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Stiffness and strength of shear diaphragms used for stability bracing of slender beams

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Abstract
Light gage metal decking is often used in structures as concrete deck formwork, roof cladding or siding. In the steel building and bridge industries, decking acts like a shear diaphragm and provides continuous lateral bracing to the top flange of non-composite beams and girders that they are attached to. The building industry has long relied on the in-plane stiffness and strength of metal decking to brace steel beams during construction. Although the current AASHTO LRFD specifications do not allow bridge deck forms to be relied upon as a bracing source for steel bridge I-girders, recent studies have demonstrated that deck forms can significantly increase the buckling capacity of bridge girders by providing a relatively simple modification to the connection. Shear diaphragm bracing of steel I-beams have been studied in the past. These studies mainly focused on beams with stocky webs. The purpose of the study outlined in this paper is to enhance the understanding of both the stiffness and strength of shear diaphragms used to brace slender steel I-beams. The parameters that are investigated include diaphragm stiffness, sheet thickness, number of side-lap fasteners, flange width, and web slenderness ratio. Beams with web slenderness ratios of 100 to 160 and span/depth ratios of 10, 15, and 20 are considered. A simple finite element analytical (FEA) model is utilized in the study. The results indicate that web slenderness ratio does not have a major effect on fastener forces and the strength behavior of shear diaphragms is dependent on the number of side-lap fasteners. The findings of the study will be used to develop strength and stiffness requirements for shear diaphragms used to brace slender steel beams.

1. Introduction
Metal deck forms are often used in the building and bridge industries as concrete deck formwork. Besides providing support to the concrete deck during construction, these forms act like a shear diaphragm and provide continuous lateral bracing to the beams that they are attached to against lateral torsional buckling. Although the building industry has long relied on metal decking for stability bracing of steel beams, the forms are generally not considered for bracing in the bridge industry. While deck forms used in the building industry are generally continuous over the

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beams, the ends of the forms used in the bridge industry are closed and they are conventionally fastened to support angles, which are eccentrically welded to beam top flanges. This eccentric connection dramatically reduces the stiffness of the deck forms, which is one of the most important properties of a bracing system. In an effort to increase the bracing capabilities of bridge metal deck forms experimental studies have been conducted in recent years (Egilmez et al., 2007 and 2012). These studies have demonstrated that metal deck forms can also significantly increase the buckling capacity of girders by providing a relatively simple modification to the connection. The results of these studies were implemented in two bridges in Houston, TX, where metal deck forms were utilized as construction bracing (Helwig and Herman, 2010). Since beams utilized in the bridge industry are generally slender, there is a need to investigate the behavior of shear diaphragms used to brace slender beams. Although the buckling behavior of diaphragm braced beams have been studied in the past (Helwig and Frank, 1999), the authors are not aware of a significant study that investigate the stiffness and strength behavior of shear diaphragms used to brace slender I-beams.

The general philosophy for the design of stability bracing systems is to enable the braced element to support the design load while controlling deformations. Winter (1960) showed that in order to achieve this, an adequate bracing system must have sufficient stiffness and strength. Shear diaphragms possess a substantial amount of stiffness and strength in the plane of the diaphragm. The buckling capacities of beams braced by a shear diaphragm have been extensively studied in the past (Errera and Apparao, 1976, Nethercot and Trahair, 1975, Helwig and Frank, 1999). The most significant work on stiffness and strength requirements of shear diaphragms were conducted by Helwig and Yura (2008a and 2008b). Helwig and Yura focused mainly on stocky steel beams with web slenderness ratio less than 60 and proposed stiffness and strength requirements for shear diaphragms used for stability bracing of such beams.

This paper presents results from an ongoing research investigation that aims to enhance the understanding of the bracing behavior of shear diaphragms used to brace slender I-beams and to develop stiffness and strength requirements. Past studies (Luttrell 1981, Davies and Bryans, 1982) on strength and stiffness of shear diaphragms subjected to lateral loads showed that the shear strength of a diaphragm is generally controlled by either the shear strength of longitudinal edge connections (sheet to structural member connections along the edges) or shear strength at interior connections between panels (sheet to sheet connections along side-laps). Hence, a thorough investigation on the stiffness and strength of shear diaphragms used to brace slender steel I-beams should address the forces that develop at both edge and side-lap fasteners. In order to achieve this, a simple finite element analytical (FEA) model, originally developed by Davies and Bryan (1982) to investigate fastener forces resulting from lateral loads applied to building frames, was utilized in this study. Edge and side-lap fasteners were modeled separately; enabling the fastener forces to be directly calculated. The parameters that were investigated include diaphragm stiffness, sheet thickness, number of side-lap fasteners, flange width, and web slenderness ratio. Beams with web slenderness ratios of 100 to 160 and span/depth ratios of 10, 15, and 20 were considered.

Background information is presented in the following section, followed by a description of the finite element model, its verification, and overview of the study. Results from the parametrical study are presented next. The final section provides a summary and conclusions.

2. Background and Previous Work

The property of shear diaphragms that is of interest for bracing purposes is the shear rigidity, which is denoted by the variable Q that has units of force per unit radian (kN/rad or kip/rad). Q is calculated as the product of the effective shear modulus, G', and the tributary width of a diaphragm bracing a single beam, s_d. These expressions are given by:

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

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

where: n = number of girders in the system, and s_g = spacing between girders. The traditional definition of the shear modulus would be the shear stress divided by the shear strain. However, since the shear stress versus strain relationship of the corrugated sheeting is generally not a linear function of the material thickness (Lutrell, 1981), an "effective shear stress" is utilized, which is not dependent on metal thickness.

