.}^2} (0.036 \text{ in.})}{36 \text{ in.}} \right) \left(\frac{2(18 \text{ ft})(12 \frac{\text{in.}}{\text{ft}})}{2(1.67) + 2(1.67) + 2(9) \left(\frac{0.0061}{0.0158} \right)} \right) 0.0061 \frac{\text{in.}}{\text{kip}} = 5.70$$

$s = 8.18 \text{ in.}$ From Example 1a at calculation of S_{nb}

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

$d = 6 \text{ in.}$ From Deck Data

$\gamma_c = 0.9$

Table 1.3-1

$$D_n = \frac{D}{L} \quad \text{Eq. 1.4-1}$$

Note: The value, D_n , can be determined using the detailed method illustrated in Example 2d and based on Appendix 1. However, the WR results can be obtained quickly from *Commentary* Table C-1.2, which is based on $t = 0.0358 \text{ in.}$ This is close enough to $t = 0.036 \text{ in.}$ and can be adjusted to refine the tabulated D .

$$D = \frac{U_1 D_1 + U_2 D_2 + U_3 D_3 + U_4 D_4}{U_1 + U_2 + U_3 + U_4} \quad \text{Eq. 1.4-2}$$

Example 1a *configuration* only requires D_1 and D_2 .

$$D_1 = 925 \text{ in.} \quad \text{From Commentary Table C-1.2}$$

$$D_2 = 7726 \text{ in.}$$

$$U_1 = 2 \quad U_2 = 4 \quad \text{See Figure in Example 1a}$$

$$D = \frac{2(925 \text{ in.}) + 4(7726 \text{ in.})}{2 + 4} = 5460 \text{ in.} \quad \text{Eq. 1.4-2}$$

Note: To refine D to $t = 0.036 \text{ in.}$, multiply D by $\left(\frac{0.0358}{0.036}\right)^{1.5} = 0.992$ Eqs. 1.4-3 and 1.4-4

$$D = 0.992(5460) = 5420$$

$$D_n = \frac{5420 \text{ in.}}{(18 \text{ ft})\left(12 \frac{\text{in.}}{\text{ft}}\right)} = 25.1 \quad \text{Eq. 1.4-1}$$

$$G' = \left(\frac{\left(29500 \frac{\text{k}}{\text{in.}^2}\right)(0.036 \text{ in.})}{2(1+0.3) \frac{8.18 \text{ in.}}{6.0 \text{ in.}} + 0.9(25.1) + 5.70} \right) (1) = 33.4 \frac{\text{k}}{\text{in.}} \quad \text{Eq. D5.1.1-1}$$

Discussion: Calculate G' using the rigorous method of Example 2d to calculate D. D_d , e , w , f are from Deck Data in Example 1a:

Eq. 1.4-8:

$$\delta_{11} = 9.90 \text{ in.}^3$$

Eq. 1.4-9:

$$\delta_{12} = 4.95 \text{ in.}^3$$

Eq. 1.4-10:

$$\delta_{22} = 2.87 \text{ in.}^3$$

Eq. 1.4-11:

$$\kappa_{t1} = 2.56 \frac{1}{\text{in.}^3}$$

Eq. 1.4-22:

$$\delta_{t2} = 59.7 \text{ in.}^{2.5}$$

Eq. 1.4-25:

$$\delta_{b2} = 129 \text{ in.}^{2.5}$$

$$\gamma_1 = \delta_{t1} = \frac{24 f}{\kappa_{t1}} \left[\frac{\kappa_{t1}}{4f^2 (f + w)} \right]^{0.25} \quad \text{Eq. 1.4-31 and Eq. 1.4-21}$$

$$= \frac{24(3.56 \text{ in.})}{2.56 \frac{1}{\text{in.}^3}} \left[\frac{2.56 \frac{1}{\text{in.}^3}}{4(3.56 \text{ in.})^2 (3.56 \text{ in.} + 1.53 \text{ in.})} \right]^{0.25} = 10.5 \text{ in.}^{2.5}$$

