Everything You Wanted to Know About Diaphragms
(But were Afraid to Ask)

February 19, 2020

Presented by: Tom Sputo and Michael Martignetti
Polling Question

• New requirement to earn PDH credits

• Two questions will be asked during the duration of today’s presentation

• The question will appear within the polling section of your GoToWebinar control panel to respond
Disclaimer

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Webinar Description

Recent changes in design codes and construction practices have made diaphragm connection design more complicated and time-consuming. This webinar identifies those changes, provides design guidance, review upcoming publications, and introduces new SDI design tools.

This webinar is provided by the Steel Joist Institute in partnership with the Steel Deck Institute.
Learning Objectives

1. Recognizing appropriate AISI and SDI Standards and literature

2. Understanding the limits of existing shear tables and evaluation reports

3. Design procedures for uniquely shaped and sloped structures

4. Design procedures and shear capacities over CF framing
What References are Available?

https://cfsei.memberclicks.net/publications
How Are Previous References Used?
How Are Previous References Used?

ESR-1227
Reissued 12/2016
This report is subject to renewal 12/2017.

DIVISION: 05 00 00—METALS
SECTION: 05 31 00—STEEL DECKING

Approval Standard
for
Profiled Steel Panels
for Use as Decking in
Class 1 Insulated Roof
Construction

UES
EVALUATION REPORT
Number: 217

Originally Issued: 11/09/2011
Revised: 01/06/2017
Valid Through: 11/30/2017

EVALUATION SUBJECT:
STEEL DECKS:

1.3 Properties assessed:
• Structural
• Fire Resistance

Wei Wen Yu Center for Cold-Formed Steel Structures
How are Shear Tables Developed?

Flat roof, Symmetric, Corner sheets
Can It Be This Simple?

\[ S_{ne} = SF_x \]
\[ S_{ni} = sM \]
\[ S_{nc} = (A^2 + B^2)^{1/2} \]
\[ S_n = \min ( S_{ne}, S_{ni}, S_{nc}, S_{nb} ) \]
Should We Understand Diaphragm Limit States?
Do Floor Diaphragms Also Have Limit States?

\[ S_n = \frac{\beta P_{nf}}{L} + kbd_c\sqrt{f'_c} \]
What’s the Most Critical Limit State? Deck, Fastener, or Support?

\[ d_e = 0.7d - 1.5t \leq 0.55d \]
\[ d_a = d - t \]
Are welds defined in contract documents? 

Inspection?

d_e = 0.7d - 1.5t ≤ 0.55d

d_a = d - t

Are Fastener (Weld) Shear Strengths Fixed?
Are Fastener (Weld) Shear Strengths Fixed?

Visible, Effective, or Average?

1.5(WR, IR, NR)20

- Design thickness = 0.0358 in.
- Support fastening: 5/8 in. arc spot welds or equivalent
- Side-lap fastening: #10 screws

<table>
<thead>
<tr>
<th>Fastener Layout</th>
<th>Side-lap Conn/Span</th>
<th>Nominal Shear Strength, $S_{nf}$, $pl^{1.2}$</th>
<th>Span, ft.</th>
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<td>$1105$</td>
</tr>
<tr>
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<td>$1275$</td>
<td>$1180$</td>
</tr>
</tbody>
</table>

15

2013

DDM04

Visible, Effective, or Average?
3.9.2 Arc Spot (Puddle) Welds: Arc spot (puddle) welds must have a minimum effective fusion area to supporting members of at least ½ inch (13 mm) in diameter. See figure 5 for details.
Are Fastener (Screw) Shear Strengths Fixed?

If support is CF43, 33 ksi = $P_{nf} = 858$
Are Fastener (Screw) Shear Strengths Fixed?

\[ P_{nv} = 4.2 \sqrt{t_2^3 d} F_{U2} \]

\[ 2.7 t_1 d F_{U1} \]

\[ 2.7 t_2 d F_{U2} \]
My Supports are Thin Cold-Formed Purlins. Does This Mean Existing Shear Tables are Useless?