There have been a number of previous research investigations on the bracing behavior of shear diaphragms. Extensive research on the buckling behavior of systems with shear diaphragm bracing was conducted during the 1960's and 1970's (Errera and Apparao 1976 at Cornell; Nethercot and Trahair 1975 in Australia and England). These studies resulted in a relatively simple design expression for the buckling behavior of diaphragm-braced beams with uniform moment loading:

$$M_{cr} = M_g + 2Qe \tag{3}$$

where: M_{cr} is the buckling capacity of the diaphragm braced beam, M_g is the capacity of the beam with no bracing, Q is the shear rigidity of the deck, and e is the distance from center of gravity of the beam to plane of shear diaphragm.

Helwig and Frank (1999) presented finite element results that demonstrated the effects of moment gradient and load height on the bracing behavior of shear diaphragms used to brace slender beams. They modified the simplified uniform moment solution from the Cornell and Australian studies to be applicable for general loading conditions and produced the following expression to approximate the ideal stiffness requirements:

$$M_{cr} = C_b^* M_g + mQd \tag{4}$$

where: M_{cr}, M_g, and Q have been defined in Eq. (3), C_b^* = moment gradient factor that considers load height effects; d = depth of the beam; and m = factor that depends on the type of loading, web slenderness of the girder, and whether there are intermediate cross frames along the girder or not.

The expressions given in Eqs. (3) and (4) are applicable to a perfectly straight girder. Therefore, using such an equation to solve for the deck stiffness for a given moment would be analogous to the ideal stiffness requirement. The brace stiffness required for a structural member to reach a specific load level or buckling capacity is often called the "ideal stiffness". Helwig and Yura (2008a and 2008b) performed finite element analytical (FEA) studies on shear diaphragm braced beams with web slenderness ratios less than 60. They conducted eigenvalue and large displacement analysis to determine "m" values and stiffness requirements for stocky beams. In their study, the "ideal stiffness" of shear diaphragms was selected as the diaphragm stiffness from an eigenvalue buckling analysis that produced a maximum beam bending stress close to 345 MPa (50 ksi). Eqs. (4) and (1) can be used to solve for the ideal shear stiffness that is required to reach a given design moment, M_u. For a design moment that produces a maximum bending stress of 345 MPa (50 ksi) Helwig and Yura (2008b) found that providing four times the ideal stiffness could effectively control deformations and brace forces. Providing four times the ideal stiffness value would therefore result in the following expression:

$$G'_{req} = 4G'_i = 4\frac{Q_i}{s_d} = 4\frac{\left(M_u - C_b^* M_g\right)}{mds_d} \tag{5}$$

where: $M_u = M_{cr}$ in Eq. (4) = design moment, G'_{ideal} = ideal shear stiffness of the diaphragm that is required to reach M_u, which is calculated from Eq. (4), G'_{req} = required shear stiffness of the diaphragm to control deformations and brace forces, and C_b^*, M_g, m, d and s_d have been defined in Eqs. (2), (3), and (4). The stiffness requirement for the shear diaphragm given in Eq. (4) is based on an analysis of beams with an initial twist, $\theta_0 = L_b/(500d)$, where d = section depth and L_b is the spacing between cross-frames. Helwig and Yura also investigated strength requirements for shear diaphragm bracing of stocky beams and proposed the following equation:

$$M'_{req'd} = 0.0011\frac{(M_u L)}{d^2} \tag{6}$$

where: M'_{br} = brace moment per unit length, L = total beam span and d = beam depth. The brace moment given in Eq. (6) represents the warping restraint provided to the top flange of the girder per unit length of the span.

3. Finite element Analytical (FEA) Model

3.1 FEA Model of Beams

The three-dimensional finite element program ANSYS (2007) was used to perform parametric studies on the behavior of steel I-beams braced with shear diaphragms. The FEA model consists of a twin beam system braced by a shear diaphragm. All of the elements used in the FEA model possessed linear elastic material properties. The beams were meshed with 8-node shell elements. Two elements were used to model the flanges and four elements were used for the webs. The aspect ratio of the elements was close to unity. The beams were simply supported with lateral movement prevented at the top and bottom flanges at the supports.

Initial imperfections play an important role in the magnitude of brace forces that develop in bracing members. Wang and Helwig (2005) showed that the brace forces are directly proportional to the magnitude of initial imperfections for beams braced by cross frames or diaphragms. The same shape and magnitude recommended by Wang and Helwig (2005) and

Helwig and Yura (2008a) for initial imperfections is adopted in this study and are shown in Fig. 1. The corresponding approximate initial twist is expressed in the following expression:

$$\theta_o = \frac{L_b}{500d} \tag{7}$$

where: L_b = unbraced length of the beam and d = depth of the beam.

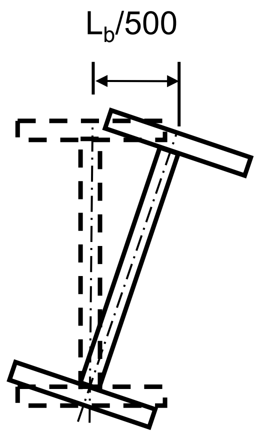

Figure 1: The shape and magnitude of imperfections used in the study

3.2 FEA Model of Shear Diaphragm

The FEA model used for shear diaphragms was adapted from a study by Davies and Bryan (1982). In their study, Davies and Bryan (1982) simulated the shear stiffness of diaphragms by a series of bars forming a truss as illustrated in Fig. 2. Each small truss shown in Fig. 2 consists of four transverse and three diagonal truss elements and represents a single deck sheet profile. The deck sheet profile modeled in this study is a typical deck sheet with three corrugations commonly used in both building and bridge applications. The transverse truss elements are located at every trough and span between the centerline of beam top flanges. This type of a representation of deck sheets enables each deck to structural member (twin beams in this study) fastener to be modeled by dimensionless spring elements and be placed at the ends of each transverse truss element. The transverse truss elements were connected to the beam top flange mid-nodes through these dimensionless spring elements. The number of transverse truss elements can be changed depending on the number of fasteners used to connect the deck sheet to structural member. These truss elements are 3-D uniaxial tension-compression spar elements. The axial stiffness of the transverse elements is taken sufficiently high for their axial strain to be neglected. Hence, the shear stiffness of the deck sheets depends only on the properties of the diagonal elements. In order to determine the required area of the diagonal truss elements that corresponds to a certain shear rigidity, an FEA model of a shear test frame was utilized.

