Steel deck diaphragms: Characterizing the hysteretic behavior of light gage steel, screw-fastened, support and sidelap connections and the influence of support connections on the stability behavior of panels

Divyansh Kapoor

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STEEL DECK DIAPHRAGMS: CHARACTERIZING THE HYSTERETIC BEHAVIOR OF LIGHT GAGE STEEL, SCREW-FASTENED, SUPPORT AND SIDELAP CONNECTIONS AND THE INFLUENCE OF SUPPORT CONNECTIONS ON THE STABILITY BEHAVIOR OF PANELS

A Dissertation Presented

by

DIVYANSH R. KAPOOR

Submitted to the Graduate School of the University of Massachusetts Amherst in partial fulfillment of the requirements for the degree of

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Civil Engineering: Structural Engineering and Mechanics
STEEL DECK DIAPHRAGMS: CHARACTERIZING THE HYSTERETIC BEHAVIOR OF LIGHT GAGE STEEL, SCREW-FASTENED, SUPPORT AND SIDELAP CONNECTIONS AND THE INFLUENCE OF SUPPORT CONNECTIONS ON THE STABILITY BEHAVIOR OF PANELS

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To my loving family
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The last few years of my life have been an incredible journey full of learning, growing, laughter, tears, and self-realization. As I conclude this exciting chapter, I feel immensely grateful towards the following people who guided, inspired, and supported me through it all:

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ABSTRACT

STEEL DECK DIAPHRAGMS: CHARACTERIZING THE HYSTERETIC BEHAVIOR OF LIGHT GAGE STEEL, SCREW-FASTENED, SUPPORT AND SIDELAP CONNECTIONS AND THE INFLUENCE OF SUPPORT CONNECTIONS ON THE STABILITY BEHAVIOR OF PANELS

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Steel deck diaphragms are planar structures that serve as critical components of a metal building’s lateral force-resisting system. These can be found in walls, floors, and roofs of metal buildings and aid with the lateral stability of the structure. The diaphragm design process involves the selection of materials, deck geometry, connections, and connection detailing across the steel panels. The American Iron and Steel Institute’s AISI S310 and Steel Deck Institutes DDM04 both provide design guidance on the selection and detailing of materials and fasteners for the applicable limit states of connections and stability of the panels. Both design codes provide uniform guidance in the treatment of these limit states. This dissertation deals with specific aspects of the connection and stability limit state fastener and material selection process, specifically focusing on the strength and hysteretic behavior of light-gage steel connections (22 gage, 24 gage, and 26 gage) and the stability behavior of steel panels due to varying support attachment patterns. To investigate the hysteretic behavior of light gage connections, 27 cyclic tests were conducted using the FEMA 461 Interim Protocol II, quasistatic cyclic displacement-controlled loading
protocol. The tests varied in ply thickness (22 gage, 24 gage, 26 gage) and support framing member thickness (14 gage and 18 gage). Results from the 27 cyclic connection tests have been presented and discussed to determine available capacity, evaluate performance of predictive design equations, and track progression of failure in the light gage connections. Simplified multi-point backbone curves have been fit and presented for incorporation into finite element analysis models and future numerical simulations of connection behavior. To determine the influence of varying support attachment pattern (end connectivity) on the out-of-plane buckling capacity of steel deck panels, 9 full-scale monotonic diaphragm tests were conducted. These tests were identical in deck profile (Type B Deck), deck gage (22 gage), span length (15 ft), support framing member thickness (16 gage), and only varied in the number of support connections per panel at the panel ends. The support attachment patterns evaluated cover a range of industry standard attachment patterns. Results from the experimental testing were utilized to calibrate finite element models and expand the study to two more commercially available deck thicknesses (20 gage and 18 gage). Results from the 9 full-scale diaphragm tests and 9 numerical simulations have been presented and discussed to determine the influence of end connectivity on peak buckling capacity, initiation of buckling, and stiffness. Results from both studies have been discussed in the context of the research questions and future works have been recommended to further investigate these phenomena and expand the state of knowledge.
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<td>$A$</td>
<td>Number of exterior support connections per flute located at the sidelap at an interior or edge panel</td>
</tr>
<tr>
<td>$B$</td>
<td>Unit length of panel</td>
</tr>
<tr>
<td>$D$</td>
<td>Panel corrugation pitch</td>
</tr>
<tr>
<td>$d = \varphi$</td>
<td>Fastener diameter</td>
</tr>
<tr>
<td>$D_d$</td>
<td>Depth of panel</td>
</tr>
<tr>
<td>$D_x$</td>
<td>Strong axis flexural stiffness per unit width</td>
</tr>
<tr>
<td>$D_y$</td>
<td>Weak axis flexural stiffness per unit width</td>
</tr>
<tr>
<td>$E$</td>
<td>one-half of the bottom flat width of panels</td>
</tr>
<tr>
<td>$E$</td>
<td>Modulus of elasticity of steel</td>
</tr>
<tr>
<td>$F$</td>
<td>Top flat width of panel</td>
</tr>
<tr>
<td>$F_u$</td>
<td>Tensile strength of sheet</td>
</tr>
<tr>
<td>$F_{u1}$</td>
<td>Tensile strength of member in contact with screw or nail head or washer</td>
</tr>
<tr>
<td>$F_{u2}$</td>
<td>Tensile strength of member not in contact with screw or nail head or washer</td>
</tr>
<tr>
<td>$F_y$</td>
<td>Specified yield stress of steel</td>
</tr>
<tr>
<td>$G'$</td>
<td>Diaphragm stiffness</td>
</tr>
<tr>
<td>$G'_{exp}$</td>
<td>Diaphragm stiffness from experimental testing</td>
</tr>
<tr>
<td>$G'_{FEA}$</td>
<td>Diaphragm stiffness from numerical simulation</td>
</tr>
<tr>
<td>$h$</td>
<td>Flat dimension of web measured in plane of the web</td>
</tr>
<tr>
<td>$I_{sg}$</td>
<td>Moment of inertia per unit width</td>
</tr>
<tr>
<td>$L_v = L$</td>
<td>Span of panel between supports</td>
</tr>
<tr>
<td>$N$</td>
<td>Number of support connections per unit width at an interior or edge panel’s end</td>
</tr>
<tr>
<td>$N_{cr}$</td>
<td>Critical buckling capacity (Easley)</td>
</tr>
<tr>
<td>$n_d$</td>
<td>Number of support connections at any given flute bottom flat along the ends of interior or edge panels into exterior supports</td>
</tr>
<tr>
<td>$n_e$</td>
<td>Number of edge support connections equally distributed along an edge panel length</td>
</tr>
<tr>
<td>$N_{ext}$</td>
<td>Bearing length at exterior supports</td>
</tr>
<tr>
<td>$n_p$</td>
<td>Number of interior supports along a panel length</td>
</tr>
<tr>
<td>$n_s$</td>
<td>Number of sidelap connections along a total panel length and not into supports</td>
</tr>
<tr>
<td>$P_{exp}$</td>
<td>Experimental connection capacity $= \min(P_{max},</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>$P_{\text{max}}$</td>
<td>Peak connection strength in the first quadrant (positive loading)</td>
</tr>
<tr>
<td>$P_{\text{max,exp}}$</td>
<td>Peak capacity observed during panel buckling experimental test</td>
</tr>
<tr>
<td>$P_{\text{max,FEA}}$</td>
<td>Peak capacity observed during panel buckling numerical simulation</td>
</tr>
<tr>
<td>$P_{\text{min}}$</td>
<td>Peak connection strength in the third quadrant (negative loading)</td>
</tr>
<tr>
<td>$P_{\text{nb, AISI}}$</td>
<td>Predicted panel buckling capacity</td>
</tr>
<tr>
<td>$P_{nf}$</td>
<td>Nominal shear strength of a support connection per fastener limited by bearing and tilting</td>
</tr>
<tr>
<td>$P_{ns}$</td>
<td>Nominal shear strength of a sidelap connection per fastener limited by bearing and tilting</td>
</tr>
<tr>
<td>$P_{nv} = P_{nf} = P_{ns}$</td>
<td>Nominal shear strength of a support or sidelap connection per fastener limited by bearing and tilting</td>
</tr>
<tr>
<td>$P_{nv,m}$</td>
<td>Predicted shear strength of a support or sidelap connection per fastener limited by bearing and tilting based on measured properties</td>
</tr>
<tr>
<td>$P_{nv,n}$</td>
<td>Predicted shear strength of a support or sidelap connection per fastener limited by bearing and tilting based on nominal properties</td>
</tr>
<tr>
<td>$P_{nw}$</td>
<td>Nominal web crippling strength per web</td>
</tr>
<tr>
<td>$q_s$</td>
<td>Perforated web adjustment factor</td>
</tr>
<tr>
<td>$R$</td>
<td>Inside bend radius</td>
</tr>
<tr>
<td>$s$</td>
<td>Developed flute width per corrugation pitch</td>
</tr>
<tr>
<td>$S_n$</td>
<td>Nominal shear strength per unit length of diaphragm</td>
</tr>
<tr>
<td>$S_{nb}$</td>
<td>Nominal shear strength per unit length of diaphragm controlled by buckling</td>
</tr>
<tr>
<td>$S_{nc}$</td>
<td>Nominal shear strength per unit length of diaphragm controlled by support connections at corners of interior or edge panels</td>
</tr>
<tr>
<td>$S_{ne}$</td>
<td>Nominal shear strength per unit length of diaphragm controlled by connections along the edge parallel to panel span in an edge panel and located at a diaphragm reaction line</td>
</tr>
<tr>
<td>$S_{nf}$</td>
<td>Nominal shear strength per unit length of diaphragm controlled by connections</td>
</tr>
<tr>
<td>$S_{ni}$</td>
<td>Nominal shear strength per unit length of diaphragm controlled by connections at interior or edge panels</td>
</tr>
<tr>
<td>$S_{nl}$</td>
<td>Nominal shear strength per unit length of diaphragm controlled by local web buckling of panel over exterior supports</td>
</tr>
<tr>
<td>$S_{no}$</td>
<td>Nominal shear strength per unit length of diaphragm controlled by panel out-of-plane buckling</td>
</tr>
<tr>
<td>Symbol</td>
<td>Definition</td>
</tr>
<tr>
<td>----------</td>
<td>---------------------------------------------------------------------------</td>
</tr>
<tr>
<td>$S_{np}$</td>
<td>Nominal shear strength per unit length of diaphragm controlled by connections along the ends of an interior or edge panel and into exterior supports</td>
</tr>
<tr>
<td>$t$</td>
<td>Base steel thickness of panel</td>
</tr>
<tr>
<td>$t_1$</td>
<td>Ply thickness of member in contact with screw or nail head or washer</td>
</tr>
<tr>
<td>$t_2$</td>
<td>Ply thickness of member not in contact with screw or nail head or washer</td>
</tr>
<tr>
<td>$w$</td>
<td>Panel cover width</td>
</tr>
<tr>
<td>$w_e$</td>
<td>Panel cover width at an exterior panel</td>
</tr>
<tr>
<td>$w_t$</td>
<td>Greatest tributary width to any given bottom flute with support connection(s) along the end perpendicular to panel span and located at exterior support</td>
</tr>
<tr>
<td>$\alpha_1$</td>
<td>Measure of exterior support fastener group distribution across panel width at an edge panel</td>
</tr>
<tr>
<td>$\alpha_2$</td>
<td>Measure of interior support fastener group distribution across panel width at an edge panel</td>
</tr>
<tr>
<td>$\alpha_e$</td>
<td>Analogous section modulus of panel exterior support connection group in an interior or exterior panel</td>
</tr>
<tr>
<td>$\alpha_p$</td>
<td>Analogous section modulus of panel interior support connection group in an interior or exterior panel</td>
</tr>
<tr>
<td>$\alpha_s$</td>
<td>Ratio of diaphragm sidelap connection strength to support connection strength</td>
</tr>
<tr>
<td>$\beta$</td>
<td>Factor defining connection interaction contribution to diaphragm shear strength per unit length</td>
</tr>
<tr>
<td>$\beta$ (Easley)</td>
<td>Buckling coefficient allowing for end restraint</td>
</tr>
<tr>
<td>$\Delta_{exp}$</td>
<td>Displacement at which peak capacity was observed in the panel buckling experimental tests</td>
</tr>
<tr>
<td>$\Delta_{FEA}$</td>
<td>Displacement at which peak capacity was observed in the panel buckling numerical simulations</td>
</tr>
<tr>
<td>$\lambda$</td>
<td>Connection strength reduction factor at corner fasteners</td>
</tr>
</tbody>
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1. CHAPTER 1:
INTRODUCTION

1.1. Introduction to metal building diaphragms

Diaphragms (Figure 1-1) serve as a critical component of a building’s lateral force resisting system (LFRS), transferring lateral loads from the façade of the structure to the designated vertical lateral force resisting system. Typical bare-deck steel diaphragms are comprised of an underlying support frame, steel deck, and connections (Figure 1-1). Though deck profile can vary, the commonly used Type B deck can be seen in Figure 1-2. The design and detailing process requires the structural engineer to select the deck geometry, support framing members, connections and their distribution in the diaphragm. All these parameters influence the capacity and behavior of steel deck diaphragms and design guidance can be found in AISI S310 – 20 (AISI 2020), SDI DDM04 (Luttrell 2015), and SDI-SDCFSFDM (Sputo 2017). The design strength of diaphragms can be either restricted by the choice of connection detailing (i.e., choice of fastener, attachment pattern at interior and exterior supports, spacing along sidelaps and edge) or by the choice of deck selection (deck profile, deck thickness, deck grade). As the industry adopts thinner optimized deck profiles, it is vital to understand how reducing deck thickness impacts connection capacity as well as how connections themselves impact the out-of-plane buckling capacity of panels. The research presented herein experimentally and numerically evaluates the impact reducing deck thickness (22 gage and higher) has on connection strengths and reducing end connectivity (fully attached, intermediately attached, and skip patterns) has on the buckling capacity of panels. In the following subsections, the different limit states and resultant design space are introduced, motivation for the
research is presented in the form of research questions, and an overview of the dissertation is provided.

Figure 1-1: Typical diaphragm components

Figure 1-2: Typical corrugation dimensions for Type B Deck (Luttrell 2015)

1.2. Overview of diaphragm design limit states and resultant design space

The nominal in plane shear strength \( (S_n) \) of a corrugated metal deck diaphragm is either governed by the limit state of connection failure \( (S_{nf}) \) or the limit state of shear buckling \( (S_{nb}) \). Connection failure is characterized by the failure of one of the sidelap, support, or edge connections. Shear buckling failure is characterized by the out of plane wave like buckling of the steel panels or local
buckling of the corrugation’s webs near the supports. Test photos showing both these failure modes can be seen in Figure 1-3 below. Current design standards and manuals such as AISI S310-20 (AISI 2020), SDI DDM04 (Luttrell 2015), and SDI-SDCFSFDM (Sputo 2017) utilize the same equations for estimating these limit states. Both limit states along with some example design configurations have been discussed in the following subsections.

![Figure 1-3: Buckling and connection failures observed during testing (Kapoor et al. 2023)](image)

1.2.1. Connection limit states

The available nominal shear strength governed by connection limit states \( S_{nf} \) is controlled by connection detailing and local failure (bearing/tilting, pull out, shear failure of fastener) of either the edge, interior, or corner connections (Fig 1-1 and Fig 1-4). The available shear capacity is the minimum of the available capacities based on edge, interior, or corner connections \( S_{nf} = \min \{S_{ni}, S_{nc}, S_{ne}\} \). Schematic illustration of these connections and the critical parameters utilized in capacity estimation can be seen in Figure 1-4 below. The following subsections discuss these limit states and their critical parameters.
Figure 1-4: Schematic illustration of typical diaphragm connections and critical parameters utilized in capacity estimation [Adapted from AISI S310 -16 (AISI 2016a)]
1.2.1.1. Individual support and sidelap fastener strength

The available shear strength of the support/edge connections ($P_{nf}$) and side lap ($P_{ns}$) connections are critical inputs for the connection limit states calculations. A typical connection comprises of the two or more plies of steel being attached and the weld, bolt, screw, or powder-actuated fastener (PAF) used. AISI S100 Chapter J2, though Chapter J5 provide design guidance for these commonly used connection types.

Strength of typical screw fastened connections can be estimated from Chapter J.4.3.1 in AISI S100 (AISI 2016b), using Equations J4.3.1-1 through J4.3.1-5. The available nominal shear strength of connections as recommended by AISI S100 (AISI 2016b) Section J4.3 Shear, are controlled by tilting and bearing of the connections (Figure 1-5) and depend on the ultimate strength ($F_u$), fastener diameter ($d = \varphi$), and ply thickness ($t$). The governing failure mode i.e., whether tilting or bearing or an interactive mode will control capacity depends on the ratio of thickness of the plies being connected. These limit states can be calculated using Equations 1-1 through 1-5, where Equations 1-1 provides the tilting capacity and Equations 1-2 through 1-5 are the bearing capacities. Here $P_{nv}$ ($P_{nv} = P_{nf} = P_{nfs} = P_{ns}$) is the nominal shear strength of sheet per screw, $t_1$ and $t_2$ are thickness of the connected plies, $d$ is the diameter of the screw, and $F_{u1}$ and $F_{u2}$ are the tensile strength of plies.

When $t_2/t_1$ is less than or equal to 1, $P_{nv}$ is the minimum of

$$P_{nv} = 4.2 \left( t_2^3 d \right)^{0.5} F_{u2} \quad (1-1)$$

$$P_{nv} = 2.7 \ t_1 d F_{u1} \quad (1-2)$$

$$P_{nv} = 2.7 \ t_2 d F_{u2} \quad (1-3)$$
When \( t_2/t_1 \) is greater than or equal to 2.5, \( P_{nv} \) is the minimum of

\[
P_{nv} = 2.7 \ t_1 dF_{u1}
\]  \( \text{(1-4)} \)

\[
P_{nv} = 2.7 \ t_2 dF_{u2}
\]  \( \text{(1-5)} \)

When \( 1.0 < t_2/t_1 < 2.5 \), \( P_{nv} \) is calculated by linearly interpolating between the two cases.

![Figure 1-5: (a) Tilting vs (b) bearing failure mode (Shi et al 2018)](image)

To further understand the relationship between ply thickness, ply ultimate strength, and fastener diameter, a brief parametric analysis was conducted on the AISI S100-16 (AISI 2016b) design equations. First, the impact of ultimate strength and varying ply thickness on the bearing and tilting limit state capacities was evaluated for 45 ksi, 65 ksi, and 80 ksi deck. These results can be seen in Figure 1-6 below. It was observed that for the thicknesses of interest (i.e., 26 gage (0.0179 in) – 22 gage (0.0295 in)), tilting was the governing failure mode. Hence for sidelap connections (\( t_2/t_1 = 1.0 \)), fastener tilting is expected to control strength. Increasing \( F_u \) from 45 ksi to 65 ksi and 80 ksi also increases the tilting and bearing capacity but this capacity increase reduces with reducing deck thickness. Similarly, the influence of increasing fastener diameter was investigated to understand whether increasing fastener diameter can enhance the capacity of light gage steel connections (Fig 1-7). The bearing and tilting limit states were evaluated for a range of thicknesses with \( F_u = 80 \) ksi. For the gages of interest, it was observed that increasing fastener diameter
increased strength, but the increase in capacity reduced with decreasing thickness as the curves converge.

