STIFFNESS OF CONCRETE-FILLED STEEL DECK DIAPHRAGMS

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Abstract

In structural analysis of building structures, the in-plane stiffness diaphragms is needed so that lateral loads will be properly distributed to elements of the lateral-force resisting system. In US building codes, diaphragm stiffness is used to determine whether a diaphragm can be assumed rigid or flexible and is also used in semi-rigid diaphragm analysis. For concrete-filled steel deck diaphragms, methods provided in AISI S310 (AISI, 2020) to calculate stiffness have relied on empirical formulas while past research by Porter and Easterling (1988) suggests that mechanical models and theoretical formulas can accurately capture stiffness.

Recently, eight cantilever diaphragm specimens were tested with variations in depth of concrete cover, deck depth, perimeter stud anchor configuration, concrete type (normal weight (NW) and lightweight (LW)), and the presence of either welded wire mesh or reinforcing steel. This report summarizes the results of this testing program as they relate to initial stiffness. The initial stiffness results of this testing program are used in conjunction with the results of a testing program performed Porter and Easterling (1988) to form a set of 25 specimens that are then used to validate a proposed prediction model for the initial stiffness of concrete-filled steel deck diaphragms.

The proposed prediction model is based on a theoretical framework proposed by Porter and Easterling (1988) which concluded that the initial stiffness of a concrete-filled steel deck diaphragm is a combination of 1) the diaphragm shear stiffness, 2) the bending stiffness of the concrete-filled steel deck diaphragm combined with the chords, and 3) the stiffness of the shear transfer connections between the concrete-filled steel deck diaphragm and the supporting steel frame. The proposed stiffness predictions using this approach resulted in an average ratio of predicted stiffness to measured stiffness equal to 0.95 with a standard deviation of 0.21. Based
on this comparison for 25 cantilever diaphragm specimens, it was deemed that the prediction model accurately represents the initial shear stiffness of concrete-filled steel deck diaphragms.

This report also includes two examples to illustrate of how the proposed prediction model can be used to calculate diaphragm deflections for two different diaphragm configurations. The results of these examples showed that for the cantilever diaphragm configuration, the deflection of the free end is mostly due to the shear deformation of the concrete-filled steel deck diaphragm or to the deformation of the shear transfer connection, depending on the spacing of headed stud anchors, with the bending deformations contributing the least to the total deflection. For the case of a simply supported diaphragm, the mid-span deflection was attributed primarily to bending deformations of the diaphragm (78% of total deflection), with shear deformations contributing to approximately 25% of the total deflection and the deformation of the shear transfer connections contributing less than 1% of the total deflection.
1 Introduction

The in-plane stiffness of diaphragms is important in the design of buildings to resist lateral loads because diaphragm stiffness plays a key role in distributing forces to elements of the lateral-force resisting system, such as braced frames and moment frames. Diaphragm stiffness is also relevant to serviceability requirements such as deflections, seismic separations between buildings, and deformation compatibility with the gravity load resisting system.

For concrete-filled steel deck diaphragms, two methods available to engineers for calculating diaphragm deflections are: 1) the empirical equation provided in AISI S310-20 (AISI, 2020) for shear stiffness combined with estimates of bending deformations, or 2) creating a structural analysis model where the diaphragm is modeled with elastic membrane elements. The two approaches produce different results for diaphragm deflections. The motivation for the current study is to help understand this apparent discrepancy in diaphragm stiffness and propose a method for predicting diaphragm deflections that is validated against a wide set of concrete-filled steel deck diaphragm tests.

A testing program was recently performed with eight cantilever diaphragm specimens, to understand the behavior of concrete-filled steel deck diaphragms with modern construction detailing. This report summarizes the results of this testing program as they relate to initial stiffness. The results of a prior research program conducted at Iowa State University (ISU) are also described (Porter and Greimann, 1980; Porter and Greimann, 1982; Neilson, 1984; Easterling, 1987; Easterling and Porter 1988; Porter and Easterling 1988; Easterling and Porter, 1994a; Easterling and Porter, 1994b; Prins, 1985). Between the two testing programs, 25 cantilever concrete-filled steel deck diaphragm specimens are identified for use in validating stiffness calculations.
A proposed prediction model is presented based on the work of Porter and Easterling (1988) in which the initial stiffness of a concrete-filled steel deck diaphragm is a combination of the 1) the shear stiffness of the diaphragm 2) the bending stiffness of the concrete-filled steel deck diaphragm in conjunction with the steel chords, and 3) the stiffness of the shear transfer connections between the concrete-filled steel deck diaphragm and the supporting steel frame. The proposed prediction model which is entirely mechanics-based and not calibrated to the data is validated against the experimental data set.

To illustrate how the different sources of stiffness are calculated using the proposed prediction model and to compare to the empirical equations in AISI S310, two examples are included in this report: Example 1 for a cantilever diaphragm specimen, and Example 2 for a realistic building diaphragm in an archetype building.
2 Literature Review

This section summarizes a past testing program used later in the report and the most relevant prediction models in the literature for the in-plane stiffness of concrete-filled steel deck diaphragms.

2.1 Previous Testing Program by Porter and Easterling (1988)

One of the most extensive testing programs investigating the performance of concrete-filled steel deck diaphragms was performed by Porter and Easterling (1988) in the 1980’s. A total of 32 cantilever diaphragm tests were performed, as reported in Porter and Easterling (1988). This experimental program is of particular interest because of the large number of tested specimens as well as the wide range of parameters that were varied and the similarities with the experimental program presented in Chapter 3 of this document. The experimental setup for this testing program is illustrated in Figure 2-1. It consisted of a cantilever configuration with W24x76 steel sections framing into steel plates which were in turn embedded into a reinforced concrete reaction block. The specimens were tested using a cyclic loading protocol introduced by two hydraulic actuators.
Different deck profiles were used, some of which do not represent common geometries used in present day construction. The perimeter fasteners consisted mostly of puddle welds, with some specimens including a combination of welds and headed stud anchors, and a few specimens including headed stud anchors as the sole method of perimeter fasteners. A summary of the specimens with a reported failure mode of diagonal tension cracking is provided in Table 2-1. Specimens 1 through 19 had in-plane dimensions of 15 ft. by 15 ft, while specimens 22 through 29 had in-plane dimensions of 12 ft. by 15 ft, similar to the specimens reported in Chapter 3 of this dissertation. Specimens 1, 2, 25, 26, and 29 included headed studs as perimeter fasteners, while the remaining specimens included welds as the perimeter fasteners. All specimens, with the exception of specimen 26, were constructed using normal weight concrete mix with compressive strengths ranging between 2500 psi and 6200 psi. Six of the specimens in this testing program introduced vertical loads in combination with lateral loads to investigate the
effect of gravity loads on diaphragm performance. One of the most important conclusions obtained from this experimental program was the identification of common limit states for concrete-filled steel deck diaphragms. While a shear interface between deck and concrete limit state was observed for some of the specimens, it was not deemed to be representative of realistic diaphragm assemblies because it requires a prohibitive number of perimeter welds. The two failure modes that were concluded to control the majority of diaphragm failures for this type of floor system were diagonal tension cracking and perimeter fastener failure. These two failure modes are a main focus of this document.

Table 2-1. Test Matrix for Specimens in Porter and Easterling (1988) Testing Program Used in Validation of Initial Stiffness Prediction Model

