Application of Diaphragm Stiffness and Strength Equations to Bridge Metal Deck Forms

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Abstract
The building and bridge industries commonly use light gage metal sheeting for concrete formwork. The building industry has long relied on the in-plane strength and stiffness of metal forms to prevent lateral torsional buckling of the beams. The flexibility in the current connection details between the forms and girders in the bridge industry often limit the amount of bracing provided by these forms. In a recent research study, modified connection details were developed that substantially improve the bracing behavior of bridge metal deck forms. However, there is currently not a design aid that a bridge engineer can utilize to predict the stiffness and strength of bridge metal deck forms. Hence, specifying the right PMDF system for bracing purposes becomes troublesome. For applications in the building industry, the Steel Deck Institute (SDI) Diaphragm Design Manual provides numerical expressions to determine the stiffness and strength of various types of metal sheeting and their corresponding connection details. The expressions in the SDI Design Manual are not directly applicable to bridge metal deck forms since PMDF used in the bridge industry differ from those utilized in the building industry by both shape and connection detail. In this paper results from shear diaphragm tests on bridge metal deck forms with modified connection details are compared with SDI expressions. Results indicate that slightly modified SDI expressions can be used to predict the stiffness and strength of bridge metal deck forms with modified connection details.

1. Introduction
The building and bridge industries commonly use light gage metal sheeting for concrete formwork. Besides supporting the wet concrete, metal sheeting also provides bracing to the beams that they are attached to during construction. The building industry has long relied on the in-plane stiffness and strength of the metal forms to prevent lateral torsional buckling of beams. In traditional steel building construction metal forms are typically continuous over the tops of the beams. In such applications metal sheeting acts like a shear diaphragm and restrains the lateral movement of the top flange of the beams; providing continuous bracing to the beams that they are attached to during pouring of concrete. An adequate bracing system for stability must possess both stiffness and strength (Winter 1960). The design of shear diaphragm bracing systems has

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been studied in this manner (Helwig and Frank 1999; Errera and Apparao 1976; Nethercot and Trahair 1975) and stiffness and strength requirements have been developed for shear diaphragms (Helwig and Yura 2008a and 2008b). Based on these requirements for shear diaphragm bracing systems, designers in the building industry can utilize design aids such as the Steel Deck Institute (SDI) Diaphragm Design Manual (Lutrell 2004) in selecting the forming system with adequate stiffness and strength.

Similar to the building industry, the bridge industry also utilizes metal forms to support wet concrete during construction. However, although metal deck forms are often relied upon for lateral bracing in the building industry, the AASHTO LRFD Specifications (AASHTO 2012) do not currently permit metal deck forms to be considered for bracing in steel bridge girders. The forms used in the bridge industry differ from those used in the building industry by both shape and method of connection. The primary difference that affects the bracing behavior is the method of connection between the forms and the girders. In the building industry, the forms are continuous over the tops of the girders and are fastened directly to the girder flanges by welding shear studs directly through the forms or by using puddle welds or mechanical fasteners. The bridge industry typically supports the forms on angles that allow the contactor to adjust the form elevation to account for changes in flange thickness as well as differential camber between adjacent girders (Fig. 1).
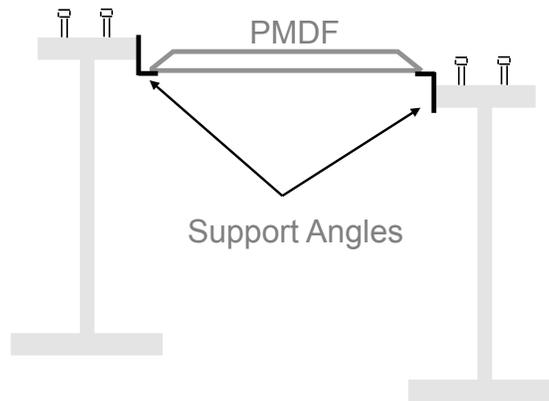

Figure 1: Conventional method of connection of PMDFs in the bridge industry

Although the support angles are beneficial for constructability issues, the conventional eccentric connections drastically reduce the in plane stiffness and bracing effectiveness of the forming system. The reason for this large stiffness reduction is because bracing systems are often governed by the equation for springs in series as illustrated in the following expression:

$$\frac{1}{\beta_{sys}} = \frac{1}{\beta_{deck}} + \frac{1}{\beta_{con}} \tag{1}$$

where β_{sys} is the stiffness of the deck and connection system, β_{deck} is the stiffness of the metal deck form, and β_{con} is the stiffness of the connection. The system stiffness in Eq. (1) is less than the smaller of either the deck or connection stiffness. Therefore, even though the deck may be very stiff, the flexible connection often leads to a low stiffness in the system. The metal forms

used in the bridge industry are often referred as permanent metal deck forms (PMDF) or Stay-in-Place (SIP) metal forms. The term PMDF will be used throughout this paper.

The authors have conducted past research to investigate the bracing behavior of PMDF systems commonly used in the bridge industry. A primary aspect of the research was to improve the connection details of PMDF systems to fully utilize the bracing potential of the formwork for bridge applications. The investigation consisted of experimental and computational studies. The experimental program consisted of tests on PMDF systems in a shear frame and also lateral displacement and buckling tests on a 15 m (50 ft) twin-girder system with PMDF for bracing. The laboratory tests have shown that the bracing behavior of PMDF in the bridge industry can be substantially enhanced by providing a modified connection detail (Egilmez et al. 2007 and 2012). The proposed modified connection detail consisted of transverse "L3×3 10ga stiffening angles" that span between the top flanges of adjacent girders, which are located at intermittent locations along the girder length. Each stiffening angle is positioned at a sidelap location between adjacent PMDF sheets so that the forms can be screwed directly to the angle (Fig. 2).

