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Predicting the stiffness of shear diaphragm panels composed of bridge metal deck forms

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Abstract. The behavior of building industry metal sheeting under shear forces has been extensively studied and equations have been developed to predict its shear stiffness. Building design engineers can make use of these equations to design a metal deck form bracing system. Bridge metal deck forms differ from building industry forms by both shape and connection detail. These two factors have implications for using these equations to predict the shear stiffness of deck form systems used in the bridge industry. The conventional eccentric connection of bridge metal deck forms reduces their shear stiffness dramatically. However, recent studies have shown that a simple modification to the connection detail can significantly increase the shear stiffness of bridge metal deck form panels. To the best of the author's knowledge currently there is not a design aid that can be used by bridge engineers to estimate the stiffness of bridge metal deck forms. Therefore, bridge engineers rely on previous test results to predict the stiffness of bridge metal deck forms in bracing applications. In an effort to provide a design aid for bridge design engineers to rely on bridge metal deck forms as a bracing source during construction, cantilever shear frame test results of bridge metal deck forms with and without edge stiffened panels have been compared with the SDI Diaphragm Design Manual and ECCS Diaphragm Stressed Skin Design Manual stiffness expressions used for building industry deck forms. The bridge metal deck form systems utilized in the tests consisted of sheets with thicknesses of 0.75 mm to 1.90 mm, heights of 50 mm to 75 mm and lengths of up to 2.7 m; which are representative of bridge metal deck forms frequently employed in steel bridge constructions. The results indicate that expressions provided in these manuals to predict the shear stiffness of building metal deck form panels can be used to estimate the shear stiffness of bridge metal deck form bracing systems with certain limitations. The SDI Diaphragm Design Manual expressions result in reasonable estimates for sheet thicknesses of 0.75 mm, 0.91 mm, and 1.21 mm and underestimate the shear stiffness of 1.52 and 1.90 mm thick bridge metal deck forms. Whereas, the ECCS Diaphragm Stressed Skin Design Manual expressions significantly underestimate the shear stiffness of bridge metal deck form systems for above mentioned deck thicknesses.

Keywords: metal deck sheet; shear stiffness; stability bracing; buckling; stiffness requirements

1. Introduction

Light gage metal sheeting is commonly used for a variety of applications in the building and bridge industries (Luttrell 2004, Cao *et al*. 2013, 2014, Massarelli *et al*. 2012, Seres and Dunai 2011). In the building industry, metal sheeting is frequently used as metal cladding for siding and roofs, and also for stay-in-place (SIP) formwork for flooring systems. The sheeting possesses a significant amount of in-plane stiffness and is often treated as a shear diaphragm for resisting lateral loads on the structure, and for supplying stability bracing to the beams and columns of the framing system. Although SIP forms are also commonly used in the bridge industry, the forms are not generally relied upon for stability bracing due to flexibility in the connection methods that can seriously reduce the stiffness of the forms. However, the bridge decking does have significant bracing potential, provided that improved connection details between the forms and the girders are utilized. Efficient design aides are desirable to effectively rely upon the forms during design. Therefore, understanding the design methods utilized for shear diaphragms in the building industry are necessary to develop an effective approach for bridge decking.

Designers in the building industry have long relied on the in-plane stiffness and strength of the metal forms to provide lateral restraint to the structural framing. Specifically germane to the topic of this paper is the bracing ability of metal sheeting to enhance the lateral torsional buckling resistance of beams. In traditional steel building constructions, metal forms are typically continuous over the tops of the beams. In such applications, metal sheeting acts as a shear diaphragm, restraining the lateral movement of the top flange of the beams. The forms provide continuous bracing to the beams to which they are attached during placement of the wet concrete flooring system. 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 2014) do not currently permit metal deck forms to be considered for bracing in

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steel bridge girders. Bridge industry forms differ from those used in the building industry in 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 generally continuous over the beams, and are fastened directly to the girder flanges by puddle welds, mechanical fasteners, or by welding the shear studs in composite flooring systems directly through the forms. Forms in the bridge industry, on the other hand, are not continuous over the tops of the girders. Instead, individual forms span between adjacent girders, and are typically supported by cold-formed angles fastened to the top flange, as depicted in Fig. 1. The angles allow the contactor to adjust the form elevation to account for changes in flange thickness, as well as differential camber between adjacent girders. Due to the large variation in flange thickness values between the positive and negative moment regions, as well as large tolerances in camber for longer span girders, it is crucial to be able to adjust the form elevation in order to achieve a uniform slab thickness along the length of the bridge.

Although the support angles are beneficial for constructability issues, the eccentricity introduced into the connection as a result of the support angle can seriously reduce the in-plane stiffness and bracing effectiveness of the forming system. The reason for this is because bracing systems are often governed by the equation for springs in series as illustrated in the following expression

$$Q = G' s_d \tag{1}$$

where β_{sys} = the stiffness of the deck and connection system; β_{deck} = the stiffness of the metal deck form; and β_{con} = 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 formwork (β_{deck}) may be very stiff, the flexibility of the 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 bracing behavior of PMDF systems commonly used in the bridge industry has been studied in the past (Currah 1993, Alenius 2002, Egilmez *et al*. 2007, 2012, and 2016a). To the best of the author's knowledge, Currah (1993) was the first researcher to investigate the stiffness and strength

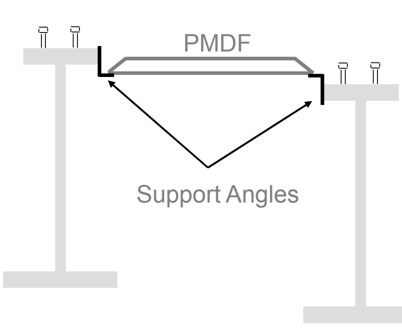


Fig. 1 Conventional method of connection of PMDF's in the bridge industry

of bridge metal deck forms. In 2002, bridge metal deck forms were relied upon for construction bracing of a 65 m long simply supported steel box girder bridge in Sweden (Alenius 2002). Unfortunately, the box girder bridge structure collapsed during deck casting. A primary aspect of the research conducted by Egilmez *et al*. (2007, 2012) 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 tests on PMDF systems in a shear frame, and lateral displacement and buckling tests on a 15 m twin-girder system with PMDF for bracing. The laboratory tests demonstrated that edge stiffened panel modification of the bridge decking can result in substantial stability bracing for steel bridge girders. The proposed edge stiffened panel modification consisted of transverse "stiffening angles" that span between the top flanges of adjacent girders 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, as depicted in Fig. 2. Results from the research study were implemented in the design and construction of two steel I-girder bridges in Houston, Texas (Egilmez *et al*. 2016a), which utilized PMDF to stabilize 15 m long simply-supported W460×177 girders (Fig. 3).

The fundamental behavior of stability bracing systems were studied by Winter (1960), who showed that effective stability bracing must possess both stiffness and strength. The behavior of shear diaphragm systems have been studied both analytically and experimentally (Errera and Apparao 1976, Currah 1993, Helwig and Frank 1999, Galanes and Godoy 2014) in the past and stiffness of strength of shear diaphragm bracing systems have been developed (Helwig and Yura 2008a and b). The results from these studies can be used to determine the necessary stiffness and strength requirements for diaphragm bracing in both the building and bridge applications. Designers in the building industry can utilize design aids, such as the Steel Deck Institute (SDI) Diaphragm Design Manual (Luttrell 2004) or the European Convention for Constructional Steelwork (ECCS) Recommendation for the Application of Metal Sheeting as a Diaphragm (ECCS 1995), to select the forming system with the appropriate degree of stiffness and strength. However, specifying the correct PMDF sheeting can be difficult for bridge designers, since there is currently no design aid appropriate for bridge decking. The SDI Diaphragm Design Manual (Luttrell 2004) and ECCS (1995) provide numerical expressions to determine the stiffness and strength of various types of metal sheeting and their corresponding connection details that are frequently used in the building industry. Due to substantial differences in the shape and connection method of bridge decking, the stiffness expressions in the SDI Design Manual and ECCS are not directly applicable to bridge metal deck forms.