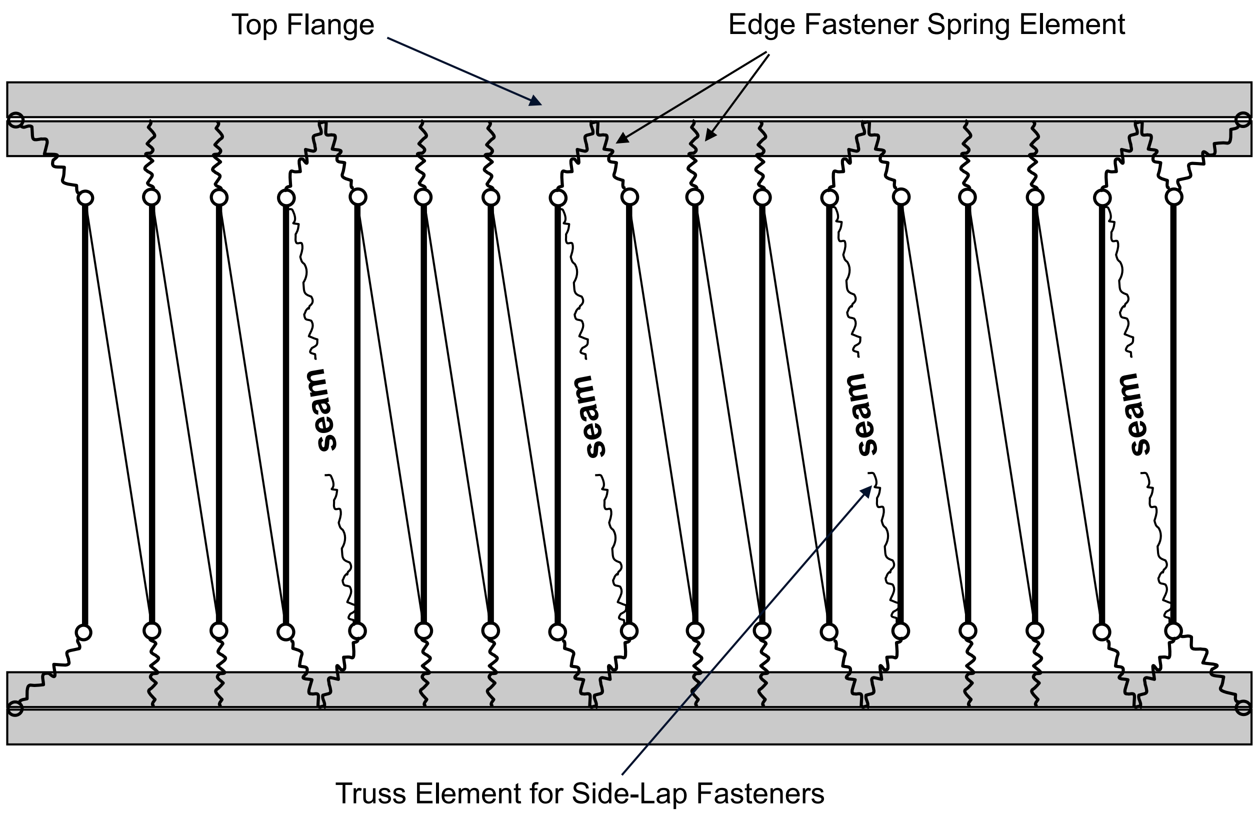

Figure 2: Finite element analytical model of shear diaphragms

3.3 FEA Model of Fasteners

In bridge forming systems deck sheets are generally fastened to supporting members along the edges at every trough and to each other at side-lap locations by mechanical fasteners. Conventional mechanical fasteners for deck sheets are generally 19 mm (¾ in) long TEKS screws with a 6.3 mm (¼ in) diameter. The fasteners that connect deck sheets to supporting members were modeled by dimensionless spring elements that possess equal stiffness in two orthogonal directions, but no rotational stiffness. These spring elements were located at the centerline of beam top flanges and connected to the mid-node of beam top flanges and the ends of the transverse truss elements as explained above. Although the dimensionless spring elements are shown to have finite length in Fig. 2, this representation is merely for illustration purposes. At side-lap locations separate spring elements were used to connect each transverse truss end to the same mid-node of beam top flange, as illustrated in Fig. 2. The transverse and lateral stiffnesses of these dimensionless spring elements were assumed to be the same as the stiffness of No. 12 and No. 14 Buildex TEKS screws used to develop the SDI Design Manual (Lutrell 2004) equations.

Side-lap fasteners were modeled by a transverse truss element that connects opposite edges of adjacent small trusses as shown in Fig. 2. The stiffness of the side-lap transverse truss elements represented the total stiffness of the number of fasteners used at a side-lap. TxDOT PMDF standards (TxDOT 2004) require a maximum center-to-center spacing of 450 mm (18-in) at side-laps. The number of fasteners at side-lap locations considered in this study ranged from five to three. Five and three fasteners correspond to deck lengths of approximately 2740 mm (108 in) and 1800 mm (71 in), respectively. These deck lengths are representative of practical deck lengths utilized in the bridge industry. The stiffness of one side-lap fastener was assumed to be the same as recommended by the SDI Design Manual (Lutrell 2004). Three different deck thicknesses (0.91, 1.22, and 1.52 mm (20, 18, and 16 ga.)) were considered in this study in order to investigate the effect of sheet thickness on fastener forces.

