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

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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.

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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.

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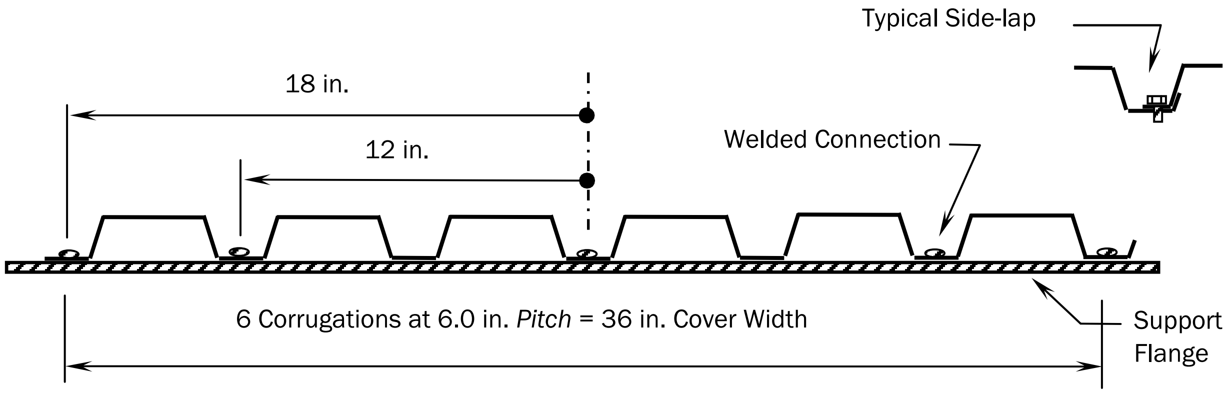
Example 1a: Nominal Diaphragm Shear Strength of a Wide Rib Deck Welded to Supports in the Absence of Uplift

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



5 *support welds* per 36 in. wide deck with one weld common - Pattern 36/5

Deck Data

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

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

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

Note: Chapter D limits (a) through (d) are satisfied, and the deck can be designed per Section D1. Material is based on ASTM 1008 SS Grade 40; other steels conforming to AISI S100 Section 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 in.) = 0.144 in < 0.15 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

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

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

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

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

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

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*

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

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

$$P_{nf} = 0.280\left[1 + 5.59\frac{\sqrt{E/F_u}}{d_a/t}\right]td_aF_u \qquad \text{For } 0.815\sqrt{\frac{E}{F_u}} < \frac{d_a}{t} \le 1.397\sqrt{\frac{E}{F_u}}$$ 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*: $d_a = (d - t) = (0.75 \text{ in.} - 0.036 \text{ in.}) = 0.714 \text{ in.}$

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

$$d_e = 0.7d - 1.5t \le 0.55d$$ AISI S100 Eq. E2.2.2.1-5

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

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

$d_e =$ 0.413 in.

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

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

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

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

$d_e =$ 0.413 in.

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

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

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

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

$d_e =$ 0.363 in. See **Note** below.

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

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

$$d_e = 0.7(0.75\text{ in.}) - 1.5(4(0.036\text{ in.})) \le 0.55(0.75\text{ in.})$$
$$= 0.309\text{ in.} < 0.413\text{ in.}$$

d_e = 0.309 in. See **Note** below.

Note: $d_e \ge 3/8$ 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:

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

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

$$P_{nf} = 0.280\left[1 + 5.59\frac{\sqrt{\dfrac{29500\text{ ksi}}{52\text{ ksi}}}}{19.8}\right](0.036\text{ in.})(0.714\text{ in.})\left(52\frac{\text{k}}{\text{in.}^2}\right) = 2.89\text{ k}$$

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

P_{nf} = same value as single-thickness *connections* = 7.03 k

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

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

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.

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

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

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.

$\frac{d_a}{t} = 4.21 \le 19.4$, 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 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):

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

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

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

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

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

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

P_{nss} = 1.5 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

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

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

$$= \max(0.806, 0.700) = 0.806$$ Unit-less

$$N = \frac{4\text{ welds}}{3\text{ ft}} = 1.33\frac{\text{welds}}{\text{ft}}$$ 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.

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

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

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

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

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

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

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

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

Calculate S_{nf}:

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

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

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

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

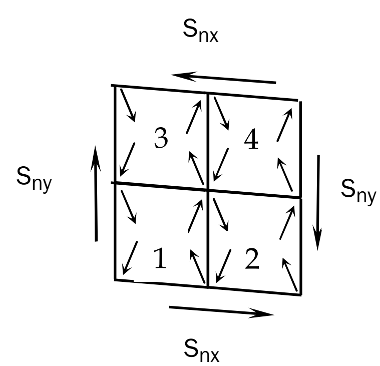
Result: S_{nf} = 1.19 klf Based on least weld strength, P_{nf} = 2.89 k.
S_{nf} applies at butt-joint or end-lap installations

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 ≈ 5.58 k/2 =2.79 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 ≈ 7.93k/3 = 2.64 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 ≈ 9.98k/4 = 2.50 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° 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 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 k at all *connections*. Common practice is to use one value of P_{nf} to determine S_{ni}, 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≈9.98 k/4=2.50 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

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

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, α_e^2, α_p^2, and λ

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

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

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

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

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

Adjustment based on strength at *exterior support,* P_{nfe}, allows S_{ni} and S_{nc} to be based on P_{nfe}.

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

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

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

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

$= \min(1.14, 1.12) = 1.12\text{ klf}$ (6% reduction vs. 1.19 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

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

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

α = 1 Conversion factor for U.S. customary units

$s = 2(e + w) + f = 2e + 2w + f$ Eq. D2.1-2

$= 1.56 \text{ in.} + 2(1.53 \text{ in.}) + 3.56 \text{ in.} = 8.18 \text{ in.}$

$$S_{nb} = \frac{7890}{(1)(6\text{ ft})^2}\left(\frac{(0.210 \text{ in.}^4/\text{ft})^3\,(0.036\text{ in.})^3(6.0\text{ in.})}{8.18\text{ in.}}\right)^{0.25} = 5.20\text{ klf} \qquad \text{Eq. D2.1-1}$$

S_{nf} = 1.19 klf < S_{nb} = 5.20 klf

Result: $S_{nb} = 5.20$ klf

Available Strength Results

Safety and resistance factors are in Section B1 (AISI S100 Table D5):

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

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

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

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

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: $S_n = 1.19$ 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.

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

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

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:

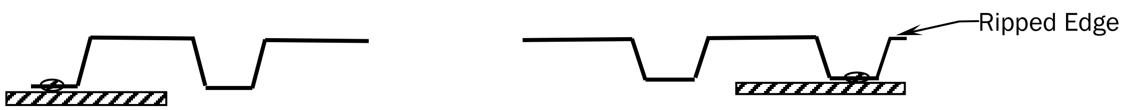
(1) Starting condition allows standard installation of full width sheet, w_e = 36 in.

(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}.



Starting Condition Ending Condition

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

$P_{nf} = P_{nfs} = 2.89$ k (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.

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

$x_{ee1} = 0$ in. $\quad x_{ee2} = 12$ in. $\quad x_{ee3} = 18$ in. 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 $n_e = n_s$

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

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

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

$$\alpha_1 = \alpha_2 = 1.00$$

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

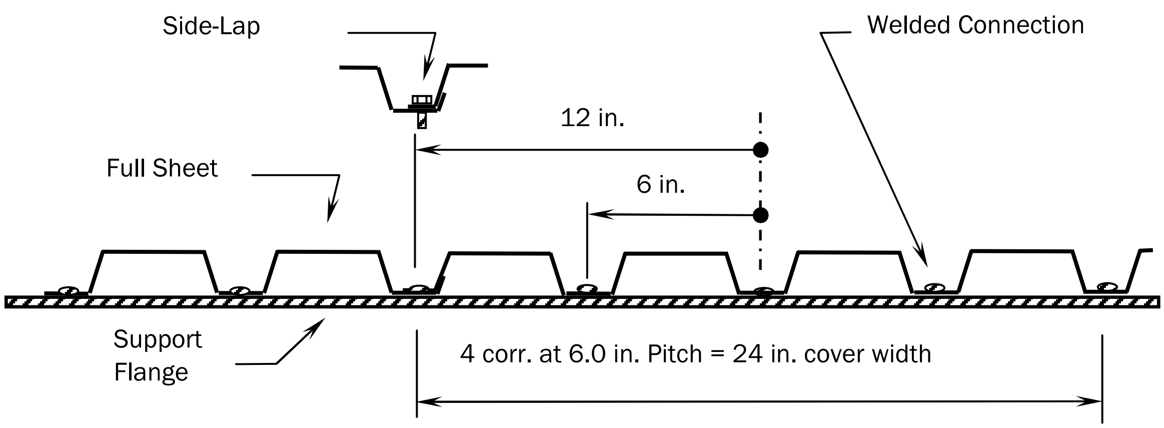
Note: End-lap weld acts as two welds along the edge—each ply develops its bearing strength, 2.89 k, and kernel strength 7.03 k > 2(2.89) k. This justifies the coefficient, 2, at α_1. See Comments at Example 1a for two thicknesses - $2.79\text{ k} \approx 2.89\text{ k}$. Simplification (2.09 klf) is 83%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

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

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

Case 2: Partial Width Sheet at Ending Condition

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



5 support welds per 24 in. wide deck with one weld common - Pattern 24/5

$x_{ee1} = 0\text{ in.} \qquad x_{ee2} = x_{ee4} = 6\text{ in.} \qquad x_{ee3} = x_{ee5} = 12\text{ in.} \qquad$ Figure D1-1

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

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

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

n_p, L, A, and λ are from Example 1a.

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

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

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

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

$S_{nf} = \min(S_{ni}, S_{nc})$
$= \min(1.28, 1.31) = 1.28\text{ klf} > S_n = 1.19\text{ klf}$ 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.

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

Example 1b Edge Detail Result:

Connection Schedule:
Support connections at panels — 3/4 in. arc spot weld, F_{XX} = 70 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*.

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

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

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

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

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

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

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

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

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

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

μ = 0.3 Poisson's ratio for steel

d = 6 in. From Deck Data

γ_c = 0.9 Table 1.3-1

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

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

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

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

$D_1 = 925$ in. From *Commentary* Table C-1.2

$D_2 = 7726$ in.