<table>
<thead>
<tr>
<th>Test Specimen</th>
<th>Total Slab Depth (in)</th>
<th>Deck Height, D₄ (in)</th>
<th>Measured Concrete Compressive Strength, fc‘ (ksi)</th>
<th>Experimental Initial Stiffness (kip/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>5.7</td>
<td>3.0</td>
<td>4.1</td>
<td>1131</td>
</tr>
<tr>
<td>4</td>
<td>5.3</td>
<td>3.0</td>
<td>3.8</td>
<td>1348</td>
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<td>5</td>
<td>3.5</td>
<td>1.5</td>
<td>3.0</td>
<td>1708</td>
</tr>
<tr>
<td>6</td>
<td>7.4</td>
<td>1.5</td>
<td>4.5</td>
<td>1891</td>
</tr>
<tr>
<td>7</td>
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<td>3.0</td>
<td>5.4</td>
<td>1336</td>
</tr>
<tr>
<td>9</td>
<td>5.5</td>
<td>3.0</td>
<td>5.4</td>
<td>1345</td>
</tr>
<tr>
<td>10</td>
<td>5.5</td>
<td>3.3</td>
<td>3.3</td>
<td>1609</td>
</tr>
<tr>
<td>11</td>
<td>5.7</td>
<td>3.0</td>
<td>3.5</td>
<td>1770</td>
</tr>
<tr>
<td>12</td>
<td>5.6</td>
<td>3.3</td>
<td>3.4</td>
<td>1712</td>
</tr>
<tr>
<td>13</td>
<td>5.5</td>
<td>3.0</td>
<td>6.2</td>
<td>2021</td>
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<td>14</td>
<td>8.2</td>
<td>3.3</td>
<td>3.7</td>
<td>1838</td>
</tr>
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<td>2.8</td>
<td>1132</td>
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<tr>
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<td>4.2</td>
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<td>3.0</td>
<td>921</td>
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<td>1.5</td>
<td>4.3</td>
<td>1595</td>
</tr>
<tr>
<td>18</td>
<td>5.6</td>
<td>3.3</td>
<td>3.1</td>
<td>1582</td>
</tr>
<tr>
<td>19</td>
<td>5.8</td>
<td>3.0</td>
<td>2.7</td>
<td>930</td>
</tr>
<tr>
<td>20</td>
<td>5.6</td>
<td>2.5</td>
<td>4.0</td>
<td>1302</td>
</tr>
<tr>
<td>21</td>
<td>5.7</td>
<td>3.3</td>
<td>3.6</td>
<td>868</td>
</tr>
<tr>
<td>22</td>
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</tr>
<tr>
<td>23</td>
<td>5.8</td>
<td>2.5</td>
<td>3.5</td>
<td>1368</td>
</tr>
<tr>
<td>24</td>
<td>5.6</td>
<td>3.0</td>
<td>4.0</td>
<td>1657</td>
</tr>
</tbody>
</table>
2.2 Current Empirical Equations for Stiffness

AISI S310-20 (AISI, 2020) provides Equation (2-1) for the shear stiffness of a concrete-filled steel deck diaphragm as a function of two additive components. The first component accounts for the stiffness contribution of the fasteners and steel deck, while the second component \( K_3 \) accounts for the stiffness contribution of the structural concrete fill above the top of the deck. The \( K_3 \) term of Equation (2-1) is empirical and may have been calibrated to test data that included not only shear deformations (for which \( K_3 \) is meant to capture), but also bending and shear transfer deformations, components of deformation that should not be included in the shear stiffness, \( K_3 \) or \( G' \). The proposed prediction model described in Section 2.3 accounts for each source of flexibility separately.

\[
G' = \frac{Et}{2(1+\mu)\frac{d}{d_c} + C} + K_3
\]  

(2-1)

Where,

\( E = \) Modulus of elasticity of steel  
\( t = \) base steel thickness of deck  
\( K_3 = 3.5d_c(f'_c)^{0.7}, \) stiffness contribution of structural concrete fill  
\( d_c = \) structural concrete thickness above top of deck  
\( f'_c = \) concrete compressive strength  
\( \mu = \) Poisson’s ratio for steel  
\( s = \) Developed flute width per pitch  
\( d = \) panel corrugation pitch  
\( C = \) slip constant considering slippage at sidelap connections and distortion at support connections (See AISI S310-16 for more information)
Examples are provided in the fourth edition of Diaphragm Design Manual (Luttrell, 2015) which illustrate how to use the $G'$ calculated from Equation (2-1) in calculating diaphragm deflections. DDM04 (Luttrell, 2015) indicates that the total deflection of the diaphragm is calculated as the combination of the bending deflections and the shear deflections. The bending deflections are inversely proportional to the moment of inertia and the modulus of elasticity of the diaphragm (bending stiffness), while the shear deflection is inversely proportional to the shear stiffness of the diaphragm, $G'$.

A proposed modification to Equation (2-1) was put forward by O’Brien (2017). O’Brien concluded that the sidelap flexibility will not contribute to slip between the deck and steel supports due to the mechanical and chemical bond between the deck and the concrete, so he proposed a modification to the slip constant, $C_2$, to neglect the contribution of the sidelap fasteners. O’Brien also modified how the prediction model accounted for the contribution of the steel deck, using an equivalent transformed effective thickness of concrete within the $K_4$ term of Equation (2-2), as opposed to the thickness of concrete above the deck flutes utilized by the $K_3$ term in Equation (2-1). The $K_4$ term was calibrated by O’Brien (2017) to produce a best fit prediction with the experimental results of the testing program conducted by Porter and Easterling (1988). This program reported initial stiffness measurements based on the displacement of the actuators at the free end of the cantilever diaphragm setup which include shear deformations, bending deformations, deformation of the shear transfer connections, and deformation of the support reactions. It is inappropriate, therefore, to calibrate the shear stiffness against data that includes deformations from sources outside of shear deformations.

$$G' = \frac{Et}{C_2} + K_4$$  \hspace{1cm} (2-2)
Where,

\[ K_d = 4.8 t_e \sqrt{f_c} \], stiffness contribution of concrete-filled steel deck diaphragm

\[ C_a = \left( \frac{E_s}{w} \right) \left( \frac{2L}{2\alpha_3 + n_p\alpha_4} \right) S_f \], slip constant considering slippage at sidelap connections and distortion at support connections

\[ t_e = t_c + t_{dt} \], equivalent total transformed concrete thickness

\[ t_c = \text{average thickness of structural concrete, including concrete in deck flutes} \]

\[ t_{dt} = n_{sc} t_s \frac{d}{s} \], transformed thickness of steel deck. d and s as defined in Figure 2-2

\[ n_{sc} = \frac{E_s}{E_c} \], shear modular ratio of steel deck to concrete

\[ E_s = \text{elastic modulus of steel deck} \]

\[ E_c = \text{elastic modulus of concrete fill} \]

\[ t_s = \text{thickness of corrugated deck} \]

\[ d = \text{width of single corrugation} \]

\[ s = \text{developed flute of single corrugation} \]

\[ = 2(e + w_c) + f \]

Centerline of deck flute

Figure 2-2. Deck Corrugation Geometry
2.3 Model for Diaphragm Stiffness

Porter and Easterling (1988) proposed a stiffness prediction method based on the assumption that the diaphragm acts like a deep beam in which the steel framing members constitute the flanges and the concrete-filled steel deck diaphragm constitutes the web. Porter and Easterling (1988) theorized that the stiffness of diaphragm specimens consisted of four components: bending stiffness of the concrete-filled steel deck diaphragm, shear stiffness of the concrete-filled steel deck diaphragm, edge zone stiffness of the concrete-filled steel deck diaphragm and the stiffness of the frame connections. Based on this assumption, the total stiffness of the diaphragm can be calculated using Equation (2-3).

\[
k_{tot} = \frac{1}{\frac{1}{k_b} + \frac{1}{k_s} + \frac{1}{k_z} + \frac{1}{k_f}}
\]  

(2-3)

Where,

\( k_b \) = bending stiffness of concrete-filled steel deck diaphragm
\( k_s \) = shear stiffness of concrete-filled steel deck diaphragm
\( k_z \) = edge zone (fastener) stiffness of concrete-filled steel deck diaphragm
\( k_f \) = frame connection stiffness

For a cantilever diaphragm configuration, the bending stiffness of the “deep beam” assumed in this model can be calculated using the deflection of a cantilever beam with a length equal to the span of the diaphragm, and a moment of inertia that accounts for both the “flanges” (frame members) and the “web” (concrete-filled steel deck diaphragm). This results in Equation (2-4).

\[
k_b = \frac{3(E_c I_c + E_b I_b)}{a^3}
\]  

(2-4)
Where,

\[ E_c = \text{concrete modulus of elasticity} \]

\[ I_c = \text{moment of inertia of concrete-filled steel deck diaphragm} \]

\[ E_b = \text{frame member modulus of elasticity} \]

\[ I_b = \text{moment of inertia of edge beams about “deep beam” neutral axis} \]

\[ a = \text{span of the diaphragm} \]

Similarly, the shear stiffness of the “deep beam” assumed in this model can be calculated using Equation (2-5). This expression neglects any contribution from the frame members since the majority of the shear is assumed to be resisted by the web.

\[
k_s = \frac{b \left( G_s t_s \left( \frac{d}{s} \right) + G_c t_c \right)}{\alpha} \quad (2-5)
\]

Where,

\[ b = \text{depth of diaphragm “deep beam”} \]

\[ G_s = \text{shear modulus of steel} \]

\[ t_c = \text{average concrete thickness considering deck geometry} \]

\[ t_s = \text{thickness of steel deck} \]

\[ d \text{ and } s \text{ as defined in Figure 2-2} \]

The edge zone stiffness is calculated using Equation (2-6) which assumes that while the diaphragm remains elastic, the edge zone deformation is equal to the deformation of the fasteners. The magnitude of these deformations depends on the type of fastener.

\[
k_z = \frac{1}{\gamma \left( \frac{\alpha}{2} + \frac{\alpha}{\Lambda} \right)} \quad (2-6)
\]
Where,

\[ \Delta_t = \text{slip of connectors on beams parallel to shear load} \]

\[ \Delta_p = \text{slip of connectors on beams perpendicular to shear load} \]

Porter and Easterling (1988) obtained the support frame stiffness experimentally since it depends on the connection details and material properties. They determined that the support frame stiffness is only a concern for the experimental setup and is not included in their recommendations for design.
## 3 Initial Stiffness of Concrete-Filled Steel Deck Diaphragms

The results of the experimental program performed at Virginia Tech as they relate to the initial stiffness of concrete-filled steel deck diaphragms are presented in this section. The experimental setup and specimen configuration are presented in a separate report (Avellaneda-Ramirez et al. 2021). The test matrix is summarized in Table 3-1.

