

DESIGN EXAMPLES FOR THE DESIGN OF PROFILED STEEL DIAPHRAGM PANELS BASED ON AISI S310  
— AISI D310-14

# Design Guide



American  
Iron and Steel  
Institute

## **AISI DESIGN GUIDE**

**Design Examples for the Design of**

**Profiled Steel Diaphragm Panels**

**Based on AISI S310-13**

2014 Edition

Approved by  
AISI Committee on Specifications  
Diaphragm Design Task Group



**Design Examples for the Design of  
Profiled Steel Diaphragm Panels  
Based on AISI S310-13**

2014 EDITION

Design Guide D310-14

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 author of this document, John Mattingly.

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 – November 2014

## Preface

This document is developed based on the 2013 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 S310-13**

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.....	24
Example 2c: Required Edge Detail Parallel to the Edge Panel Span With Panel Connected to a Thin Support in the Absence of Uplift .....	32
Example 2d: Diaphragm Stiffness Calculation .....	37
Example 3a: Nominal and Available Diaphragm Shear Strength and Stiffness of a Panel on Wood Supports in the Absence of Uplift .....	44
Example 3b: Nominal Diaphragm Shear Strength of the Example 3a Configuration Subjected to an Uplift Load .....	53
Example 3c: Nominal Diaphragm Shear Strength per Unit Length, $S_n$ , With Fiberglass Insulation .....	62
Example 3d: Edge Connections and Details Parallel to the Edge Panel Span.....	66
Example 4a: Multiple-Span Nominal Diaphragm Shear Strength Equivalent to That of a Single-Span Configuration With Large Side-Lap Spacing.....	75
Example 4b: Multiple-Span Nominal Diaphragm Shear Strength Equivalent to That of a Single-Span Configuration with Close Side-Lap Spacing.....	82
Example 5a: Available Diaphragm Shear Strength in the Absence of Uplift Where the Support Thickness Approaches the Panel Thickness .....	86
Example 5b: Stiffness of the Configuration in Example 5a.....	92

---

**This page is intentionally left blank.**



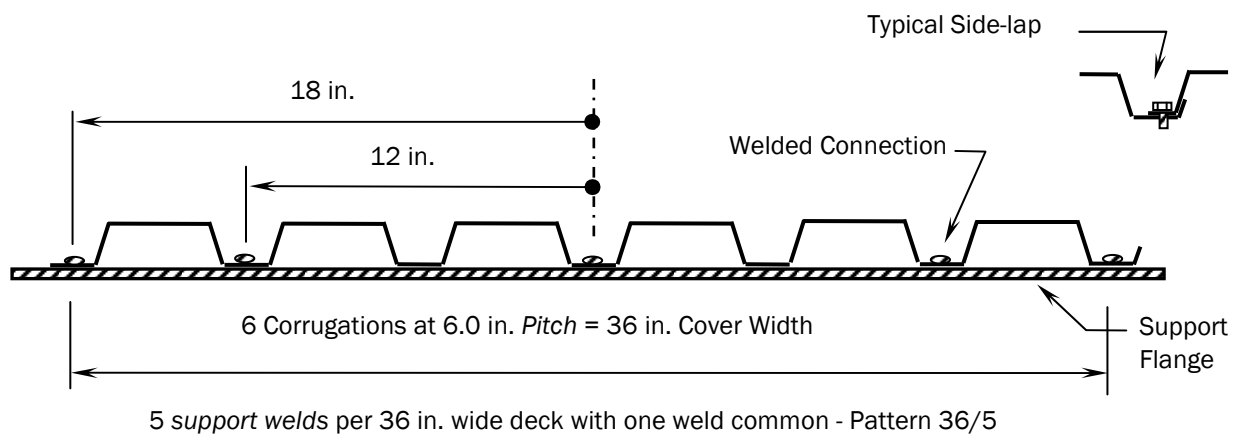
## Example 1a: Nominal Diaphragm Shear Strength of a Wide Rib Deck Welded to Supports in the Absence of Uplift

### 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 Third 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. <sup>4</sup> /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 A2 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 E2.3.1, E4.1 and E4.2 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 E2.2.2.1):**

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*

$$P_{nf} = \frac{\pi d_e^2}{4} 0.75 F_{xx} \quad \text{AISI S100 Eq. E2.2.2.1-1}$$

$$P_{nf} = 2.20 t d_a F_u \quad \text{For } \frac{d_a}{t} \leq 0.815 \sqrt{\frac{E}{F_u}} \quad \text{AISI S100 Eq. E2.2.2.1-2}$$

$$P_{nf} = 0.280 \left[ 1 + 5.59 \frac{\sqrt{E/F_u}}{d_a/t} \right] t d_a F_u \quad \text{For } 0.815 \sqrt{\frac{E}{F_u}} < \frac{d_a}{t} \leq 1.397 \sqrt{\frac{E}{F_u}} \quad \text{AISI S100 Eq. E2.2.2.1-3}$$

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

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

**Determine weld design parameters (AISI S100 Figure E2.2.2.1-1):**Single-thickness *connections*:  $d_a = (d - t) = (0.75 \text{ in.} - 0.036 \text{ in.}) = 0.714 \text{ in.}$ 

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

$$d_e = 0.7d - 1.5t \leq 0.55d \quad \text{AISI S100 Eq. E2.2.2.1-5}$$

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

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

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

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:**  $d_e \geq 3/8$  in. is required by AISI S100 Section E2.2. 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. E2.2.2.1-1 and E2.2.2.1-3:

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

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

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

Two-thickness *connections*: minimum of AISI S100 Eqs. E2.2.2.1-1 and E2.2.2.1-2:

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

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

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

**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. E2.2.2.1-1 and E2.2.2.1-2:

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

**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. E2.2.2.1-1 and E2.2.2.1-2:

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

**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 E4.3.1):**

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

$$P_{ns} = 4.2(t_2^3 d)^{1/2} F_{u2} \quad \text{AISI S100 Eq. E4.3.1-1}$$

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

$$P_{ns} = 2.7t_1 d F_{u1} \quad \text{AISI S100 Eq. E4.3.1-2}$$

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

$$P_{ns} = 2.7t_2 d F_{u2} \text{ (same)} \quad \text{AISI S100 Eq. E4.3.1-3}$$

$P_{nss} = 1.5 \text{ k}$  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  $S_{nf}$ :**

$A = 1.0$  Number of support welds at *side-lap* at deck ends

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

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

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

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

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

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

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

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

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

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

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

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

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

### 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.

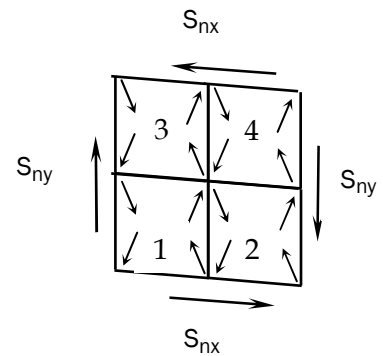
**Result:**  $S_{nf} = 1.19 \text{ klf}$  Based on least weld strength,  $P_{nf} = 2.89 \text{ k}$ .  
 $S_{nf}$  applies at butt-joint or end-lap installations

### Comment:

The justification of using "one connection strength" by considering shear flow through weld support connections at the end-lap:

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.



**Vectors On  
Deck Corner Welds**

Let's investigate the shear flow at end-laps in reference to 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 E2.2.2.1 uses a  $45^\circ$  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:

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

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

See Example 1a above for parameters:  $A$ ,  $n_s$ ,  $\alpha_e^2$ ,  $\alpha_p^2$ , and  $\lambda$

$P_{nfe} = 2.50$  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 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 (AISI S100 Table D5):

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

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

**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,  $S_{nb}$  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 (AISI S100 Table D5) 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

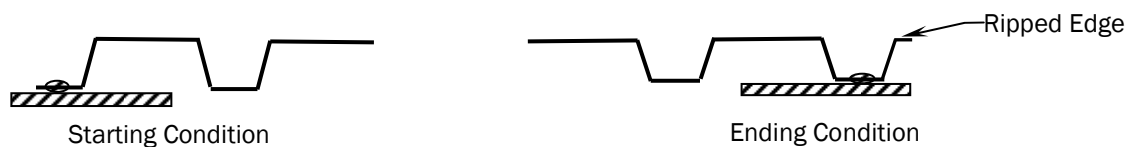
Consider two cases:

- (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  $S_{ne}$ .



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

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

### 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,  $n_p = 2$ ,  $n_e = 9$  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):

$$\alpha_1 = \alpha_2 = 1.00$$

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

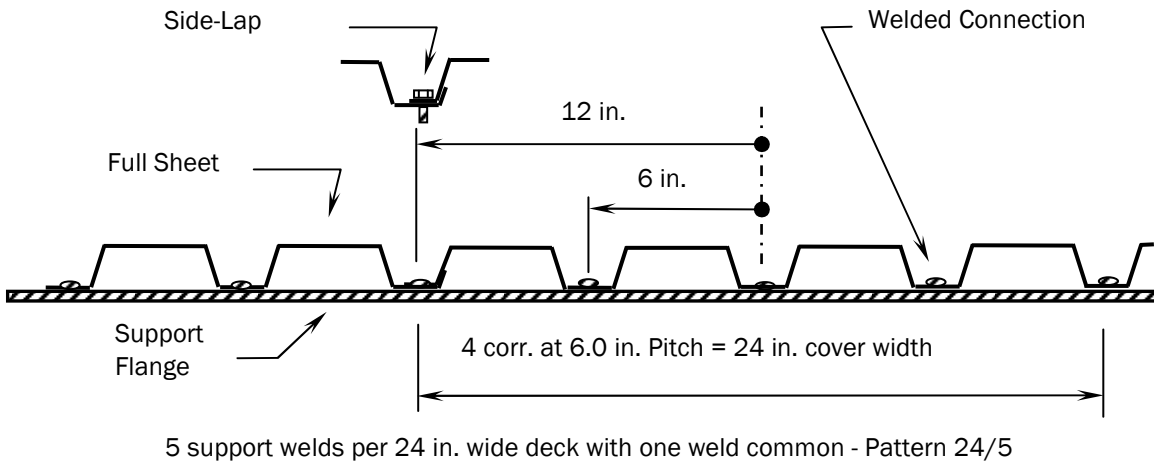
**Note:** End-lap weld acts as two welds along the edge – each ply develops its bearing strength, 2.89 k, and kernel strength  $7.03 \text{ k} > 2(2.89 \text{ k})$ . This justifies the coefficient, 2, at  $\alpha_1$ . See Comments at Example 1a for two thicknesses -  $2.79 \text{ k} \approx 2.89 \text{ k}$ . Simplification (2.09 klf) is 83% of the more precise value (2.52 klf).

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

**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  $\phi$  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

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

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

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

$n_p, L, A,$  and  $\lambda$  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  $\beta$  are greater than those at full-width sheet (36 in.).

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

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

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

**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

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

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

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

$$S_f = \frac{1.15\alpha}{1000\sqrt{t}} = \frac{1.15(1)}{1000\sqrt{0.036 \text{ in.}}} = 0.0061 \frac{\text{in.}}{\text{kip}} \quad \text{Eq. D5.2.1.1-1}$$

$$S_s = \frac{3.0\alpha}{1000\sqrt{t}} = \frac{3.0(1)}{1000\sqrt{0.036 \text{ in.}}} = 0.0158 \frac{\text{in.}}{\text{kip}} \quad \text{Eq. D5.2.2-2}$$

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

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

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

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

$d = 6$  in. From Deck Data

$\gamma_c = 0.9$

Table 1.3-1

$$D_n = \frac{D}{L}$$

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  $t = 0.0358$  in. This is close enough to  $t = 0.036$  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:

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

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

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

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

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

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

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

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

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

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

**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 1.5-5:

**Note:** Input units are defined in Section 1.5-5,  $\alpha = 1$  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**

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



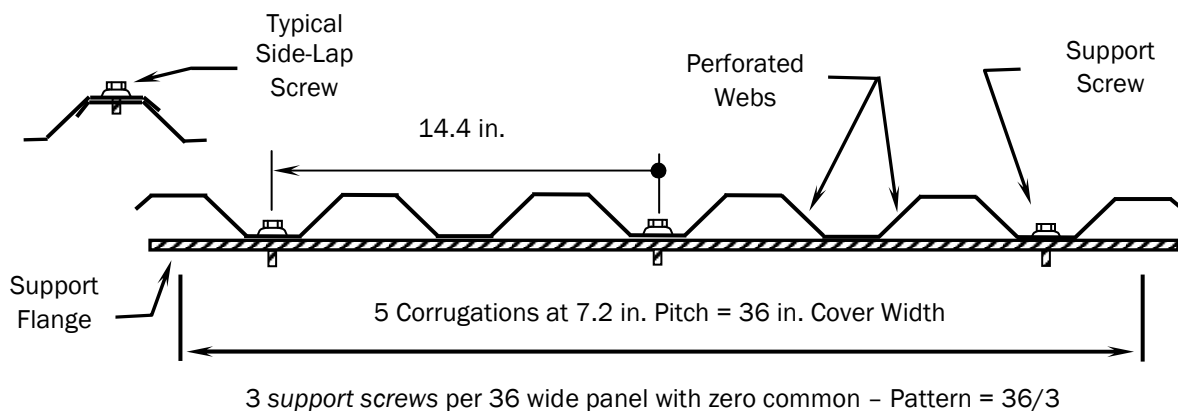
## Example 2a: Nominal and Available Diaphragm Shear Strength of a Perforated Panel Connected to a Thin Support in the Absence of Uplift

### Objective

Calculate the *nominal diaphragm shear strength* per unit length,  $S_{Nv}$ , 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 weather-tightness 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. <sup>4</sup> /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,  $S_{ni}$ , 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 E4.3.1)**

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:**

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

$$P_{nf} = 2.7t_1dF_{u1} \quad \text{AISI S100 Eq. E4.3.1-4}$$

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

$$P_{nf} = 2.7t_2dF_{u2} \quad \text{AISI S100 Eq. E4.3.1-5}$$

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

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

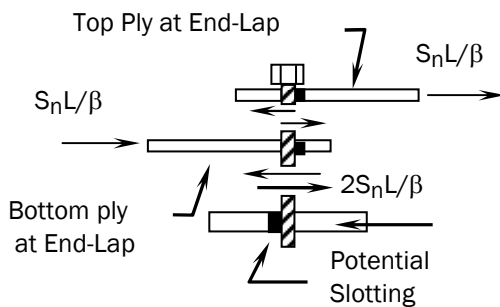
**Case 1 result:**  $P_{nf} = 0.910 \text{ kips}$  **Applies at butt-joint and end-lap installations**

**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):

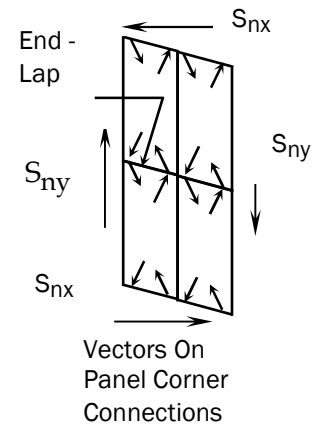
When the shear transfer plane is between the bottom ply and the support (as shown in the figure at left), the shear flow at this shear transfer plane is doubled relative to the force in a single ply. Since the bearing strength of screw against

support and screw breaking strength are greater than twice the bearing strength against one panel thickness, i.e.  $2.27 k$  and  $2 k > 2(0.910 k) = 1.82 k$ , 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  $S_{ni}$ . 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 since both components are not present at the support.



**Case 2 result: Based on the above discussion,  $P_{nf} = 0.910 k$  – 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 E4.3.1):

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

$$P_{ns} = 4.2(t_2^3 d)^{1/2} F_{u2} \quad \text{AISII S100 Eq. E4.3.1-1}$$

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

$$P_{ns} = 2.7t_1dF_{u1} \quad \text{AISI S100 Eq. E4.3.1-2}$$

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

$$P_{ns} = 2.7t_2dF_{u2} \text{ (same as above)} = 0.910 \text{ k} \quad \text{AISI S100 Eq. E4.3.1-3}$$

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

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

**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*.

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

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

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

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

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

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

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

**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  $s$ .

$$\alpha = 1 \quad \text{Conversion factor for U.S. customary units}$$

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

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

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

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

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

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

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

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

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

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

**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 (AISI S100 Table D5):

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

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

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

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

### 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:

$$\Omega_{df} = 2.35 \quad \text{and} \quad \Omega_{db} = 2.0$$

$$\frac{S_n}{\Omega} = \min\left(\frac{0.396}{2.35}, \frac{3.82}{2}\right) = 0.169 \text{ klf}$$

LRFD:

$$\phi_{df} = 0.70 \quad \text{and} \quad \phi_{db} = 0.80$$

$$\phi S_n = \min(0.70(0.396), 0.80(3.82)) = 0.277 \text{ klf}$$

LSD:

$$\phi_{df} = 0.65 \quad \text{and} \quad \phi_{db} = 0.75$$

$$\phi S_n = \min(0.65(0.396), 0.75(3.82)) = 0.257 \text{ klf}$$

**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*.

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

$\phi_d S_n$  for LRFD and LSD

*Safety Factors* and *Resistance Factors* are in Section B1 (AISI S100 Table D5) 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,  $P_{nft}$ . Section D1 will be used to determine  $S_{nf}$  based on  $P_{nft}$ .  $S_n = \min(S_{nf}, S_{nb})$ . From Example 2a,  $S_{nb}$  will not control  $S_n$ .

### Load Data

Wind uplift zone = 30 psf (0.030 ksf)

Reaction at *interior support* = 1.1(5 ft)(0.030 ksf)  
= 0.165 klf

for pull-over and pull-out

Reaction at *exterior support* = 0.4(5 ft)(0.030 ksf)  
= 0.060 klf  
= 2(0.060 klf)  
= 0.120 klf

for one-thickness pull-over

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 E4.5.2 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 E4.5.1)

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



**Note:**  $t = 0.024$  in. is slightly outside the specified limits of this equation, but will be used in this example for illustrative purposes.

At interior supports:

$$N = 1 \text{ screw/ft at all supports (See Example 2a)}$$

$$T = \frac{0.165 \text{ k/ft}}{1 \text{ screw/ft}} = 0.165 \text{ k/screw} \quad \text{Tensile force per screw}$$

$$P_{\text{nov}} = 1.5t_1d_w'F_{u1} \quad \text{AISI S100 Section E4.4.2}$$

$$= 1.5(0.024 \text{ in.})(0.5 \text{ in.})\left(65 \frac{\text{k}}{\text{in.}^2}\right) = 1.17 \text{ k}$$

**Note:** The above pull-over strength is based on one thickness with uniform pull-over.

$$\left(\frac{P_{\text{nft}}}{P_{\text{nf}}}\right) = \frac{1.1\Omega_d}{\Omega} - \left(\frac{0.71T\Omega_d}{P_{\text{nov}}}\right) \leq 1.0 \quad \text{Rewriting Eq. D3.1.2.1-2}$$

$$= \frac{1.1(2.35)}{2.35} - \left(\frac{0.71(0.165 \text{ k})(2.35)}{1.17 \text{ k}}\right)$$

$$= 0.865$$

$$P_{\text{nf}} = 0.910 \text{ k} \quad \text{From Example 2a}$$

**Result:**  $P_{\text{nft}} = 0.865(0.910 \text{ k}) = 0.787 \text{ k / screw}$  at interior supports

At exterior supports:

$$T = \frac{0.060 \text{ k/ft}}{1 \text{ screw/ft}} = 0.060 \text{ k/screw}$$

**Note:** Top ply resists 0.06 k and sees end eccentricity.  $S_{\text{nc}}$  concerns corner connection at end-lap.

$$P_{\text{nov}} = 0.5(1.17 \text{ k}) = 0.585 \text{ k} \quad \text{Eccentrically loaded connections} \quad \text{AISI S100 Section E4.5.1.1}$$

$$\left(\frac{P_{\text{nft}}}{P_{\text{nf}}}\right) = \frac{1.1\Omega_d}{\Omega} - \left(\frac{0.71T\Omega_d}{P_{\text{nov}}}\right) \leq 1.0 \quad \text{Rewriting Eq. D3.1.2.1-2}$$

$$\left(\frac{P_{\text{nft}}}{P_{\text{nf}}}\right) = \frac{1.1(2.35)}{2.35} - \left(\frac{0.71(0.06 \text{ k})(2.35)}{0.5(1.17 \text{ k})}\right) = 0.929$$

$$P_{\text{nf}} = 0.910 \text{ k} \quad \text{From Example 2a}$$

**Result:**  $P_{\text{nft}} = 0.929(0.910 \text{ k}) = 0.845 \text{ k/screw}$  at exterior supports

**Calculate  $S_{\text{nf}}$  controlled by pull-over:**

**Note:** See theory adjustment at Example 1a for variation of *support connection shear strength* along a panel length,  $L$ . Eqs. D1-1 and D1-2 are modified as follows:

Let:  $P_{\text{nfe}}$  = Nominal connection strength at exterior supports in one ply, controlled by pull-over

$P_{\text{nfi}}$  = Nominal connection strength at interior supports in one ply, controlled by pull-over

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

Rewriting so closer in form to Eq. D1-1 and to develop a modified  $\beta$ :

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

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

See Example 2a for parameters:  $A$ ,  $n_s$ ,  $\alpha_e^2$ ,  $\alpha_p^2$ , and  $\lambda$

$$P_{nfi} = 0.787 \text{ k}$$

$$P_{ns} = 0.472 \text{ k (Example 2a, and not affected by uplift.)}$$

$$P_{nfe} = 0.845 \text{ k}$$

**Calculate  $S_{ni}$  based on  $P_{nfi}$  controlled by pull-over:**

$$\beta = n_s \alpha_s + 2n_p \alpha_p^2 + 4\alpha_e^2 \frac{P_{nfe}}{P_{nfi}} \quad \text{Modified Eq. D1-5}$$

$$\alpha_s = \frac{P_{ns}}{P_{nfi}} = \frac{0.472 \text{ k}}{0.787 \text{ k}} = 0.600 \quad \text{Eq. D1-6}$$

$$\beta = 16(0.600) + 2(4)(0.32) + 4(0.32) \frac{0.845 \text{ k}}{0.787 \text{ k}} = 13.5 \quad \text{Modified Eq. D1-5}$$

$$S_{ni} = [2(0.0)(0.798 - 1) \frac{0.845 \text{ k}}{0.787 \text{ k}} + 13.5] \frac{0.787 \text{ k}}{25 \text{ ft}} \quad \text{Modified Eq. D1-1}$$

$$= 0.425 \text{ klf}$$

**Calculate  $S_{nc}$  based on  $P_{nfe}$  controlled by pull-over:**

$$\beta = n_s \alpha_s + 2n_p \alpha_p^2 \frac{P_{nfi}}{P_{nfe}} + 4\alpha_e^2 \quad \text{Modified Eq. D1-5}$$

$$\alpha_s = \frac{P_{ns}}{P_{nfi}} = \frac{0.472 \text{ k}}{0.845 \text{ k}} = 0.559 \quad \text{Eq. D1-6}$$

$$\beta = 16(0.559) + 2(4)(0.32) \frac{0.787 \text{ k}}{0.845 \text{ k}} + 4(0.32) = 12.6 \quad \text{Modified Eq. D1-5}$$

$$S_{nc} = \left( \frac{(1)^2 (12.6)^2 \frac{1}{\text{ft}^2}}{(25)^2 (1)^2 \frac{\text{ft}^2}{\text{ft}^2} + (12.6)^2} \right)^{0.5} 0.845 \text{ k} = 0.380 \text{ klf} \quad \text{Eq. D1-2}$$

$$S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.425, 0.380) = 0.380 \text{ klf} \quad \text{Section D1}$$

**Result controlled by pull-over:**

$$S_{nf} = 0.380 \text{ klf} \quad 4\% \text{ reduction vs. } 0.396 \text{ klf from Example 2a (no uplift)}$$

**(b) Interaction of shear and pull-out (Section D3.1.2.1(b) or AISI S100 Section E4.5.2.1)**

**Note:** AISI Section E4.5.2.1 limits (a) through (d) are met.

$$\left( \frac{P_{nft}}{\Omega_d P_{nf}} \right) + \left( \frac{T}{P_{not}} \right) = \frac{1.15}{\Omega} \quad \text{Eq. D3.1.2.1-6}$$

$$\left(\frac{P_{nft}}{P_{nf}}\right) = \frac{1.15\Omega_d}{\Omega} - \left(\frac{T\Omega_d}{P_{not}}\right) \leq 1.0 \quad \text{Rewriting Eq. D3.1.2.1-6}$$

$$\Omega = 2.55 \quad \Omega_d = 2.35$$

#### Calculate $P_{not}$ using AISI S100 Section E4.4.1:

**Note:** Screw threads penetrate support thickness so  $t_c = t_2 = 0.06$  in.

$$P_{not} = 0.85t_c d F_{u2} \quad \text{AISI S100 Eq. E4.4.1-1}$$

$$= 0.85(0.06 \text{ in.})(0.216 \text{ in.})\left(65 \frac{\text{k}}{\text{in.}^2}\right) = 0.716 \text{ k} < P_{nts} = 2.7 \text{ k}$$

Therefore,

$$P_{not} = 0.716 \text{ k}$$

At interior supports:

$$T = \frac{0.165 \text{ k/ft}}{1 \text{ screw/ft}} = 0.165 \text{ k/screw}$$

$$\left(\frac{P_{nft}}{P_{nf}}\right) = \frac{1.15(2.35)}{2.55} - \frac{(0.165 \text{ k})(2.35)}{0.716 \text{ k}} = 0.518 \quad \text{Eq. D3.1.2.1-6}$$

$$P_{nf} = 0.910 \text{ kips} \quad \text{From Example 2a}$$

#### Result of $P_{nft}$ at interior supports:

$$P_{nft} = 0.518(0.910 \text{ k}) = 0.471 \text{ k}$$

At exterior supports:

$$T = \frac{0.120 \text{ k/ft}}{1 \text{ screw/ft}} = 0.120 \text{ k/screw}$$

$$\left(\frac{P_{nft}}{P_{nf}}\right) = \frac{1.15(2.35)}{2.55} - \frac{(0.12 \text{ k})(2.35)}{0.716 \text{ k}} = 0.666 \quad \text{Eq. D3.1.2.1-6}$$

$$P_{nf} = 0.910 \text{ kips} \quad \text{From Example 2a}$$

#### Result of $P_{nft}$ at exterior supports:

$$P_{nft} = 0.666(0.910 \text{ k}) = 0.606 \text{ k}$$

#### Calculate $S_{nf}$ controlled by pull-out:

See Example 2a for parameters:  $A$ ,  $n_s$ ,  $\alpha_e^2$ ,  $\alpha_p^2$ , and  $\lambda$

$$P_{nfi} = 0.471 \text{ k} \quad P_{ns} = 0.472 \text{ k (Example 2a)}$$

$$P_{nfe} = 0.606 \text{ k}$$

#### Calculate $S_{ni}$ based on $P_{nfi}$ controlled by pull-out:

$$\beta = n_s \alpha_s + 2n_p \alpha_p^2 + 4\alpha_e^2 \frac{P_{nfe}}{P_{nfi}} \quad \text{Modified Eq. D1-5}$$

$$\alpha_s = \frac{P_{ns}}{P_{nfi}} = \frac{0.472 \text{ k}}{0.471 \text{ k}} = 1.00 \quad \text{Eq. D1-6}$$

$$\begin{aligned} \beta &= 16(1.00) + 2(4)(0.32) + 4(0.32) \frac{0.606 \text{ k}}{0.471 \text{ k}} \\ &= 20.2 \end{aligned}$$

$$\begin{aligned} S_{ni} &= [2(0.0)(0.798 - 1) \frac{0.606 \text{ k}}{0.471 \text{ k}} + 20.2] \frac{0.471 \text{ k}}{25 \text{ ft}} \\ &= 0.381 \text{ klf} \end{aligned} \quad \text{Modified Eq. D1-1}$$

