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DIVISION: 05 00 00—METALS
Section: 05 05 23—Metal Fastenings
Section: 05 31 00—Steel Deck

REPORT HOLDER:

HILTI, INC.
7250 DALLAS PARKWAY, SUITE 1000
PLANO, TEXAS 75024
(800) 879-8000
www.us.hilti.com

EVALUATION SUBJECT:

STEEL DECK DIAPHRAGMS ATTACHED WITH HILTI S-MD 12-24 x $1^5/_8$ M HWH5 FRAME FASTENERS

1.0 EVALUATION SCOPE

Compliance with the following codes:

2012, 2009 and 2006 *International Building Code®* (IBC)

Property evaluated:

Structural

2.0 USES

Hilti's S-MD 12-24 x $1^5/_8$ M HWH5 frame fasteners are used to attach B, BI, and Verco PLB steel roof deck panels to supporting steel framing.

3.0 DESCRIPTION

3.1 Frame Fasteners:

The Hilti S-MD 12-24 x $1^5/_8$ M HWH5 self-drilling screw fasteners are case-hardened from carbon steel conforming to ASTM A510, Grade 1018 to 1022 with an electroplated zinc coating conforming to ASTM B633-13, SC 1, Type III. The fasteners also comply with ASTM C1513 and SAE J78 and have Hex Washer head styles. The fasteners are nominally 1.625-inch (41.3 mm) long and have a nominal 0.216-inch (5.5 mm) diameter with 24 threads per inch. Table 1 provides an illustration and additional information on the S-MD 12-24 x $1^5/_8$ M HWH5 fastener. The fasteners are collated for use in a tool recommended by the report holder.

3.2 Steel Deck Panels:

Steel deck panels must be No. 16, 18, 20 or 22 gage B, BI, or Verco PLB complying with Figure 4.

The B and BI steel deck panels must conform to the requirements of ASTM A653 SS, Grade 33 (minimum) with minimum G60 galvanized coating or must be painted or phosphatized steel complying with ASTM A1008-12 SS, Grade 33 (minimum).

The B and BI steel deck panels must be 36 inches (914 mm) in width with $1^1/_2$-inch-deep (38 mm) flutes spaced 6 inches (152 mm) on center. The B steel deck panels must have nestable sidelaps and the BI steel deck panels must have interlocking (standing seam) sidelaps.

The Verco PLB (interlocking) steel deck panels must be as recognized in ESR-1735P, except that the minimum tensile and yield strengths must comply with Table 4.

3.3 Sidelap Connectors:

The steel deck panel sidelap connections must be made with either Hilti S-SLC 01 or Hilti S-SLC 02 sidelap connectors recognized in ESR-2776; minimum Hilti No. 10 x $^3/_4$-inch-long (19.1 mm) HWH screws recognized in ESR-2196; button punches as described in Section 4.1.3; or Verco VSC2 sidelap connections as described in ESR-2776. Table 2 provides illustrations and additional information on the sidelap connectors.

3.4 Steel Support Framing:

Structural steel supports of the steel deck panels (such as gage purlin, bar joists, and structural steel shapes) must be manufactured from a code-compliant steel having minimum strength requirements of ASTM A653 (minimum yield strength of 33 ksi and minimum tensile strength of 45 ksi) for gage purlins or ASTM A36 for bar joists and structural steel shapes and minimum thicknesses as noted in the tables of this report.

4.0 INSTALLATION AND DESIGN

4.1 Installation:

4.1.1 General: The B, BI, and Verco PLB steel deck panels must be attached to steel support framing with the Hilti S-MD 12-24 x $1^5/_8$ M HWH5 frame fasteners (see Table 1) and the steel deck panel sidelaps must be attached with Hilti S-SLC 01 or S-SLC 02 sidelap connectors, minimum Hilti No. 10 x $^3/_4$ HWH screws, button punches, or Verco's VSC2 connection (see Table 2).

Steel deck panel ends must overlap a minimum of 2 inches (51 mm), as shown in Figure 3b. Endlap and corner lap conditions of two- and four-deck layers must be snug and tight to one another and the supporting steel frame, prior to frame fastener attachment. Standing seam interlocking-type sidelaps must be well engaged prior to sidelap connector installation.

The Hilti fasteners must be installed in accordance with Hilti's published installation instructions and the Verco VSC2 sidelap connections must be installed in accordance with Verco's published installation instructions.

4.1.2 Frame Fasteners: The Hilti S-MD 12-24 x $1^5/_8$ HWH5 frame fasteners:

- must be installed at all steel deck panel ends, interior supports, and edges parallel to the deck corrugations;
- must be centered not less than $^3/_8$ inch (9.5 mm) from the steel deck panel ends and not less than $^5/_{16}$ inch (7.9 mm) from the steel deck panel edges parallel to corrugations at the sidelaps;
- must penetrate though the supporting steel with a minimum of three threads protruding past the back side of the supporting steel.

The number of diaphragm edge frame fasteners at walls or transfer zones parallel to the deck corrugations must be equal to or greater than the number of sidelap connectors at nearest interior sidelaps.

See Table 1 for applicable steel support framing thicknesses; Figure 1 for frame fastener patterns; and Figures 3a and 3b for frame fastener installation details.

4.1.3 Sidelap Connectors: The Hilti S-SLC 01 and Hilti S-SLC 02 sidelap connectors , minimum Hilti No. 10 x $^3/_4$ HWH screws, button punches, and Verco's VSC2 sidelap connections:

- must be installed where steel deck panels are lapped by nesting or interlocking;
- must penetrate though the steel deck panel not in contact with the sidelap connector or screw head with a minimum of three threads protruding;
- must not exceed 36 inches (914.4 mm) on center.

See Table 2 for applicable steel deck panel thicknesses; and Figures 3c, 3d, and 3e for sidelap connection details.

Button-punching must be sharp and deep. The coating of the outer protruding nose of the punched lap should be "starred," indicating a near-penetration of the button punching tool.

4.2 Design:

4.2.1 General: All end, perimeter, and interior framing members and their attachments must be designed to resist all applied loads.

4.2.2 Diaphragm Classification: Diaphragms must be classified as flexible or rigid in accordance with IBC Chapter 16.

4.2.3 Diaphragm Shear Strength *(S_{ASD} or S_{LRFD})*: Diaphragm shears due to wind, earthquake, or other load combinations in accordance with IBC Chapter 16 must not exceed the diaphragm shear strength *(S_{ASD} or S_{LRFD})* calculated in accordance with this section which is based on steel deck panels installed in a minimum three-span condition.