On the other hand, the computational studies of the research study focused on developing a simple design procedure for calculating stiffness and strength requirements of PMDF used for lateral bracing of steel bridge girders and a design methodology was proposed by Egilmez et al. (Egilmez et al. 2005). In 2006, results from the research study were used in the design and construction of two steel bridges in Houston, Texas, which utilized PMDF as bracing elements to stabilize 50 ft long simply supported W18×119 girders.
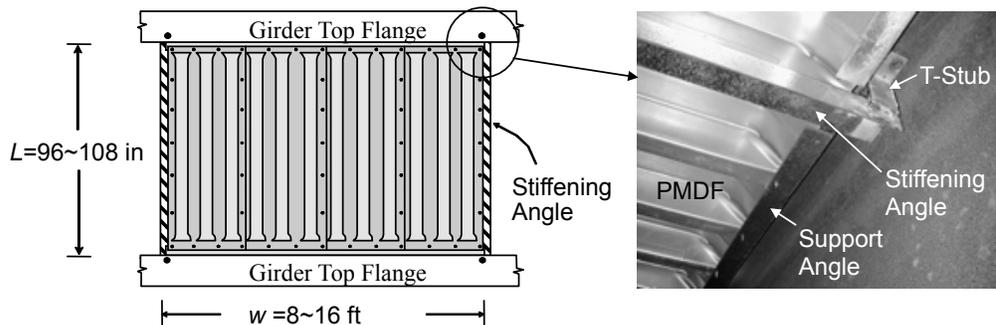

Figure 2: Modified stiffening angle connection

The proposed design procedure enables the bridge engineers to determine the required stiffness and strength of PMDF that needs to be utilized as the bracing source during the construction phase of a steel bridge. However, currently there is not a design aid that a bridge engineer can utilize to predict the stiffness and strength of bridge metal deck forms. Hence, specifying the right PMDF system for bracing purposes becomes troublesome. For applications in the building industry the SDI Diaphragm Design Manual (Luttrell, 2004) provides numerical expressions to determine the stiffness and strength of various types of metal sheeting and their corresponding connection details. However, the expressions in the SDI Design Manual are not directly applicable to bridge metal deck forms since PMDF used in the bridge industry differ from those utilized in the building industry by both shape and connection detail. In this paper results from the shear diaphragm tests are compared with SDI expressions in order to ascertain whether SDI expressions can be used to predict the stiffness and strength of bridge metal deck forms with modified connection details.

The paper will begin by presenting background information, followed by a description of the shear-panel test frame and an explanation of the testing procedure. Results from the experiments as well as comparisons of test results with values obtained from SDI Diaphragm Design Manual (Luttrell, 2004) equations will then be presented followed by a summary of the findings. An application of the SDI stiffness and strength equations is presented in Appendix I.

2. Background and Previous Work

Metal deck forms possess a considerable amount of in-plane shear stiffness and strength and are generally modeled as shear diaphragms that resist shear deformations. The elastic properties of the forms can be measured experimentally by utilizing a cantilever test frame as shown in Fig. 3. The diaphragm shown in Fig. 3 is fastened to the test frame on only two sides, which is similar to the connection detail of conventional bridge applications.
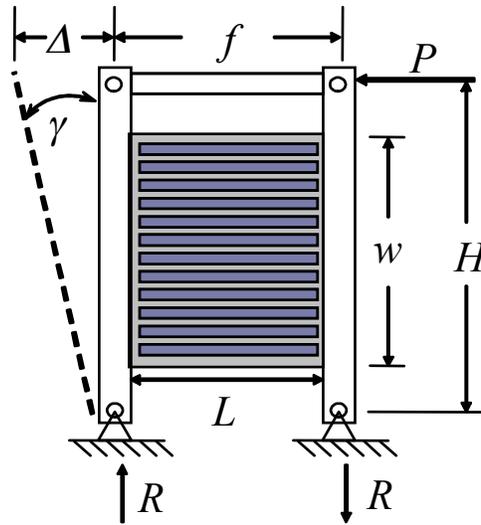

Figure 3: Cantilever shear test frame

Ideally, a shear diaphragm should be fastened on all four sides in order to be subjected to pure shear deformations. The modified connection detail used in this study enables the deck panels to be supported on four sides by means of the transverse stiffening angles. Hence, the effective shear modulus of PMDF's can be calculated as follows (Fig. 3):

$$R = \frac{PH}{f} \tag{2}$$

$$\tau' = \frac{PH}{fw} \tag{3}$$

$$\gamma = \frac{\Delta}{L} \tag{4}$$

$$G' = \frac{\tau'}{\gamma} = \frac{PH^2}{fw\Delta} \tag{5}$$

where R is the reaction force (kip), P is the shear load applied to the diaphragm (kip), L is the length of the test frame (in), f is the length of the diaphragm (in), w is the width of the diaphragm (in), τ' is the effective shear stress of the corrugated sheet (kip/in), γ is the shear strain, Δ is the shear deflection of the diaphragm (in), and G' is the effective shear modulus of the diaphragm (kip/in-rad). Since PMDF is not a thick plate, the shear stress versus strain relationship of the corrugated sheeting is not a linear function of the material thickness (Luttrell, 1981). Hence, "an effective shear modulus", G', is generally utilized which is not a function of the material thickness.

When the deck system shown in Fig. 3 reaches the ultimate capacity, the applied load becomes its maximum sustained value, R_{ult}. For this study, the ultimate effective shear capacity of the diaphragm is computed as:

$$S'_{ult} = \frac{R_{ult}L}{fw} \tag{6}$$

For applications in the building industry, in lieu of laboratory testing the effective shear modulus and strength of a diaphragm can be determined using published equations and design tables. There have been many investigations on the shear behavior of corrugated steel sheeting used in the building industry. Luttrell directed an extensive work on shear diaphragms at West Virginia University during the 1970's (Luttrell, 1981). Formulas for both diaphragm strength and stiffness predictions resulted from these studies and were published by the Steel Deck Institute (SDI) as a design manual (Luttrell, 1981). Luttrell used a stiffness based approach in his studies and showed that the shear stiffness was primarily dependent on the warping deformation of the corrugation at panel ends, shear strain in the forms, and slip at the fastener locations; all of which allow larger shear deformations to occur in the overall system. Experiments by Luttrell also showed that the shear strength of a diaphragm was generally governed by either shear strength involving the longitudinal edge of a panel over the line where force transfer is made to the structural system at joist ends, shear strength involving the force transfer at interior panel to panel connections, or shear strength involving fasteners across the ends of panels (Luttrell, 1981).

The first investigation on the bracing capabilities of PMDF used in bridge applications was conducted at University of Texas in the early 1990's by Currah (1993) and Soderberg (1994). The primary objective of Currah's study was to determine the shear stiffness of PMDF singularly, without any detraction from the deck support system. For this reason a rigid connection was used, unlike the conventional eccentric connection detail utilized in the bridge industry. Currah found reasonable agreement between experimentally measured values of shear stiffness and strength for deck forms that are used in the bridge industry and calculated values from modified SDI expressions. Soderberg investigated the decking system's connection stiffness and determined ways of improving it by welding small stiffeners to girder flanges and support angles (Soderberg, 1994).