$$\gamma_2 = 2 \delta_{t2} + \frac{2e}{f} \delta_{b2} \quad \text{Eq. 1.4-32}$$

$$= 2(59.7 \text{ in.}^{2.5}) + \frac{1.56 \text{ in.}}{3.56 \text{ in.}} (129 \text{ in.}^{2.5}) = 176 \text{ in.}^{2.5}$$

$$D_1 = \frac{\gamma_1 f}{d(t)^{1.5}} = \frac{(10.5 \text{ in.}^{2.5})(3.56 \text{ in.})}{(6 \text{ in.})(0.036 \text{ in.})^{1.5}} = 912 \text{ in.} \quad U_1 = 2 \quad \text{Eq. 1.4-3}$$

$$D_2 = \frac{\gamma_2 f}{2d(t)^{1.5}} = \frac{(176 \text{ in.}^{2.5})(3.56 \text{ in.})}{2(6 \text{ in.})(0.036 \text{ in.})^{1.5}} = 7640 \text{ in.} \quad U_2 = 4 \quad \text{Eq. 1.4-4}$$

$$D = \frac{2(912 \text{ in.}) + 4(7640 \text{ in.})}{2 + 4} = 5400 \text{ in.} \quad \text{Eq. 1.4-2}$$

$$D_n = \frac{5400 \text{ in.}}{(18 \text{ ft}) \left(12 \frac{\text{in.}}{\text{ft}} \right)} = 25.0 \quad \text{Eq. 1.4-1}$$

$$G' = \left(\frac{\left(29500 \frac{\text{k}}{\text{in.}^2} \right) 0.036 \text{ in.}}{2(1 + 0.3) \frac{8.18 \text{ in.}}{6.0 \text{ in.}} + 0.9(25.0) + 5.70} \right) 1 = 33.5 \frac{\text{k}}{\text{in.}} \quad \text{Eq. D5.1.1-1}$$

Note: The above result is the same as the one obtained based on the data from *Commentary* Table C-1.2 (Round-off error due to three significant figures.)

Check using the approximation of Appendix Section 1.5:

Note: Input units are defined in Section 1.5, $\alpha = 1$ for U.S. Customary Units

$$D_n = \frac{1}{n} \sum_{i=1}^n D_{ni} \quad \text{Eq. 1.5-1}$$

$$D_{ni} = \frac{D_d f^2}{25 \alpha L} \left[\frac{1}{t} \right]^{1.5} \quad \text{For } \psi = 1 \quad \text{Eq. 1.5-2}$$

$$D_{ni} = \frac{0.94 d \psi^2}{f} \left[\frac{D_d f^2}{25 \alpha L} \right] \left[\frac{1}{t} \right]^{1.5} \quad \text{For } 1 < \psi \leq 3 \quad \text{Eq. 1.5-3}$$

$$D_{n1} = \frac{(1.47 \text{ in.})(3.56 \text{ in.})^2}{25(1)(18 \text{ ft})} \left[\frac{1}{0.036 \text{ in.}} \right]^{1.5} = 6.06 \quad \text{Eq. 1.5-2}$$

$$D_{n2} = \frac{0.94(6 \text{ in.})(2)^2}{3.56 \text{ in.}} \left[\frac{(1.47 \text{ in.})(3.56 \text{ in.})^2}{25(1)(18 \text{ ft})} \right] \left[\frac{1}{0.036 \text{ in.}} \right]^{1.5} = 38.4 \quad \text{Eq. 1.5-3}$$

Number of corrugations with:

- a) $\psi = 1$ is 2, and (Same as U_1)
- b) $\psi = 2$ is 4 (Same as U_2)

$$D_n = \frac{1}{6} (2(6.06) + 4(38.4)) = 27.6 \quad (10\% \text{ greater than } 25.0 \text{ in rigorous calculation})$$