The equation numbers or section numbers in parenthesis correspond to the equations provided in the Steel Deck Institute Diaphragm Design Manual, 3rd edition, September 2004 (SDI DDM03).

See Figure 5 for design example.

$$S_{ne} = \frac{Pn}{\ell} = \left(2 \times \alpha_1 + 2 \times \alpha_2 + n_e\right) \times \frac{Q_f}{\ell} \quad \text{(plf or N/m)}$$

(SDI DDM03 Eq. 2.2-2)

$$S_{ni} = \left\{2 \times A \times (\lambda - 1) + B\right\} \times \frac{Q_f}{\ell} \quad \text{(plf or N/m)}$$

(SDI DDM03 Eq. 2.2-4)

$$S_{nc} = Q_f \times \sqrt{\frac{N^2 \times B^2}{\ell^2 \times N^2 + B^2}} \quad \text{(plf or N/m)}$$

(SDI DDM03 Eq. 2.2-5)

S_n = Least of S_{ne}, S_{ni}, and S_{nc}, (plf or N/m)

S_{ASD} *or* $S_{LRFD} = CF \times S_n \le S_{buckling}$ (plf or N/m)

with:

$$B = n_s \times \alpha_s + \frac{1}{w^2} \times \left[2 \times 2 \times \Sigma\left(x_p^2\right) + 4\Sigma\left(x_e^2\right)\right]$$

(SDI DDM03 Section 2.2)

$$\lambda = 1 - \frac{1.5 \times \ell_v}{240 \times \sqrt{t}} \ge 0.7 \text{ for } \mathbf{SI:}\ \lambda = 1 - \frac{38 \times \ell_v}{369 \times \sqrt{t}} \ge 0.7$$

(SDI DDM03 Section 2.2)

$$\alpha_s = \frac{Q_s}{Q_f}$$

(SDI DDM03 Section 2.4)

where:

Design equation variables are given in Tables 3 and 4.

P_n = Nominal strength of diaphragm, lbf or N.

t = Steel deck panel base-metal thickness, inch or mm.

w = Panel width, inches or mm.

S_{ASD} = Allowable Diaphragm Shear Strength, plf or N/m.

S_{LRFD} = Factored Resistance Diaphragm Shear Strength, plf or N/m.

S_n = Nominal diaphragm shear strength, plf or N/m.

$S_{buckling}$ = Appropriate ASD or LRFD steel panel buckling strength from Table 7, plf or N/m.

ℓ_v = Span, ft or m.

ℓ = Panel length = 3 x ℓ_v, ft or m.

n_e = n_s = ℓ x 12 ÷ *SS* **or** n_e = n_s = 3 × *SPS*, as applicable.

For **SI:** n_e = n_s = ℓ x 1000 ÷ *SS* **or** n_e = n_s = 3 × *SPS*, as applicable.

CF = Conversion factor for Allowable Strength Design (ASD) and Load and Resistance Factor Design (LRFD) noted in Table 6.

SS = Specified sidelap fastener spacing, inches or mm (See Figure 2a).

SPS = Specified number of sidelap fasteners per panel span (See Figure 2b).

SPS = ℓ_v (span in feet) × 12/ SS (inches).

For **SI**: *SPS* = ℓ_v (span in meters) × 1000/ SS (millimeters).

Q_f = Nominal support connection strength from Table 4.

Q_s = Nominal sidelap connection strength from Table 4.

4.2.4 Diaphragm Stiffness (G′) or Flexibility Factor (F): The steel roof deck diaphragm stiffness (G′) or flexibility factor (F) must be calculated in accordance with this section which is based on steel deck panels installed in a minimum three-span condition.

Diaphragm span/depth limitations based on flexibility must comply with Table 10.

See Figure 5 for design example.

$$G' = \frac{E \times t}{3.78 + 0.9 \times D_n + C}, \quad \text{(kips/in. or kN/mm)}$$

(SDI DDM03 Eq. 3.2-3)

$$F = \frac{1000}{G'}, \quad \text{(micro-inches/lb or µm/N)}$$

with:

$$C = E \times \frac{t}{w} \times S_f \times \left(\frac{1}{\alpha_1 + \alpha_2 + n_s \times \frac{S_f}{S_s}} \right) \times \ell \times 12$$

(SDI DDM03 Eq. 3.3-1)

For **SI:** $$C = E \times \frac{t}{w} \times S_f \times \left(\frac{1}{\alpha_1 + \alpha_2 + n_s \times \frac{S_f}{S_s}} \right) \times \ell \times 1000$$

$$D_n = \frac{D}{\ell \times 12}$$

(SDI DDM03 Eq. 3.3-2)

For **SI:** $$D_n = \frac{D}{\ell \times 1000}$$

where:

Design equation variables are given in Tables 3 and 5.

E = Modulus of elasticity of steel, 29,500 ksi (203,395 MPa).

t = Steel deck panel base-metal thickness, inch or mm.

w = panel width, inches or mm.

ℓ_v = Span, ft or m.

ℓ = Panel length = 3 x ℓ_v, ft or m.

$n_e = n_s$ = ℓ × 12 ÷ *SS* **or** $n_e = n_s$ = 3 × *SPS*, as applicable.

For **SI:** $n_e = n_s$ = ℓ × 1000 ÷ *SS* **or** $n_e = n_s$ = 3 × *SPS*, as applicable.

S_f = Nominal support connection stiffness from Table 5.

S_s = Nominal sidelap connection stiffness from Table 5.

4.2.5 Diaphragm Deflections: Diaphragm deflection (Δ) must be calculated in accordance with the footnotes of Table 10. For seismic design, diaphragm deflection limits must comply with ASCE 7/SEI Chapter 12.

4.2.6 Allowable Tension (Pullout and Pullover) Design Values for S-MD 12-24 x 1$^5/_8$ M HWH5: Allowable tension design values (pullout and pullover) of the S-MD 12-24 x 1$^5/_8$ M HWH5 fasteners installed in steel support framing and steel deck panels are provided in Tables 8 and 9.