3.4 Verification of FEA Model

The FEA simulation was verified by comparing the buckling behavior of a 14.63 m (48 ft) twin-girder system braced by permanent metal deck forms (PMDF) with that of the FEA simulation. The test used in the verification study was a full-scale twin-girder buckling test conducted at the structural engineering laboratory of University of Houston. The girders were US wide flange beams, W760×134 (US: W30×90), with a depth of 753 mm and web thickness of 15.5 mm. The top flanges of the girders were flame cut from the original 264.2 mm (10.4 in) width to 158.8 mm (6.25 in) to produce a singly symmetric section with $\rho=I_{yc}/I_y=0.18$; where I_{yc} and I_y are respective moment of inertias of the compression flange and the entire cross section about a vertical axis through the web. Point loads were applied to the girders through two gravity load simulators at third points. The deck form system utilized in the test set-up represented a forming system used in the bridge industry with a modified connection detail. The 1.22 mm (18 ga) thick PMDF sheets were 610 mm wide and were supported on cold-formed L76×51×3.3 mm (L3×2 10 ga) galvanized angles. The modified connection detail consisted of transverse stiffening angles that spanned between the top flanges of the girders. The spacing of the stiffening angles was 4.88 m (16 ft). A full description of the test set-up is given by Egilmez et. al (2012). The effective shear stiffness of the deck form system utilized in the test was measured as 7184 kN/m-rad; which corresponded to a deck shear rigidity of 9842 kN/rad (Egilmez et. al, 2007). The 9842 kN/rad shear rigidity corresponded to approximately 5.4 times the ideal shear rigidity of the deck system for a stress level of 210 MPa (30.43 ksi).

Fig. 3 shows the mid-span moment vs. mid-span total twist/initial twist behavior of the W760×134 (US: W30×90) girders braced by a shear diaphragm and FEA simulation. It can be seen from the figure that the FEA model predicted the behavior of the test beam very well in the elastic region. The rotations of the FEA model were approximately 4.7% higher that the rotations of the girders observed in the test. Since elastic materials were used in the FEA model, the inelastic behavior of the twin-girder system was not captured by the model.

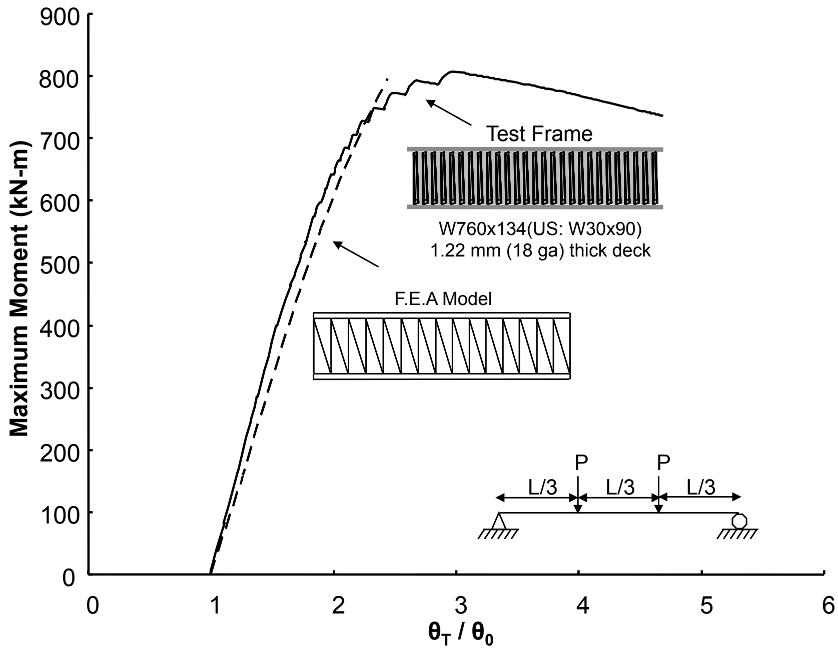

Figure 3: Normalized mid-span moment vs. twist behavior of test specimen and FEA simulation

4. Overview of Study

The beam sections that were considered in this study consisted of four doubly symmetric sections. The web slenderness ratios (WSR) of the sections were 100 and 160. The depths of sections were 1464 mm. Flange widths of 220 mm and 300 mm were considered. Flange slenderness ratios of all sections were 6. Two web slenderness ratios were considered (100 and 160) by changing the thickness of the webs. Fig. 4 shows the dimensions of the cross sections. Sections with web slenderness of 100 and flange widths of 220 mm and 300 mm are referred as Slender-100 #1 and #2; whereas sections with web slenderness of 160 and flange widths of 220 mm and 300 mm are referred as Slender-160 #1 and #2, respectively. Span-to-depth ratios (*L/d*) of 10, 15, and 20 were considered. Although an *L/d* ratio of 20 is not practical for a simply supported beam with a depth of 1464 mm, it allows a comparison of brace forces for a range of *L/d* ratios.

The only loading considered was uniformly distributed loading applied at top flange. Uniform distributed loading is representative of loading from poured concrete slab. Loading applied at mid-height was not considered since it is less critical as compared to top flange loading. The beams had transverse stiffeners spaced every 1830 mm (72 in), which is approximately 1.25 times the girder depth.