**Calculate  $S_{nc}$  based on  $P_{nfe}$  controlled by pull-out:**

$$\beta = n_s \alpha_s + 2n_p \alpha_p^2 \frac{P_{nfi}}{P_{nfe}} + 4\alpha_e^2 \quad \text{Modified Eq. D1-5}$$

$$\alpha_s = \frac{P_{ns}}{P_{nfe}} = \frac{0.472 \text{ k}}{0.606 \text{ k}} = 0.779 \quad \text{Eq. D1-6}$$

$$\begin{aligned} \beta &= 16(0.779) + 2(4)0.32 \frac{0.471 \text{ k}}{0.606 \text{ k}} + 4(0.32) \\ &= 15.7 \end{aligned}$$

$$\begin{aligned} S_{nc} &= \left( \frac{(1)^2 (15.7)^2 \frac{1}{\text{ft}^2}}{(25)^2 (1)^2 \frac{\text{ft}^2}{\text{ft}^2} + (15.7)^2} \right)^{0.5} 0.606 \text{ k} \\ &= 0.322 \text{ klf} \end{aligned} \quad \text{Eq. D1-2}$$

$$S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.381, 0.322) = 0.322 \text{ klf} \quad \text{Section D1}$$

**Result controlled by pull-out:**

$$S_{nf} = 0.322 \text{ klf} \quad 18.7\% \text{ reduction vs. } 0.396 \text{ klf from Example 2a}$$

**(c) Interaction of shear and tension in screw (Section D3.1.2.1(c) or AISI S100 Section E4.5.3)**

**Note:** Although  $P_{nf}$  in Eq. D3.1.2.1-10 should be  $P_{nss}$  ( $=2 \text{ k}$ ),  $P_{nf} = 0.910 \text{ k}$  from Example 2a is used to conservatively calculate  $P_{nft}$ .

$$\left( \frac{P_{nft}}{P_{nf}} \right) + \left( \frac{\Omega_t T}{P_{nts}} \right) = 1.3 \quad \text{Eq. D3.1.2.1-10}$$

$$\left( \frac{P_{nft}}{P_{nf}} \right) = 1.3 - \left( \frac{\Omega_t T}{P_{nts}} \right) \leq 1.0 \quad \text{Rewriting Eq. D3.1.2.1-10}$$

$$\Omega_t = 3.0 \quad P_{nts} = 2.7 \text{ k}$$

At interior supports,  $T = 0.165 \text{ k/screw}$ , and from the interaction above:

$$\left( \frac{P_{nft}}{P_{nf}} \right) = 1.3 - \left( \frac{3(0.165 \text{ k})}{2.7 \text{ k}} \right) = 1.12 > 1.0 \quad \text{Therefore, } 1.0 \text{ is used.} \quad \text{Eq. D3.1.2.1-10}$$

**Result of  $P_{nft}$  at interior supports:**

$$P_{nft} = 1(0.910) = 0.910 \text{ k} \quad (\text{no reduction})$$

At exterior supports,  $T = 0.120 \text{ k/screw}$  – See reactions from above.

$$\left( \frac{P_{nft}}{P_{nf}} \right) = 1.3 - \left( \frac{3(0.120 \text{ k})}{2.7 \text{ k}} \right) = 1.17 > 1.0 \quad \text{Therefore, 1.0 is used.}$$

**Result of  $P_{nft}$  at exterior supports:**

$$P_{nft} = 1(0.910) = 0.910 \text{ k} \quad (\text{No reduction})$$

**Result controlled by shear and tension in screw (breaking strength):**

$$S_{nf} = 0.396 \text{ klf} \quad \text{Same as no uplift – See Example 2a}$$

**Example 2b Result (With Uplift)**

$$S_n = \min(0.380, 0.322, 0.396)$$

$$\text{Nominal Strength: } S_n = 0.322 \text{ klf}$$

**Note:** Screw pull-out controls. The 30 psf uplift significantly impacts the *nominal diaphragm shear strength* per unit length of this configuration. The support is relatively thin, which resulted in a 30-50% reduction in support screw strength and an 18.7% reduction in diaphragm strength. These unproportional reductions indicate the strength contribution from *side-lap* screws. Adding support fasteners or increasing support or *panel* thickness could be considered to increase  $S_n$ .

$$\Omega_d = 2.35 \text{ for wind load}$$

$$\text{Available Strength: } \frac{S_n}{\Omega} = \frac{0.322}{2.35} = 0.137 \text{ klf}$$

**Example 2b Discussion:**

The following calculation uses a simpler design approach by using only one reaction force at the exterior support,  $T = 0.165 \text{ k/conn.}$ , to determine both  $S_{ni}$  and  $S_{nc}$  while still considering eccentricity at the *panel* ends:

**(a) Interaction of shear and pull-over**

At interior supports,

From (a) above with no eccentricity and based on  $T = 0.165 \text{ k/conn.}$

$$P_{nft} = 0.787 \text{ k}$$

At exterior supports,

$$\left( \frac{P_{nft}}{P_{nf}} \right) = \frac{1.1(2.35)}{2.35} - \frac{0.71(0.165 \text{ k})(2.35)}{0.5(1.17 \text{ k})} = 0.629$$

$$P_{nf} = 0.910 \text{ k} \quad \text{From Example 2a}$$

$$P_{nft} = 0.629(0.910) = 0.572 \text{ k}$$

**Calculate  $S_{nf}$  controlled by pull-over:**

See Example 2a for parameters:  $A$ ,  $n_s$ ,  $\alpha_e^2$ ,  $\alpha_p^2$ , and  $\lambda$

$$P_{nfi} = 0.787 \text{ k}$$

$$P_{ns} = 0.472 \text{ k (Example 2a, and not affected by uplift)}$$

$$P_{nfe} = 0.572 \text{ k}$$

**Calculate  $S_{ni}$  based on  $P_{nfi}$  controlled by pull-over:**

$$\beta = n_s \alpha_s + 2n_p \alpha_p^2 + 4\alpha_e^2 \frac{P_{nfe}}{P_{nfi}}$$

$$\alpha_s = 0.600 \quad \text{Based on } P_{nfi} = 0.787 \text{ k from (a) above} \quad \text{Eq. D1-6}$$

$$\beta = 16(0.600) + 2(4)(0.32) + 4(0.32) \frac{0.572 \text{ k}}{0.787 \text{ k}} \quad \text{Modified Eq. D1-5}$$

$$= 13.1$$

$$S_{ni} = [2(0.0)(0.798 - 1) \frac{0.572 \text{ k}}{0.787 \text{ k}} + 13.1] \frac{0.787 \text{ k}}{25 \text{ ft}} \quad \text{Modified Eq. D1-1}$$

$$= 0.412 \text{ klf}$$

**Calculate  $S_{nc}$  based on  $P_{nfe}$  controlled by pull-over:**

$$\beta = n_s \alpha_s + 2n_p \alpha_p^2 \frac{P_{nfi}}{P_{nfe}} + 4\alpha_e^2 \quad \text{Modified Eq. D1-5}$$

$$\alpha_s = \frac{0.472 \text{ k}}{0.572 \text{ k}} = 0.825 \quad \text{Eq. D1-6}$$

$$\beta = 16(0.825) + 2(4)(0.32) \frac{0.787 \text{ k}}{0.572} + 4(0.32) = 18.0$$

$$S_{nc} = \left( \frac{(1)^2 (18.0)^2 \frac{1}{\text{ft}^2}}{(25)^2 (1)^2 \frac{\text{ft}^2}{\text{ft}^2} + (18.0)^2} \right)^{0.5} 0.572 \text{ k} = 0.334 \text{ klf} \quad \text{Eq. D1-2}$$

$$S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.412 \text{ klf}, 0.334 \text{ klf}) = 0.334 \text{ klf}$$

For pull-over: 15.7% reduction vs. 0.396 klf – no uplift, and

12.1% reduction vs. 0.380 klf – more precise pull-over analysis

**(b) See interaction of shear and pull-out above**

At interior and exterior supports,

From (b) above based on  $T = 0.165 \text{ k/conn}$

$$P_{nft} = 0.518(0.910 \text{ k}) = 0.471 \text{ k}$$

$$\alpha_s = \frac{0.472 \text{ k}}{0.471 \text{ k}} = 1.00 \quad \text{Eq. D1-6}$$

$$\beta = 16(1.00) + 2(4)(0.32) + 4(0.32) = 19.8 \quad \text{Eq. D1-5}$$

Calculate  $S_{ni}$  and  $S_{nc}$  based on  $P_{nft}$  controlled by pull-out:

$$S_{ni} = [2(0.0)(0.798 - 1) + 19.8] \frac{0.471 \text{ k}}{25 \text{ ft}} = 0.373 \text{ klf} \quad \text{Eq. D1-1}$$

$$S_{nc} = \left( \frac{(1)^2 (19.8)^2 \frac{1}{\text{ft}^2}}{(25)^2 (1)^2 \frac{\text{ft}^2}{\text{ft}^2} + (19.8)^2} \right)^{0.5} 0.471 \text{ k} = 0.292 \text{ klf} \quad \text{Eq. D1-2}$$

$$S_{nf} = \min(S_{ni}, S_{ns}) = \min(0.373 \text{ klf}, 0.292 \text{ klf}) = 0.292 \text{ klf}$$

For pull-out: 26% reduction vs. 0.396 klf - no uplift, and

9.3% reduction vs. 0.322 klf - more precise pull-out analysis

**(c) See interaction of shear and tension in screw above**

At interior and exterior supports,

From (c) above based on  $T = 0.165 \text{ k/conn}$

$$P_{nft} = 1.0(0.910) = 0.910 \text{ k}$$

$$S_{nf} = 0.396 \text{ klf} \text{ Same as no uplift—See Example 2a based on } P_{nf} = 0.910 \text{ k}$$

**Example 2b Result Using Simpler Design**

$$S_n = \min(0.334, 0.292, 0.396) = 0.292 \text{ klf}$$

**Note:** The simpler design has significant impact on the *nominal diaphragm shear strength* per unit length of this configuration. Pull-out still controls, and simplification causes a 9.3% reduction vs. 0.322 klf using the more precise calculation.

### Example 2c: Required Edge Detail Parallel to the Edge Panel Span With Panel Connected to a Thin Support in the Absence of Uplift

#### Objective

Using Section D1, determine a required edge detail parallel to the *edge panel* span so  $S_{ne}$  exceeds  $S_n$  from Example 2a (no uplift) = 0.396 klf.

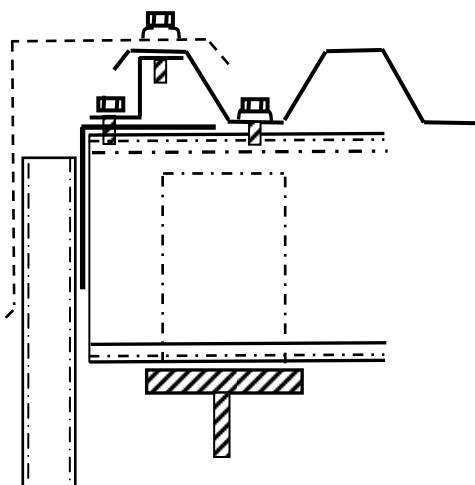
**Note:** The Section D1 requirements for  $P_{nfs}$  must be met, and Eq. D1-3 with the Eq. D4.4-1 simplification will be used. The simplification only considers the contribution of screws over the edge support angle by:

- Setting  $\alpha_1$  and  $\alpha_2$  to 1 in Eq. D1-3, and
- Requiring a screw in line with the transverse support at this detail. The designer could include the contributions of other transverse support screws through the bottom flat in the *edge panel* and calculate  $\alpha_1$  and  $\alpha_2 > 1$ .

The following conditions apply at the detail:

- Starting and ending conditions allow installation of full-width sheets,  $w_e = 36$  in.
- Starting and ending conditions do not allow a bottom flat on the edge support angle.
- The edge *connection* is through the *edge panel's* top flat; therefore, set  $P_{nfs} = 0.0$  k. ( $S_{ne} \approx 0.0$  klf) or require a detail that provides a direct path to the edge angle so *connections* are effective. See Section D1.
- #12 side-lap screw with 1/2 in. round sealing washer will be used at *panel* to Zee.
- #12 support screw without washer will connect 1 1/2 in. deep Zee to edge support.
- Try to match side-lap screw spacing = 20 in. so that  $n_e = n_s$ .
- Try to use reinforcing zee with  $t = 0.06$  in.,  $F_y = 50$  ksi, and  $F_u = 65$  ksi.
- Edge angle is same material as supports:  $t = 0.06$  in.,  $F_y = 50$  ksi, and  $F_u = 65$  ksi.

**Note:** Interior lateral force resisting systems might require a reaction ( $S_{ne}$ )  $> S_n = 0.396$  klf.



Starting *edge panel* detail will transfer diaphragm shear to a *Lateral Force Resisting System*. Anti-roll clip or other means is provided to prevent purlin roll.

1-1/2 in. deep Zee member is used to transfer longitudinal *diaphragm shear* from the top flange screw connections to the bottom flange screw connections. Due to screw eccentricity, the Zee web has potential for twisting and warping.



**Calculate nominal diaphragm shear strength,  $S_{ne}$** **Calculate connection strength along starting edge panel,  $P_{nfs}$ :**

Panel to 1-1/2 in. Zee-shaped member

**Note:** 1-1/2 in. Zee member material is the same as the support.

Typical and maximum Zee member length is 10 ft due to press-brake limits.

See Example 2a,  $P_{nfs} = P_{nf}$

$$P_{nfs} = \min(0.910, 2.27, 2.00) = 0.910 \text{ k}$$

Bearing of panel against screw controls connection strength.

Calculate required screw spacing—Edge panel to 1-1/2 in. deep Zee:

$$n_e = \frac{S_n L}{P_{nfs}} \quad \text{Rewriting} \quad \frac{L}{n_e} = \frac{P_{nfs}}{S_n} \quad \text{Eq. D4.4-1}$$

$$\frac{L}{n_e} = \frac{0.910 \frac{\text{k}}{\text{screw}}}{0.396 \frac{\text{k}}{\text{ft}}} = 2.30 \frac{\text{ft}}{\text{screw}}$$

**Note:** 2.30 ft/screw > 20 in. o.c. (1.67 ft/screw)

**Use:** Spacing = 20 in. o.c. (1.67 ft) to match  $n_s$ .

Install a screw in line with each transverse support and at a maximum spacing of 20 in. o. c. between supports.

$n_e = 16$  Edge screws along the panel length,  $L = 25 \text{ ft}$

Zee-to-edge support angle:

$$\frac{t_2}{t_1} = \frac{0.060 \text{ in.}}{0.060 \text{ in.}} = 1.0$$

$$P_{nfs} = 4.2 \left( (0.060 \text{ in.})^3 (0.216 \text{ in.}) \right)^{1/2} \left( 65 \frac{\text{k}}{\text{in.}^2} \right) \quad \text{AISI S100 Eq. 4.3.1-1}$$

$$= 1.86 \text{ k} < P_{nss} = 2.0 \text{ k}$$

$$P_{nfs} = 2.7 (0.060 \text{ in.}) (0.216 \text{ in.}) \left( 65 \frac{\text{k}}{\text{in.}^2} \right) \quad \text{AISI S100 Eq. 4.3.1-2}$$

$$= 2.27 \text{ k} > P_{nss} = 2.0 \text{ k}$$

$$P_{nfs} = \min(1.86 \text{ k}, 2.27 \text{ k}, 2.0 \text{ k}) = 1.86 \text{ k}$$

**Note:** Tilting of screw in angle controls but almost at limit of screw breaking strength= 2 k

Calculate required screw spacing—Zee-to-edge support angle:

$$\frac{L}{n_e} = \frac{1.86 \frac{\text{k}}{\text{screw}}}{0.396 \frac{\text{k}}{\text{ft}}} = 4.7 \frac{\text{ft}}{\text{screw}} \quad \text{Eq. D4.4-1}$$

**Use:** Maximum tributary length / screw = 4 ft.

Zee Length	Screws/Length	Spacing
$L_z = 10$ ft	3	12 in., 4 ft - 0 in., 4 ft - 0 in., 12 in. = 10 ft
$5$ ft < $L_z$ < 10 ft	3	
$L_z \leq 5$ ft	2	6 in., 4 ft - 0 in., 6 in. = 5 ft

**Note:** Min. of two screws/length is required to prevent twisting rotation.

**Calculate shear stress in 1-1/2 in. Zee (ASD):**

Let:  $\Omega = \Omega_d = 2.35$

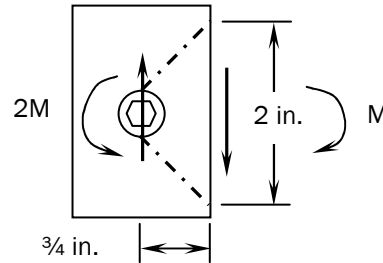
ASD Available diaphragm shear strength at edge =  $\frac{0.396 \text{ klf}}{2.35} = 0.169 \text{ klf}$

**Note:** For illustration, 2.35 is the smallest *safety factor* and creates the largest ASD shear stress in 1-1/2 in. Zee for a *nominal diaphragm shear* per unit length.

$$\tau_v = \frac{0.169 \frac{\text{k}}{\text{ft}}}{12(0.06) \frac{\text{in.}^2}{\text{ft}}} = 0.235 \text{ ksi} \quad \text{Based on entire Zee length resisting shear equally}$$

Since shear has to get out at connections, consider that not all zones of Zee work in bottom flat. Based on maximum Zee tributary length of 4.0 ft per screw (> 20 in.) and based on 1-1/2 in. Zee flat dimensions, 2 in. of bottom flat resist shear transfer:

$$\tau_v \leq \frac{4.0 \text{ ft}(0.169) \frac{\text{k}}{\text{ft}}}{2(0.06) \text{ in.}^2} = 5.63 \text{ ksi}$$



**Screw in Zee bottom flat**

**Note:** Twisting due to screw eccentricity will create secondary (x axis) reaction on bottom flat screws. Screw strength will be re-checked below.

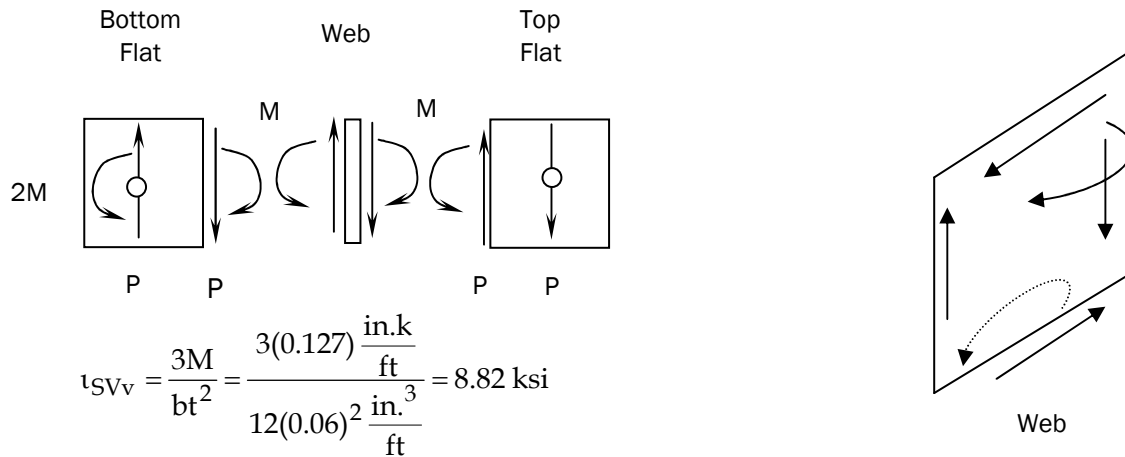
Calculate St. Venant shear in Zee web due to twisting (screw eccentricity):

**Note:** Refer to *Advanced Mechanics of Materials* by Seely and Smith or other textbooks for torsional resistance of rectangular cross-sections:

ASD Available Diaphragm Strength = 0.169 klf

$P = 0.169 \text{ klf}$  Shear transfer to Zee

$M = (0.75 \text{ in.})(0.169 \text{ klf}) = 0.127 \text{ in. k/ft}$



Calculate resultant shear stress at outer edges of web:

$$f_v \leq \tau_v + \tau_{SV} = 5.63 + 8.82 = 14.5 \text{ ksi}$$

Determine *nominal shear strength*,  $F_v$ , of web:      AISI S100 Section C3.2.1

$$\frac{h}{t} = \frac{1.5 \text{ in.}}{0.06 \text{ in.}} = 25 \quad \uparrow \begin{array}{|c|} \hline \square \\ \hline \end{array} \downarrow$$

$$\text{For } \frac{h}{t} \leq \sqrt{\frac{Ek_v}{F_y}} \quad F_v = 0.6F_y \quad \sqrt{\frac{Ek_v}{F_y}} = \sqrt{\frac{29500 \text{ ksi}(5.34)}{50 \text{ ksi}}} = 56.1 > 25$$

$$F_v = 0.6(50 \text{ ksi}) = 30 \text{ ksi} \quad \text{Allowable shear stress} = \frac{F_v}{1.6} = \frac{30}{1.6} = 18.8 \text{ ksi}$$

Result: Since 18.8 ksi > 14.5 ksi,  $t = 0.06 \text{ in.}$  OK

Find secondary (x axis) forces on Zee's bottom flat screws

**Note:** Due to screw eccentricity, re-check screw *available strength*:

$$2M = 2(0.127 \text{ in. k/ft}) = 0.254 \text{ in. k/ft}$$

Consider 10 ft length with screw spacing = 1.0, 4.0, 4.0, 1.0 = 10 ft length

Outer screws (8 ft apart)

$$P_x = \frac{0.254 \frac{\text{in. k}}{\text{ft}} (10) \text{ft}}{8 \text{ ft}(12) \frac{\text{in.}}{\text{ft}}} = 0.026 \text{ k} \quad P_y = (0.169 \text{ klf})(1 + 0.5(4.0)) \text{ ft} = 0.507 \text{ k}$$

Center screw (at pivot point of twist)

$$P_x = 0.0 \text{ k} \quad P_y = (0.169 \text{ klf})(4.0 \text{ ft}) = 0.676 \text{ k}$$

Consider 5 ft length with screw spacing = 0.5, 4.0, 0.5 = 5 ft length

Outer screws (4 ft apart)

$$P_x = \frac{0.254 \frac{\text{in. k}}{\text{ft}} (5) \text{ ft}}{4 \text{ ft} (12) \frac{\text{in.}}{\text{ft}}} = 0.026 \text{ k} \quad P_y = (0.169 \text{ klf})(0.5 + 0.5(4.0)) \text{ ft} = 0.423 \text{ k}$$

$$\text{Required Strength} \leq \sqrt{(0.507)^2 + (0.026)^2} = 0.508 \text{ k/ screw at outer screw}$$

$$= 0.676 \text{ k/ screw at center screw}$$

$$P_{\text{nfs}} = 1.86 \text{ k from above}$$

$$\text{Available Strength} = \frac{1.86 \text{ k}}{2.35} = 0.791 \text{ k/screw} > 0.676 \text{ k/screw OK}$$

**Note:** This just verifies that 4 ft. spacing is less than 4.7 ft determined above.  
Secondary force is negligible –  $P_{\text{resultant}} = 0.508 \text{ k}$  vs.  $P_y = 0.507 \text{ k}$

### Example 2c Results

**Use:** 1-1/2 in. Zee with  $t = 0.06 \text{ in.}$  and  $F_y = 50 \text{ ksi}$  Typical length = 10 ft.

**Panel to Zee:** #12 × 1 in. long *side-lap* screw with 1/2 in. round sealing washer at 20 in. o.c. (5 ft = 60 in. o.c. so it works with 20 in. o.c.)  
Flashing is not required for *diaphragm shear strength*.  
Install screws in line with transverse supports.

**Zee to Support:** #12 × 1 in. long support screw without washer

Zee Length	Screws/Length	Screw Spacing
$L_z = 10 \text{ ft}$	3	12 in., 4 ft - 0 in., 4 ft - 0 in., 12 in. = 10 ft
$5 \text{ ft} < L_z < 10 \text{ ft}$	3	6 in., 3 ft - 0 in., 3 ft - 0 in., 6 in. = 7 ft
$L_z \leq 5 \text{ ft}$	2	6 in., 4 ft - 0 in., 6 in. = 5 ft

$S_n = 0.396 \text{ klf}$  Nominal *diaphragm strength* per unit length without uplift is controlled by the corner *connections* in the panels.  $S_{ne}$  exceeds  $S_n$ .

#### Note:

- 1) Example 2c establishes a direct load path to the *lateral force resisting system* and provides design considerations for the edge connection details. Other details are possible.
- 2) The ending edge condition could require an alternate detail if the *edge panel* bottom flat lands on the edge support or if a partial-width *panel* is required. See Example 3d for such *edge panel* conditions.
- 3) Zee-member connection selection considers normal press-brake limits, use of the same stock material, and practical screw spacing limits.
- 4) Uplift is taken by transverse *support connections* and is not significantly resisted by this edge detail.
- 5) Rake flashing may be required for cosmetics or serviceability. Follow the manufacturer's guidelines for flashings and minimum attachments.

## Example 2d: Diaphragm Stiffness Calculation

### Objective

Use the *configuration* of Example 2a and determine the *diaphragm stiffness*,  $G'$ .

**Note:** Use Section D5.1.2 for perforated *panels* and Appendix Sections 1.4 and 1.6. Same  $G'$  applies with or without uplift.

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

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

$$\begin{aligned} \alpha_3 = \alpha_4 &= \frac{\sum x_e}{w} && \text{See Example 2a for parameters} \\ &= \frac{0 + 2(14.4) \text{ in.}}{36 \text{ in.}} = 0.8 && \text{Eq. D5.1.1-3} \end{aligned}$$

**Note:**  $G'$  should be evaluated using *interior panels*.

### Calculate screw flexibilities:

$$S_f = \frac{1.3\alpha}{1000\sqrt{t}} = \frac{1.3(1)}{1000\sqrt{0.024 \text{ in.}}} = 0.00839 \frac{\text{in.}}{\text{k}} \quad \text{Eq. D5.2.2-1}$$

$$S_s = \frac{3.0\alpha}{1000\sqrt{t}} = \frac{3.0(1)}{1000\sqrt{0.024 \text{ in.}}} = 0.0194 \frac{\text{in.}}{\text{k}} \quad \text{Eq. D5.2.2-2}$$

$$K = \frac{S_f}{S_s} = \frac{0.00839}{0.0194} = 0.433 \quad \text{Factor relating support and side-lap conn. flexibility. Section D5.1.1}$$

**Note:** Above  $K$  is for *configurations* with lap-up at *side-lap* and screws into steel supports at flute bottom flats.

### Calculate screw slip constant:

$$C = \left( \frac{29500 \frac{\text{k}}{\text{in.}^2} (0.024 \text{ in.})}{36 \text{ in.}} \right) \left( \frac{2(12 \frac{\text{in.}}{\text{ft}})(25 \text{ ft})}{2(0.8) + 4(0.8) + 2(16)(0.433)} \right) 0.00839 \frac{\text{in.}}{\text{k}} = 5.31$$

### Calculate warping factor, $D_n$ :

**Note:** The value,  $D_n$ , is defined in Appendix 1 and is normally determined using a spreadsheet. All constants will be determined in this example so the results can be

used as a check. Example 2d only requires  $D_1$  and  $D_2$ ; all deflection indicators and spring constants are not required.