<table>
<thead>
<tr>
<th>Fastener Layout</th>
<th>K (1/ft)</th>
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<tbody>
<tr>
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<td>2</td>
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<td>4</td>
<td>0.294</td>
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<tr>
<td>5</td>
<td>0.254</td>
</tr>
<tr>
<td>6</td>
<td>0.224</td>
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</table>

Nominal Shear Strength, $S_{nv}$, psi

Table 1: Shear Strength Values

<table>
<thead>
<tr>
<th>Loading</th>
<th>$f_{y}$</th>
<th>$f_{p}$</th>
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</thead>
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<tr>
<td>Sotonic</td>
<td>45 ksi</td>
<td>33 ksi</td>
</tr>
<tr>
<td>Wind</td>
<td>60 ksi</td>
<td>33 ksi</td>
</tr>
<tr>
<td>Other</td>
<td>60 ksi</td>
<td>33 ksi</td>
</tr>
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Initial shear resisting section $S_{nv}$, psi
Should I Defer to the CF supplier, or Can S.D.I. Help?

<table>
<thead>
<tr>
<th>Fastener Layout</th>
<th>Side-lap Connection/Span</th>
<th>Nominal Shear Strength, $S_{nf}$, psi</th>
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<tr>
<td></td>
<td>24”</td>
<td>32”</td>
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<tr>
<td>36/14</td>
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</tr>
<tr>
<td>$\alpha_f=\alpha_o$</td>
<td>4,000</td>
<td></td>
</tr>
<tr>
<td>$\alpha_p^2 = \alpha_o^2$</td>
<td>1,556</td>
<td></td>
</tr>
<tr>
<td>$N$</td>
<td>4,000</td>
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<td>$A$</td>
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<td>1575</td>
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<td>1670</td>
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<tr>
<td>6</td>
<td>2100</td>
<td>1760</td>
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</tbody>
</table>

1.5WR22
Design thickness = 0.0295 in.

- $F_{y,deck} = 33$ ksi
- $F_{u,deck} = 45$ ksi

Framing designation = 43 mils
Framing thickness = 0.0451 in.

- $F_{y,framing} = 33$ ksi
- $F_{u,framing} = 45$ ksi

Support fastening: #12 screws
Side-lap fastening: #10 screws
What About Fastener Tension?
Will Fastener Tension Also Change my Shear Tables?
(What is $P_{nft}$?)

\[ \% \text{Shear} + \% \text{Tension} \leq 1.0 \]

\[
\left( \frac{P_{nft}}{P_{nf}} \right)^{1.5} + \left( \frac{\Omega_t T}{P_{nt}} \right)^{1.5} = 1.0
\]

\[
\frac{P_{nft}}{\Omega_d P_{nf}} + \frac{T}{T_n} = \frac{1.15}{\Omega}
\]

\[
\frac{P_{nft}}{\Omega_d P_{nf}} + 0.71T = 1.1 \frac{T}{T_n} = \frac{1.15}{\Omega}
\]

\[
\frac{P_{nft}}{\Phi_{d} P_{nf}} + \frac{T}{T_n} = 1.15 \Phi
\]

\[
\frac{P_{nft}}{\Phi_{d} P_{nf}} + 0.71T = 1.1 \frac{T}{T_n} = 1.1 \Phi
\]
Is $P_{nft}$ a Direct Substitution for $P_{nf}$?

$$S_{ne} = \Sigma F_x$$

$$S_{ne} = \left(2\alpha_1 + n_p\alpha_2 + n_e\right)\frac{P_{nf}}{L}$$

$$S_{ne} = \left(2\alpha_1 + n_p\alpha_2\right)\frac{P_{nft}}{L} + n_e\frac{P_{nf}}{L}$$
My Uplift > My Gravity. Again, Are Existing Shear Tables Useless?

<table>
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<tr>
<th>Fastener Layout</th>
<th>Side-lap Conn/Span</th>
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<th>4.5</th>
<th>5</th>
<th>5.5</th>
<th>6</th>
<th>6.5</th>
<th>7</th>
<th>7.5</th>
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</table>

Nominal Shear Strength, \( S_{\text{n}} \), \( \text{psi} \):
Time to Panic?