5.0 CONDITIONS OF USE

The steel deck diaphragms attached with Hilti S-MD 12-24 x 1$^5/_8$ M HWH5 frame fasteners and the sidelap connectors described in this report comply with, or are a suitable alternative to what is specified in, those codes listed in Section 1.0 of this report, subject to the following conditions:

5.1 The Hilti S-MD 12-24 x 1$^5/_8$ M HWH5 fasteners are manufactured, identified and installed in accordance with this report, Hilti's published installation instructions and the approved plans. If there is a conflict, this report governs.

5.2 No adjustment for duration of load is permitted.

5.3 Steel deck diaphragms may be zoned by varying steel deck panel gage and/or connections across a diaphragm to meet varying shear and flexibility demands.

5.4 For intermediate steel deck panel thicknesses or panel steel strengths, diaphragm strength and stiffness values shall be based on straight-line interpolation between values determined in accordance with Section 4.1, as described in the note at the end of the diaphragm design example shown in Figure 5.

5.5 The design of the steel deck panels for vertical loads is outside the scope of this report.

5.6 Calculations demonstrating compliance with this report must be submitted to the code official for approval. The calculations must be prepared by a registered design professional where required by the statutes of the jurisdiction in which the project is to be constructed.

5.7 Hilti fasteners may be used for attachment of steel deck roof systems temporarily exposed to the exterior during construction prior to application of built-up roof covering systems. The fasteners on permanently exposed steel deck roof coverings must be covered with a corrosion-resistant paint or sealant. For permanently exposed steel deck roof covering installations, the roof covering system's compliance with Chapter 15 of the IBC must be justified to the satisfaction of the code official.

6.0 EVIDENCE SUBMITTED

6.1 Data in accordance with the ICC-ES Acceptance Criteria for Steel Deck Roof and Floor Systems (AC43), dated October 2010 (editorially revised September 2013).

6.2 Data in accordance with the ICC-ES Acceptance Criteria for Tapping Screw Fasteners (AC118), dated June 2012.

7.0 IDENTIFICATION

The S-MD 12-24 x $1^5/_8$ M HWH5 fasteners are identified by an "H" stamped on the fastener head. All fasteners are packaged in containers noting the fastener type, the report holder's name and the evaluation report number (ESR-3693).

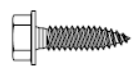
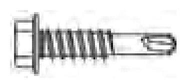
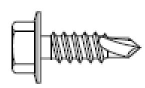
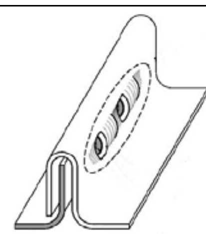
TABLE 1—FRAME FASTENER[1]

Steel Support Framing Thickness (t_f)	Fastener Type
0.0598 in. ≤ t_f ≤ $^1/_4$ in.	 S-MD 12-24 x $1^5/_8$ M HWH5

For **SI:** 1 inch = 2.54 mm

[1]Steel support framing must comply with the minimum strength requirements of ASTM A653 for gage purlins or ASTM A36 for bar joists and structural steel shapes as indicated in Section 3.4.

TABLE 2—SIDELAP CONNECTOR SELECTOR GUIDE

Steel Deck Panel Thicknesses	Fastener Type
Nos. 22, 20, 18 gage B and BI steel roof decks[1]	 Hilti S-SLC 01 M HWH (Recognized in ESR-2776)
Nos. 22, 20, 18, 16 gage B and BI steel roof decks[1]	 Hilti S-SLC 02 M HWH (Recognized in ESR-2776)
Nos. 22, 20, 18, 16 gage B and BI steel roof decks[1]	 Hilti No. 10 HWH Screw (Recognized in ESR-2196)
Nos. 22, 20, 18, 16 gage BI steel roof decks	Button Punch (As described in this report)
Nos. 22, 20, 18, 16 gage Verco PLB Deck steel roof decks	 Verco's VSC 2 Connection (As described in ESR-2776)

[1]These sidelap connectors require BI deck to be screwable.

TABLE 3—DIAPHRAGM STRENGTH (*S*) AND STIFFNESS FACTOR (*G′*) EQUATION VARIABLE VALUES
(to be used with equations in Sections 4.2.3 and 4.2.4)

DECK TYPE	PANEL WIDTH, w, inches	FRAME FASTENER PATTERN[1]	α_1 – END DISTRIBUTION FACTOR	α_2 – PURLIN DISTRIBUTION FACTOR	Σx_e^2, in.^{2}	Σx_p^2, in.^{2}	A	N ft.$^{-1}$	D – WARPING CONSTANT, IN. FOR STEEL DECK PANEL GAGE THICKNESS (MINIMUM BASE-STEEL THICKNESS IN inches)			
									No. 22 (0.0295)	No. 20 (0.0358)	No. 18 (0.0474)	No. 16 (0.0598)
B	36	36/11	3.667	3.667	1,944	1,944	2	3.000	1,548	1,164	756	540
		36/9	3.000	3.000	1,656	1,656	2	2.333	1,548	1,164	756	540
		36/7	2.000	2.000	1,008	1,008	1	2.000	1,548	1,164	756	540
		36/5	1.667	1.667	936	936	1	1.333	9,096	6,804	4,464	3,144
		36/4	1.333	1.333	720	720	1	1.000	12,864	9,624	6,312	4,452
		36/3	1.000	1.000	648	648	1	0.667	26,508	19,824	13,008	9,180
BI or Verco PLB	36	36/11	3.667	3.667	1,944	1,944	2	3.667	1,548	1,164	756	540
		36/9	3.000	3.000	1,656	1,656	2	3.000	1,548	1,164	756	540
		36/7	2.000	2.000	1,008	1,008	1	2.333	1,548	1,164	756	540
		36/5	1.667	1.667	936	936	1	1.667	9,096	6,804	4,464	3,144
		36/4	1.333	1.333	720	720	1	1.333	12,864	9,624	6,312	4,452

For **SI:** 1inch = 25.4 mm, 1 in.^{2} = 645 mm^2, 1 ft^{-1} = 3.28m^{-1}.

[1]Reference Figure 1 for depictions of frame fastener patterns.