In this paper, results from shear diaphragm tests conducted by Currah (1993) and Egilmez *et al*. (2007) on bridge metal deck form systems with and without edge stiffened panels are compared with SDI and ECCS stiffness expressions in order to evaluate the ability of these design aid expressions to predict the stiffness of bridge metal deck

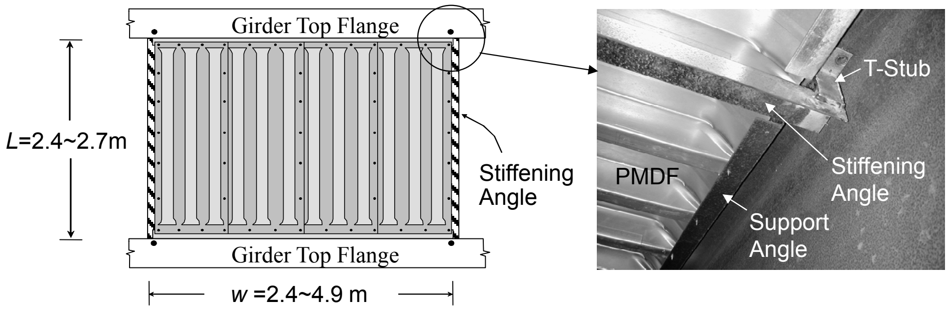
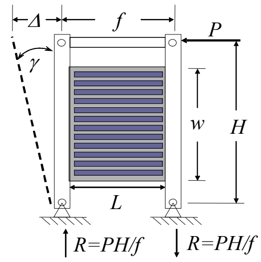


Fig. 2 Edge stiffened panel modification

Fig. 3 Photograph from the implementation project in Houston, TX (Egilmez *et al*. 2016a)



$$\tau' = \frac{PH}{f\text{w}} \qquad \gamma = \frac{\Delta}{L}$$

Fig. 4 Cantilever shear test frame

forms with conventional connection details, as well as with modified edge stiffened panels. The paper begins by presenting background information and design procedure for diaphragm bracing, followed by a description of the shear-panel tests used in the comparison. Results from the experiments as well as comparisons of test results with values obtained from equations in the SDI Diaphragm Design Manual (Luttrell 2004) and ECCS (1995) are presented next, followed by a summary of the findings. A design example with application of the stiffness equations is presented in Appendix I.

2. Background

Metal deck forms are generally modeled as shear diaphragms due to their significant in-plane shear stiffness and strength. From a bracing standpoint, the most important parameter of shear diaphragms is the shear rigidity, which represents the resistance of a diaphragm along the beam length for 1 rad shear strain. Shear rigidity has units of kN/rad and is calculated as the product of the effective shear stiffness (G') of the diaphragm and the tributary width of the deck bracing a single beam (s_d) as follows

$$s_d = \frac{(n-1)}{n} s_g \tag{2}$$

For a system with n beams with a spacing of s_g, the tributary width of the deck bracing a single beam can be calculated as

$$s_d = \frac{(n-1)}{n} s_g \tag{3}$$

The effective shear stiffness of a diaphragm can be measured experimentally by utilizing a cantilever test frame, such as the one depicted in Fig. 4. In such a frame, the diaphragm is typically fastened to the test frame on four sides to ensure a pure shear deformation. As seen in Fig. 4, the effective shear stiffness of the PMDF system can be calculated as follows

$$G' = \frac{\tau'}{\gamma} = \frac{PHL}{fw\Delta} \tag{4}$$

where G' = the effective shear stiffness of the diaphragm (kN/m-rad); P is the shear load applied to the diaphragm (kN), L is the length of the test frame (m), f is the length of the diaphragm (m), w is the width of the diaphragm (m), τ' is the effective shear stress of the corrugated sheet (kN/m), γ is the shear strain, and Δ is the shear deflection of the diaphragm (m). 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, it is common to utilize "an effective shear stiffness", G', which is not a function of the material thickness.

In lieu of laboratory testing for applications in the building industry, the effective shear stiffness 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. The most significant body of work on the shear behavior of building metal deck forms was conducted at Cornell University in the 1960's and 1970's (Errera and Apparao 1976). In 1973, Bryan presented energy-based solutions for flexibility and fastener forces for shear diaphragms (Bryan 1973). Davies also investigated the shear behavior of diaphragms modifying Bryan's approach to develop formulas that gave more accurate calculations of shear flexibility and fastener forces for diaphragms supported on two sides (Davies 1976). In 1982, Davies and Bryan published the Manual of Stressed Skin Diaphragm Design, which presented modified solutions for diaphragm flexibility and strength (Davies and Bryan 1982). Davies and Bryan (1982) provide a good summary of diaphragm behavior as well as an overview of numerical modeling techniques for the diaphragms and associated fasteners. Davies and Bryan's study also forms the basis of the ECCS Recommendations (1995). Davies and Bryan followed a flexibility based approach and defined the total flexibility of a diaphragm with sheeting perpendicular to span as the summation of four flexibility components due to: shear deformation in the sheet (c_1), distortion of the sheeting profile (also defined as warping deformation) (c_2), movement at the sheet to perpendicular member fasteners (c_3), movement in the seams (c_4). Therefore, the total flexibility of the diaphragm is given as

$$c = c_1 + c_2 + c_3 + c_4 \tag{5}$$

$$c_1 = \frac{2a(1+\nu)[1+(2h/d)]}{Etb} \tag{6}$$

$$c_2 = \frac{ad^{2.5}\alpha_1\alpha_4 K}{Et^{2.5}b^2} \tag{7}$$

$$c_3 = \frac{2as_p p}{b^2} \tag{8}$$

$$c_4 = \frac{2s_s s_p(n_{sh}-1)}{2n_s s_p + \beta_1 n_p s_s} \tag{9}$$

where a = width of panel in a direction perpendicular to the corrugation (mm); b = depth of panel in a direction parallel to the corrugation (mm); d = pitch of corrugation (mm); E = the stiffness of the metal deck form (kN/mm^2); t = net sheet thickness (mm); K = non-dimensional sheet constant, α_1 = 1.0; α_4 = 1.0; ν = Poisson's ratio; s_p = flexibility of sheet to perpendicular member fastener (mm/kN); p = pitch of sheet to perpendicular member fasteners (mm); n_s = number of seam fasteners per sidelap; n_p= total number of perpendicular members (purlins); n_{sh} = number of sheet widths per panel; s_s = flexibility of seam fastener per unit load (mm/kN); β_1 = factor that depends on n_f; n_f = number of sheet to perpendicular member fasteners per member per sheet.

During the 1970's Luttrell directed an extensive study on shear diaphragms at West Virginia University (Luttrell 1981). Formulas for diaphragm strength and stiffness predictions resulting from these studies were published by the Steel Deck Institute (SDI) as a design manual (Luttrell 1981). Luttrell (1981) showed that the shear stiffness was primarily dependent on three factors: shear strain in the forms, the warping deformation of the corrugation at panel ends, and slip at the fastener locations; given by

$$G' = \frac{Et}{(2.6[s/d] + D_n + C)} \tag{10}$$

where G' = effective shear stiffness (kN/mm); E = modulus of elasticity (N/mm^2); t = base metal thickness (mm); s = girth of corrugation per rib (mm); d = corrugation pitch (mm); D_n = warping constant; and C = slip coefficient. An example showing how the shear stiffness of a panel is calculated by the SDI and ECCS expressions is presented in Appendix I.