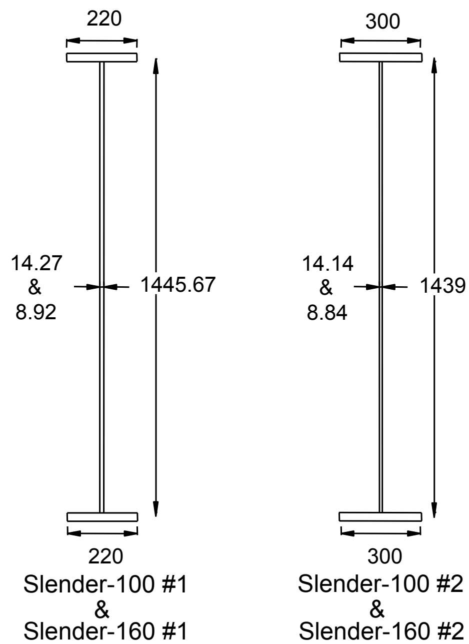

Figure 4: Sections used in the study

The design load level for the sections investigated in this study was taken as the moment corresponding to an in-plane bending stress equal to 210 MPa (30.43 ksi) at the outer fiber of the section. The stress level of 210 MPa (30.43 ksi) is somewhat arbitrary; however, it represents a reasonable level of in-plane bending stresses expected during construction. As mentioned in the background section, the property of shear diaphragms that is of interest for bracing purposes is the shear rigidity, which is denoted by the variable Q. For each section investigated in this study, the ideal shear rigidity (Q_{ideal}) was calculated by using Eq. (4). M_{cr} in Eq. (4) was taken as the moment corresponding to an in-plane bending stress of 210 MPa (30.43 ksi) at the outer fiber of the section as explained above. Large displacement analysis were then conducted by using two, three, four and six times the ideal shear rigidity, which reflects the effects of geometrical imperfections, to observe deformations and brace forces.

5. Results

Although the general twist behavior of all of the sections have been studied, mid-span twist vs. mid-span total twist over initial twist behavior will be shown for only one section. The maximum twists observed in all of the sections at the design load level will then be presented in tabular format. Fig. 5 shows the normalized applied moment versus normalized mid-span twist behavior of Slender-100 #2 and Slender-160 #2 sections with L/d ratios of 15. The thickness of the deck and number of side-lap fasteners along the seams were taken as 1.52 mm (16 ga) and five, respectively. Results are shown for diaphragm rigidities of two, three, and four times the ideal value. The normalized beam moment shown on the vertical axis is the applied moment divided by the maximum moment that corresponds to an in-plane bending stress of 210 MPa (30.43 ksi) at the extreme fiber. On the horizontal axis, the resulting twist at mid-span normalized by the initial twist is shown. It is observed from Fig. 5 that for diaphragm rigidities of two, three and four times the ideal value, the normalized twists at the design load level were 3.17, 2.63, and 2.42, and 2.90, 2.45, and 2.27 for Slender-160 #2 and Slender-100 #2 sections respectively.

Flange widths and beam depths of these two sections are identical. The reason for the difference in rotation values between the two sections is probably the web slenderness ratios. Increasing the web slenderness ratio to 160 from 100, increased the rotations about 9.31%, 7.34, and 6.61% for diaphragm rigidities of two, three, and four times the ideal value, respectively.

Table 1 lists the normalized mid-span twist values at the design load level for the four sections with *L/d* ratios of 10, 15, and 20. Results for diaphragm rigidities of two, three, and four times the ideal rigidity are presented. Deck thickness and number of side-lap fasteners along the seams were taken as 1.52 mm (16 ga) and five, respectively. It is observed in Table 1 that for a diaphragm rigidity of two times the ideal value, the total twists observed in all four sections were between 5.02 to 2.54 times the initial twits. Increasing the diaphragm rigidity to three times the ideal value dropped the total twists of the sections by approximately 20% to 16%. Further increasing the diaphragm rigidity to four times the ideal value dropped the total twists of the sections an additional 9% to 7% as compared to the twists observed for a diaphragm rigidity of three times the ideal value. For *L/d* ratios of 10 and 15, providing a shear rigidity of three times the ideal value resulted in total twists less than or equal to 3.01. For *L/d* ratios of 20, higher rotations were obtained. However, for a beam with a depth of 1464 mm, an *L/d* ratio of 20 is not practical, as mentioned earlier. It is also observed in Table 1 that the normalized twist values of sections with 220 mm flange width are approximately 10% higher than those of sections with 300 mm flange width for the same web slenderness ratios.

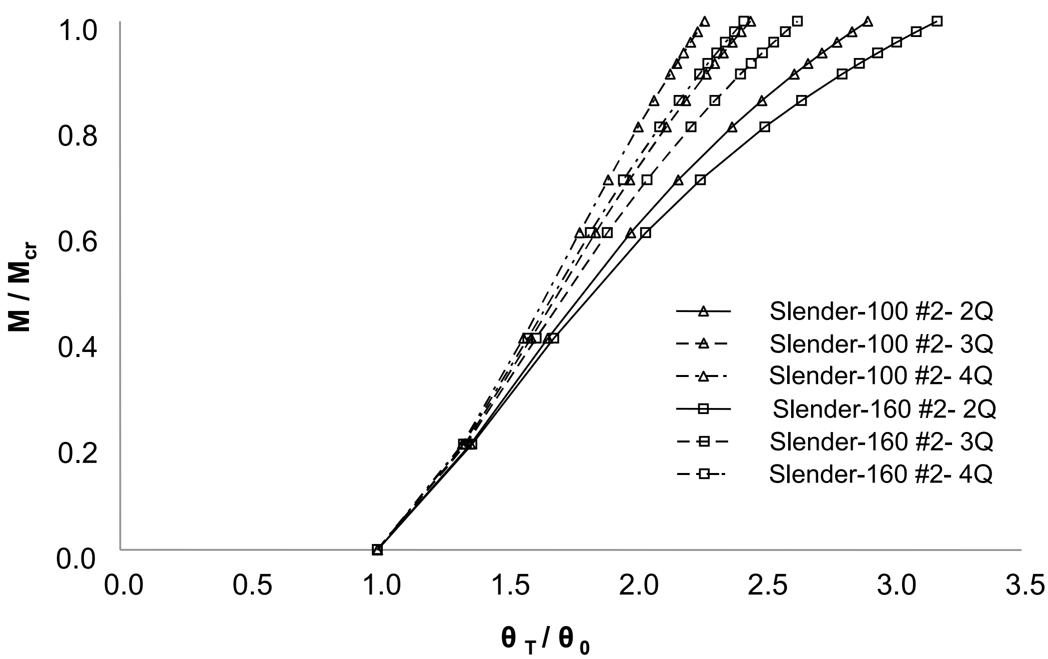

Figure 5: Normalized mid-span moment vs. twist behavior of Slender-100 #2 and Slender-160 #2 sections with *L/d* ratio of 15