Note: Compare the rigorous calculation with the one using approximate D_{ni}

From calculated D_1 above:

$$D_{n1} = \frac{912}{12 \times 18} = 4.22$$

$$\text{Ratio} = \frac{6.06}{4.22} = 1.44$$

From calculated D_2 above:

$$D_{n2} = \frac{7640}{12 \times 18} = 35.4$$

$$\text{Ratio} = \frac{38.4}{35.4} = 1.08$$

Approximation result:

$$G' = \left(\frac{29500 \frac{\text{k}}{\text{in.}^2} (0.036 \text{ in.})}{2(1+0.3) \frac{8.18 \text{ in.}}{6.0 \text{ in.}} + 0.9(27.6) + 5.70} \right) 1 = 31.2 \frac{\text{k}}{\text{in.}} \quad \text{Eq. D5.1.1-1}$$

Note: G' approximation is within 7% of $33.5 \frac{\text{k}}{\text{in.}}$ from rigorous calculation.

D_n approximation works relatively well and is best when $\psi > 1$ dominates. Variance of G' due to D_n approximation is mitigated by the other two contributing factors in the denominator: G' is 7% vs. D_n is 10% in this example.

Example 1c Result

$$G' = 33.5 \frac{\text{k}}{\text{in.}}$$

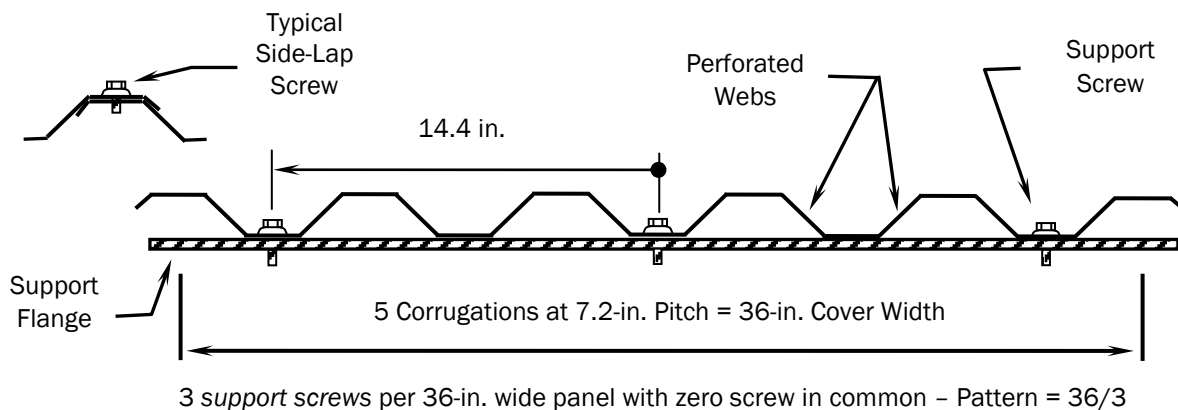
Example 2a: Nominal and Available Diaphragm Shear Strength of a Perforated Panel Connected to a Thin Support in the Absence of Uplift

Objective

Calculate the *nominal diaphragm shear strength* per unit length, S_n , and the *available strength* of a perforated panel in the absence of uplift using Chapter D. Consider the impact of a relatively thin support.

Note: *Acoustic panels* with perforated elements are addressed in Sections D1.4, D2.1, and D5.1.2.

Diaphragm Configuration



Note: Application is an exposed wall where weathertightness is not required.

Panel Data

See Figure D2.1-1 for definitions of *panel* parameters, and see *Commentary* Figure C-1.6-1 for W_p .

Yield stress, F_y	= 50 ksi	Modulus of Elasticity, E	= 29500 ksi
Tensile strength, F_u	= 65 ksi	Panel length, L	= 25.0 ft
Depth, D_d	= 1.50 in.	Cover width, w	= 36.0 in.
Thickness, t	= 0.024 in.	Pitch, d	= 7.20 in.
Top flat width, f	= 2.00 in.	Web flat width, w	= 2.19 in.
Bottom flat width, $2e$	= 2.00 in.		
Moment of Inertia, I_{xg}	= 0.123 in. ⁴ /ft	This is the I_x value from manufacturer of perforated panel and can be conservatively used for I_{xg} .	