3. Design procedure of shear diaphragms used to brace steel I-beams

A number of studies have investigated the buckling behavior of steel I-beams braced by shear diaphragms. Errera and Apparao (1976) developed equations to calculate the buckling capacity of diaphragm braced I-beams under uniform moment loading. Later, Helwig and Frank (1999) conducted a computational study to investigate the buckling behavior of diaphragm braced slender beams by considering moment gradient and load height effects. Helwig and Frank (1999) modified the uniform moment solution from the above studies to make it applicable for general loading conditions and recommended the following expression to approximate the buckling capacity of diaphragm braced beams

$$M_{cr} = C_b^* M_g + mQd \tag{11}$$

where M_{cr} = buckling capacity of the diaphragm braced beam (kN-m); C_b^* = moment gradient factor including load height effects (Ziemian 2010); M_g = buckling capacity of the girder with no bracing (kN-m); m = constant that depends on load height and web slenderness ratio; Q = shear rigidity of the diaphragm (kN-m/m-rad); and d = depth of the section (m). The expression presented in Eq. (11) is applicable to perfectly straight girders and can be used to solve for the ideal deck shear rigidity (Q_i) for a given moment level. Helwig and Yura (2008b) used Eqs. (11) and (2) to solve for the ideal deck shear stiffness for a given moment as follows

$$G'_i = \frac{Q_i}{s_d} = \frac{(M_u - C_b^* M_g)}{mds_d} \tag{12}$$

The "ideal stiffness" of a bracing system corresponds to the brace stiffness required for a structural member to reach a specific load level or buckling capacity. In order to control deformations and brace forces, a brace stiffness higher than the ideal stiffness is required. Helwig and Yura (2008b) also investigated the stiffness and strength requirements of shear diaphragms used to brace stocky beams. They found that deformations and brace forces could be effectively controlled by providing four times the ideal stiffness; therefore, the stiffness requirement for shear diaphragms is as follows

$$G'_{req'd} = 4G'_i = 4\frac{Q_i}{s_d} = 4\frac{(M_u - C_b^* M_g)}{mds_d} \tag{13}$$

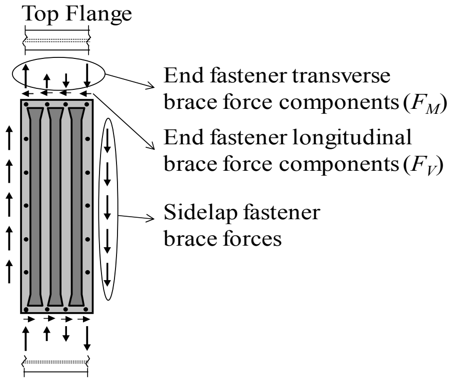


Fig. 5 Brace forces

In addition to sufficient stiffness; a bracing system should also possess sufficient strength. The strength of a diaphragm is generally governed by the shear strength of the longitudinal end connections (sheet to structural member connection along the ends), shear strength at interior connections between panels (sheet to sheet connections along sidelaps), and local or global buckling of the diaphragm (Davies and Bryan 1982, Luttrell 1981). The brace forces that develop at end and sidelap fastener connections in a single deck sheet are depicted in Fig. 5. The sheet shown in Fig. 5 has four end fasteners connecting sheet ends to beam top flanges, and five sidelap fasteners at sidelaps connecting deck sheets to each other. Helwig and Yura (2008b) also conducted large displacement analysis to develop strength requirements for shear diaphragms used to brace stocky beams. In their study, Helwig and Yura (2008b) ignored the presence of sidelap fasteners, and focused merely on the longitudinal end fasteners connecting deck sheets to beam top flanges. In mid 2010's, the study of Helwig and Yura (2008b) was extended by Egilmez *et al.* (2016b) by including the effects of sidelap fasteners. Egilmez *et al.* (2016b) found that the stiffness requirement originally proposed by Helwig and Yura (2008b) (Eq. (13)) resulted in significantly conservative estimates of the brace forces at end fastener connections. Egilmez (Egilmez *et al.* 2016b), recommended using the following expressions to achieve reasonable estimates of end and sidelap fastener brace forces

$$F_{br-e} = C_r s_e k_e w_e \frac{M_u L}{d^2} \tag{14}$$

$$F_{br-s} = C_r s_s k_s w_s \frac{M_u L}{d^2} \tag{15}$$

where F_{br-e} = maximum end fastener brace force (kN); F_{br-s} = maximum sidelap fastener brace force (kN); C_r = reduction coefficient that depends on the provided stiffness (Helwig and Yura 2008a); s_e and s_s = factors that depend on number of sidelap fasteners; k_e and k_s = factors that depend on number of end fasteners; w_e and w_s = factors that depend on deck width; M_u = maximum design moment (kN-m); L = spacing between discrete brace points that prevents twist (m); and d = depth of the section (m).

Conventional stability bracing applications in the bridge industry often consists of cross frames or diaphragms spaced approximately at 7.5 m along the length of the girders. In earlier AASHTO Standard Specifications, the maximum spacing permitted between cross-frames and diaphragms was set at 7.5 m. Although this spacing limit was abolished in the AASHTO LRFD Specification (1994), primarily due to fatigue problems that were frequently found around the brace locations, many design engineers continue to apply the 7.5 m spacing between cross frames. Therefore, utilizing PMDF as a bracing element in bridge applications can substantially reduce the number of cross frames or diaphragms necessary to stabilize the bridge during construction. Vardaroglu (Egilmez *et al.* 2014) showed that for slender beams with h/t_w of up to 160, shear diaphragms can be relied upon for providing stability bracing to singly or doubly symmetric I-beams with length to depth (L/d) ratios of up to 15 during construction. For beams with a mid-span cross frame, Vardaroglu (Egilmez *et al.* 2014) showed that shear diaphragms are effective for beams with L/d of 30.

Proper load factors should also be applied. For example, if the LRFD format is used in design, M_{cr} should be the factored design moment, and the strength of the end and sidelap fastener connections should be reduced by a resistance factor, ϕ, of 0.65 (Luttrell 2004). If the ASD format is used in design, M_{cr} should be based on service level loads and the strength of the end, and sidelap fastener connections should be reduced by a factor of safety, Ω, of 2.5 (Luttrell 2004). For a given construction design moment level, a designer in the building industry can use Eq. (13) to determine the required stiffness of the shear diaphragm that needs to be provided as a continuous bracing source. The designer can then use the stiffness expressions provided in the SDI Diaphragm Design Manual (Luttrell 2004) or ECCS Recommendations (ECCS 1995) to identify a deck sheet configuration with the required diaphragm stiffness. After the identification, Eqs. (14) and (15) can provide a reasonable estimate of the brace forces that develop at end and sidelap fastener connections. These brace forces can then be compared with the capacities of the end and sidelap fastener connections, which can be obtained by expressions provided by the above mentioned design aids. The local and global buckling of the diaphragm should also be checked.