TABLE 4—FRAME FASTENER STRENGTH (Q_f) AND SIDELAP CONNECTION STRENGTH (Q_s)
(to be used with equations in Section 4.2.3)

CONFIGURATION				STEEL DECK PANEL GAGE THICKNESS (MINIMUM BASE-STEEL THICKNESS IN inches)			
				No. 22 (0.0295)	No. 20 (0.0358)	No. 18 (0.0474)	No. 16 (0.0598)
Deck Type[1]	Minimum Deck Tensile, F_u, (Yield, F_y) Strengths, ksi	Frame Fastener & Steel Support Framing Thickness (t_f) IN inches[2]	Sidelap Connector[3]	Q_f, (lbf)	Q_f, (lbf)	Q_f, (lbf)	Q_f, (lbf)
				Q_s, (lbf)	Q_s, (lbf)	Q_s, (lbf)	Q_s, (lbf)
B or BI	45 (33)	S-MD 12-24x1$^5/_8$ M HWH5 0.0598 ≤ t_f < $^1/_8$	S-SLC 01 M HWH or S-SLC 02 M HWH	1,016	1,233	1,632	1,860
				844	1,260	1,701	2,024
		S-MD 12-24x1$^5/_8$ M HWH5 $^1/_8$ ≤ t_f ≤ $^1/_4$	S-SLC 01 M HWH or S-SLC 02 M HWH	1,193	1,661	1,860	1,860
				844	1,260	1,701	2,024
		S-MD 12-24x1$^5/_8$ M HWH5 0.0598 ≤ t_f < $^1/_8$	Minimum No. 10 x $^3/_4$ HWH Screw	1,016	1,233	1,632	1,860
				634	770	1019	1,286
		S-MD 12-24x1$^5/_8$ M HWH5 $^1/_8$ ≤ t_f ≤ $^1/_4$	Minimum No. 10 x $^3/_4$ HWH Screw	1,193	1,661	1,860	1,860
				634	770	1019	1,286
BI	45 (33)	S-MD 12-24x1$^5/_8$ M HWH5 0.0598 ≤ t_f < $^1/_8$	Button Punch	1,016	1,233	1,632	1,860
				209	308	539	858
		S-MD 12-24x1$^5/_8$ M HWH5 $^1/_8$ ≤ t_f ≤ $^1/_4$	Button Punch	1,193	1,661	1,860	1,860
				209	308	539	858
Verco PLB	65 (50)	S-MD 12-24x1$^5/_8$ M HWH5 0.0598 ≤ t_f < $^1/_8$	Verco VSC2	1,016	1,233	1,632	1,860
				2,067	2,823	4,323	5,835
		S-MD 12-24x1$^5/_8$ M HWH5 $^1/_8$ ≤ t_f ≤ $^1/_4$	Verco VSC2	1,193	1,661	1,860	1,860
				2,067	2,823	4,323	5,835

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N, 1 ksi = 6.89 MPa.

[1]See Figure 4.
[2]See Table 1.
[3]See Table 2.

TABLE 5—DIAPHRAGM STIFFNESS (G′) EQUATION VARIABLE VALUES
(to be used with equations in Section 4.2.4)

CONFIGURATION				STEEL DECK PANEL GAGE THICKNESS (MINIMUM BASE-STEEL THICKNESS IN inches)			
				No. 22 (0.0295)	No. 20 (0.0358)	No. 18 (0.0474)	No. 16 (0.0598)
Deck Type[1]	Minimum Deck Tensile, F_u, (Yield, F_y) Strengths, ksi	Frame Fastener & Steel Support Framing Thickness (t_f) in inches[2]	Sidelap Connector[3]	S_f, in./kip	S_f, in./kip	S_f, in./kip	S_f, in./kip
				S_S, in./kip	S_S, in./kip	S_S, in./kip	S_S, in./kip
B or BI	45 (33)	S-MD 12-24x1$^5/_8$ HWH5 $0.0598 \le t_f \le {}^1/_4$	S-SLC 01 M HWH S-SLC 02 M HWH Minimum No. 10 x $^3/_4$ HWH Screw	0.0076	0.0069	0.0060	0.0053
				0.0175	0.0159	0.0138	0.0123
BI	45 (33)	S-MD 12-24x1$^5/_8$ HWH5 $0.0598 \le t_f \le {}^1/_4$	Button Punch	0.0076	0.0069	0.0060	0.0053
				0.1747	0.1586	0.1378	0.1227
Verco PLB	65 (50)	S-MD 12-24x1$^5/_8$ HWH5 $0.0598 \le t_f \le {}^1/_4$	Verco VSC2	0.0076	0.0069	0.0060	0.0053
				0.0360	0.0253	0.0115	0.0074

For **SI:** 1 inch = 25.4 mm, 1 in/kip = 5.7 mm/kN, 1 ksi = 6.89 MPa.

[1]See Figure 4.
[2]See Table 1.
[3]See Table 2.

TABLE 6—CONVERSION FACTORS, CF, FOR ALLOWABLE STRENGTH DESIGN (ASD)
AND LOAD AND RESISTANCE FACTOR DESIGN (LRFD)[3]

DESIGN METHOD	LOAD CONDITIONS FOR STEEL ROOF DECK	CONVERSION FACTORS, CF[1,2]
ASD	Diaphragms subjected to earthquake loads or load combinations which include earthquake loads	0.400
ASD	Diaphragms subjected to wind loads or load combinations which include wind loads	0.426
ASD	Diaphragms subject to all other load combinations	0.400
LRFD	Diaphragms subjected to earthquake loads or load combinations which include earthquake loads	0.650
LRFD	Diaphragms subjected to wind loads or load combinations which include wind loads	0.700
LRFD	Diaphragms subject to all other load combinations	0.650

[1]The conversion factors (CF) are multiplication factors to be applied to the nominal diaphragm shear strength (S_n) to determine either S_{ASD} or S_{LRFD}.
[2]Conversion factors have been determined from Table D5, of the AISI North American Specification for the Design of Cold-Formed Steel Structural Members, AISI S100-07/S2-10 for the 2012 IBC, AISI S100-07 for the 2009 IBC, and NAS-01 with the 2004 Supplement for the 2006 IBC.
[3]Steel roof deck diaphragm resistance must be limited to lesser of values computed using Section 4.2.3 with this table, and the corresponding respective ASD and LRFD buckling diaphragm shear capacities ($S_{buckling}$) in Table 7 of this report.