Panel is end-lapped (strength of a single run of *panels* or a butt-joint condition will also be determined).

Note: Chapter D limits (a) through (d) are satisfied, and the *panel* strength can be calculated per Section D1. Material is based on ASTM 653 SS Grade 50.

Perforation Data

Perforations are located in webs only and have a band width, $W_p = 1.313$ in.

$d_p = 3/16$ in. diameter at spacing, $c_p = 3/8$ in. stagger.

Commentary Section 1.6

$$p_o = 0.9069 \frac{d_p^2}{c_p^2} = 0.9069 \frac{0.188^2}{0.375^2} = 0.228$$

Commentary Eq. C-1.6-1

Steel Support Data

Yield stress, $F_y = 50$ ksi

Tensile strength, F_u

= 65 ksi

Thickness, $t = 0.060$ in.

Spacing, L_v (shear span)

= 5.00 ft

Note: Thickness is in the lowest range of common installations.

Connection Schedule

Support connection: Pattern = 36/3 – See Figure above.

#12 screw with 1/2 in. round washer

$P_{nss} = 2.0$ k

$d = 0.216$ in.

$P_{nts} = 2.7$ k

Side-lap connection: Spacing = 20 in. o.c. (at and between supports)

#12 screw with 1/2 in. round washer

$P_{nss} = 2.0$ k

$d = 0.216$ in.

Note: Side-lap connections will be located over supports but are not into the supports.

P_{nss} = Breaking nominal shear strength of support and side-lap screws.

P_{nts} = Breaking nominal tensile strength of support screw.

Determine Available Strength per Eq. D-1 or D-2

Safety and resistance factors are in Section B1.

$$\frac{S_n}{\Omega} = \min \left(\frac{S_{nf}}{\Omega_{df}}, \frac{S_{nb}}{\Omega_{db}} \right) \quad \text{for ASD} \quad \phi S_n = \min (\phi_{df} S_{nf}, \phi_{db} S_{nb}) \quad \text{for LRFD and LSD}$$

$$S_{nf} = \min (S_{ni}, S_{nc}, S_{ne})$$

Note: S_{nf} and S_{ne} are determined in Section D1 and S_{nb} is in Section D2. S_{nb} is calculated in Example 2a. An edge detail is designed in Example 2c. Therefore, strength controlled by edge panel, S_{ne} , will not be considered in Example 2a. Uplift is in Example 2b (shear and tension interaction). Stiffness, G' , is calculated in Example 2d.

Use U.S. Customary Units.

Connections are not located in perforated zones, so testing is not required for P_{nf} or P_{ns} per Section D1.4.

Calculate Nominal Diaphragm Shear Strength per Unit Length Controlled by Panel Screws, S_{nf} , Using Section D1

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

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

Note: Eq. D1-1 for S_{ni} should be based on the *interior support* screw, and Eq. D1-2 for S_{nc} should be based on the *exterior support* corner screw. However, the same P_{nf} value will be used at all supports in determining S_{ni} and S_{nc} in this example.

Calculate Support Connection Strength, P_{nf} (Section D1.1.2 or AISI S100 J4.3.1)

Cases considered:

- (1) A single-panel thickness support connection, which applies at top ply of end-laps and at interior flutes over interior supports; and
- (2) A two-panel thickness support connection, which applies at end-laps.