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

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

$$D_1 = \frac{\gamma_1 f}{d(t)^{1.5}} \quad \text{Eq. 1.4-3} \quad D_2 = \frac{\gamma_2 f}{2d(t)^{1.5}} \quad \text{Eq. 1.4-4}$$

$$D_3 = \frac{\gamma_3 f}{3d(t)^{1.5}} \quad \text{Eq. 1.4-5} \quad D_4 = \frac{\gamma_4 f}{4d(t)^{1.5}} \quad \text{Eq. 1.4-6}$$

Calculate deflection indicators:

**Note:** See Panel Data in Example 2a for parameters. See  $S_{nb}$  determination in Example 2a for calculation of equivalent  $w_p$ .

$$w_p = K_{E_w}^{1/3} w = 2.32 \text{ in.} \quad \text{Eq. 1.6-3}$$

$$2e_p = 2e \text{ (no perf.)} = 2.00 \text{ in.} \quad f_p = f \text{ (no perf.)} = 2.00 \text{ in.}$$

Insert the equivalent element lengths in Eq. 1.4-7 through Eq. 1.4-10 to calculate the deflection indicators:

$$s = 2e_p + 2w_p + f_p = 2.0 + 2(2.32) + 2.0 = 8.64 \text{ in.} \quad \text{Eq. 1.4-7}$$

$$\delta_{11} = \frac{D_d^2}{3} (2w_p + 3f_p) = \frac{(1.5 \text{ in.})^2}{3} (2(2.32 \text{ in.}) + 3(2 \text{ in.})) = 7.98 \text{ in.}^3 \quad \text{Eq. 1.4-8}$$

$$\delta_{12} = \frac{\delta_{11}}{2} = \frac{7.98 \text{ in.}^3}{2} = 3.99 \text{ in.}^3 \quad \text{Eq. 1.4-9}$$

$$\begin{aligned} \delta_{22} &= \frac{1}{12} \left( \frac{D_d}{d} \right)^2 \left[ s(4e_p^2 - 2e_p f_p + f_p^2) + d^2 (3f_p + 2w_p) \right] \quad \text{Eq. 1.4-10} \\ &= \frac{1}{12} \left( \frac{1.5 \text{ in.}}{7.2 \text{ in.}} \right)^2 \left[ 8.64 \text{ in.} (4(1 \text{ in.})^2 - 2(1 \text{ in.})(2 \text{ in.}) + (2 \text{ in.})^2) + (7.2 \text{ in.})^2 (3(2 \text{ in.}) + 2(2.32 \text{ in.})) \right] \\ &= 2.12 \text{ in.}^3 \end{aligned}$$

Calculate spring constant indicators:

Insert the deflection indicators in Eqs. 1.4-11 through 1.4-20 to calculate the spring constant indicators:

$$\kappa_{t1} = \frac{1}{\delta_{22} - \frac{\delta_{12}}{2}} = \frac{1}{2.12 \text{ in.}^3 - \frac{3.99 \text{ in.}^3}{2}} = 8.00 \frac{1}{\text{in.}^3} \quad \text{Eq. 1.4-11}$$

$$\kappa_{t2} = \frac{1}{\left(\frac{2e}{f}\right)\left(\frac{\delta_{12}}{2}\right) + \delta_{22}} = \frac{1}{\left(\frac{2 \text{ in.}}{2 \text{ in.}}\right)\left(\frac{3.99 \text{ in.}^3}{2}\right) + 2.12 \text{ in.}^3} = 0.243 \frac{1}{\text{in.}^3} \quad \text{Eq. 1.4-12}$$

$$\kappa_{t3} = \frac{1}{\left(0.5 + \frac{2e}{f}\right)\delta_{12} + \delta_{22}} = \frac{1}{\left(0.5 + \frac{2 \text{ in.}}{2 \text{ in.}}\right)3.99 \text{ in.}^3 + 2.12 \text{ in.}^3} = 0.123 \frac{1}{\text{in.}^3} \quad \text{Eq. 1.4-13}$$

$$\kappa_{t4} = \frac{1}{\left(1 + \frac{3e}{f}\right)\delta_{12} + \delta_{22}} = \frac{1}{\left(1 + \frac{3(1 \text{ in.})}{2 \text{ in.}}\right)3.99 \text{ in.}^3 + 2.12 \text{ in.}^3} = 0.083 \frac{1}{\text{in.}^3} \quad \text{Eq. 1.4-14}$$

$$\kappa_{b2} = \frac{\frac{2e}{f}}{\frac{2e}{f} \frac{\delta_{11}}{2} + \delta_{12}} = \frac{\frac{2 \text{ in.}}{2 \text{ in.}}}{\frac{2 \text{ in.}}{2 \text{ in.}}\left(\frac{7.98 \text{ in.}^3}{2}\right) + 3.99 \text{ in.}^3} = 0.125 \frac{1}{\text{in.}^3} \quad \text{Eq. 1.4-15}$$

$$\kappa_{b3} = \frac{\frac{2e}{f}}{\left(0.5 + \frac{2e}{f}\right)\delta_{11} + \delta_{12}} = \frac{\frac{2 \text{ in.}}{2 \text{ in.}}}{\left(0.5 + \frac{2 \text{ in.}}{2 \text{ in.}}\right)7.98 \text{ in.}^3 + 3.99 \text{ in.}^3} = 0.063 \frac{1}{\text{in.}^3} \quad \text{Eq. 1.4-16}$$

$$\kappa_{b4} = \frac{\frac{2e}{f}}{\left(1 + \frac{3e}{f}\right)\delta_{11} + \delta_{12}} = \frac{\frac{2 \text{ in.}}{2 \text{ in.}}}{\left(1 + \frac{3(1 \text{ in.})}{2 \text{ in.}}\right)7.98 \text{ in.}^3 + 3.99 \text{ in.}^3} = 0.042 \frac{1}{\text{in.}^3} \quad \text{Eq. 1.4-17}$$

$$\begin{aligned} \kappa_{tc3} &= \frac{1}{\left(0.5 + \frac{2e}{f}\right)\delta_{11} + \delta_{22} + \frac{\delta_{12}}{2}} = \frac{1}{\left(0.5 + \frac{2 \text{ in.}}{2 \text{ in.}}\right)7.98 \text{ in.}^3 + 2.12 \text{ in.}^3 + \frac{3.99 \text{ in.}^3}{2}} \\ &= 0.062 \frac{1}{\text{in.}^3} \quad \text{Eq. 1.4-18} \end{aligned}$$

$$\begin{aligned} \kappa_{tc4} &= \frac{1}{\left(1 + \frac{3e}{f}\right)\delta_{11} + \delta_{22} + \left(1 + \frac{e}{f}\right)\delta_{12}} \\ &= \frac{1}{\left(1 + \frac{3(1 \text{ in.})}{2 \text{ in.}}\right)7.98 \text{ in.}^3 + 2.12 \text{ in.}^3 + \left(1 + \frac{1 \text{ in.}}{2 \text{ in.}}\right)3.99 \text{ in.}^3} = 0.036 \frac{1}{\text{in.}^3} \quad \text{Eq. 1.4-19} \end{aligned}$$

$$\kappa_{bc4} = \frac{\frac{2e}{f}}{\left(1 + \frac{4e}{f}\right)\delta_{11} + 2\delta_{12}} = \frac{\frac{2 \text{ in.}}{2 \text{ in.}}}{\left(1 + \frac{4(1 \text{ in.})}{2 \text{ in.}}\right)7.98 \text{ in.}^3 + 2(3.99 \text{ in.}^3)} = 0.031 \frac{1}{\text{in.}^3} \quad \text{Eq. 1.4-20}$$

Calculate lateral displacement indicators:

Insert the spring constant indicators in Eqs. 1.4-21 through 1.4-30 to determine the lateral displacement indicators for screw spacings = 1 through 4 flutes o.c.:

$$\delta_{t1} = \frac{24f}{\kappa_{t1}} \left[ \frac{\kappa_{t1}}{4f^2(f+w)} \right]^{0.25} = \frac{24(2 \text{ in.})}{8 \frac{1}{\text{in.}^3}} \left[ \frac{8 \frac{1}{\text{in.}^3}}{4(2 \text{ in.})^2(2 \text{ in.} + 2.19 \text{ in.})} \right]^{0.25} = 3.53 \text{ in.}^{2.5} \quad \text{Eq. 1.4-21}$$

$$\delta_{t2} = \frac{24f}{\kappa_{t2}} \left[ \frac{\kappa_{t2}}{4f^2(f+w)} \right]^{0.25} = \frac{24(2 \text{ in.})}{0.243 \frac{1}{\text{in.}^3}} \left[ \frac{0.243 \frac{1}{\text{in.}^3}}{4(2 \text{ in.})^2(2 \text{ in.} + 2.19 \text{ in.})} \right]^{0.25} = 48.5 \text{ in.}^{2.5} \quad \text{Eq. 1.4-22}$$

$$\delta_{t3} = \frac{24f}{\kappa_{t3}} \left[ \frac{\kappa_{t3}}{4f^2(f+w)} \right]^{0.25} = \frac{24(2 \text{ in.})}{0.123 \frac{1}{\text{in.}^3}} \left[ \frac{0.123 \frac{1}{\text{in.}^3}}{4(2 \text{ in.})^2(2 \text{ in.} + 2.19 \text{ in.})} \right]^{0.25} = 80.8 \text{ in.}^{2.5} \quad \text{Eq. 1.4-23}$$

$$\delta_{t4} = \frac{24f}{\kappa_{t4}} \left[ \frac{\kappa_{t4}}{4f^2(f+w)} \right]^{0.25} = \frac{24(2 \text{ in.})}{0.083 \frac{1}{\text{in.}^3}} \left[ \frac{0.083 \frac{1}{\text{in.}^3}}{4(2 \text{ in.})^2(2 \text{ in.} + 2.19 \text{ in.})} \right]^{0.25} = 108 \text{ in.}^{2.5} \quad \text{Eq. 1.4-24}$$

$$\delta_{b2} = \frac{48e}{\kappa_{b2}} \left[ \frac{\kappa_{b2}}{16e^2(2e+w)} \right]^{0.25} = \frac{48(1 \text{ in.})}{0.125 \frac{1}{\text{in.}^3}} \left[ \frac{0.125 \frac{1}{\text{in.}^3}}{16(1 \text{ in.})^2(2 \text{ in.} + 2.19 \text{ in.})} \right]^{0.25} = 79.8 \text{ in.}^{2.5} \quad \text{Eq. 1.4-25}$$

$$\delta_{b3} = \frac{48e}{\kappa_{b3}} \left[ \frac{\kappa_{b3}}{16e^2(2e+w)} \right]^{0.25} = \frac{48(1 \text{ in.})}{0.063 \frac{1}{\text{in.}^3}} \left[ \frac{0.063 \frac{1}{\text{in.}^3}}{16(1 \text{ in.})^2(2 \text{ in.} + 2.19 \text{ in.})} \right]^{0.25} = 133 \text{ in.}^{2.5} \quad \text{Eq. 1.4-26}$$

$$\delta_{b4} = \frac{48e}{\kappa_{b4}} \left[ \frac{\kappa_{b4}}{16e^2(2e+w)} \right]^{0.25} = \frac{48(1 \text{ in.})}{0.042 \frac{1}{\text{in.}^3}} \left[ \frac{0.042 \frac{1}{\text{in.}^3}}{16(1 \text{ in.})^2(2 \text{ in.} + 2.19 \text{ in.})} \right]^{0.25} = 181 \text{ in.}^{2.5} \quad \text{Eq. 1.4-27}$$

$$\delta_{tc3} = \frac{24f}{\kappa_{tc3}} \left[ \frac{\kappa_{tc3}}{4f^2(f+w)} \right]^{0.25} = \frac{24(2 \text{ in.})}{0.062 \frac{1}{\text{in.}^3}} \left[ \frac{0.062 \frac{1}{\text{in.}^3}}{4(2 \text{ in.})^2(2 \text{ in.} + 2.19 \text{ in.})} \right]^{0.25} = 135 \text{ in.}^{2.5} \quad \text{Eq. 1.4-28}$$

$$\delta_{tc4} = \frac{24f}{\kappa_{tc4}} \left[ \frac{\kappa_{tc4}}{4f^2(f+w)} \right]^{0.25} = \frac{24(2 \text{ in.})}{0.036 \frac{1}{\text{in.}^3}} \left[ \frac{0.036 \frac{1}{\text{in.}^3}}{4(2 \text{ in.})^2(2 \text{ in.} + 2.19 \text{ in.})} \right]^{0.25} = 203 \text{ in.}^{2.5}$$



Eq. 1.4-29

$$\delta_{bc4} = \frac{48e}{\kappa_{bc4}} \left[ \frac{\kappa_{bc4}}{16e^2(2e+w)} \right]^{0.25} = \frac{48(1\text{ in.})}{0.031 \frac{1}{\text{in.}^3}} \left[ \frac{0.031 \frac{1}{\text{in.}^3}}{16(1\text{ in.})^2(2\text{ in.} + 2.19\text{ in.})} \right]^{0.25} = 227 \text{ in.}^{2.5}$$

Eq. 1.4-30

Calculate final displacement indicators at top of corrugation:

Insert the lateral displacement indicators in Eqs. 1.4-31 through 1.4-34 to determine the final lateral displacement indicators for screw spacing = 1 through 4 flutes o.c.:

$$\gamma_1 = \delta_{t1} = 3.53 \text{ in.}^{2.5} \tag{Eq. 1.4-31}$$

$$\gamma_2 = 2 \delta_{t2} + \frac{2e}{f} \delta_{b2} = 2(48.5 \text{ in.}^{2.5}) + \frac{2 \text{ in.}}{2 \text{ in.}}(79.8 \text{ in.}^{2.5}) = 177 \text{ in.}^{2.5} \tag{Eq. 1.4-32}$$

$$\begin{aligned} \gamma_3 &= 2 \delta_{t3} + \delta_{tc3} + 2 \left( \frac{2e}{f} \right) \delta_{b3} \tag{Eq. 1.4-33} \\ &= 2(80.8 \text{ in.}^{2.5}) + 135 \text{ in.}^{2.5} + 2 \left( \frac{2 \text{ in.}}{2 \text{ in.}} \right) (133 \text{ in.}^{2.5}) = 563 \text{ in.}^{2.5} \end{aligned}$$

$$\begin{aligned} \gamma_4 &= 2 (\delta_{t4} + \delta_{tc4}) + \left( \frac{2e}{f} \right) (2 \delta_{b4} + \delta_{bc4}) \tag{Eq. 1.4-34} \\ &= 2(108 \text{ in.}^{2.5} + 203 \text{ in.}^{2.5}) + \left( \frac{2 \text{ in.}}{2 \text{ in.}} \right) (2(181 \text{ in.}^{2.5}) + 227 \text{ in.}^{2.5}) = 1210 \text{ in.}^{2.5} \end{aligned}$$

**Calculate warping value for screw spacing = 1 through 2 flutes o.c.:**

Insert the final displacement indicators to determine the value for warping where bottom flange screws are in every valley in Eq. 1.4-3 and where bottom flange screws are in every second valley in Eq. 1.4-4:

$$D_1 = \frac{\gamma_1 f}{d(t)^{1.5}} = \frac{3.53 \text{ in.}^{2.5}(2 \text{ in.})}{7.2 \text{ in.}(0.024 \text{ in.})^{1.5}} = 264 \text{ in.} \tag{Eq. 1.4-3}$$

$$U_1 = 1 \quad (\text{number of flutes with screw spacing} = 7.2 \text{ in.})$$

$$D_2 = \frac{\gamma_2 f}{2d(t)^{1.5}} = \frac{177 \text{ in.}^{2.5}(2 \text{ in.})}{2(7.2 \text{ in.})(0.024 \text{ in.})^{1.5}} = 6610 \text{ in.} \tag{Eq. 1.4-4}$$

$$U_2 = 4 \quad (\text{number of flutes with screw spacing} = 14.4 \text{ in.})$$

$$U_3 = 0 \quad \text{in this configuration} \quad U_4 = 0 \quad \text{in this configuration}$$

**Calculate weighted average warping value, D, for panel:**

$$D = \frac{1(264) + 4(6610) + 0 + 0}{1 + 4 + 0 + 0} = 5340 \text{ in.} \tag{Eq. 1.4-2}$$

**Calculate warping factor, D<sub>n</sub>:**

$$D_n = \frac{D}{L} = \frac{5340 \text{ in.}}{25 \text{ ft} \left( 12 \frac{\text{in.}}{\text{ft}} \right)} = 17.8 \quad (\text{Unit-less}) \quad \text{Eq. 1.4-1}$$

**Result warping factor:**  $D_n = 17.8$

$$\gamma_c = 0.71 \quad \text{Table 1.3-1}$$

**Calculate material shear strain contribution to diaphragm stiffness:**

The material shear strain contribution to the diaphragm stiffness is indicated by the first term in the denominator of Eq. D5.1.1-1.

**Note:** “Modified  $s$ ” for shear strain is not the same as “ $s$ ” for warping since different deformations and stiffnesses contribute. Warping is associated with element *flexural stiffness* while shear strain is associated with element *shear stiffness*.

$$s = 2e + 2w + f + (E_p + 2W_p + F_p) \left( \frac{1}{k} - 1 \right) \quad \text{Eq. D5.1.2-1}$$

**Note:** See Panel Data for  $W_p$ ,  $2e$ ,  $w$ ,  $f$ .  $E_p$  and  $F_p = 0$  (no perforations).  
 $k = 0.524$  - See Eq. 1.6-5; calculation for  $k$  is at  $S_{nb}$  in Example 2a.

$$s = 2 \text{ in.} + 2(2.19 \text{ in.}) + 2 \text{ in.} + (0 + 2 \times 1.313 \text{ in.} + 0) \left( \frac{1}{0.524} - 1 \right) = 10.77 \text{ in.}$$

$\mu = 0.3$  (Poisson’s ratio for steel)

### Example 2d Result

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

$$G' = \left( \frac{29500 \frac{k}{\text{in.}^2} (0.024) \text{ in.}}{2(1+0.3) \frac{10.77 \text{ in.}}{7.2 \text{ in.}} + 0.71(17.8) + 5.31} \right) 0.433 = 14.0 \frac{k}{\text{in.}}$$

### Impact of perforations:

If the panel webs are not perforated, ( $p_o = 0$ ,  $k = 1$ ,  $s = 8.38$  in.), then by similar process:

$D_n = 17.4$  or 2% less warping

$$G' = \left( \frac{29500 \frac{k}{\text{in.}^2} (0.024) \text{ in.}}{2(1+0.3) \frac{8.38 \text{ in.}}{7.2 \text{ in.}} + 0.71(17.4) + 5.31} \right) 0.433 = 14.8 \frac{k}{\text{in.}}$$

### Result:

“No perforation” case provides 6% greater *diaphragm stiffness*. The impact of web

perforations is present but not too significant. Shear strain across the perforated zone (29% increase - 10.77/8.38) contributes to the increased diaphragm deflection. Warping increase is negligible in 25 ft long panels.

### Impact of panel length and number of spans:

If the manufacturer chooses to print three span tables so  $L = 15$  ft, then for the perforated panel:

$$D_n = \frac{5340 \text{ in.}}{15 \text{ ft}(12) \frac{\text{in.}}{\text{ft}}} = 29.7 \quad \text{Eq. 1.4-1}$$

$$\gamma_c = 0.9 \quad \text{Table 1.2-1}$$

$$G' = \left( \frac{29500 \frac{\text{k}}{\text{in.}^2} (0.024) \text{ in.}}{2(1+0.3) \frac{10.77}{7.2} + 0.9(29.7) + 5.31} \right) 0.433 = 8.53 \frac{\text{k}}{\text{in.}} \quad \begin{array}{l} \text{(less stiffness)} \\ \text{39\% decrease} \\ \text{vs.} \\ \text{14.0 k/in.} \end{array}$$

### Result:

In this case, the three span table is very conservative when applied to a 25 ft *panel* since warping is dominant at 18 ft, and the 25 ft *panel* exhibits proportionately less warping.

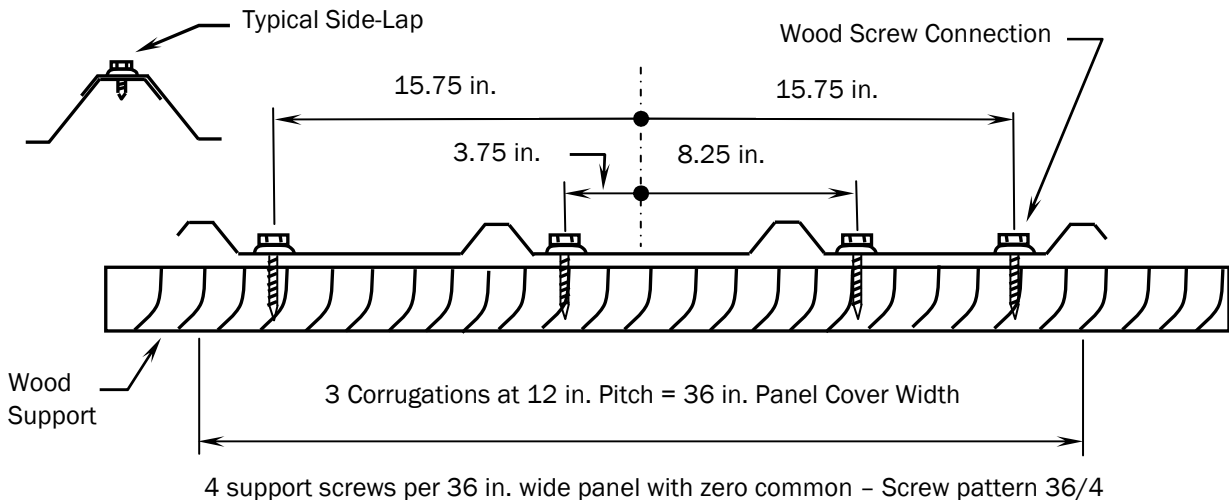
### Example 3a: Nominal and Available Diaphragm Shear Strength and Stiffness of a Panel on Wood Supports in the Absence of Uplift

#### Objective

Calculate the *nominal diaphragm shear strength* per unit length,  $S_n$ , *available strength*, and *stiffness*,  $G'$ , of a *panel* on wood supports in the absence of uplift. Compare with the impact of uplift in Example 3b.

**Note:** *Panel flexural strength* and deflection check is left to the designer and should be completed for all load combinations prior to the *diaphragm* design.

#### Diaphragm Configuration



**Note:** Application is an exposed wall or roof.

#### Panel Data

See Figure D2.1-1 for definitions of *panel* parameters.

Yield stress, $F_y$	= 80 ksi	Modulus of Elasticity, $E$	= 29500 ksi
Tensile strength, $F_u$	= 82 ksi	Panel length, $L$	= 30.0 ft
Depth, $D_d$	= 1.25 in.	Cover width, $w$	= 36.0 in.
Thickness, $t$	= 0.024 in.	Pitch, $d$	= 12.00 in.
Top flat width, $f$	= 1.00 in.	Web flat width, $w$	= 1.60 in.
Bottom flat width, $2e$	= 9.00 in.		
Moment of Inertia, $I_{xg}$	= 0.051 in. <sup>4</sup> /ft	This is $I_x$ value from manufacturer and conservatively used for $I_{xg}$ .	

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

**Note:** Chapter D limits (a) through (d) are satisfied, so calculate using Chapter D. Material is based on ASTM 653 SS Grade 80 G90 Painted. AISI S100 Section A2.3.3 design requirements apply. **Use:**  $F_y = 60$  ksi  $F_u = 62$  ksi.

### Wood Support Data

Species = Douglas Fir – South  $G = 0.45$   
 Spacing,  $L_v$  (shear span) = 5.00 ft Seasoned Wood and Application Dry

**Note:** See Table D1.1.4.2-2 for wood-specific gravity,  $G$ . Assume wood edge dimensions satisfy AWC NDS requirements, and depth will be enough for the screw thread length. Designer should verify the wood dimensions. Proof is not part of this example.

### Connection Schedule

**Support connection:** Pattern = 36/4 – See Figure above  
 #14 screw  $\times$  2 in. long with 5/8 in. round sealing washer  $P_{nss} = 2.7$  k  
 $d = 0.248$  in.  $P_{nts} = 3.2$  k

**Side-lap connection:** Spacing between supports = 20 in. o.c.  
 #12 screw  $\times$  1 in. long with 1/2 in. round sealing washer  $P_{nss} = 2.0$  k  
 $d = 0.216$  in.  $P_{nts} = 2.6$  k

**Note:** *Side-lap connections* will be located over supports but are not into supports.  
 $P_{nss}$  = Breaking *nominal shear strength* of screw  
 $P_{nts}$  = Breaking *nominal tensile strength* of screw

A large *side-lap* spacing is to illustrate the impact of *support connection strength*. For service reasons, *side-lap* spacing often varies between 12 in. and 24 in. o.c. at roofs, and spacing of 36 in. o.c. (and sometimes greater) is used at walls. Consult the manufacturer's installation guidelines.

Corrosion resistance should be considered when selecting wood support screws and *panels* over wood. Consult the wood truss provider, screw manufacturer, and the *panel* manufacturer.

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

*Safety and resistance factors* are in Section B and Section D1.1.4.1.

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

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

**Note:**  $S_{nf}$  is in Section D1;  $S_{nb}$  is in Section D2. Since an edge detail is designed in Example 3d, *strength* controlled by *edge panel*,  $S_{ne}$ , will not be considered in this example. *Stiffness*,  $G'$ , is calculated in this example and is required for serviceability checks. Uplift is in Example 3b (shear and tension interaction). Impact of insulation between *panel* and support is in Example 3c.

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

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

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

**Calculate support connection strength,  $P_{nf}$  (Section D1.1.4.2):**

$$h_s = 2 - 0.125 \text{ in.} = 1.87 \text{ in.}$$

**Note:**  $h_s$  = threaded length of screw including tapered tip that penetrates the wood. 0.125 in. is an allowance for washer and panel thickness.

$$4d = 4(0.248 \text{ in.}) = 0.992 \text{ in.} \quad \text{See Connection Schedule}$$

$$7d = 7(0.248 \text{ in.}) = 1.74 \text{ in.}$$

$h_s = 1.87 \text{ in.} > 7d = 1.74 \text{ in.}$  Also see User Note at Section D1.1.4.2 for minimum embedment length.

For  $4d \leq h_s < 7d$

$$P_{nf} = \min \left( \frac{h_s}{7d} P_{nfw}, P_{nfws}, P_{nss} \right) \quad \text{Eq. D1.1.4.2-1}$$

For  $h_s \geq 7d$

$$P_{nf} = \min(P_{nfw}, P_{nfws}, P_{nss}) \quad \text{Eq. D1.1.4.2-2}$$

**Note:** The above equation indicates that additional length will not increase  $P_{nf}$ .

$$P_{nfw} = 1.97G = 1.97(0.45) = 0.887 \text{ k} \quad \text{Table D1.1.4.2-1}$$

$$P_{nfws} = 2.7t_1 d F_{u1} \quad \text{Table D1.1.4.2-1}$$

$$= 2.7(0.024 \text{ in.})(0.248 \text{ in.}) \left( 62 \frac{\text{k}}{\text{in}^2} \right) = 0.996 \text{ k}$$

$$P_{nss} = 2.7 \text{ k} \quad \text{See Connection Schedule}$$

$$\text{Result: } P_{nf} = \min(0.887 \text{ k}, 0.996 \text{ k}, 2.70 \text{ k}) = 0.887 \text{ k} \quad \text{Eq. D1.1.4.2-2}$$

Screw bearing against wood controls the *nominal strength*.