TABLE 7—ASD AND LRFD DIAPHRAGM SHEAR STRENGTHS (plf) FOR BUCKLING, $S_{buckling}$ [1,2,3]

DECK TYPE	STEEL DECK PANEL GAGE THICKNESS (MINIMUM BASE-STEEL THICKNESS IN inches)	MINIMUM MOMENT OF INERTIA[4], I (in^4/ft)	SPAN (feet-inches)									
			3′-0"	4′-0"	5′-0"	6′-0"	7′-0"	8′-0"	9′-0"	10′-0"	11′-0"	12′-0"
ASD												
B, BI and Verco PLB	22 (0.0295)	0.152	8,444	4,750	3,040	2,111	1,551	1,188	938	760	628	528
	20 (0.0358)	0.198	11,000	6,188	3,960	2,750	2,020	1,547	1,222	990	818	688
	18 (0.0474)	0.284	15,778	8,875	5,680	3,944	2,898	2,219	1,753	1,420	1,174	986
	16 (0.0598)	0.355	19,702	11,094	7,100	4,931	3,622	2,773	2,191	1,775	1,467	1,233
LRFD												
B, BI and Verco PLB	22 (0.0295)	0.152	13,511	7,600	4,864	3,378	2,482	1,900	1,501	1,216	1,005	844
	20 (0.0358)	0.198	17,600	9,900	6,336	4,400	3,233	2,475	1,956	1,584	1,309	1,100
	18 (0.0474)	0.284	25,244	14,200	9,088	6,311	4,637	3,550	2,805	2,272	1,878	1,578
	16 (0.0598)	0.355	31,556	17,750	11,360	7,889	5,796	4,438	3,506	2,840	2,347	1,972

For **SI:** 1 inch = 25.4 mm, 1ft = 0.3048 m, 1 plf = 0.0146 N/mm, 1 in^4/ft = 1,368 mm^4/mm

[1] Load values are based upon a safety factor of 2.00 for ASD or a ϕ factor of 0.80 for LRFD.

[2] Diaphragm shears in this table are for steel deck buckling failure mode only and are to be used as prescribed in Section 4.2.3 of this report. If design condition is not tabulated, diaphragm shears for buckling may be calculated using the following equations:

For ASD, $S_{buckling} = (I*10^6/(l_v)^2)/2.0$; For LRFD, $S_{buckling} = (I*10^6/(l_v)^2)*0.8$

[3] Diaphragm resistance must be limited to lesser of values in this table and the corresponding respective ASD and LRFD shear capacities derived using Section 4.2.3 and Table 6 of this report.

[4] The tabulated moment of inertia, I, is the required gross moment of inertia of the steel deck panels about the horizontal neutral axis of the panel cross section.

TABLE 8—ALLOWABLE (ASD) TENSION PULLOUT LOADS TO RESIST TENSION (UPLIFT) LOADS FOR STEEL ROOF DECK PANELS ATTACHED WITH S-MD 12-24 X 1$^5/_8$ M HWH5 FASTENERS (lbf)[1, 4]

FASTENER	MINIMUM STEEL SUPPORT FRAMING THICKNESS IN inches (GAGE)					
	0.0598 (16)[2]	0.0747 (14)[2]	0.1046 (12)[2]	$^1/_8$ [3]	$^3/_{16}$ [3]	$^1/_4$ [3]
S-MD 12-24 x 1$^5/_8$ M HWH5	215	265	370	505	505	505

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N

[1]Tabulated allowable (ASD) values based upon a Ω safety factor of 3.0 per AISI S100. To obtain LRFD pullout capacities, the tabulated values must be multiplied by 1.6.

[2]The tabulated pullout load values are based upon calculations in accordance with Section E4 of AISI S100 with steel meeting the minimum strength requirements of ASTM A653.

[3]The tabulated pullout load values based upon testing performed in base steel meeting the minimum strength requirements of ASTM A36.

[4]Allowable tension pullout values must be compared with allowable tension pullover load values. Use lesser value.

TABLE 9—ALLOWABLE (ASD) TENSION PULLOVER LOADS TO RESIST TENSION (UPLIFT) LOADS FOR STEEL ROOF DECK PANELS ATTACHED WITH S-MD 12-24 X 1$^5/_8$ M HWH5 FASTENERS (lbf)[1, 2, 3]

FASTENER	STEEL DECK PANEL GAGE THICKNESS[2] (MINIMUM BASE-STEEL THICKNESS IN inches)			
	No. 22 (0.0295)	No. 20 (0.0358)	No. 18 (0.0474)	No. 16 (0.0598)
S-MD 12-24 x 1$^5/_8$ M HWH5	275	335	445	560

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N.

[1]Tabulated allowable (ASD) values are based upon a Ω safety factor of 3.0 per AISI S100. To obtain LRFD pullover capacities, the tabulated values must be multiplied by 1.6.

[2]Tabulated pullout load values are based upon calculations in accordance with Section E4 of AISI S100 with minimum ASTM A653 SS Grade 33 steel deck as described in Section 3.2 of this report.

[3]Allowable tension pullover values must be compared with allowable tension pullout load values. Use lesser value.

TABLE 10—DIAPHRAGM FLEXIBILITY LIMITATION[1,2,3,4,5]

F	MAXIMUM SPAN IN FEET FOR MASONRY OR CONCRETE WALLS	SPAN-DEPTH LIMITATION			
		Rotation Not Considered in Diaphragm		Rotation Considered in Diaphragm	
		Masonry or Concrete Walls	Flexible Walls	Masonry or Concrete Walls	Flexible Walls
More than 150	Not used	Not used	2:1	Not used	$1^1/_2$:1
70 – 150	200	2:1 or as required for deflection	3:1	Not used	2:1
10 – 70	400	$2^1/_2$:1 or as required for deflection	4:1	As required for deflection	$2^1/_2$:1
1 – 10	No limitation	3:1 or as required for deflection	5:1	As required for deflection	3:1
Less than 1	No limitation	As required for deflection	No limitation	As required for deflection	$3^1/_2$:1

For **SI:** 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 plf = 14.594 N/m, 1 psi = 6894 Pa.

[1]Diaphragms must be investigated regarding their flexibility and recommended span-depth limitations.

[2]Diaphragms supporting masonry or concrete walls must have their deflections limited to the following:

$$\Delta_{wall} = \frac{H^2 f_c}{0.01\, Et} \quad \text{For SI: } \Delta_{wall} = \frac{694{,}000\, H^2 f_c}{Et}$$

where:

H = Unsupported height of wall in feet or millimeters.

t = Thickness of wall in inches or millimeters.