### Table 3-1. Test Matrix

<table>
<thead>
<tr>
<th>Specimen Type</th>
<th>Test Specimen</th>
<th>Total Slab Depth (in)</th>
<th>Deck Height, $D_d$ (in)</th>
<th>Concrete Type</th>
<th>Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unreinforced</td>
<td>2/4-4-L-NF-DT</td>
<td>4</td>
<td>2</td>
<td>LW</td>
<td>none</td>
</tr>
<tr>
<td></td>
<td>3/6.25-4-L-NF-DT</td>
<td>6.25</td>
<td>3</td>
<td>LW</td>
<td>none</td>
</tr>
<tr>
<td></td>
<td>3/7.5-4-N-NF-DT</td>
<td>7.5</td>
<td>3</td>
<td>NW</td>
<td>none</td>
</tr>
<tr>
<td>Reinforcing Bars</td>
<td>2/4.5-4-L-RS-DT</td>
<td>4.5</td>
<td>2</td>
<td>LW</td>
<td>#4 bars at 12 in. spacing</td>
</tr>
<tr>
<td></td>
<td>3/6.25-4-L-RS-DT</td>
<td>6.25</td>
<td>3</td>
<td>LW</td>
<td>6x6 D2.1xD2.1</td>
</tr>
<tr>
<td></td>
<td>3/7.5-4-N-RS-DT</td>
<td>7.5</td>
<td>3</td>
<td>NW</td>
<td>#3 bars at 18 in. spacing</td>
</tr>
<tr>
<td>Perimeter Fastener Failure</td>
<td>3/6.25-4-L-NF-P</td>
<td>6.25</td>
<td>3</td>
<td>LW</td>
<td>none</td>
</tr>
<tr>
<td></td>
<td>3/7.5-4-N-NF-P</td>
<td>7.5</td>
<td>3</td>
<td>NW</td>
<td>none</td>
</tr>
</tbody>
</table>

NW = normal weight concrete  
LW = lightweight concrete

The deformation of the specimen was recorded during the test using an array of displacement sensors, as illustrated in Figure 3-1. Due to a malfunction of the data acquisition system, displacement sensor data was lost for specimen 3/7.5-4-N-NF-P. This specimen will not be included in later tables and data analysis because only actuator displacement and force readings were recorded. Specimens 2/4-4-L-NF-DT, 3/6.25-4-L-NF-DT, and 3/7.5-4-N-NF-DT did not include all the sensors shown in Figure 3-1. Sensors US 1, US 2, SP14, and SP15 were not present for those specimens. Additional details on the instrumentation arrangement are
presented in Avellaneda-Ramirez et al. (2021). Displacement sensors “BOT” were attached to the bottom flange of the beams, while “MID” indicates sensors attached to the beams just below the slab, and “TOP” refers to displacement sensors located above the slab.

Sensors US 1 and US 2 are measuring out of plane movement of the corners. These sensors are attached to the bottom flange of the steel frame.

Figure 3-1. Displacement Sensor Configuration

3.1 Initial Stiffness Experimental Results

The AISI S907 (AISI, 2013) test standard provides recommendations for the arrangement of displacement sensors. The large in-plane stiffness of concrete-filled steel deck diaphragms when compared to bare steel deck diaphragms (for which the test standard was developed) presents several unique challenges, particularly in measuring shear deformations and the
calculation of the initial stiffness. The change in shear angle during loading for each specimen was calculated using four methods (listed in paragraphs below). For each method, the initial stiffness was obtained two ways:

1. A secant stiffness at 40% of the peak shear strength
2. Linear regression analysis

The linear regression analysis was performed on the initial cycles of loading. Only the cycles prior to reaching 40% of the peak strength were used for this analysis. Since the amplitude of these cycles varied between specimens, the number of points used in this linear regression analysis was not constant.

Method 1 for Measured Stiffness – AISI S907 (AISI, 2013) with Diagonal Measurements

One of the methods proposed by AISI S907 (AISI, 2013) for the calculation of shear angle is described in Figure 3-2 which uses a linear displacement sensor to measure the elongation of the diagonal distance between two non-adjacent corners of the specimen (ΔSP1) to compute the lateral translation of the free end (δ_{dia}) using Equation (3-1), which is then divided by the length of the specimen perpendicular to the load to obtain the change in shear angle (γ_{dia}). The shear angle, γ_{dia}, calculated using this method is obtained from Equation (3-2). While only one diagonal sensor is needed to obtain the change in shear angle, AISI S907 (AISI, 2013) recommends using two sensors and averaging the results.

\[
δ_{dia} = \frac{(|ΔSP1|)\sqrt{A^2 + B^2}}{B} \quad (3-1)
\]

\[
γ_{dia} = \frac{δ_{dia}}{A} = \frac{(|ΔSP1|)\sqrt{A^2 + B^2}}{AB} \quad (3-2)
\]
Method 2 for Calculating Measured Stiffness – Law of Cosines

An alternative method for calculating change in shear angle is illustrated in Figure 3-3. This method assumes that the length of the specimen perpendicular to the loading (A) may change during loading and so it uses both the change of the diagonal distance between the non-adjacent corners of the specimen (ΔSP1) as well as the change of the perpendicular distance between the loading beam and the fixed beam (ΔSP9) to compute the change in shear angle $\gamma_{LOC}$ using the Law of Cosines. The shear angle, $\gamma_{LOC}$, calculated using this method is obtained from Equation (3-3).

$$\gamma_{LOC} = \cos^{-1} \left( \frac{(A_0 + \Delta SP9)^2 - \frac{(C_0 + \Delta SP1)^2}{2 \cdot B_0 \cdot (A_0 + \Delta SP9)}}{2 \cdot B_0 \cdot (A_0 + \Delta SP9)} \right) - \cos \left( \frac{A_0^2 - \frac{C_0^2}{2 \cdot B_0 \cdot A_0}}{2 \cdot B_0 \cdot A_0} \right) \quad (3-3)$$

Figure 3-2. Graphical Illustration of Shear Angle Calculation Using AISI S907 (AISI, 2013), Equation (3-2)
Method 3 for Measured Stiffness – AISI S907 (AISI, 2013) without Diagonal Measurements

AISI S907 (AISI, 2013) also provide a method for calculating shear angle which doesn’t include diagonal measurements. This method is illustrated in Figure 3-4. It primarily relies on the lateral translation of the free edge which can be measured with a linear displacement sensor ($\Delta SP2$) but it recognizes that some of this displacement may be due to the rigid body rotation and translation of the specimen so it uses additional sensors to subtract these effects. The rigid body rotation is accounted for by measuring the perpendicular displacement of the two corners of the fixed beam ($\Delta SP3$ and $\Delta SP7$) and the rigid body translation is accounted for by measuring the lateral translation of the fixed end ($\Delta SP4$). The shear angle, $\gamma_{LOC}$, calculated using this method is obtained from Equation (3-4).

$$\gamma_{LT} = \left[ \Delta_{SP2} - \left( \Delta_{SP4} + (\Delta_{SP3} + \Delta_{SP7}) \left( \frac{A}{B} \right) \right) \right] \frac{1}{A} \quad (3-4)$$
Figure 3-4. Graphical Illustration of Shear Angle Calculation Using AISI S907 (AISI, 2013), Equation (3-4)

Method 4 for Calculating Measured Stiffness – Three-dimensional (3D) Triangulation

Due to the flexibility of the supporting frame and the out-of-plane movement of the specimen during early cycles, the shear stiffness of the specimens was typically lower when calculated using the sensors attached to the bottom of the frame and larger for the sensors attached closest to the specimen. By taking advantage of the different sensors present for each test, a method for calculating the shear angle was developed. The purpose of this method is to obtain the approximate 3D coordinates for a point on the loading corner of the specimen at the
interface between the concrete slab and the top flange of the supporting frame (P₃) as shown in Figure 3-5. An assumption used in this method is that the corners of the specimen connected to the fixed beam are rigidly fixed and so B₀ is constant throughout the test.