**Calculate side-lap connection strength,  $P_{ns}$  (Section D1.2.5 or AISI S100 E4.3.1):**

$$\frac{t_2}{t_1} = \frac{0.024 \text{ in.}}{0.024 \text{ in.}} = 1.0 \quad \text{Therefore } \frac{t_2}{t_1} \leq 1.0 \quad F_{u2} = F_{u1}$$

$$P_{ns} = 4.2(t_2^3 d)^{1/2} F_{u2} \quad \text{AISI S100 Eq. E4.3.1-1}$$

$$= 4.2 \left[ (0.024 \text{ in.})^3 (0.216 \text{ in.}) \right]^{1/2} \left( 62 \frac{\text{k}}{\text{in}^2} \right) = 0.450 \text{ k}$$

$$P_{ns} = 2.7t_1 d F_{u1} \quad \text{AISI S100 Eq. E4.3.1-2}$$

$$= 2.7(0.024 \text{ in.})(0.216 \text{ in.})\left(62 \frac{\text{k}}{\text{in.}^2}\right) = 0.868 \text{ k}$$

$$P_{\text{NSS}} = 2.0 \text{ k} \quad \text{See Connection Schedule}$$

$$\text{Result: } P_{\text{NS}} = \min(0.450 \text{ k}, 0.868 \text{ k}, 2.00 \text{ k}) = 0.450 \text{ k}$$

Tilting of screw in *panel* controls the *nominal strength*.

### Calculate configuration parameters required for $S_{\text{nf}}$ :

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

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

$$= 1 - \frac{1.25 \text{ in.}(5.0 \text{ ft})}{240\sqrt{0.024 \text{ in.}}} = 0.832 > 0.7 \text{ OK Unit-less}$$

$$N = \frac{4 \text{ screws}}{3 \text{ ft}} = 1.33 \frac{\text{screws}}{\text{ft}} \quad \text{Number of screws into support per ft along } \textit{panel} \text{ ends}$$

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

$$n_p = \frac{L}{L_v} - 1 = \frac{30 \text{ ft}}{5 \text{ ft}} - 1 = 5 \quad \text{Number of } \textit{interior supports} \text{ along } L \quad \text{Eq. D1-9}$$

$$n_s = \frac{30 \text{ ft}(12 \frac{\text{in.}}{\text{ft}})}{20 \frac{\text{in.}}{\text{conn.}}} + 1.0 = 19 \quad \text{Number of } \textit{side-lap connections} \text{ along the } \textit{panel} \text{ length, } L$$

$$\alpha_s = \frac{P_{\text{NS}}}{P_{\text{NF}}} = \frac{0.450 \text{ k}}{0.887 \text{ k}} = 0.507 \quad \text{Connection strength ratio} \quad \text{Eq. D1-6}$$

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

$$x_{e1} = 3.75 \text{ in.} \quad x_{e3} = 8.25 \text{ in.} \quad x_{e2} = x_{e4} = 15.75 \text{ in.} \quad \text{See Configuration}$$

$$= \left(\frac{1}{(36 \text{ in.})^2}\right) \left( (3.75 \text{ in.})^2 + 2(15.75 \text{ in.})^2 + (8.25 \text{ in.})^2 \right) = 0.446$$

$$\beta = 19(0.507) + 2(5)(0.446) + 4(0.446) = 15.9 \quad \text{Eq. D1-5}$$

### Calculate Nominal Diaphragm Shear Strength, $S_{\text{nf}}$ , With Butt-Joint

**Note:** This applies to a single run of 30 ft *panels* or cases without end-laps. All *support connection shear strength* is associated with one thickness of panel and  $P_{\text{nf}}$  equals 0.887 k. End-lap strength and alternate shear planes will be considered later.

$$S_{\text{ni}} = [2A(\lambda - 1) + \beta] \frac{P_{\text{NF}}}{L} = [(2)(0.0)(0.832 - 1) + 15.9] \frac{0.887 \text{ k}}{30 \text{ ft}} = 0.470 \text{ klf} \quad \text{Eq. D1-1}$$

$$S_{\text{nc}} = \left(\frac{N^2 \beta^2}{L^2 N^2 + \beta^2}\right)^{0.5} P_{\text{NF}} = \left(\frac{(1.33)^2 (15.9)^2 \frac{1}{\text{ft}^2}}{(30)^2 (1.33)^2 \frac{\text{ft}^2}{\text{ft}^2} + (15.9)^2}\right)^{0.5} 0.887 \text{ k} = 0.437 \text{ klf} \quad \text{Eq. D1-2}$$

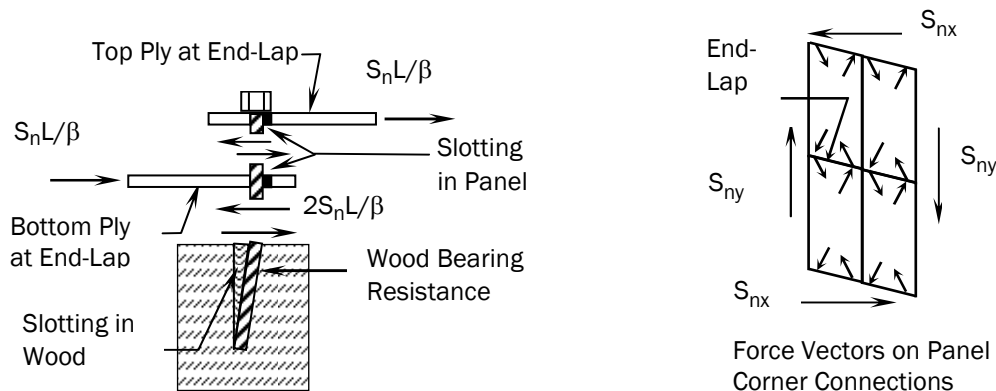
**Result at Butt-Joint (Section D1):**

$$S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.470 \text{ klf}, 0.437 \text{ klf}) = 0.437 \text{ klf}$$

Based on  $P_{nf} = 0.887 \text{ k}$ .  $S_{ne}$  is determined in Example 3d.

**Calculate Nominal Diaphragm Shear Strength,  $S_{nf}$ , With End-Lap**

**Note:** Consider the shear transfer at the end-lap. Since  $P_{nf}$  is controlled by screw bearing against wood, the force in each steel ply cannot develop the full bearing strength,  $P_{nf}$ , at the end-lap. See the sketch below. In determining  $S_{ni}$ , the *connection* force component in each ply parallel with the *panel* span is added to create the force in the screw shank that is embedded in the wood. At the corners of the end-lap *panel*, the maximum force occurs in each ply and is limited to one-half the strength in the wood support. At *interior supports* there is one ply and *connection* strength is  $P_{nf}$ . A more detailed discussion is included in Example 2a.



**Note:** See theory adjustment in Example 1a for variation of *support connection shear strength* along a panel length,  $L$  - Modified Eqs. D1-1 and D1-2.

Let:  $P_{nfe} = 0.5 P_{nf}$  at *exterior supports* in one ply

$P_{nfi} = P_{nf}$  at *interior supports* in one ply

**Calculate  $S_{ni}$  based on  $P_{nfi} = 0.887 \text{ k}$ :**

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

$$\beta = n_s \alpha_s + 2n_p \alpha_p^2 + 4\alpha_e^2 \frac{P_{nfe}}{P_{nfi}} \quad \text{Modified Eq. D1-5}$$

$$\frac{P_{nfe}}{P_{nfi}} = 0.5 \quad \text{See free body sketch}$$

See butt-joint calculation for parameters:  $\alpha_s, \alpha_p^2, \alpha_e^2, n_s, n_p$

$$\beta = 19(0.507) + 2(5)(0.446) + 4(0.446)(0.50) = 15.0 \text{ unit-less}$$

$$S_{ni} = \left[ 2(0.0)(0.832 - 1)(0.5) + 15.0 \right] \frac{0.887 \text{ kips}}{30 \text{ ft}} = 0.444 \text{ klf}$$

5.5% less than  $S_{ni} = 0.470 \text{ klf}$  with butt-joint.



**Note:**  $S_{nc}$  considers both orthogonal force components at a corner connection in each *panel*. At the two-ply end-lap, the opposing components in each *panel* that are parallel with the wood support will cancel in the screw shank so  $S_{nc}$  will not be limited by wood bearing. Instead, steel bearing in one ply will control since both components are present. The additive longitudinal limitation is in  $S_{ni}$ . For the end-lap case,  $S_{nc}$  will be based on  $P_{nfe} = 0.996$  k at the end-lap per ply and 0.887 k at interior supports. Many engineers would neglect this refinement. In the case of five spans, the benefit is a little academic, but it is there.

**Calculate  $S_{nc}$  based on  $P_{nfe} = 0.996$  k:**

$$\beta = n_s \alpha_s + 2n_p \alpha_p^2 \frac{P_{nfi}}{P_{nfe}} + 4\alpha_e^2 \quad \text{Modified Eq. D1-5}$$

$$\alpha_s = \frac{P_{ns}}{P_{nf}} = \frac{0.450 \text{ kips}}{0.996 \text{ kips}} = 0.452 \quad \text{Eq. D1-6}$$

$$\beta = 19(0.452) + 2(5)(0.446) \frac{0.887}{0.996} + (4)(0.446) = 14.3$$

$$S_{nc} = \left( \frac{N^2 \beta^2}{L^2 N^2 + \beta^2} \right)^{0.5} = \left( \frac{(1.33)^2 (14.3)^2 \frac{1}{\text{ft}^2}}{(30)^2 (1.33)^2 \frac{\text{ft}^2}{\text{ft}^2} + (14.3)^2} \right)^{0.5} 0.996 \text{ k} = 0.447 \text{ klf}$$

2% greater than  $S_{nc} = 0.437$  klf with butt-joint.

**Result at end-lap (Section D1):**

$$S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.444, 0.447) = 0.444 \text{ klf}$$

**Note:** The butt-joint *diaphragm strength* (using a single  $P_{nf} = 0.887$  k) is 0.437 klf. The end-lap refinement provides a 2% increase in this case. The controlling limit state changed from  $S_{nc}$  to  $S_{ni}$ . The case of one end with end-lap and one without will be between these values. The result would be different at three spans and with fewer side-lap screws, but many manufacturers publish tables based on the butt-joint case and this is a rational approach in this example.

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

$$\alpha = 1 \quad \text{Conversion factor for U.S. customary units}$$

$$s = 2e + 2w + f = 9 \text{ in.} + 2(1.6 \text{ in.}) + 1 \text{ in.} = 13.2 \text{ in.}$$

$$\text{Eq. D2.1-2}$$

$$S_{nb} = \frac{7890}{(1)(5 \text{ ft})^2} \left( \frac{(0.051 \text{ in.}^4/\text{ft})^3 (0.024 \text{ in.})^3 (12 \text{ in.})}{13.2 \text{ in.}} \right)^{0.25} = 2.02 \text{ klf}$$

**Result:**  $S_{nb} = 2.02 \text{ klf}$

$$S_{nf} = 0.444 \text{ klf} < S_{nb} = 2.02 \text{ klf}$$

**Result at end-lap installations (No uplift):**

$$\text{Nominal Strength: } S_n = 0.444 \text{ klf}$$

**Result: Available diaphragm strength per unit length (with end-lap):**

*Safety and resistance factors are in Section B1 (AISI S100 Table D5) and Section D1.1.4.1:*

**ASD**

$$\frac{S_n}{\Omega} = \min\left(\frac{S_{nf}}{\Omega_{df}}, \frac{S_{nb}}{\Omega_{db}}\right)$$

$$\frac{S_n}{\Omega} = \min\left(\frac{0.444}{3}, \frac{2.02}{2}\right) = 0.148 \text{ klf}$$

**LRFD**

$$\phi S_n = \min(\phi_{df} S_{nf}, \phi_{db} S_{nb})$$

$$\phi S_n = \min(0.55(0.444), 0.8(2.02)) = 0.244 \text{ klf}$$

**LSD**

$$\phi S_n = \min(\phi_{df} S_{nf}, \phi_{db} S_{nb})$$

$$\phi S_n = \min(0.50(0.444), 0.75(2.02)) = 0.222 \text{ klf}$$

**Calculate Diaphragm Stiffness,  $G'$ , Using Sections D5.1.1 and D5.2**

**Note:** Calculation is the same for end-lap or butt joint conditions.

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

$K = 0.5$	For <i>panel</i> on wood supports	$E = 29500 \text{ ksi}$
$s = 13.2 \text{ in.}$	From determination of $S_{nb}$	$\mu = 0.3$ (Poisson's ratio for steel)
$d = 12 \text{ in.}$	From the Panel Data	$t = 0.024 \text{ in.}$

**Calculate warping factor,  $D_n$ , using Section 1.4 (Appendix 1):**

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

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

This example only requires  $D_1$  - See *Diaphragm Configuration* and *Panel Data*

$$U_1 = 3 \quad U_2 = 0 \quad U_3 = 0 \quad U_4 = 0$$

**Note:**  $D_1$  is calculated using the detailed method in Example 2d. The designer should verify  $\kappa_{t1}$  and expect some round-off variation:

$$D_1 = \frac{\gamma_1 f}{d(t)^{1.5}} \quad \text{Eq. 1.4-3}$$

$$\kappa_{t1} = 1.15 \frac{1}{\text{in.}^3} \quad \text{Eq. 1.4-11}$$

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

$$= \frac{24(1.0 \text{ in.})}{1.15 \frac{1}{\text{in.}^3}} \left[ \frac{1.15 \frac{1}{\text{in.}^3}}{4(1 \text{ in.})^2 (1 \text{ in.} + 1.6 \text{ in.})} \right]^{0.25} = 12.0 \text{ in.}^{2.5}$$

$$D_1 = \frac{(12.0 \text{ in.}^{2.5})(1 \text{ in.})}{(12 \text{ in.})(0.024 \text{ in.})^{1.5}} = 269 \text{ in.} \quad \text{Eq. 1.4-3}$$

$$D = \frac{3(269 \text{ in.}) + 0 + 0 + 0}{3 + 0 + 0 + 0} = 269 \text{ in.} \quad \text{Eq. 1.4-2}$$

**Result:**

$$D_n = \frac{269 \text{ in.}}{(30 \text{ ft}) \frac{12 \text{ in.}}{\text{ft}}} = 0.747 \quad \text{Eq. 1.4-1}$$

$$\gamma_c = 0.64 \quad \text{Table 1.3-1}$$

**Calculate slip constant, C:**

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

$$\alpha_3 = \alpha_4 = \frac{\sum x_e}{w} \quad \text{See Figure D1-1 for } x_e \text{ and Example 3a Configuration} \quad \text{Eq. D5.1.1-3}$$

$$x_{e1} = 3.75 \text{ in.} \quad x_{e3} = 8.25 \text{ in.} \quad x_{e2} = x_{e4} = 15.75 \text{ in.}$$

$$\alpha_3 = \alpha_4 = \frac{3.75 + 8.25 + 2(15.75)}{36} \frac{\text{in.}}{\text{in.}} = 1.21$$

**Note:**  $x_e$  and  $w$  are used for both *interior panels* and *edge panels*, and  $G'$  should be evaluated with the dominant *interior panels*.

**Calculate screw flexibilities:**

$$S_f = \frac{1.5\alpha}{1000\sqrt{t}} = \frac{1.5(1)}{1000\sqrt{0.024 \text{ in.}}} = 0.00968 \frac{\text{in.}}{\text{kip}} \quad \text{Eq. D5.2.3-1}$$

$$S_s = \frac{3.0\alpha}{1000\sqrt{t}} = \frac{3.0(1)}{1000\sqrt{0.024 \text{ in.}}} = 0.0194 \frac{\text{in.}}{\text{kip}} \quad \text{Eq. D5.2.2-2}$$

**Result of slip constant, C:**

$$C = \left( \frac{29500 \frac{\text{k}}{\text{in.}^2} (0.024 \text{ in.})}{36 \text{ in.}} \right) \left( \frac{2(30 \text{ ft}) \frac{12 \text{ in.}}{\text{ft}}}{2(1.21) + 5(1.21) + 2(19) \frac{0.00968 \frac{\text{in.}}{\text{k}}}{0.0194 \frac{\text{in.}}{\text{k}}}} \right) 0.00968 \frac{\text{in.}}{\text{k}} = 5.00$$

**Result of diaphragm stiffness, G':**

$$G' = \left( \frac{29500 \frac{\text{k}}{\text{in.}^2} (0.024 \text{ in.})}{2(1 + 0.3) \frac{13.2 \text{ in.}}{12 \text{ in.}} + 0.64(0.747) + 5.00} \right) 0.5 = 42.5 \frac{\text{k}}{\text{in.}} \quad \text{Eq. D5.1.1-1}$$

**Note:** The denominator of all three components of  $G'$  is 8.34. Warping is represented by  $0.64(0.747) = 0.478$ . In this case, warping represents 6% of the *diaphragm* deflection ( $0.478/8.34$ ). Slip at connections is dominant – 60% of the *diaphragm* deflection ( $5/8.34$ ). Shear strain represents 34% ( $2.86/8.34$ ).

**Example 3a Result (No Uplift)****Nominal diaphragm strength per unit length:**

$$S_n = 0.437 \text{ klf}$$

Controlled by bearing of screw against wood at *interior supports*.

Butt-joint model is chosen for a usable and slightly conservative answer.

2 in. long support screws work.

**Note:** Required screw length changes in Example 3b to resist uplift.

**Available diaphragm strength per unit length:**

$$\begin{aligned} S_{\text{available}} &= 0.437 \text{ klf} / 3.00 = 0.146 \text{ klf} \quad (\text{ASD}) \\ &= (0.55)(0.437 \text{ klf}) = 0.240 \text{ klf} \quad (\text{LRFD}) \\ &= (0.50)(0.437 \text{ klf}) = 0.219 \text{ klf} \quad (\text{LSD}) \end{aligned}$$

**Diaphragm stiffness:**

$$G' = 42.5 \frac{\text{k}}{\text{in.}}$$

### Example 3b: Nominal Diaphragm Shear Strength of the Example 3a Configuration Subjected to an Uplift Load

#### Objective

Use the *configuration* of Example 3a and calculate the *nominal diaphragm shear strength* per unit length,  $S_n$ , in the presence of a large uplift load. Compare with no uplift.

**Note:** Section D3.1.2.2 will be used to determine  $P_{nft}$  and  $S_n$ .  
End-laps are at the ends of 30 ft *panels*.

#### Load Data

Wind uplift zone	= 50 psf (0.05 ksf)	
Reaction at <i>interior support</i>	= 1.1(5 ft)(0.05 ksf)	
	= 0.275 klf	Pull-over and pull-out
Reaction at <i>exterior support</i>	= 0.4(5 ft)(0.05 ksf)	
	= 0.100 klf	Pull-over for one thickness
Reaction at end-lap	= 2(0.100 klf)	
	= 0.200 klf	Pull-out at end-lap

**Note:** Tributary length doubles for pull-out at end-lap - two plies.  
Neglect dead load of panel in wind *load combination* (would reduce force on screw).  
1.1 and 0.4 are rational coefficients for multiple-span applications subject to uniform loads. More precise six-6 span values are available, and some engineers use 1.0 and 0.5.

#### Design Method: ASD Selected (Eq. C2-1)

Calculate  $S_{nf}$  ( $S_{nf}$  is the least calculated at cases (a), (b), and (c) with the corresponding  $P_{nft}$ ):

(a) Interaction of shear and pull-over (AISI S100 Section E4.5.1.1)

Calculate  $P_{nft}$

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

$\Omega = 2.35$       $\Omega_d = 3.00$  (Wood)     (Section D1.1.4.1)

**Note:** The given limits in AISI S100 Section E4.5.1.1 are slightly exceeded. This example, however, will apply the design provisions for illustrative purposes. Since the pull-over and shear interaction considers failure in steel,  $\Omega_d = 2.35$  could be used to determine *available strength* at this limit. The conservative wood system  $\Omega_d = 3.00$  will be used in the example.

At interior supports:

$$N = 1.33 \frac{\text{screws}}{\text{ft}} \quad \text{See Example 3a}$$

$$T = \frac{0.275 \text{ k/ft}}{1.33 \text{ screws/ft}} = 0.207 \text{ k/screw} \quad \text{See Load Data}$$

$$P_{\text{nov}} = 1.5t_1d'_wF_{u1} \quad \text{AISI S100 Eq. E4.4.2-1}$$

**Note:** See AISI *Cold-Formed Steel Design Manual*, 2013 Edition, Table IV-11a for screw head diameter,  $d_h$ .

$$d_h = 0.409 \text{ in. (screw head diameter for \#14 support screw)}$$

$$t_w = 0.040 \text{ in. (washer thickness estimate)}$$

$$d'_w = d_h + 2t_w + t_1 \leq d_w \quad \text{AISI S100 Eq. E4.4.2-2}$$

$$= 0.409 \text{ in.} + 2(0.04 \text{ in.}) + 0.024 \text{ in.} = 0.513 \text{ in.} < 0.625 \text{ in.} \quad \text{OK}$$

For domed washer,  $d'_w < 3/4 \text{ in.}$  per AISI S100 Section E4.4.2.

For one-panel thickness and uniform pull-over,

$$P_{\text{nov}} = 1.5(0.024 \text{ in.})(0.513 \text{ in.}) \left( 62 \frac{\text{k}}{\text{in.}^2} \right) = 1.15 \text{ k} \quad \text{AISI S100 Eq. E4.4.2-1}$$

$$\left( \frac{P_{\text{nft}}}{P_{\text{nf}}} \right) = \frac{1.1(3.00)}{2.35} - \left( \frac{0.71(0.207 \text{ k})(3.00)}{1.15 \text{ k}} \right) \leq 1.0 \quad \text{Rewriting Eq. D3.1.2.1-2}$$

$$= 1.02 > 1.0, \text{ Therefore, } 1.0 \text{ is used.}$$

$$P_{\text{nf}} = \min(0.996 \text{ k}, 2.70 \text{ k}) = 0.996 \text{ k} \quad \text{from Example 3a}$$

**Note:** See Section D3.1.2.2. Pull-over interaction relates to steel, so wood bearing is excluded as a limit on  $P_{\text{nf}}$ . However, wood bearing is included at pull-out interaction - See (b) below.

**Result of  $P_{\text{nft}}$  at interior support controlled by pull-over:**

$$P_{\text{nft}} = 1.0(0.996 \text{ k}) = 0.996 \text{ k} \quad \text{no reduction}$$

At exterior supports:

$$T = \frac{0.100 \text{ k/ft}}{1.33 \text{ screws/ft}} = 0.075 \text{ k/screw} \quad \text{See Load Data}$$

For one-panel thickness and eccentric pull-over,

$$P_{\text{nov}} = 0.5(1.15 \text{ k}) = 0.575 \text{ k} \quad \text{AISI S100 Section E4.5.1.1}$$

$$\left( \frac{P_{\text{nft}}}{P_{\text{nf}}} \right) = \frac{1.1(3.00)}{2.35} - \left( \frac{0.71(0.075 \text{ k})(3.00)}{0.5(1.15) \text{ k}} \right) \leq 1.0 \quad \text{Rewriting Eq. D3.1.2.1-2}$$

$$= 1.13 > 1.0 \text{ Therefore, } 1.0 \text{ is used.}$$

$$P_{\text{nf}} = \min(0.996 \text{ k}, 2.70 \text{ k}) = 0.996 \text{ k} \quad \text{from Example 3a}$$

Result of  $P_{nft}$  at *exterior support* controlled by pull-over:

$$P_{nft} = 1.0(0.996 \text{ k}) = 0.996 \text{ k} \quad \text{no reduction}$$

Calculate  $S_{nf}$  controlled by pull-over:

Since  $P_{nft}$  has not been reduced in considering the pull-over and shear interaction, the *nominal diaphragm strength* will be the same as the case without the uplift:

$$S_{ni} = 0.444 \text{ klf} \quad (\text{based on } P_{nf} = 0.887 \text{ k}) \quad S_{nc} = 0.447 \text{ klf} \quad (\text{based on } P_{nf} = 0.996 \text{ k})$$

Result of Case (a):  $S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.444 \text{ k}, 0.447 \text{ k}) = 0.444 \text{ klf}$

**Note:** The Case (a) results indicate that for a relatively short span and multiple support connections as shown in this example, the pull-over strength over wood support does not reduce the *diaphragm strength* at 50 psf uplift.

### (b) Interaction of shear and pull-out (Section D3.1.2.2)

Calculate  $P_{nft}$ :

$$\frac{P_{nft}}{P_{nfw}} = \frac{\cos\theta}{\left[\cos^2\theta\right] + \left[\frac{P'_{nfw}}{P_{nT}} \sin^2\theta\right]} \quad \text{Eq. D3.1.2.2-1a}$$

$$P'_{nfw} = P_{nfw} \quad \text{For } h_s \geq 7d \quad \text{Eq. D3.1.2.2-6}$$

$$P_{nfw} = 0.887 \text{ k} \quad \text{From Example 3a}$$

$$h_s = 1.87 \text{ in.} \geq 7d = 1.74 \text{ in.} \quad \text{From Example 3a (for 2 in. long screw)}$$

$$P'_{nfw} = 0.887 \text{ k} \quad \text{Will not increase for greater } h_s \quad \text{Eq. D3.1.2.2-6}$$

$$P_{nT} = 6.16\alpha G^2 d h_s \quad \text{Will increase for greater } h_s \quad \text{Eq. D3.1.2.2-7}$$

$$= 6.16(1)(0.45)^2 (0.248 \text{ in.})(1.87 \text{ in.}) = 0.578 \text{ k/screw}$$

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

**Note:** Without  $T$ ,  $P_{nf} = 0.887 \text{ k}$ . The maximum shear force,  $V_{max}$ , in the screw cannot exceed the allowable strength of the screw, i.e.:

$$\Omega_d = 3.00$$

$$V_{max} \leq \frac{P_{nf}}{\Omega_d} = \frac{0.887}{3} = 0.296 \text{ k/screw} \quad \text{ASD}$$

S100 Section E4.5.2.1 does not require eccentricity reduction for pull-out.

At *interior supports*:

$$T = \frac{0.275 \text{ k/ft}}{1.33 \text{ screws/ft}} = 0.207 \text{ k/screw} \quad \text{See Load Data}$$

Pull-out *available tension strength* (no shear):

$$\frac{P_{nT}}{\Omega} = \frac{0.578 \text{ k}}{3} = 0.193 \text{ k} < 0.207 \text{ k} \quad \text{2 in. long screw NG}$$

Need more or longer screws just to resist tension load.