Case 1: Single-panel thickness support connection:

$$\frac{t_2}{t_1} = \frac{0.060 \text{ in.}}{0.024 \text{ in.}} = 2.5 \quad \text{Therefore, } \frac{t_2}{t_1} \geq 2.5$$

$$P_{nf} = 2.7 t_1 d F_{u1} \quad \text{AISI S100 Eq. J4.3.1-4}$$

$$= 2.7(0.024 \text{ in.})(0.216 \text{ in.})\left(65 \frac{\text{k}}{\text{in.}^2}\right) = 0.910 \text{ k} < P_{nss} = 2.0 \text{ k}$$

$$P_{nf} = 2.7 t_2 d F_{u2} \quad \text{AISI S100 Eq. J4.3.1-5}$$

$$= 2.7(0.060 \text{ in.})(0.216 \text{ in.})\left(65 \frac{\text{k}}{\text{in.}^2}\right) = 2.27 \text{ k} > P_{nss} = 2.0 \text{ k}$$

$$P_{nf} = \min(0.910, 2.27, 2.0) = 0.910 \text{ k}$$

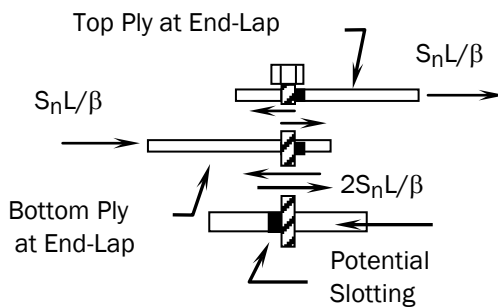
Case 1 result: $P_{nf} = 0.910 \text{ kips}$ **Applies at butt-joint and end-lap installations**

Note: Bearing of the panel against the screw at one thickness controls connection strength.

This “one-connection strength” will be used:

1. As required for other fasteners in Section D1.1.5, and
2. To calculate S_{ni} and S_{nc} in Section D1 per standard industry practice.

Case 2: Two-panel thickness support connection:



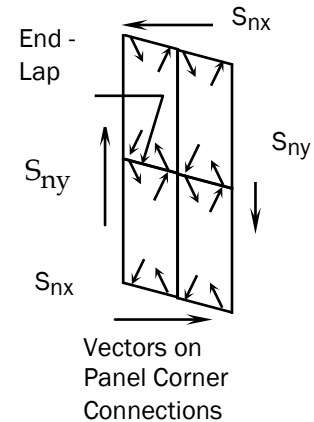
Justify using “one-connection strength” by considering shear flow through screw support connection at end-lap (also see Appendix 2 Figure 2.2-1):

When the shear transfer plane is between the bottom ply and the support (as shown in the figure at left), the shear flow at this shear transfer plane is doubled relative to the force in a single ply. Since the bearing strength of screw against

support and screw breaking strength are greater than twice the bearing strength against one panel thickness, i.e. 2.27 k and $2\text{ k} > 2(0.910\text{ k}) = 1.82\text{ k}$, the two-panel thickness support connection does not control the design. It is, therefore, permissible to use the one-thickness connection strength for design in each ply at the end-lap.

Connection strength at the end-lap top ply shear plane is the same as that over an interior support – both with one bearing thickness. Let’s investigate the panels at the end-lap. In this case, the support connections are through two plies and located in only two panels. There are four plies at the elevated side-lap over the support.

Force vectors in support connections are shown in the figure at right. At the corner of a four-panel connection, the vertical components of the two panels add to each other and are transferred to the support. The adjacent two panels’ vertical components are in the opposing direction and are also transferred to the support. The horizontal components in the two panels cancel and do not show up at the support. However, both components are present in each ply and cause bearing at one thickness. As stated above, the shank breaking strength and bearing against the support will not control the strength in this configuration. Therefore, it is permissible to use the one-thickness connection strength to determine S_{ni} , which considers the components parallel with the span, and the common screw develops the full available strength in each ply. The value in the end-lap top ply can be limited to $1/2$ the support bearing or screw breaking capacity when determining S_{ni} . In this configuration, S_{nc} is always controlled by the one-thickness connection strength and both components are present in each ply.



Case 2 result: Based on the above discussion, $P_{nf} = 0.910\text{ k}$ – Used to calculate S_{ni} and S_{nc} for butt-joint or end-lap.