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

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

Note: To refine D to t = 0.036 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

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

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

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

Eq. 1.4-8: $\delta_{11} = 9.90\text{ in.}^3$

Eq. 1.4-9: $\delta_{12} = 4.95\text{ in.}^3$

Eq.1.4-10: $\delta_{22} = 2.87\text{ in.}^3$

Eq. 1.4-11: $\kappa_{t1} = 2.56\frac{1}{\text{in.}^3}$

Eq.1.4-22: $\delta_{t2} = 59.7\text{ in.}^{2.5}$

Eq. 1.4-25: $\delta_{b2} = 129\text{ in.}^{2.5}$

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

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

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

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

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

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

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

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

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

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, α = 1 for U.S. Customary Units

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

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

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

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

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

Number of corrugations with:

a) ψ = 1 is 2, and (Same as U_1)

b) ψ = 2 is 4 (Same as U_2)

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

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

From calculated D_1 above:

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

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

From calculated D_2 above:

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

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

Approximation result:

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

Note: G′ approximation is within 7% of $33.5 \frac{k}{in.}$ 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

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

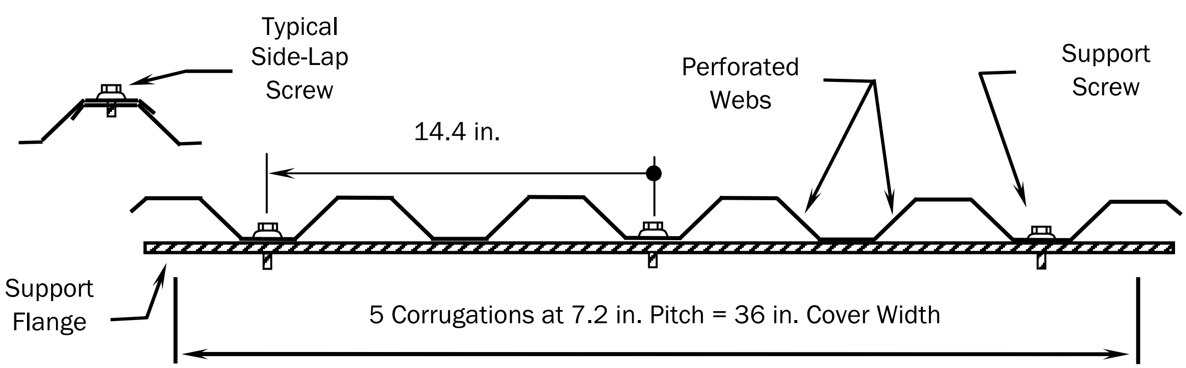
Example 2a: Nominal and Available Diaphragm Shear Strength of a Perforated Panel Connected to a Thin Support in the Absence of Uplift

Objective

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

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

Diaphragm Configuration



3 *support screws* per 36 wide panel with zero common – Pattern = 36/3

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.^{4}/ft	This is the I_x value from manufacturer of perforated panel and can be conservatively used for I_{xg}.	

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

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

Perforation Data

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

d_p = 3/16 in. diameter at spacing, c_p = 3/8 in. stagger. *Commentary* Section 1.6

$$p_o = 0.9069 \frac{{d_p}^2}{{c_p}^2} = 0.9069 \frac{0.188^2}{0.375^2} = 0.228$$ *Commentary* Eq. C-1.6-1

Steel Support Data

Yield stress, F_y	= 50 ksi	*Tensile strength*, F_u	= 65 ksi
Thickness, t	= 0.060 in.	Spacing, L_v (shear span)	= 5.00 ft

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

Connection Schedule

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

#12 screw with ½ 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 ½ 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.

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

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

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

Use U.S. Customary Units.

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

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

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

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

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:*

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

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

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

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

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

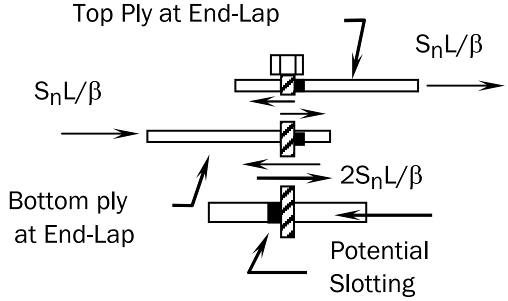
$$P_{nf} = \min(0.910,\ 2.27,\ 2.0) = 0.910\text{ k}$$

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.

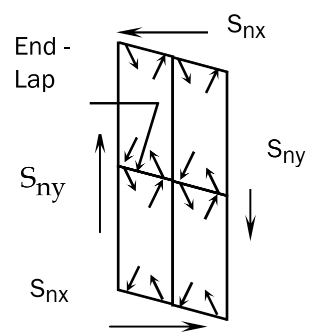
Case 2: Two-*panel* thickness *support connection:*



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.



Vectors On Panel Corner Connections

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

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

$$\frac{t_2}{t_1} = \frac{0.024\ \text{in.}}{0.024\ \text{in.}} = 1.0 \qquad \text{Therefore, } \frac{t_2}{t_1} \le 1.0$$

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

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

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

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

$P_{ns} = 2.7 t_2 d F_{u2}$ (same as above) = 0.910 k AISI S100 Eq. E4.3.1-3

$P_{nss} = 2.0\,\text{kips}$ 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}:

$A = 0.0$ Number of support screws at side-lap at *panel* ends

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

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

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

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

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

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

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

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

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

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

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

Calculate nominal diaphragm shear strength, S_{nf}:

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

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

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

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

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

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

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

Result: **S_{nf} = 0.396 klf**   Based on least support screw strength, P_{nf} = 0.910 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

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

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.

α = 1   Conversion factor for U.S. customary units

$s = 2(e + w_p) + f = 2e + 2w_p + f$ — Eq. D2.1-2

$w_p = K_{E_w}^{1/3} w$ — Eq. 1.6-3

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

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

$k = 0.9 + p_o^2 - 1.875 p_o$ (for $0.2 \le p_o \le 0.58$) — Eq. 1.6-5

$= 0.9 + (0.228)^2 - 1.875(0.228) = 0.524$ See Panel Data for p_o

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

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

s = 2.0 in. + 2(2.32 in.) + 2.0 in. = 8.64 in. — Eq. D2.1-2

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

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

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

Available Strength

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

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

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

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

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

Discussion:

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

ASD:

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

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

LRFD:

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

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

LSD:

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

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

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: $S_n = 0.396$ 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: $\dfrac{S_n}{\Omega_d}$ 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	for one-thickness pull-over
	= 2(0.060 klf)	
	= 0.120 klf	for pull-out at end-lap

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

AISI S100 Section 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)

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

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 screw/ft at all supports (See Example 2a)

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

$$P_{nov} = 1.5 t_1 d'_w F_{u1} \qquad \text{AISI S100 Section E4.4.2}$$

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

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

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

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

$$= 0.865$$

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

Result: P_{nft} = 0.865(0.910 k) = 0.787 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_{nc} concerns corner connection at end-lap.

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

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

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

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

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

Calculate S_{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_{nfe} = *Nominal connection strength* at *exterior supports* in one ply, controlled by pull-over

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

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

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

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

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

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

P_{nfi} = 0.787 k P_{ns} = 0.472 k (Example 2a, and not affected by uplift.)

P_{nfe} = 0.845 k

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

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

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

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

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

= 0.425 klf

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

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

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

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

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

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

Result controlled by pull-over:

$S_{nf} = 0.380$ klf 4% reduction vs. 0.396 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.

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

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

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

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

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

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

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

Therefore,

$P_{not} = 0.716$ k

At *interior supports*:

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

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

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

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

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

At *exterior supports*:

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

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

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

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

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

Calculate S_{nf} controlled by pull-out:

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

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

$P_{nfe} = 0.606$ k

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

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

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

$$\beta = 16(1.00) + 2(4)(0.32) + 4(0.32)\frac{0.606\ \text{k}}{0.471\ \text{k}}$$

$$= 20.2$$

$$S_{ni} = [2(0.0)(0.798 - 1)\frac{0.606\,\text{k}}{0.471\,\text{k}} + 20.2]\frac{0.471\ \text{k}}{25\ \text{ft}} \qquad \text{Modified Eq. D1-1}$$

$$= 0.381\ \text{klf}$$

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

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

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

$$\beta = 16(0.779) + 2(4)0.32\frac{0.471\ \text{k}}{0.606\ \text{k}} + 4(0.32)$$

$$= 15.7$$

$$S_{nc} = \left(\frac{(1)^2(15.7)^2\dfrac{1}{\text{ft}^2}}{(25)^2(1)^2\dfrac{\text{ft}^2}{\text{ft}^2} + (15.7)^2}\right)^{0.5} 0.606\,\text{k} \qquad \text{Eq. D1-2}$$

$$= 0.322\ \text{klf}$$

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

Result controlled by pull-out:

$S_{nf} = 0.322\ \text{klf}$ 18.7% reduction vs. 0.396 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 k), P_{nf} = 0.910 k from Example 2a is used to conservatively calculate P_{nft}.

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

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

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

At *interior supports,* T = 0.165 k/screw, and from the interaction above:

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

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

P_{nft} = 1(0.910) = 0.910 k (no reduction)

At *exterior supports*, T = 0.120 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 k (No reduction)

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

$S_{nf} = 0.396$ klf Same as no uplift—See Example 2a

Example 2b Result (With Uplift)

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

Nominal Strength: $S_n = 0.322$ 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.

Ω_d = 2.35 for wind load

Available Strength: $\dfrac{S_n}{\Omega} = \dfrac{0.322}{2.35} = \mathbf{0.137\ klf}$

Example 2b Discussion:

The following calculation uses a simpler design approach by using only one reaction force at the exterior support, T = 0.165 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 k/conn.

P_{nft} = 0.787 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 k From Example 2a

P_{nft} = 0.629(0.910) = 0.572 k

Calculate S_{nf} controlled by pull-over:

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

P_{nfi} = 0.787 k $\quad$ P_{ns} = 0.472 k (Example 2a, and not affected by uplift)

P_{nfe} = 0.572 k

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

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

$\alpha_s = 0.600$ Based on P_{nfi} = 0.787 k from (a) above 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 klf

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

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

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

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

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

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

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 k/conn

P_{nft} = 0.518(0.910 k) = 0.471 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:

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

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

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

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 k/conn

P_{nft} = 1.0(0.910) = 0.910 k

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

a) Setting α_1 and α_2 to 1 in Eq. D1-3, and

b) 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 α_1 and $\alpha_2 > 1$.

The following conditions apply at the detail:

a) Starting and ending conditions allow installation of full-width sheets, w_e = 36 in.

b) Starting and ending conditions do not allow a bottom flat on the edge support angle.

c) 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.

d) #12 side-lap screw with 1/2 in. round sealing washer will be used at *panel* to Zee.

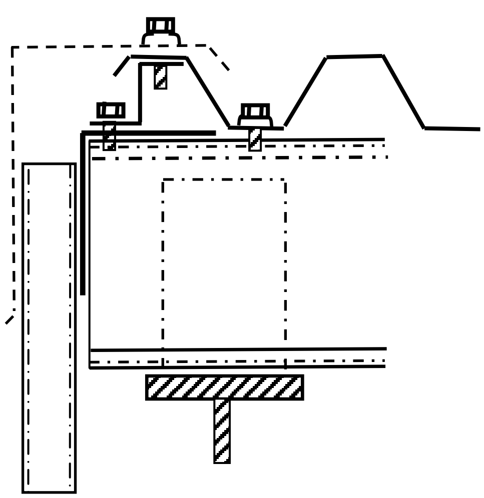
e) #12 support screw without washer will connect 1 ½ in. deep Zee to edge support.

f) Try to match side-lap screw spacing = 20 in. so that $n_e = n_s$.

g) Try to use reinforcing zee with t = 0.06 in., F_y = 50 ksi, and F_u = 65 ksi.

h) 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:

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

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

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 ft

Zee-to-edge support angle:

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

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

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

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

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

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

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:

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

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: $\Omega = \Omega_d = 2.35$

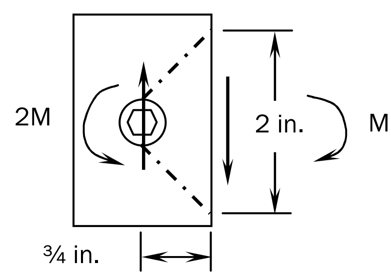
ASD Available diaphragm shear strength at edge $= \dfrac{0.396 \text{ klf}}{2.35} = 0.169 \text{ klf}$

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.

$$\tau_v = \frac{0.169 \frac{\text{k}}{\text{ft}}}{12(0.06)\frac{\text{in.}^2}{\text{ft}}} = 0.235 \text{ ksi}$$ 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.