Bridge industry designers can still use Eq. (13) to determine the required stiffness of the shear diaphragm for a given design construction moment level. However, specifying the appropriate deck sheet configuration can be challenging because the stiffness expressions in the SDI Design Manual (Luttrell 2004) and ECCS Recommendations (1995) are not directly applicable to bridge metal deck form systems. Currently, bridge designers benefit from the result of previous cantilever shear test results of specific PMDF configurations. For example, for an implementation project in Houston, TX (Egilmez *et al.* 2016a), in which, edge stiffened PMDF panels were utilized to stabilize 15 m long bridge girders, Texas Department of Transportation (TxDOT) required not only cantilever shear tests but also a full scale buckling test to verify that the edge stiffed PMDF panels could adequately brace the 15 m long girders. By

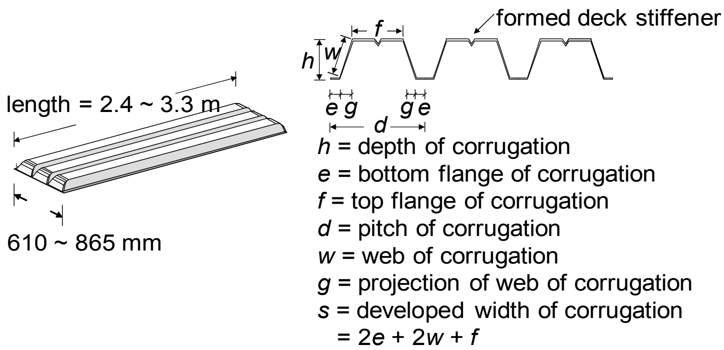


Fig. 6 Profrile and dimensions of PMDF specimens

utilizing edge stiffened PMDF panels as the bracing source, it was possible to eliminate a total of 680 intermediate diaphragms required in the case of conventional bracing. Once the PMDF configuration has been determined, the bridge designer can then use Eqs. (14) and (15) to obtain a reasonable estimate of the brace forces that develop at end and sidelap fastener connections. SDI Design Manual (Luttrell 2004) or ECCS Recommendations (1995) expressions can then be used to check the strength of the fastener connections. An example on how to use these expressions is presented in Appendix I.

4. Overview of PMDF Shear Diaphragm Tests

Results from two test programs were used to evaluate the capability of SDI (Luttrell 2004) and ECCS (1995) stiffness expressions to predict the stiffness of bridge metal deck forms: (a) Currah (1993); (b) Egilmez *et al*. (2007). The deck panel specimens tested in both of these studies were pre-closed (tapered closure) open profile metal deck forms. Fig. 6 shows the dimensions and profile configuration of the PMDF specimens tested in these investigations. Fig. 6 also illustrates the pitch, depth, thickness, width, rib trough, and rib crest of a typical open profile deck panel. The pitch is the spacing between consecutive ribs, rib trough is the valley between the ribs, and rib crest is the top of the ribs. Various open profile decks were tested in these studies. Egilmez *et al*. (2007) focused mainly on 75×203 (75 mm depth, 203 m pitch) bridge metal deck forms with 610 mm cover width and metal thicknesses of 0.75 mm, 0.91 mm, 1.21 mm, and 1.52 mm. PMDF spans of 2440 mm and 2740 mm were considered that will be henceforth referred to as 2.4 m and 2.7 m, respectively. A total of 17 shear panel tests with different connection details were conducted. In all of the tests, PMDFs were fastened to support angles in every rib trough, creating a fully fastened deck configuration. A wider range of deck profiles were tested in Currah's study. The thickness, depth, pitch, cover width, and span of the open profile decks tested in Currah's (1993) study ranged between 0.75 mm to 1.90 mm, 50 mm to 75 mm, 150 mm to 220 mm, 610 mm to 865 mm, and 2.4 m to 3.30 m, respectively. Currah (1993) conducted six fully fastened and 6 partially fastened PMDF shear panel tests. Results from only the fully fastened systems are presented since bridge designers will most likely specify fully fastened deck panels in bracing applications.

Metal deck forms commonly used in bridge constructions generally consist of deck sheets with a cover length of 610 and 914 mm (CANAM 2006, Luttrell 2004). These forms are manufactured with different thicknesses, heights, and lengths. Typical respective thicknesses, heights and lengths of these deck forms range between 0.75 mm to 1.52 mm, 38 mm to 75 mm, and 2 m to 3 m. The deck heights, thicknesses, and spans used in both Currah's (1993) and Egilmez *et al*.'s (2007) studies were representative of PMDF heights, thicknesses, and spans frequently employed in steel bridge constructions.

In both of the test programs a test frame similar to that shown in Fig. 4 was used to conduct shear tests on different PMDF panels. A detailed description of the test frames can be found in Egilmez *et al*. (2007) and Currah (1993). In both of the studies deck form sheets spanned between the loading beams of the test frame, and were supported on cold-formed, L76×51×3.3 galvanized angles, typically employed in bridge construction. 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 the steel bridge I-girders). Deck sheets were fastened to support angles at the ends and to each other at sidelaps by 6.4 mm diameter No.14 TEKS screws, similar to those in Luttrell's (1981) study. Support angles were welded directly to the top plate of the shear test frame loading beams with 3 mm fillet welds 37.5 mm long, intermittently spaced 300 mm on center; similar to field applications. 76 mm long fillet welds were used at the ends of the individual support angles. In regions where welding to the girder flange is not allowed, the support angles are typically welded to loose strap angles

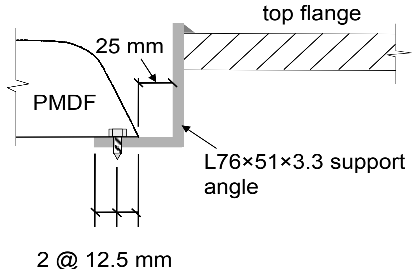


Fig. 7 Conventional PMDF connection utilized in shear diaphragm tests

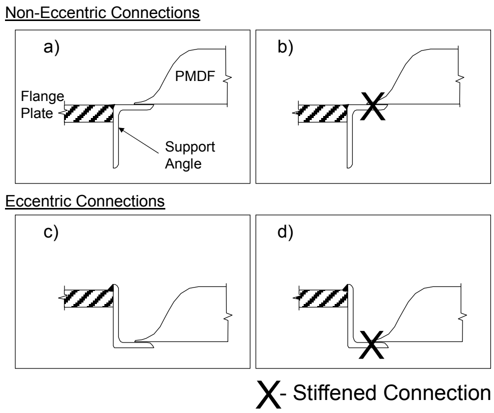


Fig. 8 Eccentric and non-eccentric connection configurations

spaced approximately at 30 cm. These strap angles are not welded to the girders. Hold down clips are used to prevent any uplifting of the deck panels. Deck panels with strap angle connections were not considered in this study. Strap angle connections generally result in a reduction in the stiffness of the deck form system as compared to the stiffness of systems that can be directly welded to girder top flanges.

Currah's (1993) study focused mainly on determining the stiffness of open profile bridge metal deck forms. Therefore, the tests conducted by Currah (1993) included only deck panels with non-eccentric conventional connection detail as depicted in Fig. 8. Egilmez *et al.* (2007) focused on developing a modified panel configuration to increase the stiffness of eccentrically connected PMDF panels, and conducted PMDF panel tests with conventional eccentric and non-eccentric connections, as well as with edge stiffened panel modification. In the edge stiffened tests, transverse L76×51×3.3 galvanized stiffening angles were positioned to coincide with a side-lap seam, with the deck screwed directly to the angle. Spacing between the stiffening angles ranged from 2.4 to 4.8 m. To ensure 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 (63.5 mm long, L76×51×3.3, long leg back-to-back) bolted to the underside of the top flange plate, as shown in Fig. 2. Although these T-sections may also be welded to the flange, the researchers aimed to test the bolted connection in case a contractor used a connection that was less stiff compared to welding. In many of the tests, the PMDF panels were conservatively tested with the largest possible eccentricity. Smaller eccentricities will generally occur along much of the girder length, thereby resulting in stiffer systems. For tests without eccentricity, the long leg of the support angle was oriented vertically downward 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 3 mm above the top flange plate of the support beam, as depicted in Fig. 7.