Calculate side-lap connection strength, P_{ns} (Section D1.2.5 or AISI S100 J4.3.1):

$$\frac{t_2}{t_1} = \frac{0.024\text{ in.}}{0.024\text{ in.}} = 1.0 \quad \text{Therefore, } \frac{t_2}{t_1} \leq 1.0$$

$$P_{ns} = 4.2(t_2^3 d)^{1/2} F_{u2}$$

AISI S100 Eq. J4.3.1-1

$$= 4.2 \left((0.024 \text{ in.})^3 (0.216 \text{ in.}) \right)^{1/2} \left(65 \frac{\text{k}}{\text{in.}^2} \right) = 0.472 \text{ k}$$

$$P_{ns} = 2.7 t_1 d F_{u1} \quad \text{AISI S100 Eq. J4.3.1-2}$$

$$= 2.7 (0.024 \text{ in.}) (0.216 \text{ in.}) \left(65 \frac{\text{k}}{\text{in.}^2} \right) = 0.910 \text{ k}$$

$$P_{ns} = 2.7 t_2 d F_{u2} \text{ (same as above)} = 0.910 \text{ k} \quad \text{AISI S100 Eq. J4.3.1-3}$$

$$P_{nss} = 2.0 \text{ kips} \quad \text{From Connection Schedule}$$

$$P_{ns} = \min(0.472, 0.910, 0.910, 2.00) = 0.472 \text{ k}$$

Result: $P_{ns} = 0.472 \text{ kips}$ **Tilting of screw in panel controls**

Calculate configuration parameters required for S_{nf} :

$$A = 0.0 \quad \text{Number of support screws at side-lap at panel ends}$$

$$\lambda = 1 - \frac{D_d L_v}{240 \sqrt{t}} \geq 0.7 \quad \text{Input units are defined in Section D1.} \quad \text{Eq. D1-4a}$$

$$= 1 - \frac{(1.5 \text{ in.})(5.0 \text{ ft})}{240 \sqrt{0.024 \text{ in.}}} = 0.798 > 0.7 \quad \text{OK}$$

$$N = \frac{3 \text{ screws}}{3 \text{ ft}} = 1.0 \frac{\text{screw}}{\text{ft}} \quad \text{Number of screws into support per ft along panel ends}$$

No support screws are common at *panel side-laps*.

$$\beta = n_s \alpha_s + 2 n_p \alpha_p^2 + 4 \alpha_e^2 \quad \text{Factor defining screw interaction} \quad \text{Eq. D1-5}$$

$$n_p = \frac{L}{L_v} - 1 = \frac{25 \text{ ft}}{5 \text{ ft}} - 1 = 4.0 \quad \text{Number of interior supports along L} \quad \text{Modified Eq. D1-9}$$

$$n_s = \frac{25 \text{ ft} (12 \text{ in.} / \text{ft})}{20 \frac{\text{in.}}{\text{conn.}}} + 1.0 = 16 \quad \text{Number of side-lap screws along the panel length, L}$$

$$\alpha_s = \frac{P_{ns}}{P_{nf}} = \frac{0.472 \text{ k}}{0.910 \text{ k}} = 0.519 \quad \text{Connection strength ratio} \quad \text{Eq. D1-6}$$

$$\alpha_p^2 = \alpha_e^2 = \left(\frac{1}{w^2} \right) \sum x_e^2 \quad \text{See Figure D1-1} \quad \text{Eq. D1-8}$$

$$x_{e1} = 0.0 \quad x_{e2} = x_{e3} = 2(7.2 \text{ in.}) = 14.4 \text{ in.}$$

$$\alpha_p^2 = \alpha_e^2 = \left(\frac{1}{(36 \text{ in.})^2} \right) \left((0 \text{ in.})^2 + 2(14.4 \text{ in.})^2 \right) = 0.32$$

Calculate nominal diaphragm shear strength, S_{nf} :

$$\beta = 16(0.519) + 2(4)(0.32) + 4(0.32) = 12.1 \quad \text{Eq. D1-5}$$

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

$$= [2(0.0)(0.798 - 1) + 12.1] \frac{0.910 \text{ kips}}{25 \text{ ft}} = 0.440 \text{ klf}$$

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

$$= \left(\frac{(1)^2 (12.1)^2 \frac{1}{\text{ft}^2}}{(25)^2 (1)^2 \frac{\text{ft}^2}{\text{ft}^2} + (12.1)^2} \right)^{0.5} 0.910 \text{ kips} = 0.396 \text{ klf}$$

$$S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.440, 0.396) = 0.396 \text{ klf} \quad S_{ne} \text{ is considered in Example 2c}$$

Comment: S_{ni} and S_{nc} values often are close.