3. Overview of Shear Diaphragm Tests and Connection Details
A test frame similar to the frame shown in Fig. 3 was constructed for the diaphragm tests (Fig. 4). The test frame was designed so that a wide range of deck systems with a variety of panel

widths and deck spans could be tested. The frame was racked by a hydraulic actuator, which applied a shear load at the end of the frame. Loads and displacements were monitored throughout the testing so that the shear stiffness and strength of the panel could be measured. A detailed description of the test frame can be found in Egilmez et al. (2007).
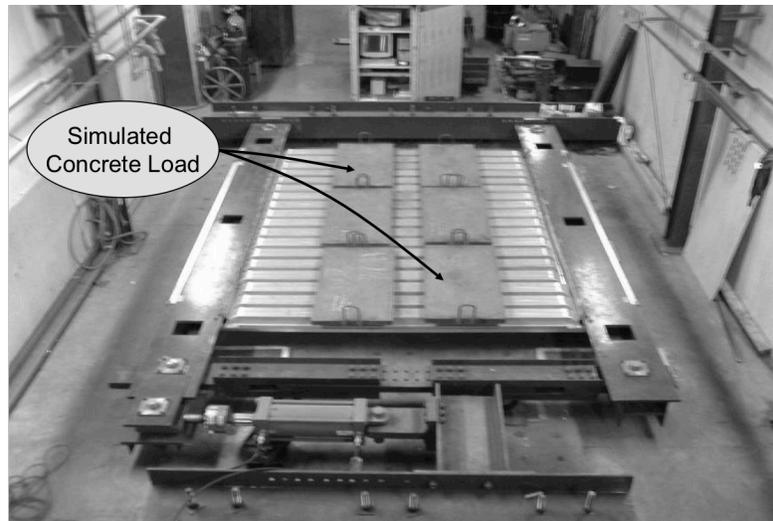

Figure 4: Test frame used for shear diaphragm tests

Once each deck panel was assembled into the frame, three separate phases of loading were conducted: 1) elastic cyclic loading without dead load, 2) elastic cyclic loading with dead load, and 3) ultimate capacity test in one direction with dead load. The first two phases of testing were performed in the elastic range of the material up to shear stresses not exceeding 0.01 k/in. The purpose of these initial elastic tests was to determine the elastic shear stiffness of the PMDF system with and without superimposed dead load. The superimposed dead load, provided by steel plates (Fig. 4), was applied to measure the effects of friction between the contact surfaces of the PMDF system that would normally be caused by fresh concrete. The weight of the steel plates was large enough to indicate the effect of internal friction between the form sheets on the shear stiffness. The third phase of testing was the ultimate load test, which was conducted to get a measure of the shear stiffness at higher load levels while also measuring the ultimate shear strength of the PMDF systems.

The deck panel specimens tested during this study were pre-closed (tapered closure), 3x8 (3 in depth, 8 in pitch) bridge deck forms with a 24 in cover width and metal thicknesses of 22, 20, 18, and 16 ga. PMDF spans of 96 in and 108 in were considered. These spans are representative of the PMDF spans frequently employed in steel girder construction. Fig. 5 shows the dimensions and profile configuration of the PMDF specimens tested in this investigation.

Deck form sheets spanned between the loading beams of the test frame, and were supported on cold-formed, L3×2 10 ga galvanized angles, which are typical of those employed in bridge constructions. Similar to conventional bridge applications the support angles were welded directly to the top plate of the shear test frame loading beams (which represented the top flange of actual steel bridge I-girders) with a variety of support details for the cold-formed support angles. Initial tests were conducted using conventional connection details with and without

eccentricities. For tests without eccentricity, the long leg of the support angle was oriented vertically down and the short leg was placed flush with the level of the top flange plate. For tests with maximum eccentricity, the long leg of the support angle was oriented vertically upward with its edge extending ⅛ in above the top flange plate of the support beam as shown in Fig. 6. Preceding tests were conducted with PMDF panels that utilized the proposed modified connection, which involved a transverse stiffening angle (L3×3 10 ga) that spans between adjacent girder flanges as shown in Fig. 2. The stiffening angles were positioned to coincide with a side-lap seam so the deck could be screwed directly to the angle with several fasteners. The spacing between the stiffening angles was varied between 8 to 16 ft. In order to have the same eccentricity of the stiffening angle and the support angle, the ends of the stiffening angle were welded to the webs of fabricated T-stubs (2.5 in long, L3×52 10 ga, long leg back-to-back) that were bolted to the underside of the top flange plate as shown in Fig. 2. In the preceding phases of the study it was recommended that the conventional L3×2 10 ga support angles be replaced by L3×3 10 ga angles, and that the L3×3 10 ga size also be used for the stiffening angles (Egilmez et al. 2012). A more practical connection between the stiffening angles and girders were also recommended in the preceding phases of the study (Egilmez 2005). In the recommended connection detail, stiffening angles were welded to gusset plates that were welded to the web and bottom of top flange of girders.

h = depth of corrugation = 3 in
e = bottom flange of corrugation = 1 in
f = top flange of corrugation = 5.5 in
d = pitch of corrugation = 8 in
w = web of corrugation = 3 in
g = projection of web of corrugation = ¼ in
s = developed width of corrugation
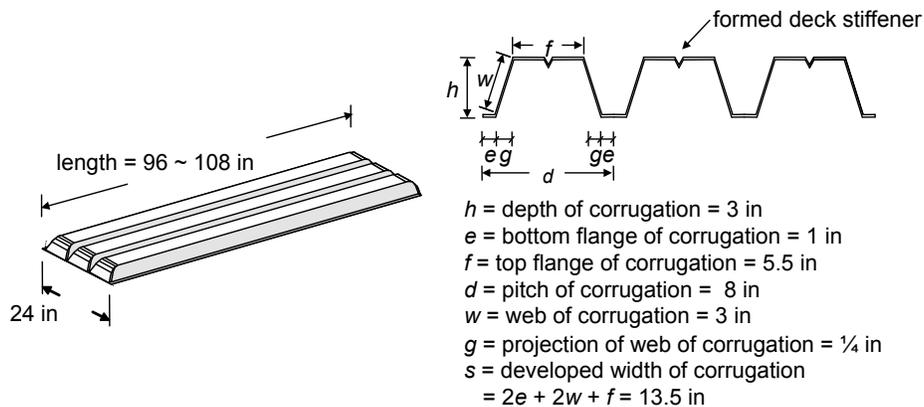   = $2e + 2w + f$ = 13.5 in
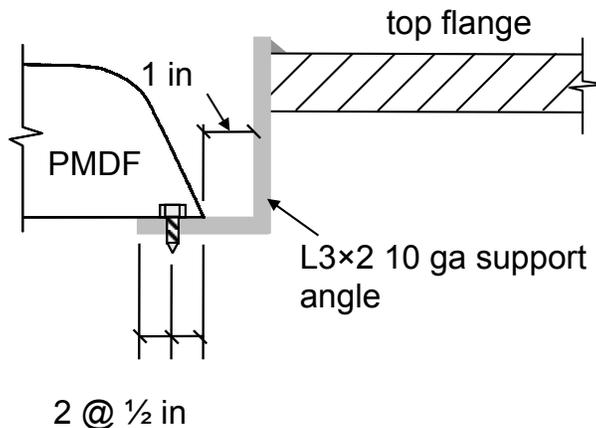
Figure 5: Profile and dimensions of PMDF specimens

