COMMENTARY S310-C

D1 Diaphragm Shear Strength per Unit Length Controlled by Connection Strength, S_{nf}

The *diaphragm nominal shear strength* [*resistance*] per unit length controlled by *connection nominal strengths* [*resistance*] in each *panel* includes fastener failure or *connection* failure in the *panel*. *Standard* Eq. D1-1 includes a (λ-1) relaxation term, which represents edge of *panel* corner *buckling* at *support connections* along *side-laps* at *panel* ends. That corner *connection* cannot develop its full *nominal strength* [*resistance*]. This relaxation occurs at the compression corners as the *panel* racks in-plane but the reduction is applied in both directions (tension and compression) for simplicity. *Standard* Eq. D1-2 recognizes the orthogonal force components and greater demand at the corner *connections* in each *panel* as illustrated in *Standard* Appendix 2 Figure 2.2-1, which is showing a particular *panel* at the *diaphragm* edge and not the general case at *interior panels* (in the general case, w_a or w_u are not present).

The *diaphragm nominal shear strength* [*resistance*] controlled by *connections* at *edge panels* includes fastener failure or local *panel* failure at fasteners along lines where the shear is transmitted from the *diaphragm* to the structure's *lateral force-resisting system* (*shear walls*, braced bents, or *moment frames*). These *connections* are spaced parallel to the *panel*, and typically edge supports are required in the shear transfer plane to allow installation of these edge *connections*. *Standard* Eq. D1-3 addresses this *diaphragm* strength limit, and the *required strength* [reaction due to *factored loads*] at frames is compared with this *available shear strength* [*factored resistance*]. It should be noted that the reaction at an interior support usually does not equal the shear in the *diaphragm panel* at the point of transfer, as beam reactions do not equal shear at interior reactions.

Standard Eq. D1-3, S_{ne}, includes the contribution of all edge fasteners, n_e, and the *support connections* between the *panel* center line and the reaction line in the *edge panel*. *Standard* Eq. D1-3 is based on a symmetric *connection* pattern at each support in an *edge panel*, while the patterns can vary between *interior* and *exterior supports*. It is acceptable to only consider n_e and the *support connections* at the edge by letting $\alpha_1 = 1$ and $\alpha_2 = 1$ in *Standard* Eq. D1-3. Since adding *connections* at the edge parallel with the *panel* span normally does not significantly impact installation time, many designers will not let S_{ne} control *diaphragm* capacity. Common practice requires n_e to equal or exceed n_s.

The *diaphragm nominal shear strength* [*resistance*] controlled by *connections* along the edge perpendicular to the *panel* span includes fastener failure or local *panel* failure at *exterior supports*. Attachment pattern has a significant influence on this *diaphragm* strength limit which is addressed by *Standard* Eq. D1-4. This theory was presented in a report by Nunna (2018a).

S_{np} is the smallest value of bottom flute fastener(s) shear strength per tributary width among all the bottom flute support *connections*. Some examples of the controlling combination for w_t and n_d used in *Standard* Eq. D1-4 are shown in Figure C-D1-1. For fastener patterns with a support fastener in every bottom flute, w_t is equal to the pitch of the deck. For *cellular deck*, the average number of *support* fasteners per unit width is used to calculate S_{np}.