Result: $S_{nf} = 0.396 \text{ klf}$ Based on least support screw strength, $P_{nf} = 0.910 \text{ k}$.
 S_{nf} applies at butt-joint or end-lap installations.

Calculate Nominal Diaphragm Shear Strength per Unit Length Controlled by Panel Buckling, S_{nb} , Using Section D2.1 and Appendix 1 Section 1.6

$$S_{nb} = \frac{7890}{\alpha L_v^2} \left(\frac{I_{xg}^3 t^3 d}{s} \right)^{0.25} \quad \text{Eq. D2.1-1}$$

Note: See Panel and Perforation Data for parameters. Required units are defined in Section D2.1. Coefficient, 7890, includes necessary adjustments – See *Commentary* Section D2.1. Since perforations are only in web, only $w = w_p$ needs to be inserted at Eq. D2.1-2 to determine s .

$\alpha = 1$ Conversion factor for U.S. Customary Units

$$s = 2(e + w_p) + f = 2e + 2w_p + f \quad \text{Eq. D2.1-2}$$

$$w_p = K_{E_w}^{1/3} w \quad \text{Eq. 1.6-3}$$

$$K_{E_w} = 1 + A_w^3 \left(\frac{1}{k} - 1 \right) \quad \text{Eq. 1.6-4}$$

$$A_w = \frac{W_p}{w} = \frac{1.313 \text{ in.}}{2.19 \text{ in.}} = 0.600 \quad \text{Eq. C-1.6-3}$$

$$k = 0.9 + p_o^2 - 1.875 p_o \quad (\text{for } 0.2 \leq p_o \leq 0.58) \quad \text{Eq. 1.6-5}$$

$$= 0.9 + (0.228)^2 - 1.875(0.228) = 0.524 \quad \text{See Panel Data for } p_o$$

$$K_{E_w} = 1 + (0.600)^3 \left(\frac{1}{0.524} - 1 \right) = 1.196 \quad \text{Eq. 1.6-4}$$

$$w_p = (1.196)^{1/3} (2.19 \text{ in.}) = 2.32 \text{ in.} \quad \text{Eq. 1.6-3}$$

$$s = 2.0 \text{ in.} + 2(2.32 \text{ in.}) + 2.0 \text{ in.} = 8.64 \text{ in.} \quad \text{Eq. D2.1-2}$$

$$S_{nb} = \frac{7890}{(1)(5 \text{ ft})^2} \left(\frac{(0.123 \text{ in.}^4/\text{ft})^3 (0.024 \text{ in.})^3 (7.2 \text{ in.})}{8.64 \text{ in.}} \right)^{0.25} = 3.82 \text{ klf} \quad \text{Eq. D2.1-1}$$

Result: $S_{nb} = 3.82 \text{ klf}$

$$S_{nf} = 0.396 \text{ klf} < S_{nb} = 3.82 \text{ klf}$$

Available Strength

Safety and resistance factors are in Section B1, Table B1.1:

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

$$\text{Since } \Omega_{df} > \Omega_{db}, S_{nf} < S_{nb}, \quad \frac{S_{nf}}{\Omega_{df}} < \frac{S_{nb}}{\Omega_{db}}$$

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

$$\text{Since } \phi_{df} < \phi_{db}, S_{nf} < S_{nb}, \quad \phi_{df} S_{nf} < \phi_{db} S_{nb}$$

Discussion:

Wind load is the probable load that allows S_{nb} to control *available strength*. The *available strength* based on S_{nf} is greatest for that event.