E = Modulus of elasticity of wall material for deflection determination in pounds per square inch or kilopascals.

f_c = Allowable compression strength of wall material in flexure in pounds per square inch or kilopascals. For concrete $f_c = 0.45\, f'_c$. For masonry, $f_c = F_b = 0.33\, f'_m$.

[3]The total deflection Δ of the diaphragm may be computed from the equation: $\Delta = \Delta_f + \Delta_w$.

where:

Δ_f = Flexural deflection of the diaphragm determined in the same manner as the deflection of beams.
Δ_w = The web deflection may be determined by the equation:

$$\Delta_w = \frac{q_{ave}\, L\, F}{10^6} \quad \text{For SI: } \Delta_w = \frac{q_{ave}\, L\, F}{175}$$

where:

L = Distance in feet between vertical resisting element (such as shear wall) and the point to which the deflection is to be determined.

q_{ave} = Average shear in diaphragm in pounds per foot or newtons per meter over length *L*.

F = Flexibility factor: The average microinches or micrometers (μm) a diaphragm web will deflect in a span of 1 foot (m) under a shear of 1 pound per foot (N/m).

[4]When applying these limitations to cantilevered diaphragms, the allowable span depth ratio will be half that shown.

[5]Diaphragm classification (flexible or rigid) and deflection limits must comply with Section 4.1.

[6]The general deflection equation for rectangular symmetrical diaphragms only:

$(d^2y)/(dx^2) = M / EI + q / B\, G'$

For a uniformly loaded rectangular diaphragm on a simple span, the maximum deflection at the centerline of the diaphragm is:

Δ = $5(1728)qL^4 / 384\, EI + qLF/ 10^6$ For SI: $\Delta = 5\,(1000)^4 qL^4/384\, EI + qLF/10^6$

Δ = Diaphragm deflection, inches (mm).

q = Wind or seismic load, kips per lineal foot (N/m)

L = Length of diaphragm normal to load, feet (m).

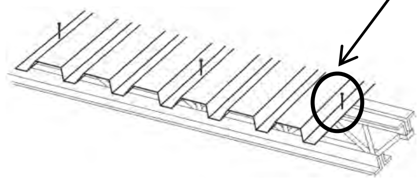
B = Width of diaphragm parallel to load, feet (m).

E = Modulus of elasticity of supporting steel.

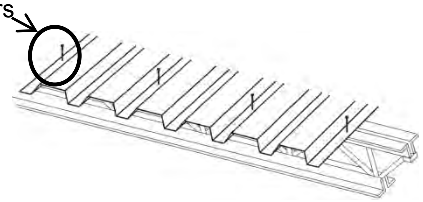
I = Moment of inertia, inches^4 (mm^4).

Nonrectangular diaphragms, nonsymmetrical diaphragms with re-entrant corners or diaphragms subjected to torsional loadings require special design consideration.

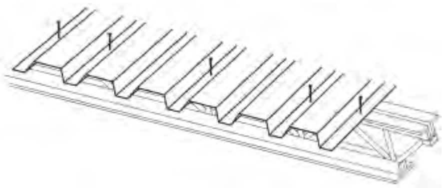
side frame fasteners



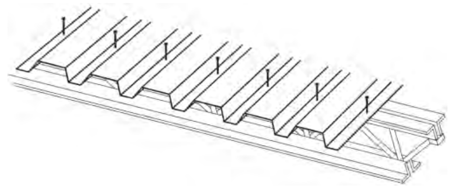
Pattern 36/3



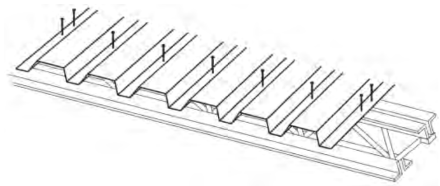
Pattern 36/4



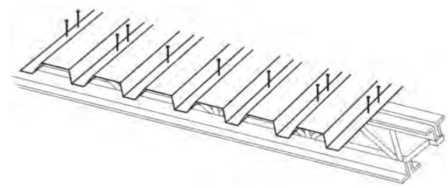
Pattern 36/5



Pattern 36/7



Pattern 36/9

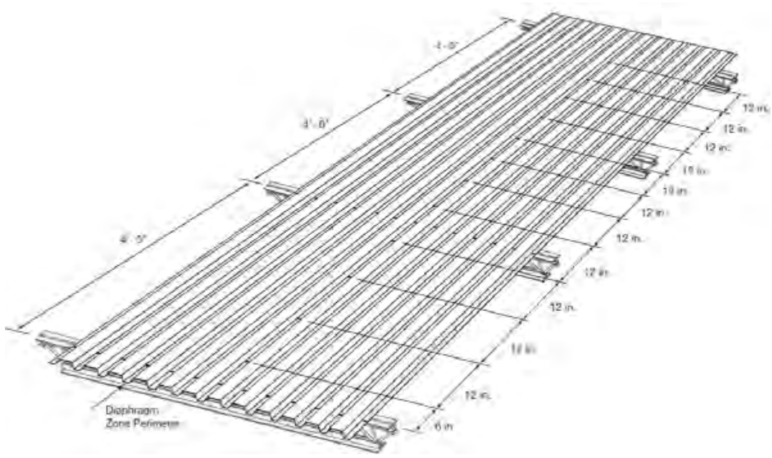


Pattern 36/11

Notes:

1. B-Deck shown for illustration purposes only. See Section 3.2 and Figure 4 for applicable deck types.
2. Bar joist shown for illustration purposes only. Connection to structural steel members and gage purlins also allowed by this report as set forth in Table 1.
3. For B-Deck, the side frame fasteners are installed through both connecting steel decks and into the supporting framing.
4. For BI-Deck and Verco PLB Deck, the same number of side frame fasteners are installed on each side of the sidelap and into supporting framing.