Four different support angle configurations were tested: (a) no eccentricity-unstiffened; (b) no eccentricity-stiffened; (c) maximum eccentricity-unstiffened; and (d) maximum eccentricity-stiffened. Although not recommended for bracing applications, results from tests of maximum eccentricity and no stiffening are also presented to show the effect of edge stiffened panel modification on shear stiffness of panels with eccentric support angles. The four support angle configurations that are considered are illustrated in Fig. 8. The symbol "X" indicates a stiffened connection with the approximate eccentricity shown by the vertical position of the "X".

5. Comparison of experimental and SDI-ECCS shear stiffness values

As previously mentioned, SDI and ECCS has published design manuals which enable the designer to estimate the diaphragm shear stiffness and strength (Luttrell 2004, ECCS 1995). These manuals are primarily for metal decks used in the building industry which are open ended and fastened directly to the structural members on four sides. The purpose of this section is to provide comparisons between the SDI and ECCS design manual stiffness equations, and measured stiffness of the forms used in the bridge industry with edge stiffened panel modifications. The aim of these comparisons is to determine whether the SDI and ECCS expressions have the potential to predict the behavior of forms used in the bridge industry.

The SDI and ECCS shear stiffness equations are not directly applicable to bridge industry deck forms for two reasons. First, the stiffness equations of the SDI and ECCS design manual is specifically for open end deck forms, but bridge industry deck forms have tapered closed ends with the sheeting folded at the edges. The closed ends result in an increase in the warping resistance and hence the shear stiffness of the diaphragm. The second reason arises due to the assumption made in the SDI and ECCS stiffness equations 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, which are fastened to stiff structural members on four sides. However, in the bridge industry, even though the edge stiffened panel modification depicted earlier in Fig. 2 enables shear transfer in the deck panels on all four sides by means of the transverse stiffening angles, the L76×51×3.3 stiffening angles are not generally stiff enough to result in a deformed shape consistent with pure shear. Therefore, the assumption that corrugations remain straight does not necessarily apply to deck forms used in the bridge industry. Hence, these two factors have implications for using SDI and ECCS shear stiffness equations to predict the shear stiffness of deck form systems used in the bridge industry.

This section provides a comparison of experimentally measured shear stiffness values for 0.75 mm, 0.91 mm, 1.21 mm, 1.52 mm, and 1.90 mm thick PMDF systems to stiffness values computed using both SDI (Luttrell 2004) and ECCS (1995) equations for diaphragm stiffness (previously presented as Eqs. (5) and (10)). Results from tests conducted by Currah (1993) and Egilmez *et al.* (2007)

and stiffness predictions from SDI and ECCS equations are summarized in Table 1, which has been divided into seventeen columns labeled (a) through (r). Column (a) shows the test number of the connection detail used. Columns (b) through (e) present the deck sheet geometry, which consists of the metal thickness, the depth, the pitch, and the width of the individual deck sheets. Columns (f) through (h) 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. The term "panel" refers to the width of the collection of sheets tested in the frame, ranging from 4 to 8 sheets connected along the sidelaps of the individual sheets. Columns (i) through (m) list the computed SDI and ECCS stiffnesses and the measured shear stiffness. In order to improve the comparison between the test results and SDI and ECCS equations, two SDI and ECCS stiffness values are presented: Closed end deck stiffness and open end deck stiffness. The stiffness for closed end decks (theoretically warping restrained) was computed by removing the

Table 1 Shear stiffness of deck systems from laboratory tests, SDI and ECCS expressions

Researcher	Connection type	(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)	(k)	(l)	(m)	(n)	(o)	(p)	(r)
			Deck sheet				Deck panel			G'_{closed} (kN/m-rad)			G'_{open} (kN/m-rad)		G'_{closed}/G'_{exp}		G'_{exp}/G'_{open}	
		Test #	Thickness: t (mm)	Depth: h (mm)	Pitch: d (mm)	Cover width: s (mm)	Span: L (m)	Width: w (m)	Stiffener spacing: s_s (m)	SDI	ECCS	Experimental stiffness: G'_{exp} (kN/m-rad)	SDI	ECCS	SDI	ECCS	SDI	ECCS
Currah (1993)	No Eccentricity I. Unstiffened	1	1.21	63.5	165	660	2.4	2.6	N/A	16140	13902	13501	12057	6428	1.20	1.03	1.12	2.10
		2	1.52	50	152.5	610	2.4	2.4	N/A	17999	16526	16482	16382	8143	1.09	1.00	1.01	2.02
		3	1.90	63.5	165	660	2.6	2.6	N/A	19744	18358	17183	17298	11550	1.15	1.07	0.99	1.49
		4	1.52	75	203	610	3.0	3.7	N/A	15302	12369	10345	6949	3398	1.48	1.20	1.49	3.04
		5	1.21	63.5	203	813	2.4	2.4	N/A	16933	15942	12274	11589	4532	1.38	1.30	1.06	2.71
		6	1.52	50	216	864	2.4	2.6	N/A	22553	20865	16657	13750	6711	1.35	1.25	1.21	2.48
Egilmez *et al.* (2007)	No eccentricity I. Unstiffened	1	0.75	75	203	610	2.4	2.4	NA	10167	10374	2365	1506	716	4.30	4.39	1.57	3.30
		2	0.91	75	203	610	2.4	2.4	NA	11510	11661	5102	2295	1094	2.26	2.29	2.22	4.66
		3	1.21	75	203	610	2.4	2.4	NA	13773	13985	6376	4104	2093	2.16	2.19	1.55	3.05
	No eccentricity II. Stiffened	4	0.75	75	203	610	2.4	2.4	2.4	10167	10374	2962	1506	716	3.43	3.50	1.97	4.14
		5	0.91	75	203	610	2.4	2.4	2.4	11510	11661	6954	2295	1094	1.66	1.68	3.03	6.36
		6	1.21	75	203	610	2.4	2.4	2.4	13773	13985	10520	4104	2090	1.31	1.33	2.56	5.03
	Maximum eccentricity III. Unstiffened	7	0.75	75	203	610	2.4	2.4	2.4	10167	10374	733	1506	716	13.9	14.2	0.49	1.02
		8	0.91	75	203	610	2.4	2.4	2.4	11510	11661	1679	2295	1094	6.86	6.95	0.73	1.53
		9	1.21	75	203	610	2.4	2.4	2.4	13773	13985	2195	4104	2090	6.26	6.37	0.53	1.05
	Maximum eccentricity IV. Stiffened	10	0.75	75	203	610	2.4	2.4	2.4	10167	10374	2244	1506	716	4.53	4.62	1.49	3.13
		11	0.91	75	203	610	2.4	2.4	2.4	11510	11661	4198	2295	1094	2.74	2.78	1.83	3.84
		12	1.21	75	203	610	2.4	2.4	2.4	13733	13985	7295	3739	2096	1.88	1.92	1.95	3.49
		13	1.21	75	203	610	2.7	2.4	2.4	12606	14658	6530	4316	2569	1.93	2.24	1.51	2.54
		14	1.21	75	203	610	2.7	3.6	3.6	13453	12690	6793	4412	1785	1.98	1.87	1.54	3.81
		15	1.21	75	203	610	2.7	4.8	4.8	14021	11189	7184	4471	1368	1.95	1.56	1.61	5.25
		16	1.21	75	203	610	2.7	4.8	2.4	12606	14658	7662	4316	2569	1.65	1.91	1.78	2.98
		17	1.52	75	203	610	2.7	2.4	2.4	14480	16871	6381	6317	4153	2.27	2.64	1.01	1.54
		18	1.52	75	203	610	2.7	3.6	3.6	15480	10426	6054	6634	2716	2.56	1.72	0.91	2.23
		19	1.52	75	203	610	2.7	4.8	4.8	16152	7544	6486	6755	2018	2.49	1.16	0.96	3.21
		20	1.52	75	203	610	2.7	4.8	2.4	14480	16871	7959	6443	4153	1.82	2.21	1.24	1.92