MWFRS?  CC?
Tilting?
Uplift?

CF?
Sne? Pnft

ASCE 7-05? 7-10? 7-16?
I Want a 5 minute solution for CF and Uplift. Can S.D.I. Help?

SDI Diaphragm Interaction Calculator

<table>
<thead>
<tr>
<th>Deck Profile</th>
<th>1.5 x 6 WR</th>
<th>Support Fastener Pattern</th>
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<tbody>
<tr>
<td>Deck Gage</td>
<td>22</td>
<td>Support Fastener</td>
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<tr>
<td>Deck $F_y$, $F_u$</td>
<td>33, 45 ksi</td>
<td>Sidelap Fastener</td>
</tr>
<tr>
<td>(-) Uplift, ASCE 7-10</td>
<td>50 psf</td>
<td>Substrate thickness, $t_2$</td>
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<tr>
<td></td>
<td></td>
<td>Substrate $F_y$, $F_u$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Number of Spans</td>
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<td></td>
<td></td>
<td>AISI S-310</td>
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</table>

#12 screw |
| 36/7      |

#10 screw |
| 33, 45 ksi |

CF33 mil (20 gage) |
| 2016

CF, DDM04 and S310 Diaphragm Tool
Nominal, ASD and LFRD outputs
Tension – Shear Interaction
Calculations Page
I Want a 5 minute solution.
Can S.D.I. Help?

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<th>7.5</th>
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<td>596</td>
<td>561</td>
<td>529</td>
<td>500</td>
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- $P_{nf} = 0.589 \text{ kips}$
- $P_{ns} = 0.417 \text{ kips}$
I Want a 5 minute solution. Can S.D.I. Help?

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<td>F</td>
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<td>270</td>
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<td>127</td>
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<td>20</td>
<td>F</td>
<td>F</td>
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</table>

| LRFD Interactive Shear Strength, F_{d} S_{nf}, plf |

F  Fasteners Fail in tension. See AISI S310

\[
S_{ne} = (2\alpha_1 + n_p\alpha_2) \frac{P_{nft}}{L} + n_e \frac{P_{nf}}{L} = 297 \text{ plf}
\]

\[
S_{ni} = 2A(\lambda - 1) + n_s\alpha_s + \frac{1}{w^2} \left(2n_p\Sigma x_p^2 + 4\Sigma x_e^2\right) \frac{P_{nft}}{L} = 215 \text{ plf}
\]

\[
S_{nc} = \frac{P_{nft} \sqrt{N^2B^2}}{\sqrt{L^2N^2 + B^2}} = \text{LIMIT} = 198 \text{ plf}
\]

\[
S_n = \min(S_{ne}, S_{ni}, S_{nc}) \phi = 159 \text{ plf}
\]
Accessories . . . Should we open the can?
3.4 Accessory Attachment

A. Structural accessories shall be attached to supporting structure or deck as required for transfer of forces, but not to exceed 12” (300mm) on center. Non-structural accessories shall be attached to supporting structure or deck as required for serviceability, but not to exceed 24” (600mm) on center.
Who Selects Accessories and Can They Transfer Shear?

6” wide 22 gage flat plate. \( P_{ns} \#10 = 417 \text{ lbs} \)

\[ S_{ni} = SM = 1005 \text{ plf} = (P_{ns})(\#\text{ screws})(0.5') \]

4.8 screws / span  (Use 5, not 4)

What did the deck detailer put on the erection plans?
We’ve reviewed codes, shear limit states, potential problems, and future solutions . . . . . Tom Sputo will help with defining and applying shear loads before selecting fasteners and patterns.

Part 2
Polling Question #1

Which of the following are limit states for determining diaphragm strength of deck?

A. Buckling Strength of the Deck
B. Edge Fastener Strength
C. Interior Fastener Strength
D. Corner Fastener Strength
E. All of the Above
When Checking Interaction, Should I Use . . .