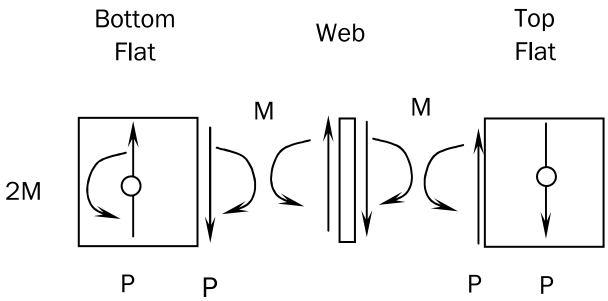
Calculate St. Venant shear in Zee web due to twisting (screw eccentricity):

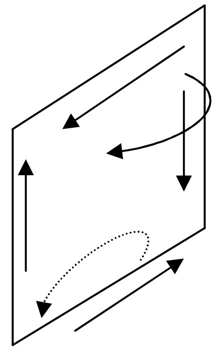
Note: Refer to *Advanced Mechanics of Materials* by Seely and Smith or other textbooks for torsional resistance of rectangular cross-sections:

ASD Available Diaphragm Strength = 0.169 klf

P = 0.169 klf Shear transfer to Zee

M = (0.75 in.)(0.169 klf) = 0.127 in. k/ft





Web

$$\tau_{SVv} = \frac{3M}{bt^2} = \frac{3(0.127)\,\frac{\text{in.k}}{\text{ft}}}{12(0.06)^2\,\frac{\text{in.}^3}{\text{ft}}} = 8.82 \text{ ksi}$$

Calculate resultant shear stress at outer edges of web:

$$f_v \le \tau_v + \tau_{SVv} = 5.63 + 8.82 = 14.5 \text{ ksi}$$

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

$$\frac{h}{t} = \frac{1.5 \text{ in.}}{0.06 \text{ in.}} = 25$$

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

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

Result: Since 18.8 ksi > 14.5 ksi, t = 0.06 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)

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

Center screw (at pivot point of twist)

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

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

Outer screws (4 ft apart)

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

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

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

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

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

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

Example 2c Results

Use: 1-1/2 in. Zee with t = 0.06 in. and F_y = 50 ksi Typical length = 10 ft.

Panel to Zee: #12 × 1 in. long *side-lap* screw with ½ 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 ft	3	12 in., 4 ft - 0 in., 4 ft - 0 in., 12 in. = 10 ft
5 ft < L_z < 10 ft	3	6 in., 3 ft - 0 in., 3 ft - 0 in., 6 in. = 7 ft
$L_z \le$ 5 ft	2	6 in., 4 ft - 0 in., 6 in. = 5 ft

$\mathbf{S_n = 0.396}$ **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.

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

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

$$\alpha_3 = \alpha_4 = \frac{\sum x_e}{w}$$ See Example 2a for parameters

$$= \frac{0 + 2(14.4)}{36} \frac{\text{in.}}{\text{in.}} = 0.8 \qquad \text{Eq. D5.1.1-3}$$

Note: G′ should be evaluated using *interior panels*.

Calculate screw *flexibilities*:

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

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

$$K = \frac{S_f}{S_s} = \frac{0.00839}{0.0194} = 0.433$$ 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.

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

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

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

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

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.

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

$2e_p$ = 2e (no perf.) = 2.00 in. f_p = f (no perf.) = 2.00 in.

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

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

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

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

$$\delta_{22} = \frac{1}{12}\left(\frac{D_d}{d}\right)^2\left[s\left(4e_p^2 - 2e_p f_p + f_p^2\right) + d^2\left(3f_p + 2w_p\right)\right] \qquad \text{Eq. 1.4-10}$$

$$= \frac{1}{12}\left(\frac{1.5\text{in.}}{7.2\text{in.}}\right)^2\left[8.64\text{in.}\left(4(1\text{in.})^2 - 2(1\text{in.})(2\text{in.}) + (2\text{in.})^2\right) + (7.2\text{in.})^2\left(3(2\text{in.}) + 2(2.32\text{in.})\right)\right]$$

$$= 2.12 \text{ in.}^3$$

Calculate spring constant indicators:

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

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

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

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

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

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

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

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

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

$$= 0.062\frac{1}{\text{in.}^3}$$

$$\kappa_{tc4} = \frac{1}{\left(1 + \frac{3e}{f}\right)\delta_{11} + \delta_{22} + \left(1 + \frac{e}{f}\right)\delta_{12}} \qquad \text{Eq. 1.4-19}$$

$$= \frac{1}{\left(1 + \frac{3(1\text{ in.})}{2\text{ in.}}\right)7.98\text{ in.}^3 + 2.12\text{ in.}^3 + \left(1 + \frac{1\text{ in.}}{2\text{ in.}}\right)3.99\text{ in.}^3} = 0.036\frac{1}{\text{in.}^3}$$

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

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.:

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

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

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

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

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

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

Eq. 1.4-26

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

Eq. 1.4-27

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

Eq. 1.4-28

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

Eq. 1.4-29

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

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.:

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

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

$$\gamma_3 = 2\,\delta_{t3} + \delta_{tc3} + 2\left(\frac{2e}{f}\right)\delta_{b3}$$ Eq. 1.4-33

$$= 2(80.8\,\text{in.}^{2.5}) + 135\,\text{in.}^{2.5} + 2\left(\frac{2\,\text{in.}}{2\,\text{in.}}\right)(133\,\text{in.}^{2.5}) = 563\,\text{in.}^{2.5}$$

$$\gamma_4 = 2\,(\delta_{t4} + \delta_{tc4}) + \left(\frac{2e}{f}\right)(2\,\delta_{b4} + \delta_{bc4})$$ Eq. 1.4-34

$$= 2\left(108\,\text{in.}^{2.5} + 203\,\text{in.}^{2.5}\right) + \left(\frac{2\,\text{in.}}{2\,\text{in.}}\right)\left(2(181\,\text{in.}^{2.5}) + 227\,\text{in.}^{2.5}\right) = 1210\,\text{in.}^{2.5}$$

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:

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

$U_1 = 1$ (number of flutes with screw spacing = 7.2 in.)

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

$U_2 = 4$ (number of flutes with screw spacing = 14.4 in.)

$U_3 = 0$ in this *configuration* $U_4 = 0$ in this *configuration*

Calculate weighted average warping value, D, for panel:

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

Calculate warping factor, D_n:

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

Result warping factor: $D_n = 17.8$

$\gamma_c = 0.71$ 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*.

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

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

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

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

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

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

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:

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

γ_c = 0.9 Table 1.2-1

$$G' = \left(\frac{29500\dfrac{\text{k}}{\text{in.}^2}(0.024)\,\text{in.}}{2(1+0.3)\dfrac{10.77}{7.2} + 0.9(29.7) + 5.31} \right) 0.433 = 8.53\frac{\text{k}}{\text{in.}}$$

(less stiffness)
39% decrease
vs.
14.0 k/in.

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.

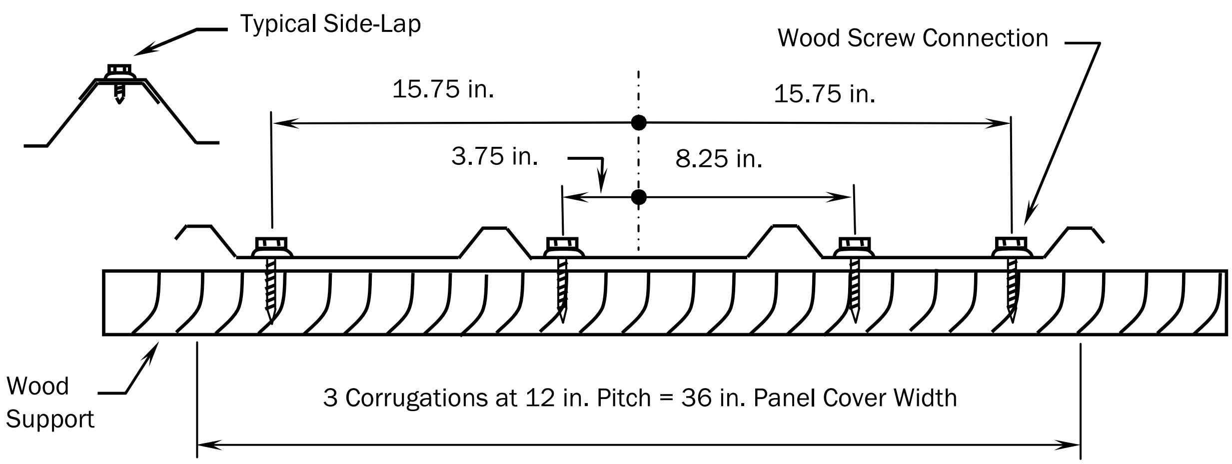
Example 3a: Nominal and Available Diaphragm Shear Strength and Stiffness of a Panel on Wood Supports in the Absence of Uplift

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



4 support screws per 36 in. wide panel with zero common – Screw pattern 36/4

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.^{4}/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 × 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 × 1 in. long with ½ 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.

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

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

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

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

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

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

h_s = 2 - 0.125 in. = 1.87 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 in.) = 0.992 in. See Connection Schedule
7d = 7(0.248 in.) = 1.74 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 $4d \le h_s < 7d$

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

For $h_s \ge 7d$

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

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

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

$P_{nfws} = 2.7t_1dF_{u1}$ Table D1.1.4.2-1

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

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

Result: $P_{nf} = \min(0.887k, 0.996k, 2.70k) = 0.887k$ 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):

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

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

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

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

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

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

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

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

Calculate configuration parameters required for S_{nf}:

$A = 0.0$ Number support screws at side-lap at *panel* ends

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

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

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

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

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

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

$\alpha_s = \dfrac{P_{ns}}{P_{nf}} = \dfrac{0.450\text{ k}}{0.887\text{ k}} = 0.507$ *Connection strength* ratio Eq. D1-6

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

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

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

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

Calculate Nominal Diaphragm Shear Strength, S_{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_{nf} equals 0.887 k. End-lap strength and alternate shear planes will be considered later.

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

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

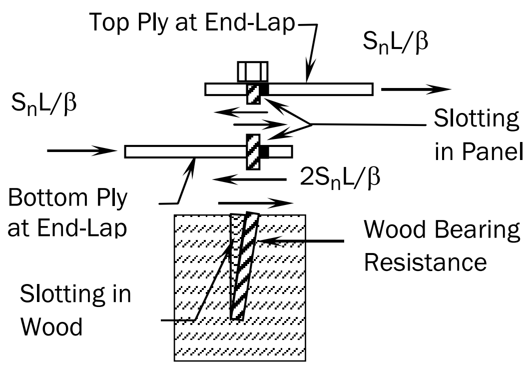
Result at Butt-Joint (Section D1):

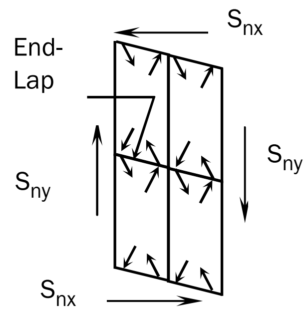
$S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.470 \text{ klf}, 0.437 \text{ klf}) = 0.437 \text{ klf}$

Based on P_{nf} = 0.887 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.





Force Vectors on Panel Corner Connections

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 k:

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

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

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

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

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

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

5.5% less than S_{ni} = 0.470 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:

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

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

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

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

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

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

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

α = 1 Conversion factor for U.S. customary units

s = 2e + 2w + f = 9 in. + 2(1.6 in.) + 1 in. = 13.2 in. 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: $\mathbf{S_{nb} = 2.02\ klf}$

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

Result at end-lap installations (No uplift):

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

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

$$\phi S_n = \min(0.55(0.444), 0.80(2.02)) = 0.244\text{ klf}$$

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.