Figure 3-5. Illustration of 3D Coordinate Method for Computing Shear Angle

Figure 3-6. Three Points Used for 3D Coordinate Method for Computing Shear Angle
To obtain the coordinates of P₃, it is necessary to first obtain the 3D coordinates of a point below the bottom flange of the testing frame (P₁ in Figure 3-6) and a point above the concrete slab (P₂ in Figure 3-6). As, shown in Figure 3-7, the in-plane movement of P₂ (x and y coordinates) can be obtained by mathematically solving for the intersection of the perimeters of two circles with radius equal to C₀+ΔSP₁ and A₀+ΔSP₉, respectively. The perimeters of these two circles represent the possible location of the corner based on the change in length of the sensors connected to said corner. These perimeters intersect at two points. The farthest of these two intersection points from the original coordinate is considered a trivial solution so only the closest value is used. The out-of-plane coordinate (z-coordinate) is taken directly from the uplift sensor connected to this corner (US₂).

![Figure 3-7. Obtaining In-Plane Coordinates of P₂](image)

Once the 3D coordinate for P₁ is obtained, a similar process can be performed to obtain the coordinates of P₂ by using the sensors connected to that corner above the slab (SP₁₂ and
SP15), as shown in Figure 3-8. The out-of-plane coordinate for this point is obtained by assuming that the distance $H_{1-2}$ in Figure 3-6 remains constant throughout the test.

![Figure 3-8. Obtaining In-Plane Coordinates of P₃](image)

Now that the 3D coordinates of $P_1$ and $P_2$ have been obtained, the coordinates of $P_3$ can be calculated using a vector of length $H_{2-3}$ (as defined in Figure 3-6) with origin in $P_2$ and oriented parallel to the vector formed by points $P_1$ and $P_2$. The change in shear angle is then calculated using Equation 3-4.

$$
Y_{3D} = \cos \left( \frac{\vec{V}_1 \cdot \vec{V}_2}{|\vec{V}_1| \cdot |\vec{V}_2|} \right) - Y_0
$$

(3-5)

Where,

$\vec{V}_1$ = Vector going from $F_2$ (as defined in Figure 3-5) and $P_3$

$\vec{V}_2$ = Vector going from $F_2$ (as defined in Figure 3-5) and $F_1$ (as defined in Figure 3-5)

$Y_0$ = Initial angle based on original coordinate of $P_3$
Results for Measured Stiffness

The initial stiffness measurements for each tested diaphragm specimen are summarized in Table 3-2 for all four methods of shear angle calculation as described above. For each method, the stiffness was calculated using either linear regression and a secant stiffness through at point at 40% of the peak strength as described at the beginning of this subsection. Method 1 which uses diagonal displacement sensors was calculated for displacement sensors near the top of the steel beams (labeled as mid layer) and for displacement sensors located at the bottom of the steel beams (labeled as bottom layer). Calculations for Method 2 only used the bottom layer of displacement sensors because required measurements were not available at the mid layer.

The 3D triangulation method for calculating shear angle is deemed to be the most accurate since it accounts for the effects of out-of-plane movement of the frame, axial elongation of the connections, and flexibility of the frame. However, the instrumentation necessary to perform this calculation was only present in four of the tested specimens. The accuracy of Methods 1 through 3 were thus judged by comparing them to the results of the 3D triangulation Method 4. This comparison shows that on average, the initial stiffness obtained from Method 1 with the mid layer of instrumentation yields a reasonable estimate of initial stiffness. In the following sections, the measured stiffness using the 3D triangulation (Method 4) is used for the first four specimens, while the specimens for which Method 4 cannot be implemented will be omitted.

The results in Table 3-2 show that the initial stiffness measurements calculated using the middle layer of displacement sensors generally resulted in substantially larger stiffness values compared to the bottom layer. Twisting of the steel beams was visually observed during cycles at large displacements, and the stiffness results indicate that even during initial loading, twisting of
the steel beams was likely the cause of the discrepancy between stiffness calculated using the bottom and middle layers. For this reason, the measured stiffness calculated using the bottom layer of displacement sensors is not considered representative of the stiffness of the concrete-filled steel deck diaphragm.

The stiffness measurements calculated using linear regression analysis are on average lower than those obtained as a secant stiffness at 40% of peak shear strength. This indicates that there is some hysteresis in the response (i.e., the unloading portions of the initial cycles have smaller load values than the loading portions), even during these initial cycles. For specimen 3/6.25-4-L-NF-P (designed to fail the studs), the 3D triangulation method (Method 4) resulted in an initial stiffness almost identical to the initial stiffness calculated using Method 1 with the mid layer of diagonal sensors and using the average secant stiffness at 40% of peak strength. The similarity is likely due to the flexibility of the shear transfer connection which decreases the in-plane stiffness of the concrete-filled steel deck diaphragms, resulting in less twisting of the steel beams.
Table 3-2. Summary of Stiffness Measurements

<table>
<thead>
<tr>
<th>Method Used for Calculating Shear Angle</th>
<th>Specimen</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2/4.5-4-L-RS-DT</td>
</tr>
<tr>
<td>Bottom Layer of Instrumentation</td>
<td></td>
</tr>
<tr>
<td>2. Law of Cosines Method</td>
<td>Linear Regression</td>
</tr>
<tr>
<td></td>
<td>Secant Stiffness @ 40%</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>1. AISI S907 Diagonal Measurements Method</td>
<td>Linear Regression</td>
</tr>
<tr>
<td></td>
<td>Secant Stiffness @ 40%</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Mid Layer of Instrumentation</td>
<td>Linear Regression</td>
</tr>
<tr>
<td>1. AISI S907 Diagonal Measurements Method</td>
<td>Secant Stiffness @ 40%</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>3. AISI S907 without Diagonal Measurements Method</td>
<td>Linear Regression</td>
</tr>
<tr>
<td></td>
<td>Secant Stiffness @ 40%</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>4. 3D Triangulation Method</td>
<td>Linear Regression</td>
</tr>
</tbody>
</table>

N/A = Not Available

Secant stiffness calculated through point at 40% of peak shear strength in either the positive (pos) or negative (neg) loading direction

Linear regression stiffness calculated using initial loading cycles prior to reaching 40% of peak experimental strength
3.2 Prediction of Initial Stiffness for Cantilever Diaphragm Specimens

A proposed prediction model for the initial shear stiffness of a concrete-filled steel deck diaphragm in a cantilever configuration is outlined in this section. The way to apply this prediction model to generalized diaphragms is presented in Chapter 5. The proposed approach is based on the model proposed by Porter and Easterling (1988) wherein the deflection of the free end for their cantilever diaphragm specimens consists of four components: 1) the deflection of the concrete-filled steel deck diaphragm due to bending deformations, 2) the deflection of the concrete-filled steel deck diaphragm due to shear deformations, 3) the deformation of the edge zone of the concrete-filled steel deck diaphragm, and 4) displacement of the support reaction connections at the fixed end. Based on this assumption, the total deflection of the free end of a cantilever diaphragm specimen can be calculated using Equation (3-6).

\[ \Delta_{tot} = \Delta_s + \Delta_b + \Delta_z + \Delta_f \]  

(3-6)

Where,

\( \Delta_{tot} \) = Total deflection of concrete-filled steel deck diaphragm
\( \Delta_s \) = Deflection of concrete-filled steel deck diaphragm due to shear
\( \Delta_b \) = Deflection of concrete-filled steel deck diaphragm due to bending
\( \Delta_z \) = Deflection of concrete-filled steel deck diaphragm due to deformations at the edge zone of the diaphragm (shear transfer slip)
\( \Delta_f \) = Deflection of concrete-filled steel deck diaphragm due displacement of support reaction connections
The deflection of the free end of a cantilever diaphragm specimen due to bending (\(\Delta_b\)), as illustrated in Figure 3-9, can be calculated using Equation (3-7). The bending is resisted by the flanges of the deep beam consisting of the W-sections oriented perpendicular to the direction of loading (weak axis bending), as well as the web of the deep beam consisting of the concrete-filled steel deck diaphragm. The moment of inertia of the web and the flanges of the deep beam with respect to the centroidal axis of the assembly is calculated using the parallel axis theorem.

\[
\Delta_b = \frac{V \cdot a^3}{3 \cdot (E_c I_c + E_s I_b)}
\]  

Where,

\(V\) = Load applied at free end of cantilever diaphragm specimen

\(a\) = span of diaphragm “deep beam”

\(E_c\) = modulus of elasticity of concrete

\(E_s\) = modulus of elasticity of steel

\(I_c\) = moment of inertia of concrete-filled steel deck diaphragms with respect to the centroidal axis of the assembly

\(I_s\) = moment of inertia of the steel beams with respect to the centroidal axis of the assembly
The deflection of the free end of a cantilever diaphragm specimen due to the deformations of the edge zone \( \Delta_z \), as illustrated in Figure 3-10, can be calculated using Equation (3-8). Both the in-plane translation of the slab with respect to the supporting frame due to the slip of the shear connection on the beams oriented parallel to the loading, and the rotation of the slab with respect to the supporting frame due to the slip of the shear connection on the beams oriented perpendicular to the loading are taken into account in Equation (3-8).