MWFRS
CC
ASCE 7-05
ASCE 7-10
ASCE 7-16

<table>
<thead>
<tr>
<th>TRIB. AREA (FT^2)</th>
<th>ZONE 1 INTERIOR</th>
<th>ZONE 2 PERIMETER</th>
<th>ZONE 3 CORNERS</th>
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<td>35.3</td>
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<td>34.4</td>
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<tr>
<td>50</td>
<td>33.2</td>
<td>44.5</td>
<td>53.5</td>
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<tr>
<td>100</td>
<td>32.3</td>
<td>38.2</td>
<td>38.2</td>
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</tbody>
</table>

a = 12.0 ft.
When Checking Interaction, Should I Use . . .

This is an interesting question, but the answer is quite simple.

• Roof deck performs 2 functions:
  • As Roof SHEATHING, carrying gravity and wind uplift (or downward) loads
  • As a DIAPHRAGM, transferring lateral loads into the Lateral Force Resisting System
When Checking Interaction, Should I Use . . .

And **ASCE 7-10** (and 7-16) provide the answer.

- **C26.2 DEFINITIONS: COMPONENTS AND CLADDING:**

  … Cladding receives wind loads directly. Examples of **components include fasteners, … roof decking …** Components can be part of the MWFRS when they act as … roof diaphragms, but they may also be loaded as individual components.
When Checking Interaction, Should I Use . . .

And **ASCE 7-10** (and 7-16) provide the answer.

- **C26.2 DEFINITIONS: MAIN WIND-FORCE RESISTING SYSTEM (MWFRS)**

  Can consist of ... an assemblage of structural elements that work together to transfer wind loads acting on the entire structure to the ground. Structural elements such as ... **roof diaphragms** are part of the Main Wind-Force Resisting System (MWFRS) ...
When Checking Interaction, Should I Use . . .

And ASCE 7-10 (and 7-16) provide the answer.

- ROOF DECK AS SHEATHING
  - Roof Dead, Roof Live, and C&C Wind Pressures (uplift or downward)

- ROOF DECK AS DIAPHRAGM
  - Roof Dead, Roof Live, and MWFRS Wind Pressures (uplift or downward) and MWFRS Wind Diaphragm Shear
Openings. What Changes?
Openings. What Changes?
Openings. What Changes?

\[ V_1 = 16.7(20')/100' = 3.3K \]
\[ V_2 = 6.7(40')/30'/2 = 4.5K \]
\[ V_3 = -10(60')/100' = -6K \]
\[ SF_x = 1.8K \]
\[ SF_y = 0 \]
How do I handle a roof (or floor) diaphragm with holes?

\[ V_x = V_A - w(x) \]
How do I handle a roof (or floor) diaphragm with holes?

\[ V_L = V_x \left( \frac{b}{B-b} \right) \]

\[ V_R = V_L - a(w) \]
How do I handle a roof (or floor) diaphragm with holes?

\[ S_H (b) = \frac{wa^2}{2} + (V_L - aw)a \]

\( S_H \) is the “hole chord” force that must be dragged into the diaphragm.
How do I handle a roof (or floor) diaphragm with holes?

\[ S_H (b) = \frac{wa^2}{2} + (V_L - aw)a \]

Observations:
- Chord force depends on magnitude of “w”
- As “a” approaches zero, chord force approaches zero
How about a simple computer solution using frame analysis software?

It is possible to calculate diaphragm deflections using frame analysis software that has the ability to calculate shear deflections in addition to flexural deflections. Most, but not all available educational or commercial software has this capability.

In order to properly model the diaphragm, a beam is constructed using the proper shear stiffness, and analyzed under the load case(s).

Program input should use the following values:

\[
\begin{align*}
I &= \text{Infinite (or extremely large value)} \\
E &= \text{Infinite (or extremely large value)} \\
G &= 11300 \text{ ksi (for steel diaphragm)} \\
B &= \text{Diaphragm depth (length parallel to direction of loading)} \\
A &= \text{Beam element cross sectional area} \\
&= (B \cdot G') / G
\end{align*}
\]
How about a simple computer solution using frame analysis software?

**DDM04 – Example 15**

\[ w = 0.200 \text{ kip/ft} \]

\[ G' = 30 \text{ kip/inch} \]
How about a simple computer solution using frame analysis software?