Figure 1-6: Influence of ply thickness and ultimate strength on the bearing and tilting capacity of connections (#12 fastener shown)

Figure 1-7: Influence of fastener diameter on the bearing and tilting capacity of connections (80 ksi deck shown)
For other connection types, such as arc spot welds and arc seam welds AISI S310 (AISI 2016a) references AISI S100-16 (AISI 2016b) design equations from sections J2.2.1 and J2.3.1. Other connections such as PAFs and proprietary connections are also permitted for use but AISI S310 recommends that capacity be established via testing. Deck and fastener manufacturers, such as NUCOR® and HILTI® respectively often provide predictive equations based on independent testing. For example, the available shear capacity of proprietary fasteners such as the PunchLok-II connection (VSC2) can be estimated using Equation 1-6 (IAPMO ER-0652 2018) provided by the deck manufacturer. Capacities of other proprietary fasteners can also be calculated in a similar manner by referring to SDI DDM04 (Luttrell 2015) or manufacturer reports. SDI DDM04 (Luttrell 2015) provides predictive equations for proprietary power-actuated fasteners and non-proprietary button-punched sidelap connections.

\[ P_{ns} = 137.42t - 2.01 \]  

(1-6)

1.2.1.2. Edge connection \((S_{ne})\)

The available shear strength per unit length from edge connections \((S_{ne})\) can be estimated by considering the contribution of all fasteners along the edge of an end panel and supports, and then averaging it across the length of the panel (Equation 1-7). This includes fasteners through both the interior and exterior supports and fasteners connected directly to the edge member. Interior and exterior support fasteners are accounted for by the measure of fastener group distribution across a panel width \((w_e)\) by \(\alpha_1\) and \(\alpha_2\) respectively. The remaining fasteners are directly accounted for by \(n_e\).

\[ S_{ne} = \frac{(2\alpha_1 + n_p \alpha_2)P_{nf} + n_e P_{nf}}{L} \]  

(1-7)
1.2.1.3. Interior connection ($S_{ni}$)

The available shear strength per unit length governed by the failure of interior connections ($S_{ni}$) accounts for connection detailing at interior or edge panels (Figure 1-4) and can be calculated using Equation 1-8 below. The number of fasteners through each flute at the supports ($A$), the distribution of fasteners across the panel (accounted for with $\beta$), and individual connection strength ($P_{nf}$) are all critical parameters for this calculation. Here, a relaxation term ($\lambda - 1$), is used to account for the reduced capacity of corner fasteners due to buckling at support connections along sidelaps at panel ends. $\beta$ (Equation 1-9) is the factor which defines connection contribution and interaction with the diaphragm shear strength. This further depends on the analogous section modulus of exterior ($\alpha_{e}^2$) and interior supports ($\alpha_{p}^2$), the number of sidelap connections ($n_{s}$) and connection strength ratio ($\alpha_{s}$).

\[
S_{ni} = [2A(\lambda - 1) + \beta] \frac{P_{nf}}{L} \quad (1-8)
\]

\[
\beta = n_{s}\alpha_{s} + 2n_{p}(\alpha_{p})^2 + 4(\alpha_{e})^2 \quad (1-9)
\]

1.2.1.4. Corner connection ($S_{nc}$)

The available shear strength per unit length controlled by the failure of the corner fastener ($S_{nc}$) can be estimated using Equation 1-10 below. The resultant of forces existing along the perpendicular and parallel edge of the deck is considered through $N$, $\beta$, and $L$ and related to individual connection strength to estimate the limit state. Here $N$ is the number of support fasteners per panel width ($w_{e}$), $\beta$ is as described in 1.2.1.3, and $L$ is the panel length.

\[
S_{nc} = \left(\frac{N^2\beta^2}{N^2L^2+\beta}\right)^{0.5} P_{nf} \quad (1-10)
\]
1.2.1.5. Connections along perpendicular edge ($S_{np}$)

The available shear strength per unit length controlled by the failure of the connections along the perpendicular edge ($S_{np}$) can be estimated using Equation 1-11 below. Here $n_d$ is the number of support fasteners per flute bottom and $w_t$ is the greatest tributary width to any support connection perpendicular to the span and located at the exterior support.

$$S_{np} = n_d P_{nf} \frac{1}{w_t}$$  \hspace{1cm} (1-11)

1.2.1.6. Sample connection limit states

The impact of changing attachment pattern on the connection limit states for a 22 gage (0.76 mm) Type B deck (Figure 1-2) of variable length can be seen in Figure 1-9 below. Here, two different types of support connections, namely #14 (6 mm) fasteners [$P_{nv} = 0.86$ kips (3.83 kN)] and 0.85 in (21.6 mm) spot welds [$P_{nv} = 2.56$ kips (11.4 kN)], were considered for the calculations. These connections were assumed to either have the 36/7, 36/5, or 36/4 attachment patterns (Figure 1-8). Edge and sidelay spacing was held constant at 6 inches (152.4 mm) on center and were made with #12 (5.3 mm) fasteners [$P_{nv} = .74$ kips (3.30 kN)]. The governing predicted failure mode for all of these configurations ($S_{nf} = \min[S_{nc}, S_{ni}, S_{nc}]$) was corner fastener failure ($S_{nc}$) and the capacity indicated in Figure 1-9 below is this capacity.

Figure 1-8: Typical support attachment patterns analyzed for resultant design space
1.2.2. Panel buckling limit states ($S_{nb}$)

The available nominal shear strength governed by panel buckling limit states ($S_{nb}$) is the minimum of the out-of-plane buckling capacity of the panel ($S_{no}$) and the local buckling capacity of the corrugation web over the supports ($S_{nl}$). The following subsections discuss these limit states and their critical parameters.

1.2.2.1. Global (Out-of-plane) buckling ($S_{no}$)

The nominal shear strength per unit length of a diaphragm system controlled by out-of-plane buckling (Figure 1-10) of the panel can be estimated using Equation 1-12. Here, the panel is assumed to be simply supported and capacity depends upon span length ($L_v$), moment of inertia per unit width ($I_{xg}$), thickness ($t$), corrugation pitch ($d$), and the developed flute width per pitch ($s$).
The developed flute width is the flat width of sheet steel required to form an individual corrugation. The variation in $S_{no}$ due to increasing span length for a 22 gage (0.76 mm) Type-B deck can be seen in Figure 1-12.

$$S_{no} = \frac{7890}{\alpha L_v^2} \left( \frac{l_{kg}^3 t^3 d}{s} \right)^{0.25}$$  \hfill (1-12)

1.2.2.2. Local buckling (Web crippling) ($S_{nl}$)

Local buckling in steel panels can be observed over the exterior supports and occurs after significant end warping of the panel ends. Both slenderness of the web and bearing length ($N_{ext}$) are important parameters that influence this diaphragm limit state. The failure mode can be seen in Figure 1-11 and can be estimated using Equations 1-13 and 1-14 below.

$$S_{nl} = P_{nw} \left( \frac{d-e}{D_d} \right) \left( \frac{1}{d} \right)$$  \hfill (1-13)
\[ p_{nw} = 4.36 t^2 F_p \sin \theta \left( 1 - 0.04 \sqrt{\frac{R}{t}} \right) \left( 1 + 0.25 \sqrt{\frac{N_{ext}}{t}} \right) \left( 1 + 0.025 \sqrt{\frac{R}{q_{st}}} \right) \] (1-14)

Here, \( d, e, D_d, R, t, \) and \( h \) are corrugation pitch, half bottom flat width, panel depth, corner radii, thickness, and flat dimension of the web respectively and have been depicted for a typical trapezoidal corrugation in Figure 1-2.

![Image](image.png)

Figure 1-11: Web crippling failure at panel ends (Avci and Easterling 2002)

1.2.3. Resultant design space

The unfactored capacity resultant design space for a 22 gage (0.76 mm) Type B deck due to connection and stability limit states can be seen in Figure 1-12 below. For the local buckling calculations, \( N_{ext} \) was assumed to be the minimum recommended value of 0.75 in (19 mm) and corner radii were conservatively neglected. Sidelap and edge fastener spacings were set to 6 in (152.4 mm) on center. Here, for typical span lengths [<7.5 ft (2286 mm)] the governing limit state is either local buckling \( (S_{nb}) \) or connections \( (S_{nf}) \) for the welded and screw-fastened connections respectively. \( S_{nb} \) does not govern design until 9.6 feet (2926 mm) and 14 feet (4267 mm) of span length for the for the evaluated configurations.
1.3. Research questions addressed through dissertation

1.3.1. How does individual connection strength, $P_{nf}$, vary due to reducing deck and support framing member thickness in screw fastened connections? What is the hysteretic behavior of these light-gage connections?

As can be seen from the guidance provided in design codes, accurately characterizing individual connection strength is a critical step in determining a metal building diaphragm’s capacity as limited by connection limit states. There exists a wealth of literature focusing on steel-to-steel connections behavior, and a variety of fasteners such as screw fastened (Tao et al. 2017, Torabian and Schafer 2021, Pham and Moen 2015, Rogers and Tremblay 2003), arc seam and spot welds.
(Torabian and Schafer 2021, Rogers and Tremblay 2003), and powder actuated fasteners (Torabian and Schafer 2021, Rogers and Tremblay 2003) have already been investigated. However, corrugated deck sidelap and structural tests with screw fastened connections for gages higher than 22 gage are not present in literature. To expand the state of knowledge on connection behavior, 27 cyclic connection tests were conducted, and results have been reported and analyzed. These tests vary in deck gage (22, 24, and 26 gage) and underlying support thickness (14 and 18 gage) and examine the thinner end of commercially available deck thicknesses. #10 – 16 – 3/4 inch and #12 – 16 – 3/4-inch hex head self-drilling and self-tapping screws and used to construct the sidelap and structural connections respectively. The fabricated connections were tested using the FEMA 461 Interim Protocol II, quasistatic cyclic displacement-controlled loading protocol (FEMA 2007).

1.3.2. Does end connectivity (support attachment) influence the out-of-plane buckling capacity of corrugated steel deck diaphragms?

Connection detailing and individual connection strength are examined in detail when determining the available shear strength as restricted by connection failure. In contrast, the panel out-of-plane buckling limit state only considers cross section geometric and base metal material properties when determining the available shear strength. The support connections are assumed to be nearly pinned (Panel buckling factor, $\beta = 1.07$) and the influence of connections along the edges of the panels (sidelap locations) is also neglected. Existing literature focusing on the strength and stiffness behavior of metal building diaphragms (O’Brien et al. 2017, Moen et al 2016, Ammar and Nilson 1971, Wright and Hossain 1997, Luttrell 1973, Duerr and Saal 2006) is also dominated by tests limited by connection strength and limited data exists on diaphragms failing due to panel buckling (Nunna 2011, Pinkham 1999). Industry practice (Nucor Roof Deck Design Tool) and limited literature (Wright and Hossain 1997) suggest that end attachment can have a detrimental impact
on available buckling capacity (25% and 50% respectively). However, no comprehensive test series exists where specimens were designed to isolate the influence of support attachment pattern on the buckling capacity. To determine the influence of support attachment pattern on the out-of-plane buckling limit state, 9 full-scale (10 ft X15 ft) specimens were designed and tested. These specimens varied in support attachment pattern (36/7 (fully attached), 36/5 (intermediately attached), and 36/4 (skip pattern)), and were constructed with 22 gage Type B deck. Results from the experimental testing were analyzed to determine peak capacity as well as onset of buckling to assess the adequacy of the existing panel buckling equation (Equation 1-12). These results were also used to calibrate and numerically evaluate the influence of reducing support connections on two more commercially available deck gages (18 gage and 20 gage).

1.4. Organization of dissertation

The dissertation is organized into 5 chapters and 8 appendices.

- **CHAPTER 1: INTRODUCTION**
  This chapter introduces metal building diaphragms, applicable design limit states, sample design space, and topics and motivations of the research presented in this dissertation. Motivation is presented in the form of two research questions which are addressed in the subsequent chapters. An overview of the organization of the dissertation is also presented at the end.

- **CHAPTER 2: CHARACTERIZING THE HYSTERETIC BEHAVIOR OF LIGHT GAGE STEEL-TO-STEEL SIDE LAP AND FRAME CONNECTIONS**
  This chapter focuses on the first research question, “How does individual connection strength, $P_{nf}$, vary due to reducing deck and support framing member thickness in screw fastened connections? What is the hysteretic behavior of these light-gage connections?”
and presents background information, review of existing works, as well as the experimental test program and results and conclusions. The results from three sidelap and six structural (frame) fastener tests with three repetitions each are presented, multi-point backbone curves are determined which are utilized to determine peak force and strain energy. Progression of failure in the light gage connection tests is documented and presented. Adequacy of existing design equations is also evaluated by comparing tested strengths with predicted strengths.

- **CHAPTER 3: DETERMINING THE INFLUENCE OF SUPPORT ATTACHMENT PATTERN OF THE OUT-OF-PLANE BUCKLING CAPACITY OF LIGHT-GAGE STEEL DECK DIAPHRAGMS**

This chapter focuses on the second research question, “Does end connectivity (support attachment) influence the out-of-plane buckling capacity of corrugated steel deck diaphragms?” and presents a review of existing works, experimental test matrix, test procedure, test results, numerical modelling matrix, FEA modelling methodology, and numerical results. Results from the 9 monotonic tests are presented and utilized to calibrate finite element models to expand to 6 untested configurations. Results from the numerical and experimental investigations are utilized to evaluate the performance of predictive equations and assess the impact changing attachment patterns has on strength (ultimate and initiation of buckling), stiffness, and end warping behavior.

- **CHAPTER 4: DISCUSSION OF RESEARCH QUESTIONS**

This chapter reviews the results obtained through experimental and numerical evaluations presented in Chapter 2 and Chapter 3 in context of the research questions.

- **CHAPTER 5: RECOMMENDED FUTURE WORKS**
This chapter presents and discusses future works recommended to better understand the connection and stability behavior of light-gage corrugated deck diaphragms as well as future refinements and expansions of the works presented herein.

- **APPENDICES**
  - Appendix A: Individual connection test force-displacement and deck slip results
  - Appendix B: Individual connection test field observation notes
  - Appendix C: Coupon testing results
  - Appendix D: Diaphragm test rig specifications
  - Appendix E: Sensor layout and loading protocol for panel buckling tests
  - Appendix F: Detailed per specimen results from panel buckling experiments
  - Appendix G: Estimation of initiation of buckling
  - Appendix H: Detailed numerical results
2. CHAPTER 2:

CHARACTERIZING THE HYSTERETIC BEHAVIOR OF LIGHT GAGE STEEL-TO-STEEL SIDELAP AND FRAME CONNECTIONS

2.1. Introduction

Connection behavior influences diaphragm behavior and capacity and is a critical parameter in diaphragm design and numerical simulations. Diaphragm connections can be categorized into sidelap connections (Figure 2-1), those which connect deck panels with one another, and structural (frame) connections (Figure 2-1) which connect the panels to the underlying structural framing members. Design guidance on the strength and flexibility of these connections can be found in AISI S100 (AISI 2016b) and in SDI DDM04 (Luttrell 2015) and AISI S310 (AISI 2016a) respectively. The strength equations have been reproduced and discussed in Section 1.2.1.1, Individual support and sidelap connection strength.

Figure 2-1: Typical sidelap and structural (frame) connections in a steel deck diaphragm
The behavior of these connections has been the focus of many experimental studies: Torabian and Schafer (Torabian and Schafer 2021), Torabian and Fratamico (Torabian and Fratamico 2018), and Torabian et al. (Torabian et al 2018) investigated light gage steel deck sidelap and structural framing connections; Shi et al (Shi et al 2018), Pham and Moen (Pham and Moen 2015), and Tao et al (Tao et al 2017) investigated the behavior of flat sheet to sheet connections, but corrugated deck sidelap and structural tests with screw fastened connections for gages higher than 22 are not present in literature.

To bridge this gap and further the understanding of light gage steel connection hysteretic behavior, 27 cyclic deformation-controlled connection tests were conducted. These tests are comprised of three sidelap configurations and six support fastener configurations with three repetitions each. Type B deck (Fig 2-2), 22, 24, and 26 gage flutes were tested with #10 fasteners for the sidelap configurations. Flutes with the same thickness were tested with 43 mil (18 gage) and 68 mil (14 gage) light gage strap for the structural (frame) connections. The objective of this work is to characterize the hysteretic behavior of tested connections, provide comparison with code predictions for strength, and estimate backbone curve parameters.

![Figure 2-2: Typical corrugation dimensions for Type B Deck (Luttrell 2015)](image)
2.2. Relevant works

2.2.1. Torabian and Schafer (2021)

Torabian and Schafer (Torabian and Schafer 2021) experimentally evaluated the seismic performance of a suite of sidelap and structural connections commonly used in steel deck construction. The experimental test matrix comprised of a total of 64 sidelap and 60 structural connections. The deck gages evaluated were 22, 20, and 18 gage and support framing thickness was 3/16 in and 3/8 in. The sidelap connections included in the study were #10 and #12 screw fasteners, top arc seam welds, button punch, and the PunchLok II connection in two configurations. Support connections were made with either arc spot welds, arc seam welds or powder actuated fasteners (PAFs). The test apparatus used in the study can be seen in Figure 2-3 and specimen were loaded using the FEMA 461 (FEMA 2007) displacement-controlled loading protocol.

Screw fastened connections had capacities ranging from 0.85 to 1.26 kips (3.5 to 5.6 kN) vs PunchLok-II – 2.05 to 5.56 kips (9.1 to 24.7 kN) or arc seam welds – 2.43 to 3.64 kips (10.8 to

Figure 2-3: (a) Sidelap and (b) Structural connection test setup (Torabian and Schafer 2021)
16.2 kN) (Torabian and Schafer 2021). The force-displacement response of the various connections were also processed into four-point backbone curves (Figure 2-4) to compare characteristics such as stiffness ($k_i$), strength ($P_{peak}$), ductility ($\mu$), minimum total deformation capacity, and residual strength ($P_4$). In the four-point backbone curve, Point 3 corresponds with peak capacity ($P_{peak}$, $\delta_p$), initial stiffness ($k_i$) * 20% of the peak load secant stiffness, Points 1, 2, and 4 are determined by balancing the energy under the four-point curve to the multi-point backbone. Point 4 also corresponds with the residual force level observed during testing. Ductility ($\mu$) is defined as the ratio of displacement at 80% of peak ($\delta_{pp80}$) to the estimated yield displacement ($\delta_p = P_p/k_i$). Minimum total deformation capacity is defined as the ratio of displacement at point 4 to the yield displacement ($\delta_4/\delta_y$).

![Figure 2-4: Four point backbone fit to cyclic backbone (Torabian and Schafer 2021)](image)

Some sample backbone curves from Torabian et al (2021 and 2017) have been reproduced below for comparing behavior of welded and screw fastened connections. Based on the above given definitions of ductility, initial stiffness and minimum total deformation capacity, screw fastened connections were observed to have modest capacity but significantly higher ductility and minimum
total deformation at failure (Figure 2-4). Ductility for the screw fastened connections ranged from 12.92 to 25.53 versus 2.08 to 3.73 for arc seam welds. Similarly, minimum total deformation capacity for the screw fastened connections ranged from 37.02 to 40.70 versus 4.95 to 9.07 for arc seam welds. Although the study considered and characterized a large suite of fastener types and deck gages, screw fastened support connections with deck thicknesses thinner than 22 gage were not included in this study. Furthermore, the study did not include support framing members thinner than 3/16 in.

Figure 2-5: Comparison of behavior of light gage sidelap screw fastened and top arc seam connections (Torabian and Schafer 2021)
2.2.2. Shi et al (2018)

Shi et al. (Shi et al 2018) experimentally tested 80 flat sheet sidelap and structural connections to remove the effect that corrugations and edge distances can have on the capacity of light gage connections. Two deck gages (20 and 18) were evaluated in combination with one support thickness (3/16 in). The sidelap connections were made with #10 and 12 screws and support connections were made with powder actuated fasteners (PAFs), pneumatic power actuated fasteners, arc seam welds and #12 screws. Additionally, 1, 2, and 4 ply connections were tested to simulate lapping of deck panels at ends and corners. The test apparatus used for testing can be seen in Figure 2-6 below and the specimens were tested both monotonically and cyclically using the FEMA 461 (FEMA 2007) protocol.

![Connection test setup (Shi et al 2018)](image)

In the screw fastened sidelap tests, tilting of the screws was observed which was followed by pullout of the screw as the screw threads pulled through the plies one at a time. Unlike the thicker 18 gage specimens, the thinner 22-gage specimens exhibited more sensitivity to fastener diameter.
and underwent a more severe reduction in stiffness due to tilting of the screws. The recorded capacities of the #12 fasteners were 40% higher than #10 fasteners. Failure mode of the screw fastened #12 support connections changed from pull out to bearing when loading protocol changed from monotonic to cyclic. This also had an impact on capacity (Figure 2-7.a) and a capacity drop was observed for the 18 gage specimens (Figure 2-7.b), but capacity increased for the 22-gage specimen. However, this trend was not observed in the companion Torabian and Schafer (Torabian and Schafer 2021) study and was attributed by the authors to shift in failure mode from bearing to tilting.

When results from the flat sheet sidelay tests were compared with results from the corrugated flute tests by Torabian and Schafer (Torabian and Schafer 2021), the corrugated tests had an average increase of 14% in capacity. This difference was hypothesized to be due to stiffening of the material at the corner of the corrugations and cold-working effects.

Figure 2-7: Response of (a) #12 support and (b) #10 sidelay screw fastened connection (Shi et al. 2018)
2.2.3. Tao et al (2017)

Tao et al (Tao et al 2017) evaluated and characterized the behavior of light gage steel screw fastened connections between steel-to-steel plies, steel plies with wood sheathing, and steel plies with OSB sheathing. 25, 20, 18, 16, 14, and 12 gage flat steel plies were tested with #8, #10, and #12 fasteners resulting in a total of 222 steel-to-steel tests. The specimens were tested monotonically and cyclically using the FEMA 461 protocol, and the cyclic behavior of the connections was characterized through backbone curves and Pinching 04 – OpenSees models. The predicted hysteretic behavior from Pinching 4 models was found to be in excellent agreement with experimental data (Figure 2-8) and a methodology for estimating backbone and pinching 4 parameters was presented.

![Figure 2-8: Example response of Pinching 4 model and comparison with test data (Tao et al 2017)](image)

Although the works discussed herein characterized various connection types such as welds, screws, PAFs with various ply thicknesses in the range of 12 gage to 25 gage in the flat sheet and deck flute configurations, steel deck flute tests in gages higher than 22 gage with screw fastened connections were not present. To add to existing literature, 24 and 26 gage deck were selected for the experimental test program presented in the dissertation. The 22 gage deck was also included.
in the study to provide overlap with existing literature for comparisons. Further, to simulate steel deck to light gage steel framing connections, the support thicknesses were also selected to be 18 gage (43 mil) and 14 gage (68 mil). The next subsection presents the detailed experimental test matrix.

2.3. Experimental test matrix

The experimental test matrices for sidelap and structural (frame) support connections can be seen in Table 2-3 and Tables 2-4 through 2-5 respectively. Here the reported \( P_{nv} \) was calculated using the methodology described in Section 1.2.1.1 with nominal \( (P_{nv,n}) \) and measured \( (P_{nv,m}) \) material properties. ASTM E8 (ASTM 2008) compliant coupon tests were conducted to estimate the measured material properties of the steel utilized in the sidelap and structural connection tests. The steel was donated by industry sponsors and was sourced from two different plants. All 22-gage decks were provided by Manufacturer 1 and all 24 and 26 gage decks were provided by Manufacturer 2. Three coupons were cut and tested for each ply thickness from the web flats and tested in an Instron 11.5 kips universal testing machine. The average of these repetitions for each gage have been reported in Table 2-1 below. Per-test results can also be found in Appendix C.