\[
\Delta_z = 2 \left( \Delta_t + \frac{a}{b} \Delta_p \right) \tag{3-8}
\]

Where,

\( \Delta_t \) = slip of shear transfer connections on beams oriented parallel to shear loading (see Figure 3-10)
Δ_p = slip of shear transfer connections on beams oriented perpendicular to shear loading
(see Figure 3-10)

b = depth of diaphragm “deep beam”

Porter and Easterling (1988) concluded that the stiffness of a headed stud can be estimated using an expression proposed by Dodd (1986) in which the stiffness of the headed stud is linearly proportional to the peak strength of the stud with a constant of proportionality of 145.3, which corresponds to the stiffness (kip/in) of a headed stud with a peak strength of one kip. Therefore, the slip of the shear transfer connection can be calculated using Equation (3-9).

\[
\Delta_{t,p} = \frac{V}{k_{st} \cdot Q_{su} \cdot n_s}
\]

(3-9)

Where,

Δ_t = slip of shear transfer connections on beams oriented parallel to shear loading (see Figure 3-10), (in. for Imperial or mm for SI units)
\( \Delta_p \) = slip of shear transfer connections on beams oriented perpendicular to shear loading (see Figure 3-10), (in. for Imperial or mm for SI units)

\( k_{st} \) = stiffness of a single headed stud anchor with a peak strength of 1 kip (Imperial) or 1 kN (SI units). Recommended value of 145.3 in.\(^{-1}\) (Imperial units) or 5.72 mm\(^{-1}\) (SI units)

\( Q_{su} \) = capacity of headed stud anchors in load direction, (kip for Imperial or kN for SI units)

\( n_s \) = number of headed stud anchors in load direction

\[ \Delta_f = \frac{2 \cdot a}{b} \Delta_c \]  \hspace{1cm} (3-10)

Where,

\( \Delta_c \) = displacement of support reaction connections
The expression proposed by Porter and Easterling (1988) for calculating the deflection of the free end of a cantilever diaphragm specimen due to shear ($\Delta_s$) (as illustrated in Figure 3-12) is presented in Equation (3-11).

$$\Delta_s = \frac{V \cdot a}{b \cdot G'_s} \quad (3-11)$$

Where,

$G'_s =$ shear stiffness of concrete-filled steel deck diaphragm
Porter and Easterling (1988) provide Equation (3-12) for calculating the $G'_{s}$ term, which represents the shear stiffness of the concrete-filled steel deck diaphragm. The $G'_{s}$ term includes the contribution of the concrete within the deck ribs as well as the contribution from the steel deck.

$$G'_{s} = G_{deck} t_s \left( \frac{d}{s} \right) + G_{concrete} t_c$$  \hspace{1cm} (3-12)

Where,

$G_{s}$ = shear modulus of steel deck

$G_{c}$ = shear modulus of concrete

$t_c$ = average concrete thickness considering deck geometry

$t_s$ = thickness of steel deck

$d$ and $s$ as defined in Figure 2-2
The expression for $G'_s$ can be further simplified by implementing some assumptions deemed reasonable for design purposes. The definition of the shear modulus of the concrete and steel materials can be substituted into Equation (3-12), while defining the modular ratio as $n_{sc} = E_s/E_c$, resulting in Equation (3-13).

$$G'_s = \frac{n_{sc}E_c}{2(1 + \nu_s)} t_s \left( \frac{d}{s} \right) + \frac{E_c}{2(1 + \nu_c)} t_c$$  \hspace{1cm} (3-13)

Grouping common terms and defining the transformed deck thickness as $t_{cd} = n_{sc} t_s \left( \frac{d}{s} \right)$ and the modulus of elasticity of concrete as $E_c = 0.033 \cdot w_c^{1.5} \sqrt{f'_c}$ results in Equation (3-14).

$$G'_s = 0.0165 \cdot w_c^{1.5} \sqrt{f'_c} \left( \frac{t_{cd}}{(1 + \nu_s)} + \frac{t_c}{(1 + \nu_c)} \right)$$  \hspace{1cm} (3-14)

Assuming that the Poisson’s ratio for both the concrete and the steel is 0.3 ($\nu_s = \nu_c = 0.3$) and defining the effective slab thickness as $t_e = t_{cd} + t_c$ results in Equation (3-15).

$$G'_s = 0.013 \cdot w_c^{1.5} \cdot t_e \sqrt{f'_c}$$  \hspace{1cm} (3-15)

Finally, assuming a unit weight for lightweight concrete and normal weight concrete of 110 pcf and 145 pcf respectively, a lightweight factor ($\lambda$) can be implemented, resulting in Equation (3-16).

$$G'_s = 22 \cdot \lambda^{1.5} \cdot t_e \sqrt{f'_c}$$  \hspace{1cm} (3-16)

Where,

$\lambda =$ factor for lightweight concrete

$= 0.75$ for $w_c = 110$ pcf
\[ = 1 \text{ for } w_c = 145 \text{pcf} \]

Equation (3-16) is the proposed expression for the calculation of the shear stiffness of a concrete-filled steel deck diaphragm to be used for the calculation of the shear deformations contributing to diaphragm deflections. The proposed expressions account for the effect of lightweight concrete on shear stiffness, as illustrated in Figure 3-13. The difference between the predicted stiffness using the equation provided in AISI S310-20 (AISI, 2020) and the proposed prediction model is attributed to the empirical approach used in calibrating the AISI prediction model. The experimental data used in the calibration of the AISI S310-20 (AISI, 2020) prediction model included more than just shear deformations (bending, shear transfer, etc.) resulting in a prediction model with lower predicted shear stiffness than the proposed prediction model which accounts only for the shear deformations to be consistent with the purpose of \( G' \).
The prediction model presented by AISI S310-20 (AISI, 2020) does not account for the effect of lightweight concrete. Therefore, $G'_s$ using AISI S310 is the same for NW and LW.

Figure 3-13. Comparison of Proposed Prediction Model for $G'_s$ (Equation (3-16)) to Existing Prediction Model in AISI S310-20 (Equation (2-1)) as a Function of Concrete Compressive Strength, $f'_c$.

### 3.3 Comparison of Experimental Results to Proposed Prediction Method

The accuracy of the proposed prediction model for the initial shear stiffness of concrete-filled steel deck diaphragms is examined in this section by comparing the results from the cantilever diaphragm testing programs performed by Porter and Easterling (1988) and the testing program described in this document to predictions using the proposed prediction model. A summary of the predicted initial stiffness for the cantilever diaphragm specimens in both testing
programs is provided in Table 3-3. The instrumentation setup for specimens 4/2-4-LW-NF-DT, 3/6.25-4-L-NF-DT, and 3/7.5-4-N-NF-DT did not include sufficient sensors to implement the 3D triangulation method for calculating shear angle and the uplift bracing present in these specimens was deemed insufficient in preventing out-of-plane movement which affected deformation measurements. Therefore, these specimens are not included in Table 3-3 and in further analysis. Additional restraint against out of plane translation was provided for the remaining specimens (Avellaneda-Ramirez et al. 2021).

The predicted initial stiffness at 40% of peak load is calculated using Equation (3-17).

$$K'_{pred} = \frac{V_{40\%}\cdot a}{b\cdot \Delta_{tot40\%}}$$ \hspace{1cm} (3-17)

Where,

- $K'_{pred}$ = predicted initial stiffness for cantilever diaphragm specimen
- $\Delta_{tot40\%}$ = total predicted deflection of the free end of cantilever diaphragm at 40% of peak load calculated using Equation (3-6).
- $V_{40\%}$ = 40% of peak load
### Table 3-3. Comparison of Prediction of Initial Shear Stiffness to Experimental Results of Cantilever Diaphragm Specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>40% of Peak Load</th>
<th>Experimental Initial Stiffness, (K_{\text{exp}}) (kip)</th>
<th>Predicted Total Deflection @ 40% of Peak Load (kip/in)</th>
<th>Predicted Initial Stiffness, (K_{\text{pred}}) (kip/in)</th>
<th>Experiment/Prediction ((K_{\text{exp}}/K_{\text{pred}}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>2/4.5-4-L-RS-DT</td>
<td>106</td>
<td>2273</td>
<td>0.0398</td>
<td>2125</td>
<td>1.07</td>
</tr>
<tr>
<td>3/6.25-4-L-RS-DT</td>
<td>80</td>
<td>3528</td>
<td>0.0246</td>
<td>2598</td>
<td>1.36</td>
</tr>
<tr>
<td>3/7.5-4-N-RS-DT</td>
<td>130</td>
<td>3378</td>
<td>0.0285</td>
<td>3634</td>
<td>0.93</td>
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<td>3/6.25-4-L-NF-P</td>
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<td>1500</td>
<td>0.0136</td>
<td>1415</td>
<td>1.06</td>
</tr>
<tr>
<td>3</td>
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<td>1326</td>
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<td>1637</td>
<td>0.0318</td>
<td>1687</td>
<td>0.98</td>
</tr>
</tbody>
</table>

Mean 0.98
Median 0.98
Std. Dev. 0.21
A graphical comparison of the reported initial stiffness experimental results to predictions using the proposed prediction model is provided in Figure 3-14. The majority of experimental values are within 30% of predicted values with an average experiment to prediction ratio of 0.98 with a standard deviation of 0.21. The proposed model is shown to predict the measured initial stiffness of the cantilever diaphragm specimens with reasonable accuracy.