Table 1: Normalized mid-span twist values at the design load level

Section Designation	Section Properties	L/d	θ_T/θ_o 2Q ideal	3Q ideal	4Q ideal
SLENDER-100 #1	b_f [1]= 220 mm	10	2.85	2.41	2.23
	d[2] = 1464 mm	15	3.32	2.75	2.53
	WSR[3] = 100	20	4.30	3.38	3.07
SLENDER-100 #2	b_f = 300 mm	10	2.54	2.16	2.00
	d = 1464 mm	15	2.90	2.45	2.27
	WSR = 100	20	3.63	2.95	2.70
SLENDER-160 #1	b_f = 220 mm	10	3.33	2.74	2.52
	d = 1464 mm	15	3.71	3.01	2.75
	WSR = 160	20	5.02	3.76	3.36
SLENDER-160 #2	b_f = 300 mm	10	2.96	2.46	2.26
	d = 1464 mm	15	3.17	2.63	2.42
	WSR = 160	20	3.98	3.16	2.87

1. Flange width, 2. Section depth, 3. Web Slenderness ratio

The required brace stiffness for a particular section should also be large enough to control brace forces. Therefore, the maximum brace forces that develop along the length of the beams need to be identified. Brace forces that develop in a single deck sheet are shown in Fig. 6. As seen in Fig. 6 both transverse and longitudinal forces develop at edge fasteners; whereas at side-lap fasteners only transverse shear forces develop. It was observed in the analyses that the resultant edge fastener forces were much higher than the forces that develop at side-lap fasteners. Hence, the forces that develop in the side-lap fasteners are not discussed in this paper.

The distribution of the resultant edge fastener forces at the design load level along the length of the beam is shown in Fig. 7 for Slender-160 #2 section with *L/d* ratios of 10, 15, and 20 for a diaphragm rigidity of three times the ideal value, five side-lap fasteners along the seams, and a deck thickness of 1.52 mm (16 ga). The resultant forces shown in the figure are calculated by taking the square root of the summation of squares of the transverse and longitudinal force components that develop in each edge fastener. The horizontal axis covers only half of the beams, since brace forces are symmetrical with respect to mid-span. For *L/d* ratios of 10, 15, and 20 there are a total of 12, 18, and 24 deck sheets along half of the beam lengths, respectively. Each line shown in the figure belongs to a single deck sheet and each cross in a single line shows the force in one out of four edge fasteners in a single sheet. It is observed in Fig. 7 that the brace forces increase as *L/d* ratio increases and the forces maximize around quarter span for all *L/d* ratios.

Also shown in the figure is the edge fastener force for *L/d* ratio of 10 calculated by Eq. (6) that was recommended by Helwig and Yura (2008b). Helwig and Yura (2008) recommended Eq. (6) for beams with web slenderness ratios less than 60. However, results from Table 2, which is discussed below, indicate that fastener forces are not dramatically affected by the web slenderness ratio. Edge fastener force calculated from Eq. (6) overestimates the force calculated from the analyses by 1.94 times. For *L/d* ratio of 15, the overestimation factor increases to 2.70.

Analyses were also conducted with deck thicknesses of 1.22 mm (18 ga) and 0.91 mm (20 ga) thick decks. The shear rigidities of these deck systems were kept the same as the shear rigidity of the deck system with a thickness of 1.52 mm (16 ga). Decreasing the deck thickness did not have an effect on the twist behavior of the sections, since diaphragm rigidities were kept the same, and resulted in a negligible decrease in fastener forces. Hence, the results shown in this paper belong to deck systems with a thickness of 1.52 mm (16 ga).