Try 2-½ in. long screw but same number.

$$h_s = 2.5 - 0.125 \text{ in.} = 2.37 \text{ in.} \quad \text{See } h_s \text{ note in Example 3a}$$

$$P_{nT} = 6.16(1)(0.45)^2(0.248 \text{ in.})(2.37 \text{ in.}) = 0.733 \text{ k/screw} \quad \text{Eq. D3.1.2.2-7}$$

Pull-out *available tension strength* (no shear):

$$\frac{P_{nT}}{\Omega} = \frac{0.733 \text{ k}}{3} = 0.244 \text{ k} > 0.207 \text{ k} \quad 2\text{-}1/2 \text{ in. long screw OK}$$

To determine  $P_{nft}$  from Eq. D3.1.2.2-1a, iteration is needed. Assume  $P_{nft} = 0.475 \text{ k}$ :

$$\text{Let } V = \frac{P_{nft}}{\Omega} = \frac{0.475 \text{ k}}{3} = 0.158 \frac{\text{k}}{\text{screw}} \quad \text{Required shear strength in presence of } T = 0.207 \text{ k}$$

$$\frac{P_{nft}}{P'_{nfw}} = \frac{0.475 \text{ k}}{0.887 \text{ k}} = 0.536 \quad \text{Based on assumption}$$

$$\theta = \tan^{-1}\left(\frac{0.207 \text{ k}}{0.158 \text{ k}}\right) = 52.6 \text{ degree} \quad \text{Eq. D3.1.2.2-2a}$$

$$\frac{P_{nft}}{P'_{nfw}} = \frac{0.607}{(0.607)^2 + \left[\frac{0.887 \text{ k}}{0.733 \text{ k}}(0.794)^2\right]} = 0.537 \quad \text{Eq. D3.1.2.2-1a}$$

$$P_{nft} = 0.537(0.887 \text{ k}) = 0.476 \text{ k} \quad \text{Close enough to assumed } 0.475 \text{ k}$$

### Result of $P_{nft}$ at interior support controlled by pull-out:

$$P_{nft} = 0.475 \text{ k/screw}$$

The strength is controlled by wood bearing (against screw shank), and is reduced due to shear and tension interaction.

At *exterior supports* (pull-out at end-lap):

$$T = \frac{0.200 \text{ k/ft}}{1.33 \text{ screws/ft}} = 0.150 \text{ k/screw} \quad \text{See Load Data}$$

Pull-out *available tension strength* (no shear):

$$\frac{P_{nT}}{\Omega} = \frac{0.578 \text{ k}}{3} = 0.193 \text{ k} > 0.150 \text{ k} \quad 2 \text{ in. long screw OK}$$

Try 2-½ in. long screws for consistency with interior supports.

$$P_{nT} = 0.733 \text{ k} \quad \text{from above}$$

Pull-out *available strength* (no shear):

$$\frac{P_{nT}}{\Omega} = \frac{0.733 \text{ k}}{3} = 0.244 \text{ k} > 0.150 \text{ k} \quad \text{OK in tension only}$$

Iteration to obtain  $P_{nft}$ , assume:  $P_{nft} = 0.700 \text{ k}$

$$P'_{nfw} = 0.887 \text{ k} \quad \text{See } P_{nfw} \text{ Example 3a} \quad \text{Eq. D3.1.2.2-6}$$



$$\text{Let } V = \frac{P_{nft}}{\Omega} = \frac{0.700 \text{ k}}{3} = 0.233 \frac{\text{k}}{\text{screw}} \quad \text{Required shear strength in presence of } T = 0.150 \text{ k}$$

$$\frac{P_{nft}}{P'_{nfw}} = \frac{0.700 \text{ k}}{0.887 \text{ k}} = 0.789 \quad \text{Based on assumption}$$

$$\theta = \tan^{-1} \left( \frac{0.150 \text{ k}}{0.233 \text{ k}} \right) = 32.8 \text{ degree} \quad \text{Eq. D3.1.2.2-2a}$$

$$\frac{P_{nft}}{P'_{nfw}} = \frac{0.841}{(0.841)^2 + \left[ \frac{0.887 \text{ k}}{0.733 \text{ k}} (0.542)^2 \right]} = 0.791 \quad \text{Eq. D3.1.2.2-1a}$$

$$P_{nft} = 0.791(0.887 \text{ k}) = 0.702 \text{ k} \quad \text{Close enough to assumed } 0.700 \text{ k}$$

### Result of $P_{nft}$ at exterior support controlled by pull-out:

$$P_{nft} = 0.700 \text{ k/screw}$$

The strength is controlled by wood bearing (against screw shank) and is reduced due to shear and tension interaction.

Calculate  $S_{nf}$  controlled by pull-out:

**Note:** See sketch and end-lap discussion in Example 3a.

To determine  $S_{ni}$ , use:

$$P_{nfe} = 0.700/2 = 0.350 \text{ k/ply} \quad \text{At end-lap (shank limits strength)}$$

$$P_{nfi} = 0.475 \text{ k/ply} \quad \text{At interior supports (only one ply)}$$

To determine  $S_{nc}$ , use:

$$P_{nfe} = 0.996 \text{ k/ply} \quad \text{At the end-lap}$$

$$P_{nfi} = 0.475 \text{ k} \quad \text{At interior supports}$$

The opposing forces at the end-lap eliminate the force component in the screw shank that is parallel to the wood support and that is considered in  $S_{nc}$ . Both components are in the top panel ply in steel bearing at  $S_{nc}$ . The additive force in the shank parallel to panel span is considered in  $S_{ni}$ .

Calculate  $S_{ni}$  controlled by pull-out based on  $P_{nfi} = 0.475 \text{ k}$ ,  $P_{nfe} = 0.350 \text{ k}$ :

**Note:** See theory adjustment in Example 1a for variation of *support connection shear strength* along a panel length,  $L$  - Modified Eqs. D1-1 and D1-2.

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

$$\beta = n_s \alpha_s + 2n_p \alpha_p^2 + 4\alpha_e^2 \frac{P_{nfe}}{P_{nfi}} \quad \text{Modified Eq. D1-5}$$

See Example 3a for parameters:  $P_{ns}, \alpha_p^2, \alpha_e^2, n_s, n_p, A, \lambda$

$$\alpha_s = \frac{P_{ns}}{P_{nf}} = \frac{0.450 \text{ kips}}{0.475 \text{ kips}} = 0.947 \quad \text{Eq. D1-6}$$

$$\beta = 19(0.947) + 2(5)(0.446) + 4(0.446) \frac{0.350}{0.475} = 23.8$$

$$S_{ni} = \left[ (2)(0.0)(0.832 - 1) \frac{0.350 \text{ k}}{0.475 \text{ k}} + 23.8 \right] \frac{0.475 \text{ k}}{30 \text{ ft}} \quad \text{Modified Eq. D1-1}$$

$$= 0.377 \text{ klf}$$

Calculate  $S_{nc}$  controlled by pull-out based on  $P_{nfe} = 0.996 \text{ k}$ ,  $P_{nfi} = 0.475 \text{ k}$ :

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

$$\beta = n_s \alpha_s + 2n_p \alpha_p^2 \frac{P_{nfi}}{P_{nfe}} + 4\alpha_e^2 \quad \text{Modified Eq. D1-5}$$

$$\alpha_s = \frac{P_{ns}}{P_{nf}} = \frac{0.450 \text{ kips}}{0.996 \text{ kips}} = 0.452 \quad \text{Eq. D1-6}$$

$$\beta = 19(0.452) + 2(5)(0.446) \frac{0.475 \text{ k}}{0.996 \text{ k}} + 4(0.446) = 12.5$$

$$S_{nc} = \left( \frac{N^2 \beta^2}{L^2 N^2 + \beta^2} \right)^{0.5} P_{nfe} = \left( \frac{\left( 1.33 \frac{1}{\text{ft}} \right)^2 (12.5)^2}{(30 \text{ ft})^2 \left( 1.33 \frac{1}{\text{ft}} \right)^2 + (12.5)^2} \right)^{0.5} 0.996 \text{ k} \quad \text{Eq. D1-2}$$

$$= 0.396 \text{ klf}$$

**Result of Case (b):**  $S_{nf}$  controlled by pull-out with end-lap

$$S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.377, 0.396) = 0.377 \text{ klf}$$

**Note:** Uplift pressure (50 psf) reduces the *diaphragm strength* by 15% (0.444 klf to 0.377 klf) even when the support screw length is increased to 2-1/2 in. The *interior support connection shear strength* was reduced by 46% (0.887 k to 0.475 k), so the benefit of the *side-lap screws* shows up.

**(c) Interaction of shear and tension in screw (Section D3.1.2.1 or AISI S100 Section E4.5.3.1)**

$$\left( \frac{P_{nft}}{P_{nf}} \right) + \left( \frac{\Omega_t T}{P_{nts}} \right) = 1.3 \quad \text{Eq. D3.1.2.1-10}$$

$$\Omega_t = 3.0 \quad P_{nts} = 3.2 \text{ k}$$

**Note:**  $P_{nf}$  should be  $P_{nss}$  (= 2.7 k) in equation; conservatively use  $P_{nf} = 0.887 \text{ k}$  (from Example 3a).

At interior supports:

$$T = \frac{0.275 \text{ k/ft}}{1.33 \text{ screws/ft}} = 0.207 \text{ k/screw} \quad \text{See Load Data}$$

$$\left(\frac{P_{nft}}{P_{nf}}\right) = 1.3 - \left(\frac{3(0.207 \text{ k})}{3.2 \text{ k}}\right) = 1.11 \quad \text{No reduction} \quad \text{Eq. D3.1.2.1-10}$$

At exterior support:

$$T = \frac{0.200 \text{ k/ft}}{1.33 \text{ screws/ft}} = 0.150 \text{ k/screw} \quad \text{See Load Data}$$

$$\left(\frac{P_{nft}}{P_{nf}}\right) = 1.3 - \left(\frac{3(0.150 \text{ k})}{3.2 \text{ k}}\right) = 1.16 \quad \text{No reduction} \quad \text{Eq. D3.1.2.1-10}$$

**Result of Case (c):** Since the screw failure does not reduce or control the diaphragm strength, the *nominal diaphragm strength* is the same as that in Example 3a.

$$S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.444, 0.447) = 0.444 \text{ klf}$$

**Result of  $S_{nf}$  with uplift and end-lap:**

$$S_{nf} = \min(0.444, 0.377, 0.444) = 0.377 \text{ klf} \quad \text{Controlled by pull-out}$$

**Calculate  $S_n$  for Same Configuration With Uplift and a Butt-Joint:**

**(a) Interaction of shear and pull-over (one ply)**

At interior supports:

$$T = 0.207 \text{ k/screw} \quad P_{nft} = 0.996 \text{ k/ connection} \quad \begin{array}{l} \text{Same as end-lap} \\ \text{No reduction} \end{array}$$

At exterior supports:

$$T = 0.075 \text{ k/screw} \quad P_{nft} = 0.996 \text{ k/ connection} \quad \begin{array}{l} \text{Same as end-lap} \\ \text{No reduction} \end{array}$$

Result of Case (a) with butt-joints:

$$S_{nf} = 0.437 \text{ klf} \quad \text{No better than Example 3a at butt-joint } (P_{nf} = 0.887 \text{ k})$$

**(b) Interaction of shear and pull-out (one ply)**

At interior supports:

$$T = 0.207 \text{ k/screw} \quad P_{nft} = 0.475 \text{ k/ connection} \quad \text{Same as end-lap}$$

At exterior supports:

$$T = 0.075 \text{ k/screw}$$

Using same iterative process as at end-lap:

**Assume:**  $P_{nft} = 0.845 \text{ k}$

$$P'_{nfw} = 0.887 \text{ k} \quad \text{Eq. D3.1.2.2-6}$$

$$V = \frac{0.845 \text{ k}}{3} = 0.282 \frac{\text{k}}{\text{screw}} \quad \text{Required shear strength in presence of } T = 0.075 \text{ k}$$

$$\frac{P_{nft}}{P'_{nfw}} = \frac{0.845 \text{ k}}{0.887 \text{ k}} = 0.953 \quad \text{Based on assumption}$$

$$\theta = \tan^{-1}\left(\frac{0.075 \text{ k}}{0.282 \text{ k}}\right) = 14.9 \text{ degree} \quad \text{Eq. D3.1.2.2-2a}$$

$$\frac{P_{nft}}{P'_{nfw}} = \frac{0.966}{(0.966)^2 + \left[\frac{0.887 \text{ k}}{0.733 \text{ k}}(0.257)^2\right]} = 0.954 \quad \text{Eq. D3.1.2.2-1a}$$

$$P_{nft} = 0.954(0.887 \text{ k}) = 0.846 \text{ k} \quad \text{Close enough to assumed } 0.845 \text{ k}$$

**Result:**  $P_{nft} = 0.845 \text{ k/ screw}$  at exterior support

Calculate  $S_{nf}$  controlled by pull-out:

Calculate  $S_{ni}$  controlled by pull-out based on  $P_{nfi} = 0.475 \text{ k}$ ,  $P_{nfe} = 0.845 \text{ k}$ :

See Example 3a for parameters:  $P_{ns}, \alpha_p^2, \alpha_e^2, n_s, n_p, A, \lambda$

$$\alpha_s = \frac{P_{ns}}{P_{nf}} = \frac{0.450 \text{ kips}}{0.475 \text{ kips}} = 0.947 \quad \text{Eq. D1-6}$$

$$\beta = 19(0.947) + 2(5)(0.446) + 4(0.446) \frac{0.845}{0.475} = 25.6 \quad \text{Modified Eq. D1-5}$$

$$S_{ni} = \left[ (2)(0.0)(.832 - 1) \frac{0.845 \text{ k}}{0.475 \text{ k}} + 25.6 \right] \frac{0.475 \text{ k}}{30 \text{ ft}} = 0.405 \text{ klf} \quad \text{Modified Eq. D1-1}$$

Calculate  $S_{nc}$  controlled by pull-out based on  $P_{nfi} = 0.475 \text{ k}$ ,  $P_{nfe} = 0.845 \text{ k}$ :

**Note:** See Example 3a discussion about end-lap.  $S_{nc}$  is based on  $P_{nfe}$ , and there is one ply at end for butt-joint so both components apply to screw shank.

$$\alpha_s = \frac{0.450 \text{ kips}}{0.845 \text{ kips}} = 0.533$$

$$\beta = 19(0.533) + 2(5)(0.446) \frac{0.475}{0.845} + 4(0.446) = 14.4$$

$$S_{nc} = \left( \frac{N^2 \beta^2}{L^2 N^2 + \beta^2} \right)^{0.5} P_{nfe} = \left( \frac{\left(1.33 \frac{1}{\text{ft}}\right)^2 (14.4)^2}{(30 \text{ ft})^2 \left(1.33 \frac{1}{\text{ft}}\right)^2 + (14.4)^2} \right)^{0.5} 0.845 \text{ k} = 0.382 \text{ klf}$$

**Result of Case (b) with butt-joints:**

$$S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.405, 0.382) = 0.382 \text{ klf}$$

Controlled by pull-out at corner connection.

**Note:** The answer is about the same as the design at the end-lap (1% increase from 0.377 klf to 0.382 klf). The controlling limit state changed from  $S_{ni}$  to  $S_{nc}$ . The case of one end with end-lap and one without end-lap is between the two.

### (c) Interaction of shear and tension in screw

At interior supports:

$$T = 0.207 \text{ k/screw} \quad \text{Same as end-lap with no reduction}$$

At exterior supports:

$$T = 0.075 \text{ k/screw}$$

$$\left( \frac{P_{nft}}{P_{nf}} \right) = 1.3 - \left( \frac{3(0.075 \text{ k})}{3.2 \text{ k}} \right) = 1.23 \quad \text{No reduction}$$

Result of Case (c) with butt-joints:

$$S_{nf} = 0.437 \text{ klf} \quad \text{Same as in Example 3a at butt-joint } (P_{nf} = 0.887 \text{ k})$$

### Example 3b Results (With 50 psf Uplift)

#### Nominal diaphragm shear strength per unit length:

$$S_{nf} = \min(0.444, 0.377, 0.444) = 0.377 \text{ klf} \quad \text{with end-laps}$$

$$S_{nf} = \min(0.437, 0.382, 0.437) = 0.382 \text{ klf} \quad \text{with butt-joints}$$

#### Available diaphragm strength per unit length:

$$\Omega_d = 3.00 \text{ (Wood)}$$

$$ASD \quad S_a = 0.382 \text{ klf} / 3 = 0.127 \text{ klf} \quad \text{Use butt-joint result for design}$$

#### Diaphragm stiffness:

$$G' = 42.5 \text{ k/in.} \quad \text{Same as no uplift}$$

Requirements to achieve this *strength* and *stiffness*:

Screw pattern and schedule should be as shown in the sketch on page 1 of Example 3a.

Screw length = 2-1/2 in. and screw to be fully threaded.

**Note:** Uplift pressure (50 psf) reduces the *nominal diaphragm strength* by 15% (0.444 klf to 0.377 klf) with end-laps.

Uplift pressure (50 psf) reduces the *nominal diaphragm strength* by 12.6% (0.437 klf to 0.382 klf) with butt-joints.

Interaction of shear (wood bearing) and pull-out in the wood support controls *diaphragm strength*. Screw bearing against wood controls where no uplift is present. Butt-joint model is rational for design in this example (0.382 klf at butt-joint vs. 0.377 klf at end-lap or 1% difference).

Corrosion resistance could affect screw or *panel* specifications. Consult the wood truss provider and screw manufacturer. Consult the *panel* manufacturer for material and coating specifications.

### Example 3c: Nominal Diaphragm Shear Strength per Unit Length, $S_n$ , With Fiberglass Insulation

#### Objective

Use the *diaphragm configuration* of Example 3a and calculate the *nominal diaphragm shear strength* per unit length,  $S_n$ , and the *available strength* with R-19 fiberglass insulation over supports and beneath the *panel*. Compare the impact with no insulation.

**Note:** The Section D1.3 limits (a) to (f) are met. Sections D1.3 and D1.3.1.1 are used for the design.

All support screws are at interior flutes in this *configuration*; *side-lap* is not fastened to the support. The support screw's *diaphragm* shear contribution at interior flutes is neglected but the *connection* provides uplift resistance.

The shear and tension interaction is not considered since *shear resistance* is not provided by *support connections*.

The R-19 insulation will be compressed to less than 3/8 in. thickness beneath the *panel*.

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

*Safety and resistance factors* are in Section B and Section D1.1.4.1:

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

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

**Note:**  $S_{nf}$  is in Section D1.3, and  $S_{nb}$  is in Section D2. Strength will be determined with and without uplift.  $S_{ne}$  is calculated in Example 3d.

#### Calculate Nominal Diaphragm Shear Strength per Unit Length Controlled by Panel Screws, $S_{nf}$

##### Calculate support connection strength, $P_{nf}$ (Section D1.3):

$$P_{nf} = 0 \text{ k} \quad \text{Interior flute with insulation beneath the panel}$$

##### Calculate side-lap connection strength, $P_{ns}$ :

$$P_{ns} = 0.450 \text{ k} \quad \text{From Example 3a}$$

$$S_{nf} = \frac{n_s P_{ns}}{L} = \frac{19(0.450 \text{ k})}{30 \text{ ft}} = 0.285 \text{ klf} \quad \text{Eq. D1.3.1.1-1}$$

**Note:** Eq. D1-1 simplifies to Eq. D1.3.1.1-1 when  $P_{nf} = 0$ .

#### Calculate the *nominal strength* at the *diaphragm edge* perpendicular to the *panel span* at the *perimeter chord*:

**Note:** Section D1.3 allows screw  $P_{nfs}$  and  $P_{nf}$  to be determined using Section D1.1.4 at reaction lines when the compressed insulation thickness is less than or equal to 3/8 in. Edge detail parallel to *edge panel span* is in Example 3d.

At perimeter chord (perpendicular to *panel* span):

$$P_{nf} = \min(0.887 \text{ k}, 0.996 \text{ k}, 2.70 \text{ k}) = 0.887 \text{ k} \quad \text{from Example 3a} \quad \text{Eq. D1.1.4.2-2}$$

Screw bearing against wood controls the *nominal strength*.

$$S_{nf} = NP_{nf} = 1.33 \frac{\text{screw}}{\text{ft}} \left( 0.887 \frac{\text{k}}{\text{screw}} \right) = 1.18 \text{ klf} > 0.285 \text{ klf OK}$$

**Note:** See Eq. D4.4-3 and Figure 2.2-1 for similar considerations.

Shear has to get in and out at perimeters.  $P_{nf}$  contribution is neglected in the field.

**Result:**  $S_{nf} = 0.285 \text{ klf}$       No Uplift

### Calculate Nominal Diaphragm Shear Strength per Unit Length, $S_{nf}$ , Controlled by Panel Screws in the Presence of Uplift

#### Load Data

Wind uplift zone = 50 psf (0.05 ksf)

Uplift reactions are the same as Example 3b.

**Note:** There is no shear and tension interaction since the *diaphragm* does not resist shear in the *support connections*.

**Result:**  $S_{nf} = 0.285 \text{ klf}$       With Uplift

### Determine the Nominal Strength of the Support Screw in Tension

**Note:** Tension in *support connections* must be considered. The impact of insulation on pull-over, pull-out, and shear and tension in the screw is not explicitly covered in AISI S310 or AISI S100. **Small-scale testing per AISI S905 is the required option.**

*Rational engineering* might make the following judgments in determining the pull-over nominal strength of the screw:

Since the insulation is compacted beneath the washer head,  $P_{nov}$  could approach the strength per AISI S100 Section E4.4.2. However, one-half of the pull-over *strength* from Example 3b is used. The rationalization is that the zone closest to the *panel* web will work, while dishing beneath the washer into insulation will make the other zones less effective and cause progressive tearing. See Example 3b Case (a).

Therefore,

$$P_{nov} = 0.5(1.15 \text{ k}) = 0.575 \text{ k} \quad \text{At interior supports}$$

$$P_{nov} = 0.5(0.575 \text{ k}) = 0.288 \text{ k} \quad \text{At exterior supports}$$

Note: 0.575 k includes AISI S100 prying adjustment.

At *interior supports*:

$$T = 0.207 \text{ k/screw}$$

#### Available tension strength (AISI S100 Section E4):

$$\Omega = 3.00$$

$$\Omega = 2.50 \text{ for rational engineering at connections (AISI S100 Section A1.2(c))}$$

$$\frac{P_{\text{nov}}}{\Omega} = \frac{0.575 \text{ k}}{2.50} = 0.230 \text{ k} > 0.207 \text{ k} \quad \text{OK for pull-over}$$

At exterior supports:

$$T = 0.075 \text{ k/screw}$$

Available tension strength:

$$\frac{P_{\text{nov}}}{\Omega} = \frac{0.288 \text{ k}}{2.50} = 0.115 \text{ k} > 0.075 \text{ k} \quad \text{OK for pull-over}$$

**Note:** Since pull-over failure is steel-related, use  $\Omega = 2.5$  (rational engineering analysis per AISI S100 Section A1.2(c)) in lieu of more severe  $\Omega = 3$ , which is consistent with  $\Omega_d = 3.0$  at wood supports (Section D1.1.4.1).

**Determine the pull-out nominal strength of the #14 × 2-1/2 in. long screw:**

**Note:** Pull-out strength is not impacted by insulation.  $P_{\text{not}}$  is the same as Example 3b.

$$P_{\text{nT}} = 6.16(1)(0.45)^2(0.248)(2.37) = 0.733 \text{ k} \quad \text{Eq. D3.1.2.2-7}$$

At interior supports:

$$T = 0.207 \text{ k/screw}$$

Available tension strength:

$$\frac{P_{\text{not}}}{\Omega} = \frac{0.733 \text{ k}}{3.00} = 0.244 \text{ k} > 0.207 \text{ k} \quad \text{OK for pull-out}$$

At exterior supports:

$$T = 0.150 \text{ k/screw} \quad \text{Two plys at end-lap}$$

Available tension strength:

$$\frac{P_{\text{not}}}{\Omega} = \frac{0.733 \text{ k}}{3.00} = 0.244 \text{ k} > 0.150 \text{ k} \quad \text{OK for pull-out}$$

**Determine the breaking strength of connection:**

**Note:** Breaking strength of screw will not be impacted by insulation.

At interior and exterior supports:

Available tension strength (AISI S100 Section E4):

$$\Omega_t = 3.0$$

$$\frac{P_{\text{nts}}}{\Omega} = \frac{3.2 \text{ k}}{3.00} = 1.07 \text{ k} > 0.207 \text{ k} > 0.150 \text{ k} \quad \text{OK for breaking strength}$$

**Result:**

Connection configuration of Example 3b is acceptable to resist the required uplift load of 50 psf with insulation between *panel* and support based on rational engineering analysis.

**Example 3c Results (With R-19 Insulation Between Panel and Support)**



$S_{nf} = 0.285 \text{ klf}$  With or without uplift— Applies at butt-joint or end-lap installations

**Available diaphragm strength per unit length:**

$$\Omega_d = 3.00 \text{ ASD (Wood)}$$

Section D1.1.4.1

$$\phi_d = 0.55 \text{ LRFD}$$

$$\phi_d = 0.50 \text{ LSD}$$

$$\begin{aligned} S_a &= 0.285 \text{ klf}/3 = 0.095 \text{ klf (ASD)} \\ &= (0.55)(0.285 \text{ klf}) = 0.157 \text{ klf (LRFD)} \\ &= (0.50)(0.285 \text{ klf}) = 0.143 \text{ klf (LSD)} \end{aligned}$$

**Use:** The screw pattern shown in the *configuration* on page 1 of Example 3a.  
Screw length = 2-1/2 in. to resist an **uplift load** of 50 psf and require screw to be fully threaded. Screw length = 2 in. **without uplift** and require screw to be fully threaded.

**Note:** Insulation reduces the *nominal strength* by 35% (0.437 klf to 0.285 klf). Additional *side-lap* screws could be added to increase *nominal strength*. This example is representative of a generic profile. Considerable testing with insulation and field experience exists. Consult the manufacturer for allowable spans and uplift resistance in lieu of rational analysis.  
Corrosion resistance over wood can impact screw specification. Corrosion resistance can impact *panel* material and coating specification.

### Example 3d: Edge Connections and Details Parallel to the Edge Panel Span

#### Objective

Determine the required edge *connections* and details parallel to the *edge panel* span so that  $S_{ne}$  exceeds  $S_n$ :

Part 1 - With insulation from Example 3c, 0.285 klf (with or without uplift), and

Part 2 - Without insulation from Example 3a, 0.437 klf (without uplift).

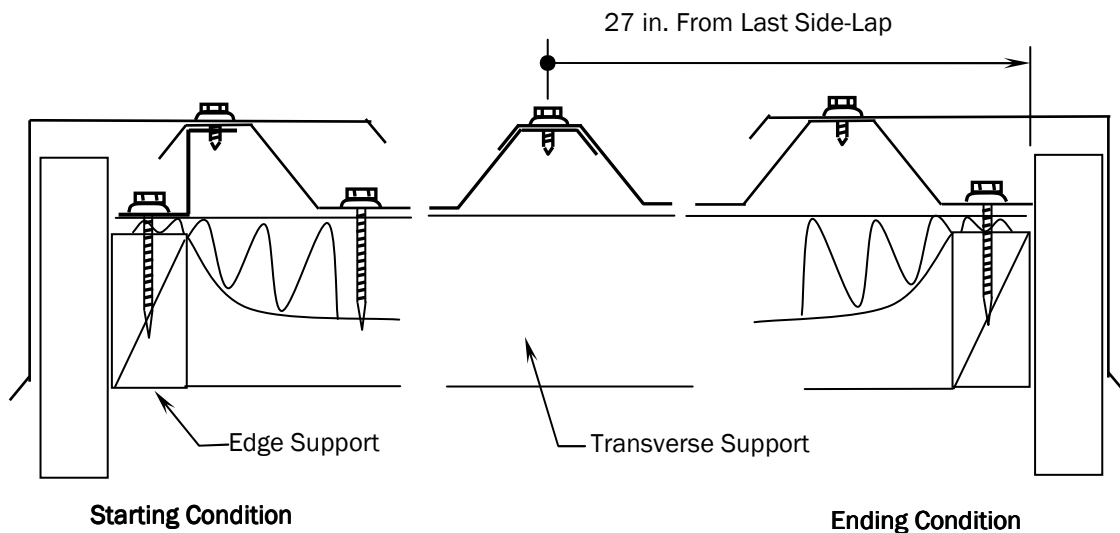
**Note:** The *configuration* of Example 3a applies.