Figure 6: Conventional PMDF connection utilized in shear diaphragm tests

The parameters that were investigated for this study included panel width and length, metal deck gage, stiffener spacing, and support angle configuration. Four different support angle configurations were tested: a) no eccentricity-unstiffened, b) no eccentricity-stiffened, c) maximum eccentricity-unstiffened, and d) maximum eccentricity-stiffened. These three support angle configurations are illustrated in Fig. 7. The symbol "X" indicates a stiffened connection with the approximate eccentricity indicated by the vertical position of the "X". The effects of stiffening angles can be observed in Fig. 8, which shows a typical shear stress vs. shear strain behavior of a deck system tested with and without stiffening angles. This particular deck system had 18 ga metal deck forms. The graph shows that utilizing stiffening angles tremendously increases the stiffness of the system. Fig.9 shows the failure modes of systems without and with stiffening angles. The unstiffened conventional deck system failed due to excessive support angle rotations, whereas the stiffened system failed due to local buckling of the stiffening angle. Although there was not much strength increase by the addition of stiffening angles, the stiffness increase was considerable as can be observed in Fig. 8. Complete results of the shear diaphragm tests are presented by Egilmez et al. (2007).
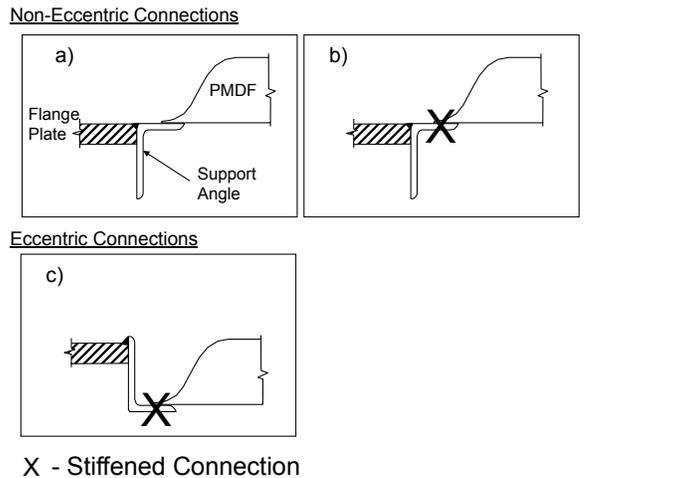

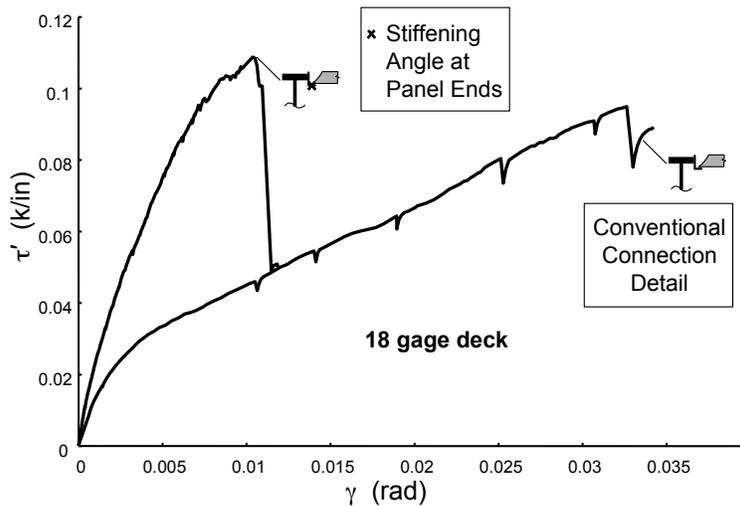Figure 7: Eccentric and non-eccentric connection configurations

Figure 8: Effect of stiffened connections on PMDF stress vs. strain behavior
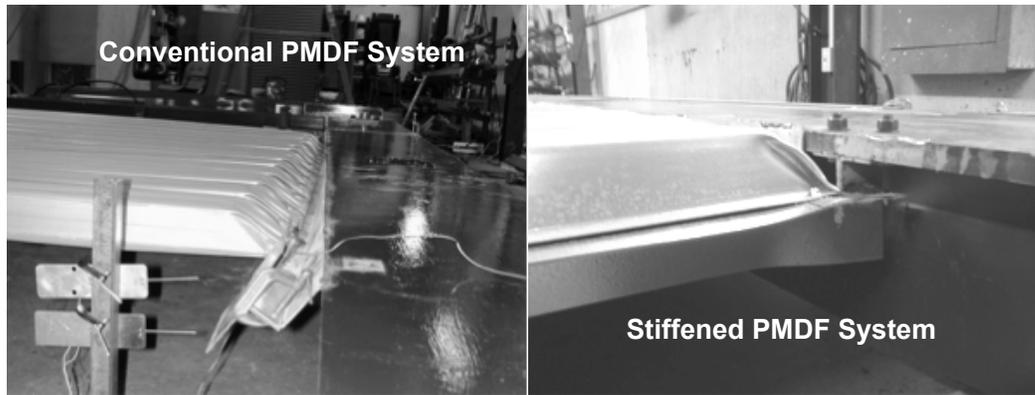
Figure 9: Failure modes of conventional and modified connection details

4. Comparison of Experimental and SDI Design Manual Shear Stiffness and Strength Values

As previously mentioned, Steel Deck Institute (SDI) has published a design manual which enables the designer to estimate the shear strength and stiffness of a particular shear diaphragm (Luttrell, 2004). The SDI Design Manual is primarily for metal decks used in the building industry where the decks are open ended and fastened directly to the structural members on four sides. The purpose of this section is to determine whether the SDI Design Manual equations can be used to provide an adequate estimation for shear stiffness and strength capacities of bridge metal deck form systems with stiffening angles. Comparison of test results (from deck specimens without superimposed dead load) with values obtained from SDI equations will be presented. Test specimens with conventional eccentric connections were excluded from these comparisons, since the stiffness of such systems is governed by the stiffness of the eccentric connection, which is not accounted for in the SDI equations.