<table>
<thead>
<tr>
<th>Gage</th>
<th>Thickness (in)</th>
<th>( F_y ) (ksi)</th>
<th>( \varepsilon_y ) (%)</th>
<th>( F_u ) (ksi)</th>
<th>( \varepsilon_u ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>0.0680</td>
<td>32.06</td>
<td>0.20</td>
<td>48.38</td>
<td>23.46</td>
</tr>
<tr>
<td>18</td>
<td>0.0488</td>
<td>18.31</td>
<td>0.20</td>
<td>39.54</td>
<td>30.04</td>
</tr>
<tr>
<td>22</td>
<td>0.0305</td>
<td>51.56</td>
<td>0.20</td>
<td>81.96</td>
<td>11.74</td>
</tr>
<tr>
<td>24</td>
<td>0.0253</td>
<td>60.01</td>
<td>0.20</td>
<td>99.47</td>
<td>1.52</td>
</tr>
<tr>
<td>26</td>
<td>0.0203</td>
<td>58.24</td>
<td>0.20</td>
<td>95.76</td>
<td>0.82</td>
</tr>
</tbody>
</table>
Due to the significantly low elongations ($\varepsilon_u < 3.0\%$) observed while testing the 24 and 26 gage steel decks, AISI S100 (AISI 2016b) recommendations from Section A3.1.3 for steels with minimum elongation of less than three percent were considered when estimating capacities. AISI S100 (AISI 2016b) permits the use of low elongation steels but restricts the ultimate strength ($F_u$) to the lesser of 62 ksi or 75% of specified value. Measured and nominal capacities reported in Table 2-3 through 2-5 account for this reduction. Nominal material properties, ply thicknesses, and fastener diameters from AISI S310 (AISI 2016a) and DDM04 (Luttrell 2015) were used for calculating the nominal capacities of the tested connections. During these calculations, $F_u$ was restricted to $F_{u,\text{calculation}}$ to account for low elongation in 24 and 26 gage. These properties have been summarized below in Table 2-2.

<table>
<thead>
<tr>
<th>Description</th>
<th>Thickness (mil)</th>
<th>Thickness (gage)</th>
<th>Thickness (in)</th>
<th>$F_{u,\text{specified}}$ (ksi)</th>
<th>$F_{u,\text{calculation}}$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>22 gage deck</td>
<td>30</td>
<td>22</td>
<td>0.0295</td>
<td>45</td>
<td>45</td>
</tr>
<tr>
<td>24 gage deck</td>
<td>24</td>
<td>24</td>
<td>0.0239</td>
<td>80</td>
<td>60</td>
</tr>
<tr>
<td>26 gage deck</td>
<td>18</td>
<td>26</td>
<td>0.0179</td>
<td>80</td>
<td>60</td>
</tr>
<tr>
<td>43 mil strap</td>
<td>43</td>
<td>16</td>
<td>0.045</td>
<td>65</td>
<td>65</td>
</tr>
<tr>
<td>68 mil strap</td>
<td>68</td>
<td>14</td>
<td>0.071</td>
<td>65</td>
<td>65</td>
</tr>
</tbody>
</table>

2.3.1. Sidelap (deck to deck) connections

The sidelap specimen comprises of two pieces of nestable steel deck flutes, 36 inches long, fastened together through the center of the bottom flats using #10 screws placed 18 inches on center as can be seen in Figure 2-9. The expected capacities of these specimens based on nominal and measured properties and can be seen in Table 2-3.
Table 2-3: Sidelap test matrix and expected capacities

<table>
<thead>
<tr>
<th>ID</th>
<th>Deck Thickness (ga.)</th>
<th>Deck Grade (ksi)</th>
<th>Fastener (No.)</th>
<th>$P_{nv,n}$ (kips)</th>
<th>$P_{nv,m}$ (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>22</td>
<td>45</td>
<td>10</td>
<td>0.42</td>
<td>0.76</td>
</tr>
<tr>
<td>2</td>
<td>22</td>
<td>45</td>
<td>10</td>
<td>0.42</td>
<td>0.76</td>
</tr>
<tr>
<td>3</td>
<td>22</td>
<td>45</td>
<td>10</td>
<td>0.42</td>
<td>0.76</td>
</tr>
<tr>
<td>4</td>
<td>24</td>
<td>80</td>
<td>10</td>
<td>0.41</td>
<td>0.41</td>
</tr>
<tr>
<td>5</td>
<td>24</td>
<td>80</td>
<td>10</td>
<td>0.41</td>
<td>0.41</td>
</tr>
<tr>
<td>6</td>
<td>24</td>
<td>80</td>
<td>10</td>
<td>0.41</td>
<td>0.41</td>
</tr>
<tr>
<td>7</td>
<td>26</td>
<td>80</td>
<td>10</td>
<td>0.26</td>
<td>0.26</td>
</tr>
<tr>
<td>8</td>
<td>26</td>
<td>80</td>
<td>10</td>
<td>0.26</td>
<td>0.26</td>
</tr>
<tr>
<td>9</td>
<td>26</td>
<td>80</td>
<td>10</td>
<td>0.26</td>
<td>0.26</td>
</tr>
</tbody>
</table>

Figure 2-9: Typical sidelap connection specimen
2.3.2. Frame (structural) connections

Typical frame connection specimen can be seen in Figure 2-10. This comprises of a single piece of deck flute, 36 inches long, connected to a piece of steel strap, using #12 fasteners placed 18 inches on center. The expected capacities of these specimens based on nominal and measured properties can be seen in Table 2-4 and Table 2-5 for the 43 mil (16 gage) and 68 mil (14 gage) specimens.

Table 2-4: Frame test matrix and expected capacities (I/II)

<table>
<thead>
<tr>
<th>ID</th>
<th>Deck Thickness (ga.)</th>
<th>Deck Grade (ksi)</th>
<th>Fastener (No.)</th>
<th>CFS Joist Thickness (mil)</th>
<th>$P_{nv,n}$ (kips)</th>
<th>$P_{nv,m}$ (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>22</td>
<td>45</td>
<td>12</td>
<td>43</td>
<td>0.77</td>
<td>0.78</td>
</tr>
<tr>
<td>11</td>
<td>22</td>
<td>45</td>
<td>12</td>
<td>43</td>
<td>0.77</td>
<td>0.78</td>
</tr>
<tr>
<td>12</td>
<td>22</td>
<td>45</td>
<td>12</td>
<td>43</td>
<td>0.77</td>
<td>0.78</td>
</tr>
<tr>
<td>13</td>
<td>24</td>
<td>80</td>
<td>12</td>
<td>43</td>
<td>0.84</td>
<td>0.77</td>
</tr>
<tr>
<td>14</td>
<td>24</td>
<td>80</td>
<td>12</td>
<td>43</td>
<td>0.84</td>
<td>0.77</td>
</tr>
<tr>
<td>15</td>
<td>24</td>
<td>80</td>
<td>12</td>
<td>43</td>
<td>0.84</td>
<td>0.77</td>
</tr>
<tr>
<td>16</td>
<td>26</td>
<td>80</td>
<td>12</td>
<td>43</td>
<td>0.63</td>
<td>0.63</td>
</tr>
<tr>
<td>17</td>
<td>26</td>
<td>80</td>
<td>12</td>
<td>43</td>
<td>0.63</td>
<td>0.63</td>
</tr>
<tr>
<td>18</td>
<td>26</td>
<td>80</td>
<td>12</td>
<td>43</td>
<td>0.63</td>
<td>0.63</td>
</tr>
</tbody>
</table>

Table 2-5: Frame test matrix and expected capacities (II/II)

<table>
<thead>
<tr>
<th>ID</th>
<th>Deck Thickness (ga.)</th>
<th>Deck Grade (ksi)</th>
<th>Fastener (No.)</th>
<th>CFS Joist Thickness (mil)</th>
<th>$P_{nv,n}$ (kips)</th>
<th>$P_{nv,m}$ (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>19</td>
<td>22</td>
<td>45</td>
<td>12</td>
<td>68</td>
<td>0.77</td>
<td>0.78</td>
</tr>
<tr>
<td>20</td>
<td>22</td>
<td>45</td>
<td>12</td>
<td>68</td>
<td>0.77</td>
<td>0.78</td>
</tr>
<tr>
<td>21</td>
<td>22</td>
<td>45</td>
<td>12</td>
<td>68</td>
<td>0.77</td>
<td>0.78</td>
</tr>
<tr>
<td>22</td>
<td>24</td>
<td>80</td>
<td>12</td>
<td>68</td>
<td>0.84</td>
<td>0.77</td>
</tr>
<tr>
<td>23</td>
<td>24</td>
<td>80</td>
<td>12</td>
<td>68</td>
<td>0.84</td>
<td>0.77</td>
</tr>
<tr>
<td>24</td>
<td>24</td>
<td>80</td>
<td>12</td>
<td>68</td>
<td>0.84</td>
<td>0.77</td>
</tr>
<tr>
<td>25</td>
<td>26</td>
<td>80</td>
<td>12</td>
<td>68</td>
<td>0.63</td>
<td>0.63</td>
</tr>
<tr>
<td>26</td>
<td>26</td>
<td>80</td>
<td>12</td>
<td>68</td>
<td>0.63</td>
<td>0.63</td>
</tr>
<tr>
<td>27</td>
<td>26</td>
<td>80</td>
<td>12</td>
<td>68</td>
<td>0.63</td>
<td>0.63</td>
</tr>
</tbody>
</table>
Figure 2-10: Typical frame (structural) connection specimen

2.4. Experimental setup

The test rig for the experimental program can be seen in Figure 2-11 below. This testing apparatus was designed and used by Torabian et al (NBM 2017, Torabian and Fratamico 2018, Torabian and Schafer 2021) to investigate the cyclic shear performance of bare-deck structural and sidelap connections and can be used in two configurations (Figure 2-11(a) and 2-11(b)). The specimens are connected to the stationary (fixed) and moving (load) square tubing via bolts through the flutes. The fixed square tubing is connected to the reaction beam and the free square tubing rests on linear guide bearings which slide on the underlying guiderail. Displacement is applied to the load beam
via the actuator. Detailed specifications on the testing apparatus design can be found in Torabian and Schafer (Torabian and Schafer 2021) and 2017 NBM report to AISI, SDI, SJI (NBM 2017).

Figure 2-11: Sidelap testing configuration (b) Sidelap testing configuration (NBM 2017)

2.5. Loading protocol and data acquisition

FEMA 461 (FEMA 2007) Interim Protocol II, quasistatic cyclic displacement-controlled loading protocol was utilized to load the specimen (Figure 2-12). The displacement amplitude was ramped up from initial displacement ($\Delta_0$) to the maximum expected displacement amplitude ($\Delta_m$) of 0.75 inches over 15 steps (30 cycles). Peak displacement in each step ($\Delta_i$) was 40% higher than the peak displacement of the previous step ($\Delta_{i-1}$). Each step comprised of primary and trailing positive and negative cycles (Figure 2-12). Two additional displacement steps were defined to reach 1.47 inches of displacement (34 cycles total) to capture any residual force levels which might exist in the testing post the maximum expected displacement. The maximum expected displacement amplitude ($\Delta_m$) of 0.75 in over 15 steps was also selected to ensure that at the very minimum, 10 complete steps are completed before failure of the specimen.
The displacement was applied to the free beam (HSS section) via an MTS 150-kip hydraulic actuator and FLEXTEST 60 controller at a load rate of 0.005 inches/sec. Applied load and overall displacement of the free beam was measured using the internal load cell and position transducer of the actuator. Four additional position transducers (PT1 – PT4) were located on the free and fixed HSS sections and deck flutes. These sensors were used to verify actuator displacement results (PT1), measure test rig translations (PT1 and PT2), and deck slip (PT1 – PT4). Although deck slip had been minimized by providing significant overstrength in the fixed and free tubing bolted connections, deck slip was measured to validate this assumption. These sensors can be seen attached to the test rig and specimen in Figure 2-13 below. Displacement and force data from the position transducers and the actuator were recorded using a custom LabVIEW virtual instrument (VI) at an acquisition rate of 10 Hz.
2.6. Experimental test program results

The following sections and subsections present and discuss the results of the sidelap connections and structural connections experimental test program. Individual test actuator force-displacement, test rig translations, and deck slip plots along with detailed test observation notes can be found in Appendix A and Appendix B respectively. Hysteresis and backbone curves from the 27 connection tests have been presented (Figure 2-15 through 2-17) along with a Table (Table 2-6) summarizing maximum ($P_{\text{max}}$) and minimum ($P_{\text{min}}$) observed peak loads and strain energy under the backbone curves. Influence of Ply 1 and Ply 2 thickness on the peak observed capacities ($P_{\text{max}}$ & $P_{\text{min}}$) and predictive performance of design equations has been investigated and presented. Descriptions of the progression of failure in the connection tests and estimations of strain energy under the backbone curves have also been presented to characterize the hysteretic behavior of connections.
2.6.1. Determining hysteresis curves, backbone curves, $P_{\text{max}}$, $P_{\text{min}}$, and strain energy under the backbone curves

Hysteresis curves of the screw fastened sidelap and structural (frame) connections were determined from the actuator force and displacement results. The hysteresis curves were then utilized to determine multi-point backbone curves for each of these repetitions. While determining backbone curves, points were selected to reflect the overall response of the connection. Prior to peak, the multi-point backbone curve points correspond to points with maximum load in that displacement step. Post first peak and onset of degradation, points were selected to correspond with maximum displacement in the displacement step. Degradation was defined to initiate when the primary cycle in a displacement step resulted in a lower peak force than the primary cycle in the previous step. The first peak in positive and negative loading were recorded as the $P_{\text{max}}$ and $P_{\text{min}}$ respectively. Once degradation had initiated, further peaks observed in latter parts of the loading protocol were intentionally neglected. This was done to avoid capturing secondary peaks that were observed at very large displacements due to excessive tilting leading to bearing of the fastener heads into the plies or piling of steel around fastener holes due to excessive bearing. A sample backbone curve for the 22 gage sidelap tests (22-22-10-R1), repetition one can be seen in Figure 2-14.a.

Figure 2-14: Backbone curves(a) and strain energy calculation (b & c)
Strain energy under the multi point backbone curves is an important input when fitting four-point backbone curves to the data for numerical modelling of connections and hence was calculated and reported to allow future expansions to this work. The backbone curves were numerically integrated using the trapezoidal method. The trapezoidal method is well suited for accurately integrating the generated backbone curves as points along the backbone curve are connected with linear line segments, essentially diving the entire response into smaller trapezoids and triangles. The resulting discretization of the area under the backbone curve can be seen in Figure 2-14.b and 2-14.c.

2.6.2. Summary results

Individual hysteresis and backbone curves from the 27 connection tests have been grouped by test type and presented in Figures 2-15 through 2-17 below. The sidelay connection testing summary can be seen in Figure 2-15. Figures 2-16 and 2-17 summarize the hysteresis and backbone curves for the 18 gage and 14 gage support tests respectively. Figure 2-18 depicts the superimposed backbone curves for comparing specimens as well as the strain energy. Table (Table 2-6) presents maximum ($P_{\text{max}}$) and minimum ($P_{\text{min}}$) observed peak loads and strain energy under the backbone curves.
Figure 2-15: Hysteresis and backbone curves for sidelap testing
Figure 2-16: Hysteresis and backbone curves for 18 gage support testing
Figure 2-17: Hysteresis and backbone curves for 14 gage support testing
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Figure 2-18: Superimposed backbone curves and strain energy per quadrant
2.6.3. Sidelap specimens

Typical failure mode observed during the testing of the 22 gage sidelaps (22-22-10-R1, R2, and R3) was tilting of the screw and bearing in plies. Cyclic loading would cause the screw to no longer be flush (Figure 2-16(a)) in the 0.071-inch cycle and start tilting (Figure 2-16(b)) in the 0.100-inch cycle. This eventually led to thread by thread pull through of the fastener which resulted in hole ovalisation of Ply 1 (Figure 2-16(e)) and bearing in Ply 2 (Figure 2-16(f)). The mean observed capacity, $P_{\text{max}}$ and $P_{\text{min}}$, for the 22 gage sidelaps with #10 fasteners (#10 – 14 – 3/4" inch) was found to be 0.61 kips and -0.64 kips respectively.

![Image](image_url)

**Figure 2-19: Progression of failure observed in 22-gage sidelap testing**

Similar to the 22 gage sidelap specimen, typical failure mode observed during the testing of the 24 gage sidelaps (24-24-10-R1, R2, and R3) was also tilting of the screws and bearing in plies. Cyclic loading would cause the screw to no longer be flush (Figure 2-17(a)) in the 0.051-inch cycle, start tilting (Figure 2-17(b)) in the 0.071-inch cycle, and eventually lead to thread by thread pull through of the fastener. This resulted in hole ovalization and bearing of Ply 1 (Figure 2-17(d)) and bearing.
in Ply 2 (Figure 2-17(f)). The mean observed capacity, $P_{max}$ and $P_{min}$, for the 24 gage sidelaps with #10 fasteners (#10 – 14 – 3/4" inch) was found to be 0.51 kips and -0.46 kips respectively.

Typical failure mode observed during the testing of the 26 gage sidelaps (26-26-10-R1, R2, and R3) was tilting of the screw and bearing in plies. Additionally, tearing of the fastener hole, ply separation, and warping near the fasteners was also observed. Cyclic loading would cause the screw to no longer be flush (Figure 2-18(a)) in the 0.051-inch cycle and start tilting (Figure 2-18(b)) in the 0.100-inch cycle. Thread pull-through of the fasteners was initiated in the 0.195-inch cycle which resulted in tearing of the fastener hole and hole ovalization in Ply 1 (Figure 2-18(c)). The plies began to separate in the 0.38-inch cycle. Bearing was also observed in Ply 2 (Figure 2-18(f)). The mean observed capacity, $P_{max}$ and $P_{min}$, for the 26 gage sidelaps with #10 fasteners (#10 – 14 – 3/4" inch) was found to be 0.33 kips and -0.27 kips respectively.
The common failure mode in sidelap tests was tilting of fasteners leading to threads pulling through which eventually resulted in complete loss of the fastener. Hole ovalization in Ply 1 and bearing of fastener hole in Ply 2 was also observed. As mentioned in DDM04 (Luttrell 2015), failure mechanism followed pullout of the screw after sufficient tilting had occurred.

2.6.4. Structural specimens

Typical failure mode observed during the testing of the 22-gage structural connection with 18 gage supports (22-18-10-R1, R2, and R3) was tilting of the screw and bearing in plies. Cyclic loading resulted in the screws to no longer be flush (Figure 2-19(a)) with the deck in the 0.051-inch cycle. The threads would start to pull through and eventually result in tilting of the fastener (Figure 2-19(b)) in the 0.100-inch cycle. Ply separation along with hole elongation and bearing in Ply 1 was observed in the 0.195-inch cycle (Figure 2-19(c)). Bearing was also observed in Ply 2 (Figure 2-19(e)). The mean observed capacity, $P_{\text{max}}$ and $P_{\text{min}}$, for the 22-gage structural connections with #12 fasteners (#12 – 14 – 3/4" inch) was found to be 0.55 kips and -0.51 kips respectively.
Figure 2-22: Progression of failure observed in 22-gage deck with 18 gage support testing

Typical failure mode observed during the testing of the 24-gage structural connection with 18 gage supports (24-18-10-R1, R2, and R3) was tilting of the screw and bearing in plies. Additionally, ply separation, warping near the fasteners, and fastener pull-through was also observed in these tests. As the connections were loaded, initiation of warping was observed near the fasteners in the direction of loading. Continued cyclic loading resulted in the screws to no longer be flush and tilt in the 0.03-inch cycle (Figure 2-20(a)). Fastener pull through and ply separation ensued (Figure 2-20(b)). Eventually bearing and tilting of the fastener caused Ply 1 to pull over the fastener (Figure 2-20(c) through Figure 2-20(e)). Bearing was also observed in Ply 2 (Figure 2-20(f)). The mean observed capacity, $P_{max}$ and $P_{min}$, for the 24-gage structural connections with #12 fasteners (#12 – 14 – 3/4" inch) was found to be 0.51 kips and -0.41 kips respectively.
Figure 2-23: Progression of failure observed in 24-gage deck with 18 gage support testing

Typical failure mode observed during the testing of the 26-gage structural connection with 18 gage supports (26-18-10-R1, R2, and R3) was tilting of the screw and bearing in plies. Warping around the fasteners, ply separation, and fastener head bearing into the steel plies after sufficient tilting was also observed in these tests. Ply separation and warping around the fasteners in the direction of loading was observed to initiate in the 0.019-inch cycle (Figure 2-21(b)). Continued cyclic loading resulted in the screws to no longer be flush (Figure 2-21(a)) in the 0.036-inch cycle. The threads would start to pull through and eventually result in tilting of the fastener (Figure 2-21(c)) in the 0.100-inch cycle. As the applied displacement magnitude increased, the fastener continued to tilt more and eventually the fastener head made contact with the steel plies (Figure 2-21(d)). This resulted in load increases in the force-displacement response after degradation due to bearing and tilting had already initiated. Hole elongation and bearing were observed in both plies (Figure 2-21(e) and Figure 2-21(f)). The mean observed capacity, $P_{max}$ and $P_{min}$, for the 26-gage structural
connections with #12 fasteners (#12 – 14 – 3/4" inch) was found to be 0.48 kips and -0.38 kips respectively.