The Section D1.3 requirements for  $S_{ne}$  and  $P_{nfs}$  are applicable.

This design must address the same concerns as Example 2c.

The following field conditions apply:

- Starting condition allows installation of full-width sheets,  $w_e = 36$  in., but does not allow a bottom flat on the edge support.
- Ending condition requires that sheet be ripped so  $w_e = 27$  in. but allows a bottom flat on the edge support.



#### Part 1 Example 3d - Starting Condition With Insulation

**Note:** The gap between the panel and the support is less than  $3/8$  in. but an edge support fastener is not located in the panel's bottom flat. Section D1.3 requires a detail to develop  $S_{ne}$ .

The starting detail requires a  $1\text{-}1/4$  in. deep Reinforcing Zee that also has insulation less than  $3/8$  in. between the bottom flat and the edge support.  $P_{nfs}$  can be calculated using Section D1.1.

Top flashing is not necessary for *diaphragm strength*. Follow the manufacturer's service guidelines for weathertightness, maximum connection spacing, and screw type and length.

**Design reinforcing Zee to transfer  $S_n = 0.285$  klf to side wall:**

$$\text{Try: } t = 0.036 \text{ in.} \quad F_y = 50 \text{ ksi} \quad F_u = 65 \text{ ksi}$$

Calculate screw *nominal shear strength* at panel-to-Zee connection:

**Note:** Use Section D1.1.2 (AISI S100 E4.3). Try same screw and spacing as at panel side-lap (#12 × 1 in. long)

$$\frac{t_2}{t_1} = \frac{0.036 \text{ in.}}{0.024 \text{ in.}} = 1.5 \quad 1 < \frac{t_2}{t_1} < 2.5$$

$$P_{ns} = 4.2(t_2^3 d)^{1/2} F_{u2} \quad \text{AISI S100 Eq. E4.3.1-1}$$

$$= 4.2((0.036 \text{ in.})^3 (0.216 \text{ in.}))^{1/2} (65 \frac{\text{k}}{\text{in.}^2}) = 0.867 \text{ k}$$

$$P_{ns} = 2.7 t_1 d F_{u1} \quad \text{AISI S100 Eq. E4.3.1-4}$$

$$= 2.7(0.024 \text{ in.})(0.216 \text{ in.})(62 \frac{\text{k}}{\text{in.}^2}) = 0.868 \text{ k}$$

$$P_{ns} = 2.7 t_2 d F_{u2} \quad \text{AISI S100 Eq. E4.3.1-5}$$

$$= 2.7(0.036 \text{ in.})(0.216 \text{ in.})(65 \frac{\text{k}}{\text{in.}^2}) = 1.36 \text{ k}$$

$$P_{nss} = 2.0 \text{ k} \quad \text{See Connection Schedule}$$

$$\frac{t_2}{t_1} = 1.0 \quad P_{ns} = \min(0.867 \text{ k}, 0.868 \text{ k}, 1.36 \text{ k}, 2.00 \text{ k}) = 0.867 \text{ k}$$

$$\frac{t_2}{t_1} = 2.5 \quad P_{ns} = \min(0.868 \text{ k}, 1.36 \text{ k}, 2.00 \text{ k}) = 0.868 \text{ k}$$

$$\frac{t_2}{t_1} = 1.5 \quad P_{ns} = 0.867 \text{ k} + \frac{1.5 - 1.0}{2.5 - 1} (0.868 \text{ k} - 0.867 \text{ k}) = 0.867 \text{ k}$$

Result:  $P_{nfs} = 0.867$  kips Screw tilting in Zee controls but is balanced with screw bearing against panel.

Calculate screw *nominal shear strength* at Zee-to-wood edge support connection:

**Note:** Use Section D1.1.4.2 (AISI S100 E4.3.1). Use same screw as at wood transverse supports in Example 3c (#14 screw × 2-1/2 in. long with 5/8 in. round sealing washer required for uplift).

$$P_{nfw} = 1.97G = 1.97(0.45) = 0.887 \text{ k} \quad \text{Table D1.1.4.2-1}$$

$$P_{nfw_s} = 2.7 t_1 d F_{u1} = 2.7(0.036 \text{ in.})(0.248 \text{ in.}) \left( 65 \frac{\text{k}}{\text{in.}^2} \right) = 1.57 \text{ k} \quad \text{Table D1.1.4.2-1}$$

$$P_{nss} = 2.7 \text{ k} \quad \text{From Connection Schedule in Example 3a}$$

Result:

$P_{nfs} = \min(0.887 \text{ k}, 1.57 \text{ k}, 2.70 \text{ k}) = 0.887 \text{ k}$  Screw bearing against wood controls shear strength at edge support.

**Calculate required screw spacing along Zee bottom flat at starting condition:**

Typical and maximum Zee length is 10 ft due to press-brake limits.

$$n_e = \frac{S_n L}{P_{nfs}} \quad \text{or} \quad \frac{L}{n_e} = \frac{P_{nfs}}{S_n} = \frac{0.887 \frac{k}{\text{conn.}}}{0.285 \frac{k}{\text{ft}}} = 3.11 \frac{\text{ft}}{\text{conn.}} \quad \text{Eq. D4.4-1}$$

**Calculate required screw spacing between panel and Zee top flat:**

$$\frac{L}{n_e} = \frac{P_{nfs}}{S_n} = \frac{0.867 \frac{k}{\text{conn.}}}{0.285 \frac{k}{\text{ft}}} = 3.04 \frac{\text{ft}}{\text{conn.}} > 20 \text{ in. o.c.}$$

**Result Part 1 at Starting Condition—With Insulation****Panel to the Reinforcing Zee over the edge support:**

Use: #12 screw × 1 in. long with ½ in. round sealing washer

Spacing = 20 in. o.c. to match the side-lap spacing

**Reinforcing Zee to the edge support:**

Use: #14 × 2-½ in. long screw with 5/8 in. round sealing washer

Maximum spacing = 3 ft.-0 in. o.c.

Number: 4 screws per Zee length > 8 ft and ≤ 10 ft

Spacing: 6 in., 3 ft - 0 in., 3 ft - 0 in., 3 ft - 0 in., 6 in. = 10 ft

6 in., 2 ft - 4 in., 2 ft - 4 in., 2 ft - 4 in., 6 in. = 8 ft

3 screws per Zee length > 5 ft and ≤ 8 ft

Spacing: 12 in., 3 ft - 0 in., 3 ft - 0 in., 12 in. = 8 ft

6 in., 2 ft - 0 in., 2 ft - 0 in., 6 in. = 5 ft

2 screws per Zee length ≤ 5 ft

Spacing: 12 in., 3 ft - 0 in., 12 in. = 5 ft

**Note:** Could use 2 in. long screws without sealing washers along the wood edge supports.

The 50 psf uplift of Example 3b is resisted by the transverse support connections at the starting *edge panel*. 2-1/2 in. long screw should be used with sealing washer if only one screw type is used on the project.

It is also acknowledged that the secondary (x-axis) force on bottom flat screws is negligible from study in Example 2c.

**Calculate shear stress in reinforcing Zee:**

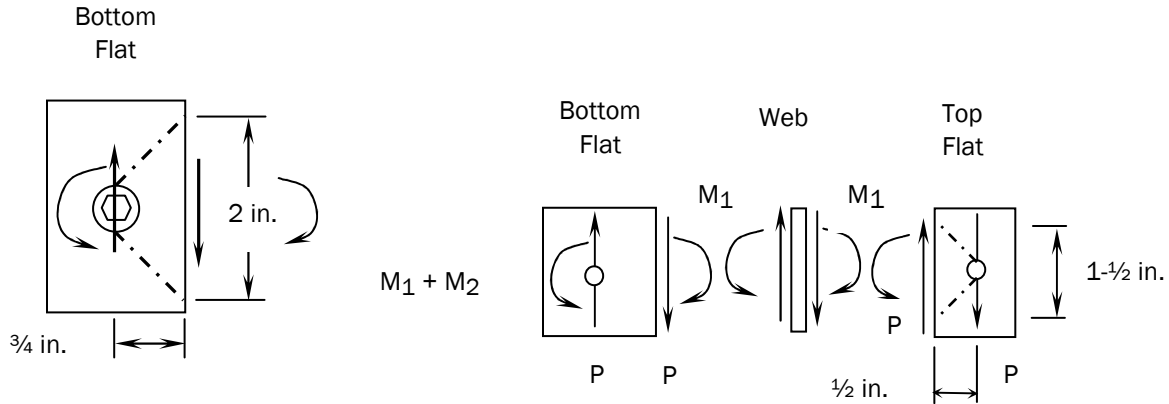
**Note:** Use 2 in. tributary length at the Zee bottom flat – assume 1-½ in. flange. Because panel has a 1 in. top flat, use a 1-1/2 in tributary length at the top flat.

ASD Available Diaphragm Strength = 0.095 klf From Example 3c

P = 0.095 klf Shear transfers to Zee

M<sub>1</sub> = 0.5 in.(0.095 klf) = 0.048 in. k/ ft

M<sub>2</sub> = 0.75 in.(0.095 klf) = 0.071 in. k/ ft



Stress in Top Flat of Zee

$$\tau_v \leq \frac{(1.67 \text{ ft})(0.095) \frac{\text{k}}{\text{ft}}}{1.5(0.036) \text{ in.}^2} = 2.94 \text{ ksi}$$

Stress in Bottom Flat of Zee

$$\tau_v \leq \frac{(3.0 \text{ ft})(0.095) \frac{\text{k}}{\text{ft}}}{2(0.036) \text{ in.}^2} = 3.96 \text{ ksi}$$

Calculate St. Venant shear stress in Zee web (see Example 2c):

$$\tau_{SVV} = \frac{3M_1}{bt^2} = \frac{3(0.048) \frac{\text{in.k}}{\text{ft}}}{12(0.036)^2 \frac{\text{in.}^3}{\text{ft}}} = 9.26 \text{ ksi}$$

Calculate total shear stress in Zee web:

$$f_v \leq \tau_v + \tau_{SVV} = 3.96 + 9.26 = 13.2 \text{ ksi} \quad \text{At outer edges of web}$$

Determine nominal shear strength,  $F_v$ , of web (AISI S100 Section C3.2.1):

$$\frac{h}{t} = \frac{1.25 \text{ in.}}{0.036 \text{ in.}} = 34.7 \quad \uparrow \square \downarrow \quad \text{For } \frac{h}{t} \leq \sqrt{\frac{Ek_v}{F_y}} \quad F_v = 0.6F_y$$

$$\sqrt{\frac{Ek_v}{F_y}} = \sqrt{\frac{29500 \text{ ksi} (5.34)}{50 \text{ ksi}}} = 56.1 > 34.7$$

$$F_v = 0.6(50 \text{ ksi}) = 30 \text{ ksi} \quad \text{Allowable shear stress} = \frac{F_v}{1.6} = \frac{30}{1.6} = 18.8 \text{ ksi}$$

18.8 ksi > 13.2 ksi OK Using rational screw locations and a conservative (tributary length) approximation

Result of Part 1 at starting condition with insulation:

Reinforcing Zee Use: minimum thickness = 0.036 in. and  $F_y = 50 \text{ ksi}$ .

### Part 1 Example 3d - Ending Condition With Insulation, $S_n = 0.285$ klf

**Note:** The gap between the *panel* and the support is less than 3/8 in. and the *panel's* bottom flat at the ripped length is over the wood edge support. Section D1.3 states that  $P_{nf}$  and  $P_{nfs}$  can be calculated using Section D1.1 to develop  $S_{ne}$ .

The impact of the partial-width sheet need not be investigated. The transverse support connection contribution is neglected and  $w_e$  does not appear in Eq. D1.3.1.1-1.

From Example 3a:

$$P_{nf} = P_{nfs} = \min(0.887 \text{ k}, 0.996 \text{ k}, 2.70 \text{ k}) = 0.887 \text{ k} \quad \text{Screw bearing against wood controls}$$

$$\frac{L}{n_e} = \frac{P_{nfs}}{S_n} = \frac{0.887 \frac{\text{k}}{\text{conn.}}}{0.285 \frac{\text{k}}{\text{ft}}} = 3.11 \frac{\text{ft}}{\text{conn.}}$$

#### Result of Part 1 at ending condition with insulation:

##### Screw spacing along edge support:

**Use:** #14 screw  $\times$  2-1/2 in. long with 5/8 in. round sealing washer

Install in line with transverse supports

Install at mid-span between transverse supports -  $\frac{5 \text{ ft}}{2} = 2.5 \text{ ft} < 3.11 \text{ ft}$

**Note:** 2 in. long screws could be used between transverse supports, but 2-1/2 in. long screws at transverse supports are required to resist the 50 psf uplift per Example 3b. The option of using 2-1/2 in. screws also results in the same screw for consistency and ease of installation. Flashing is not necessary for *diaphragm strength*. However, follow the manufacturer's service guidelines for weathertightness, maximum connection spacing, and screw type and length.

#### Objective of Example 3d—Part 2

Using Section D1, determine the required edge *connections* and details parallel to the *edge panel* span so  $S_{ne}$  exceeds  $S_n$  without insulation from Example 3a, 0.437 klf (no uplift).

**Note:** The gap between the panel and the supports is less than 3/8 in., but an edge support fastener is not located in the panel's bottom flat at the starting condition. A detail is required per Section D1.3 to develop  $S_{ne}$ .

The starting detail requires a 1-1/4 in. deep Reinforcing Zee.  $P_{nfs}$  can be calculated using Section D1.1.

#### Part 2 - Starting Condition Without Insulation

Design reinforcing Zee to transfer  $S_n = 0.437$  klf:

**Try:**  $t = 0.048$  in.  $F_y = 50$  ksi  $F_u = 65$  ksi

Calculate screw *nominal shear strength* at panel-to-Zee connection (Section D1.1.2):

Try same screw and spacing as at panel side-lap (#12 × 1 in. long)

$$\frac{t_2}{t_1} = \frac{0.048 \text{ in.}}{0.024 \text{ in.}} = 2.0 \quad 1 < \frac{t_2}{t_1} < 2.5$$

$$P_{ns} = 4.2(0.048 \text{ in.})^3 (0.216 \text{ in.})^{1/2} \left(62 \frac{\text{k}}{\text{in.}^2}\right) = 1.33 \text{ k} \quad \text{AISI S100 Eq. E4.3.1-1}$$

$$P_{ns} = 2.7(0.024 \text{ in.})(0.216 \text{ in.}) \left(62 \frac{\text{k}}{\text{in.}^2}\right) = 0.868 \text{ k} \quad \text{AISI S100 Eq. E4.3.1-4}$$

$$P_{ns} = 2.7(0.048 \text{ in.})(0.216 \text{ in.}) \left(65 \frac{\text{k}}{\text{in.}^2}\right) = 1.82 \text{ k} \quad \text{AISI S100 Eq. E4.3.1-5}$$

$$P_{nss} = 2.0 \text{ kips} \quad \text{From Connection Schedule in Example 3a}$$

$$\frac{t_2}{t_1} = 1.0 \quad P_{ns} = \min(1.33 \text{ k}, 0.868 \text{ k}, 1.82 \text{ k}, 2.00 \text{ k}) = 0.868 \text{ k}$$

$$\frac{t_2}{t_1} = 2.5 \quad P_{ns} = \min(0.868 \text{ k}, 1.82 \text{ k}, 2.00 \text{ k}) = 0.868 \text{ k}$$

$$\frac{t_2}{t_1} = 2.0 \quad P_{ns} = 0.868 \text{ k} + \frac{2.0 - 1.0}{2.5 - 1} (0.868 \text{ k} - 0.868 \text{ k}) = 0.868 \text{ k}$$

**Result:**  $P_{ns} = 0.868 \text{ k}$  Screw bearing against panel controls

Calculate screw nominal shear strength at Zee-to-wood edge support connection:

**Note:** Use Section D1.1.4.2 and same screw as at wood transverse supports.

$$P_{nfw} = 1.97G = 1.97(0.45) = 0.887 \text{ k} \quad \text{Table D1.1.4.2-1}$$

$$P_{nfws} = 2.7(0.048 \text{ in.})(0.248 \text{ in.}) \left(65 \frac{\text{k}}{\text{in.}^2}\right) = 2.09 \text{ k} \quad \text{Table D1.1.4.2-1}$$

$$P_{nss} = 2.7 \text{ k} \quad \text{From Connection Schedule in Example 3a}$$

$$\mathbf{Result:} \quad P_{nfs} = \min(0.887 \text{ k}, 2.09 \text{ k}, 2.70 \text{ k}) = 0.887 \text{ k}$$

Screw bearing against wood controls shear *strength* at edge support.

Calculate required screw spacing between panel and Zee top flat:

$$\frac{L}{n_e} = \frac{P_{nfs}}{S_n} = \frac{0.868 \frac{\text{k}}{\text{conn.}}}{0.437 \frac{\text{k}}{\text{ft}}} = 1.99 \frac{\text{ft}}{\text{conn.}} > 20 \text{ in. o.c.} = 1.67 \text{ ft o.c.}$$

Calculate required screw spacing along Zee bottom flat at starting condition:

$$\frac{L}{n_e} = \frac{P_{nfs}}{S_n} = \frac{0.887 \frac{\text{k}}{\text{conn.}}}{0.437 \frac{\text{k}}{\text{ft}}} = 2.03 \frac{\text{ft}}{\text{conn.}}$$

**Result of Part 2 at Starting Condition Without Insulation**

**Panel to the reinforcing Zee over the edge support:**

**Use:** #12 × 1 in. long screw with 1/2 in. round sealing washer  
Spacing = 20 in. o.c. to match the side-lap spacing

**Reinforcing Zee to the edge support:**

**Use:** #14 × 2 in. long screw with 5/8 in. round sealing washer  
Maximum spacing = 2 ft. - 0 in. o.c.

Number:

5 screws per Zee length > 8 ft and ≤ 10 ft

Spacing: 12 in., 2 ft - 0 in., 2 ft - 0 in., 2 ft - 0 in., 2 ft - 0 in., 12 in. = 10 ft  
6 in., 1 ft - 9 in., 1 ft - 9 in., 1 ft - 9 in., 1 ft - 9 in., 6 in. = 8 ft

4 screws per Zee length > 5 ft and ≤ 8 ft

Spacing: 12 in., 2 ft - 0 in., 2 ft - 0 in., 2 ft - 0 in., 12 in. = 8 ft  
3 in., 1 ft - 6 in., 1 ft - 6 in., 1 ft - 6 in., 3 in. = 5 ft

3 screws per Zee length ≤ 5 ft

Spacing: 6 in., 2 ft - 0 in., 2 ft - 0 in., 6 in. = 5 ft

**Calculate shear stress in Reinforcing Zee:**

*ASD Available Diaphragm Strength* = 0.146 klf From Example 3a

See Sketch at Example 3d Part 1

$P = 0.146$  klf Shear transfers to Zee

$M_1 = 0.50$  in.(0.146 klf) = 0.073 in. k/ft

$M_2 = 0.75$  in.(0.146 klf) = 0.110 in. k/ft

Stress in top flat of Zee:

$$\tau_v \leq \frac{1.67 \text{ ft}(0.146) \frac{\text{k}}{\text{ft}}}{1.5(0.048) \text{ in.}^2} = 3.39 \text{ ksi}$$

Stress in bottom flat of Zee:

$$\tau_v \leq \frac{2.00 \text{ ft}(0.146) \frac{\text{k}}{\text{ft}}}{2(0.048) \text{ in.}^2} = 3.04 \text{ ksi}$$

**Calculate St. Venant shear stress in Zee web (See Example 2c):**

$$\tau_{SV} = \frac{3M_1}{bt^2} = \frac{3(0.073) \frac{\text{in. k}}{\text{ft}}}{12(0.048)^2 \frac{\text{in.}^3}{\text{ft}}} = 7.92 \text{ ksi}$$

**Calculate total shear stress in Zee web:**

$$f_v \leq \tau_v + \tau_{SV} = 3.39 + 7.92 = 11.3 \text{ ksi} \quad \text{At outer edges of web}$$



**Determine nominal shear strength,  $F_v$ , of web (AISI S100 Section C3.2.1):**

$$\frac{h}{t} = \frac{1.25 \text{ in.}}{0.048 \text{ in.}} = 26.0 \quad \text{See the check in Example 3d Part 1 with } t = 0.036 \text{ in.}$$

$$F_v = 0.6(50 \text{ ksi}) = 30 \text{ ksi} \quad \text{Allowable shear stress} = \frac{F_v}{1.6} = \frac{30}{1.6} = 18.8 \text{ ksi}$$

18.8 ksi > 11.3 ksi OK Using rational screw locations and a conservative (tributary length) approximation

**Result of Part 2 at Starting Condition:**

**Reinforcing Zee Use:** minimum thickness = 0.048 in. and  $F_y = 50$  ksi

**Note:** If investigate  $t = 0.036$  in. and with no screw spacing change,  $\tau_v = 4.52$  ksi,  $\tau_{SV} = 14.1$  ksi, and the total *shear stress* = 18.6 ksi. It can, therefore, be justified to use  $t = 0.036$  in., but this is at the limit. Use greater thickness Zee for detail stability.

**Part 2 - Ending Condition Without Insulation,  $S_n = 0.437$  klf**

From Example 3a:

$$P_{nf} = P_{nfs} = \min(0.887 \text{ k}, 0.996 \text{ k}, 2.70 \text{ k}) = 0.887 \text{ k} \quad \text{Screw bearing against wood controls}$$

**Calculate required screw spacing between ripped edge panel and edge support:**

$$\frac{L}{n_e} = \frac{P_{nfs}}{S_n} = \frac{0.887 \frac{\text{k}}{\text{conn.}}}{0.437 \frac{\text{k}}{\text{ft}}} = 2.03 \frac{\text{ft}}{\text{conn.}}$$

**Result of Part 2 at ending condition without insulation:****Panel to the edge support:**

**Use:** #14 × 2 in. long screw with 5/8 in. round sealing washer

Spacing:

Install in line with transverse supports

$$\text{Install at } 1/3 \text{ points between transverse supports} - \frac{5 \text{ ft}}{3} = 1.67 \text{ ft} < 2.03 \text{ ft}$$

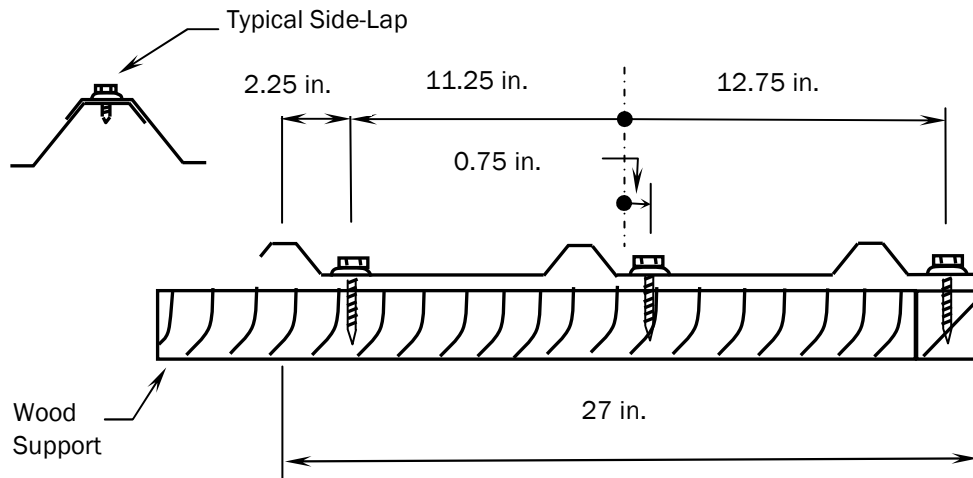
**Calculate diaphragm strength of partial-width panel:**

Try side-lap screws at 15 in. o.c. at last side-lap between last full- and partial-width *panel* (screws at quarter points rather than third points per span.)

$$\alpha_e^2 = \alpha_p^2 = \left( \frac{1}{(27 \text{ in.})^2} \right) \left( (0.75 \text{ in.})^2 + (12.75 \text{ in.})^2 + (11.25 \text{ in.})^2 \right) = 0.397$$

$$N = \frac{3 \text{ screws}}{2.25 \text{ ft}} = 1.33 \frac{\text{screws}}{\text{ft}}$$

$$n_s = \frac{30 \text{ ft} (12 \frac{\text{in.}}{\text{ft}})}{15 \frac{\text{in.}}{\text{conn.}}} + 1.0 = 25$$



See Example 3a for parameters:  $P_{ns}, \alpha_s, n_p, A, \lambda$

$$\beta = 25(0.507) + 2(5)(0.397) + 4(0.397) = 18.2 \quad \text{Eq. D1-5}$$

$$S_{ni} = [(2)(0.0)(0.832 - 1) + 18.2] \frac{0.887 \text{ k}}{30 \text{ ft}} = 0.538 \text{ klf}$$

$$S_{nc} = \left( \frac{(1.33)^2 (18.2)^2 \frac{1}{\text{ft}^2}}{(30)^2 (1.33)^2 \frac{\text{ft}^2}{\text{ft}^2} + (18.2)^2} \right)^{0.5} 0.887 \text{ k} = 0.490 \text{ klf}$$

**Result:**  $S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.538, 0.490) = 0.490 \text{ klf}$

based on  $P_{nf} = 0.887 \text{ k} > 0.437 \text{ klf}$  **OK** butt-joint case used

**Note:** If additional side-lap screws are not added (keep spacing = 20 in. o.c.),  $\beta = 15.2$  and  $S_{nf} = 0.420 \text{ klf}$  or 4% reduction vs. 0.437 klf. Some designers might let this go.

### Result of Part 2 - Example 3d at Ending Condition Without Insulation:

**Use:** #12 × 1 in. long screw with 1/2 in. round sealing washer at the *side-lap* between the last full-width *panel* and the partial-width *edge panel*. Spacing = 15 in. o.c.

**Use:** #14 × 2 in. long screw with 5/8 in. round sealing washer. Spacing along edge support:  
Install in line with transverse supports; and  
Install at 1/3 points between transverse supports.

**Note:** If *diaphragm* will experience an uplift event (say 50 psf), 2-1/2 in. long screws might be required. Part 2 considered no uplift.

### Example 4a: Multiple-Span Nominal Diaphragm Shear Strength Equivalent to That of a Single-Span Configuration With Large Side-Lap Spacing

#### Objective

Determine the *nominal shear strength* per unit length,  $S_n$ , of a multiple-span *diaphragm* that is equivalent to the *nominal shear strength* of a single-span *diaphragm* with the same *configuration* but a large *side-lap* screw spacing.