The SDI Design Manual shear stiffness equation is not directly applicable to the deck forms used in the bridge industry for two reasons. The first reason is that the stiffness equation of the SDI Design Manual is specifically for open end deck forms. Deck forms used in the bridge industry have tapered closed ends with the sheeting folded at the edges, which increase the warping resistance and hence the shear stiffness of the diaphragm. The second reason arises due to the assumption made in the SDI stiffness equation that sidelaps and two edges of the diaphragm parallel to the corrugations remain straight as shear displacements occur. This assumption is accurate for diaphragms with short lengths and which are fastened to stiff structural members on four sides. In the bridge industry, even though the improved connection detail that utilizes stiffening angles enables the deck panels to be supported on four sides, the L3x2 10 ga stiffening angles are not stiff enough to prevent the corrugations to curve in the longitudinal direction as the diaphragm distorts. Therefore, the assumption that corrugations remain straight becomes questionable for deck forms used in the bridge industry. Hence, due to these two reasons there are concerns related to using SDI Design Manual shear stiffness equation to predict the shear stiffness of deck form systems used in the bridge industry. On the other hand, the SDI Design Manual strength equations were expected to predict the strength of deck forms used in the bridge industry accurately, due to the fact that the failure modes of shear diaphragms in both industries

are identical. The appropriate SDI equations along with stiffness and strength calculation examples are presented in Appendices I and II, respectively.

5. SDI Design Manual Stiffness vs. Experimental Stiffness Results
This section provides a comparison of experimentally measured effective shear stiffness values for 18 ga and 16 ga deck form systems to stiffness values computed using SDI Design Manual (Luttrell, 2004) Equation 3.2-3 for diaphragm stiffness (presented in Appendix I). In the application of the SDI Manual equation to PMDF used in the bridge industry the following assumptions were made: 1) The flexibilities of the ¼ in diameter No.14 TEKS screws used in this study were calculated using SDI Design Manual Equations 4.5.1-1 and 4.5.1-2 for fastener flexibilities. These fastener flexibility equations presented in the SDI Design Manual are based on single lap joint shear tests for No.12 and No.14 Buildex TEKS screws used in the test programs conducted by Luttrell (Luttrell, 1981). It was assumed that the ¼ in TEKS screws used in this study would have the same flexibility as the flexibility of the screws used in Luttrell's studies; 2) Warping constant, D, was calculated according to the equations presented in Appendix IV of SDI Design Manual. However, radius corners and formed decked stiffeners in the deck profile as seen in Fig. 6 were neglected.

Table 1, which is divided into nine columns labeled (a) through (i), provides values for the effective shear stiffness from both laboratory and SDI Design Manual Equation 3.2-3 for diaphragm stiffness. Column (a) shows the PMDF thickness; whereas column (b) shows the test number and a pictorial illustration of the connection detail that was used. Columns (c) through (e) present the panel geometry, which consists of the span of the deck, the width of the shear panel, and the spacing between the stiffening angles. Columns (f) through (h) list the computed SDI stiffness for closed end decks, measured effective shear stiffness from each test (without superimposed dead load), and SDI stiffness for open ended decks. The ratio of measured stiffness to SDI stiffness value for open ended decks is presented in column (i). Warping constant, D, calculations of the SDI Manual are for open ended decks. In order to make a better comparison between the test results and SDI equations, two SDI stiffness values are presented. In addition to the stiffness of open ended decks given by Eq. 3.2-3, a stiffness for closed end (theoretically warping restrained) decks computed by removing the warping term, D_n, from Eq.3.2-3 of the SDI Manual, is also presented. A detailed example showing how the SDI stiffness values presented in columns (f) and (h) are calculated is presented in Appendix I for Test #9.

Results from eccentric stiffened connection details are presented. The eccentric stiffened connection detail used is the modified connection detail recommended in the previous studies (Egilmez et al. 2007, and 2012). The comparison of shear stiffness of deck form systems with modified connection details with SDI formulations gives an indication on whether the SDI Design Manual (Lutrell, 2004) stiffness equations are applicable or not for such systems.

Most of the forming systems used in bridge deck systems have span lengths in the range of 7 ft to 9 ft. The range of deck spans tested experimentally in this study consisted of either 8 ft or 9 ft decks. Egilmez et al. (2007) showed that the deck span had a small effect on the performance of the deck forms, where the strength and stiffness of the deck panel decreased by approximately

10%. The aspect ratios (deck span to panel width) of the tests shown in Table 1 ranged between 0.56 and 1.0.

Test number 1 belongs to 18 ga deck system with an aspect ratio of 1.0. The ratio of measured stiffness values to stiffness estimates computed by the SDI Manual Equation 3.3-3 (for open ended deck forms) for this test is 1.41. Tests 2 through 5 and 6 through 9 are for 18 ga and 16 ga deck form panels with different aspect ratios, respectively. The widths of the panels were 8 ft, 12 ft, 16 ft, and 16ft for Tests 2 through 5 and 6 through 9, respectively. Tests 2 through 4 and 6 through 8 had two stiffening angles at the edges of the panel and Tests 5 and 9 had three stiffening angles, two at the edges and one at the middle. Due to the presence of the intermediate stiffening angle, in Tests 5 and 9 the overall diaphragm panel width was taken as 8 ft instead of 16 ft in SDI Design Manual stiffness calculations. It can be seen from the results presented in Table 1 that changing the aspect ratio does not have a great impact on the computed SDI stiffnesses for neither open nor closed end decks for both 18 ga and 16 ga PMDF systems. A similar behavior can be observed in the measured stiffness values for 18 ga and 16 ga PMDF systems. The only exception was the stiffness of the panel with the highest aspect ratio (Test 2) for 18 ga PMDF systems. The measured stiffness of Test 2 was about 25% smaller than those of Tests 3 to 5. The ratio of measured stiffness values to stiffness estimates computed by the SDI Manual Equation 3.3-3 (for open ended deck forms) were 1.02, 1.32, 1.36, and 1.39 for Tests 2 to 5, respectively. The stiffness estimates computed by the SDI Manual Equation 3.3-3 (for closed end deck forms) were about 2.5 times higher than the measured stiffnesses of Tests 3 to 5. On the other hand for 16 ga PMDF systems the ratio of measured stiffness values to stiffness estimates computed by the SDI Manual Equation 3.3-3 (for open ended deck forms) were 0.79, 0.91, 0.84, and 0.94 for Tests 6 to 9, respectively.