Figure 2-24: Progression of failure observed in 26-gage deck with 18 gage support testing

Typical failure mode observed during the testing of the 22-gage structural connection with 14 gage supports (22-14-10-R1, R2, and R3) was tilting of the screw and bearing in plies. Initiation of warping was observed in the 0.019-inch cycle (Figure 2-22(a)). Fastener hole bearing was observed in 0.139-inch cycle (Figure 2-22(b)) and was followed by fastener tilting (Figure 2-22(c)). After the fastener threads had pulled though and displacement amplitude was increased, the fastener head tilted and made contact with the ply in the 0.273-inch cycle (Figure 2-22(d)). Fastener pull-through continued until the fastener had completely pulled out. Bearing was observed in Ply 1 [Figure 2-22(d)] and fastener hole ovalization was observed in Ply 2 (Figure 2-22(d)). The mean observed capacity, $P_{max}$ and $P_{min}$, for the 22-gage structural connections with #12 fasteners (#12 – 14 – 3/4" inch) was found to be 0.96 kips and -0.86 kips respectively.
Figure 2-25: Progression of failure observed in 22-gage deck with 14 gage support testing

Typical failure mode observed during the testing of the 24-gage structural connection with 14 gage supports (24-14-10-R1, R2, and R3) was bearing in plies leading to eventual pull over. The warping of deck was observed in the 0.0139-inch cycle (Figure 2-23(a)) and progressively increased with increasing displacement amplitude. Bearing of Ply 1 was observed in 0.100-inch cycle (Figure 2-23(b)), and after significant bearing, fastener started to tilt (Figure 2-23(c)). The fastener was ultimately pulled over by Ply 1 (Figure 2-23(d) and Figure 2-23(e)). Although tilting was observed later on in the loading protocol, the primary failure mode was bearing, and the fastener remained in position in Ply 2 (Figure 2-23(f)). The mean observed capacity, $P_{\text{max}}$ and $P_{\text{min}}$, for the 24-gage structural connections with #12 fasteners (#12 – 14 – 3/4" inch) and 14 gage supports was found to be 0.90 kips and -0.86 kips respectively.
Figure 2-26: Progression of failure observed in 24-gage deck with 14 gage support testing

Typical failure mode observed during the testing of the 26-gage structural connection with 14 gage supports (26-14-10-R1, R2, and R3) was bearing in plies leading to eventual pull over. Deck warping and ply separation was observed in the 0.026-inch cycle (Figure 2-24(b)) and progressively increased with increasing displacement amplitude. Bearing of Ply 1 was observed in 0.100-inch cycle (Figure 2-24(c)) and progressively increased as applied displacement amplitude increased (Figure 2-24(d)). The fastener was ultimately pulled over by Ply 1 (Figure 2-24(e)). The mean observed capacity, $P_{\text{max}}$ and $P_{\text{min}}$, for the 26-gage structural connections with #12 fasteners (#12 – 14 – 3/4" inch) and 14 gage supports was found to be 0.71 kips and -0.65 kips respectively.
The common failure modes observed during structural connection testing were tilting of fasteners, bearing of plies, deck warping near fasteners in direction of loading. When tilting is dominating bearing as was observed in the 18 gage tests, failure mode leans towards fastener pull out as the screws are individually pulled through thread by thread. This also shows up as spikes in the force-displacement response of the connection. As observed with the sidelap specimen, excessive tilting can also cause the fastener head to tilt and contact the plies causing a load increase even after load and stiffness degradation have started which can lead to large increases in capacity at large displacements. When bearing is dominating tilting, as was observed in the 14 gage support tests with 24 gage and 26 gage deck, failure mode leans towards fastener pull over. The fastener shank continues to bear and tear the ply until the ply pulls over the fastener. This is preceded by large deformations in Ply 1 near the connection.
2.6.5. Influence of deck and support thickness on peak connection capacity and performance of predictive design equations

In the sidelap tests, as expected and predicted by the design equations, reducing deck thickness caused a drastic reduction in capacity. This reduction was on average 24% and 60% when comparing the 22 gage sidelaps with the 24 and 26 gage sidelap tests (a 17% and 34% reduction in thickness). Similarly, a predictable reduction in capacity was also observed for the 18 gage supports when deck thickness was reduced from 22 gage to 24 and 26 gage (20 and 26% respectively). For the 14 gage supports, negligible capacity reduction was observed between the 22 gage and 24 gage tests when a slight increase was predicted. When Ply 1 thickness was held constant and ply 2 thickness was reduced from 14 gage to 18 gage (28.3% reduction in thickness), a reduction in capacity was observed for the connections. This reduction was on average 40.5%, 52.7%, and 46% for the 22 gages, 24 gage, and 26 gage deck respectively. This was directly in contrast with code predictions as these connections were expected to have similar capacities.

Tables 2-8 through 2-10 and Figure 2-28 show the mean test to predicted ratios for the sidelap, 18 gage support, and 14 gage support tests respectively. Table 2-7 below summarizes the mean test to predicted ratios by testing series (i.e., sidelap vs 18 gage supports vs 14 gage supports). The peak tested strength, $P_{exp}$, is taken as the minimum of $P_{max}$ and $P_{min}$ for each repetition and compared with AISI S100 design equations with nominal ($P_{nv,n}$) and measured ($P_{nv,m}$) material properties. While the sidelap tests and 14 gage support tests were well predicted by the design equations, significant overpredictions were observed for the 18 gage support tests. This can be observed in Table 2-8 and 2-10. The mean test to predicted ratios for 18 gage support tests based on nominal and measured properties was found to be 0.58 and 0.59 respectively.
Table 2-7: Mean and standard deviation of predictability of AISI S100 (AISI 2016b) equations

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Figure 2-28: Mean test to predicted ratios by ratio of ply thickness

The connection capacity for the 18 gage support tests is interpolated between the tilting and bearing capacity of the support plies based on the relative ply thickness. However, both the tilting and bearing capacities lie above the experimentally observed capacities indicating that not only are the interpolated capacities higher but so are the calculated bearing and tilting capacities. This indicates that either the connection failed to develop its design strength or that predictive equation did not perform well in the relative ply ratio range of 1.00 and 2.50. The 18 gage support members had a high ultimate to yield strength ratio (2.16 in comparison 1.51 of the 14 gage) as well as an elongation of greater than 30%. It is possible that the full ultimate strength was not developed which caused the reductions in capacity. Also, the observed failure mode in the 18 gage tests was
tilting and bearing in Ply 1 or 2 which matched the code predictions. While the predictive equation did not perform well in predicting capacity for the 18 gage tests, it was able to predict failure mode.

Table 2-8: Mean test to predicted ratios for sidelap tests

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<td><strong>Mean</strong></td>
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<td>0.26</td>
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Table 2-9: Mean test to predicted ratios for 18 gage structural tests

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<th>No</th>
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<th>Support Gage (Ply 2)</th>
<th>$P_{\text{max}}$ (kips)</th>
<th>$P_{\text{min}}$ (kips)</th>
<th>$P_{\text{nv,n}}$ (kips)</th>
<th>$P_{\text{nv,m}}$ (kips)</th>
<th>$P_{\text{exp}}/P_{\text{nv,n}}$ (kips)</th>
<th>$P_{\text{exp}}/P_{\text{nv,m}}$ (kips)</th>
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<td>0.78</td>
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<td>0.78</td>
<td>0.66</td>
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<tr>
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<td><strong>Mean</strong></td>
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<td>$P_{min}$ (kips)</td>
<td>$P_{nv,n}$ (kips)</td>
<td>$P_{nv,m}$ (kips)</td>
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<td>$P_{exp}/P_{nv,m}$</td>
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</tr>
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<td>0.84</td>
<td>1.03</td>
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<td>0.63</td>
<td>1.13</td>
<td>1.12</td>
</tr>
</tbody>
</table>

2.6.6. Comparison of strain energy

To compare strain energy values, the averages were computed for all repetitions of a configuration and have been reported in Table 2-11. In the sidelap connection tests, as expected, strain energy was observed to increase as ply thickness was increased. Along with reductions in capacity (2-18), the lighter gage steel connections also had reduced energy absorption capability. When comparing the 22-gage tests with 24 gage and 26 gage tests, the strain energy was on average 80% and 52% of the energy of the 22 gage specimens respectively. In contrast, structural framing connections comprising of either the 18 gage or 14 gage supports had similar energy absorption capabilities across the steel thicknesses. In the 18 gage tests, the strain energy of the 24 gage and 26 gage specimens was 87% and 117% of the strain energy of the 22-gage specimen. In the 14 gage tests, the strain energy absorbed by the 24 gage and 26 gage specimens was 89% of the energy absorbed by the 22-gage specimen.
<table>
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<th>Negative ft-lb</th>
<th>Total ft-lb</th>
<th>$P_{\text{exp}}$ (average) (kips)</th>
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<td>32.58</td>
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<td>9</td>
<td>26</td>
<td>14</td>
<td>30.75</td>
<td>33.43</td>
<td>64.18</td>
<td>0.70</td>
</tr>
</tbody>
</table>

2.7. Experimental test program conclusions

To investigate the influence of ply thickness on the hysteretic behavior of light gage steel sidelap and structural connections, 27 cyclic tests were performed using the FEMA 461 quasi-static loading protocol. Applied displacement was ramped over 34 cycles to 1.5 inches and 3 unique sidelap and 6 unique structural configurations were tested. Three repetitions were performed for each configuration and based on the observations from these tests, the following key conclusions were drawn:

- Sidelap connections failed due to a combination of bearing and tilting. Fastener threads were pulled through the plies, thread by thread, due to excessive tilting until the fastener was ejected from the connection. This behavior was observed independent of deck thickness and was accompanied by a reduction in capacity and strain energy.

- Structural connections failed either in bearing or a combination of bearing and tilting based on the relative thickness of the plies and the support thickness. When deck thickness was 24 gage or 26 gage, bearing dominated and the connection failed due to pull over of the
fastener for the 14 gage tests. When deck thickness was 22 gage, tilting was observed for both 14 and 18 gage tests. The structural connections had comparable strain energy capacity for both the 14 gage and 18 gage tests.

- During testing, both excessive bearing and tilting of the fastener heads were observed to cause secondary peaks at large displacements which were excluded from the backbone curves. The increase in capacity was due to bearing of the fastener head against the plies after sufficient tilting had occurred or pilling of steel around the fastener hole due to bearing.

- The design equations accurately predicted failure modes across the test programs but there were variabilities in capacity predictions. While the 14 gage support tests and sidelap tests were well predicted (mean test to predicted ratios = 1.08 and 1.14 respectively), poor predictions were observed for the 18-gage specimen (mean test to predicted ratio = 0.58).

- The poorly predicted 18 gage support tests fall in the transition range of relative ply thicknesses (1.00 < \( t_2/t_1 < 2.50 \)), and the 18 gage support members had a significantly higher ultimate to yield strength ratio when compared with the 14 gage support members (2.16 in comparison with 1.51). It is hypothesized that either the equation had poor performance in the transition region, or the 18 gage support member could not develop its coupon tested ultimate strength, leading to poor predictions.

- Low elongation steel reductions as stipulated by AISI S100-16 needed to be accounted for the 24 gage and 26 gage nominal and measured capacity calculations due to the low elongation observed during coupon testing for accurate predictions.
3. CHAPTER 3:

DETERMINING THE INFLUENCE OF SUPPORT ATTACHMENT PATTERN OF THE OUT-OF-PLANE BUCKLING CAPACITY OF LIGHT-GAGE STEEL DECK DIAPHRAGMS

3.1. Introduction

While adequate design guidance is provided in design codes and manuals such as AISI S310 – 20 (AISI 2020) and SDI DDM04 (Luttrell 2015) to account for the impact of support attachment patterns (Figure 3-1) on the connection limit states, the out of plane panel buckling limit state does not consider this impact. Existing panel buckling research is also limited to the fully attached (36/7) attachment pattern (i.e., deck is attached through every bottom flute) and no comprehensive dataset exists where tests were specifically designed and performed to study the impact reducing support fasteners (36/5 and 36/4) can have on buckling capacity and behavior. To further understand the relationship between support attachment pattern and the out-of-plane buckling limit state, nine monotonic tests were conducted at the University of Massachusetts Amherst Robert B. Brack Structural testing facility. These tests were performed on the cantilever test frame and comprised of three unique configurations with three repetitions each. The specimens were all constructed with 22 gage (0.76 mm) Type B deck (Figure 3-2) and had identical span lengths ($L_o$), thickness ($t$), sidelap, and edge connections and only differed in the number of fasteners at the supports to simulate industry standard attachment patterns (Figure 3-1). This Chapter summarizes the relevant background literature, utilized testing procedure, key results, and conclusions of the experimental study and finite element analysis (FEA) investigation.
Figure 3-1: Diaphragm components and typical attachment patterns

Figure 3-2: Typical corrugation dimensions for Type B Deck (Luttrell 2015)

3.2. Relevant works

Although there exists a wealth of literature which focuses on the shear strength and stiffness of diaphragm systems and applicable limit states (O’brien et al. 2017, Moen et al 2016, Ammar and Nilson 1971, Wright and Hossain 1997, Luttrell 1973, Duerr and Saal 2006), the works of Easley
(Easley 1975), Wright and Hossain (Wright and Hossain 1997), and Nunna (Nunna 2011) are discussed below due to the unique insights they offer on the panel buckling limit state. Through the works of Easley (Easley 1975) and Nunna (Nunna 2011), the origin of the panel buckling equation is traced to its current calibrated and modified form. Wright and Hossain’s (Wright and Hossain 1997) investigation on the impact of attachment method on end restraint coefficient $\beta$, has also been included to examine the differences end attachment methodology can have on strength.

3.2.1. Easley (1975)

The existing shear buckling limit state equation (Equation 1-12) is a modified and calibrated form of the elastic buckling equation (Equation 3-1) developed by Easley and McFarland (Easley 1975). The calibration was done by Nunna (Nunna 2011). Easley and McFarland developed elastic buckling equations (Equation 3-1) to predict critical shear load per unit length of a corrugated metal panel by utilizing the Ritz energy method. Panels were treated as plates with different flexural rigidities in the two perpendicular directions and the ends were assumed to be simply supported through the mid-plane of every flute. These equations were validated via a suite of eight experimental tests which varied in aspect ratio (Table 3-1 and Figure 3-3), corrugation pitch, and stiffness in the orthogonal directions. Tabs attached to the mid-plane of corrugations were utilized to apply load through the neutral axis of the panel. These tabs were clamped into the test frame in an attempt to create a simply supported condition.

$$S_{no} = 36\beta \frac{D_x^{1/4}D_y^{3/4}}{b^2}$$

(3-1)

Equation 1 predicted buckling capacities within 1.06 to 1.25 times the experimentally measured capacities. Easley discovered that tabs had an end restraining effect which caused deviation from
the equation as the supports no longer behaved as purely simply supported connections. The restraining effect of the tabs were accounted for by the end restraint coefficient \( \beta \) which theoretically varied between 1.0 (simply supported) and 1.9 (fully fixed) (Easley 1975, Hlavacek 1968). They concluded that the elastic buckling equation was accurate for simply supported panels \( (\beta = 1.0) \), but true variation of \( \beta \) with end restraint is unknown and depended upon the attachment conditions.

Table 3-1: Experimental configurations (Adapted from Easley 1975)

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Dimensions (in)</th>
<th>( N_{cr} ) (lb/in)</th>
<th>( \beta )</th>
<th>( \beta N_{cr} ) (lb/in)</th>
</tr>
</thead>
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<td></td>
<td>Length Width q measured tested</td>
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<td></td>
<td></td>
</tr>
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</tbody>
</table>

Figure 3-3: Summary of results of Easley test program (Adapted from Easley 1975)
3.2.2. Wright and Hossain (1997)

While developing analytical models to predict strength and stiffness of profiled sheets, Wright and Hossain (Wright and Hossain 1975) looked into the impact boundary attachment has on the buckling capacity of these sheets. Three distinct boundary conditions (Figure 3-4) were analyzed using finite element analysis (FEA) and compared to small scale tests: welded through both top flat and bottom flute (Type 1), welded through bottom flute (Type 2), discretely welded with spot welds in bottom flute (Type 3). They found that Easley’s buckling equations can accurately predict the shear buckling capacity but needed specific values of $\beta$ to account for the effect of different boundary conditions i.e., end restraints. The reported $\beta$ values varied from 1.72, 1.42, and 1.00 for Type 1, Type 2, and Type 3 boundary conditions respectively. These $\beta$ values were back calculated from the FEA and experimental results. Further, Wright and Hossain also recommended a 50% reduction in buckling capacity if the sheets were only attached in alternate flutes. This 50% reduction also agrees with industry practice and is a significant deviation from what is recommended in the code as the code.

![Figure 3-4: Evaluated boundary conditions (adapted from Wright and Hossain 1997)](image)

3.2.3. Nunna (2011)

Nunna (Nunna 2011) evaluated the performance of panel buckling equations from TSM (Army, Navy, and Air Force 1982), SDI DDM03 (Luttrell 2004), Easley and McFarland (Easley 1975) and the proposed AISI S310 – 16 (AISI 2016a) equation (Equation 1-12). The equations were used
to predict the buckling capacities for a historical dataset comprising of twenty-eight full-scale experiments where the failure mode was deck out-of-plane buckling without localized failure of fasteners. The objective of this work was to evaluate the validity of commonly available panel out of plane buckling equations and provide recommendations for resistance factors (LRFD and LSD) and safety factors (ASD). The specimens varied in corrugation depth [1.5 in specimen (26 nos.), 2 in specimen (1 no.), and 1- 5/8 in specimen (1 no.)], corrugation pitch – “d” [6 in (152.4 mm), 9 in (228.6 mm), and 12 in (304.8 mm)], gage – “t” (29, 22, 20, 18 and 16 gage), number of spans, and span length – “L”. Twenty-seven of the twenty-eight specimens were fully attached to the test frame with connections through each flute. The strength to predicted ratios for TSM, Easley and McFarland, and proposed AISI S310 equations can be seen in Figure 3-5 below.

![Figure 3-5: Comparison of existing equations](image)

The comparison of the AISI S310 panel buckling equation with historical data can be seen in Figure 3-6. In addition to the calibration tests used by Nunna (Nunna 2011), other panel buckling tests from literature (Pinkham 1999) have also been added to this plot to show performance of the equation. The equation had an average strength-to-predicted ratio and correlation coefficient of
1.002 and 0.910 respectively and could be utilized for single and multi-span applications (Nunna 2011). The standard deviation for test to predicted ratios was 0.213. The calculated LRFD, LSD, and ASD resistance and safety factors were 0.70, 0.55, and 2.27 respectively. Nunna recommended that either the TSM, modified Easley, or proposed AISI S310 equation be used for estimating the out-of-plane buckling capacity of the deck. However, there was high variability in strength to predicted ratios (0.61 – 1.44) for the S310 out-of-plane buckling equation and all specimens had the same fully attached 36/7 pattern.

![Comparison of current equation with historical test data](image)

**Figure 3-6: Comparison of current equation with historical test data**
3.3. Experimental test matrix and measured properties

3.3.1. Test matrix

The experimental test matrix (Table 3-2) was designed to ensure that panel out-of-plane buckling [AISI S310 Chapter D2-1 (AISI 2020)] is the governing limit state across all the attachment patterns. In this table, $S_{nf}$ is the minimum of $S_{ni}$, $S_{nc}$, and $S_{ne}$. $S_{nf}/S_{nb}$ is the ratio defining overstrength in connections when compared to buckling. Panel buckling was ensured by providing adequate overstrength in the connection limit states [min $(S_{nf}/S_{nb}) = 1.78$]. Support and edge fasteners connections were made with #14 (6 mm) screws which had a connection strength ($P_{nf}$) of 1.24 kips (5.52 kN) each when used with the 22-gage deck (0.76 mm) and 54 mil (1.37 mm) CFS support angles. Sidelap connections were made with the proprietary Punchlok – II tool which provided a $P_{nf}$ of 2.10 kips (9.34 kN) for the 22-gage (0.76 mm) deck. However, the sidelap capacity was conservatively restricted to support connection capacity (1.24 kips (5.52 kN)) for calculations to as it was assumed that failure of the support connections would occur first along the sidelaps initiating failure in the test as the sidelaps had higher load carrying capacity than the support connections. Sidelaps and edge fasteners were installed 6 inches on center (152.4 mm). Nominal geometric dimensions and material properties ($E = 29,500$ ksi (203,400 MPa), $F_y = 50$ ksi (345 MPa), $F_u = 65$ ksi (448 MPa)) were used for all calculations.
<table>
<thead>
<tr>
<th>Specimen</th>
<th>Attachment Pattern</th>
<th>$L_v$ (ft)</th>
<th>$S_{ni}$ (klf)</th>
<th>$S_{nc}$ (klf)</th>
<th>$S_{ne}$ (klf)</th>
<th>$S_{nf}$ (klf)</th>
<th>$S_{nb}$ (klf)</th>
<th>$S_{nf}/S_{nb}$ (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>36/7 - R1</td>
<td>36/7</td>
<td>15.00</td>
<td>2.61</td>
<td>1.82</td>
<td>2.73</td>
<td>1.82</td>
<td>0.63</td>
<td>2.89</td>
</tr>
<tr>
<td>36/7 - R2</td>
<td>36/5</td>
<td>15.00</td>
<td>2.59</td>
<td>1.40</td>
<td>2.68</td>
<td>1.40</td>
<td>0.63</td>
<td>2.22</td>
</tr>
<tr>
<td>36/7 - R3</td>
<td>36/4</td>
<td>15.00</td>
<td>2.54</td>
<td>1.12</td>
<td>2.62</td>
<td>1.12</td>
<td>0.63</td>
<td>1.78</td>
</tr>
<tr>
<td>36/5 - R1</td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>36/5 - R2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>36/5 - R3</td>
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<td></td>
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<tr>
<td>36/4 - R1</td>
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<tr>
<td>36/4 - R2</td>
<td></td>
<td></td>
<td></td>
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<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>36/4 - R3</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
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<td></td>
</tr>
</tbody>
</table>

**Notes** -
1. Edge and sidelap fastener spacing - 6 in (152.4 mm) on center
2. Exterior edge fastener type - #14 Hex head ($P_{nf} = 1.24$ kips)
3. Sidelap connections - VSC - II ($P_{nr} = 2.10$ kips, restricted to 1.24 kips)

### 3.3.2. Measured properties and dimensions

Thickness and bearing length ($N_{ext}$) measurements were taken from each of the specimen to compare with the nominal properties used in the test matrix design calculations. Thickness ($t$) of the panel has an impact on the nominal shear strength ($P_{nv}$) of the connection (AISI 2016b) and thereby impacts the connection limit states. Thirty-six thickness measurements were taken from various locations on the panel prior to construction and testing with a pair of digital Vernier Calipers. Figure 3-7(a) below shows the variation in $t$ and the empirical cumulative distribution function for these measurements. The mean thickness including galvanization was 0.0305 in (77 mm) vs 0.0295 in (76 mm) as published in the deck manufacturer’s specification and evaluation report (IAPMO ER-0652 2018). This was 2% higher than the nominal values used in the calculations and 97% of the measurements were above the nominal thickness value.
Ten measurements of bearing length of deck on both the East and West supports were also recorded for each of the nine repetitions to compare against the minimum required value in AISI S310 of 0.75 in (19.0 mm). The recorded bearing lengths can be seen in Figure 3-7(b). The mean supported length was 0.83 in (21.0 mm). Although the measurements varied from 0.46 in (11.7 mm) to 1.18 in (29.9 mm), at least 70% of the measurements were above the minimum required value.