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

K = 0.5	For *panel* on wood supports	E = 29500 ksi
s = 13.2 in.	From determination of S_{nb}	μ = 0.3 (Poisson's ratio for steel)
d = 12 in.	From the Panel Data	t = 0.024 in.

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

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

$$D = \frac{U_1D_1 + U_2D_2 + U_3D_3 + U_4D_4}{U_1 + U_2 + U_3 + U_4} \qquad \text{Eq. 1.4-2}$$

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

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

Note: D1 is calculated using the detailed method in Example 2d. The designer should verify κ_{t1} and expect some round-off variation:

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

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

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

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

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

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

Result:

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

$$\gamma_c = 0.64$$ Table 1.3-1

Calculate slip constant, C:

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

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

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

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

$$S_s = \frac{3.0\alpha}{1000\sqrt{t}} = \frac{3.0(1)}{1000\sqrt{.024\text{ in.}}} = 0.0194\frac{\text{in.}}{\text{kip}}$$ Eq. D5.2.2-2

Result of slip constant, C:

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

Result of *diaphragm* stiffness, G':

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

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:**

$S_n = 0.437$ 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:**

$S_{available}$ = 0.437 klf/3.00 = 0.146 klf (*ASD*)
= (0.55)(0.437 klf) = 0.240 klf (*LRFD*)
= (0.50)(0.437 klf) = 0.219 klf (*LSD*)

***Diaphragm stiffness*:**

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

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 *interior support*	= 1.1(5 ft)(0.05 ksf)	
	= 0.275 klf	Pull-over and pull-out
Reaction at *exterior support*	= 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}

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

$\Omega = 2.35$ $\quad$ $\Omega_d = 3.00$ (Wood) $\quad$ (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*:

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

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

$$P_{nov} = 1.5t_1d'_wF_{u1}$$ 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 in. (screw head diameter for #14 support screw)
t_w = 0.040 in. (washer thickness estimate)

$$d'_w = d_h + 2t_w + t_1 \le d_w$$ 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 in. per AISI S100 Section E4.4.2.

For one-*panel* thickness and uniform pull-over,

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

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

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

$P_{nf} = \min(0.996\ \text{k},\ 2.70\ \text{k}) = 0.996\ \text{k}$ 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 $P_{nf.}$ However, wood bearing is included at pull-out interaction - See (b) below.

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

P_{nft} = 1.0(0.996 k) = 0.996 k no reduction

At *exterior supports*:

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

For one-*panel* thickness and eccentric pull-over,

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

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

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

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

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

P_{nft} = 1.0(0.996 k) = 0.996 k   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:

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

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

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

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

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

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

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

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

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

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

Ω_d = 3.00

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

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

At *interior supports*:

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

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

$\frac{P_{nT}}{\Omega} = \frac{0.578\text{ k}}{3} = 0.193\text{ k} < 0.207\text{ k}$ 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 in. = 2.37 in. See h_s note in Example 3a

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

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

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

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

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

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

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

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

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

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

P_{nft} = 0.475 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):

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

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

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

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

P_{nT} = 0.733 k from above

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

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

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

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

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

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

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

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

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

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

P_{nft} = 0.700 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 k/ply At end-lap (shank limits strength)

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

To determine S_{nc}, use:

P_{nfe} = 0.996 k/ply At the end-lap

P_{nfi} = 0.475 k 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 k, P_{nfe} = 0.350 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.

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

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

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

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

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

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

$= 0.377$ klf

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

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

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

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

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

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

$= 0.396$ 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)

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

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

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

At *interior supports*:

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

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

At *exterior support*:

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

$$\left(\frac{P_{nft}}{P_{nf}}\right) = 1.3 - \left(\frac{3(0.150\ k)}{3.2\ k}\right) = 1.16$$ No reduction 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.

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

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

$$S_{nf} = \min(0.444,\ 0.377,\ 0.444) = 0.377\ \ klf$$ 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:*

$T = 0.207$ k/screw $P_{nft} = 0.996$ k/ connection Same as end-lap

No reduction

At *exterior supports:*

$T = 0.075$ k/screw $P_{nft} = 0.996$ k/ connection Same as end-lap

No reduction

Result of Case (a) with butt-joints:

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

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

At interior supports:

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

At exterior supports:

$T = 0.075$ k/screw

Using same iterative process as at end-lap:

Assume: P_{nft} = 0.845 k

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

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

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

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

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

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

Result: P_{nft} = 0.845 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 k, P_{nfe} = 0.845 k:

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

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

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

$$= 25.6$$

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

$$= 0.405\text{ klf}$$

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

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

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

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

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 k/screw Same as end-lap with no reduction

At *exterior supports:*

T = 0.075 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:

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

Example 3b Results (With 50 psf Uplift)

Nominal diaphragm shear strength per unit length:

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

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

Available diaphragm strength per unit length:

Ω_d = 3.00 (Wood)

ASD S_a = 0.382 klf/3 = 0.127 klf Use butt-joint result for design

Diaphragm stiffness:

$G' = 42.5\ \text{k}/\text{in.}$ 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 = 2-½ 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:

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

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

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):

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

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

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

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

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):

$P_{nf} = \min(0.887\,k, 0.996\,k, 2.70\,k) = 0.887\,k$ from Example 3a 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: $S_{nf} = 0.285$ 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,

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

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

Note: 0.575 k includes AISI S100 prying adjustment.

At *interior supports*:

T = 0.207 k/screw

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

Ω = 3.00

Ω = 2.50 for rational engineering at connections (AISI S100 Section A1.2(c))

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

At *exterior supports*:

T = 0.075 k/screw

Available tension strength:

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

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

Determine the *pull-out nominal strength* of the #14 × 2-½ in. long screw*:*

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

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

At *interior supports*:

T = 0.207 k/screw

Available tension strength:

$$\frac{P_{not}}{\Omega} = \frac{0.733\text{ k}}{3.00} = 0.244\text{ k} > 0.207\text{ k} \quad \text{OK for pull-out}$$

At *exterior supports*:

T = 0.150 k/screw Two plys at end-lap

Available tension strength:

$$\frac{P_{not}}{\Omega} = \frac{0.733\text{ k}}{3.00} = 0.244\text{ k} > 0.150\text{ k} \quad \text{OK for pull-out}$$

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):

Ω_t = 3.0

$$\frac{P_{nts}}{\Omega} = \frac{3.2\text{ k}}{3.00} = 1.07\text{ k} > 0.207\text{ k} > 0.150\text{ k} \quad \text{OK for breaking strength}$$

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 klf With or without uplift—Applies at butt-joint or end-lap installations

Available diaphragm strength per unit length:

Ω_d = 3.00 *ASD* (Wood) Section D1.1.4.1

ϕ_d = 0.55 *LRFD*

ϕ_d = 0.50 *LSD*

S_a = 0.285 klf/3 = 0.095 klf (*ASD*)

= (0.55)(0.285 klf) = 0.157 klf (*LRFD*)

= (0.50)(0.285 klf) = 0.143 klf (*LSD*)

Use: The screw pattern shown in the *configuration* on page 1 of Example 3a.
Screw length = 2-½ 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:

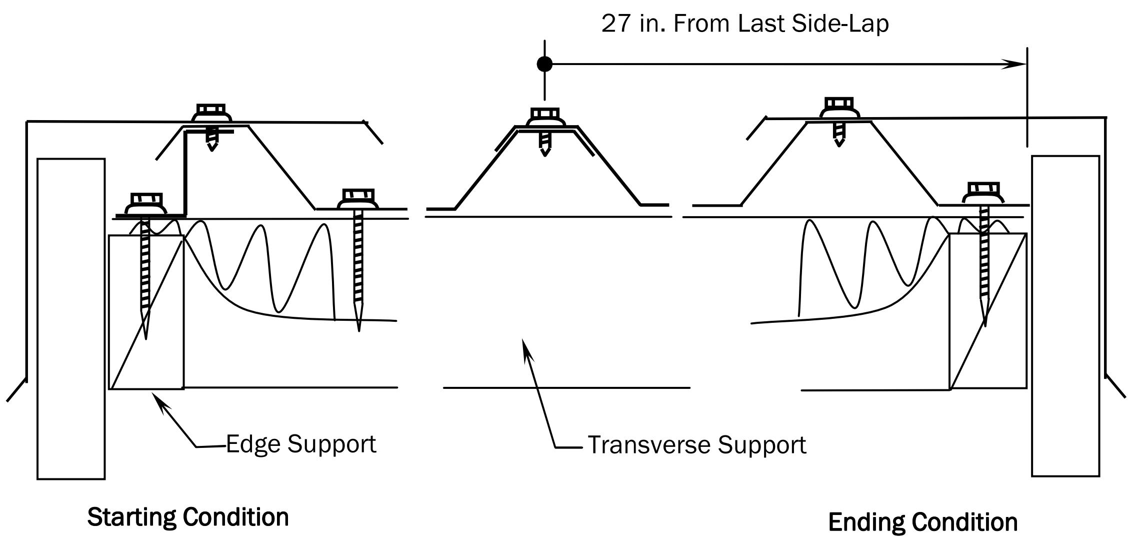
Part 1 - With insulation from Example 3c, 0.285 klf (with or without uplift), and

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:

a) Starting condition allows installation of full-width sheets, w_e = 36 in., but does not allow a bottom flat on the edge support.

b) 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 1-¼ 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:

Try: t = 0.036 in. F_y = 50 ksi F_u = 65 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)

$$\frac{t_2}{t_1} = \frac{0.036\text{ in.}}{0.024\text{ in.}} = 1.5 \qquad 1 < \frac{t_2}{t_1} < 2.5$$

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

$$= 4.2\left((0.036\text{ in.})^3(0.216\text{ in.})\right)^{1/2}\left(65\frac{\text{k}}{\text{in.}^2}\right) = 0.867\text{ k}$$

$$P_{ns} = 2.7t_1 dF_{u1} \qquad \text{AISI S100 Eq. E4.3.1-4}$$

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

$$P_{ns} = 2.7t_2 dF_{u2} \qquad \text{AISI S100 Eq. E4.3.1-5}$$

$$= 2.7(0.036\text{ in.})(0.216\text{ in.})\left(65\frac{\text{k}}{\text{in.}^2}\right) = 1.36\text{ k}$$

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

$$\frac{t_2}{t_1} = 1.0 \qquad P_{ns} = \min(0.867\text{k}, 0.868\text{k}, 1.36\text{k}, 2.00\text{k}) = 0.867\text{k}$$

$$\frac{t_2}{t_1} = 2.5 \qquad P_{ns} = \min(0.868\text{k}, 1.36\text{k}, 2.00\text{k}) = 0.868\text{k}$$

$$\frac{t_2}{t_1} = 1.5 \qquad P_{ns} = 0.867\text{ k} + \frac{1.5 - 1.0}{2.5 - 1}(0.868\text{ k} - 0.867\text{ k}) = 0.867\text{ k}$$

Result: $P_{nfs} = 0.867\text{kips}$ 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).