Table 1: Effective shear stiffness values from laboratory tests and SDI Eq. 3.2-3

Connection Type	a) Metal Thickness	b) Test # and Detail	c) Span, L (ft)	d) Width, w (ft)	e) Stiffener Spacing, s_s (ft)	f) SDI Stiffness Closed End G'_{closed} (Eq. 3.2-3) (kip/in-rad)	g) Experimental Stiffness G'_{exp} (kip/in-rad)	h) SDI Stiffness Open End G'_{open} (Eq. 3.2-3) (kip/in-rad)	i) G'_{exp}/G'_{open}
Stifened Eccentric	18 ga	1	8	8	8	102.2	35.0	24.8	1.41
	18 ga	2	9	8	8	94.2	27.0	26.5	1.02
	18 ga	3	9	12	12	93.7	35.0	26.5	1.32
	18 ga	4	9	16	16	93.5	36.0	26.5	1.36
	18 ga	5	9	16	8	94.2	37.0	26.5	1.39
	16 ga	6	9	8	8	108.8	32.0	40.6	0.79
	16 ga	7	9	12	12	108.2	37.0	40.5	0.91
	16 ga	8	9	16	16	107.9	34.0	40.4	0.84
	16 ga	9	9	16	8	108.8	38.0	40.6	0.94

6. SDI Design Manual Strength vs. Measured Strength Results
This section summarizes the measured and predicted shear strength of the deck systems from both laboratory and SDI Design Manual (Lutrell, 2004) strength equations. Results are presented for 18 ga and 16 ga deck form systems. In the application of the SDI Manual strength equations the strength of the ¼ in diameter No.14 TEKS screws used in this study were calculated using SDI Design Manual Equations 4.5-1 and 4.5-2 for fastener strength. Similar to stiffness comparisons, it was assumed that the ¼ in TEKS screws used in this study would have the same strength as the screws used in Luttrell's studies. As previously mentioned the shear strength of a diaphragm system is limited by the strength of the connections or by general plate-like buckling of the whole diaphragm area (Lutrell, 2004). Table 2 provides values for the shear strength of the deck systems from both laboratory and SDI Design Manual Equations (Lutrell, 2004). Equations 2.2-2, 2.2-4, and 2.2-5 from the SDI Design Manual were used to estimate the strength of the edge fastener, interior panel fastener, and corner fastener failures, respectively. The shear strength due to general plate-like buckling of the whole diaphragm area is not presented since this failure mode was not observed in any of the laboratory tests. This type of a failure generally controls the behavior of thin diaphragms or in certain 22 ga diaphragms with very closely spaced side lap connections and where larger than usual purlin spacing values are involved (Lutrell, 2004), both of which are not valid in this study. Equation 2.2-5 was modified to be applicable to bridge metal deck forms. The modification and resulting applicable equation are presented in Appendix II for Test #5.

Columns (f) through (j) in Table 2 provide values for the shear strength from both laboratory and SDI Design Manual Equations 2.2-2, 2.2-4, and 2.2-5 for diaphragm strength due to fastener failures. Columns (f), (g), and (h) list the computed SDI strength of the diaphragms due to edge fastener, interior panel fastener, and corner fastener failures, respectively. The estimated strengths for all of the panels in the table are controlled by the strength of the corner fastener, S_{nc} (column h), which is consistent with the observed failures from all of the tests. Therefore to compare the estimated and measured strengths, the ratio of the measured strength (column i) to S_{nc} is given in the last column (column j).

The ratio of the measured to predicted values ranged from 0.79 to 1.15 with an average value of 1.01. Most of the values of the ratio that were less than 1.0 occurred for the heavier gage deck that had a thickness of 16 ga, which is consistent with the shear stiffness results summarized in the last section in which the ratios of measured to predicted values were also less than unity. The effect of stiffener spacing can be observed Table 2. For both 18 ga and 16 ga deck panels the measured strength for panels with stiffening angles only at the edges were similar to each other for widths of 8 ft and 12 ft. When the overall deck panel width was increased to 16 ft with stiffening angles only at the edges, the measured strength decreased by about 15% and the ratio of measured to predicted values of shear strength fall below 1.00 (0.97 and 0.79 for 18 ga and 16 ga deck forms, respectively). Introducing a stiffening angle at mid-span increased the shear strength and brought the ratio to 1.15 and 0.92 for 18 ga and 16 ga deck forms, respectively. Overall, SDI equations provided relatively accurate estimates of the strength of shear diaphragms used in the bridge industry.

Table 1: Shear strength values from laboratory tests and SDI Equations

	a) Metal Thickness	b) Test # and Detail	c) Span, L (ft)	d) Width, w (ft)	e) Stiffener Spacing, s_s (ft)	f) SDI Strength S_{ne} (Eq. 2.2-2) (kip/in)	g) SDI Strength S_{ni} (Eq. 2.2-4) (kip/in)	h) SDI Strength S_{nc} (Eq. 2.2-5) (kip/in)	i) Experimental Strength S_{ult} (kip/in)	j) S_{ult}/S_{nc}
Connection Type: Stiffened Eccentric	18 ga	1	8	8	8	0.198	0.113	0.107	0.116	1.08
	18 ga	2	9	8	8	0.176	0.100	0.097	0.109	1.13
	18 ga	3	9	12	12	0.176	0.100	0.097	0.103	1.06
	18 ga	4	9	16	16	0.176	0.100	0.097	0.093	0.96
	18 ga	5	9	16	8	0.176	0.100	0.097	0.111	1.15
	16 ga	6	9	8	8	0.186	0.118	0.112	0.108	0.97
	16 ga	7	9	12	12	0.186	0.118	0.112	0.115	1.02
	16 ga	8	9	16	16	0.186	0.118	0.112	0.088	0.79
	16 ga	9	9	16	8	0.186	0.118	0.112	0.103	0.92

7. Summary and Conclusions

This paper has focused on methods of predicting the shear stiffness and strength of formwork that is used in the bridge industry. Although the formwork is not currently relied up for bracing in the bridge industry, the authors of this paper have conducted research to improve the bracing potential and the forms. The connection detail for formwork that is used in the bridge industry has an eccentric connection that greatly reduces the bracing potential of the metal forms. The modified connection details incorporated transverse stiffening angles that span between adjacent girder top flanges, where each stiffening angle is positioned at a sidelap location between adjacent PMDF sheets so that the forms can be fastened directly to the angle. The stiffening angle provides a connection for the diaphragm panel on all four sides, which is consistent with many diaphragms that are used in the building industry. However, there is not a design aid that a bridge engineer can utilize to predict the stiffness and strength of bridge metal deck forms in order to specify the right PMDF system for bracing purposes. Comparisons were made in this paper between laboratory measurements of the stiffness and strength of shear diaphragms comprised bridge decking and predictions using equations from the SDI Design Manual (Lutrell, 2004).