3.4. Experimental setup

The following subsections discuss the experimental test frame setup, sensor scheme, fabrication methodology, loading protocol, and data acquisition system utilized in the tests.

3.4.1. Test frame

The cantilever test frame (Figure 3-8) was made up of hot-rolled steel plate girders and can be used to test specimens in 10 ft X 15 ft (3048 mm X 4572 mm) and 15 ft X 15 ft (4572 mm X 4572 mm) configurations. The south side free beam is connected to the actuator and the north beam is fixed to the strong floor through the fixed supports shown in Figure 3-8. Additionally, the free
beam has a roller support near either end to prevent out-of-plane motion of the rig. These supports are connected to the free beam using high-capacity rollers [25 kips (111 kN)]. The bottom of the free beam also rests on rollers to allow translation of the beam due to action of the actuator. The free and fixed beams are connected by transverse beams on the east and west side with pins in all four corners to allow the test frame to pivot about the points of fixity. Detailed design, fabrication, and construction specifications of the rig have been discussed by Castaneda (Castaneda 2022).

![Diagram of Cantilever Test Rig]

**Figure 3-8: Cantilever test rig**

3.4.2. Sensor locations

Linear variable differential transducers (LVDTs) were used to measure translations of the test rig and out of plane motion of the specimen. Three LVDTs each (X1, Y1, Z1, X2, Y2, and Z2) were located in the north-east and south-west corners of the test frame which measured the X, Y, and Z motion of the rig. Nine LVDTs (A1 through C3) in three sensor frames comprising of three LVDTs each were placed along the width of the specimen to measure out-of-plane displacements at various
locations during testing. These sensors were stationary relative to the panel and the specimen moved under them during testing. Hence to avoid loss of contact, the sensors were placed in the middle of top or bottom of flute flats. Complete sensor layout and sensor coordinate system can be seen in Figure 3-9.

3.4.3. Specimen fabrication

The 22 gage (0.76 mm) steel deck was connected to the test rig using a supporting frame made from 600S200 – 54 cold-formed steel angles (Figure 3-10) which was created by splitting 1200S200 – 54 studs in half through the web. The frame members had holes drilled into them and
were connected to the test rig with 0.75 in (19 mm) bolts placed approximately 2 ft (610 mm) on center along the fixed, free, and transverse beams. Hilti #14 (6 mm) self-drilling, self-tapping, hex head screws were used to make the support and edge connections (Figure 3-11). The individual pieces of deck were then connected at sidelaps using the proprietary Punchlok II tool (Figure 3-11).

Figure 3-10: Perimeter framing members

Figure 3-11: Typical sidelap (VSC-II), support (Hex-14), and edge connections (Hex-14)
To prevent any accidental damage to the specimen or unintentional loading during the sidelap construction, a 20 ft (6096 mm) long construction platform (Figure 3-12) was constructed using a ladder and OSB sheathing. This platform was used as the working platform during crimping of the seams and supported all construction loads.

![Construction platform for sidelap connections. Punchlok – II tool was supported by the crane during specimen fabrication](image)

**Figure 3-12:** Construction platform for sidelap connections. Punchlok – II tool was supported by the crane during specimen fabrication

### 3.4.4. Loading protocol and data acquisition

A monotonic load protocol (Figure 3-13) was utilized to load the specimen using the MTS 244.41 hydraulic actuator and FLEXTEST 60 controller. Peak displacement was set to 3 in (76.2 mm) and the specimen was loaded at a rate of 0.0033 in/sec (0.084 mm/sec) to achieve approximately 1 in (25.4 mm) of displacement every 300 seconds of testing. Data acquisition was done with National Instruments data acquisition system and LabView program at an acquisition rate of 10 Hz.
Figure 3-13: Typical monotonic loading protocol (A_{max} = 3\text{in})

3.5. Experimental test program results

The following sections and subsections present and discuss the results and post processing methods utilized in the experimental testing and interpretation of data. Detailed per test results have been provided in Appendix F.

3.5.1. Summary of experimental results

The data acquisition system provided unfiltered sensor data from the actuator and out-of-plane displacement sensors. Unprocessed actuator force-displacement results for the specimen have been depicted in Figure 3-14 below. Table 3-3 summarizes the ultimate capacity, displacement, and stiffness results for the tests. Conversion of unprocessed actuator force-displacement to shear force-displacement data was required to estimate the ultimate capacity (P_{max}), displacement at maximum load (\Delta P_{max}), and stiffness (G') of the specimen. The processing methods for the same are discussed in section 3.5.1.1 and 3.5.1.3. Table 3-4 summarizes the load and displacement levels at the initiation of out-of-plane buckling (P_{nb}). Out-of-plane sensor data was utilized to establish load level for initiation of buckling. The methodology for the same has been discussed in Section 3.5.1.2. Figure 3-15 shows the corrected force-shear displacement results, P_{max}, P_{nb}, and P_{40} for all
the specimens. Individual sensor force-displacement results for all the repetitions have been summarized in Figure 3-16.

![Graph showing unprocessed actuator force displacement results summary](image)

Figure 3-14: Unprocessed actuator force displacement results summary
**Figure 3-15:** Processed force-shear displacement results summary

**Table 3-3:** Summary of Results (Ultimate load and stiffness)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$P_{\text{max}}$ (kips)</th>
<th>$P_{\text{max,ave}}$ (kips)</th>
<th>$\Delta P_{\text{max}}$ (in)</th>
<th>$\Delta P_{\text{max,ave}}$ (in)</th>
<th>$G'$ (kips/in)</th>
<th>$G'_{\text{ave}}$ (kips/in)</th>
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</thead>
<tbody>
<tr>
<td>36_7_R1</td>
<td>15.5</td>
<td></td>
<td>1.50</td>
<td></td>
<td>22.8</td>
<td></td>
</tr>
<tr>
<td>36_7_R2</td>
<td>15.7</td>
<td>15.5</td>
<td>1.33</td>
<td>1.38</td>
<td>26.4</td>
<td>24.5</td>
</tr>
<tr>
<td>36_7_R3</td>
<td>15.3</td>
<td></td>
<td>1.32</td>
<td></td>
<td>24.2</td>
<td></td>
</tr>
<tr>
<td>36_5_R1</td>
<td>14.9</td>
<td></td>
<td>1.60</td>
<td></td>
<td>12.5</td>
<td></td>
</tr>
<tr>
<td>36_5_R2</td>
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<td>14.2</td>
<td>1.44</td>
<td>1.55</td>
<td>11.5</td>
<td>11.9</td>
</tr>
<tr>
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<td>14.4</td>
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<td>1.60</td>
<td></td>
<td>11.7</td>
<td></td>
</tr>
<tr>
<td>36_4_R1</td>
<td>14.5</td>
<td></td>
<td>2.26</td>
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<td>8.40</td>
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</tr>
<tr>
<td>36_4_R2</td>
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<td>14.4</td>
<td>1.96</td>
<td>2.16</td>
<td>8.08</td>
<td>8.20</td>
</tr>
<tr>
<td>36_4_R3</td>
<td>14.6</td>
<td></td>
<td>2.25</td>
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</table>
Table 3-4: Summary of results [initiation of buckling and comparison with AISI S310 Section D2 (AISI 2020)]

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$P_{nb}$ (kips)</th>
<th>$P_{nb,ave}$ (kips)</th>
<th>$\Delta P_{nb}$ (in)</th>
<th>$\Delta P_{nb,ave}$ (in)</th>
<th>$P_{nb,AISI}$ (kips)</th>
<th>$P_{nb}/P_{nb,AISI}$</th>
<th>$(P_{nb}/P_{nb,AISI})_{ave}$</th>
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<tbody>
<tr>
<td>36_7_R1</td>
<td>10.9</td>
<td>0.38</td>
<td>9.45</td>
<td>1.15</td>
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</tr>
<tr>
<td>36_7_R2</td>
<td>10.2</td>
<td>0.29</td>
<td>9.45</td>
<td>1.08</td>
<td>1.09</td>
<td></td>
<td></td>
</tr>
<tr>
<td>36_7_R3</td>
<td>9.78</td>
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<td>9.45</td>
<td>1.03</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>36_5_R1</td>
<td>10.4</td>
<td>0.62</td>
<td>9.45</td>
<td>1.10</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>36_5_R2</td>
<td>9.62</td>
<td>0.62</td>
<td>9.45</td>
<td>1.02</td>
<td>1.03</td>
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<td></td>
</tr>
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<td>0.97</td>
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<tr>
<td>36_4_R1</td>
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<td>0.92</td>
<td>9.45</td>
<td>1.05</td>
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</tr>
<tr>
<td>36_4_R2</td>
<td>9.93</td>
<td>0.94</td>
<td>9.45</td>
<td>1.05</td>
<td>1.05</td>
<td></td>
<td></td>
</tr>
<tr>
<td>36_4_R3</td>
<td>9.76</td>
<td>0.91</td>
<td>9.45</td>
<td>1.03</td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

Table 3-5: Summary of results [Stiffness comparison with AISI S310 Section D5 (AISI 2020)]

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$G'$ (kips/in)</th>
<th>$G'_{ave}$ (kips/in)</th>
<th>$G'_{AISI}$ (kips/in)</th>
<th>$G'<em>{ave}/G'</em>{AISI}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>36_7_R1</td>
<td>22.8</td>
<td>24.5</td>
<td>72.9</td>
<td>0.34</td>
</tr>
<tr>
<td>36_7_R2</td>
<td>26.4</td>
<td>24.5</td>
<td>72.9</td>
<td>0.34</td>
</tr>
<tr>
<td>36_7_R3</td>
<td>24.2</td>
<td>24.5</td>
<td>72.9</td>
<td>0.34</td>
</tr>
<tr>
<td>36_5_R1</td>
<td>12.5</td>
<td>11.9</td>
<td>18.7</td>
<td>0.64</td>
</tr>
<tr>
<td>36_5_R2</td>
<td>11.5</td>
<td>11.9</td>
<td>18.7</td>
<td>0.64</td>
</tr>
<tr>
<td>36_7_R3</td>
<td>11.7</td>
<td>11.9</td>
<td>18.7</td>
<td>0.64</td>
</tr>
<tr>
<td>36_4_R1</td>
<td>8.40</td>
<td>8.20</td>
<td>13.6</td>
<td>0.60</td>
</tr>
<tr>
<td>36_4_R2</td>
<td>8.08</td>
<td>8.20</td>
<td>13.6</td>
<td>0.60</td>
</tr>
<tr>
<td>36_4_R3</td>
<td>8.12</td>
<td>8.20</td>
<td>13.6</td>
<td>0.60</td>
</tr>
</tbody>
</table>
Figure 3-16: Summary of sensor force-out-of-plane displacement ($\Delta z$)
3.5.1.1. Estimation of peak load ($P_{\text{max}}$) and corrected displacement

To account for initial actuator load and lateral translation of the rig, the measured force and displacement results were corrected using initial force readings and adapted forms of the equations (Equation 3-2) presented in AISI S907 (AISI 2017) respectively. Figure 3-17 below shows the sensor locations that were used to measure in-plane translations of the rig. These displacements were subtracted from the measured displacements using Equation 3-2 to obtain the pure shear displacement ($\Delta_s$) of the specimen.

\[ \Delta_s = \Delta_3 - \Delta_1 \]  

Figure 3-17: Estimation of corrected displacement [Adapted from AISI S907 (AISI 2017)]
3.5.1.2. Estimation of load \((P_{nb})\) and displacement \((\Delta_{nb})\) level corresponding with onset of out-of-plane buckling

Results from the out-of-plane sensors (Sensors A1 through C3, Figure 11) were used to establish onset of out-of-plane buckling in the specimen. Sensor displacement-time results were evaluated to visually identify the time step at which out of plane displacement was recorded by the sensors. This time step was correlated with the actuator force-time results to determine force required \((P_{nb})\) to initiate out of plane buckling in the specimen. This force level was then correlated with the corrected force-shear displacement plots to determine displacement level \((\Delta_{nb})\) at which first signs of out-of-plane motion were recorded. The force level \((P_{nb})\) was also verified by comparing results with sensor force-displacement results. This process has been visualized for the 36/5-R3 specimen in Figure 3-18 through 3-20.

![Graphs](image)

**Figure 3-18:** Determining time step corresponding with onset of out-of-plane buckling. Bottom three plots represent insets of corresponding top three plots.
Figure 3-19: Determining force level ($P_{nb}$) corresponding with time step

Figure 3-20: (a) Determining displacement level $\Delta_{nb}$ (b) verification of $P_{nb}$
3.5.1.3. Estimation of Stiffness (\(G'\))

Stiffness of the specimen were calculated from the corrected force-shear displacement results at the 40% peak load level (\(P_d\) and \(\Delta_d\)) as depicted in Figure 3-21 below. This was based on recommendations from AISI S907 – Test Standard for determining the strength and stiffness of cold-formed steel diaphragms by the cantilever test method (AISI 2017).

![Figure 3-21: Estimation of diaphragm stiffness, G' (AISI 2017)](image)

\[
G' = \frac{P_d \times a}{\Delta_d \times b}
\] (12)

3.5.2. 36-7 Fully attached specimen

The 36/7 fully attached specimen had an average peak capacity (\(P_{max,ave}\)) of 15.5 kips (66.7 kN) at an average displacement (\(\Delta_{Pmax}\)) of 1.50 in (38.1 mm). Initiation of out-of-plane buckling, as indicated by the out-of-plane sensors, occurred at an average load (\(P_{nb,ave}\)) of 10.3 kips (45.8 kN) (Table 3-3 and Figure 3-15) at 0.32 in (8.12 mm) of average displacement (\(\Delta_{pb,ave}\)). Strength to
predicted ratios for individual repetitions have been summarized in Table 3-4. The average strength to predicted capacity ratios for the maximum capacity \( \left( \frac{P_{\text{max}}}{P_{nb, AISI}} \right) \) and initiation of buckling \( \left( \frac{P_{nb}}{P_{nb, AISI}} \right) \) were 1.64 and 1.09 respectively. The average stiffness \( \left( G'_{\text{ave}} \right) \) for the 36/7 repetitions was calculated to be 24.5 kips/in \((4.29 \text{ kN/mm}) \) (Table 3-3). Typical buckled shape observed during testing can be seen in Figure 3-22 below.

These tests were terminated either due to large post-peak deflections, flattening of flutes, or post-peak connection failures. The 36/7 – R1 test was terminated due to excessive flattening of the corner flute (Figure 3-23.a) in Panel 01 at the West support. The 36/7 – R2 test was terminated due to a post-peak pull out failure of the support fastener in Panel 01 on the West support (Figure 3-23.b). The 36/7 – R3 test was terminated due to excessive post-peak deformations and flattening of the corner flute on the West support (Figure 3-23.c).

![Typical buckled shape observed in the 36/7 repetitions](image)

**Figure 3-22: Typical buckled shape observed in the 36/7 repetitions**
Figure 3-23: Observed post-peak failure modes in the 36/7 repetitions: (a) 36/7-R1 (b) 36/7-R2 (c) 36/7-R3

3.5.3. 36-5 Intermediately attached specimen

The 36/5 intermediately attached specimen had an average peak capacity ($P_{\text{max,ave}}$) of 14.2 kips (63.2 kN) at an average displacement ($A_{\text{Pmax}}$) of 1.62 in (41.1 mm). Initiation of out-of-plane buckling, as indicated by the out-of-plane sensors, occurred at an average load ($P_{\text{nb,ave}}$) of 9.70 kips (43.1 kN) (Table 3-3 and Figure 3-15) at 0.61 in (15.5 mm) of average displacement ($A_{\text{Pnb,ave}}$). The average strength to code predicted capacity ratios for the maximum capacity ($P_{\text{max}}/P_{\text{AISI}}$) and initiation of buckling ($P_{\text{nb}}/P_{\text{AISI}}$) were 1.50 and 1.03 respectively. The average stiffness ($G'_{\text{ave}}$) for the 36/5 repetitions was calculated to be 11.9 kips/in (2.08 kN/mm) (Table 3-3). Typical buckled shape observed during testing can be seen in Figure 3-24 below.

Test repetitions 36/5-R1 and 36/5 R3 were terminated due to post-peak flattening of flutes and bearing/pull-over failures of the support fasteners (Figure 3-25). The 36/5 – R1 specimen had excessive flattening of the flutes at the supports as can be seen in Figure 3-23.a. The 36/5 – R3 repetition failed due to post peak bearing/tilting of fasteners which was followed by pull over (Figure 3-25.b). Repetition 36/5-R2 was terminated due to unzipping of the sidelaps caused by failure of the sidelap connections between panel 03 and panel 04 at sidelap seam 3-4 (Figure 3-25.b).
Figure 3-24: Typical buckled shape observed in the 36/5 repetitions

Figure 3-25: Observed ultimate failure modes in the 36/5 repetitions:
(a) 36/5-R1 (b) 36/5-R2 (c) 36/5-R3

This occurred due to a fabrication/construction error during crimping of the panels as the overlapping ends were not sufficiently lapped to ensure a proper connection. The difference in failure mode of the connection due to the improper crimping can be seen in Figure 3-26. Figure 3-26 shows two pictures comparing engagement of the vertical end with the overlapping end after the overlapping end has been removed. Here, when the two overlapping ends were lapped
adequately, the connection failed after tearing off the overlapping edges. With the inadequate connection, the seams show minimal signs of tearing, and the connection fails due to opening of the crimp. Since the failure of the sidelap occurred in the post-peak range, the test was deemed acceptable. However, this repetition has a noticeably lower ultimate ($P_{\text{max}}$) capacity, 10% lower on average, when compared to other repetitions in the set.

![Figure 3-26: (a) Proper vs (b) improper sidelap connection at failure](image)

3.5.4. 36-4 Skip pattern specimen

The 36/4 skip pattern specimen had an average peak capacity ($P_{\text{max,ave}}$) of 14.4 kips (64.1 kN) at an average displacement ($A_{P_{\text{max}}}$) of 2.21 in (56.1 mm). Initiation of out-of-plane buckling occurred at an average load ($P_{\text{nb,ave}}$) of 9.87 kips (43.9 kN) (Table 3-3 and Figure 3-15) at 0.92 in (23.4 mm) of average displacement ($A_{P_{\text{nb,ave}}}$). The average strength to code predicted capacity ratios for the maximum capacity ($P_{\text{max}}/P_{\text{nb, AISI}}$) and initiation of buckling ($P_{\text{nb}}/P_{\text{AISI}}$) were 1.52 and 1.05 respectively. The average stiffness ($G'_{\text{ave}}$) for the 36/4 repetitions was calculated to be 8.20 kips/in (1.44 kN/mm) (Table 3-3). Typical buckled shape observed during testing can be seen in Figure 3-27 below.
The 36/4 test repetitions were terminated due to significant warping and flattening of flutes over the supports and post peak connection failures. Pull over, pullout, bearing, edge tear out, and tilting were observed in the repetitions. The 36/4 – R1 specimen ultimately failed due to pull over of a support fastener over the west support [Figure 3-28(a)] Panel 01. The 36/4 – R2 specimen test run was terminated due to bearing and subsequent pull out of the fastener over the East support in Panel 01 [Figure 3-28(b)]. The 36/4 – R3 specimen failed due to fastener failures caused by pullout, edge tear out, and bearing/tilting over the west support [Figure 3-28(c) and 3-28(d)]. Significant warping present throughout the panel ends.