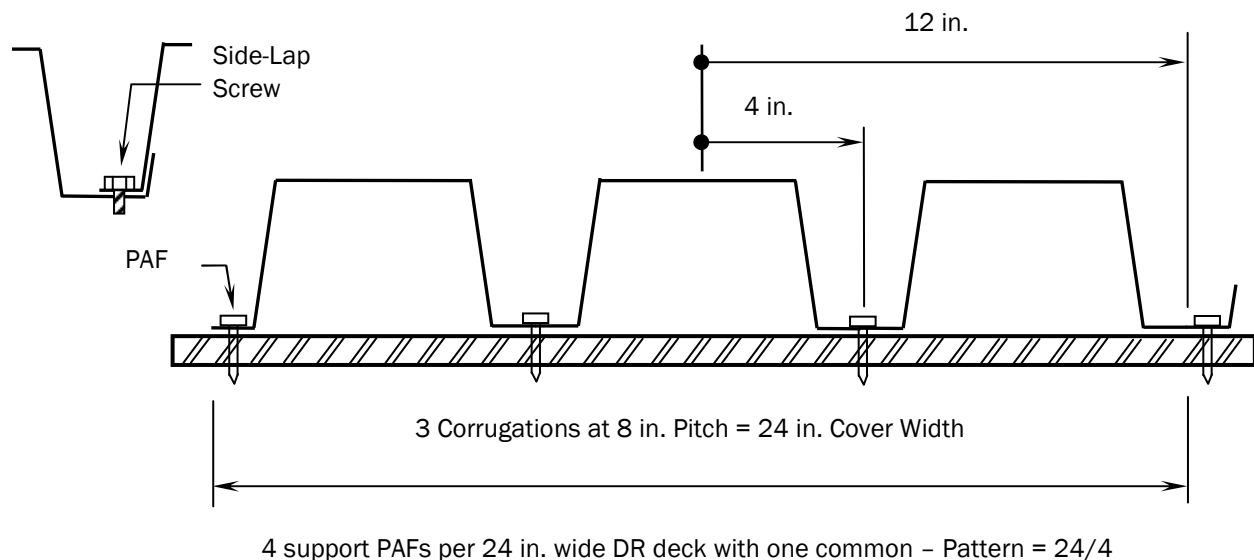
**Example 4a** Uses a minimal amount of *side-lap* screws with a spacing – 3 ft o.c.

**Example 4b** Increases the number of *side-lap* screws with a spacing = 6 in. o.c. to illustrate the impact on the required additional *support connections*.

**Note:** The calculation process described in this example may be used to determine the strength of a multiple-span *diaphragm* that matches the test results of a single-span *diaphragm* with the same *configuration*. The objective is to have the multiple-span *diaphragm* strength equivalent to that of the single-span *diaphragm* by increasing the number of interior support connections. Chapter D is used for the strength comparison, and uplift is not considered in this example. If uplift is present, the impact must be investigated in design.

#### Diaphragm Configuration

Note: The following configuration may be tested as a single span.



#### Deck Data

See Deep Rib Deck (DR) in *Commentary* Table C-1.1a.

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

SDI Generic load table shows: DL +LL ASD available flexural strength = 43 psf

Recommended maximum single-span = 15 ft

*Configuration* is a representative application.

Yield stress, $F_y$	= 40 ksi	Modulus of Elasticity, $E$	= 29500 ksi
Tensile strength, $F_u$	= 52 ksi	Panel length, $L$	= 12.0 ft
Depth, $D_d$	= 3.00 in.	Cover width, $w$	= 24.0 in.
Thickness, $t$	= 0.036 in.	Pitch, $d$	= 8.00 in.
Top flat width, $f$	= 5.24 in.	Web flat width, $w$	= 3.07 in.
Bottom flat width, $2e$	= 1.49 in.		
Moment of Inertia, $I_{xg}$	= 0.781 in. <sup>4</sup> /ft	This is $I_x$ value from SDI generic load tables and conservatively used for $I_{xg}$ .	

*Configuration* is a single run of *panels* (strength of a butt-joint condition will be determined).

**Note:** Chapter D limits (a) through (d) are satisfied. Therefore, the provisions of Section D1 will be used. Material is based on ASTM 1008 SS Grade 40. Deck profile will provide the required edge dimensions—Designer to verify.

### 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)	= 12.0 ft

### Connection Schedule

**Support connection:** Pattern = 24/4 – See *configuration* above.

Power-Actuated Fastener (PAF): Test-based *strength* and *flexibility* equations are obtained from the PAF manufacturer

$P_{npa}$	= 3.0 k	PAF nominal shear breaking strength
$P_{ntp}$	= 5.0 k	PAF nominal tension breaking strength

**Side-lap connection:**

#10 screw	$d$	= 0.190 in.
$P_{nss}$	= 1.5 k	Screw nominal shear breaking strength
Spacing	= 3 ft – 0 in. o.c.	(Between supports)

**Note:** Section D1.1.3 requires that  $P_{nf}$  for PAFs be established by tests.

Section D1 requires a side-lap screw spacing  $\leq 3$  ft o.c. for  $L_v > 5$  ft.

The *Commentary* on Section D1.1.3 directs designers to SDI DDM03 for test-based *strength* and *flexibility* equations for particular PAFs from several manufacturers. AISI S310 permits the use of test-based proprietary information from manufacturers. In this example, a proprietary PAF resistance equation is assumed. The assumed PAF resistance equation is not listed in DDM03 but is representative to illustrate the calculation process. The designer should consult the PAF manufacturer for test-based design values. The assumed PAF resistance is applicable to  $\frac{1}{4}$  in. and thicker steel supports.

### 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:**  $S_{nf}$  is in Section D1,  $S_{nb}$  is in Section D2, and  $S_{ne}$  is in Section D1.

### Calculate Nominal Diaphragm Shear Strength per Unit Length Controlled by Connections, $S_{nf}$ , 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:**  $S_{nf}$  is minimum of  $S_{ni}$  and  $S_{nc}$ . This example requires that sufficient edge connections will be provided at the edge panels parallel with the deck span so that  $S_{ne}$  will not control  $S_{nf}$ . The same  $P_{nf}$  value will be used at all supports.

### Calculate support connection strength, $P_{nf}$ (Section D1.1.3):

Use the equation provided by the manufacturer, which is applicable within the following limits:

Support thickness:  $t \geq 0.250$  in.

Panel thickness:  $0.028$  in.  $\leq t \leq 0.060$  in.

$$\begin{aligned} P_{nf} &= 60t(1 - 2t), k \quad \text{for } t \text{ in in.} \quad \text{Manufacturer-provided equation} \\ &= 60(0.036)(1 - 2(0.036)) = 2.00 k \end{aligned}$$

$$P_{npa} = 3.0 k \quad \text{See Connection Schedule}$$

**Result:**  $P_{nf} = \min(2.00, 3.0) = 2.00$  kips Bearing of deck against PAF controls

**Note:** The above manufacturer-provided equation is only applicable for a specific PAF. It cannot be used for other PAFs. Each equation is test-based.

### Calculate side-lap connection strength, $P_{ns}$ (Section D1.2.5 or AISI S100 E4.3.1):

$$\frac{t_2}{t_1} = \frac{0.036 \text{ in.}}{0.036 \text{ in.}} = 1.0 \quad \text{Therefore, } \frac{t_2}{t_1} \leq 1.0 \quad F_{u2} = F_{u1}$$

$$\begin{aligned} P_{ns} &= 4.2(t_2^3 d)^{1/2} F_{u2} \quad \text{AISI S100 Eq. E4.3.1-1} \\ &= 4.2((0.036 \text{ in.})^3 (0.19 \text{ in.}))^{1/2} (52 \frac{k}{\text{in.}^2}) = 0.650 k \end{aligned}$$

$$\begin{aligned} P_{ns} &= 2.7 t_1 d F_{u1} \quad \text{AISI S100 Eq. E4.3.1-2} \\ &= 2.7(0.036 \text{ in.})(0.19 \text{ in.})(52 \frac{k}{\text{in.}^2}) = 0.960 k \end{aligned}$$

$P_{nss} = 1.5 \text{ k}$  See Connection Schedule

**Result:**  $P_{ns} = \min(0.650, 0.960, 1.50) = 0.650 \text{ k}$  Tilting of screw in deck controls

### Calculate configuration parameters required for $S_{nf}$ :

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

$$\lambda = 1 - \frac{D_d L_v}{240\sqrt{t}} \geq 0.7 \quad \text{Required input units are defined in AISI S310 Eq. D1-4a}$$

$$= 1 - \frac{(3 \text{ in.})(12.0 \text{ ft})}{240\sqrt{0.036 \text{ in.}}} = 0.209 < 0.7$$

Therefore,  $\lambda = 0.700$  Unit-less

$$N = \frac{3 \text{ PAF}}{2 \text{ ft}} = 1.50 \frac{\text{PAF}}{\text{ft}}$$

Number of PAFs into support per ft along DR deck ends  
(Although 4 PAFs appear in the sketch, one is common to each deck at side-lap, so  $N = 3/2$ .)

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

$$n_p = \frac{L}{L_v} - 1 = \frac{12 \text{ ft}}{12 \text{ ft}} - 1 = 0.0 \quad \text{Number of interior supports} \quad \text{Eq. D1-9}$$

$$n_s = \left( \frac{(12 \text{ ft}) \left( \frac{12 \text{ in.}}{\text{ft}} \right) - 1}{36 \frac{\text{in.}}{\text{conn.}}} \right) \frac{12 \text{ ft}}{12 \text{ ft}} = 3 \quad \text{Number of side-lap screws along the panel length, } L$$

$$\alpha_s = \frac{P_{ns}}{P_{nf}} = \frac{0.650 \text{ k}}{2.0 \text{ k}} = 0.325 \quad \text{Connection strength ratio} \quad \text{Eq. D1-6}$$

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

$$x_{e1} = x_{e3} = 4.00 \text{ in.} \quad x_{e2} = x_{e4} = 12 \text{ in.}$$

$$= \left( \frac{1}{(24 \text{ in.})^2} \right) (2(4 \text{ in.})^2 + 2(12 \text{ in.})^2) = 0.556$$

$$\beta = 3(0.325) + 2(0)(0.556) + 4(0.556) = 3.20 \quad \text{Eq. D1-5}$$

### Calculate nominal diaphragm shear strength, $S_{nf}$ , for single span:

$$S_{ni} = [2A(\lambda - 1) + \beta] \frac{P_{nf}}{L} = [2(1.0)(0.700 - 1) + 3.20] \frac{2.00 \text{ k}}{12 \text{ ft}} = 0.433 \text{ klf} \quad \text{Eq. D1-1}$$

$$S_{nc} = \left( \frac{N^2 \beta^2}{L^2 N^2 + \beta^2} \right)^{0.5} P_{nf} = \left( \frac{(1.50 \frac{1}{\text{ft}})^2 (3.20)^2}{(12 \text{ ft})^2 (1.50 \frac{1}{\text{ft}})^2 + (3.20)^2} \right)^{0.5} 2.00 \text{ k} = 0.525 \text{ klf} \quad \text{Eq. D1-2}$$

### Result of single span:

$$S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.433, 0.525) = 0.433 \text{ klf}$$

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

$$\alpha = 1 \quad \text{Conversion factor for U.S. customary units}$$

$$s = 2e + 2w + f = 1.49 \text{ in.} + 2(3.07 \text{ in.}) + 5.24 \text{ in.} = 12.87 \text{ in.} \quad \text{Eq. D2.1-2}$$

$$S_{nb} = \frac{7890}{(1)(12 \text{ ft})^2} \left( \frac{(0.781 \text{ in.}^4/\text{ft})^3 (0.036 \text{ in.})^3 (8.0 \text{ in.})}{12.87 \text{ in.}} \right)^{0.25} = 3.34 \text{ klf}$$

$$S_{nf} = 0.433 \text{ klf} < S_{nb} = 3.34 \text{ klf}$$

**Single-span result (no uplift):**  $S_n = 0.433 \text{ klf}$

### Calculate an Equivalent Multiple-Span Configuration

**Note:** Most projects limit *panel* lengths to 30 or 40 ft for ease of handling and shipping. This might allow two- or three-span applications of this *configuration*. Equivalence will be calculated based on the two-span case,  $L = 24 \text{ ft}$ ; and then on the three-span case,  $L = 36 \text{ ft}$ .

**Calculate nominal diaphragm shear strength per unit length in a two-span application with the same support connections at interior and exterior supports as the single-span case:**

The following calculation will establish a baseline for comparison between single- and two-spans.

See single-span calculation for parameters:  $A, N, \lambda, \alpha_s, \alpha_p^2, \alpha_e^2, P_{ns}, P_{nf}$

$$A = 1 \quad L = 24 \text{ ft} \quad \lambda = 0.7 \quad \alpha_s = 0.325$$

$$\alpha_p^2 = \alpha_e^2 = 0.556 \quad P_{ns} = 0.650 \text{ k} \quad P_{nf} = 2 \text{ k} \quad N = 1.50 \frac{\text{PAF}}{\text{ft}}$$

$$n_p = \frac{L}{L_v} - 1 = \frac{24 \text{ ft}}{12 \text{ ft}} - 1 = 1.00 \quad \text{Eq. D1-9}$$

$$n_s = \left( \frac{L_v}{\text{Spacing}} - 1 \right) \frac{L}{L_v} = \left( \frac{(12 \text{ ft}) \left( \frac{12 \text{ in.}}{\text{ft}} \right)}{36 \frac{\text{in.}}{\text{conn.}}} - 1 \right) \frac{24 \text{ ft}}{12 \text{ ft}} = 6$$

$$\beta = 6(0.325) + 2(1)(0.556) + 4(0.556) = 5.29 \quad \text{Eq. D1-5}$$

$$S_{ni} = [2(1.0)(0.700 - 1) + 5.29] \frac{2.00 \text{ k}}{24 \text{ ft}} = 0.391 \text{ klf} \quad \text{Eq. D1-1}$$

$$S_{nc} = \left( \frac{(1.50 \frac{1}{ft})^2 (5.29)^2}{(24 \text{ ft})^2 (1.50 \frac{1}{ft})^2 + (5.29)^2} \right)^{0.5} 2.00 \text{ k} = 0.436 \text{ klf} \quad \text{Eq. D1-2}$$

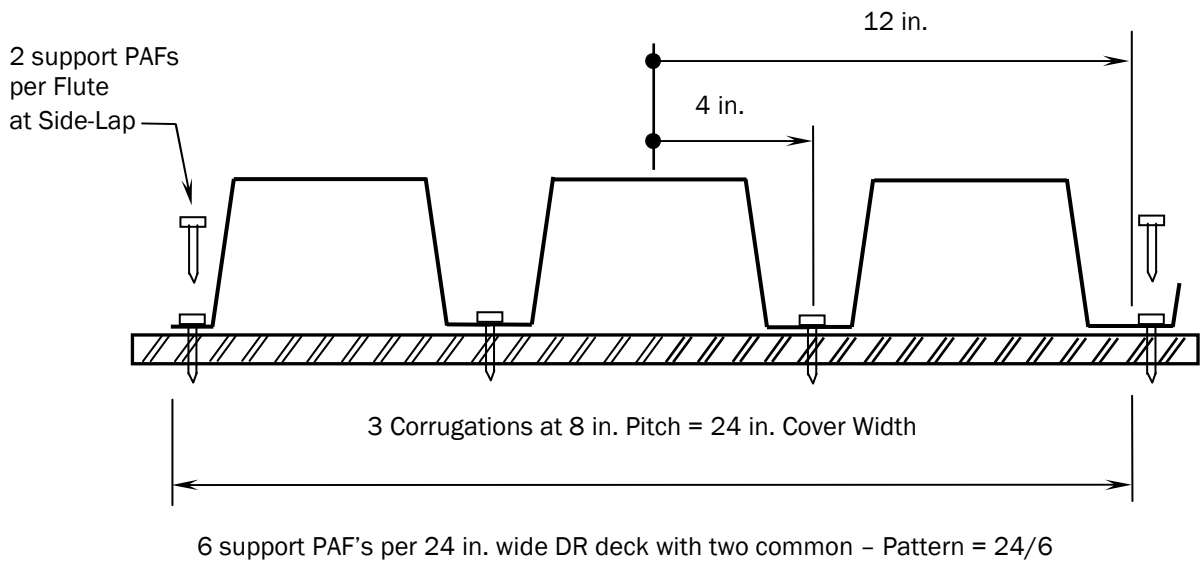
$$S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.391, 0.436) = 0.391 \text{ klf}$$

$$S_{nb} = 3.34 \text{ klf (Same as single-span)} \gg 0.391 \text{ klf} \quad \text{Eq. D2.1-1}$$

**Result of two-spans:**  $S_n = 0.391 \text{ klf}$  With no increase in support connections

**Note:** This is 10% less than the single-span case (0.433 klf), and indicates that additional *interior support connections* are required to obtain the strength equivalent to that of the single-span.

Try *new interior support connection* configuration (no change at exterior support):



**Calculate equivalence based on additional *interior support connections*:**

See two-span calculation above for:  $L, A, N, n_s, n_p, \lambda, \alpha_s, \alpha_e^2, P_{ns}, P_{nf}$

$$\alpha_p^2 = \left( \frac{1}{(24 \text{ in.})^2} \right) \left( 2(4 \text{ in.})^2 + 2(2)(12 \text{ in.})^2 \right) = 1.06 \quad \text{Eq. D1-8}$$

$$\beta = 6(0.325) + 2(1)(1.06) + 4(0.556) = 6.29 \quad \text{Eq. D1-5}$$

$$S_{ni} = [2(1.0)(0.700 - 1) + 6.29] \frac{2.00 \text{ k}}{24 \text{ ft}} = 0.474 \text{ klf} \quad \text{Eq. D1-1}$$

$$S_{nc} = \left( \frac{(1.50 \frac{1}{ft})^2 (6.29)^2}{(24 \text{ ft})^2 (1.50 \frac{1}{ft})^2 + (6.29)^2} \right)^{0.5} 2.00 \text{ k} = 0.516 \text{ klf} \quad \text{Eq. D1-2}$$

$$S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.474, 0.516) = 0.474 \text{ klf} > 0.433 \text{ klf OK}$$



$$S_{nb} = 3.34 \text{ klf (Same as single span)} > 0.474 \text{ klf} \quad \text{Eq. D2.1-1}$$

**Result of equivalent two span:**  $S_n = 0.474 \text{ klf}$  OK as compared to the single-span  
 $S_n = 0.433 \text{ klf}$

**Note:** The number of *interior support connections* increases 33% (4 per cover width vs. 3 per cover width) and the *nominal shear diaphragm strength* is 9.5% greater than that of a single-span with the same *configuration*.

### Calculate equivalence based on additional interior support connections at three spans:

See equivalent two-span calculation for:  $A, N, \lambda, \alpha_s, \alpha_e^2, \alpha_p^2, P_{ns}, P_{nf}$

$L = 36 \text{ ft}$  Three-span case with  $L_v = 12 \text{ ft}$

$$n_s = \left( \frac{(12 \text{ ft}) \left( \frac{12 \text{ in.}}{\text{ft}} \right)}{36 \frac{\text{in.}}{\text{conn.}}} - 1 \right) \frac{36 \text{ ft}}{12 \text{ ft}} = 9$$

$$n_p = \frac{L}{L_v} - 1 = \frac{36 \text{ ft}}{12 \text{ ft}} - 1 = 2.0$$

$$\beta = 9(0.325) + 2(2)(1.06) + 4(0.556) = 9.39 \quad \text{Eq. D1-5}$$

$$S_{ni} = [2(1.0)(0.700 - 1) + 9.39] \frac{2.00 \text{ k}}{36 \text{ ft}} = 0.488 \text{ klf} \quad \text{Eq. D1-1}$$

$$S_{nc} = \left( \frac{(1.50 \frac{1}{\text{ft}})^2 (9.39)^2}{(36 \text{ ft})^2 (1.50 \frac{1}{\text{ft}})^2 + (9.39)^2} \right)^{0.5} 2.00 \text{ k} = 0.514 \text{ klf} \quad \text{Eq. D1-2}$$

$$S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.488, 0.514) = 0.488 \text{ klf} > 0.433 \text{ klf} \text{ OK}$$

$$S_{nb} = 3.34 \text{ klf (Same as single span)} > 0.488 \text{ klf}$$

**Result of equivalent three span:**  $S_n = 0.488 \text{ klf}$  OK as compared to that of single span  
 $S_n = 0.433 \text{ klf}$

**Note:** The number of *interior support connections* increases 33% (4 per cover width vs. 3 per cover width), and the *nominal diaphragm shear strength* is 13% greater than that of a single span with the same *configuration*.

### Result Example 4a

The proposed 24/6 pattern *configuration* at the interior supports provides an equivalent *nominal diaphragm shear strength* per unit length relative to the single-span *configuration* while all other parameters are held constant.

**Note:** Although the trial *configuration* provides more *strength* than needed, physically cannot add less than one PAF per interior support per DR cover width. Comparing the two- and three-span cases, the two-span case is slightly more severe: 0.474 klf vs. 0.488 klf in this example, where  $S_{ni}$  controls the resistance. The extra *connection* at the *interior support* increased *nominal strength* from 0.391klf to 0.474 klf or 21% at the two-span case.

### Example 4b: Multiple-Span Nominal Diaphragm Shear Strength Equivalent to That of a Single-Span Configuration with Close Side-Lap Spacing

#### Objective

Determine a multiple-span *configuration* to provide equivalent or greater *nominal diaphragm shear strength* per unit length,  $S_n$ , relative to a single-span *configuration* where side-lap screw spacing is very small.

**Note:** *Interior support connection* numbers are the parameter for equivalence. *Diaphragm configuration* that might be tested as a single span is the same as Example 4a with one modification.

#### Connection Schedule

##### Side-lap connection:

#10 screw	$d = 0.190$ in.
$P_{nss} = 1.5$ kips	Screw <i>nominal shear breaking strength</i>
Spacing = 0 ft – 6 in. o.c.	(Between supports)

**Note:** A 6 in. o.c. *side-lap* spacing is a very close spacing. Some designers might consider this the limit (or impractical) and will require either more *support connections*, a different *side-lap connection*, or greater deck thickness to increase *diaphragm strength* rather than placing *side-lap* screws more closely together.

#### Calculate Nominal Diaphragm Shear Strength per Unit Length Controlled by Connections, $S_{nf}$ , for the Single-Span Case Using Section D1

See single-span calculation in Example 4a for:  $A, N, n_p, \lambda, \alpha_s, \alpha_e^2, P_{ns}, P_{nf}$

$$L = 12 \text{ ft}$$

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

$$n_s = \left( \frac{(12 \text{ ft}) \left( \frac{12 \text{ in.}}{\text{ft}} \right)}{6 \frac{\text{in.}}{\text{conn.}}} - 1 \right) \frac{12 \text{ ft}}{12 \text{ ft}} = 23 \quad \text{Side-lap screws along the panel length, } L$$

$$\beta = 23(0.325) + 2(0)(0.556) + 4(0.556) = 9.70 \quad \text{Eq. D1-5}$$

$$S_{ni} = [2(1.0)(0.700 - 1) + 9.70] \frac{2.00 \text{ k}}{12 \text{ ft}} = 1.52 \text{ klf} \quad \text{Eq. D1-1}$$

$$S_{nc} = \left( \frac{(1.50 \frac{1}{\text{ft}})^2 (9.7)^2}{(12 \text{ ft})^2 (1.50 \frac{1}{\text{ft}})^2 + (9.7)^2} \right)^{0.5} 2.00 \text{ k} = 1.42 \text{ klf} \quad \text{Eq. D1-2}$$

$$S_{nf} = \min(S_{ni}, S_{nc}) = \min(1.52, 1.42) = 1.42 \text{ klf}$$

$$S_{nb} = 3.34 \text{ klf} \quad (\text{Same as Example 4a single span}) > 1.42 \text{ klf} \quad \text{Eq. D2.1-1}$$

**Result of single span:**  $S_n = 1.42$  klf

**Note:** *Diaphragm strength* is controlled by the corner *connections* at the deck ends. The *side-lap* screw increase (23 vs. 3) increases  $S_n$  from 0.433 klf to 1.42 klf with a ratio of  $1.42/0.433 = 3.28$ .

### Calculate an Equivalent Multiple-Span Configuration

Calculate *nominal diaphragm shear strength per unit length* in a two-span application with the same *support connections* at interior and exterior supports as the single-span case:

See single-span calculation for parameters:  $A, N, \lambda, \alpha_s, \alpha_p^2, \alpha_e^2, P_{ns}, P_{nf}$

$L = 24$  ft Two-span case with  $L_v = 12$  ft

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

$$n_p = \frac{L}{L_v} - 1 = \frac{24 \text{ ft}}{12 \text{ ft}} - 1 = 1.0$$

$$n_s = \left( \frac{(12 \text{ ft}) \left( \frac{12 \text{ in.}}{\text{ft}} \right)}{6 \frac{\text{in.}}{\text{conn.}}} - 1 \right) \frac{24 \text{ ft}}{12 \text{ ft}} = 46$$

$$\beta = 46(0.325) + 2(1)(0.556) + 4(0.556) = 18.3 \quad \text{Eq. D1-5}$$

$$S_{ni} = [2(1.0)(0.700 - 1) + 18.3] \frac{2.00 \text{ k}}{24 \text{ ft}} = 1.48 \text{ klf} \quad \text{Eq. D1-1}$$

$$S_{nc} = \left( \frac{(1.50 \frac{1}{\text{ft}})^2 (18.3)^2}{(24 \text{ ft})^2 (1.50 \frac{1}{\text{ft}})^2 + (18.3)^2} \right)^{0.5} 2.00 \text{ k} = 1.36 \text{ klf} \quad \text{Eq. D1-2}$$

$$S_{nf} = \min(S_{ni}, S_{nc}) = \min(1.48, 1.36) = 1.36 \text{ klf}$$

$$S_{nb} = 3.34 \text{ klf} \quad (\text{Same as Example 4a single span}) > 1.36 \text{ klf} \quad \text{Eq. D2.1-1}$$

**Result two spans:**  $S_n = 1.36$  klf With no increase in support connections

**Note:** *Diaphragm strength* is controlled by the corner *connections* at the deck ends. Reduction is 4% and some designers would let go:  $1.36 \text{ klf} \approx 1.42 \text{ klf}$ . This implies that many *side-lap connections* could require no increase in *support connections* to apply single-span test results to multiple-span applications.