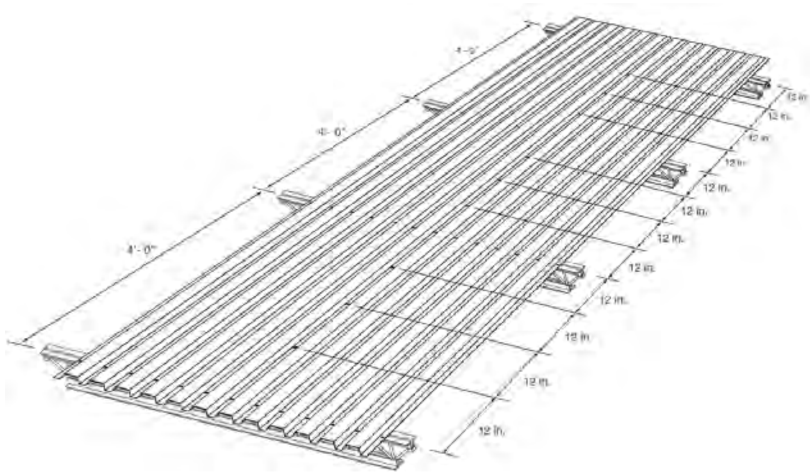
FIGURE 1—B-, BI- AND PLB DECK FRAME FASTENER PATTERNS



Example: A 4'-0" span with a 12 in. sidelap connector spacing will typically start 6 in. from the first joist / beam line at the diaphragm zone perimeter, and then have equal spacings of 12 in. across the entire diaphragm length or width, off-set at the interior joist / beam locations. The interior joist / beam fastening locations are frame fasteners and not sidelap connectors. This convention of specifying sidelap connectors by spacing does not consider each deck span independently as a discrete element, but rather as a larger steel deck diaphragm system consisting of 3 or more spans.

Note: If the sidelap connector spacing does not divide evenly into the span length, some spans may have more sidelap connectors than adjacent spans. For this reason, n_e and n_s may not be whole numbers.

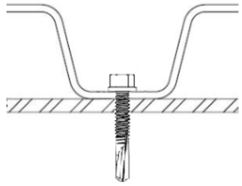
2a: SPECIFIED BY SIDELAP CONNECTOR SPACING (SS)



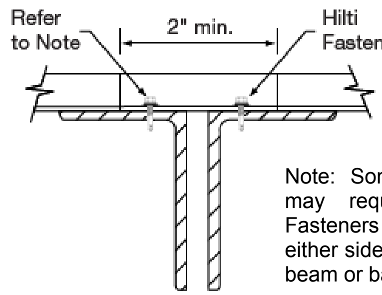
Example: A 4'-0" span specified with 3 sidelap connectors per span will have 3 sidelap connectors evenly spaced 12 in. from each joist/ beam line and each other making 4 equal 12 in. spaces per span. This convention of specifying sidelap connectors by the number of sidelap connectors per span considers each deck span independently as a discrete element.

2b: SPECIFIED BY NUMBER OF SIDELAP CONNECTORS PER SPAN (SPS)

FIGURE 2—EXAMPLE ILLUSTRATION OF SIDELAP CONNECTOR SPECIFICATION CONVENTIONS SPACING OR NUMBER PER SPAN

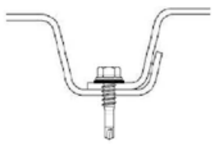


3a. S-MD 12-24 x 1$^5/_8$ M HWH Fastener Attachment of Steel Deck to Frame

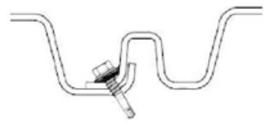


Note: Some fastener patterns may require two fasteners. Fasteners may be installed on either side of the structural steel beam or bar joist.

3b. Steel Deck Endlap Condition

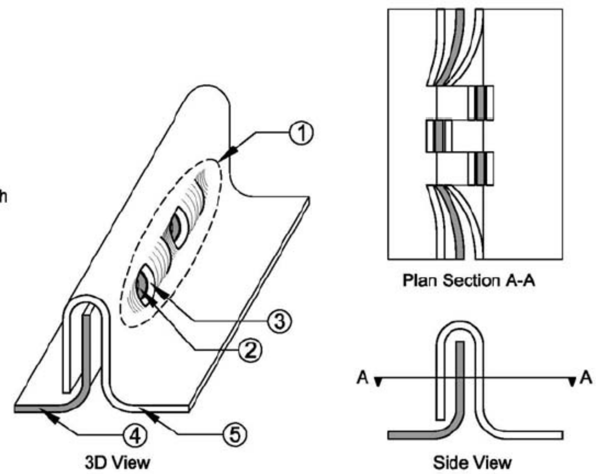


3c. Sidelap Connector with B Deck



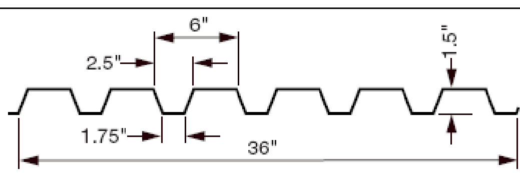
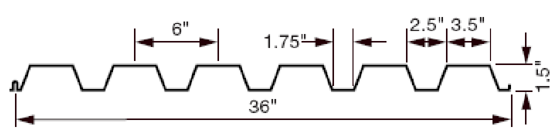
3d. Sidelap Connector with BI Deck with Screwable Sidelap

① PunchLok®II system connection - "2M/3F" condition as shown, the side of the tool with two blades starts off placed on the male side of the sidelap.
② Sheared surface of male leg.
③ Sheared surface of female leg.
④ Male leg / sheet.
⑤ Female leg / sheet.



3e. Verco VSC 2 Connection with PLB

FIGURE 3—TYPICAL FRAME, ENDLAP AND SIDELAP CONNECTIONS

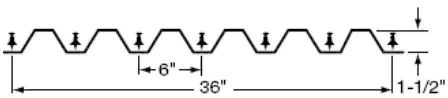
Deck Type	Nominal Dimensions	Deck Type	Nominal Dimensions
B-Deck		BI-Deck and Verco PLB Deck	

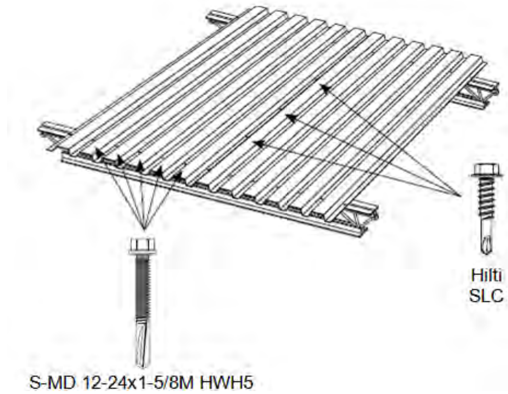
Notes:

1. B-Deck (nestable) and BI-Deck (interlocking) deck panel thicknesses must be 16, 18, 20 or 22 gage steel [(54, 43, 33 or 27 mil designations) (0.0598, 0.0474, 0.0358 or 0.0295 inch) (1.51, 1.21, 0.91 or 0.76 mm)], respectively. Intermediate steel deck panel thicknesses may be used (Reference Section 5.4 of this report).
2. PLB (interlocking) deck panel thicknesses must be 16, 18, 20 or 22 gage steel [(54, 43, 33 or 27 mil designations) (0.0598, 0.0478, 0.0359 or 0.0299 inch) (1.51, 1.21, 0.91 or 0.76 mm)], respectively. Intermediate steel deck panel thicknesses may be used (Reference Section 5.4 of this report).
3. BI-Deck (interlocking) deck panels must have screwable sidelap edges for use with Hilti SLC01 and SLC02 sidelap connectors and minimum Hilti No. 10 x $^3/_4$ HWH Screws.