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

$$P_{nfws} = 2.7t_1 dF_{u1} = 2.7(0.036\text{ in.})(0.248\text{ in.})\left(65\frac{\text{k}}{\text{in.}^2}\right) = 1.57\text{ k} \qquad \text{Table D1.1.4.2-1}$$

$P_{nss} = 2.7\text{ k}$ 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.

$$n_e = \frac{S_n L}{P_{nfs}} \quad \text{or} \quad \frac{L}{n_e} = \frac{P_{nfs}}{S_n} = \frac{0.887\,\frac{k}{conn.}}{0.285\,\frac{k}{ft}} = 3.11\,\frac{ft}{conn.} \qquad \text{Eq. D4.4-1}$$

Calculate required screw spacing between panel and Zee top flat:

$$\frac{L}{n_e} = \frac{P_{nfs}}{S_n} = \frac{0.867\,\frac{k}{conn.}}{0.285\,\frac{k}{ft}} = 3.04\,\frac{ft}{conn.} > 20 \text{ in. o.c.}$$

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.

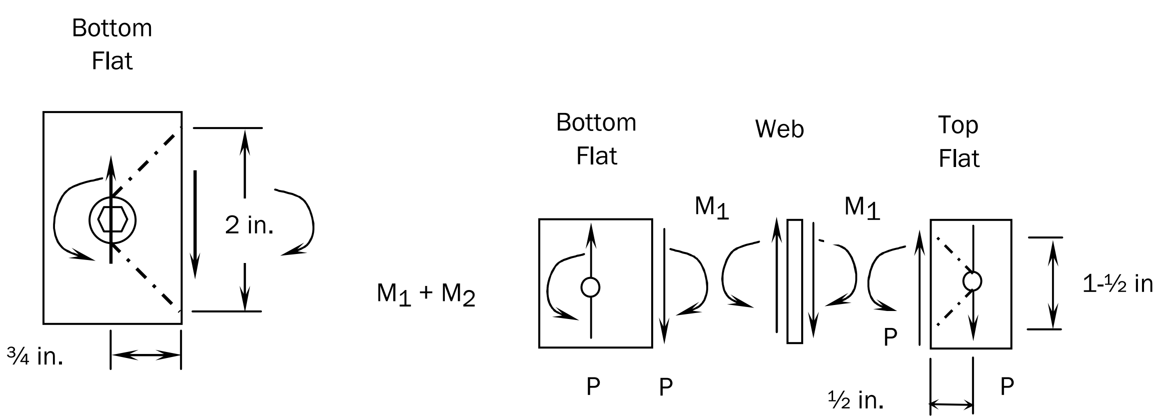
Calculate shear stress in reinforcing Zee:

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_1 = 0.5 in.(0.095 klf) = 0.048 in. k/ ft
M_2 = 0.75 in.(0.095 klf) = 0.071 in. k/ ft



Stress in Top Flat of Zee

$$\tau_v \le \frac{(1.67\ \text{ft})(0.095)\dfrac{\text{k}}{\text{ft}}}{1.5(0.036)\ \text{in.}^2} = 2.94\ \text{ksi}$$

Stress in Bottom Flat of Zee

$$\tau_v \le \frac{(3.0\ \text{ft})(0.095)\dfrac{\text{k}}{\text{ft}}}{2(0.036)\ \text{in.}^2} = 3.96\ \text{ksi}$$

Calculate St. Venant shear stress in Zee web (see Example 2c):

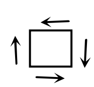
$$\tau_{SVv} = \frac{3M_1}{bt^2} = \frac{3(0.048)\dfrac{\text{in.k}}{\text{ft}}}{12(0.036)^2\dfrac{\text{in.}^3}{\text{ft}}} = 9.26\ \text{ksi}$$

Calculate total shear stress in Zee web:

$f_v \le \tau_v + \tau_{SV_v} = 3.96 + 9.26 = 13.2\ \text{ksi}$ At outer edges of web

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

$$\frac{h}{t} = \frac{1.25\ \text{in.}}{0.036\ \text{in.}} = 34.7$$



For $\dfrac{h}{t} \le \sqrt{\dfrac{Ek_v}{F_y}}$ $\quad F_v = 0.6F_y$

$$\sqrt{\frac{Ek_v}{F_y}} = \sqrt{\frac{29500\ \text{ksi}\,(5.34)}{50\ \text{ksi}}} = 56.1 > 34.7$$

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

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 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:

$P_{nf} = P_{nfs} = \min(0.887\,k, 0.996\,k, 2.70\,k) = 0.887\,k$ Screw bearing against wood controls

$$\frac{L}{n_e} = \frac{P_{nfs}}{S_n} = \frac{0.887\frac{k}{conn.}}{0.285\frac{k}{ft}} = 3.11\frac{ft}{conn.}$$

Result of Part 1 at ending condition with insulation:

Screw spacing along edge support:

Use: #14 screw × 2-½ 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 1-¼ 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)

$$\frac{t_2}{t_1} = \frac{0.048\text{ in.}}{0.024\text{ in.}} = 2.0 \qquad 1 < \frac{t_2}{t_1} < 2.5$$

$$P_{ns} = 4.2(0.048\text{ in.})^3(0.216\text{ in.})^{1/2}(62\frac{\text{k}}{\text{in.}^2}) = 1.33\text{ k} \qquad \text{AISI S100 Eq. E4.3.1-1}$$

$$P_{ns} = 2.7(0.024\text{ in.})(0.216\text{ in.})(62\frac{\text{k}}{\text{in.}^2}) = 0.868\text{ k} \qquad \text{AISI S100 Eq. E4.3.1-4}$$

$$P_{ns} = 2.7(0.048\text{ in.})(0.216\text{ in.})(65\frac{\text{k}}{\text{in.}^2}) = 1.82\text{ k} \qquad \text{AISI S100 Eq. E4.3.1-5}$$

$P_{nss} = 2.0\text{ kips}$ From Connection Schedule in Example 3a

$$\frac{t_2}{t_1} = 1.0 \qquad P_{ns} = \min(1.33\text{k}, 0.868\text{k}, 1.82\text{k}, 2.00\text{k}) = 0.868\text{k}$$

$$\frac{t_2}{t_1} = 2.5 \qquad P_{ns} = \min(0.868\text{k}, 1.82\text{k}, 2.00\text{k}) = 0.868\text{k}$$

$$\frac{t_2}{t_1} = 2.0 \qquad P_{ns} = 0.868\text{ k} + \frac{2.0 - 1.0}{2.5 - 1}(0.868\text{ k} - 0.868\text{ k}) = 0.868\text{ k}$$

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.

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

$$P_{nfws} = 2.7(0.048\text{ in.})(0.248\text{ in.})(65\frac{\text{k}}{\text{in.}^2}) = 2.09\text{ k} \qquad \text{Table D1.1.4.2-1}$$

$P_{nss} = 2.7\text{ k}$ From Connection Schedule in Example 3a

Result: $P_{nfs} = \min(0.887\text{ k}, 2.09\text{ k}, 2.70\text{ k}) = 0.887\text{ k}$
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:

$\mathit{ASD\ Available\ Diaphragm\ Strength} = 0.146\text{ klf}$ 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:

$$\tau_v \le \frac{1.67\text{ ft}(0.146)\frac{\text{k}}{\text{ft}}}{1.5(0.048)\text{ in.}^2} = 3.39\text{ ksi}$$

Stress in bottom flat of Zee:

$$\tau_v \le \frac{2.00\text{ ft}(0.146)\frac{\text{k}}{\text{ft}}}{2(0.048)\text{ in.}^2} = 3.04\text{ ksi}$$

Calculate St. Venant shear stress in Zee web (See Example 2c):

$$\tau_{SVv} = \frac{3M_1}{bt^2} = \frac{3(0.073)\frac{\text{in.k}}{\text{ft}}}{12(0.048)^2\frac{\text{in.}^3}{\text{ft}}} = 7.92\text{ ksi}$$

Calculate total shear stress in Zee web:

$f_v \le \tau_v + \tau_{SV_v} = 3.39 + 7.92 = 11.3\text{ ksi}$ At outer edges of web

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

$$\frac{h}{t} = \frac{1.25 \text{ in.}}{0.048 \text{ in.}} = 26.0$$ See the check in Example 3d Part 1 with t = 0.036 in.

$F_v = 0.6(50 \text{ ksi}) = 30 \text{ ksi}$ *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, τ_v = 4.52 ksi, τ_{SVv} = 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:

$P_{nf} = P_{nfs} = \min(0.887 \text{ k}, 0.996 \text{ k}, 2.70 \text{ k}) = 0.887 \text{ k}$ Screw bearing against wood controls

Calculate required screw spacing between ripped *edge panel* and edge support:

$$\frac{L}{n_e} = \frac{P_{nfs}}{S_n} = \frac{0.887 \frac{k}{\text{conn.}}}{0.437 \frac{k}{\text{ft}}} = 2.03 \frac{\text{ft}}{\text{conn.}}$$

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

Install at 1/3 points between transverse supports - $\frac{5 \text{ ft}}{3} = 1.67 \text{ ft} < 2.03 \text{ ft}$

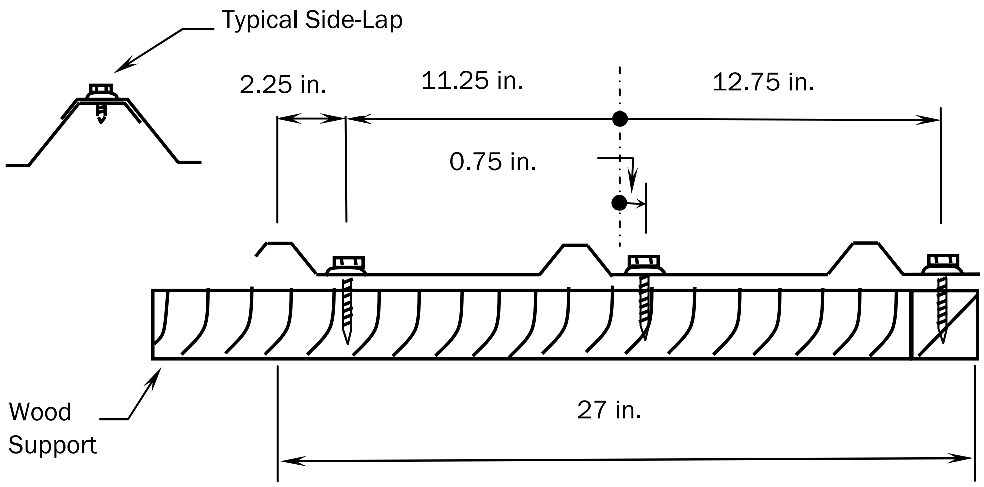
Calculate *diaphragm strength* of partial-width *panel*:

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.)

$$\alpha_e^2 = \alpha_p^2 = \left(\frac{1}{(27 \text{ in.})^2}\right)\left((0.75 \text{ in.})^2 + (12.75 \text{ in.})^2 + (11.25 \text{ in.})^2\right) = 0.397$$

$$N = \frac{3 \text{ screws}}{2.25 \text{ ft}} = 1.33 \frac{\text{screws}}{\text{ft}}$$

$$n_s = \frac{30 \text{ ft}(12 \text{ in.}/\text{ft})}{15 \frac{\text{in}}{\text{conn.}}} + 1.0 = 25$$



See Example 3a for parameters: $P_{ns}, \alpha_s, n_p, A, \lambda$

$$\beta = 25(0.507) + 2(5)(0.397) + 4(0.397) = 18.2 \qquad \text{Eq. D1-5}$$

$$S_{ni} = [(2)(0.0)(0.832 - 1) + 18.2]\frac{0.887\ \text{k}}{30\ \text{ft}} = 0.538\ \text{klf}$$

$$S_{nc} = \left(\frac{(1.33)^2(18.2)^2\ \dfrac{1}{\text{ft}^2}}{(30)^2(1.33)^2\ \dfrac{\text{ft}^2}{\text{ft}^2} + (18.2)^2}\right)^{0.5} 0.887 \quad \text{k} = 0.490\ \text{klf}$$

Result: $S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.538, 0.490) = 0.490\,\text{klf}$

based on P_{nf} = 0.887 k > 0.437 klf **OK** butt-joint case used

Note: If additional side-lap screws are not added (keep spacing = 20 in. o.c.), β = 15.2 and S_{nf} = 0.420 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.