respective warping terms, c_2 and D_n, from Eqs. (5) and (10) for ECCS and SDI stiffness calculations. Hence, the SDI and ECCS stiffness values for closed end decks presented in columns (i) and (j) represent the upper bound for deck panel stiffnesses. The measured shear stiffnesses of the deck panels are presented in column (k) and the SDI and ECCS stiffness values for open end decks are presented in columns (l) and (m). A detailed example showing the calculations for SDI and ECCS stiffness values presented in columns (i), (j), (l), and (m) is presented in Appendix I for Test #12. Columns (n), (o), (p) and (r) respectively present the ratios of the SDI and ECCS closed end stiffness values to measured stiffness values and ratios of measured stiffness values to the SDI and ECCS stiffness values for open end decks.

The tests have been divided into four categories based upon testing parameters and comparison of results: (I) unstiffened with no eccentricity; (II) edge stiffened panels with no eccentricity; (III) unstiffened with eccentricity; and (IV) edge stiffened panels with eccentricity. The Category I test results demonstrate the performance of unstiffened deck forms with no eccentricity. Although the connection detail between the girders and the deck forms utilized in this category of tests is similar to the building industry detail, there is an important difference. The deck panels in this category were fastened on only two sides; in the building industry, panels are fastened on all four sides. Tests conducted by Currah (1993) fall only into this category. The Category II test results demonstrate the effect of the stiffening angles on non-eccentric connections. Although the deck panels in this category are fastened on four sides, the transverse L76×51×3.3 stiffening angles used in this study are generally more flexible than the purlins used in the building industry. In Category III, results from test specimens with conventional eccentric connections are presented. The stiffness of such systems is governed by the stiffness of the eccentric connection, which is not accounted for in the SDI and ECCS equations. In Category IV, the results from eccentric edge stiffened panels are presented. The eccentric edge stiffened panel detail used in Category IV tests is the edge stiffened panel modification recommended by Egilmez *et al*. (2007, 2012).

It can be seen in Table 1 that for deck panels with an aspect ratio of 1.0 (deck span to panel width) closed end (no warping) stiffness values predicted by the SDI and ECCS expressions are very close to each other. This implies that for deck panels with an aspect ratio of 1.0 the stiffness expressions for shear strain in the forms and slip at the fastener locations result in similar values. For deck panels with an aspect ratio different than 1.0, closed end stiffness predictions of the ECCS expressions deviate from those of the SDI expressions by 6% to 25%. The ECCS expressions take the effects of the aspect ratio directly. On the other hand, comparison of open end stiffness values predicted by the SDI and ECCS expressions reveal that the ECCS expressions result in much lower stiffness predictions. For open end decks, stiffness due to distortion of the sheeting profile (warping) is also included in the total stiffness calculation. The warping term in the ECCS (1995) yields to significantly conservative stiffness estimates as compared to those predicted by the SDI.

For Category I tests, all but one measured stiffness values fell between the computed SDI and ECCS stiffness values for closed and open end decks. For all of the deck panels in these categories, closed end stiffness predictions by the SDI and ECCS expressions overestimated the stiffness of the panels, and open end stiffness predictions by the ECCS expressions significantly underestimated the stiffness of the panels. The ratio of $G'_{exp}/(G'_{open})_{ECCS}$ ranged between 1.49 to 6.36. SDI expressions for open end decks did a better job in predicting the stiffness of the panels. The ratio of $G'_{exp}/(G'_{open})_{SDI}$ ranged between 0.99 to 2.22. The fact that all nine experimental stiffness values were either greater than or almost equal to the computed SDI stiffness values for open end decks implies that the gain in panel stiffness due to the tapered closure of the bridge metal deck forms overcomes the reduction in panel stiffness due to the panels being fastened on only two sides. However, the tapered closure of the deck forms does not increase the panel shear stiffness to comparable values with the shear stiffness of warping restrained deck forms (Presented in columns (i) and (j)).

The Category II test results (Egilmez *et al*. 2007) demonstrate the effect of the addition of the two stiffening angles at the outside edges of the deck panels as shown earlier in Fig. 2. Since no eccentricity was used in the Category II tests, the only difference relative to the Category I tests was the addition of the stiffening angles on the edges of the panels. As expected, the stiffening angles in the Category II tests increased the measured stiffness of the bridge decking as compared to the Category I tests. For deck thicknesses of 0.75 mm, 0.91 mm, and 1.21 mm the ratio of experimental shear stiffness values to the computed SDI and ECCS stiffness values for open end decks (Column (l)) were 1.97, 3.03, 2.56 and 4.14, 6.36, 5.03, respectively; whereas the ratio of the computed SDI and ECCS stiffness values for closed end decks to experimental shear stiffness values were 3.43, 1.66, 1.31 and 3.50, 1.68, 1.33, respectively. As the thickness of the decks increased, the experimental stiffness values approached the computed stiffness values for closed end decks. In contrast, the stiffening angles resulted in no increase in the stiffness estimates computed by the SDI and ECCS expressions for either open or closed end decks, since the expressions were developed for deck panel systems supported on four sides.

Results from eccentric unstiffened panel tests are presented in Category III. The measured stiffness values of these tests were substantially lower than the computed SDI stiffness values for open end decks. As previously explained, the stiffness of such systems is governed by the stiffness of the eccentric connection, unaccounted for in the SDI equations. Results from edge stiffened panels with eccentricity are presented in Category IV. The only difference between the three Category III tests (Tests 7, 8, and 9) and the first three Category IV tests (Tests 10, 11, and 12) was the addition of the stiffening angles on the edges of the panels. When the deck panels with eccentric connections are modified by transverse stiffening angles at the two edges of the panels, the shear stiffness of the panels increases significantly, and approaches to the stiffness of non-eccentric deck panels of Category II tests. The measured stiffness values of deck panels in Category III exceeded the computed SDI and ECCS stiffness values for open end decks. The shear stiffness of Tests 10, 11, and 12 with eccentric edge stiffened panels were greater than the shear stiffness of respective unstiffened eccentric panels by 306%, 250%, and 332%, and smaller than the shear stiffness of respective unstiffened non-eccentric panels by 5.1%, 17.7%, and -14.4%. The aspect ratio of the deck panels in these tests (Tests 10, 11, and 12) was 1.0, similar to those of Category I, II, and III tests conducted by Egilmez *et al*. (2007). The ratio of measured stiffness values to stiffness estimates computed by the SDI and ECCS expressions (for open end deck forms) for these three tests were 1.49, 1.83, 1.95, and 3.13, 3.84, 3.49, respectively.