### Calculate equivalence based on additional interior support connections:

Try the same increased *interior support connection* configuration (Pattern = 24/6) as in Example 4a in the two-span case (no change at *exterior support*):

See two-span calculation in Example 4b for:  $A, L, N, n_s, n_p, \lambda, \alpha_s, \alpha_e^2, P_{ns}, P_{nf}$

See two-span equivalent calculations in Example 4a for extra support connections.

$$\alpha_p^2 = 1.06$$

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

$$= 46(0.325) + 2(1)(1.06) + 4(0.556) = 19.3$$

$$S_{ni} = [2(1.0)(0.700 - 1) + 19.3] \frac{2.00 \text{ k}}{24 \text{ ft}} = 1.56 \text{ klf} \quad \text{Eq. D1-1}$$

$$S_{nc} = \left( \frac{(1.50 \frac{1}{\text{ft}})^2 (19.3)^2}{(24 \text{ ft})^2 (1.50 \frac{1}{\text{ft}})^2 + (19.3)^2} \right)^{0.5} 2.00 \text{ k} = 1.42 \text{ klf} \quad \text{Eq. D1-2}$$

$$S_{nf} = \min(S_{ni}, S_{nc}) = \min(1.56, 1.42) = 1.42 \text{ klf}$$

$$S_{nb} = 3.34 \text{ klf} \quad (\text{Same as Example 4a single span}) > 1.42 \text{ klf} \quad \text{Eq. D2.1-1}$$

**Result of equivalent two spans:**  $S_n = 1.42 \text{ klf}$  OK as compared to

$$S_n = 1.42 \text{ klf} \text{ at single span}$$

Try the same *interior support connection configuration* (Pattern = 24/6) as in Example 4a in a three-span case (no change at *exterior support*):

See two-span equivalent calculations in Example 4b for:  $A, N, \lambda, \alpha_s, \alpha_e^2, \alpha_p^2, P_{ns}, P_{nf}$

$L = 36 \text{ ft}$  Three-span case with  $L_v = 12 \text{ ft}$

$$n_p = \frac{L}{L_v} - 1 = \frac{36 \text{ ft}}{12 \text{ ft}} - 1 = 2.0$$

$$n_s = \left( \frac{(12 \text{ ft}) \left( 12 \frac{\text{in.}}{\text{ft}} \right)}{6 \frac{\text{in.}}{\text{conn.}}} - 1 \right) \frac{36 \text{ ft}}{12 \text{ ft}} = 69$$

$$\beta = 69(0.325) + 2(2)(1.06) + 4(0.556) = 28.9 \quad \text{Eq. D1-5}$$

$$S_{ni} = [2(1.0)(0.700 - 1) + 28.9] \frac{2.00 \text{ k}}{36 \text{ ft}} = 1.57 \text{ klf} \quad \text{Eq. D1-1}$$

$$S_{nc} = \left( \frac{(1.50 \frac{1}{\text{ft}})^2 (28.9)^2}{(36 \text{ ft})^2 (1.50 \frac{1}{\text{ft}})^2 + (28.9)^2} \right)^{0.5} 2.00 \text{ k} = 1.42 \text{ klf} \quad \text{Eq. D1-2}$$

$$S_{nf} = \min(S_{ni}, S_{nc}) = \min(1.57, 1.42) = 1.42 \text{ klf}$$

$$S_{nb} = 3.34 \text{ klf} \quad (\text{Same as Example 4a single span}) > 1.42 \text{ klf} \quad \text{Eq. D2.1-1}$$

**Result of equivalent three spans:**  $S_n = 1.42 \text{ klf}$  OK as compared to

$$S_n = 1.42 \text{ klf} \text{ at single span}$$

Same strength as equivalent two spans.

### Result Example 4b

The proposed 24/6 pattern configuration at the interior supports provides an equivalent nominal diaphragm shear strength per unit length relative to the single-span configuration while all other parameters are held constant.

**Note:** When *side-lap connections* are very close in a *configuration*, it may not be efficient to add additional *interior support connections* to increase the diaphragm strength. These additional *interior support connections* can only increase the strength slightly. It might be more efficient to add *exterior support connections* and hold *interior support connections* constant. The following calculation illustrates the effectiveness of increasing the *exterior support connections*:

**Calculate equivalence based on additional exterior support connections:**

Try 24/6 pattern at *exterior supports* and 24/4 pattern at *interior supports* at three-span case:

**Collect configuration parameters required for  $S_{nf}$ :**

$$\begin{array}{llll}
 A = 2.0 & \lambda = 0.7 & n_p = 2 & P_{nf} = 2 \text{ kips} \\
 \alpha_s = 0.325 & \alpha_p^2 = 0.556 & \alpha_e^2 = 1.06 & n_s = 69 \\
 N = \frac{4 \text{ PAF}}{2 \text{ ft}} = 2.00 \frac{\text{PAF}}{\text{ft}} & & & \text{(Although six PAFs appear in the sketch, two are} \\
 & & & \text{common to each deck at } \textit{side-lap}, \text{ so } N = 4/2.) \\
 \beta = 69(0.325) + 2(2)(0.556) + 4(1.06) = 28.9 & & & 
 \end{array}$$

**Calculate  $S_{nf}$ :**

$$\begin{aligned}
 S_{ni} &= [2(1.0)(0.700 - 1) + 28.9] \frac{2.00 \text{ k}}{36 \text{ ft}} = 1.57 \text{ klf} \\
 S_{nc} &= \left( \frac{(2.0 \frac{1}{\text{ft}})^2 (28.9)^2}{(36 \text{ ft})^2 (2.0 \frac{1}{\text{ft}})^2 + (28.9)^2} \right)^{0.5} 2.00 \text{ k} = 1.49 \text{ klf} > 1.42 \text{ klf}
 \end{aligned}$$

**Result:**  $S_{nf} = \min(S_{ni}, S_{nc}) = \min(1.57, 1.49) = 1.49 \text{ klf} > 1.42 \text{ klf}$  OK

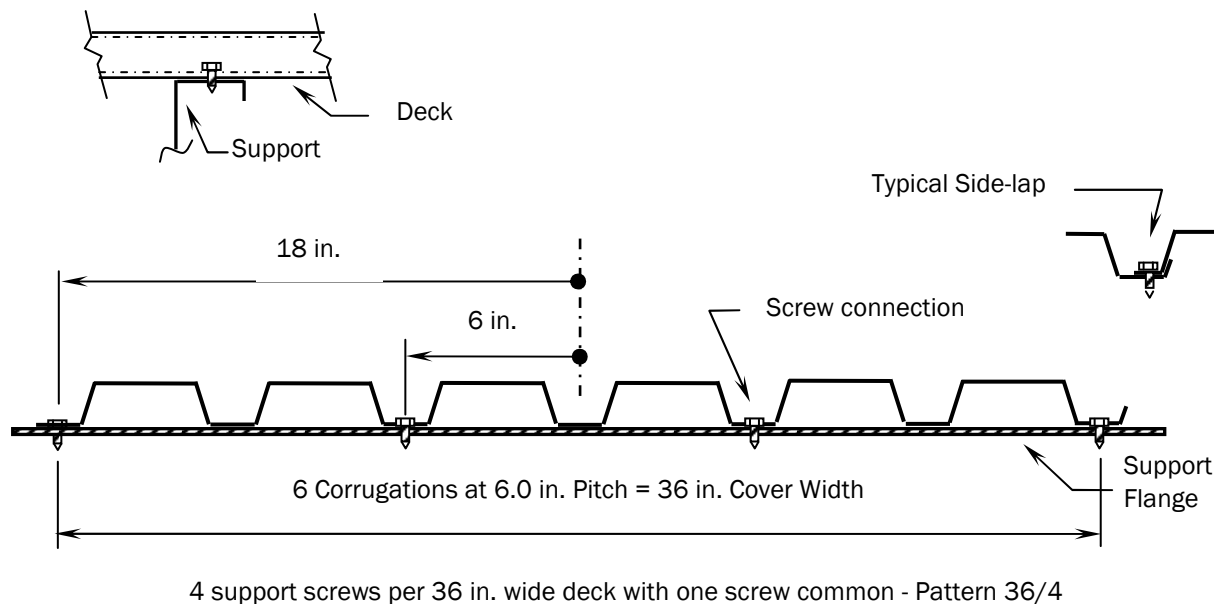
**Note:** In this *configuration*, the addition of end *connections* is more efficient where  $S_{nc}$  controls  $S_n$ . At the three-span case for a butt-jointed *panel*, this configuration requires the same number of additional support connections (two at *panel ends* vs. two at *interior supports*) but provides greater strength (1.49 klf vs. 1.42 klf).

### Example 5a: Available Diaphragm Shear Strength in the Absence of Uplift Where the Support Thickness Approaches the Panel Thickness

#### Objective

Calculate the *nominal* and *available diaphragm shear strength* per unit length in the absence of uplift using Chapter D where the support thickness approaches the *panel* thickness.

#### Diaphragm Configuration



#### Deck Data

See Wide Rib Deck (WR) in *Commentary* Table C-1.1a.

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

Yield stress, $F_y$	= 80 ksi	Modulus of Elasticity, $E$	= 29500 ksi
Tensile strength, $F_u$	= 82 ksi	Panel Length, $L$	= 20.0 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, $I_{xg}$	= 0.190 in. <sup>4</sup> /ft	This is $I_x$ value from manufacturer and conservatively used for $I_{xg}$ .	

WR deck will be end-lapped.

**Note:** Chapter D limits (a) through (d) are satisfied. Therefore, Section D1 is used to determine the diaphragm strength.

Specified material is ASTM A653 SS Grade 80. AISI S100 Section A2.3.3 design requirements apply. **Use:**  $F_y = 60$  ksi  $F_u = 62$  ksi  
 Edge dimensions must be checked using AISI S100 Sections E4.1 and E4.2 as applicable. This example assumes that the “as produced” deck provides adequate edge dimensions. Consult panel manufacturer for dimensions and determine whether screws are permitted at *side-laps*, and review panel end dimensions at end-laps.

### Steel Support Data

Yield stress, $F_y$	= 40 ksi	Tensile strength, $F_u$	= 55 ksi
Thickness, $t$	= 0.048 in.	Spacing, $L_v$ (shear span)	= 4.00 ft

Material is based on ASTM A653 SS Grade 40.

### Connection Schedule

**Support connection:** Pattern = 36/4 – See configuration above.

#12 screw	$d = 0.216$ in.
$P_{nss} = 2.0$ kips	$P_{nts} = 2.7$ kips

Same support connection type and spacing will be used at interior and exterior supports.

**Note:**  $P_{nss}$  = Breaking *nominal shear strength* of screw.  
 $P_{nts}$  = Breaking *nominal tensile strength* of screw.

### Side-lap connection:

#10 screw	$d = 0.190$ in.
$P_{nss} = 1.5$ kips	Screw <i>nominal shear breaking strength</i>
Spacing = 24 in. o.c.	(Between supports)

### 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:**  $S_{nf}$  is in Section D1,  $S_{nb}$  is in Section D2, and  $S_{ne}$  is in Section D1.

**Calculate nominal diaphragm shear strength per unit length controlled by 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:**  $S_{nf}$  is minimum of  $S_{ni}$  and  $S_{nc}$ . This example requires that sufficient edge *connections* will be provided at the *edge panels* so  $S_{ne}$  will not control  $S_{nf}$ . Same  $P_{nf}$  will be used at all supports.

**Calculate support connection strength,  $P_{nf}$  (Section D1.1.2):**

$$\frac{t_2}{t_1} = \frac{0.048 \text{ in.}}{0.036 \text{ in.}} = 1.5 \quad \text{Therefore, } 1.0 < \frac{t_2}{t_1} \leq 2.5$$

$$\begin{aligned} P_{nf} &= 4.2(t_2^3 d)^{1/2} F_{u2} && \text{AISI S100 Eq. E4.3.1-1} \\ &= 4.2((0.048 \text{ in.})^3 (0.216 \text{ in.}))^{1/2} (55 \frac{\text{k}}{\text{in.}^2}) = 1.13 \text{ k} \end{aligned}$$

$$\begin{aligned} P_{nf} &= 2.7 t_1 d F_{u1} && \text{AISI S100 Eq. E4.3.1-4} \\ &= 2.7(0.036 \text{ in.})(0.216 \text{ in.})(62 \frac{\text{k}}{\text{in.}^2}) = 1.30 \text{ k} \end{aligned}$$

$$\begin{aligned} P_{nf} &= 2.7 t_2 d F_{u2} && \text{AISI S100 Eq. E4.3.1-5} \\ &= 2.7(0.048 \text{ in.})(0.216 \text{ in.})(55 \frac{\text{k}}{\text{in.}^2}) = 1.54 \text{ k} \end{aligned}$$

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

$$\frac{t_2}{t_1} = 1.0 \quad P_{nf} = \min(1.13, 1.30, 1.54, 2.00) = 1.13 \text{ k}$$

$$\frac{t_2}{t_1} = 2.5 \quad P_{nf} = \min(1.30, 1.54, 2.00) = 1.30 \text{ k}$$

$$\frac{t_2}{t_1} = 1.5 \quad P_{nf} = 1.13 + \frac{1.5 - 1.0}{2.5 - 1} (1.30 - 1.13) = 1.19 \text{ k}$$

**Result:**  $P_{nf} = 1.19 \text{ k}$  Screw tilting in support controls strength, but this value is also close to the screw bearing against deck limit (1.3 k).

**Calculate side-lap connection strength,  $P_{ns}$  (Section D1.2.5 or AISI S100 Section E4.3.1):**

$$\frac{t_2}{t_1} = \frac{0.036 \text{ in.}}{0.036 \text{ in.}} = 1.0 \quad \text{Therefore, } \frac{t_2}{t_1} \leq 1.0 \quad F_{u2} = F_{u1}$$

$$\begin{aligned} P_{ns} &= 4.2(t_2^3 d)^{1/2} F_{u2} && \text{AISI S100 Eq. E4.3.1-1} \\ &= 4.2((0.036 \text{ in.})^3 (0.216 \text{ in.}))^{1/2} (62 \frac{\text{k}}{\text{in.}^2}) = 0.827 \text{ k} \end{aligned}$$

$$\begin{aligned} P_{ns} &= 2.7 t_1 d F_{u1} && \text{AISI S100 Eq. E4.3.1-2} \\ &= 2.7(0.036 \text{ in.})(0.216 \text{ in.})(62 \frac{\text{k}}{\text{in.}^2}) = 1.30 \text{ k} \end{aligned}$$

$$P_{nss} = 2.0 \text{ k} \quad \text{See Connection Schedule}$$

**Result:**  $P_{ns} = \min(0.827, 1.30, 2.00) = 0.827 \text{ kips}$  Tilting of screw in panel controls.



**Calculate configuration parameters required for  $S_{nf}$ :**

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

$$\lambda = 1 - \frac{D_d L_v}{240\sqrt{t}} \geq 0.7 \quad \text{Required input units are defined in AISI S310} \quad \text{Eq. D1-4}$$

$$= 1 - \frac{(1.47 \text{ in.})(4.0 \text{ ft})}{240\sqrt{0.036 \text{ in.}}} = 0.871 > 0.7 \text{ OK} \quad \text{Unit-less}$$

$$N = \frac{3 \text{ screws}}{3 \text{ ft}} = 1.00 \frac{\text{screw}}{\text{ft}} \quad \text{Number of screws into support per ft along deck ends}$$

(Although four screws appear in the sketch, one is common to each deck at *side-lap* so  $N = 3/3$ .)

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

$$n_p = \frac{L}{L_v} - 1 = \frac{20 \text{ ft}}{4 \text{ ft}} - 1 = 4.0 \quad \text{Number of interior supports} \quad \text{Eq. D1-9}$$

$$n_s = \left( \frac{(4 \text{ ft}) \left( \frac{12 \text{ in.}}{\text{ft}} \right)}{24 \frac{\text{in.}}{\text{conn.}}} - 1 \right) \frac{20 \text{ ft}}{4 \text{ ft}} = 5 \quad \text{Number of side-lap screws along the panel length, L}$$

$$\alpha_s = \frac{P_{ns}}{P_{nf}} = \frac{0.827 \text{ k}}{1.19 \text{ k}} = 0.695 \quad \text{Connection strength ratio} \quad \text{Eq. D1-6}$$

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

$$x_{e1} = x_{e3} = 6 \text{ in.} \quad x_{e2} = x_{e4} = 18 \text{ in.}$$

$$\alpha_p^2 = \alpha_e^2 = \left( \frac{1}{(36 \text{ in.})^2} \right) \left( 2(6 \text{ in.})^2 + 2(18 \text{ in.})^2 \right) = 0.556$$

$$\beta = 5(0.695) + 2(4)(0.556) + 4(0.556) = 10.1 \quad \text{Eq. D1-5}$$

**Calculate nominal diaphragm shear strength,  $S_{nf}$ :**

**Note:** The simplification is that the butt-joint strength ( $P_{nf} = 1.19 \text{ k}$ ) will be applied to the end-lap strength in calculating  $S_{nf}$ .

$$S_{ni} = [2(1.0)(0.871 - 1) + 10.1] \frac{1.19 \text{ k}}{20 \text{ ft}} = 0.580 \text{ klf} \quad \text{Eq. D1-1}$$

$$S_{nc} = \left( \frac{(1.0 \frac{1}{\text{ft}})^2 (10.1)^2}{(20 \text{ ft})^2 (1.0 \frac{1}{\text{ft}})^2 + (10.1)^2} \right)^{0.5} 1.19 \text{ kips} = 0.536 \text{ klf} \quad \text{Eq. D1-2}$$

**Result:**  $S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.580, 0.536) = 0.536 \text{ klf}$   
Controlled by corner screws at deck ends

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

$\alpha = 1$  Conversion factor for U.S. customary units  
 $s = 2e + 2w + f = 1.56 \text{ in.} + 2(1.53 \text{ in.}) + 3.56 \text{ in.} = 8.18 \text{ in.}$  Eq. D2.1-2

$$S_{nb} = \frac{7890}{(1)(4 \text{ ft})^2} \left( \frac{(0.190 \text{ in.}^4/\text{ft})^3 (0.036 \text{ in.})^3 (6.0 \text{ in.})}{8.18 \text{ in.}} \right)^{0.25} = 10.9 \text{ klf}$$

$$S_{nf} = 0.536 \text{ klf} \ll S_{nb} = 10.9 \text{ klf}$$

**Result Nominal Diaphragm Shear Strength per Unit Length,  $S_n$**

$S_n = 0.536 \text{ klf}$  Based on one support screw connection strength,  $P_{nf} = 1.19 \text{ k}$ .

**Note:** Designer must develop an edge detail so that  $S_{ne}$  is greater than 0.536 klf using the method of Example 1b or 2c, as applicable.

A detailed discussion of shear flow through end-laps is presented in Examples 1a and 2a, which include a comparison with *diaphragm strength* determined at butt-joints based on one *support connection strength*,  $P_{nf}$ . Consider shear flow through end-laps in this example and use:

$P_{nf} = 0.60 \text{ kips}$  (1.19 k/2) at *exterior supports* in the determination of  $S_{ni}$   
and 1.3 k at *exterior supports* in the determination of  $S_{nc}$ , while

$P_{nf} = 1.19 \text{ k}$  at *interior supports*.

The shear in the screw shank at the first interior flute at the end-lap is twice the shear that is in each deck ply while the four corner end-lap shear at the shear plane above the point of support tilting theoretically is zero in the shank. The selection of  $P_{nf} = 0.60 \text{ k}$  per ply is the most severe case.

**Consider shear flow through end-lap and determine  $S_{nf}$ :**

Note: See theory adjustment at Example 1a for variation of *support connection shear strength* along a panel length,  $L$ , using Modified Eqs. D1-1 and D1-2.

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

**Calculate  $S_{ni}$  based on  $P_{nfi} = 1.19 \text{ k}$ ,  $P_{nfe} = 0.60 \text{ k}$ :**

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

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

See butt-joint calculation on the previous page for:  $A, N, L, n_p, n_s, \lambda, \alpha_e^2, \alpha_p^2, P_{ns}$

$$\beta = n_s \alpha_s + 2n_p \alpha_p^2 + 4\alpha_e^2 \frac{P_{nfe}}{P_{nfi}} \quad \text{Modified Eq. D1-5}$$

$$\alpha_s = \frac{0.827 \text{ k}}{1.19 \text{ k}} = 0.695 \quad \text{Eq. D1-6}$$

$$\beta = 5(0.695) + 2(4)(0.556) + 4(0.556) \frac{0.60 \text{ k}}{1.19 \text{ k}} \\ = 9.04$$

$$S_{ni} = [2(1.0)(0.827 - 1) \frac{0.60 \text{ k}}{1.19 \text{ k}} + 9.04] \frac{1.19 \text{ k}}{20 \text{ ft}} \quad \text{Modified Eq. D1-1} \\ = 0.528 \text{ klf}$$

**Calculate  $S_{nc}$  based on  $P_{nfe} = 1.30 \text{ k}$ ,  $P_{nfi} = 1.19 \text{ k}$**

$$\beta = n_s \alpha_s + 2n_p \alpha_p^2 \frac{P_{nfi}}{P_{nfe}} + 4\alpha_e^2 \quad \text{Modified Eq. D1-5}$$

$$\alpha_s = \frac{0.827 \text{ k}}{1.30 \text{ k}} = 0.636 \quad \text{Eq. D1-6}$$

$$\beta = 5(0.636) + 2(4)(0.556) \frac{1.19 \text{ k}}{1.30 \text{ k}} + 4(0.556) \\ = 9.48$$

$$S_{nc} = \left( \frac{(1)^2 (9.48)^2 \frac{1}{\text{ft}^2}}{(20)^2 (1)^2 \frac{\text{ft}^2}{\text{ft}^2} + (9.48)^2} \right)^{0.5} 1.30 \text{ k} \quad \text{Modified Eq. D1-2} \\ = 0.557 \text{ klf}$$

**Result:**  $S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.528, 0.557) = 0.528 \text{ klf}$   
Controlled by tilting of screws at interior supports

**Note:** The butt-joint *strength* (without all the refinements) is 0.536 klf vs. 0.528 klf considering shear flow across the end-laps. The refinement provides a negligible 1.5% decrease and neglects the additional stability at the four corner end-lap. The controlling limit state changed –  $S_{nc}$  to  $S_{ni}$ . The other case of one end with end-lap and one without will be between these values – say, 0.532 klf. The result would be different at three spans and with fewer side-lap screws, but many manufacturers publish tables based on the butt-joint case and this is a rational approach in this example.

### Example 5b: Stiffness of the Configuration in Example 5a

#### Objective

Calculate the *stiffness* of the *configuration* in Example 5a.

**Note:** Use Sections D5.1.1 and D5.2. Use Appendix Section 1.4 for  $D_n$ . *Stiffness* considers the dominant *panels* in the *diaphragm* field.

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

$K = 1.0$  for steel *panels* with lap-down on steel supports

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

See *diaphragm configuration* and Figure D1-1 for  $x_e$ .

$$\alpha_3 = \alpha_4 = \frac{\sum x_e}{w} = \left( \frac{1}{36 \text{ in.}} \right) (2(6 \text{ in.}) + 2(18 \text{ in.})) = 1.33 \quad \text{Eq. D5.1.1-3}$$

#### Calculate support screw flexibility, $S_f$ , using *Commentary of Section D5.2.2*:

**Note:** Since the support is relatively thin, tilting of screw may occur at the support. Eq. D5.2.2-1 is based on thick supports where distortion is dominated by local slotting and buckling in the deck, and it will not be applicable to this configuration with a thin support. *Commentary* Section D5.2 should be considered.

#### Calculate, $t_3$ , balance point between tilting and bearing control in support:

$$4.2(t_3^3 d)^{1/2} F_{u2} = 2.7 t_2 d F_{u2}$$

**Rationale:** Beyond thickness,  $t_3$ , tilting at support will not control, and *flexibility* is relatively independent of support thickness.

$$(t_3^3)^{1/2} = \frac{2.7(0.048 \text{ in.})(0.216 \text{ in.})}{4.2(0.216 \text{ in.})^{0.5}} = 0.014 \text{ in.}^{1.5}$$

$$t_3 = 0.058 \text{ in.}$$

$$S_f = \left( 3 - 1.7 \left( \frac{t_2 - t_1}{t_3 - t_1} \right) \right) \left( \frac{\alpha}{1000 t_1^{0.5}} \right) \quad \text{Figure C-D5.2.2-1}$$

$$= \left( 3 - 1.7 \left( \frac{0.048 - 0.036}{0.058 - 0.036} \right) \right) \left( \frac{\alpha}{1000(0.036)^{0.5}} \right)$$

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

$$S_f = \frac{2.07\alpha}{1000\sqrt{t}} = \frac{2.07(1)}{1000\sqrt{0.036 \text{ in.}}} = 0.0109 \frac{\text{in.}}{\text{k}} \quad (59\% \text{ greater than } S_f \text{ with thick support})$$

Calculate side-lap screw flexibility,  $S_s$ :

$$S_s = \frac{3.0\alpha}{1000\sqrt{t}} = \frac{3.0(1)}{1000\sqrt{0.036 \text{ in.}}} = 0.0158 \frac{\text{in.}}{\text{k}} \quad \text{Eq. D5.2.2-2}$$

Calculate slip constant at connections,  $C$ :

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

$$\alpha_3 = \frac{\sum x_e}{w} = \frac{(2(6) + 2(18))}{36} = 1.33; \quad \alpha_4 = \frac{\sum x_p}{w} = \alpha_3 = 1.33$$

$$C = \left( \frac{29500 \frac{\text{k}}{\text{in.}^2} (0.036 \text{ in.})}{36 \text{ in.}} \right) \left( \frac{2(12 \frac{\text{in.}}{\text{ft}})(20 \text{ ft})}{2(1.33) + 4(1.33) + 2(5) \left( \frac{0.0109}{0.0158} \right)} \right) 0.0109 \frac{\text{in.}}{\text{k}} = 10.4 \quad \text{Eq. D5.1.1-2}$$

Calculate  $D_n$  using S310 Appendix 1:

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

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

**Note:** Only  $D_2$  is required in this example, and Example 1c determined that value,  $D_2$ , for the same WR deck.

$$D_2 = 7640 \text{ in.} \quad \text{Eq. 1.4-4}$$

As check, *Commentary* Table C-1.2 lists  $D_2 = 7726$  at  $t = 0.0358$  in. for WR deck and this value could be used in design.

$$U_2 = 6 \quad U_1 = U_3 = U_4 = 0$$

$$D = \frac{0 + 6(7640) + 0 + 0}{0 + 6 + 0 + 0} = 7640 \text{ in.} \quad \text{Eq. 1.4-2}$$

$$D_n = \frac{7640 \text{ in.}}{20 \text{ ft} \times 12 \frac{\text{in.}}{\text{ft}}} = 31.8 \text{ (Unit-less)} \quad \text{Eq. 1.4-1}$$

$$\gamma_c = 0.71 \quad \text{Table 1.3-1}$$

Calculate  $G'$ :

From Deck Data

$$s = f + 2e + 2w = 3.56 \text{ in.} + 1.56 \text{ in.} + 2(1.53 \text{ in.}) = 8.18 \text{ in.} \quad \text{Eq. 2.1-2}$$

$$d = 6 \text{ in.}$$

$$\mu = 0.3 \text{ (Poisson's ratio for steel)}$$

$$G' = \left( \frac{29500 \frac{\text{k}}{\text{in.}^2} (0.036) \text{ in.}}{2(1+0.3) \frac{8.18 \text{ in.}}{6 \text{ in.}} + 0.71(31.8) + 10.4} \right) 1 = 29.1 \frac{\text{k}}{\text{in.}} \quad \text{Eq. D5.1.1-1}$$

**Note:** If the support is much thicker:

$$S_f = \frac{1.3\alpha}{1000\sqrt{t}} = \frac{1.3(1)}{1000\sqrt{0.036} \text{ in.}} = 0.00685 \frac{\text{in.}}{\text{k}} \quad \text{Eq. D5.2.2-1}$$

$$C = 7.88 \quad 24\% \text{ less slippage and distortion at connections}$$

$$G' = 31.2 \text{ k/in.} \quad 7\% \text{ less deflection because warping dominates deflection}$$

### Result Example 5b - Stiffness:

$$G' = 29.1 \frac{\text{k}}{\text{in.}}$$

### Result of Example 5:

#### Nominal diaphragm shear strength per unit length:

$$S_n = 0.536 \text{ klf} \quad \text{Controlled by } S_{nf}$$

#### Available diaphragm shear strength per unit length, $S_a$ :

Depending on different load types, select *safety* and *resistance factors* from Section B1.

Use  $\Omega_d$  and  $\phi_d$  for connection-related diaphragm strength in AISI S100 Table D5.

$$\frac{S_n}{\Omega} = \frac{S_{nf}}{\Omega_{df}} \text{ for ASD} \quad \phi S_n = \phi_{df} S_{nf} \text{ for LRFD and LSD}$$

#### Diaphragm stiffness

$$G' = 29.1 \frac{\text{k}}{\text{in.}}$$





**American  
Iron and Steel  
Institute**

25 Massachusetts Avenue NW  
Suite 800  
Washington, DC 20001  
[www.steel.org](http://www.steel.org)