ASD:

$$\Omega_{df} = 2.00 \text{ and } \Omega_{db} = 2.00$$

$$\frac{S_n}{\Omega} = \min \left(\frac{0.396}{2.00}, \frac{3.82}{2.00} \right) = 0.198 \text{ klf}$$

LRFD:

$$\phi_{df} = 0.80 \text{ and } \phi_{db} = 0.80$$

$$\phi S_n = \min(0.80(0.396), 0.80(3.82)) = 0.317 \text{ klf}$$

LSD:

$$\phi_{df} = 0.75 \text{ and } \phi_{db} = 0.75$$

$$\phi S_n = \min(0.75(0.396), 0.75(3.82)) = 0.297 \text{ klf}$$

Result: S_{nf} will control both S_n and *available strength* for all load combinations in this diaphragm configuration. S_{nb} does not control in this example.

Example 2a Result (No Uplift)

Nominal Strength: $S_n = 0.396 \text{ klf}$ Applies at butt-joint or end-lap installations.

Nominal strength per unit length is controlled by the corner *connections* in the panels.

$$\text{Available Strength: } \frac{S_n}{\Omega_d} \quad \text{for ASD}$$

$\phi_d S_n$ for *LRFD* and *LSD*

Safety Factors and *Resistance Factors* are in Section B1, Table B1.1 for different loading events.

Example 2b: Nominal and Available Diaphragm Shear Strength of a Perforated Panel Connected to a Thin Support With Uplift

Objective

Use the *configuration* of Example 2a and calculate the *nominal* and *available diaphragm shear strength* per unit length in the presence of an uplift load. Consider the impact of a relatively thin support.

Note: Section D3.1.2.1 will be used to determine shear and tension interaction and the *nominal shear strength* per screw, P_{nt} . Section D1 will be used to determine S_{nf} based on P_{nt} . $S_n = \min(S_{nf}, S_{nb})$. From Example 2a, S_{nb} will not control S_n .

Load Data

Wind uplift zone = 30 psf (0.030 ksf)

$$\text{Reaction at interior support} = 1.1(5 \text{ ft})(0.030 \text{ ksf})$$
$$= 0.165 \text{ klf}$$

for pull-over and pull-out

$$\text{Reaction at exterior support} = 0.4(5 \text{ ft})(0.030 \text{ ksf})$$
$$= 0.060 \text{ klf}$$

for one-thickness pull-over

$$= 2(0.060 \text{ klf})$$
$$= 0.120 \text{ klf}$$

for pull-out at end-lap

Note: Two plies exist at end-lap, so tributary length doubles for screw pull-out. Neglect dead load of *panel* in *load combination* (would reduce force on the screw). Quick design could avoid refinements and conservatively use 0.165 klf as the universal connection load – See Example 2b; discussion below. Coefficients 1.1 and 0.4 are rational for multiple-span applications subject to uniform loads. More precise five-span values are available, and some engineers use coefficients 1.0 and 0.5.

AISI S100 Section J4.5.2 does not require a *strength* reduction for eccentric loading in pull-out. Even if required, opposing forces at an end-lap would partly offset this eccentricity (prying).

Design Method – ASD (Eq. C2-1)

Calculate Nominal Diaphragm Shear Strength per Unit Length Controlled by Panel Screws, S_{nf} , Using Section D1 ($P_{nf} = P_{nft}$)

Calculate P_{nft} (screw *nominal shear strength* in the presence of tension) and corresponding S_{nf} :

Note: S_{nf} is least value calculated at cases (a), (b), and (c) below using the corresponding P_{nft}

(a) Interaction of shear and pull-over (Section D3.1.2.1(a) or AISI S100 Section J4.5.1)

$$\left(\frac{P_{\text{nft}}}{\Omega_d P_{\text{nf}}}\right) + \left(\frac{0.71T}{P_{\text{nov}}}\right) = \frac{1.1}{\Omega} \quad \Omega = 2.35 \quad \Omega_d = 2.00 \quad \text{Eq. D3.1.2.1-2}$$