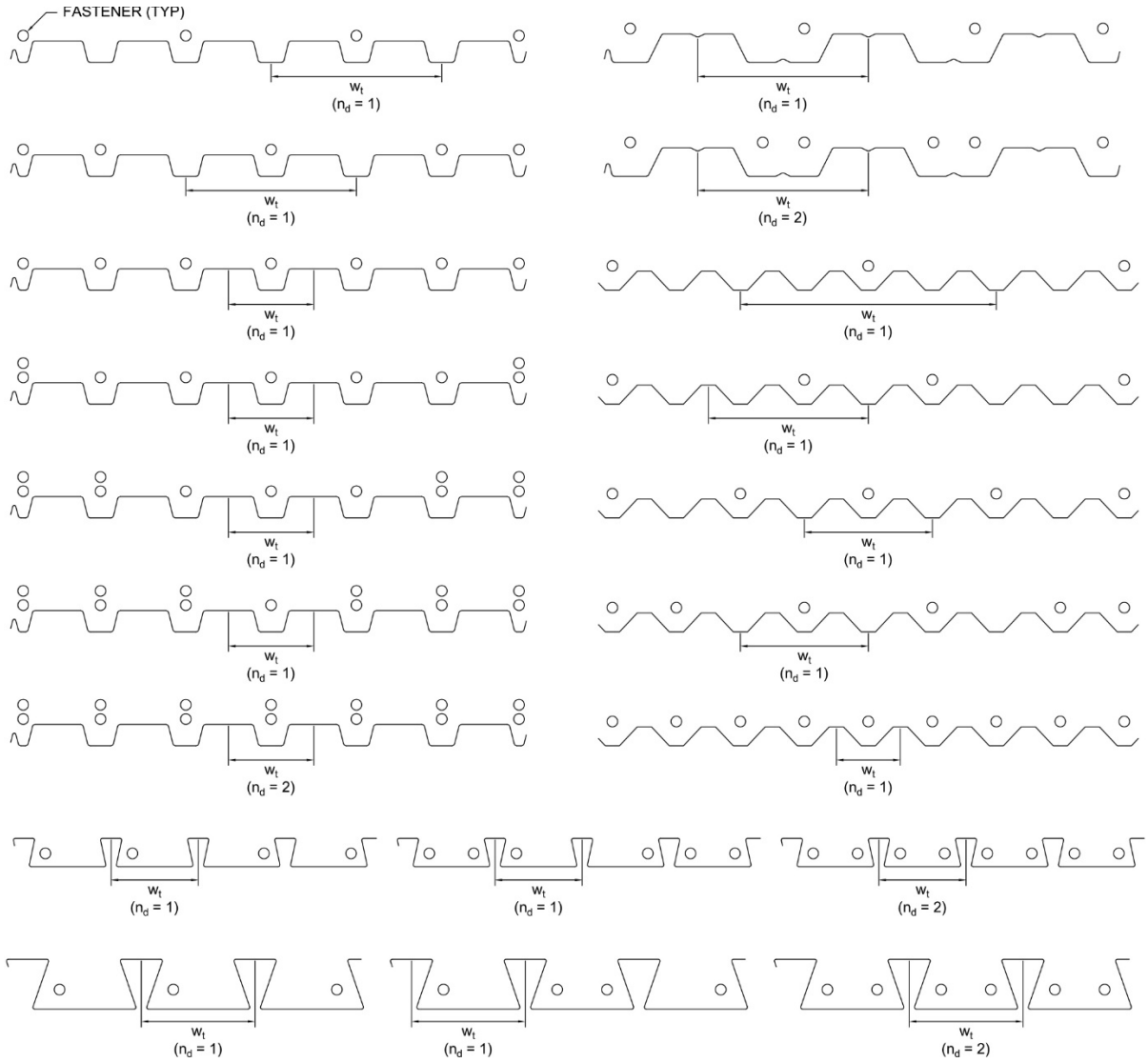

Figure C-D1-1 Tributary Width and Number of Fasteners for Various Attachment Patterns

Standard Figure D1-1 assumes that a common *support connection* occurs at the *side-lap* and that the *panel* ends are lapped. In *Standard* Figure D1-1, N = the *connections*/unit width. For a cover width, w, = 3 ft (0.914 m), A= 2 and N = 5/3 (1/ft) (5.47 (1/m)) at the top *exterior support*, while A = 1 and N = 4/3 (1/ft) (4.38 (1/m)) at the bottom *exterior support*. If a fastener is installed at either side of the side-lap (See Figure C-D1-2), then with everything else being equal, N at the top is 7/3 (1/ft) (7.64 (1/m)) and N at the bottom is 5/3 (1/ft) (5.47 (1/m)). In either case, when determining α_e and α_e^2, there are 7 values of x_e at the top and 5 values of x_e at the bottom.

In the *Standard* Figure D1-1, the number of *side-lap connections*, n_s, is 6 and is distributed over the entire *panel* length, L. Note that n_s is not the number of *side-lap connections* per span, L_v. The number of *side-lap connections*, n_e, at the edge (*lateral force-resisting system* line in the schematic) is

9. The schematic in *Standard* Figure D1-1 conforms to common practice, $n_e \geq n_s$.

The terms, *exterior support* and *interior support*, are relative to each *panel* and not descriptive of the location within the structure.

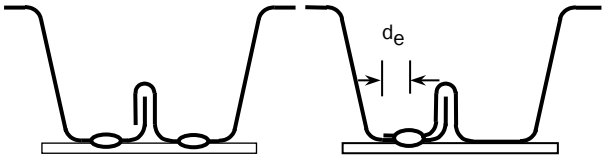

Figure C-D1-2 Interlocking Side-Laps

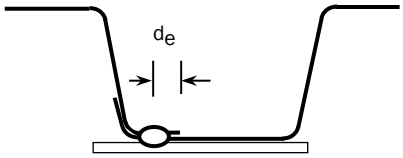

Figure C-D1-3 Nestable Side-Lap

A *support connection* should be installed at either side of the *side-lap* when the *panel* design does not allow a single *support connection* to engage both sections of *panel* at the *side-lap* while developing full *nominal strength* [*resistance*]. Figure C-D1-2 shows examples of *interlocking side-lap connection* over an *interior support*; one option requires two *support connections* while the other may allow one. The critical edge dimension, d_e, is shown for the second option. Figure C-D1-3 shows that d_e is also critical at nestable *side-laps*. If the horizontal lip is too short, an arc spot weld might not be acceptable but an erector might use an equivalent *strength* fillet weld or a mechanical fastener to engage the support. The designer should determine equivalence.