![Graphical Comparison](image)

Figure 3-14. Comparison of Experimental Initial Stiffness for Cantilever Diaphragm Specimens to Proposed Prediction Model
4 Example: Diaphragm Deflection

To illustrate the relative magnitude of the components of diaphragm deflection for two different diaphragm configurations, as well as to provide guidance on how the proposed prediction model for the initial shear stiffness of concrete-filled steel deck diaphragms can be used for calculating diaphragm deflections, two examples are provided in this section. The first example consists of a cantilever diaphragm configuration while the second example consists of a simply supported diaphragm configuration.
4.1 Example 1: Cantilever Diaphragm Specimen

**Given Information**

\[
\begin{align*}
a &= 12 \text{ ft} \\
b &= 15 \text{ ft} \\
V &= 80 \text{ kip} \\
w_c &= 110 \text{pcf} \\
f'_c &= 4.35 \text{ksi} \\
E_s &= 29000 \text{ksi} \\
\text{Verco W3 Formlok with 3.25 in. lightweight concrete cover} \\
\text{1 headed stud welded at 36 in. spacing on all four beams}
\end{align*}
\]

**Objective**

Calculate the deflection of the free end of a cantilever diaphragm specimen (Figure 4-1), due to a shear load, \(V=80\) kips.
Calculations

1. Calculate relevant material properties

\[ E_c = w_c^{1.5} \cdot 33 \cdot \sqrt{f_c'} = (110 \text{ pcf})^{1.5} \cdot 33 \cdot \sqrt{4350 \text{ psi}} = 2511 \text{ ksi} \]

\[ n_{sc} = \frac{E_x}{E_c} = \frac{29000 \text{ ksi}}{2511 \text{ ksi}} = 11.5 \]

2. Calculate deflection of the free end due to bending

\[ \Delta_b = \frac{V \cdot a^3}{3(E_c I_c + E_b I_b)} \]

\[ I_c = \frac{\left(n_{sc} t_s \left( \frac{d}{2} \right) + t_c \right) b^3}{12} = \frac{\left(11.5 \cdot 0.0358 \text{ in} \left( \frac{12 \text{ in}}{16 \text{ in}} \right) + 4.75 \text{ in} \right) (180 \text{ in})^3}{12} = 2,459,000 \text{ in}^4 \]

\[ I_b = 2 \left( I_{yW24X84} + A_{W24X84} \cdot \left( \frac{b}{2} \right)^2 \right) = 2 \left( 94.4 \text{ in}^4 + 24.7 \text{ in}^2 \cdot \left( \frac{180 \text{ in}}{2} \right)^2 \right) = 400,300 \text{ in}^4 \]

\[ \Delta_b = \frac{(80 \text{ kip})(144 \text{ in})^3}{3[(2511 \text{ ksi})(2,459,000 \text{ in}^4) + (29,000 \text{ ksi})(400,300 \text{ in}^4)]} = 0.0045 \text{ in} \]

3. Calculate deflection of the free end due to deformation of shear transfer connections

\[ \Delta_z = 2 \left( \Delta_t + \frac{a}{b} \Delta_p \right) \]

The cantilever diaphragm specimen included four studs on the beams parallel to the shear loading and three studs on the beams perpendicular to the shear loading. Therefore, the slip of the headed studs can be calculated as:

\[ \Delta_t = \frac{V}{145.3 \cdot Q_{su}} = \frac{80 \text{ kip}}{(145.3 \text{ in}^{-1})(15.3 \text{ kip}) \cdot 4} = 0.009 \text{ in} \]

\[ \Delta_p = \frac{V(a)}{145.3 \cdot Q_{su}} = \frac{(80 \text{ kip}) \left(144 \text{ in} \cdot 180 \text{ in}^{-1} \cdot 24 \text{ kip} \cdot 3 \right)}{(145.3 \text{ in}^{-1})(24.4 \text{ kip}) \cdot 3} = 0.006 \text{ in} \]

The deflection of the free end due to the deformation of the shear transfer connection can then be calculated as:
\[ \Delta_z = 2 \left( 0.009 \text{ in} + \frac{144 \text{ in}}{180 \text{ in}} (0.006 \text{ in}) \right) = 0.0276 \text{ in} \]

4. Calculate deflection of the free end due to shear

\[ \Delta_s = \frac{V_a}{bG_s} \]

For comparison, the calculation of \( G_s' \) will be performed in two ways:

- Using the proposed prediction model as described in Section 3.2 (Equation (3-16))

\[ G_s' = 22 \cdot \lambda^{1.5} \cdot t_e \sqrt{f_c} = 22 \cdot (0.75)^{1.5} \cdot \left( (11.5)(0.0358 \text{ in}) \left( \frac{12 \text{ in}}{1.6 \text{ in}} \right) + 4.75 \text{ in} \right) \sqrt{4350 \text{ psi}} = 4769 \text{ kip/in} \]

\[ \Delta_s = \frac{(80 \text{ kip}) \cdot (144 \text{ in})}{(180 \text{ in}) \cdot (4769 \text{ kip/in})} = 0.013 \text{ in} \]

- Using the prediction model provided in AISI S310-20 (AISI, 2020) as described in Chapter 2 (Equation (2-1)). For ease of calculation, the shear stiffness will be taken solely as the \( K_3 \) term in Equation (2-1). \( K_3 \) accounts for the shear resistance of the concrete slab and the steel deck and encompasses the majority of the predicted shear stiffness.

\[ G_s' = K_3 = 3.5 \cdot d_c \cdot f_{c}^{0.7} = 3.5 \cdot (3.25 \text{ in.})(4350 \text{ psi})^{0.7} = 4008 \text{ kip/in} \]

\[ \Delta_s = \frac{(80 \text{ kip}) \cdot (144 \text{ in})}{(180 \text{ in}) \cdot (4008 \text{ kip/in})} = 0.016 \text{ in} \]

5. Calculate the total deflection of the free end

Since the calculation for the deflection of the free end due to shear was performed in two ways, the total deflection will is also calculated two ways:
• Current approach described in DDM04 (Luttrell, 2015) (only bending and shear deflections) using the prediction model provided in AISI S310-20 (AISI, 2020)
for the calculation of initial shear stiffness

\[ \Delta_{\text{total}} = \Delta_b + \Delta_s = 0.0045 \text{ in.} + 0.016 \text{ in.} \]

\[ \Delta_{\text{total}} = 0.0205 \text{ in.} \quad \text{AISI S310-16} \]

• Proposed prediction model

\[ \Delta_{\text{total}} = \Delta_b + \Delta_s + \Delta_z = 0.0045 \text{ in.} + 0.013 \text{ in.} + 0.0276 \text{ in.} \]

\[ \Delta_{\text{total}} = 0.046 \text{ in.} \quad \text{Proposed Method} \]

The results of the example indicate that for the case of a cantilever diaphragm, the approach currently recommended by DDM04 (Luttrell, 2015) results in smaller deflection predictions when compared to the proposed prediction model. The majority of the deformations in the proposed prediction model come from the deformation of the shear transfer connections which are not directly accounted for in the prediction model provided by AISI S310-20 (AISI, 2020).
4.2 Example 2: Simply Supported Diaphragm Configuration

**Given Information**

\[
\begin{align*}
    a &= 300 \text{ ft} \\
    b &= 100 \text{ ft} \\
    w &= 0.333 \text{ kip/ft} \\
    w_c &= 100 \text{ pcf} \\
    f'_c &= 4 \text{ ksi} \\
    E_s &= 29000 \text{ ksi}
\end{align*}
\]

Verco W3 Formlok with 3.25 in. lightweight concrete cover
Structural steel shapes and headed stud anchor configuration are as shown in Figure 4-3.