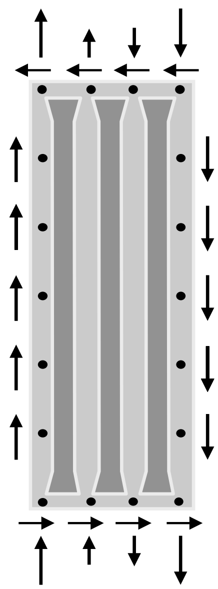

Figure 6: Fastener forces at a single deck sheet

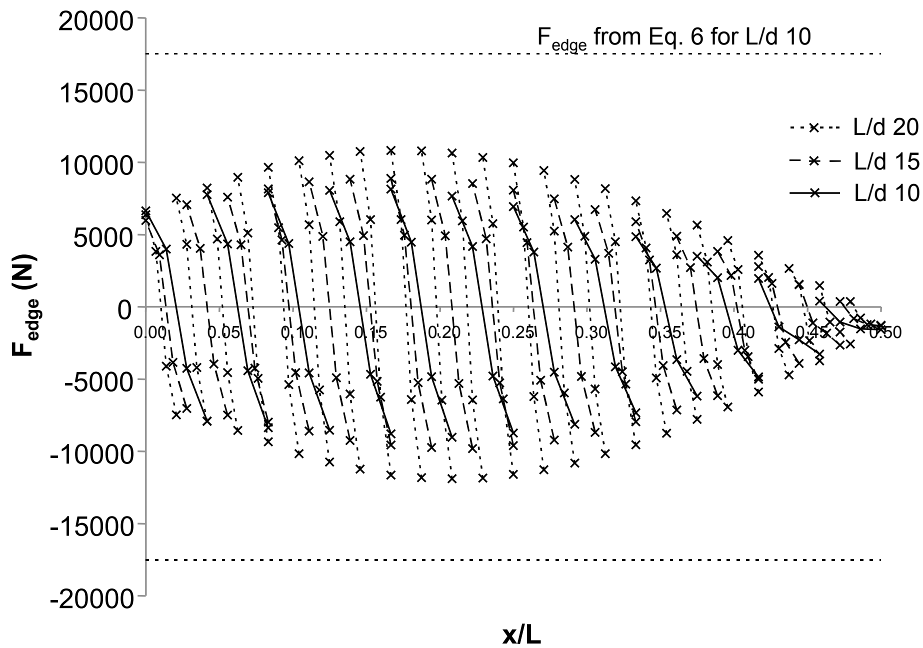

Figure 7: Edge fastener force distribution along half the beam length for Slender-160 #2 section for *L/d* ratios of 10, 15, and 20

Fig. 8 shows the normalized applied moment versus normalized maximum fastener forces for Slender-160 #2 section with *L/d* ratio of 15, five side-lap fasteners along the seams, and a deck thickness of 1.52 mm (16 ga). Results for diaphragm rigidities of two, three, and four times the ideal value are illustrated. The normalized maximum edge fastener force shown on the horizontal axis is calculated by dividing the maximum fastener force along the length of the beam (obtained from Fig. 7) to the capacity of the deck to structural member fastener connection given by Luttrell (2004) for a deck thickness of 1.52 mm (16 ga). Although comparing the forces in the edge fasteners with the capacity of the deck to structural member fastener connection with a 1.52 mm (16 ga) thick deck sheet is arbitrary, it gives a reasonable indication of how large the fastener forces are. It is seen from Fig. 8 that for diaphragm rigidities of two, three, and four times the ideal value, the maximum fastener forces were 92% and 79%, and 74%, respectively, of the capacity of the edge fastener connection.

Table 2 lists the normalized maximum edge fastener force (similarly calculated as explained above) observed at the design load level for the four sections with *L/d* ratios of 10, 15, and 20. Results for diaphragm rigidities of two, three, and four times the ideal rigidity are presented. Deck thickness and number of side-lap fasteners along the seams were taken as 1.52 mm (16 ga) and five, respectively. It is observed in Table 2 that increasing the flange width while keeping the web slenderness ratio the same increased the brace forces. For example, for Slender-100 #1 and #2 sections the maximum normalized edge fastener forces were 0.67 and 0.84, respectively, for *L/d* of 15 and a diaphragm rigidity of three times the ideal value. Similar behavior was observed for Slender-160 sections. It can also be seen in Table 2 that for each section, the maximum normalized edge fastener forces ranged between 0.63 and 1.23 for a diaphragm rigidity of two times the ideal value. Increasing the diaphragm rigidity to three times the ideal value dropped the maximum normalized edge fastener forces by approximately 16% to 19%. Further increasing the diaphragm rigidity to four times the ideal value dropped the maximum normalized edge fastener forces of the sections an additional 8% to 9% as compared to those for a diaphragm rigidity of three times the ideal value. Another interesting observation is that increasing the web slenderness ratio while keeping everything else the same did not have a major effect on the maximum normalized edge fastener forces. For example, for Slender-100 #2 and Slender-160 #2 sections with *L/d* ratios of 15 and a diaphragm rigidity of three times the ideal value, the maximum normalized edge fastener forces were 0.84 and 0.79, respectively.

The effect of deck span on the strength behavior of shear diaphragms used to brace slender beams was also investigated. Using a shorter deck span will result in using fewer side-lap fasteners. For example, using a 1350 mm (53 in) long diaphragm will require two side-lap fasteners instead of five side-lap fasteners required for a span of 2740 mm (108 in) (TxDOT PMDF standards, 2004). It was observed in the analyses that reducing the number of side-lap fasteners while keeping the deck shear rigidity the same resulted in increased edge fastener forces. The effect of number of side-lap fasteners on the magnitude of maximum edge fastener force is illustrated in Fig. 9.

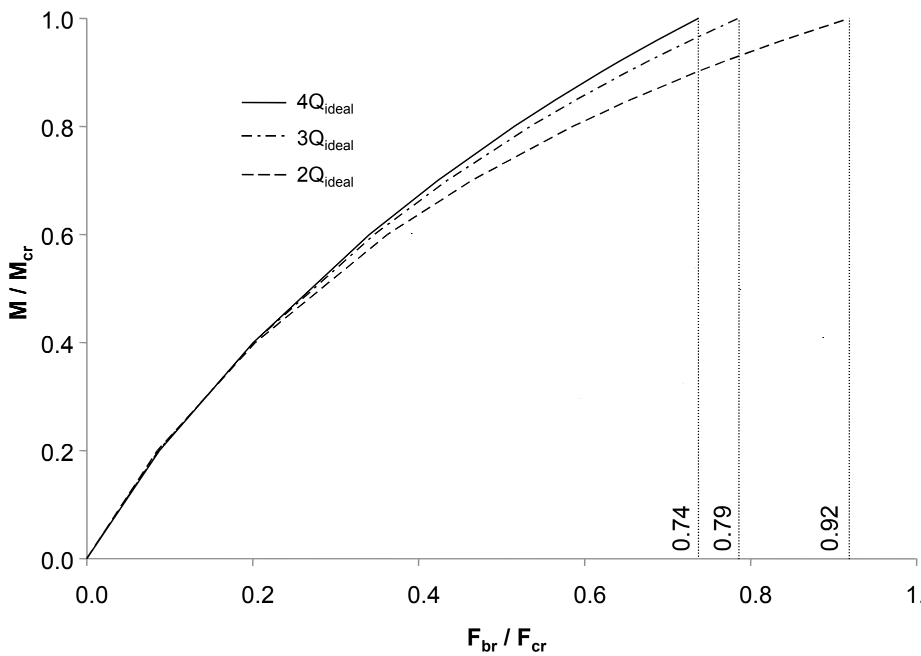

Figure 8: Normalized applied moment vs. normalized maximum edge fastener forces for Slender-160 #2 section with *L/d* ratio of 15