The span of the panels in Tests 13-20 were approximately 12% longer than the panels used in the first 12 tests. In addition, a wider range of deck widths were tested in tests 13-20: 2.44 m, 3.66 m, and 4.88 m. Another variable that was considered in tests 13-20 were the spacing between the stiffening angles. In Tests 13, 14, 15, and 17, 18, 19, stiffening angles were only at the edges of the panel, while in Tests 16 and 20, stiffening angle spacing was equal to half the width of the panel, which resulted in stiffening angles at the two edges and one at the middle. The difference between Tests 15-19 and 16-20 is the thickness of the deck with respective values of 1.21 mm and 1.52 mm. The extra stiffening angle in Tests 16 and 20 resulted in respective increases of 3% and 12% in the measured stiffness compared to Tests 15 and 19, in which stiffening angles were positioned only at the edges of the panels. A comparison of the measured stiffness values in Tests 14-20 show that the panel aspect ratio did not have a significant impact on the stiffness behavior of the panels.

Due to the presence of the intermediate stiffening angle, in Tests 16 and 20 the overall diaphragm panel width was taken as 2.44 m instead of 4.88 m in the SDI and ECCS stiffness calculations. For Tests 13, 14, 15, and 16, which had a deck span of 2.7 m and deck thickness of 1.21 mm, the ratios of measured stiffness values to stiffness estimates computed by the SDI and ECCS expressions (for open end deck forms) were 1.51, 1.54, 1.61, 1.78, and 2.54, 3.81, 5.25, 2.98, respectively. The ratios of stiffness estimates computed by the SDI and ECCS expressions (for closed end deck forms) to the measured stiffness values were 1.93, 1.98, 1.95, 1.65, and 2.24, 1.87, 1.56, 1.91, respectively. Although the SDI expressions were conservative and provided reasonable approximations for the panels with deck thickness values of 0.75-1.21 mm, the expressions were unconservative for two out of four deck panel tests with deck sheet thickness of 1.52 mm (Tests 17-20). The ratio of the measured to predicted stiffness using SDI expressions for the 1.52 mm thick PMDF systems with stiffeners only at the edges of the 2.4 m, 3.6 m, and 4.8 m wide panels were 1.01, 0.91, and 0.96 for Tests 17 to 19, respectively. This indicates that for edge stiffened panels with deck sheet thickness of 1.52, as the stiffening angle spacing increases the shear stiffness is dominated by the stiffness of the eccentric connection. The overall panel widths of Test 20 and 19 were 4.8 m. However, the panel in Test 20 had an additional stiffening angle at midspan, which increased the shear stiffness of the panel system, as well as the ratio of measured stiffness value to the computed SDI stiffness value for open end decks to 1.24. Once again, the ECCS expressions resulted in conservative estimates of shear stiffness for all 1.52 mm deck panels in Category IV. The ratio of the measured to ECCS shear stiffness estimates for 1.52 mm thick PMDF systems were 1.54, 2.23, 3.21, and 1.92 for Tests 17 to 20, respectively.

6. Conclusions

This paper focused on methods of predicting the shear stiffness of formwork used in the bridge industry. The conventional connection detail for bridge industry formwork has an eccentric connection that greatly reduces the bracing potential of the metal forms. Egilmez *et al.* (2007) recommended using transverse stiffening angles that span between adjacent girder top flanges, such that each stiffening angle is positioned at a sidelap location between adjacent PMDF sheets, allowing the forms to be fastened directly to the angle. However, there is currently no design aid to allow bridge engineers to predict the stiffness of bridge metal deck forms in order to specify the most appropriate PMDF system for bracing purposes. Comparisons were made in this paper between laboratory measurements of the stiffness of shear diaphragms comprised of bridge decking, and predictions using stiffness equations from the SDI (Luttrell 2001) and ECCS (1995).

The comparisons presented in this paper indicate that the SDI stiffness expressions from the building industry result in reasonable estimates of shear stiffness of edge stiffened deck panels with deck sheet thicknesses of 0.75 mm, 0.91 mm, and 1.21 mm. The recommended spacing between stiffening angles would typically be 3 m, which is the usual length of support angles used with metal deck forms in the bridge industry. For thicker deck sheets, most probably the stiffness of the eccentric connection dominates the shear stiffness of the 3.6 m and 4.8 m wide edge stiffened deck panels, and the SDI stiffness expressions overestimate the stiffness of such systems. On the other hand, the ECCS expressions resulted in significantly conservative estimates for all of the 1.52 mm thick deck panels tested in both studies. Since SDI expressions resulted in unconservative estimates for the shear stiffness of above mentioned 1.52 mm thick deck panel systems, expressions from ECCS can be used to predict the shear stiffness of such systems. There are a wide range of bridge metal deck forms which have different geometries and connection details than those used in Currah's (1993) and Egilmez *et al.*'s studies (2007). This study needs to be extended so that SDI and ECCS shear stiffness expressions can be more reliably used to predict the shear stiffness of different bridge metal deck form systems.

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DL

Appendix I: Design example

Flooring system

Consists of a series of ten simply supported W760×134 beams spanning 15 m. The tributary width of deck bracing a single beam is 2.4 m. There is no intermediate discrete bracing system. Determine the required thickness of the deck sheet to provide stability bracing to interior beams during concrete cast. The beam is subjected to a maximum factored moment of 700 kN-m. This moment level is less than ϕM_r = 994 kN-m, the upper limit of elastic behavior (AISC 2010), and corresponds to a construction stress level of 175 MPa.

Check lateral torsional buckling of beam with L_b = 15 m

Although self-weight of the beam acts at mid-height, the majority of the load is applied to the top flange (construction loads and fresh concrete). Therefore, assume the entire load is applied at the top flange. Due to top flange loading, moment gradient factor, C_b, should be modified as follows: $C_b^* = C_b/1.4$ (Ziemian 2010). For top flange uniform distributed loading C_b can be taken as 1.14 (Ziemian 2010). Therefore, $C_b^* = C_b/1.4 = 1.14/1.4 = 0.81$. Assuming elastic buckling, the capacity of the beam can be obtained by the following expression (AISC 2010)

$$\phi M_n = \phi C_b^* M_g = \phi C_b^* \frac{\pi}{L_b}\sqrt{EI_y GJ + \frac{\pi^2 E^2 C_w I_y}{L_b^2}}$$
$$= 181 \text{ kN} - \text{m}$$

Brace stiffness requirement

Use Eq. (13) to calculate the required effective stiffness of the metal deck sheet bracing system

$$G'_{req'd} = 4G'_i = 4\frac{\left(M_u - \phi C_b^* M_g\right)}{s_d md} = 4\frac{(700-181)}{2.4x0.5x0.75}$$
$$= 2306 \text{ kN/m/rad}$$

A diagram with $G'_{req'd}$ = 2306 kN/m/rad should be provided.

Dimensions and necessary parameters (Test #12)

A PMDF bracing system with edge stiffened panel modification will be utilized. Try the PMDF system of Test #12 and check its stiffness by using SDI and ECCS stiffness equations.

Deck Profile: 1.21 mm thick fully fastened deck form, stiffener spacing = 2400 m, eccentricity = 7 cm, L = deck sheet span length = 2400 mm, a = overall diaphragm panel width = 2400 mm, 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 = 610 mm, t = thickness of sheet metal = 1.21 mm, s = girth of corrugation per rib = 343 mm, d = corrugation pitch = 203 mm, h = deck sheet depth = 75 mm, n_e = number of edge connectors = 0, 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), $\sum x_e$ = (101.5 + 101.5 + 304.5 + 304.5) = 812.0 mm, $\sum(x_e)^2$ = (10302.3×2 + 92720.2×2) = 206045 mm^2, α_1 = end distribution factor = $\sum(x_e)$ / w = 812 / 610 = 1.33.