Test results for shear diaphragms with stiffened eccentric connections were compared with stiffness and strength values computed using SDI Design Manual (Lutrell, 2004) equations. The stiffening angle spacing that was tested generally consisted of 8 ft and 16 ft, however isolated cases were also conducted with a spacing of 12 ft. In applying the SDI Design Manual equations to bridge metal deck forms the flexibility and strength of sheet to sheet and sheet to support and stiffening angles utilizing ¼ in TEKS screws used in this study were assumed to possess the

same flexibility and strength of the connections tested in Luttrell's studies. In addition, in calculating the warping constant of the formwork, the radius corners and formed decked stiffeners in the deck profile were neglected.

Among the nine different deck form systems, the stiffness of the four 16 ga deck systems was about 10 to 15 % lower than the stiffness predicted with SDI equations. The greatest difference was observed in deck system with stiffening angles spaced 16 ft apart. Comparison of laboratory and SDI strength values in revealed that the SDI Design Manual equation that defines the strength of corner fasteners had good agreement with the measured strength of PMDF systems from the shear tests in this study. The SDI Design Manual strength equations over predicted the strength of the corner fasteners by approximately 26% for the case with stiffening angles spaced 16 ft apart, however this appeared to be an isolated case.

Although the test results documented in this paper showed cases where the SDI equations were unconservative, this fact is likely to be offset by recommendations that were made following the shear tests. The final phase of the testing program consisted of buckling tests on a twin girder system with metal deck forms for the bracing. It was recommended during this final phase that the conventional L3×2 10 ga support and stiffening angles utilized in the shear diaphragm tests be replaced by L3×3 10 ga angles (Egilmez et al. 2012). It was observed that utilizing L3×3 10 ga angles not only increased the strength of the deck to support angle fastener connections, but also improved the effective shear stiffness of the deck system. Further research is needed to verify the positive affect of L3×3 10 ga angles on the stiffness and strength of PMDF used in the bridge industry to ensure that equations proposed by SDI Design Manual are adequate for bridge design.

Appendix I: SDI Diaphragm Stiffness Example
Dimensions and Necessary Parameters (Test #5):
Deck Profile (Fig. 10): 18 ga deck form, stiffener spacing = 8 ft (Two at edges and one at midspan), eccentricity = 3 in (fully fastened).
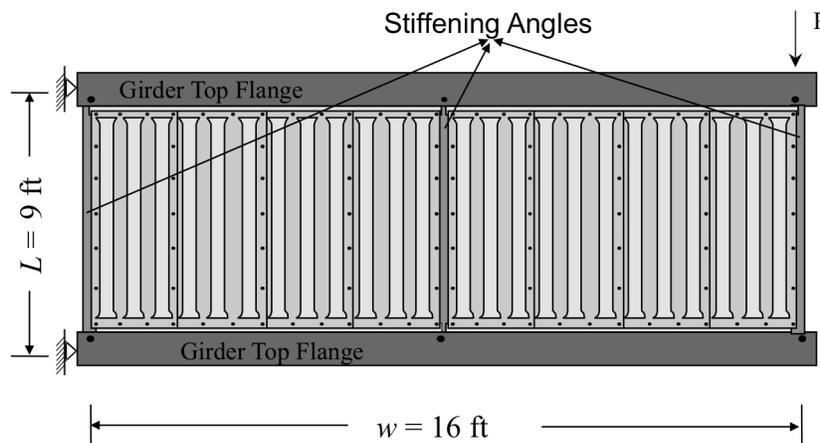L = deck sheet span length = 9 ft, a = overall diaphragm panel width = 8 ft,

Figure 10: Deck panel and fastener configuration (Test #5)

n_{sh} = number of individual deck sheets in panel =4, n_s =number of side lap fasteners per seam =5, w = individual deck sheet width = 24 in, t = thickness of sheet metal = 0.0478 in, s = girth of corrugation per rib = 13.5 in (Fig. 5), d = corrugation pitch = 8 in (Fig. 5), h = deck sheet depth = 3 in, n_e = number of edge connectors = 5, n_p = number of purlins = 0, α_2 = purlin distribution factor = 0, $\sum(x_p)^2$ = 0 for no purlins, x_e = distance from individual panel centerline to any fastener in a panel along the end support members (mm), $\sum x_e$ = (4 + 4 + 12 + 12) = 32 in, $\sum(x_e)^2$ = (16x2 + 144x2) = 176 in^2, α_1 = end distribution factor = $\sum(x_e)$ / w = 32 / 24 = 1.33. S_f = structural connector flexibility (SDI Eq. 4.5.1-1) = $1.3\times10^{-3}/(t)^{0.5}$ = 0.005946 in/kip, S_s = side lap connector flexibility (SDI Eq. 4.5.1-2), = $3\times10^{-3}/(t)^{0.5}$ = 0,013722 in/kip,

Shear Stiffness Equation:
The effective shear stiffness of a diaphragm is defined by equation 3.3-3 of the SDI Design Manual (Luttrell, 2004) as:

$$G' = \frac{Et}{\left(2.6(s/d) + D_n + C\right)} \qquad \text{(SDI Eq. 3.2-3)}$$

where G' = effective shear modulus (kip/in), E = modulus of elasticity = 29000 (ksi), t = base metal thickness (in), s = girth of corrugation per rib (in), d = corrugation pitch (in), D_n = warping constant, and C = slip coefficient.