Figure 3-27: Typical buckled shape observed in the 36/4 repetitions
3.5.5. Influence on buckling capacity and displacement

Changing support attachment pattern had a negligible impact on peak force ($P_{\text{max}}$) observed in the tests, about 7.7% average. When support fasteners were reduced from the 36/7 pattern to the reduced 35/5 and 36/4 patterns, capacity reduced by 8.3% and 7.3% respectively. However, $P_{\text{max}}$ was achieved at varying displacement levels as can be seen in Figure 3-15 and Table 4. The average displacement increased from 1.50 in (38.1 mm) to 1.62 in (41.1 mm) for the 36/5 and to 2.21 in (56.1 mm) for the 36/4 pattern tests respectively. This was an increase of 7.8% for the 36/5 and 47.6% for the 36/4 repetitions. The average ultimate strength to predicted ratios ($P_{\text{max}}/P_{nb,AISI}$) for the 36/7, 36/5, and 36/4 repetitions were 1.64, 1.50, and 1.52 respectively.

Changing support attachment pattern from the 36/7 pattern to the 36/5 and 36/4 pattern reduced the load at which out-of-plane buckling initiated ($P_{nb}$) by 5% and 4% respectively. The average strength to predicted ratios when comparing initiation of buckling strength with expected capacity ($P_{nb}/P_{nb,AISI}$) for the 36/7, 36/5, and 36/4 repetitions were 1.09, 1.03, and 1.05, respectively.
3.5.6. Influence on stiffness

Changing support attachment pattern had a significant impact on the specimen’s stiffness (Figure 3-15 and Table 3-3). As support fasteners were reduced from the fully attached pattern (36/7) to the intermediate (36/5) and skip patterns (36/4), the average stiffness for the set of repetitions reduced from 24.5 kips/in (4.29 kN/mm) to 11.9 kips/in (2.08 kN/mm) and 8.20 kips/in (1.44 kN/mm) respectively. This is a 51% and 67% percent reduction for the 36/5 and 36/4 patterns, respectively, when compared to the fully attached case. The stiffness observed from the experimental tests ($G'_{exp}$) did not agree well with predictive methods in AISI S310 – 20 Section D5 (AISI 2020) \([mean(G'_{FEA}/G'_{AISI}) = 0.52]\). The 36/5 specimens were best predicted \([mean(G'_{FEA}/G'_{AISI})_{36/5} = 0.90]\) and the fully attached 36/7 specimens had the lowest prediction ratio \([mean(G'_{FEA}/G'_{AISI})_{36/7} = 0.34]\).

3.5.7. Influence on end-warping behavior

Although the ends of the specimen were not instrumented, video recordings of the tests show that the three attachment patterns exhibited different end-warping behavior. The 36/7 pattern (Figure 3-29, highlighted in red) showed uniform warping across all the flutes with all the flutes warping approximately uniformly in the same direction. The 36/5 attachment pattern showed non-uniform warping behavior with differences observed in the attached and unattached flutes (Figure 3-30, highlighted in red). The two fully attached flutes on either ends of the panels warped similarly and the warping behavior was different from the partially attached interior four flutes. The 36/4 specimen warped uniformly with subsequent flutes alternatingly warping upwards or downwards as can be seen in Figure 3-31 (highlighted in red).
Figure 3-29: Typical end-warping behavior observed in the 36/7 specimens

Figure 3-30: Typical end-warping behavior observed in the 36/5 specimens
3.6. Experimental program conclusions

To investigate the influence of industry standard support fastener attachment patterns on the AISI S310 and DDM04 panel buckling limit state, nine monotonic tests were conducted. These tests were identical in configuration except for the support attachment pattern. Three unique support attachment patterns were evaluated, and three repetitions were performed for each set. The specimens were instrumented with displacement sensors to capture the onset of buckling. Based on the observations from these tests, the following key conclusions were drawn:

- Support attachment pattern had negligible impact on the ultimate capacity (7.7%) and load at which buckling initiates (4.5%)
- Reducing support fasteners from the 36/7 pattern to the 36/5 and 36/4 patterns increased displacement at ultimate load level by 7.80% and 47.6% respectively
• Current design equations provided a conservative estimate for the ultimate capacity of the specimen (Mean test to predicted ratio = 1.55) but provided accurate estimates of the load at which out-of-plane buckling initiated (Mean test to predicted ratio = 1.05)
• Reducing support attachments from the 36/7 pattern to the 36/5 and 36/4 reduced the initial stiffness of the test specimen by 51% and 67% respectively
• Current design equations provided stiffness estimates that were significantly stiffer than experimental results (Mean test to predicted ratio = 0.52)

3.7. Finite element analysis (FEA) expansion

The experimental results presented in Section 6 of this report were utilized to develop and calibrate finite element analysis (FEA) models capable of capturing experimental strength and buckling behavior observed during testing. The FEA models utilized non-linear idealized material properties, idealized connection behavior, and contact definitions. The calibrated models were used to predict onset of buckling, ultimate capacities, and stiffness for 18 gage and 20 gage Type B deck for comparison with the predictive equation in AISI S310 (AISI 2020) and SDI DDM04 (Luttrell 2015). This section of the report presents and discusses the FEA modelling methodology, validation process, findings of the numerical simulations, and performance of predictive equations.

3.7.1. Numerical modelling matrix

The numerical modeling matrix for the FEA expansion can be seen in Table 3-6 below. The 20 gage and 18 gage simulation models are identical to the tested 22 gage specimen in deck profile (Type B), span length ($L_v$), support member thickness, and attachment patterns (36/7, 36/5, and 36/4) and only differ in the simulated base metal thickness. This ensures that deck thickness is the only variable across the models and the effect of thickness on panel buckling capacity can be isolated.
Table 3-6: Numerical modelling matrix for FEA Expansion

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Attachment Pattern</th>
<th>$L_v$ (ft)</th>
<th>Deck Thickness (in)</th>
<th>Support Thickness (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>36/7 - 22 – 16</td>
<td>36/7</td>
<td>15</td>
<td>0.0295</td>
<td></td>
</tr>
<tr>
<td>36/7 - 20 – 16</td>
<td>36/7</td>
<td>15</td>
<td>0.0358</td>
<td>0.054</td>
</tr>
<tr>
<td>36/7 - 18 – 16</td>
<td>36/7</td>
<td>15</td>
<td>0.0474</td>
<td></td>
</tr>
<tr>
<td>36/5 - 22 – 16</td>
<td>36/5</td>
<td>15</td>
<td>0.0295</td>
<td></td>
</tr>
<tr>
<td>36/5 - 20 – 16</td>
<td>36/5</td>
<td>15</td>
<td>0.0358</td>
<td>0.054</td>
</tr>
<tr>
<td>36/5 - 18 – 16</td>
<td>36/5</td>
<td>15</td>
<td>0.0474</td>
<td></td>
</tr>
<tr>
<td>36/4 - 22 – 16</td>
<td>36/4</td>
<td>15</td>
<td>0.0295</td>
<td></td>
</tr>
<tr>
<td>36/4 - 20 – 16</td>
<td>36/4</td>
<td>15</td>
<td>0.0358</td>
<td>0.054</td>
</tr>
<tr>
<td>36/4 - 18 – 16</td>
<td>36/4</td>
<td>15</td>
<td>0.0474</td>
<td></td>
</tr>
</tbody>
</table>

3.8. FEA modelling methodology

Specimens based on the modelling matrix proposed in Section 3.7.1 were modelled and analyzed using the commercially available finite element analysis software ABAQUS Version 6.14 (ABAQUS 2014). The following sub-sections present the modelled simplified deck and support geometry, material properties, interactions/constraints, and boundary conditions utilized for the non-linear analysis.

3.8.1. Model geometry and material properties

The FEA model geometry was defined to be identical to the tested specimens and the idealization can be seen in Figure 3-32 below. Three interconnected full width (36-inches) light gage steel deck panels and one partial panel (12-inches) were modelled which were connected to the underlying frame. The overall size of the FEA model was 10 feet by 15 feet (Figure 3-32).
The light gage steel deck was modelled with repeating configurations as can be seen in Figure 3-33. The dimensions for the corrugations were based on nominal dimensions provided by the deck manufacturer. The geometry of the interlocking sidelaps was simplified into 0.75-inch-high flat plates as can be seen in Figure 3-33 below (0.75 in vertical position at the left of the corrugation).

Figure 3-32: FEA idealization of test specimen

Figure 3-33: Repeating corrugation dimensions (in inches)
The support angles and cold-formed steel (CFS) framing members were modelled based on nominal dimensions of the framing members used in the experimental testing. The modelled cross-section geometry can be seen in Figure 3-34 below.

![Figure 3-34: Cross-section dimensions of the support framing members (in inches)](image)

An idealized bi-linear material model was utilized to model the nominal stress-strain behavior of steel (Figure 3-35). The yield strength, $F_y$, and ultimate strength, $F_u$, were set to 50 ksi and 65 ksi respectively. Yield point, $\varepsilon_y$, and elongation at failure, $\varepsilon_u$, were assumed to be 0.02% and 18% respectively. The modulus of elasticity, $E$, and Poisson’s ratio was assumed to be 29,500 ksi and 0.3 respectively. Engineering stress and strain were converted to plastic stress and strain to include material plasticity in the models.
3.8.2. Interactions and constraints

Inbuilt ABAQUS (ABAQUS 2014) multi-point constraints (MPC) and point based fasteners were utilized in the model to apply boundary conditions and simulate connections respectively. Both the fixed and load joist were constrained to reference nodes located in the middle of the joist webs using MPCs (Figure 3-36). These reference points were then used to restrict degrees of freedom and apply displacements in the static general load step.

Connections between the deck and the underlying frame were modelled using the inbuilt Abaqus point-based fasteners. The fastener behavior in U1, U2, and U3 direction was defined as a rigid
MPC to prevent any slip or deformation at fastener locations. Here, U1, U2, and U3 correspond with the X, Z, and Y axis depicted in Figure 3-32. A second bi-linear fastener model was also used as an alternative to the rigid fasteners to study the influence of fastener deformation on panel buckling and stiffness. The bi-linear fastener stiffness and peak capacity were based on tests by Tao et al. (Tao et al. 2017) and the idealized behavior can be seen in Figure 3-37 below in orange. While the connection test was not identical to the experimentally tested configuration, it served as a reasonable approximation to determine the impact of connection models on strength and stiffness. Sidelap connections were not modelled explicitly in the FEA simulations and a tie constraint was used to join the vertical flats of the panels to simulate the interlocking deck and VSC-II connection (Figure 3-36). Implications of the fastener and VSC-II modelling methodology have been discussed in the results section of this report.

![F-Δ response 54-33-12-M](image)

**Figure 3-37: Fastener behavior (orange) utilized in FEA models to improve stiffness**

Contacts were defined between the ends of the panels and exterior support beams and the bottom flanges of the exterior panels and free and fixed support joist flanges (Figure 3-38). ABAQUS
default “hard” contacts were utilized in the U2 direction and “frictionless” behavior was defined in the U1 and U3 direction (ABAQUS 2014). This was done to prevent the steel panels from penetrating the underlying frame in the FEA models and to simulate realistic warping restraint at the deck ends. The frictionless behavior also ensured that the only restraint the panel ends had in the U1 and U2 direction were due to the MPC connections.

![Deck end and exterior support](image1.png) ![Exterior panel and load/fix beams](image2.png)

**Figure 3-38: Contact definition locations in the model for panel ends and support members and panels edges and loaded/fixed beams**

### 3.8.3. Meshing details

The deck panels were meshed with S4R quadrilateral shell elements with 7 integration points through the thickness. The support angles and CFS members were also modelled with S4R elements. A 0.5 in global mesh size was used to discretize the deck panel and 1.0 in global mesh size was used to discretize the support framing members. The panel mesh size ensured that at least 7, 3, and 4 numbers of elements were present in the top flat, web flat, and bottom flat respectively. This ensured that local buckling could also be captured in the models in addition to global buckling behavior.
3.8.4. Solvers, boundary conditions, and applied loading/displacement

A static general load step was defined to incrementally apply the displacement in the FEA simulation. Maximum iterations were set to 500 and the initial and maximum increment size was set to 0.0015 inches and 0.03 inches respectively. The minimum size was set to 3E-25 inch to aid with convergence during the local buckling portion of the analysis. Non-linear geometry was also activated for the load step. Boundary conditions were imposed on the FEA model by restraining degrees of freedom of the underlying frame to replicate experimental conditions as can be seen in Figure 39 below. The fixed beam was restricted in all degrees of freedom (U1 = U2 = U3 = UR1 = UR2 = UR3 = 0). The free/load beam was restricted in all degrees of freedom except for U3 and U1 (U2 = UR1 = UR2 = UR3 = 0). Both exterior support beams were restricted by setting U2, UR1, and UR3 to 0 to allow these beams to pivot about their connection to the fixed beam and loaded beam. This also allowed the underlying frame to behave like a pin-jointed frame. A displacement of 3-inches was applied to the free beam in the U3 direction as was done in the experimental tests. The beam was free to move in the U1 direction.
Figure 3-40: Applied Boundary conditions to the underlying frame for the non-linear analysis
(U1 = X, U2 = Z, U3 = Y)

3.9. Comparison with experimental data

Figure 3-38 compares the force versus shear displacement response of the FEA models with experimental tests. The FEA models accurately predicted peak capacity and mean experimental to simulated capacity ($P_{max, exp}/P_{max, FEA}$) ratio was 0.95. However, displacement at peak load ($\Delta_{exp}$) was not accurately predicted (Table 3-7) and the mean predicted displacement at ultimate load ($\Delta_{exp}$) versus predicted ($\Delta_{FEA}$) was 1.43. The FEA models were also stiffer (Table 8) than the experimental tests (Mean experimental to FEA predicted stiffness ratio, $G'_{exp}/G'_{FEA} = 0.78$). Global (out-of-plane) buckling, local buckling, and end-warping behavior observed in the experimental tests were accurately captured by the FEA models and Figures 3-40 through Figure 3-42 compare the FEA simulations with experimental tests. The FEA models accurately predicted overall buckled shapes, local buckling near deck ends, and the end-warping behavior of flutes.
Figure 3-41: Comparison of experimental and FEA force displacement response

Table 3-7: Comparison of FEA and experimental peak strength and displacement

<table>
<thead>
<tr>
<th>Configuration</th>
<th>$P_{\text{max,exp}}$</th>
<th>$P_{\text{max,FEA}}$</th>
<th>$P_{\text{max,exp}}/P_{\text{max,FEA}}$</th>
<th>$\Delta_{\text{exp}}$</th>
<th>$\Delta_{\text{FEA}}$</th>
<th>$\Delta_{\text{exp}}/\Delta_{\text{FEA}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>36/7 - 22 gage</td>
<td>15.48</td>
<td>16.02</td>
<td>0.97</td>
<td>1.50</td>
<td>0.82</td>
<td>1.82</td>
</tr>
<tr>
<td>36/5 - 22 gage</td>
<td>14.93</td>
<td>15.54</td>
<td>0.96</td>
<td>1.62</td>
<td>1.24</td>
<td>1.31</td>
</tr>
<tr>
<td>36/4 - 22 gage</td>
<td>14.36</td>
<td>15.47</td>
<td>0.93</td>
<td>2.21</td>
<td>1.93</td>
<td>1.15</td>
</tr>
<tr>
<td>Mean</td>
<td></td>
<td></td>
<td>0.95</td>
<td>1.43</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3-8: Comparison of FEA and experimental stiffness and reduction in stiffness due to changing fastener pattern

<table>
<thead>
<tr>
<th>Configuration</th>
<th>$G'_{\text{exp}}$</th>
<th>$G'_{\text{FEA}}$</th>
<th>$G'<em>{\text{exp}}/G'</em>{\text{FEA}}$</th>
<th>Reduction in $G'$</th>
</tr>
</thead>
<tbody>
<tr>
<td>36/7 - 22 gage</td>
<td>24.46</td>
<td>34.08</td>
<td>0.72</td>
<td>$\frac{0.66}{0.73} = 0.90$</td>
</tr>
<tr>
<td>36/5 - 22 gage</td>
<td>11.79</td>
<td>16.89</td>
<td>0.70</td>
<td>$\frac{0.52}{0.50} = 1.03$</td>
</tr>
<tr>
<td>36/4 - 22 gage</td>
<td>8.24</td>
<td>9.06</td>
<td>0.91</td>
<td>$\frac{0.30}{0.46} = 0.65$</td>
</tr>
<tr>
<td>Mean</td>
<td></td>
<td></td>
<td></td>
<td>0.86</td>
</tr>
</tbody>
</table>
Possible causes for the deviation in stiffness ($G'$) and displacement at ultimate load ($Δ$) were investigated and determined to be a combination of connection stiffness and the material model utilized in the simulations. FEA models with multi-linear fastener behavior were analyzed and results were compared with experimental response (Figure 3-39). Coupon tests available in literature (Torabian and Schafer 2021) of identical deck from the manufacturer also showed that the steel had slightly higher elongation at failure ($ε_u$) of average 20% vs 18%. Material models based on the increased elongation were utilized to rerun the FEA models. The revised models showed great improvements in stiffness predictions for the 36/5 and 36/4 tests ($\frac{G'_\text{exp}}{G'_\text{FEA}} = 1.11$ and 0.96 respectively) but lead to convergence issues and terminated prematurely (Figure 3-42). This was also accompanied by a large increase in computational run time (more than 1500 increments), significantly reducing the efficiency of the models. Further, negligible improvement was observed for the 36/7 simulations and hence the multi-linear faster modelling methodology was not utilized in the numerical modelling.

![Figure 3-42: Comparison of experimental and FEA force displacement response with bi-linear fastener data and increased ductility material models](image-url)
Although the models did not capture experimental stiffness behavior, the strength, buckling, and end-warping behavior predictions were in good agreement with experimental results. The rigid connections resulted in overly stiff models as connector and deck slip at the connections was completely eliminated in the models. However, the models were efficient, and the modelling methodology did not require experimental fastener data or coupon-tested material properties for strength predictions. Hence, the models were deemed suitable for predicting buckling capacities of similar untested configurations.
Figure 3-43: Comparison of FEA and experimental deformed states for the 36/4 skip pattern specimen
Figure 3-44: Comparison of FEA and experimental deformed states for the 36/5 specimen

Comparison of overall specimen buckled shape

Local buckling near deck ends

Comparison of end warping behavior
Figure 3-45: Comparison of FEA and experimental deformed states for the 36/7 fully attached specimen
3.10. Parametric study summary results

The FEA models developed and presented in section 3.8 and 3.9 were utilized to predict capacities for 20-gage and 18-gage Type B deck to evaluate the influence of support attachment patterns across two more commonly used deck thicknesses. The force vs. shear displacement results for the entire parametric evaluation can be seen in Figure 3-43 below. Table 3-9 summarized the observed peak strength ($P_{max,FEA}$), displacement at $P_{max,FEA}$, ($\Delta_{FEA}$), and stiffness ($G'_{FEA}$).

As observed in the experimental tests, onset of buckling did not coincide with a change in overall force-displacement behavior. Hence out-of-plane displacement at midspan nodes of the FEA specimen were extracted and plotted against overall force to qualitatively determine onset of buckling. The location of measurement of out-of-plane displacement and force versus out-of-plane
displacement results for the 9 configurations can be seen in Figure 3-44 and Figure 3-46 respectively. For all the specimens, the AISI S310 out-plane buckling equation provided conservative estimates for initiation of buckling as can be seen in Figure 3-43.

Table 3-9: Summary of FEA predicted ultimate capacities and stiffness (non-linear analysis)

<table>
<thead>
<tr>
<th>Configuration</th>
<th>( P_{\text{max,FEA}} ) (kips)</th>
<th>( \Delta_{\text{FEA}} ) (in)</th>
<th>( G'_{\text{FEA}} ) (kips/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>36/7 - 22 gage</td>
<td>16.02</td>
<td>0.82</td>
<td>34.08</td>
</tr>
<tr>
<td>36/5 - 22 gage</td>
<td>15.54</td>
<td>1.24</td>
<td>16.89</td>
</tr>
<tr>
<td>36/4 - 22 gage</td>
<td>15.47</td>
<td>1.93</td>
<td>9.06</td>
</tr>
<tr>
<td>36/7 - 20 gage</td>
<td>23.02</td>
<td>0.70</td>
<td>64.39</td>
</tr>
<tr>
<td>36/5 - 20 gage</td>
<td>20.62</td>
<td>1.61</td>
<td>22.41</td>
</tr>
<tr>
<td>36/4 - 20 gage</td>
<td>19.17</td>
<td>1.58</td>
<td>12.93</td>
</tr>
<tr>
<td>36/7 - 18 gage</td>
<td>36.24</td>
<td>0.64</td>
<td>101.36</td>
</tr>
<tr>
<td>36/5 - 18 gage</td>
<td>31.89</td>
<td>1.16</td>
<td>39.49</td>
</tr>
<tr>
<td>36/4 - 18 gage</td>
<td>30.06</td>
<td>1.67</td>
<td>26.48</td>
</tr>
</tbody>
</table>

Figure 3-47: Location of out-of-plane displacement measurements for determining onset of buckling
Determining onset-of-buckling visually from the out-of-plane versus force results was possible for the 20-gage and 18-gage deck but proved to be challenging for the 22-gage 36/5 and 36/4 attachment pattern simulations. Unlike the other 7 simulated configurations, both the 22-gage 36/5 and 36/4 simulations did not have a clear point where sudden change in stiffness could be clearly detected (Figure 3-48).
Figure 3-48: Summary of FEA out-of-plane displacement ($\Delta z$) of flutes at midspan versus applied force.
3.11. Comparison with predictive methods

The ultimate capacity \( P_{\text{max,FEA}} \) and stiffness \( G' \) predicted by the FEA models were compared with predictive equations from AISI S310 – 20 (AISI 2020) and DDM04 (Luttrell 2015). The following subsections discuss the performance of the predictive equations.