S_f = structural connector flexibility
= 0.0374/(t)0.5 = 0.034 mm/kN
S_s = side lap connector flexibility,
= 0.0863/(t)0.5 = 0.0785 mm/kN

Shear stiffness (SDI)

The effective shear stiffness of a diaphragm is defined as

$$G' = \frac{Et}{\left(2.6(s/d) + D_n + C\right)}$$

where G' = effective shear stiffness (kN/mm), E = modulus of elasticity = 200 (kN/mm^2), t = base metal thickness (mm), s = girth of corrugation per rib (mm), d = corrugation pitch (mm), D_n = warping constant, and C = slip coefficient.

Connector slip parameter

The 3rd edition of the SDI Design Manual (Lutrell 2004) presents a simplified equation 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

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

$$\text{where,}\quad C = \left(\frac{2x200x1.21x2740x0.034}{2400}\right)\quad [\,],$$

$$[\,] = \left(\left(\frac{4-1}{2x1.33 + 0 + \dfrac{2x5x0.034}{0.0785}}\right) + \left(\frac{1}{2x1.33+0+4}\right)\right),$$

and C = 11.39

Warping constant, D_n

The warping constant, D_n, is defined in SDI Design Manual (Lutrell 2004)

$$D_n = \frac{D}{12L},$$

where D = warping constant depending on fastener arrangement.

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. Warping constant, D_n, is calculated here as explained in Appendix-IV of SDI Design Manual (Lutrell 2004)

$WT = 4f^2(f+w) = 17\text{E}06\ \text{mm}^3$,
$WB = 16e^2(2e+w) = 1.27\text{E}06\ \text{mm}^3$,
$PW = 1/(t)^{1.5} = 0.751\ \text{mm}^{-1.5}$
$A = 2e/f = 0.357$,
$D1 = h^2(2w+3f)/3 = 1{,}076{,}250\ \text{mm}^3$,
$D2 = D1/2 = 538{,}125\ \text{mm}^3$,
$V = 2(e+w)+f = 344\ \text{mm}$,
$D3 = (h^2/12d^2)((V)(4e^2\text{-}2ef+f^2)+d^2(3f+2w)) = 328{,}148\ \text{mm}^3$,
$C1 = 1/(D3\text{-}D2/2) = 0.000017\ \text{mm}^{-3}$,
$D4[1] = (24f/C1)(C1/WT)^{0.25} = 198{,}271\ \text{mm}^{5/2}$,
$G4[1] = D4[1] = 198{,}271\ \text{mm}^{5/2}$,
$DW[1] = (G4[1])(f/d)(PW) = 102{,}734\ \text{mm}$,
$D_n = D/12L = DW[1]/L = 102{,}734/2100 = 48.9$

Shear stiffness calculation

G'_{open}
$= (200\text{E}06\times0.00121)/[2(1+0.3)(344/203)+48.9+11.39)]$
$= 3739$ kN/m (open end deck)
$\phi G'_{prov} = 0.65\times3739 = 2430\ \text{kN/m} > G'_{req'd} = 2306\ \text{kN/m}$ **OK**

Shear stiffness (ECCS)

The shear flexibility of a diaphragm is defined as

$$c = c_1 + c_2 + c_3 + c_4$$

Flexibility component due to shear deformation in the sheet (c_1)

$$c_1 = \frac{2a(1+v)[1+(2h/d)]}{Etb} = \frac{2x2400^{\text{mm}}(1+0.3)[1+(2x75^{\text{mm}}/203^{\text{mm}})]}{200^{\text{kN/mm}^2}x1.21^{\text{mm}}x2400^{\text{mm}}} = 0.0187\ \text{mm/kN}$$

Flexibility component due to distortion of the sheeting profile (c_2)

$$c_2 = \frac{ad^{2.5}\alpha_1\alpha_4 K}{Et^{2.5}b^2} = \frac{2400^{\text{mm}}x(203^{\text{mm}})^{2.5}x1.0x1.0x0.534}{200^{\text{kN/mm}^2}x(1.21^{\text{mm}})^{2.5}x2400^{\text{mm}}} = 0.406\ \text{mm/kN}$$

Flexibility component due to movement at the sheet to perpendicular member fasteners (c_3)

$$c_3 = \frac{2as_p p}{b^2} = \frac{2x2400^{\text{mm}}x0.034^{\text{mm/kN}}x203^{\text{mm}}}{(2400^{\text{mm}})^2} = 0.0058\ \text{mm/kN}$$

Flexibility component due to movement in the seams (c_4)

$$c_4 = \frac{2s_s s_p(n_{sh}-1)}{2n_s s_p + \beta_1 n_p s_s} = \frac{2x0.034^{\text{mm/kN}}x0.0784^{\text{mm/kN}}(4-1)}{2x5x0.0784^{\text{mm/kN}}+0.0} = 0.0471^{\text{mm/kN}}$$

The total flexibility of the diaphragm is

$$c = c_1 + c_2 + c_3 + c_4 = 0.0214^{\text{mm/kN}} + 0.53^{\text{mm/kN}} + 0.0075^{\text{mm/kN}} + 0.0471^{\text{mm/kN}} = 0.477^{\text{mm/kN}}$$

The stiffness of the diaphragm is

$$G'_{open} = \frac{1}{c} = \frac{1}{0.477^{\text{mm/kN}}} = 2096^{\text{kN/mm}}$$

Use the stiffness obtained from the SDI expression as recommended.

Brace strength requirement

Use Eqs. (14) and (15) to obtain reasonable estimates of maximum end and sidelap fastener brace forces, respectively. These expressions were developed for a twin-beam system. For a system with multiple beams, these expressions can be modified by the reduction factor $N_g = 0.5\times8/(8\text{-}1) = 0.57$ (Egilmez *et al.* 2016b). $G'_{prov} = 3739$ kN/m, is slightly higher than $G'_{req'd} = 2430$ kN/m. Therefore

$$C_r = \frac{3}{4} + \frac{1}{4}\left(\frac{G'_{req'd}}{G'_{prov}}\right)^2 = 0.86$$

$$F_{br-e} = N_g C_r s_e k_e w_e \frac{M_u L}{d^2} = 0.57x0.85x0.0003x1.0x1.0\frac{700\times15}{0.75^2} = 2.71\ \text{kN}$$

$$F_{br-s} = N_g C_r s_s k_s w_s \frac{M_u L}{d^2} = 0.57x0.85x0.00025x1.0x1.0\frac{700\times15}{0.75^2} = 2.26\ \text{kN}$$

The shear strength of a deck sheet to structural member fastener connection with mechanical fasteners, such as No. 12 and 14 screws, is given by Luttrell (2004). Use $\phi = 0.65$, F_y = yield strength of sheet metal = 345 MPa; and t = sheet thickness = 1.21 mm:

The shear strength of a sheet to sheet fastener connection at sidelaps is given by Luttrell (2004). Use ϕ = 0.65, d = diameter of the screw = 12 mm; and t = sheet thickness = 1.21 mm

$$\begin{aligned}\phi Q_f &= \frac{F_y t}{31.5}\left(1-\frac{F_y}{1380}\right)\\ &= 0.65\frac{345x1.21}{31.5}\left(1-\frac{345}{1380}\right) \qquad \textbf{OK}\\ &= 6.46\,\mathrm{kN}\rangle(F_{br-e}) = 2.71\,\mathrm{kN}\ (0.5\ kip)\end{aligned}$$

Local and global buckling capacity of the diaphragm should also be checked.