It is possible to have multiple *connections* in each flute over any support and that effect is included in α_1 and α_2 in Eq. D1-3, and α_e^2 and α_p^2 in Eq. D1-5, respectively. The term "A" in *Standard* Eq. D1-1 accounts for the reduced corner *connection resistance* as limited by the compressive *stiffness* of the *panel* at the *side-lap* over the support. With multiple fasteners per flute, proper spacing and edge dimension must be maintained and the group effect might become a *limit state* at the fastener cluster. See *Standard* Section D1.1.6.

Standard Equations D1-1 through D1-11 were developed by Luttrell and first published in The SDI *Diaphragm Design Manual*, First Edition (SDI, 1981). They are also listed in SDI DDM03 (SDI, 2004). The *Standard* equations are based on fluted *panels* with the *configuration* illustrated in *Standard* Figure D2.1-1 and parameters shown in *Standard* Figure D1-1 and defined in *Standard* Section D1. The basic *Standard* equations and the mechanical model can be modified to be applicable to *diaphragms* with concrete fill over *deck* or with insulation between the *panel* and the support. The *Standard* applies such modifications in Sections D1.3 and D4. The modifications consider the potential for corner *buckling* and end warping, and the relative *flexibilities* of *support connections*.

D2 Diaphragm Shear Strength per Unit Length Controlled by Stability, S_{nb}

Standard Section D2 determines the *diaphragm nominal shear strength* [*resistance*] that is controlled by *shear buckling* (*out-of-plane panel buckling* and not *local buckling* at fasteners). This *shear buckling* might manifest as several relatively large diagonal waves across several *panels* or as general (column-like) *buckling* between supports. When several diagonal waves occur, post-*buckling strength* can be present until *connection* failure occurs, which further controls the *diaphragm resistance*. The *load* on these *connections* is redistributed by tension field action and may not follow the model of *Standard* Section D1. *Buckling* initially is a material limit, so the *Standard* Table B1.1 *factors* in *Standard* Section B1 vary from the *connection*-related limits.

Standard Eq. D2-1 is a theoretical limit that includes the orthotropic nature of the

diaphragm fluted *panel* and represents the same theory used to design corrugated *webs* in girders. This theory was presented in SDI DDM03 (2004) and was initially evaluated by Easley (1975). The Easley research contained *confirmatory tests* limited to single spans. For practical cases, Easley and McFarland showed that the elastic *buckling load* for thin corrugated metal *diaphragms* is predicted using Eq. C-D2-1. Since the strong axis *flexural stiffness* is more commonly based on I_x (I_{xg} in *the Standard*), the axes presented in Eq. C-D2-1 are shifted for convenience relative to that presented in the Easley paper.

$$S_{nb} = \frac{36\beta_E D_y^{1/4} D_x^{3/4}}{L_v^2} \qquad \text{(C-D2-1)}$$

where

β_E = *Buckling* coefficient allowance for end restraint and determined by tests
= 1.07

D_x = Strong axis *flexural stiffness* per unit width, k-in. (kN-mm)

$$= \frac{EI_x'}{d} = EI_x \qquad \text{(C-D2-2)}$$

where

E = Modulus of elasticity of steel, 29500 ksi (203000 MPa)

I_x' = Moment of inertia of one corrugation, in.^{4}/*pitch* (mm^4/*pitch*)

I_x = Moment of inertia per unit width, in.^{4}/in. (mm^4/mm)

d = *Panel* corrugation *pitch*, in./*pitch* (mm/*pitch*). See *Standard* Figure D2-1

$$I_x = \frac{I_x'}{d}$$

D_y = Weak axis *flexural stiffness* per unit length, k-in. (kN-mm)

$$= \left(\frac{Et^3}{12}\right)\frac{d}{s} \qquad \text{(C-D2-3)}$$

where

s = Developed flute width per *pitch* determined in accordance with *Standard* Eq. D2-5 in./*pitch* (mm/*pitch*)

t = Base steel *thickness of panel*, in. (mm)

L_v = Span of *panel* between supports with fasteners, ft (m)

$$S_{no} = \frac{36\beta_E E}{L_v^2}\sqrt[4]{\frac{t^3 I_x^3}{12}\left(\frac{d}{s}\right)} \qquad \text{(C-D2-4)}$$