DDM04 – Example 15

\[ w = 0.200 \text{ kip/ft} \]

A = 2 at hole

= 3 away from hole
How about a simple computer solution using frame analysis software?

**DDM04 – Example 15**

Disregarding Hole:

\[ V_L = 30k \quad V_R = 30k \]
\[ \Delta c = 0.287'' \quad \Delta d = 0.669'' \]

Considering Hole:

\[ V_L = 28.36k \quad V_R = 31.64k \]
\[ \Delta c = 0.269'' \quad \Delta d = 0.791'' \]

But because magnitude of “w” does not change, the chord forces at the hole do not change
Sloped Roofs. What Changes?

![Diagram of Monoslope Roof]

**Figure 2.3 - Monoslope Roof**

Where:

- \( d \) = Diaphragm depth (parallel to force)
- \( L \) = Diaphragm length (perpendicular to force)
Sloped Roofs. What Changes?

Using this geometrical relationship it can be seen that the force in the sloped diaphragm per unit length (pounds per linear foot) is the same as if the roof were flat.

\[ V_{\text{diaphragm-sloped}} = \frac{F_{\text{sloped}}}{d_{\text{sloped}}} = \frac{F_x}{\cos \phi} = \frac{F_x}{d(\alpha)} = \frac{F_x}{d} = V_{\text{diaphragm-flat}} \]
Gable Roof. What Changes?

\[ \text{plf} = \text{plf} \]
Hipped Roofs. What Changes?
Hipped Roof. What Changes?
Hipped Roof. What Changes?

\[
\Sigma F_x = 0 \\
\Sigma F_y = 1600 - 1116 + T \\
T = 484 \text{ lbs}
\]

\[
\sqrt{(61 + 279)^2 + 395^2} \\
\frac{1.732'}{1.732'} = 301 \text{ plf}
\]
Skewed Shear Walls. What Changes?
Does Diaphragm Shear Capacity Change for 1-span Conditions?

![Graph showing the change in shear capacity with increasing number of spans.](image)
Does Diaphragm Shear Capacity Change for 1-span Conditions?

\[ \Sigma M_A = (633 + 774)(36") (2) + 417(36") (6) + 258(24)(4) - 258(12)(4) - S_n (180") = 0 \]

\[ S_n 3 - \text{span} = 1132# = 377 \text{ plf} \]
\[ S_n 1 - \text{span} = 1363# = 454 \text{ plf} \]

\[ \Sigma M_A = (633 + 417)(36") (2) + 258(24)(2) - 258(12)(2) - S_n (60") = 0 \]
Should I use “Shear-Area” or “Shear-Stiffness” to calculate deflections?

\[ G' \text{ shear modulus} \]

\[ D_S = \frac{wL^2}{8bG'} \]
Is Shear Deflection based on Bending or Stiffness?

W = 100 plf ASD

WR22
5/8 arc spot weld
#10 TEK
6-0 spans
Net U = 20 psf

36/4/1
G' = 15.6 K/in

36/9/2
G' = 67.4 K/in

36/4/1
G' = 15.6 K/in
Is Shear Deflection based on Bending or Stiffness?

Bending Method

Energy Method
How do I design an unbalanced diaphragm, like a strip mall?

Statics will never fail you!

Force in Side L = $P$
= $P / L$ (pounds / foot)

Force in Side W = $P(e - XL) / L$
= $P(e - XL) / WL$ (pounds / foot)

$XL = W^2 / (L + 2W)$

The torsion in the diaphragm results in an opposing force couple in the transverse walls that must be taken out into the drag struts at those walls.
How do I design an unbalanced diaphragm, like a strip mall?

Statics will never fail you!

\[
\begin{align*}
L & = 100 \text{ feet} \\
W & = 30 \text{ feet} \\
P & = 15,000 \text{ pounds} \\
e & = 15 \text{ feet} \\
XL & = 5.62 \text{ feet}
\end{align*}
\]

Force in L  = 15,000 pounds  
= 150 plf

Force in W  = \( (15,000)(15-5.62)/(100) = 1407 \text{ pounds} \)  
= 47 plf

NOTE 1: The location of the centroid of the walls is determined by statics, and the two transverse walls do not have to be of equal length.