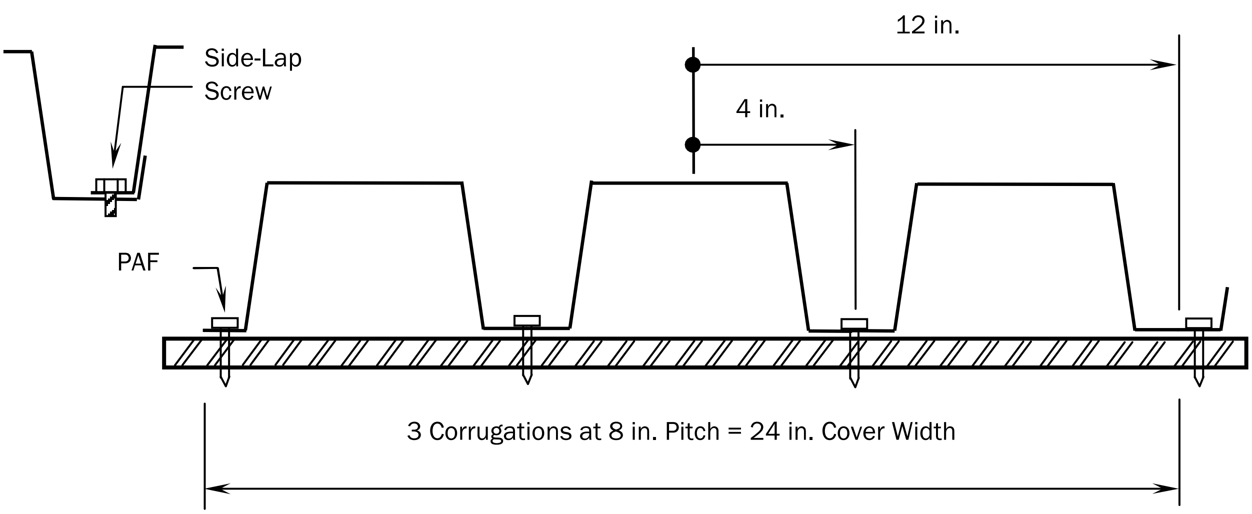
Example 4a Uses a minimal amount of *side-lap* screws with a spacing – 3 ft o.c.

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.



4 support PAFs per 24 in. wide DR deck with one common – Pattern = 24/4

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.^{4}/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 ≤ 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 ¼ in. and thicker steel supports.

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

Safety and *resistance factors* are in Section B1.

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

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

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

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

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

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: $0.028 \text{ in.} \leq t \leq 0.060$ in.

$$P_{nf} = 60t(1 - 2t), \text{k} \quad \text{for t in in.} \qquad \text{Manufacturer-provided equation}$$

$$= 60(0.036)(1 - 2(0.036)) = 2.00 \text{ k}$$

$P_{npa} = 3.0$ k 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):

$$\frac{t_2}{t_1} = \frac{0.036 \text{ in.}}{0.036 \text{ in.}} = 1.0 \text{ Therefore, } \frac{t_2}{t_1} \leq 1.0 \qquad F_{u2} = F_{u1}$$

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

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

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

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

P_{nss} = 1.5 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}:

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

$\lambda = 1 - \dfrac{D_d L_v}{240\sqrt{t}} \geq 0.7$ Required input units are defined in AISI S310 Eq. D1-4a

$= 1 - \dfrac{(3\text{ in.})(12.0\text{ ft})}{240\sqrt{0.036\text{ in.}}} = 0.209 < 0.7$

Therefore, $\lambda = 0.700$ Unit-less

$N = \dfrac{3\text{ PAF}}{2\text{ ft}} = 1.50\dfrac{\text{PAF}}{\text{ft}}$ 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.)

$\beta = n_s\alpha_s + 2n_p\alpha_p^2 + 4\alpha_e^2$ Factor defining PAF interaction Eq. D1-5

$n_p = \dfrac{L}{L_v} - 1 = \dfrac{12\text{ ft}}{12\text{ ft}} - 1 = 0.0$ Number of *interior supports* Eq. D1-9

$n_s = \left(\dfrac{(12\text{ ft})\left(12\dfrac{\text{in.}}{\text{ft}}\right)}{36\dfrac{\text{in.}}{\text{conn.}}} - 1\right)\dfrac{12\text{ ft}}{12\text{ ft}} = 3$ Number of *side-lap* screws along the *panel* length, L

$\alpha_s = \dfrac{P_{ns}}{P_{nf}} = \dfrac{0.650\text{ k}}{2.0\text{ k}} = 0.325$ *Connection strength* ratio Eq. D1-6

$\alpha_e^2 = \alpha_p^2 = \left(\dfrac{1}{w^2}\right)\sum x_e^2$ See *diaphragm configuration* Eq. D1-8

$x_{e1} = x_{e3} = 4.00\text{ in.}$ $x_{e2} = x_{e4} = 12\text{ in.}$

$= \left(\dfrac{1}{(24\text{ in.})^2}\right)\left(2(4\text{ in.})^2 + 2(12\text{ in.})^2\right) = 0.556$

$\beta = 3(0.325) + 2(0)(0.556) + 4(0.556) = 3.20$ Eq. D1-5

Calculate nominal diaphragm shear strength, S_{nf}, for single span:

$S_{ni} = [2A(\lambda - 1) + \beta]\dfrac{P_{nf}}{L} = [2(1.0)(0.700 - 1) + 3.20]\dfrac{2.00\text{ k}}{12\text{ ft}} = 0.433\text{ klf}$ Eq. D1-1

$S_{nc} = \left(\dfrac{N^2\beta^2}{L^2N^2 + \beta^2}\right)^{0.5} P_{nf} = \left(\dfrac{(1.50\frac{1}{\text{ft}})^2(3.20)^2}{(12\text{ ft})^2(1.50\frac{1}{\text{ft}})^2 + (3.20)^2}\right)^{0.5} 2.00\text{ k} = 0.525\text{ klf}$ 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:

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

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

$s = 2e + 2w + f = 1.49$ in. + 2(3.07 in.) + 5.24 in. = 12.87 in. Eq. D2.1-2

$$S_{nb} = \frac{7890}{(1)(12\text{ ft})^2}\left(\frac{(0.781\text{ in.}^4/\text{ft})^3\ (0.036\text{ in.})^3(8.0\text{ in.})}{12.87\text{ in.}}\right)^{0.25} = 3.34\text{ klf}$$

$S_{nf} = 0.433$ klf < $S_{nb} = 3.34$ 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 ft; and then on the three-span case, L = 36 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: $A, N, \lambda, \alpha_s, \alpha_p^2, \alpha_e^2, P_{ns}, P_{nf}$

$A = 1$ L = 24 ft $\lambda = 0.7$ $\alpha_s = 0.325$

$\alpha_p^2 = \alpha_e^2 = 0.556$ $P_{ns} = 0.650\text{ k}$ $P_{nf} = 2$ k $N = 1.50\dfrac{\text{PAF}}{\text{ft}}$

$$n_p = \frac{L}{L_v} - 1 = \frac{24\text{ ft}}{12\text{ ft}} - 1 = 1.00 \qquad \text{Eq. D1-9}$$

$$n_s = \left(\frac{L_v}{\text{Spacing}} - 1\right)\frac{L}{L_v} = \left(\frac{(12\text{ ft})\left(12\dfrac{\text{in.}}{\text{ft}}\right)}{36\dfrac{\text{in.}}{\text{conn.}}} - 1\right)\frac{24\text{ ft}}{12\text{ ft}} = 6$$

$$\beta = 6(0.325) + 2(1)(0.556) + 4(0.556) = 5.29 \qquad \text{Eq. D1-5}$$

$$S_{ni} = [2(1.0)(0.700 - 1) + 5.29]\frac{2.00\text{ k}}{24\text{ ft}} = 0.391\text{ klf} \qquad \text{Eq. D1-1}$$

$$S_{nc} = \left(\frac{(1.50 \frac{1}{ft})^2 (5.29)^2}{(24\ ft)^2 (1.50 \frac{1}{ft})^2 + (5.29)^2} \right)^{0.5} 2.00\ k = 0.436\ klf \qquad \text{Eq. D1-2}$$

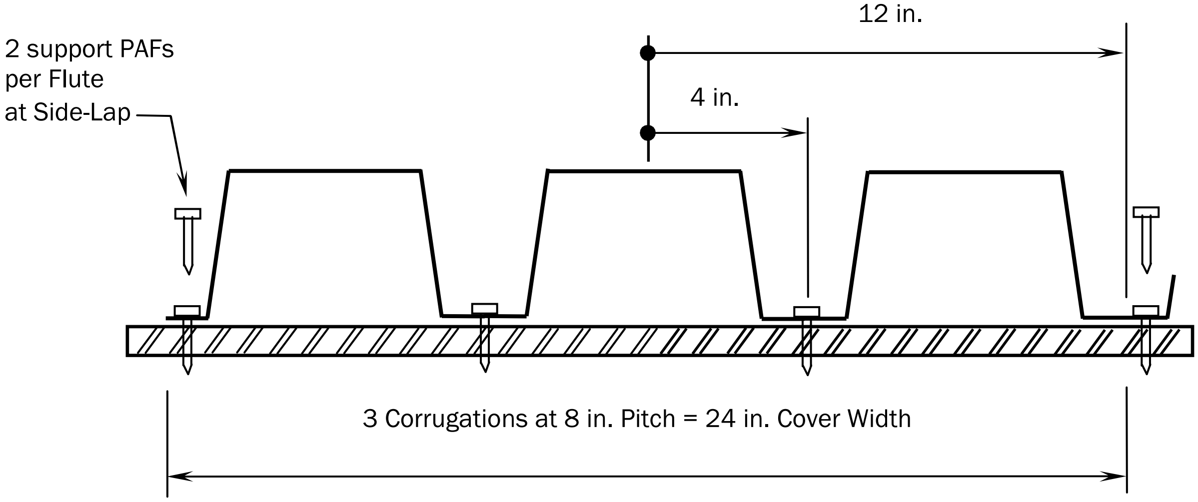
$$S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.391, 0.436) = 0.391\ klf$$

$$S_{nb} = 3.34\ klf\ \text{(Same as single-span)}\ >> 0.391\ klf \qquad \text{Eq. D2.1-1}$$

Result of two-spans: $S_n = 0.391$ 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):



6 support PAF's per 24 in. wide DR deck with two common – Pattern = 24/6

Calculate equivalence based on additional *interior support connections:*

See two-span calculation above for: $L, A, N, n_s, n_p, \lambda, \alpha_s, \alpha_e^2, P_{ns}, P_{nf}$

$$\alpha_p^2 = \left(\frac{1}{(24\ in.)^2} \right) \left(2(4\ in.)^2 + 2(2)(12\ in.)^2 \right) = 1.06 \qquad \text{Eq. D1-8}$$

$$\beta = 6(0.325) + 2(1)(1.06) + 4(0.556) = 6.29 \qquad \text{Eq. D1-5}$$

$$S_{ni} = [2(1.0)(0.700 - 1) + 6.29] \frac{2.00\ k}{24\ ft} = 0.474\ klf \qquad \text{Eq. D1-1}$$

$$S_{nc} = \left(\frac{(1.50 \frac{1}{ft})^2 (6.29)^2}{(24\ ft)^2 (1.50 \frac{1}{ft})^2 + (6.29)^2} \right)^{0.5} 2.00\ k = 0.516\ klf \qquad \text{Eq. D1-2}$$

$$S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.474, 0.516) = 0.474\ klf > 0.433\ klf\ \text{OK}$$