![Figure 4-2. Schematic of Simply Supported Diaphragm](image)

![Figure 4-3. Typical Floor Plan of Archetype Building (Wei, 2021)](image)
Objective

Calculate the mid-span deflection of simply-supported concrete-filled steel deck diaphragm (Figure 4-2)

Calculations

For the case of a simply supported diaphragm, the total mid-span deflection is calculated using Equation (4-1).

\[ \Delta_{tot} = \Delta_s + \Delta_b + \Delta_z \]  \hspace{1cm} (4-1)

Where,

\[ \Delta_{tot} = \text{total mid-span deflection of simply-supported diaphragm} \]

\[ \Delta_s = \text{deflection of simply-supported diaphragm due to shear (Equation (4-2))} \]

\[ \Delta_b = \text{deflection of simply-supported diaphragm to bending (Equation (4-3))} \]

\[ \Delta_z = \text{deflection of simply-supported diaphragm due to deformations at the edge zone of the diaphragm (shear transfer slip) (Equation (4-4))} \]

For the case of a simply-supported diaphragm with an applied distributed load, the shear mid-span deflection is calculated using Equation (4-2).

\[ \Delta_s = \frac{w \cdot a^2}{8 \cdot b \cdot G'_s} \]  \hspace{1cm} (4-2)

Where,

\[ w = \text{distributed load applied to simply supported diaphragm} \]

\[ a = \text{span of diaphragm “deep beam”} \]

\[ b = \text{depth of diaphragm “deep beam”} \]
The bending mid-span deflection is calculated using Equation (4-3). The bending deflections are affected by the flexibility of the shear transfer connections, as illustrated in Figure 4-4. The slip of the shear transfer connection between the chords and the concrete-filled steel deck diaphragm result in additional deflection. The magnitude of this slip for the largest spacing of headed stud anchor allowed by design specifications was on the order of $4 \cdot 10^{-6}$ in., which was deemed negligible and will therefore be ignored in this example.

$$\Delta_b = 5 \cdot \frac{w \cdot a^4}{384 \cdot (E_c I_c + E_b I_b)} \quad (4-3)$$

Where,

$E_c = \text{modulus of elasticity of concrete}$

$E_b = \text{modulus of elasticity of concrete}$

$I_c = \text{moment of inertia of concrete-filled steel deck diaphragm with respect to the centroidal axis of the assembly}$

$I_b = \text{moment of inertia of the steel beams with respect to the centroidal axis of the assembly}$
Figure 4-4. Mid-Span Deflection of Simply Supported concrete-filled steel deck diaphragm Due to Bending and Shear Only

The contribution of the deformation of the shear transfer connections to the mid-span deflection are illustrated in Figure 4-5. The deflection due to the shear transfer connections is solely due to the slip of the headed stud anchors on the collectors.

\[ \Delta_z = \Delta_t \]  

(4-4)

Where,

\( \Delta_t \) = slip of shear transfer connections on beams oriented parallel to shear loading
Figure 4-5. Mid-Span Deflection of Simply Supported Concrete-Filled Steel Deck Diaphragm Due to Deformation of Shear Transfer Connections Only

1. Calculate relevant material properties:

\[ E_c = w_c^{1.5} \cdot 33 \cdot \sqrt{f'_c} = (110 \text{pcf})^{1.5} \cdot 33 \cdot \sqrt{4000 \text{ psi}} = 2408 \text{ ksi} \]

\[ n_{sc} = \frac{E_s}{E_c} = \frac{29000 \text{ ksi}}{2408 \text{ ksi}} = 12 \]

2. Calculate mid-span deflection due to bending

\[ \Delta_b = \frac{5w \cdot d^4}{384 \cdot (E_d l_c + E_b l_b)} \]

\[ t_c = n_{sc} \cdot t_s \cdot \frac{d}{s} + t_c = (12)(0.0358 \text{ in.}) \left( \frac{12 \text{ in.}}{16 \text{ in.}} \right) + 4.75 \text{ in.} = 5.07 \text{ in.} \]

\[ l_c = \frac{t_c \cdot b^3}{12} = \frac{5.07 \text{ in.} (1200 \text{ in.})^3}{12} = 7.3 \cdot 10^8 \text{in.}^4 \]

\[ \Delta_b = \frac{5(0.028 \text{kip/ in.})^{3600 \text{ in.}^4}}{384 \cdot [(2408 \text{ ksi})(7.3 \cdot 10^8 \text{in.}^4) + (29.000 \text{ ksi})(1.1 \cdot 10^7 \text{ in.}^4)]] = 0.029 \text{ in.} \]
3. Calculate mid-span deflection due to deformation of shear transfer at the collectors

\[ \Delta_z = \Delta_i = \frac{100 \, \text{kip}}{(145.3 \, \text{in}^{-1}) \cdot (19.2 \, \text{kip}) \cdot (132 \, \text{studs})} = 0.000272 \, \text{in.} \]

4. Calculate mid-span deflection due to shear

\[ \Delta_s = \frac{w \cdot a^2}{6 \cdot b \cdot G_s'} \]

Similar to Example 1, the calculation of \( G_s' \) will be performed in two ways:

- Using the proposed prediction model as described in Section 3.2 (Equation (3-16))

\[ G_s' = 22 \cdot L^{1.5} \cdot \frac{w}{t_c} = 22 \cdot (0.75)^{1.5} \cdot \left( \frac{(12)(0.0358 \, \text{in.})}{16 \, \text{in.}} + 4.75 \, \text{in.} \right) \cdot \sqrt{4000 \, \text{psi}} = 4317 \, \frac{\text{kip}}{\text{in}} \]

\[ \Delta_s = \left( \frac{0.026 \, \text{kip}}{\text{in}} \right) \cdot (3600 \, \text{in})^2 = 0.008 \, \text{in} \]

- Using the prediction model provided in AISI S310-20 (AISI, 2020) as described in Chapter 2 (Equation (2-1)).

\[ G_s' = K_3 = 3.5 \cdot d_c \cdot f_c^{0.7} = 3.5 \cdot (3.25 \, \text{in.}) \cdot (4000 \, \text{psi})^{0.7} = 3800 \, \frac{\text{kip}}{\text{in}} \]

\[ \Delta_s = \left( \frac{0.026 \, \text{kip}}{\text{in}} \right) \cdot (3600 \, \text{in})^2 = 0.01 \, \text{in.} \]

5. Calculate total mid-span deflection

Since the calculation for the deflection of the free end due to shear was performed in two ways, the total deflection is also calculated two ways:

- Current approach described in DDM04 (Luttrell, 2015) (only bending and shear deflections) using the prediction model provided in AISI S310-20 (AISI, 2020) for the calculation of initial shear stiffness

\[ \Delta_{tot} = \Delta_b + \Delta_z = 0.029 \, \text{in.} + 0.01 \, \text{in.} \]
\[ \Delta_{\text{tot}} = 0.039 \text{ in.} \quad \text{AISI S310-16} \]

- Proposed prediction model

\[ \Delta_{\text{tot}} = \Delta_b + \Delta_s + \Delta_z = 0.029 \text{ in} + 0.008 \text{ in} + 0.000272 \text{ in} \]

\[ \Delta_{\text{tot}} = 0.037 \text{ in.} \quad \text{Proposed Method} \]

The results of this example indicate that for the case of a simply supported diaphragm with a 3:1 aspect ratio, the proposed prediction model predicts similar deflections to the prediction model outlined in DDM04 (Luttrell, 2015). The majority of the deformations for this diaphragm configuration are bending deformations and the shear stiffness was similar between the proposed model and DDM04.

4.3 Summary

The results for the two design examples are summarized and discussed in this section. The components of the predicted deflection as well as the total predicted deflection for each example are summarized in Table 4-1.

<table>
<thead>
<tr>
<th>Example</th>
<th>Prediction Model</th>
<th>Predicted Deflection (in.)</th>
<th>Total Predicted Deflection (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Bending</td>
<td>Shear</td>
</tr>
<tr>
<td>Example 1. Cantilever Diaphragm</td>
<td>Proposed Prediction Model</td>
<td>0.0045</td>
<td>0.013</td>
</tr>
<tr>
<td></td>
<td>DDM04 (Luttrell, 2015)</td>
<td>0.0045</td>
<td>0.016</td>
</tr>
<tr>
<td>Example 2. Simply Supported Diaphragm</td>
<td>Proposed Prediction Model</td>
<td>0.029</td>
<td>0.008</td>
</tr>
<tr>
<td></td>
<td>DDM04 (Luttrell, 2015)</td>
<td>0.029</td>
<td>0.01</td>
</tr>
</tbody>
</table>
Discussion of Example 1 – Cantilever Diaphragm Specimen

The results of this example illustrate that for a cantilever diaphragm specimen with headed stud anchors placed with the largest spacing allowed by AISC 360 (AISC, 2016), the largest contribution to the deflection of the free end is due to the slip of the shear transfer connections (61% of total deflection for this example), followed by the shear deformations of the concrete-filled steel deck diaphragm (29% of total deflection for this example), with the bending deflections contributing 10% of the total deflection.