Connector Slip Parameter:
The third edition of the SDI Design Manual (Lutrell 2004) presents a simplified equation (Equation 3.3-1) for the connector slip parameter (C). The simplified equation is based on the assumption that the number of intermediate edge connectors (n_e) is equal to the number of side lap fasteners (n_s). For bridge PMDF systems there are no intermediate edge connectors. Hence, n_e is not equal to n_s, and the simplified equation is not usable. For this reason, the exact equation of the connector slip parameter, C, given in page 28 of the 1st Ed. of the SDI Diaphragm Design Manual (Lutrell, 1981) will be used for both unstiffened and stiffened deck systems:

$$C = \left(\frac{2EtLS_f}{a}\right)\left(\left(\frac{n_{sh}-1}{2\alpha_1 + n_p\alpha_2 + \dfrac{2n_sS_f}{S_s}}\right) + \left(\frac{1}{2\alpha_1 + n_{\alpha_2} + n_e}\right)\right)$$

$$C = \left(\frac{2 x 29000^{ksi} x 0.0478^{in} x 108^{in} x 0.005946^{in/kip}}{96^{in}}\right)\left(\left(\frac{4-1}{2 x 1.33 + 0 + \dfrac{2 x 5 x 0.005946^{in/kip}}{0.013722^{in/kip}}}\right) + \left(\frac{1}{2 x 1.33 + 0 + 5}\right)\right)$$

C = 10.55

Warping Constant, D_n:
The warping constant, D_n, is defined by Eq. 3.3-2 in SDI Design Manual (Lutrell, 2004):

$$D_n = \frac{D}{12L} \qquad \text{(SDI Eq. 3.3-2)}$$

where D = warping constant depending on fastener arrangement.
For this study all of the test specimens were fully fastened. A detailed solution for D values is given in Appendix-IV of SDI Design Manual (Lutrell, 2004). Deck profile dimensions (f, w, e, h, and d) used in warping constant equations are presented in Fig. 5. These dimensions were the average of three measurements and were almost the same for all four deck types. Warping constant, D_n, is calculated here as shown in Appendix-IV of SDI Design Manual (Lutrell, 2004):
WT = $4f^2(f+w)$ = 1028.5 in^3, WB = $16e^2(2e+w)$ = 80 in^3, PW = $1/(t)^{1.5}$ = 95.7 in$^{-1.5}$, A = 2e/f = 0.363, D1 = $h^2(2w+3f)/3$ = 67.5 in^3, D2 = D1/2 = 33.8 in^3,
V = 2(e+w)+f = 13.5 in, D3 = $(h^2/12d^2)((V)(4e^2-2ef+f^2)+d^2(3f+2w))$ = 20.7 in^3,
C1 = $1/(D3-D2/2)$ = 0.263 in^{-3}, D4[1] = $(24f/C1)(C1/WT)^{0.25}$ = 62.45 in$^{5/2}$,
G4[1] = D4[1] = 62.45 in$^{5/2}$, DW[1] = (G4[1])(f/d)(PW) = 4108.8 in,
D_n = D/12L = DW[1]/L = 4108.8/108 = 38.0

Shear Stiffness Calculation:
G'_{closed} = $(29000^{ksi}\times0.0478^{in})/[2(1+0.3)(13.5^{in}/8^{in})+0+10.55]$ = 94.2 kip/in (closed end deck)
G'_{open} = $(29000\times0.0478)/[2(1+0.3)(13.5^{in}/8^{in})+38.0+10.55]$ = 26.5 kip/in (open end deck)

Appendix II: SDI Diaphragm Strength Example
Dimensions (Test #13): Same as given in Appendix I.
Shear Strength Equation:
The shear strength of a diaphragm is given by either one of Eqs. 2.2-2, 2.2-4, and 2.2-5 in the SDI Design Manual (3rd Edition, 2004):

$$S_n = \frac{\left(2\alpha_1 + n_p\alpha_2 + n_e\right)Q_f}{L} \qquad \text{(SDI Eq. 2.2-2)}$$

$$S_{ni} = \frac{\left(2A(\lambda-1)+B\right)Q_f}{L} \qquad \text{(SDI Eq. 2.2-4)}$$

$$S_{nc} = \left(\frac{N^2B^2}{L^2N^2+B^2}\right)^{0.5} Q_f \qquad \text{(SDI Eq. 2.2-5)}$$

where,

$$B = n_s\alpha_s + \frac{2n_p\sum x_p^2 + 4\sum x_e^2}{w_2}, \lambda = 1 - \frac{DL_v}{240(t)^{0.5}}, \alpha_s = \frac{Q_s}{Q_f}$$

S_{ne} = shear strength involving the longitudinal edge of a panel over the line where force transfer is made to the structural system at joist ends, S_{ni} = shear strength involving force transfer at interior panel to panel connections, S_{nc} = shear strength involving corner fasteners across the ends of the panels, Q_f and Q_s = sheet to structural member and sheet to sheet fastener strengths, A = 1, λ = factor that depends on panel depth, purlin spacing, and sheet thickness, B = factor that depends on the number of stitch connectors, width of the panel, number of purlins, distance from panel centerline to fasteners, and ratio of stitch connector strength to fastener strength, N = number of fasteners per unit length along the panel width, L_v = purlin spacing (in).

SDI Eq. 2.2-5 assumes that λ=1. However, this assumption is not valid for the deck profiles used in this study. For bridge metal deck form systems n_p=0 and L_v=L. SDI Eq. 2.2-5 is rewritten here by including the λ term (given also by Currah, 1993). The following expanded equation was used in the strength calculations:

$$S_{nc} = \sqrt{\dfrac{Q_f^2}{\left(\dfrac{L}{2A(\lambda-1)+B}\right)+\dfrac{1}{N^2}}} \qquad \text{(SDI Eq. 2.2-5 wit } \lambda \text{ term)}$$

<u>Structural and Side Lap Fastener Strengths (Q_f, and Q_s):</u>

$$Q_f = 1.25 F_y t \left(1 - 0.005 F_y\right)\left(kip\right) \qquad \text{(SDI Eq. 4.5-1)}$$

$$Q_s = 115 dt \left(kip\right) \qquad \text{(SDI Eq. 4.5-2)}$$

where F_y = yield strength of sheet metal (ksi) = 58.7 ksi (Egilmez, 2005), d = major diameter of the screw (mm) = ¼ in, and t = base sheet metal thickness (in).
Q_f = 1.25×58.7×0.0478×(1-0.005×58.7) = 2.48 kip, Q_s = 115×0.25×0.0478 = 1.37 kip α_s = 0.552,

<u>Shear Strength Calculation:</u>
λ = 1-3×107/(240×(0.0478)$^{0.5}$) = 0.557<0.70→λ=0.7, B = 4.99.
A = 1 for single fasteners at panel edges, L_v = L = 107 in (single span),
N = number of end fasteners per meter = 13/96 = 0.135 fasteners/in,
S_{ne} = (2×1.33+0+5)×2.48/107 = 0.176 kip/in,
S_{ni} = (2×1×(0.7-1)+4.99) ×2.48/107 = 0.100 kip/in, and
S_{nc} = (0.135^2×4.99^2/(107^2×0.135^2+4.99^2))$^{0.5}$×2.48 = 0.097 kip/in

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