Eq. C-D2-4 is dimensionally admissible for any unit system, but dimensional analysis is required to adjust for product and material data as commonly presented. Examples are:

U.S. Customary Units

Where I_{xg} has units, in.^{4}/ft, while other parameters and units are as shown in the definitions (I_{xg} is substituted for I_x to agree with the *Standard*):

$$S_{no} = \frac{36(1.07)(29500)}{L_v^2}\left(\frac{k}{in.^2 ft^2}\right)\sqrt[4]{\frac{t^3 I_{xg}^3}{12}\left(\frac{d}{s}\right)\left(\frac{in.^3 in.^{12} in.}{ft^3 in.}\right)\left(\frac{1ft^3}{1728in.^3}\right)}$$

$$S_{no} = \frac{36(1.07)(29500)}{12L_v^2}\left(\frac{k\,in.^3}{in.^2\,ft^2}\right)\left(\frac{1ft}{12in.}\right)\sqrt[4]{t^3 I_{xg}^3\left(\frac{d}{s}\right)}$$

$$S_{nb} = \frac{7890}{L_v^2}\sqrt[4]{t^3 I_{xg}^3\left(\frac{d}{s}\right)}\frac{k}{ft}$$

SI Units

Where I_{xg} is substituted for I_x, while other parameters and units are as shown in the Definitions:

$$S_{no} = \frac{36(1.07)(203000)1000}{L_v^2}\left(\frac{kN}{m^2 m^2}\right)\sqrt[4]{\frac{t^3 I_{xg}^3}{12}\left(\frac{d}{s}\right)\left(\frac{mm^3 mm^{12} mm}{mm^3 mm}\right)}$$

$$S_{no} = \frac{36(1.07)(203000000)}{1.861L_v^2}\left(\frac{kN\,mm^3}{m^2 m^2}\right)\left(\frac{1m^3}{10^9\,mm^3}\right)\sqrt[4]{t^3 I_x^3\left(\frac{d}{s}\right)}$$

$$S_{no} = \frac{4.20}{L_v^2}\sqrt[4]{t^3 I_{xg}^3\left(\frac{d}{s}\right)}\frac{kN}{m}$$

Nunna (2011) compared existing *diaphragm* test data with the equations in existing analytical models. The 28 tests exhibited *panel buckling* and included five multiple-span tests plus one hybrid test mixing multiple- and single-spans. The equation in the *Standard* represents a best fit between theory and tests. The *buckling* coefficient increased relative to the previous SDI DDM03 (2004) value. The same *buckling strength* is attributed to single and multiple-span applications. The Nunna report indicates that the *resistance factors* are reasonable when determined in accordance with *Standard* Table B-1. The evaluation results were rationally extended to the entire acceptable range of *Standard* Section D1.1.

The *gross section* moment of inertia should be used in the stability analysis, and this might be more representative of the available *buckling stiffness* of the *panel* during tests. However, section properties are typically published at a *stress* level consistent with *service loads* and these values are commonly used in *diaphragm* design to determine *diaphragm* strength and develop *load* tables.

The *nominal shear strength* [*resistance*] of *diaphragms* formed by fluted *panels* is based on the typical fluted *panel* section illustrated in *Standard* Figure D2-1. Perforations can affect the moment of inertia and the d/s ratio. Luttrell (SDI, 2011) provided an analytical method to determine this effect. Contact the *deck* or *panel* manufacturer for these parameters.

Standard Eq. D2-2 determines the *diaphragm nominal shear strength* [*resistance*] that is controlled by *exterior support local web buckling*. This theory was presented in a report by Nunna (2018b). *Exterior support local web buckling* occurs at the *exterior support* of deck panels and may occur at both lapped and butted joints.

Significant end warping is observed prior to *exterior support local web buckling*. As end warping behavior becomes more extreme, forces are transferred through the panel webs in the form of tension and compression until the web in compression eventually fails. This failure mode is very similar to the failure mode observed in web-crippling tests of steel deck panels. A modified *web crippling* equation which was based on, AISI S100, Equation G5-1, is used to calculate the web-crippling strength of the *panel web* in the test analysis. Slenderness of the web as well as bearing length influence the *diaphragm nominal shear strength* [*resistance*]

that is controlled by *exterior support local web buckling*. *Standard* Eq. D2-4 addresses *panels* with perforated webs and is based on methods developed by Luttrell, SDI (2011).

Testing is always allowed to verify *buckling* capacity.

References

Nunna, R. V. (2018a), "Report No. 18-02, Exterior Support Fastener Limitation for Profiled Steel Diaphragms," S. Barnes Associates, 2018.

Nunna, R. V. (2018b), "Report No. 18-01, Exterior Support Local Web Buckling for Profiled Steel Diaphragms," S. Barnes Associates, 2018.