NOTE 2: For both wind and seismic loading, the load P is located at the centroid of the load application.
How is Acoustical Deck affected by the holes in the webs?

DDM04 – Example 19 and 22 – WR20 Roof Deck

PERFORATIONS: 5/32” ROUND HOLES IN A STAGGERED PATTERN

1.53”

W

Wp

29/32”

3/8”
How is Acoustical Deck affected by the holes in the webs?

DDM04 – Example 19 and 22 – WR20 Roof Deck

\[
\begin{align*}
367 & \text{ WELD AT FLUTE} \\
U_1 &= 6, \quad U_2 = U_3 = U_4 = 0
\end{align*}
\]

\[
D \text{ with perforations } = \frac{6(930)}{6} = 930 \text{ inches (0.6\% more warping)}
\]

\[
D \text{ without perforations } = 924 \text{ inches}
\]
How is Acoustical Deck affected by the holes in the webs?

**DDM04 – Example 19 and 22 – WR20 Roof Deck**

\[
G' = \frac{Et}{2(1 + \mu)(s/d) + \gamma_c D_n + C} \times K
\]

\[
= \left[ \frac{29500(0.0358)}{2(1 + 0.3)\left(\frac{9.124}{6}\right) + 0.9(4.306) + 4.86} \right] \times (1)
\]

\[
= 83.23 \text{ kips / inch} \rightarrow 96.6\% \text{ of stiffness of unperforated deck}
\]

Determine Strength

The nominal diaphragm shear strength is not affected since fasteners are installed in unperforated regions. (AISI S310 D1.4)

Observation: Acoustical Deck diaphragms behave much like unperforated deck diaphragms
Do I need reinforcing steel in a floor diaphragm?

Short answer, no … with some explanation needed.

• Testing to develop concrete filled diaphragm design method did not contain reinforcing (neither welded wire reinforcing or rebar).

• ANSI/SDI C “Standard for Composite Steel Floor Deck-Slabs” permits the use of either fibers (steel or synthetic) or steel reinforcing for temperature and shrinkage reinforcement.

• ANSI/SDI NC “Standard for Non-Composite Steel Floor Deck” permits 2 methods.
  • Deck as stay in place form, slab designed using ACI 318. ACI 318 will require temperature and shrinkage reinforcing.
  • Deck as total structural support, in this case the slab is not required to be designed by ACI 318, therefore no T&S reinforcing required.
Do I need to have an Evaluation Report for every diaphragm

**Short answer, no … with some explanation needed.**

- Evaluation reports are useful for evaluation of alternate products IAW Section 104 of the IBC

- Evaluation reports are NOT REQUIRED for products designed using IBC provisions or referenced standards within the IBC.

- 2018 IBC includes the AISI S310 Diaphragm Standard through IBC references to AISI S100, SDI-RD, SDI-C, SDI-NC Standards.
  - If diaphragm uses fasteners (welds or generic screws) that have strength and stiffness included in S310, the diaphragm is designed completely using Standards included in the IBC. No Report required.
  - If diaphragm uses fasteners that are proprietary, then the strength and stiffness of the fasteners must be determined, and an Evaluation Report is prudent.
Polling Question #2

The calculated shear capacity of a diaphragm is greatest for a diaphragm that spans:

A. 1 span
B. 2 spans
C. 3 spans
D. 4 spans
Polling Answers

Which of the following are limit states for determining diaphragm strength of deck?

A. Buckling Strength of the Deck
B. Edge Fastener Strength
C. Interior Fastener Strength
D. Corner Fastener Strength
E. All of the Above

The calculated shear capacity of a diaphragm is greatest for a diaphragm that spans:

A. 1 span
B. 2 spans
C. 3 spans
D. 4 spans
Register for SJI’s Next Webinar

Best Practices for Steel Joist, Joist Girder, Steel Deck Construction

March 18, 2020 1:00 pm EDT
THANK YOU

Questions?

and don’t be afraid to ask