FIGURE 4—STEEL DECK PANELS

Given:
Load Type: Wind Design: ASD
Span (ℓ_v): 6′-0"
Deck: No. 20 gage $1^1/_2$" B-Deck (33 ksi)
Steel Support Framing: Bar Joist with $^1/_4$" Thick Top Chord
Frame Fastener Pattern: 36/7
Sidelap Fastener Spacing (SS): 12" o.c.
Frame Fastener: S-MD 12-24 x $1^5/_8$ M HWH5
Sidelap Fastener: S-SLC 02 M HWH





Problem:
Determine Allowable (ASD) Diaphragm Shear Strength (S_{ASD}) and Stiffness (G’) for the given steel deck diaphragm.

Calculation Steps	SDI DDM03 Ref.	Report Ref.
Step 1: Calculate Nominal Diaphragm Shear Strength Limited by Edge Fasteners: $S_{ne} = \{2\times\alpha_1 + 2\times\alpha_2 + n_e\}\times\frac{Q_f}{\ell} = \{2\times 2 + 2\times 2 + 18\}\times\frac{1661}{18} = 2{,}399 \text{ plf}$ where: $\alpha_1 = \alpha_2 = 2$ $\ell_v = \text{Span} = 6 \text{ ft}$ $\ell = \text{n}\times\ell_v = 3\times 6 = 18 \text{ ft}$ $\text{n} = 3$ $n_e = n_s = \frac{n\times\ell_v\times 12}{SS} = \frac{3\times 6\times 12}{12} = 18$	Eq. 2.2-2	Section 4.2.3 and Tables 3 & 4
Step 2: Calculate Nominal Diaphragm Shear Strength Limited by Interior Panel Fasteners: $S_{ni} = \{2\times A\times(\lambda - 1) + B\}\times\frac{Q_f}{\ell} = \{2\times 1\times(0.802 - 1) + 19.88\}\times\frac{1661}{18} = 1{,}798 \text{ plf}$ where: $\lambda = 1 - \frac{1.5\times\ell_v}{240\times\sqrt{t}} = 1 - \frac{1.5\times 6}{240\times\sqrt{0.0358}} = 0.802 \geq 0.7$ $B = n_s\times\alpha_s + \frac{1}{\text{w}^2}\times\left[2\times 2\times\sum(x_p^2) + 4\sum(x_e^2)\right] = 18\times 0.759 + \frac{[2\times 2\times 1008 + 4\times 1008]}{1296} = 19.88$ $\alpha_s = \frac{Q_s}{Q_f} = \frac{1260}{1661} = 0.759$	Eq. 2.2-4	Section 4.2.3 and Tables 3 & 4
Step 3: Calculate Nominal Diaphragm Shear Strength Limited by Corner Fasteners: $S_{nc} = Q_f\times\sqrt{\frac{N^2\times B^2}{\ell^2\times N^2 + B^2}} = 1661\times\sqrt{\frac{2.00^2\times 19.88^2}{18^2\times 2.00^2 + 19.88^2}} = 1{,}606 \text{ plf}$	Eq. 2.2-5	Section 4.2.3 and Tables 3 & 4
Step 4: Determine Nominal Diaphragm Shear Strength: S_n = least of S_{ne}, S_{ni}, and S_{nc} = S_{nc} = 1,606 plf		Section 4.2.3 and Table 4

FIGURE 5—DIAPHRAGM DESIGN EXAMPLE

Calculation Steps (continued)	SDI DDM03 Ref.	Report Ref.
Step 5: Apply Appropriate Conversion Factor to Determine Allowable Diaphragm Shear Strength: $S_{ASD\ (wind)} = S_n \times$ Conversion Factor $= 1{,}606 \times 0.426 = 684$ plf	Section 2.4	Section 4.2.3 and Table 6
Step 6: Check to See if Steel Deck Buckling Controls: $S_{buckling\ (ASD)} = 2{,}750$ plf $> S_{ASD\ (wind)}$ therefore, $S_{ASD\ (wind)} = \boxed{684 \text{ plf}}$		Section 4.2.3 and Table 7
Step 7: Calculate Diaphragm Stiffness: $G' = \dfrac{29{,}500 \times t}{3.78 + 0.9 \times D_n + C} = \dfrac{29{,}500 \times 0.0358}{3.78 + 0.9 \times 5.39 + 3.70} = \boxed{85.65 \text{ kips/in.}}$ $F = \dfrac{1000}{G'} = \dfrac{1000}{85.65} = 11.68 \text{ micro-inches/lb}$ where: $D_n = \dfrac{D}{\ell \times 12} = \dfrac{1164}{18 \times 12} = 5.39$ $\qquad E = 29{,}500$ $C = E \times \dfrac{t \times S_f}{w} \times \dfrac{1}{\alpha_1 + \alpha_2 + n_s \times \dfrac{S_f}{S_s}} \times \ell \times 12$ $= 29{,}500 \times \dfrac{0.0358 \times 0.0069}{36} \times \dfrac{1}{2 + 2 + 18 \times \dfrac{0.0069}{0.0159}} \times 18 \times 12 = 3.70$	Eq. 3.3-1, 3.3-2, and 3.3-3	Section 4.2.4 and Tables 3 & 5

FIGURE 5—DIAPHRAGM DESIGN EXAMPLE (continued)

NOTE: Straight-line interpolation between different steel deck thicknesses and steel deck strengths for the calculation of diaphragm shear strength values is permitted.