3.11.1. Comparison of predicted capacity from AISI S310 - 20 (AISI 2020) with ultimate capacity from FEA

The ultimate capacities \( P_{\text{max,FEA}} \) predicted by the FEA models were compared with capacities \( P_{\text{nb,AISI}} \) predicted by Equation D2-1 (Equation 1-12 in this report) from AISI S310 - 20 (AISI 2020). Table 10 summarizes the measured (FEA) to predicted (AISI) ratios for the parametric evaluation. The FEA models predicted significant overstrength in the out-of-plane buckling limit state \( \text{mean}(P_{\text{max,FEA}}/P_{\text{nb,AISI}}) = 1.50 \), but these capacities were achieved well into the non-linear range of the force-displacement response and after initiation of out-of-plane buckling (Figure 45 and Figure 48). Also, from experimental testing, it was observed that the peak capacity achieved is highly dependent on connection integrity and behavior (Experimental specimen 36/5 R1/R3 vs 36/5 R2) and hence can vary based on connection detailing. Further, there was a drop in capacity in the FEA simulations when comparing the fully attached simulations (36/7) with partial attachments (36/5 and 36/4) for the 18-gage and 22-gage deck. This reduction was about 6% and 17% when comparing the fully attached (36/7) pattern with the intermediate (36/5) and skip (36/4) patterns respectively. However, the predictive equation predicted buckling capacities that coincided well with onset of buckling and hence no modifications have been suggested.
Table 3-10: Comparison of non-linear analysis predicted capacities and AISI S310 -20 Section D2 capacities (AISI 2020)

<table>
<thead>
<tr>
<th>Configuration</th>
<th>thickness</th>
<th>$P_{\text{max,FEA}}$ (kips)</th>
<th>$P_{\text{nb,AISI}}$ (kips)</th>
<th>$P_{\text{max,FEA}}/P_{\text{nb,AISI}}$ (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>36/7 - 22 gage</td>
<td>0.0300</td>
<td>16.02</td>
<td>10.59</td>
<td>1.51</td>
</tr>
<tr>
<td>36/5 - 22 gage</td>
<td>0.0300</td>
<td>15.54</td>
<td>10.59</td>
<td>1.47</td>
</tr>
<tr>
<td>36/4 - 22 gage</td>
<td>0.0300</td>
<td>15.47</td>
<td>10.59</td>
<td>1.46</td>
</tr>
<tr>
<td>36/7 - 20 gage</td>
<td>0.0358</td>
<td>23.02</td>
<td>13.84</td>
<td>1.66</td>
</tr>
<tr>
<td>36/5 - 20 gage</td>
<td>0.0358</td>
<td>20.62</td>
<td>13.84</td>
<td>1.49</td>
</tr>
<tr>
<td>36/4 - 20 gage</td>
<td>0.0358</td>
<td>19.17</td>
<td>13.84</td>
<td>1.39</td>
</tr>
<tr>
<td>36/7 - 18 gage</td>
<td>0.0474</td>
<td>36.24</td>
<td>21.57</td>
<td>1.68</td>
</tr>
<tr>
<td>36/5 - 18 gage</td>
<td>0.0474</td>
<td>31.89</td>
<td>21.57</td>
<td>1.48</td>
</tr>
<tr>
<td>36/4 - 18 gage</td>
<td>0.0474</td>
<td>30.06</td>
<td>21.57</td>
<td>1.39</td>
</tr>
</tbody>
</table>

Mean 1.50

St. Dev 0.10

3.11.2. Comparison of predicted stiffness from AISI S310 -20 (AISI 2020) with stiffness from FEA

The stiffness ($G'_{\text{FEA}}$) predicted by the FEA models were compared with stiffness ($G'_{\text{AISI}}$) predicted by following the procedure outlined in Section D5 from AISI S310 - 20 (AISI 2020). Table 3-11 summarizes the measured (FEA) to predicted (AISI) ratios for the parametric evaluation. The FEA models were more flexible than the AISI predictions [$mean(G'_{\text{FEA}}/G'_{\text{AISI}}) = 0.67$] and there was high variability associated with the predictions. The intermediately attached pattern (36/5) was best predicted [$mean(G'_{\text{FEA}}/G'_{\text{AISI}})_{36/5} = 0.80$] and the fully attached pattern (36/7) [$mean(G'_{\text{FEA}}/G'_{\text{AISI}})_{36/7} = 0.56$] had the highest difference in prediction. The reduction in stiffness predicted by the FEA models when attachment pattern was changed from 36/7 to 36/5 and 36/4 was on average 59% and 76% respectively. In comparison, the experimental predicted reductions when attachment pattern was changed from 36/7 to 36/5 and 36/4 was on average 52% and 66% respectively.
3.12. FEA parametric study conclusions

To investigate the influence of industry standard support fastener attachment patterns on the AISI S310 -20 (AISI 2020) and DDM04 panel buckling limit state, a FEA modelling methodology was developed to predict capacity for nine unique configurations. The FEA modelling methodology was validated against experimental results and utilized non-linear idealized material properties, idealized connection behavior, and contact definitions. The calibrated models were used to predict onset of buckling, ultimate capacities, and stiffness for 18-gage, 20-gage, 22-gage Type B deck. These simulations were identical in configuration except for the support attachment pattern and base metal thickness. Based on the observations from these numerical simulations, the following key conclusions were drawn:

- Developed FEA models can capture experimental strength with high accuracy (Experimental/FEA = 95%) and predict buckling (local and global) and end warping behavior.
• Stiffness predicted by the FEA models is higher than that observed during experimental testing [mean(Experimental/FEA = 78%)]. This difference is largest for the 36/5 intermediate attached specimen (observed/predicted = 70%)
• Improved stiffness prediction is possible with more involved fastener models [mean(Experimental/FEA = 89%)] and measured material properties, but such models can cause convergence issues and add significant runtime
• Reduction in stiffness when comparing the fully attached specimen with intermediate and skip patterns show good agreement with reductions observed during experimental testing (Experimental/FEA = 90% and 103% for 36/7 versus 36/5 and 36/4 respectively)
• FEA models show similar trends in peak capacity as observed in experimental testing and a significant reserve was observed after initiation of out-of-plane buckling
• A reduction (6% and 17% average) in peak capacity was observed in the FEA models when comparing the fully attached 36/7 specimens with the 36/5 and 36/4 specimens
• In the FEA simulations and experimental tests, onset of buckling was observed to not coincide with a notable change in the overall force-displacement response. While predicting onset of buckling visually through sensor/out-of-plane displacement buckling data is possible, a homogenous method is required to uniformly predict initiation and required for assessing the accuracy of the panel buckling equation
4. CHAPTER 4:
DISCUSSION OF RESEARCH QUESTIONS

4.1. How does individual connection strength, $P_{nf}$, vary due to reducing deck and support framing member thickness in screw fastened connections? What is the hysteretic behavior of these light-gage connections?

To determine the influence of reducing deck and support thickness on the strength and hysteretic performance of light gage steel screw fastened connections, 27 cyclic connections tests were conducted. These tests varied in steel gage and ultimate strength. 22, 24, and 26 gage steel Type B deck flutes were tested with #12 and #10 fasteners in the structural (frame) and sidelap configuration respectively. The tests were used to determine key characteristics of connection response such as hysteresis data, load carrying capacity, and strain energy capacity.

The test results show that light-gage steel connection detailing can have significant impact on the strength and hysteretic behavior of these connections. Sidelaps were shown to be highly sensitive to Ply thickness and both strength and strain energy reduced when ply thickness was reduced. The common failure mode was tilting of the screw leading to eventual failure of the connection. Plies typically used for steel deck (16 – 26 gage) fall in the thickness range where tilting is the dominant failure mode for screw fastened sidelap connections. Theoretical design equations were observed to have good predictability for the strength and failure mode of these connections. The design equations consider the relationship between ply thickness, ply ultimate strength, and fastener diameter when determining the resistance to bearing and tilting. Hence by increasing fastener diameter, lighter gage sidelap connections can theoretically have increased strength. The influence of increasing fastener diameter on the energy dissipation properties is also
vital information needed to understand and predict the seismic response of these connections. An expansion to the sidelp test program presented in this dissertation has been proposed in the future works section.

Failure modes for the support connection tests were well predicted by the design equations and depended on relative thickness of the plies. Capacity predictions for the 14 gage supports showed excellent agreement (mean test to predicted ratio = 1.08, standard deviation = 0.07) but the 18 gage connections were poorly predicted (mean test to predicted ratio = 0.58, standard deviation = 0.08). The 18 gage support tests fall into the interpolation range of the design equations and review of individual limit states capacities revealed that all capacities were overpredicted. The 18 gage ply also had a high ultimate to yield strength ratio (2.16 in comparison 1.51 of the 14 gage) as well as an elongation of greater than 30%. This could also cause an underprediction as the coupon tested ultimate strength of the material could not be developed during testing. While the predictive equation did not perform well in predicting capacity for the 18 gage tests, it was able to predict failure mode.

Screw fastened connections that have undergone damage due to tilting and bearing show clear evidence of so in the form of deformation around the fastener holes, tilting of the fasteners, ply separation, and ply piling. As these connections are typically visible in bare deck diaphragm, construction such as in end and sidewall panels of metal buildings, visual inspection of these connections after an event can lend helpful insights into the response and condition of the structure. This topic has been explored further in the recommended future works.
4.2. Does end connectivity (support attachment) influence the out-of-plane buckling capacity of corrugated steel deck diaphragms?

To determine whether end connectivity (support attachment patterns) influences the out-of-plane buckling capacity of light-gage steel diaphragms, 9 full-scale, monotonic tests were conducted. These tests varied in support attachment pattern, while maintaining other key parameters such as material properties (E), span length, and corrugation geometry (Type B Deck – 22 gage). The test results were utilized to calibrate FEA models and predict the stability behavior of 6 untested configurations that were identical to the experimental tests but differed in deck thickness. Results from the experimental and numerical investigations conclusively indicate that accounting for support attachment pattern is not necessary when predicting buckling capacity, and that significant overstrength exists in predictive equations utilized by available design standards such as AISI S310 (AISI 2016a), SDI DDM04 (Luttrell 2015), and SDCFSFDM (Sputo 2017). However, achieving the full panel buckling capacity requires managing the connection limit states which typically govern design. If connections are not adequately overdesigned, failure of the connections initiates and prohibits the panels from achieving their maximum buckling capacity possible for the connection type being used. This was especially observed when the 36/5 intermediate attachment patterns were being tested and test repetition 2 failed prematurely due to improper sidelap construction of the proprietary VSC-II PunchLok connection.

As the industry moves towards lighter (thinner) highly optimized steel deck profiles, fully addressing the panel buckling limit state becomes increasingly important. The typical overstrength in buckling vs connections failure associated with diaphragm design diminishes due to the reduced cross section properties. The thinner steel decks can have comparable connection strengths with their thicker counterparts due to the increased ultimate strength but suffer significant reductions in
the panel buckling capacity due to reduction in the moment of inertia per unit width and thickness. Further, reducing support attachment pattern also influences stiffness and the displacement level at which ultimate capacity is achieved. This is of particular importance when addressing serviceability and ultimate limit states as although the diaphragms have similar capacity, they deform differently, with the skip pattern specimens almost having a third of the stiffness of their fully attached counterparts. Further, connections are an important source of energy dissipation in light gage steel diaphragms, and typical diaphragm construction features a large number of support and sidelap connections. This can be utilized to further improve the energy dissipation capacities of metal deck diaphragms.
5. CHAPTER 5:

RECOMMENDED FUTURE WORKS

5.1. Characterizing the hysteretic behavior of light gage steel-to-steel sidelap and frame connections

To further understand, characterize, and compare the behavior of light gage steel screw fastened connections, the following future works are recommended –

- Characterization of the energy dissipated during testing for each of the connection tests presented in the dissertation. Energy dissipation can be estimated by integrating the area enclosed during each loop in the force-displacement response and summing them together.

- Estimation of four-point backbone curves based on the multi-point backbone curves to estimate and compare key characteristics such as ductility, minimum total elongation, and residual force level. As there was variability associated with the repetition in the testing, the four-point backbone also provides a homogenous method to compare repetitions and connection configurations.

- There were large differences noted in the measured connection capacities ($P_{exp}$) and the expected connection capacities predicted with measured ($P_{nv,m}$) and nominal material properties ($P_{nv,n}$) for the 18 gage support tests. Base metal properties in the 18 gage plies were observed to have a very high ultimate to yield strength ratio (2.2) and elongation (>30%). It is hypothesized that due to the high ratio and high elongation, the measured ultimate strength could not be developed in the support ply, leading to lower tested capacities. It is also possible that due to relative thickness of plies falling into the interpolation range, the current design equation might require modifications to account for
the low thickness of the plies as these plies fall towards the lower limit of the applicability of the code. Conclusively determining the cause of this underprediction is important for understanding the accuracy of the connection strength equations and a sample testing matrix covering a range of relative ply thicknesses from 1.12 to 3.01 have been proposed here –

Table 5-1: structural connection test matrix

<table>
<thead>
<tr>
<th>No</th>
<th>Ply 1 Gage (ga)</th>
<th>Ply 1 t₁ (in)</th>
<th>Ply 1 Grade (ksi)</th>
<th>Ply 2 Gage (ga)</th>
<th>Ply 2 t₂ (in)</th>
<th>Ply 2 Grade (ksi)</th>
<th>Fastener</th>
<th>T2/T₁</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>22</td>
<td>0.0295</td>
<td>45</td>
<td>16</td>
<td>0.0538</td>
<td>65</td>
<td>14 and 12</td>
<td>1.82</td>
</tr>
<tr>
<td>2</td>
<td>24</td>
<td>0.0239</td>
<td>45</td>
<td>16</td>
<td>0.0538</td>
<td>65</td>
<td>14 and 12</td>
<td>2.25</td>
</tr>
<tr>
<td>3</td>
<td>26</td>
<td>0.0179</td>
<td>45</td>
<td>16</td>
<td>0.0538</td>
<td>65</td>
<td>14 and 12</td>
<td>3.01</td>
</tr>
<tr>
<td>4</td>
<td>22</td>
<td>0.0295</td>
<td>80</td>
<td>20</td>
<td>0.0329</td>
<td>65</td>
<td>14 and 12</td>
<td>1.12</td>
</tr>
<tr>
<td>5</td>
<td>24</td>
<td>0.0239</td>
<td>80</td>
<td>20</td>
<td>0.0329</td>
<td>65</td>
<td>14 and 12</td>
<td>1.38</td>
</tr>
<tr>
<td>6</td>
<td>26</td>
<td>0.0179</td>
<td>80</td>
<td>20</td>
<td>0.0329</td>
<td>65</td>
<td>14 and 12</td>
<td>1.84</td>
</tr>
</tbody>
</table>

• Further testing of light-gage steel sidelap connections focusing on the 22, 24, 26 gage deck range with different sized fasteners (ex - #8, #10, #12, #14) to expand the state of knowledge on the performance of these connections. A sample testing matrix has been proposed here –

Table 5-2: Nestable sidelap test matrix

<table>
<thead>
<tr>
<th>No</th>
<th>Ply gage (ga)</th>
<th>Grade (ksi)</th>
<th>Fastener (No.)</th>
<th>No</th>
<th>Ply gage (ga)</th>
<th>Grade (ksi)</th>
<th>Fastener (No.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>22</td>
<td>45</td>
<td>14</td>
<td>10</td>
<td>22</td>
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<td>26</td>
<td>45</td>
<td>14</td>
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<td>26</td>
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<td>12</td>
</tr>
<tr>
<td>4</td>
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<td>14</td>
<td>13</td>
<td>22</td>
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<td>5</td>
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<tr>
<td>6</td>
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<td>80</td>
<td>14</td>
<td>15</td>
<td>26</td>
<td>80</td>
<td>12</td>
</tr>
<tr>
<td>7</td>
<td>22</td>
<td>80</td>
<td>14</td>
<td>16</td>
<td>22</td>
<td>80</td>
<td>12</td>
</tr>
<tr>
<td>8</td>
<td>24</td>
<td>80</td>
<td>14</td>
<td>17</td>
<td>24</td>
<td>80</td>
<td>12</td>
</tr>
<tr>
<td>9</td>
<td>26</td>
<td>80</td>
<td>14</td>
<td>18</td>
<td>26</td>
<td>80</td>
<td>12</td>
</tr>
</tbody>
</table>
• Adoption of estimated backbone curve parameters in FEA simulation to simulate non-linear connection behavior. Multi-point and four-point backbone curves generated from experimental tests results can be used to idealize the cyclic behavior of connections in monotonic FEA simulations. While the simplified connection non-linearity modelling methodology can provide estimates for peak load, it does not account for the pinching effects noted during testing that are characteristic of light gage steel connections. Including connection pinching effects for cyclic simulations in finite element programs is possible with ABAQUS user elements (UEL) (Ding 2015). These UELs rely on Pinching 4 parameters which makes it possible to include degradation and pinching behavior in FEA simulations.

• As structural (frame) and sidelap connections are often exposed in metal building end walls and side walls, they are easily accessible for inspections. Visual documentation of the progression of failure in the connections as they undergo testing and correlating the observed damage state with normalized energy dissipation or displacement can provide valuable insights for qualitative visual inspection of these connections in actual structures after a loading event.

5.2. Determining the influence of support attachment pattern of the out-of-plane buckling capacity of light-gage steel deck diaphragms

To further understand the influence of end connectivity on the stability behavior of light gauge steel panels, the following future works are recommended –

• Development of robust FEA models capable of accurately predicting stiffness in addition to buckling capacity and behavior. Stiffness prediction in the numerical models greatly depend on material properties \( (E, F_y, F_u, \varepsilon_y, \varepsilon_u) \) as well as fastener behavior. ASTM 8
compliant coupon tests are recommended for accurately capturing material behavior. Further, support and sidelap fastener tests are recommended for determining multi-linear connection behavior inputs for the FEA models. Some sample connection test configurations that would expand the available connection test data and aid with the modelling and analysis have been presented in Section 5.1.

- Developed models should be used to predict behavior for a variety of commercially available deck cross section types (For example Type N deck, Type F, Dovetail deck, shallow deck, etc.) and at a range of spans \( L_v \). Additionally, the newly added limit state of local buckling should also be investigated by varying panel end support lengths. End warping behavior should also be included as an objective of the study as stiffness greatly depends on it and FEA models can provide detailed warping behavior.

- A homogenous method of identifying onset of buckling from out-of-plane results should be developed to accurately assess the predictability of the AISI S310 equation. While visual inspection of the out-of-plane displacement serves as a good indicator for buckling, a homogenous method will eliminate any bias or error. Further, an automated method makes evaluating and processing large amounts of FEA data feasible.