S_{nb} = 3.34 klf (Same as single span) > 0.474 klf Eq. D2.1-1

Result of equivalent two span: S_n = 0.474 klf OK as compared to the single-span S_n = 0.433 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, λ, α_s, α_e^2, α_p^2, P_{ns}, P_{nf}

L = 36 ft Three-span case with L_v = 12 ft

$$n_s = \left(\frac{(12\text{ ft})\left(12\dfrac{\text{in.}}{\text{ft}}\right)}{36\dfrac{\text{in.}}{\text{conn.}}} - 1\right)\frac{36\text{ ft}}{12\text{ ft}} = 9$$

$$n_p = \frac{L}{L_v} - 1 = \frac{36\text{ ft}}{12\text{ ft}} - 1 = 2.0$$

$$\beta = 9(0.325) + 2(2)(1.06) + 4(0.556) = 9.39 \qquad \text{Eq. D1-5}$$

$$S_{ni} = [2(1.0)(0.700-1) + 9.39]\frac{2.00\text{ k}}{36\text{ ft}} = 0.488\text{ klf} \qquad \text{Eq. D1-1}$$

$$S_{nc} = \left(\frac{(1.50\frac{1}{\text{ft}})^2(9.39)^2}{(36\text{ ft})^2(1.50\frac{1}{\text{ft}})^2 + (9.39)^2}\right)^{0.5} 2.00\text{ k} = 0.514\text{ klf} \qquad \text{Eq. D1-2}$$

$$S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.488, 0.514) = 0.488\text{ klf} > 0.433\text{ klf OK}$$

S_{nb} = 3.34 klf (Same as single span) > 0.488 klf

Result of equivalent three span: S_n = 0.488 klf OK as compared to that of single span S_n = 0.433 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 *nominal shear breaking strength*
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 ft

$$\beta = n_s\alpha_s + 2n_p\alpha_p^2 + 4\alpha_e^2 \qquad \text{Eq. D1-5}$$

$$n_s = \left(\frac{(12\text{ ft})\left(12\dfrac{\text{in.}}{\text{ft}}\right)}{6\dfrac{\text{in.}}{\text{conn.}}} - 1\right)\frac{12\text{ ft}}{12\text{ ft}} = 23$$

Side-lap screws along the panel length, L

$$\beta = 23(0.325) + 2(0)(0.556) + 4(0.556) = 9.70 \qquad \text{Eq. D1-5}$$

$$S_{ni} = [2(1.0)(0.700-1) + 9.70]\frac{2.00\text{ k}}{12\text{ ft}} = 1.52\text{ klf} \qquad \text{Eq. D1-1}$$

$$S_{nc} = \left(\frac{(1.50\frac{1}{\text{ft}})^2(9.7)^2}{(12\text{ ft})^2(1.50\frac{1}{\text{ft}})^2 + (9.7)^2}\right)^{0.5} 2.00\text{ k} = 1.42\text{ klf} \qquad \text{Eq. D1-2}$$

$$S_{nf} = \min(S_{ni}, S_{nc}) = \min(1.52, 1.42) = 1.42\text{ klf}$$

S_{nb} = 3.34 klf (Same as Example 4a single span) > 1.42 klf 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

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

$$n_p = \frac{L}{L_v} - 1 = \frac{24\text{ ft}}{12\text{ ft}} - 1 = 1.0$$

$$n_s = \left(\frac{(12\text{ ft})\left(12\dfrac{\text{in.}}{\text{ft}}\right)}{6\dfrac{\text{in.}}{\text{conn.}}} - 1\right)\frac{24\text{ ft}}{12\text{ ft}} = 46$$

$$\beta = 46(0.325) + 2(1)(0.556) + 4(0.556) = 18.3 \quad \text{Eq. D1-5}$$

$$S_{ni} = [2(1.0)(0.700 - 1) + 18.3]\frac{2.00\text{ k}}{24\text{ ft}} = 1.48\text{ klf} \quad \text{Eq. D1-1}$$

$$S_{nc} = \left(\frac{(1.50\frac{1}{\text{ft}})^2(18.3)^2}{(24\text{ ft})^2(1.50\frac{1}{\text{ft}})^2 + (18.3)^2}\right)^{0.5} 2.00 \quad \text{k} = 1.36\text{ klf} \quad \text{Eq. D1-2}$$

$$S_{nf} = \min(S_{ni}, S_{nc}) = \min(1.48, 1.36) = 1.36\text{ klf}$$

S_{nb} = 3.34 klf (Same as Example 4a single span) > 1.36 klf 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.

$$\alpha_p^2 = 1.06$$

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

$= 46(0.325) + 2(1)(1.06) + 4(0.556) = 19.3$

$S_{ni} = [2(1.0)(0.700-1)+19.3]\dfrac{2.00\ \text{k}}{24\ \text{ft}} = 1.56\ \text{klf}$ Eq. D1-1

$S_{nc} = \left(\dfrac{(1.50\frac{1}{\text{ft}})^2(19.3)^2}{(24\ \text{ft})^2(1.50\frac{1}{\text{ft}})^2 + (19.3)^2}\right)^{0.5} 2.00\ \ \text{k} = 1.42\ \text{klf}$ Eq. D1-2

$S_{nf} = \min(S_{ni}, S_{nc}) = \min(1.56, 1.42) = 1.42\ \text{klf}$

S_{nb} = 3.34 klf (Same as Example 4a single span) > 1.42 klf Eq. D2.1-1

Result of equivalent two spans: $S_n = 1.42\ \text{klf}$ OK as compared to $S_n = 1.42\ \text{klf}$ 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 ft Three-span case with L_v = 12 ft

$n_p = \dfrac{L}{L_v} - 1 = \dfrac{36\ \text{ft}}{12\ \text{ft}} - 1 = 2.0$

$n_s = \left(\dfrac{(12\ \text{ft})\left(12\dfrac{\text{in.}}{\text{ft}}\right)}{6\dfrac{\text{in.}}{\text{conn.}}} - 1\right)\dfrac{36\text{ft}}{12\ \text{ft}} = 69$

$\beta = 69(0.325) + 2(2)(1.06) + 4(0.556) = 28.9$ Eq. D1-5

$S_{ni} = [2(1.0)(0.700-1)+28.9]\dfrac{2.00\ \text{k}}{36\ \text{ft}} = 1.57\ \text{klf}$ Eq. D1-1

$S_{nc} = \left(\dfrac{(1.50\frac{1}{\text{ft}})^2(28.9)^2}{(36\ \text{ft})^2(1.50\frac{1}{\text{ft}})^2 + (28.9)^2}\right)^{0.5} 2.00\ \ \text{k} = 1.42\ \text{klf}$ Eq. D1-2

$S_{nf} = \min(S_{ni}, S_{nc}) = \min(1.57, 1.42) = 1.42\ \text{klf}$

S_{nb} = 3.34 klf (Same as Example 4a single span) > 1.42 klf Eq. D2.1-1

Result of equivalent three spans: $S_n = 1.42\ \text{klf}$ OK as compared to $S_n = 1.42\ \text{klf}$ 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}:

$A = 2.0$ $\lambda = 0.7$ $n_p = 2$ $P_{nf} = 2$ kips

$\alpha_s = 0.325$ $\alpha_p^2 = 0.556$ $\alpha_e^2 = 1.06$ $n_s = 69$

$N = \dfrac{4\text{ PAF}}{2\text{ ft}} = 2.00\dfrac{\text{PAF}}{\text{ft}}$ (Although six PAFs appear in the sketch, two are common to each deck at *side-lap*, so N = 4/2.)

$\beta = 69(0.325) + 2(2)(0.556) + 4(1.06) = 28.9$

Calculate S_{nf}:

$$S_{ni} = [2(1.0)(0.700 - 1) + 28.9]\frac{2.00\text{ k}}{36\text{ ft}} = 1.57\text{ klf}$$

$$S_{nc} = \left(\frac{(2.0\frac{1}{\text{ft}})^2(28.9)^2}{(36\text{ ft})^2(2.0\frac{1}{\text{ft}})^2 + (28.9)^2}\right)^{0.5} 2.00\ \text{k} = 1.49\text{ klf} > 1.42\text{ klf}$$

Result: $S_{nf} = \min(S_{ni}, S_{nc}) = \min(1.57, 1.49) = 1.49\text{ klf} > 1.42\text{ klf}$ OK

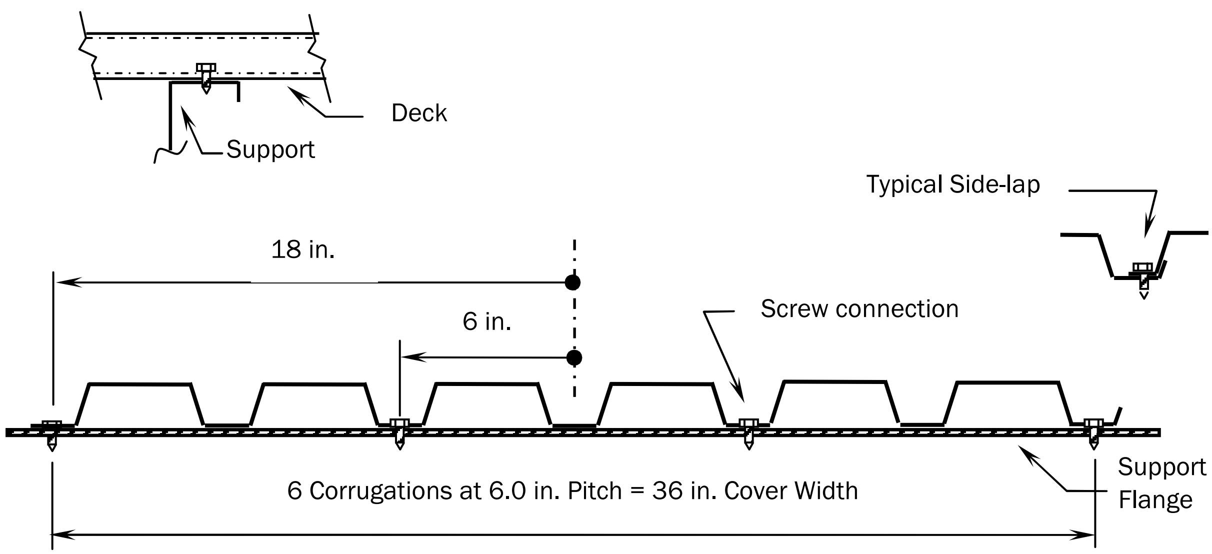
Note: In this *configuration*, the addition of end *connections* is more efficient where S_{nc} controls S_n. At the three-span case for a butt-jointed *panel*, this configuration requires the same number of additional support connections (two at *panel* ends vs. two at *interior supports*) but provides 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



4 support screws per 36 in. wide deck with one screw common - Pattern 36/4

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.^{4}/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 *nominal shear breaking strength*

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.