The results also illustrate that the proposed prediction model results in larger predicted deflections, even if both prediction methods result in similar shear deflections. The discrepancy is due to the additional deformation of the shear connections that is directly accounted for in the proposed prediction method.

Discussion of Example 2 Simply Supported Diaphragm

The results of this example illustrate that for this case of a simply-supported diaphragm configuration, the largest contribution to the mid-span deflection is due to bending (78% of total deflection for this example), followed by the shear deformations of the concrete-filled steel deck diaphragm (21% of total deflection for this example), with the shear transfer deflections contributing 0.8% of the total deflection. The results also illustrate that the approach described in DDM04 (Luttrell, 2015) using the prediction model provided in AISI S310-20 (AISI, 2020) for the shear stiffness of the diaphragm results in similar total deflections when compared to the proposed prediction model. This is because both methods calculate the bending deflections in the same way, which account for the majority of the total deflection. Therefore, the differences in
predicted shear deformations and deformation of the shear transfer connections are not as pronounced as for the case of a cantilever diaphragm.

The results given by the proposed prediction model are consistent with deflections obtained from commercial structural analysis software. A structural analysis model of this structure was produced by Swecker (2021) in RAM Structural System v17.02. The diaphragm was modeled as semi-rigid with elastic membrane elements having thickness equal to the transformed slab thickness, \( t_e \). The analysis yielded a mid-span diaphragm deflection of 0.037 in. relative to the edges of the diaphragm, which is consistent with the mid-span deflection due to shear and bending calculated using the proposed prediction model (0.037 in.). The structural analysis software doesn’t include the contribution of the deformation of the shear transfer connection. The agreement between the structural analysis and the proposed prediction model suggests that the proposed prediction model produces consistent results compared to the approach of some practicing engineers to calculate diaphragm deflections using structural analysis.
5 Conclusions and Recommendation

5.1 Summary

This report outlined the efforts in understanding and predicting the initial stiffness of concrete-filled steel deck diaphragms by utilizing the experimental results of a cantilever diaphragm testing program performed at Virginia Tech as well as existing experimental data and prediction models in the literature. This report includes the details of a proposed prediction model for the initial shear stiffness of concrete-filled steel deck diaphragms. The proposed model is based on a model put forward by Porter and Easterling (1988) in which the initial stiffness of the diaphragm should be calculated based on the total deformation of the diaphragm.

5.2 Proposed Stiffness for Concrete-Filled Steel Deck Diaphragms

The total deformation of a diaphragm is calculated using Equation (5-1), which includes bending and shear deformations of the diaphragm as well as the slip of the shear transfer connections (if deemed significant).

\[ \Delta_{tot} = \Delta_s + \Delta_b + \Delta_z \]  \hspace{1cm} (5-1)

Where,

\[ \Delta_{tot} = \text{Total deflection of concrete-filled steel deck diaphragm} \]
\[ \Delta_s = \text{Deflection of concrete-filled steel deck diaphragm due to shear} \]
\[ \Delta_b = \text{Deflection of concrete-filled steel deck diaphragm due to bending} \]
\[ \Delta_z = \text{Deflection of concrete-filled steel deck diaphragm due to deformations of the shear transfer connections} \]
For the case of a simply supported diaphragm with an applied distributed load (See Figure 4-2), the midspan deflection due to shear can be calculated using Equation (5-2).

\[ \Delta_s = \frac{w \cdot a^2}{8 \cdot b \cdot G_s'} \]  

(5-2)

Where,

\( w \) = distributed load applied to simply supported diaphragm

\( a \) = span of diaphragm “deep beam”

\( b \) = depth of diaphragm “deep beam”

The shear stiffness of the concrete-filled steel deck diaphragm is given by Equation (5-3).

\[ G_s' = 22 \cdot \lambda^{15} \cdot t_e \sqrt{f_c'} \left( \frac{kip}{in} \right) \]  

(5-3)

Where,

\( \lambda \) = factor for lightweight concrete

\( = 0.75 \) for \( w_c = 110 \) pcf

\( = 1 \) for \( w_c = 145 \) pcf

\( t_e = t_{cd} + t_c \), effective thickness of the concrete-filled steel deck diaphragm, in.

\( t_{cd} \) = transformed thickness of the steel deck, in.

\( t_c \) = average concrete thickness considering deck geometry, in.

\( f_c' \) = compressive strength of concrete, psi

The midspan deflection of a simply supported diaphragm span with uniform loading due to bending deformations can be calculated using Equation (5-4).

\[ \Delta_b = \frac{5 \cdot w \cdot a^4}{384 \cdot (E_sI_s + E_cI_c)} \]  

(5-4)
Where,

\[ E_c = \text{modulus of elasticity of concrete} \]
\[ E_b = \text{modulus of elasticity of steel} \]
\[ I_c = \text{moment of inertia of concrete-filled steel deck diaphragm with respect to the centroidal axis of the assembly} \]
\[ I_b = \text{moment of inertia of the steel beams oriented perpendicular to the loading with respect to the centroidal axis of the assembly} \]

The deformation of the shear transfer connections for a simply supported diaphragm can be calculated using Equation (5-5), as the deformation of the connectors on the collectors.

\[ \Delta_z = \frac{w \cdot a}{145.8 \cdot Q_{su} \cdot n_s} \text{ (in.)} \]  

(5-5)

Where,

\[ Q_{su} = \text{capacity of headed stud anchors in load direction, kip} \]
\[ n_s = \text{number of headed stud anchors in load direction along the collector} \]

The following is a list of recommendations for design:

- It is proposed that the shear stiffness of concrete on steel deck diaphragms, \( G' \), be revised in AISI S310 to match Equation (5-3). This equation is mechanics based instead of empirical, produces results that are consistent with structural analysis models, and has been shown to match experimental data.

- For typical diaphragm configurations, spans, and typical headed stud anchor size and spacing, the shear transfer deformation associated with the headed stud anchors in load direction along the collector...
anchors may be neglected. This recommendation is based on the results of the example diaphragm analysis. For diaphragm configurations with small aspect ratios (span to depth), deformation associated with shear transfer would make up a larger portion of the deformation, but in these cases, diaphragm deflection will likely be small and inconsequential. For especially unusual diaphragm configurations, or diaphragms with shear transfer connections more flexible than headed shear studs, it may be worthwhile to check the contribution of shear transfer to diaphragm deflections.

- For diaphragms modeled using elastic membrane elements in a structural analysis model (i.e. semi-rigid diaphragm analysis), it is reasonable to use the transformed average concrete thickness and typical concrete material properties. The resulting diaphragm deflection should match the results of hand calculations using the proposed equation for $G'$ as demonstrated by the results of Example 2.

- Based on the results presented in this report, the prediction model provided by AISI S310-20 (AISI, 2020) for shear stiffness of concrete on steel deck diaphragms results in smaller stiffness than expected of an actual diaphragm. However, because the shear deformations are only one component of diaphragm deflection, the total diaphragm displacement was shown to be similar to the proposed model in Example 2. Furthermore, the magnitude of the diaphragm deflections for concrete on steel deck assemblies are relatively small when compared to the story drifts associated with the rest of the lateral force resisting system (e.g. braced frame, moment frame). For these reasons, diaphragm
deflections calculated using the shear stiffness in AISI S310-20 are likely not problematic.

5.3 Conclusions

Existing equations for shear stiffness, $G'$, of concrete-filled steel deck diaphragms were empirical and may have been calibrated to experimental data which included additional sources of flexibility (such as bending deformations, deformation of the shear transfer connections, and deformation of the experimental support reactions). The proposed prediction model separately accounts for each source of flexibility and the proposed equation for shear stiffness $G'$, is mechanics-based and accounts solely for the shear deformation of the concrete-filled steel deck diaphragm.

Predicted stiffness values using the proposed model were compared to the measured initial stiffness of cantilever diaphragm specimens from two experimental programs. The average ratio of predicted stiffness to experimentally measured stiffness was found to be 0.98 with a standard deviation of 0.21. Based on this comparison, the prediction model provides a reasonably accurate estimate of initial shear stiffness for concrete-filled steel deck diaphragms.

This report also includes two examples to illustrate how the proposed prediction model can be used when calculating diaphragm deflections for two different diaphragm configurations. The results showed that for a small cantilever diaphragm configuration, similar to test configurations, the deflection of the free end is mostly due to the shear deformation of the concrete-filled steel deck diaphragm and to the deformation of the shear transfer connection. For the example simply supported diaphragm, representative of a building floor, the mid-span deflection was attributed primarily to bending deformations (78% of total deflection), and shear
deformations (21% of the total deflection), while the deformation of the shear transfer connections contributed less than 1% of the total deflection.
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