Table 2: Normalized maximum edge fastener force at the design load

Section Designation	Section Properties	L/d	F_{br}/F_{cap} 2Q$_{ideal}$	3Q$_{ideal}$	4Q$_{ideal}$
SLENDER-100 #1	b_f [1]= 220 mm	10	0.66	0.57	0.54
	d[2] = 1464 mm	15	0.79	0.67	0.63
	WSR[3] = 100	20	1.04	0.83	0.76
SLENDER-100 #2	b_f = 300 mm	10	0.81	0.72	0.68
	d = 1464 mm	15	0.96	0.84	0.79
	WSR = 100	20	1.23	1.02	0.95
SLENDER-160 #1	b_f = 220 mm	10	0.63	0.54	0.51
	d = 1464 mm	15	0.73	0.60	0.56
	WSR = 160	20	0.98	0.75	0.68
SLENDER-160 #2	b_f = 300 mm	10	0.83	0.72	0.68
	d = 1464 mm	15	0.92	0.79	0.74
	WSR = 160	20	1.17	0.95	0.88

1. Flange width, 2. Section depth, 3. Web Slenderness ratio

For Slender-100 #2 section with an *L/d* ratio of 10, deck thickness of 1.52 mm (16 ga), and diaphragm rigidity of four times the ideal value additional analyses were conducted for deck systems with three and four side-lap fasteners. As seen in Fig. 9, for a deck system with five side-lap fasteners, providing four times the ideal shear rigidity resulted in a brace force that was 68% of the capacity of an edge fastener connection with a 1.52 mm (16 ga) thick deck sheet. For deck systems with four and three side-lap fasteners, this value increased to 77% and 89%, respectively.

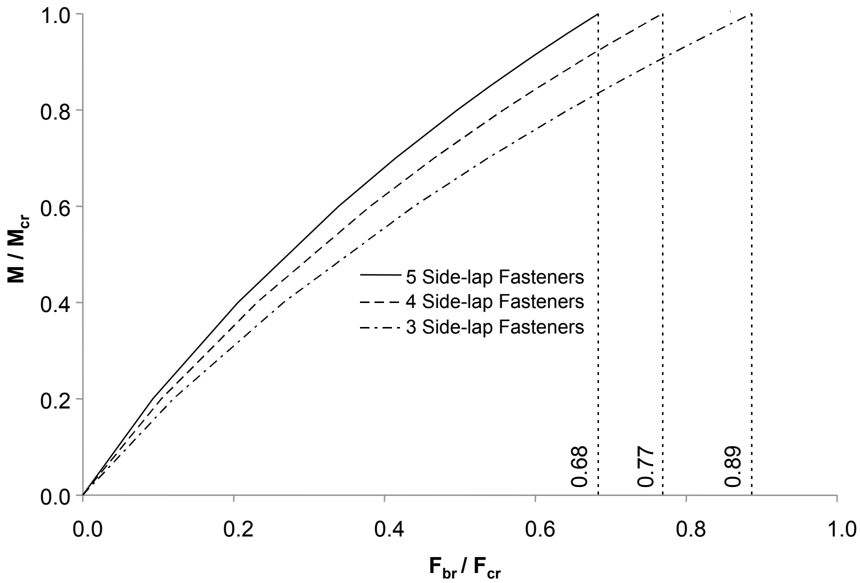

Figure 9: Effect of number of side-lap fasteners on edge fastener forces

6. Summary and Conclusions

A parametric study was conducted to enhance the understanding of stiffness and strength behavior of shear diaphragms used to brace slender steel I-beams. A finite element analytical model was used that enabled each edge fastener to be simulated separately. Fasteners at each side-lap seam were also modeled by single truss elements with a stiffness equal to the stiffness of the total number of fasteners at a side-lap seam. The parameters that were investigated included diaphragm stiffness, sheet thickness, number of side-lap fasteners, flange width, and web slenderness ratio. Only uniformly distributed loading was considered. The design load level was taken as the moment corresponding to an in-plane bending stress equal to 210 MPa (30.43 ksi) at the outer fiber of the sections. Hence, the ideal shear rigidity of a diaphragm system corresponded to a diaphragm rigidity that allowed the beams to buckle at the above mentioned design load level.

The results indicate that the number of side-lap fasteners along a seam appears to affect the magnitude of edge fastener forces. Utilizing a shorter deck that requires three side-lap fasteners instead of a longer deck that requires five fasteners increases the edge fastener forces by approximately 30%. Changing the deck thickness changes the stiffness of the fastener connections. However, based on the results from this study, utilizing different deck thickness does not have a major effect on fastener forces. The results also indicate that utilizing a diaphragm rigidity of three times the ideal rigidity is sufficient to control the deformations and fastener forces. Two web slenderness ratios were considered. It was observed that increasing the web slenderness ratio increased the rotations by about 5%-10% and slightly decreased fastener forces. However, increasing the flange width to 300 mm from 220 mm, decreased the rotations by approximately 10% and increased fastener forces by about 25%-35%.

Comparison of edge fastener force results with results obtained from recommended equations from literature revealed that the recommended equations could yield to overestimation of the forces by more than a factor of two. Although these recommendations are intended for stocky beams with web slenderness ratios less than 60, the comparison indicates that further studies are necessary for slender beams. Additional parametrical studies are currently underway that also take into account sections with higher depths and intermediate cross frames. Results from this study along with results from the ongoing investigation will be used to develop stiffness and strength requirements for shear diaphragms used to brace slender beams.

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