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

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

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:

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

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

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):

$$\frac{t_2}{t_1} = \frac{0.048 \text{ in.}}{0.036 \text{ in.}} = 1.5 \qquad \text{Therefore,} \quad 1.0 < \frac{t_2}{t_1} \le 2.5$$

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

$$= 4.2\left((0.048 \text{ in.})^3 (0.216 \text{ in.})\right)^{1/2} \left(55 \frac{k}{\text{in.}^2}\right) = 1.13 \text{ k}$$

$$P_{nf} = 2.7 t_1 d F_{u1} \qquad \text{AISI S100 Eq. E4.3.1-4}$$

$$= 2.7(0.036 \text{ in.})(0.216 \text{ in.})\left(62 \frac{k}{\text{in.}^2}\right) = 1.30 \text{ k}$$

$$P_{nf} = 2.7 t_2 d F_{u2} \qquad \text{AISI S100 Eq. E4.3.1-5}$$

$$= 2.7(0.048 \text{ in.})(0.216 \text{ in.})\left(55 \frac{k}{\text{in.}^2}\right) = 1.54 \text{ k}$$

$P_{nss} = 2.0$ kips See Connection Schedule

$$\frac{t_2}{t_1} = 1.0 \qquad P_{nf} = \min(1.13, 1.30, 1.54, 2.00) = 1.13 \text{ k}$$

$$\frac{t_2}{t_1} = 2.5 \qquad P_{nf} = \min(1.30, 1.54, 2.00) = 1.30 \text{ k}$$

$$\frac{t_2}{t_1} = 1.5 \qquad P_{nf} = 1.13 + \frac{1.5 - 1.0}{2.5 - 1}(1.30 - 1.13) = 1.19 \text{ k}$$

Result: $P_{nf} = 1.19$ 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):

$$\frac{t_2}{t_1} = \frac{0.036 \text{ in.}}{0.036 \text{ in.}} = 1.0 \qquad \text{Therefore,} \quad \frac{t_2}{t_1} \le 1.0 \qquad F_{u2} = F_{u1}$$

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

$$= 4.2\left((0.036 \text{ in.})^3 (0.216 \text{ in.})\right)^{1/2} \left(62 \frac{k}{\text{in.}^2}\right) = 0.827 \text{ k}$$

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

$$= 2.7(0.036 \text{ in.})(0.216 \text{ in.})\left(62 \frac{k}{\text{in.}^2}\right) = 1.30 \text{ k}$$

$P_{nss} = 2.0$ k See Connection Schedule

Result: $P_{ns} = \min(0.827, 1.30, 2.00) = 0.827$ kips Tilting of screw in panel controls.

Calculate configuration parameters required for S_{nf}:

$A = 1.0$ Number support screws at side-lap at deck ends

$\lambda = 1 - \frac{D_d L_v}{240\sqrt{t}} \geq 0.7$ Required input units are defined in AISI S310 Eq. D1-4

$= 1 - \frac{(1.47\text{ in.})(4.0\text{ ft})}{240\sqrt{0.036\text{ in.}}} = 0.871 > 0.7$ OK Unit-less

$N = \frac{3\text{ screws}}{3\text{ ft}} = 1.00\frac{\text{screw}}{\text{ft}}$ 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.)

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

$n_p = \frac{L}{L_v} - 1 = \frac{20\text{ ft}}{4\text{ ft}} - 1 = 4.0$ Number of *interior supports* Eq. D1-9

$n_s = \left(\frac{(4\text{ ft})\left(12\frac{\text{in.}}{\text{ft}}\right)}{24\frac{\text{in.}}{\text{conn.}}} - 1\right)\frac{20\text{ ft}}{4\text{ ft}} = 5$ Number of *side-lap* screws along the *panel* length, L

$\alpha_s = \frac{P_{ns}}{P_{nf}} = \frac{0.827\text{ k}}{1.19\text{ k}} = 0.695$ *Connection strength* ratio Eq. D1-6

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

$x_{e1} = x_{e3} = 6\text{ in.}$ $x_{e2} = x_{e4} = 18\text{ in.}$

$\alpha_p^2 = \alpha_e^2 = \left(\frac{1}{(36\text{ in.})^2}\right)\left(2(6\text{ in.})^2 + 2(18\text{ in.})^2\right) = 0.556$

$\beta = 5(0.695) + 2(4)(0.556) + 4(0.556) = 10.1$ Eq. D1-5

Calculate nominal diaphragm shear strength, S_{nf}:

Note: The simplification is that the butt-joint strength (P_{nf} =1.19 k) will be applied to the end-lap strength in calculating S_{nf}.

$S_{ni} = [2(1.0)(0.871 - 1) + 10.1]\frac{1.19\text{ k}}{20\text{ ft}} = 0.580\text{ klf}$ Eq. D1-1

$S_{nc} = \left(\frac{(1.0\frac{1}{\text{ft}})^2(10.1)^2}{(20\text{ft})^2(1.0\frac{1}{\text{ft}})^2 + (10.1)^2}\right)^{0.5} 1.19\text{ kips} = 0.536\text{ klf}$ Eq. D1-2

Result: $S_{nf} = \min(S_{ni}, S_{nc}) = \min(0.580, 0.536) = 0.536$ 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.

α = 1 Conversion factor for U.S. customary units
s = 2e + 2w + f = 1.56 in. + 2(1.53 in.) + 3.56 in. = 8.18 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 klf << S_{nb} = 10.9 klf

Result Nominal Diaphragm Shear Strength per Unit Length, S_n

S_n = 0.536 klf Based on one support screw connection strength, P_{nf} = 1.19 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 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 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 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 k, P_{nfe} = 0.60 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: $A, N, L, n_p, n_s, \lambda, \alpha_e^2, \alpha_p^2, P_{ns}$

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

$$\alpha_s = \frac{0.827\ \text{k}}{1.19\ \text{k}} = 0.695$$ Eq. D1-6

$$\beta = 5(0.695) + 2(4)(0.556) + 4(0.556)\frac{0.60\ \text{k}}{1.19\ \text{k}}$$

$$= 9.04$$

$$S_{ni} = [2(1.0)(0.827 - 1)\frac{0.60\ \text{k}}{1.19\ \text{k}} + 9.04]\frac{1.19\ \text{k}}{20\ \text{ft}}$$ Modified Eq. D1-1

$$= 0.528\ \text{klf}$$

Calculate S_{nc} based on P_{nfe} = 1.30 k, P_{nfi} = 1.19 k

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

$$\alpha_s = \frac{0.827\ \text{k}}{1.30\ \text{k}} = 0.636$$ Eq. D1-6

$$\beta = 5(0.636) + 2(4)(0.556)\frac{1.19\ \text{k}}{1.30\ \text{k}} + 4(0.556)$$

$$= 9.48$$

$$S_{nc} = \left(\frac{(1)^2 (9.48)^2 \frac{1}{\text{ft}^2}}{(20)^2 (1)^2 \frac{\text{ft}^2}{\text{ft}^2} + (9.48)^2} \right)^{0.5} 1.30\ \text{k}$$ 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.

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

K = 1.0 for steel *panels* with lap-down on steel supports

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

See *diaphragm configuration* and Figure D1-1 for x_e.

$$\alpha_3 = \alpha_4 = \frac{\sum x_e}{w} = \left(\frac{1}{36 \text{ in.}} \right) \left(2(6 \text{ in.}) + 2(18 \text{ in.}) \right) = 1.33 \qquad \text{Eq. D5.1.1-3}$$

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:

$$4.2\left(t_3^3 d\right)^{1/2} F_{u2} = 2.7 t_2 d F_{u2}$$

Rationale: Beyond thickness, t_3, tilting at support will not control, and *flexibility* is relatively independent of support thickness.

$$\left(t_3^3\right)^{1/2} = \frac{2.7(0.048 \text{ in.})(0.216 \text{ in.})}{4.2(0.216 \text{ in.})^{0.5}} = 0.014 \text{ in.}^{1.5}$$

$$t_3 = 0.058 \text{ in.}$$

$$S_f = \left(3 - 1.7\left(\frac{t_2 - t_1}{t_3 - t_1} \right) \right) \left(\frac{\alpha}{1000 t_1^{0.5}} \right) \qquad \text{Figure C-D5.2.2-1}$$

$$= \left(3 - 1.7\left(\frac{0.048 - 0.036}{0.058 - 0.036} \right) \right) \left(\frac{\alpha}{1000(0.036)^{0.5}} \right)$$

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

$$S_f = \frac{2.07\alpha}{1000\sqrt{t}} = \frac{2.07(1)}{1000\sqrt{0.036\text{ in.}}} = 0.0109\frac{\text{in.}}{\text{k}}$$ (59% greater than S_f with thick support)

Calculate side-lap screw flexibility, S_s:

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

Calculate slip constant at connections, C:

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

$$\alpha_3 = \frac{\sum x_e}{w} = \frac{(2(6)+2(18))}{36} = 1.33; \qquad \alpha_4 = \frac{\sum x_p}{w} = \alpha_3 = 1.33$$

$$C = \left(\frac{29500\frac{\text{k}}{\text{in.}^2}(0.036\text{ in.})}{36\text{ in.}}\right)\left(\frac{2(12\frac{\text{in.}}{\text{ft}})(20\text{ ft})}{2(1.33)+4(1.33)+2(5)\left(\frac{0.0109}{0.0158}\right)}\right)0.0109\frac{\text{in.}}{\text{k}} = 10.4$$ Eq. D5.1.1-2

Calculate D_n using S310 Appendix 1:

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

$$D = \frac{U_1D_1 + U_2D_2 + U_3D_3 + U_4D_4}{U_1 + U_2 + U_3 + U_4}$$ 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 in. 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.

$U_2 = 6 \qquad U_1 = U_3 = U_4 = 0$

$$D = \frac{0 + 6(7640) + 0 + 0}{0 + 6 + 0 + 0} = 7640\text{ in.}$$ Eq. 1.4-2

$$D_n = \frac{7640\text{ in.}}{20\text{ ft}\times 12\frac{\text{in.}}{\text{ft}}} = 31.8 \quad \text{(Unit-less)}$$ Eq. 1.4-1

γ_c = 0.71 Table 1.3-1

Calculate G′:

From Deck Data

s = f + 2e + 2w = 3.56 in. + 1.56 in.+2(1.53 in.) = 8.18 in. Eq. 2.1-2

d = 6 in.

μ = 0.3 (Poisson's ratio for steel)

$$G' = \left(\frac{29500 \frac{k}{in.^2} (0.036)\, in.}{2(1+0.3) \frac{8.18\, in.}{6\, in.} + 0.71(31.8) + 10.4} \right) 1 = 29.1 \frac{k}{in.}$$ Eq. D5.1.1-1

Note: If the support is much thicker:

$$S_f = \frac{1.3\alpha}{1000\sqrt{t}} = \frac{1.3(1)}{1000\sqrt{0.036\, in.}} = 0.00685 \frac{in.}{k}$$ Eq. D5.2.2-1

C = 7.88 24% less slippage and distortion at connections

G′ = 31.2 k/in. 7% less deflection because warping dominates deflection

Result Example 5b - Stiffness:

$$G' = 29.1 \frac{k}{in.}$$

Result of Example 5:

Nominal diaphragm shear strength per unit length:

S_n = 0.536 klf Controlled by S_{nf}

Available diaphragm shear strength per unit length, S_a:

Depending on different load types, select s*afety* and *resistance factors* from Section B1.

Use Ω_d and ϕ_d for connection-related diaphragm strength in AISI S100 Table D5.

$\frac{S_n}{\Omega} = \frac{S_{nf}}{\Omega_{df}}$ for *ASD* $\phi S_n = \phi_{df} S_{nf}$ for *LRFD* and *LSD*

Diaphragm stiffness

$$G' = 29.1 \frac{k}{in.}$$

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