- Predictions of the FEA models should be spot checked with full-scale experiments to verify the accuracy of the models and recalibrate if necessary.
APPENDICES

A. Appendix A – Detailed per-specimen experimental results of connection tests
Figure A-1: 01-22-22-R1
Figure A-2: 02-22-22-R2
Figure A-3: 03-22-22-R3
Figure A-4: 04-24-24-R1
Figure A-5: 05-24-24-R2
Figure A-6: 06-24-24-R3
Figure A-7: 07-26-26-R1
Figure A-8: 08-26-26-R2
Figure A-9: 09-26-26-R3
Figure A-10: 10-22-18-R1
Figure A-11: 11-22-18-R2
Figure A-12: 12-22-18-R3
Figure A-13: 13-24-18-R1
Figure A-14: 14-24-18-R2
Figure A-15: 15-24-18-R3
Figure A-16: 16-26-18-R1
Figure A-17: 17-26-18-R2
Figure A-18: 18-26-18-R3
Figure A-19: 19-22-14-R1
Figure A-20: 20-22-14-R2
Figure A-21: 21-22-14-R3
Figure A-22: 22-24-14-R1
Figure A-23: 23-24-14-R2
Figure A-24: 24-24-14-R3
Figure A-25: 25-26-14-R1
Figure A-27: 27-26-14-R3
B. Appendix B – Test observation notes
# UMass Connections testing program

<table>
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<th>Test Details:</th>
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<td>Test ID:</td>
<td>08-22-02-10-RR</td>
</tr>
<tr>
<td>Date:</td>
<td>07/05/23</td>
</tr>
<tr>
<td>Time:</td>
<td>2:50 PM</td>
</tr>
</tbody>
</table>

### Specimen Details:
- Deck thickness 1, $t_1$: 0.29 in
- Deck thickness 2, $t_2$: 0.29 in
- Fastener: #10 1/2, 5/8 [No, threads/in, length (in)]

### Loading protocol details:
- Protocol: FEMA
- Load Rate: 0.005 in/sec
- Max Displacement, $\Delta_m$: 0.05 in

### Test Notes:
- Actuator initial force: -210 lbs
- Actuator initial displacement: -0.001 in

* Pre-tension: \( F_0 = -392 \text{ lbs} \) & 0.01 mm
* Fastener head last contact 0.071" cycle
* Tilt observed in 0.10" cycle
* Fastener head visibly off the nut in 0.15" cycle
* Chatter noise heard in 0.195" cycle, severe tilting observed
* Thread sheared through 0.23" in 0.29" cycle
* Pull up through 0.25" in 0.15" cycle
* Fastener almost out in 0.5" cycle
* F2 (635 peak in 0.7") cycle (but a note on ovalization)

### End of test Notes:
- Max force: lbs
- Min Force: lbs
- Failure Mode: Exceeding tilt, resulting in fastener pull-out of...
# UMass Connections testing program

### Test Details:
- Test ID: 03 - 22 - 22 - 10 - K3
- Date: 03/05/23
- Time: 4:30 PM

### Specimen Details:
- Deck thickness 1, $t_1$: 0.38 in
- Deck thickness 2, $t_2$: 0.38 in
- Fastener: 1/10, 1/8, 5/32 (No, threads/in, length in)

### Loading protocol details:
- Protocol: FEMA
- Load Rate: 0.005 in/sec
- Max Displacement, $\Delta_m$: 0.75 in

### Test Notes:
- Actuator initial force: 1854 lbs
- Actuator initial displacement: 0.005 in

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### End of test Notes:
- Max force: lbs
- Min Force: lbs
- Failure Mode: 0.4 of fast with loss in $t_2$
<table>
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<tr>
<th>Test Details:</th>
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<td>07/06/23</td>
</tr>
<tr>
<td>Time:</td>
<td>1:50pm</td>
</tr>
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</table>

**Specimen Details:**
- Deck thickness 1, \( t_1 \): 28ga in
- Deck thickness 2, \( t_2 \): 20ga in
- Fastener: \( \pm 10 / 15 / 3/4 \) [No, threads/in, length (in)]

**Loading protocol details:**
- Protocol: FEMA
- Load Rate: 0.005 / 0.01 in/sec
- Max Displacement, \( \Delta_m \): 0.75 in

**Test Notes:**
- Actuator initial force: 2046 lbs
- Actuator initial displacement: 0.005 in

- 0.05" cycle fastener heads no longer flush
- 0.07" cycle fully above
- Click drives head down new drop to 0.1" drop
- 0.13" fastener clearance eigen of exabden

- Unload variable @ 0.13" trailing cycle, start fully through too
- Easily raises load drop about 0.196" cycle SPRING TRAILING CYCLE
- Significantly over

- Ply separation show near Fl at 0.25" cycle ply 1 was damage underثر
- Local buckly under Fasten head of Flail @ 0.0" spring 0.75" cgy

**End of test Notes:**
- Max force: lbs
- Min Force: lbs
- Failure Mode: Lifty & beamy clearly on class 1 fastener.
# UMass Connections testing program

<table>
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<tr>
<th>Test Details:</th>
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<td>Time:</td>
<td>3:00 PM</td>
</tr>
</tbody>
</table>

## Specimen Details:

| Deck thickness 1, \( t_1 \): 24 ga | in |
| Deck thickness 2, \( t_2 \): 24 ga | in |
| Fastener: | [No, threads/in, length (in)] |

## Loading protocol details:

| Protocol: | FEMA |
| Load Rate: | \( 0.05 \) / \( 0.01 \) in/sec |
| Max Displacement, \( \Delta_m \): | 0.25 in |

## Test Notes:

| Actuator initial force: | 1854 lbs |
| Actuator initial displacement: | 0.058 in |
| 0.05” cycle fastener breaks no longer flush, tiltly initiated |
| 0.07” clear signs of tilting, 0.13” large tilting |
| Load drop & large heave \( 0.19” \) going to \( 0.14” \) |
| Threads pull through \( 0.14” \) no trinity no pin \( 0.04” \) |
| Escare tilting \( 0.21” \) load drop \( 0.35” \) followed by \( 0.38” \) |
| Large load breaks no in \( 0.37” \) ply separation & slip |
| Threads cont. but pull through \( 0.35” \) end |
| Large load drop & in \( 0.02” \) going & \( 0.15” \) protruding |
| Top popped out \( 0.35” \) cycle |

## End of test Notes:

| Max force: | lbs |
| Min Force: | lbs |
| Failure Mode: | Lifting & beams clearly due to \( t \) & fastener |

152
UMass Connections testing program

<table>
<thead>
<tr>
<th>Test Details:</th>
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</table>

**Specimen Details:**
- Deck thickness 1, $t_1$: 0.484 in
- Deck thickness 2, $t_2$: 2.486 in
- Fastener: #10-16, 3/4" [No, threads/in, length (in)]

**Loading protocol details:**
- Protocol:  
- Load Rate: in/sec  
- Max Displacement, $\Delta_m$: in

**Test Notes:**
- Actuator initial force: -960.00 lbs  
- Actuator initial displacement: -0.099 in  
- Tilt documented @ 0.099"  
- Fastener tilted observed @ 0.1"  
- Threads winkle @ 0.29" ply separation in -0.29" division  
- Cycled in @ 0.30" cycle till dynamic pops through  
- Fastener popped (+2) in 0.05" cycle

**End of test Notes:**
- Max force: lbs  
- Min Force: lbs  
- Failure Mode: 

"Loosy & Breaking loads up to (off screw)"
# UMass Connections testing program

## Test Details:
- **Test ID:** 07-26-28-10-R1
- **Date:** 07/09/23
- **Time:** 10:10am

## Specimen Details:
- **Deck thickness 1, \( t_1 \):** 26 ga
- **Deck thickness 2, \( t_2 \):** 26 ga
- **Fastener:** ±10, 16, 314 [No, threads/in, length (in)]

## Loading protocol details:
- **Protocol:** FEMA
- **Load Rate:** 0.005/0.01 in/sec
- **Max Displacement, \( \Delta_m \):** 0.75 in

## Test Notes:
- **Actuator initial force:** 196 lbs
- **Actuator initial displacement:** 0.00 in
- Installation issues:
  - Bottom nuts are loose
  - Clicking noise heard @ 0.
  - 0.19" fastener not fully engaged: try 0.01" with 0.1" primary thread in
  - 0.01" fastener not fully engaged (third shear)
  - Large washer bit off at 0.24", instead pull through washer head @ 0.6".
  - Large washer hit 0.24" with 0.01" new drip sound from 0.
  - Ply drop also over @ 0.31" cycle: 0.24" cycle.
  - 0.01" pop into place @ 0.75" cycle and fastens itself.

## End of test Notes:
- **Max force:** lbs
- **Min Force:** lbs
- **Failure Mode:**
# UMass Connections testing program

<table>
<thead>
<tr>
<th>Test Details:</th>
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<td>08-26-26-10-K2</td>
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<tr>
<td>Date:</td>
<td>07/15/23</td>
</tr>
<tr>
<td>Time:</td>
<td>11:40am</td>
</tr>
</tbody>
</table>

**Specimen Details:**
- Deck thickness 1, t₁: 26 ga in
- Deck thickness 2, t₂: 26 ga in
- Fastener: ± 10, 16, 3/4 [No, threads/in, length (in)]

**Loading protocol details:**
- Protocol: FEMA
- Load Rate: 0.005 / 0.01 in/sec
- Max Displacement, Δₘ: 0.25 in

**Test Notes:**
- Actuator initial force: - 4000 lbs
- Actuator initial displacement: 0.005 in
- Clicking noises heard @ 0.026”
- Possible heads no longer pull in 0.05” cycle
- Clicking noises heard @ 0.026” cycle
- No positive heads heard in 0.24” cycle
- Negative 0.1” pinching cycle
- Load drop observed @ 0.056”
- 0.25” cycle load drop damage observed
- Load dropped poppy noises heard in 0.24” cycle
- Load separation observed @ 0.3” in 0.53” cycle
- Load drop observed @ 0.3” in 0.53” cycle
- Load drop observed @ 0.5” in 0.53” cycle
- Pulling out of hole
- Severe load drop in -0.5” in 0.5 cycle
- Test terminated due to almost zero capacity in 0.5 cycle

**End of test Notes:**

<table>
<thead>
<tr>
<th>Max force:</th>
<th>lbs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Min Force:</td>
<td>lbs</td>
</tr>
</tbody>
</table>

Failure Mode: [signature]
UMass Connections testing program

Test Details: 09-26-10-R3
Test ID: 09-26-10-R3
Date: 02/10/21
Time: 2:30pm

Specimen Details:
Deck thickness 1, t1: 2.69 in
Deck thickness 2, t2: 2.58 in
Fastener: #10, 1/6 3/4 [No, threads/in, length (in)]

Loading protocol details:
Protocol: FEMA
Load Rate: 0.005/s 0.01 in/sec
Max Displacement, Δm: 0.75 in

Test Notes:
Actuator initial force: 5000 lbs
Actuator initial displacement: 0.005 in

Clicking noises heard 0.051" tilky observ in 0.1" cycle
Popping noises & clean egn at 0.139" cycle
Odd noise load drop observ 0.168" go to 0.19" cycle
Seeds pulled out 0.079" in 0.195" " cycle
Load drops 0.11" in 0.195" trailing cycle
Excess void dew in 0.213" cycle - steel hole damage dew
On 0.383" cycle - ply separation also observ
0.5" load drop pappy end load drops observ
Large load drop observ 20.74" 0.1" disp, 71 (free seven required)

End of test Notes:
Max force: lbs
Min Force: lbs
Failure Mode:
### UMass Connections testing program

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<thead>
<tr>
<th>Test Details:</th>
<th>10-22-18-12-R1</th>
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<tr>
<td>Test ID:</td>
<td>10-72-18-12-R1</td>
</tr>
<tr>
<td>Date:</td>
<td>07/10/23</td>
</tr>
<tr>
<td>Time:</td>
<td>1:55 p.m.</td>
</tr>
</tbody>
</table>

#### Specimen Details:
- Deck thickness 1, $t_1$: 2.59 in
- Deck thickness 2, $t_2$: 1.89 in
- Fastener: 1/2-14-3/4
  - [No, threads/in, length (in)]

#### Loading protocol details:
- Protocol: FEMA
- Load Rate: 0.005 to 0.01 in/sec
- Max Displacement, $\Delta_m$: 1.50 in
  - 17 steps in 0.75" in 15.

#### Test Notes:
- Actuator initial force: -370 lb
- Actuator initial displacement: 0.50 in
  - X-bar: Head is longer than 0.051" cycle
  - Power actuation DBA decreased to 0.07"
  - Very obvious 0.04" in 0.1" cycle
  - Threads visible 0.127" above 0.139" trailing
  - Beating 0.15" in 0.196" cycles in first 5:
    - Many noisy heads (0.03) in 0.233" fastening cycle
    - Threads continue to pull through by step at 0.05 in 0.263"
    - E1 (Beating) completely lost at 0.536" cycle, shut-out at 0.29 in
    - 6.9" cycle
    - Two 20.246" in 0.07" (tried cycle)

#### End of test Notes:
- Max force: lbs
- Min Force: lbs
- Failure Mode: Beating & losing cycle unevenly in con. 1 fastener
# UMass Connections testing program

<table>
<thead>
<tr>
<th>Test Details:</th>
<th>11-22-18-12-R2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test ID:</td>
<td>11-22-18-17-R2</td>
</tr>
<tr>
<td>Date:</td>
<td>07/11/25</td>
</tr>
<tr>
<td>Time:</td>
<td>11:45 am</td>
</tr>
</tbody>
</table>

## Specimen Details:

- Deck thickness 1, $t_1$: 22 ga in
- Deck thickness 2, $t_2$: 18 ga in
- Fastener: 7/12/19, 3/4 [No, threads/in, length (in)]

## Loading protocol details:

- Protocol: FEMA
- Load Rate: 0.05/0.01 in/sec
- Max Displacement, $\Delta_m$: 0.15 in, 1750/2 in

## Test Notes:

- Actuator initial force: -760 lb in lbs
- Actuator initial displacement: 0.03 in in
- 0.02” cycle: 47.71 mm in 0.03” cycle
- Citip rim 0.03” in 0.09” cycle
- Fractured hole damage shear head in 0.12” cycle
- A443A25 head no good (0.03” in 0.19” cycle)
- Bearing in F1 Observed 0.03” in 0.19” cycle
- A324A25 at 0.02” in 0.27” in 0.38” cycle
- A443A25 at 0.02” in 0.38” cycle
- F1 load at 0.03” in 0.53” cycle
- F2 load at 0.39” return from 0.77” cycle

## End of test Notes:

- Max force: lbs
- Min Force: lbs

**Failure Mode:** Bearing in F1/2, 4A25 in F1/2, F1 load upon F2, test can end now.
# UMass Connections testing program

## Test Details:
- **Test ID**: 12-22-18-12-R3
- **Date**: 04/11/23
- **Time**: 1:05 pm

## Specimen Details:
- **Deck thickness 1, \( t_1 \)**: 2.29 in
- **Deck thickness 2, \( t_2 \)**: 1.88 in
- **Fastener**: ±12, 14, 3/4. [No. threads/in, length (in)]

## Loading protocol details:
- **Protocol**: TMA
- **Load Rate**: 0.008/0.01 in/sec
- **Max Displacement, \( \Delta_m \)**: 1.5" in 17 steps

## Test Notes:
- **Actuator initial force**: -275 lbs
- **Actuator initial displacement**: 6.50" in
- 

## End of test Notes:
- **Max force**: lbs
- **Min Force**: lbs
- **Failure Mode**: Bearing 2 tension
# UMass Connections testing program

## Test Details:
- **Test ID:** 13-24-18-12-R1
- **Date:** 07/11/23
- **Time:** 2:50 PM

## Specimen Details:
- **Deck thickness 1, t₁:** 2.48 in
- **Deck thickness 2, t₂:** 1.85 in
- **Fastener:** 11/4, 3/4, [No, threads/in, length (in)]

## Loading protocol details:
- **Protocol:** FEMA
- **Load Rate:** 0.05 0.01 in/sec
- **Max Displacement, Δₘ:** 1.50 0.17 steps in

## Test Notes:
- Actuator initial force: 60 lbs
- Actuator initial displacement: 0.03 in
- Warping & ply caused fastener bolts @ 0.02 in
- LIly observed approx 0.03" cycle
- Ply needs winkle @ no layer pullin 0.1" cycle
- Ply sep. observed near F1 in 0.33" cycle
- 0.06" torol cycle to 0.10" unthread pull in notes head
- Load poppin & wind drop @ 0.08"
- Substrate 20.2" dead drop @ 0.083" large coat cap 0.15" when done from 0.387" to 0.536"

## End of test Notes:
- **Max Force:** lbs
- **Min Force:** lbs
- **Failure Mode:** Winging in R1, Bearing in R2
# UMass Connections testing program

## Test Details:
- **Test ID:** 14_24_R_12_R2
- **Date:** 09/11/23
- **Time:** 4:10 PM

## Specimen Details:
- **Deck thickness 1**, $t_1$: 0.05 in
- **Deck thickness 2**, $t_2$: 0.05 in
- **Fastener:** #12 1/4, 3/4
  - [No, threads/in, length (in)]

## Loading protocol details:
- **Protocol:** FEMA
- **Load Rate:** $0.03/0.01$ in/sec
- **Max Displacement, $\Delta_m$:** 1.5" in 17 steps

## Test Notes:
- **Actuator initial force:** $-100$ lb
- **Actuator initial displacement:** $-0.05$" in
- Cycle 1 to 0.01 cycle (0.02" in 0.5")
- Cycle 2 to 0.071 in 0.51"
- Cycle 3 to 0.01 cycle (0.02" in 0.51")
- Cycle 4 to 0.071 in 0.51"
- Cycle 5 to 0.01 cycle (0.02" in 0.51")
- Cycle 6 to 0.071 in 0.51"
- Cycle 7 to 0.01 cycle (0.02" in 0.51")
- Cycle 8 to 0.071 in 0.51"
- Cycle 9 to 0.01 cycle (0.02" in 0.51")
- Cycle 10 to 0.071 in 0.51"
- Cycle 11 to 0.01 cycle (0.02" in 0.51")
- Cycle 12 to 0.071 in 0.51"
- Cycle 13 to 0.01 cycle (0.02" in 0.51")
- Cycle 14 to 0.071 in 0.51"
- Cycle 15 to 0.01 cycle (0.02" in 0.51")
- Cycle 16 to 0.071 in 0.51"
- Cycle 17 to 0.01 cycle (0.02" in 0.51")

## End of test Notes:
- **Max force:** lbs
- **Min Force:** lbs
- **Failure Mode:** Eddy in P1, Beany in P2
# UMass Connections testing program

## Test Details:
- **Test ID:** 15_24_18_12_R3
- **Date:** 04/12/23
- **Time:** 9 a.m.

## Specimen Details:
- **Deck thickness 1, \( t_1 \):** 0.9 in
- **Deck thickness 2, \( t_2 \):** 1.8 in
- **Fastener:** # 12/14/24 [No, threads/in, length (in)]

## Loading protocol details:
- **Protocol:** FEMA
- **Load Rate:** 0.005/0.01 in/sec
- **Max Displacement, \( \Delta_m \):** 1.5 in

## Test Notes:
- **Actuator initial force:** 2724 lbs
- **Actuator initial displacement:** 0.05 in

- Initial deflection away 0.015 in due to 0.036 in cleat head.
- Fastener 90°, nominating cycle, no longer flush with head.
- Tracing cycle command 6.051 in pop-in head 0.08 in.
- Ply strip 0.14 in. from outer thread pull-through.
- Thread pull-through 0.065 in - 0.195 in cycle.
- F1 bearing on deck observed @ 0.195 in cycles.
- F1 showing sign of bearing in 0.27 cycle - cycle overload.
- @ 0.3 in cycles - F12 bearing in ply 1 causing 0.28 in cycles in #8. Full over in Ply 1 large hole next 1/16 in.

## End of test Notes:
- F1 pulled through Ply 1. F2 load @ 0.5 in 1.0 cycle.

## Final Results:
- **Max Force:** lbs
- **Min Force:** lbs
- **Failure Mode:** Pull-out in Ply 1, high bearing in Ply 2.
### UMass Connections testing program

**Test Details:**
- **Test ID:** 16 - 26 - 18 - 12 - R1
- **Date:** 07/12/23
- **Time:** 9: 55 a.m.

**Specimen Details:**
- Deck thickness 1, t₁: 2.68 in
- Deck thickness 2, t₂: 1.89 in
- Fastener: # 12, 3A, 3/4 [No, threads/in, length (in)]

**Loading protocol details:**
- **Protocol:**
- **Load Rate:** 0.005/0.01 in/sec
- **Max Displacement, Δ m:** 1.50 in Δ steps in

**Test Notes:**
- Actuator initial force: lbs
- Actuator initial displacement: in

> 6.019" ply sup'd & wrapping around 1/4" obvo - 0.026" F111 no change or
> 6.036" ply sup'd & wrapping around F2 - extended pull through nose head
> 0.01" in 0.051 (7) cycle - tiltig over in 0.34" cycle
> 0.74" (P) cycle - tilting & threads.wrinkle - 0.195" (P) cycle - 1.012" F111
> Beating into ply 1 - 1 cycle primarily cause lean cycle, 0.195-
> 1 cycle - fasten hole beating obvi in 0.195" (7) cycle - BAG & COAD
> drop(0.026" P) - load incr. seen (0.023" P) due to F111 beating -
> Coating into pice - poppy & COAD drop (0.02" in 0.523" P)
> 0.385 (P) cycle higher than pice cycle - COAD drop 0.5" in 0.54" (P) my cho.

**End of test Notes:**
- **Max force:** lbs
- **Min Force:** lbs
- **Failure Mode:**

> F2 lost (0.18" in 0.54" (7) cycle, popped again @ 0.18" (P) (P)
> 0.5" (F1 Pulling out) (P)
> 0.5" (F1 completely gone 208" in/0.15" (P)
# UMass Connections testing program

## Test Details:
- **Test ID:** 17-26-18-12-R2
- **Date:** 07/12/23
- **Time:** 11:30 am

## Specimen Details:
- **Deck thickness 1, \( t_1 \):** 2.68 in
- **Deck thickness 2, \( t_2 \):** 1.88 in
- **Fastener:** \( \#12, 14, 3/4 \) in [No, threads/in, length (in)]

## Loading protocol details:
- **Protocol:** Test A
- **Load Rate:** 0.05 in/sec
- **Max Displacement, \( \Delta_m \):** 1.5 in 17 steps in

## Test Notes:
- **Actuator initial force:** lbs
- **Actuator initial displacement:** in

   - Ply up & working above in 0.026" cycle, FH nose flush in 0.05" cycle around & deck up 3/4" stroke, cycle (0.03")
   - Ply is also observed in 0.1" cycle - fasten head down against 0.14" from top - fasten hole bearing observed in F1 with 0.15" (P)
   - Fasten hole bearing also observed in F2 - 0.14" (P) - large thread pitch
   - Noise heard Q - 0.37" in - 0.37" cycle, fast has been pulled through ply 1 - threads pull along nose 0.15" in - 0.37" (P)

   - F1 lost return @ 0.14" in 0.15" (P) - instead came to max capacity and stopped.

## End of test Notes:
- **Max force:** lbs
- **Min Force:** lbs
- **Failure Mode:**
# UMass Connections testing program

## Test Details:
- **Test ID:** 16-26-18-12-R3
- **Date:** 07/12/23
- **Time:** 2:45 PM

## Specimen Details:
- **Deck thickness 1, \( t_1 \):** 0.688 in
- **Deck thickness 2, \( t_2 \):** 0.896 in
- **Fastener:** #12, 14, 3/4 in. [No, threads/in, length (in)]

## Loading protocol details:
- **Protocol:** FEMA
- **Load Rate:** 0.05/0.01 in/sec
- **Max Displacement, \( \Delta_m \):** 1.5 in

## Test Notes:
- **Actuator initial force:** 10.2 lbs
- **Actuator initial displacement:**
  - Ply x 10
  - FH 10" long in 0.036" cycle
  - 10 cycles, 10 in

## End of test Notes:
- **Max force:** 10 lbs
- **Min Force:** 0 lbs
- **Failure Mode:**
# UMass Connections testing program

## Test Details:

- **Test ID:** 94_22_14_12_R1
- **Date:** 07/13/03
- **Time:** 1:30pm

## Specimen Details:

- **Deck thickness 1, t₁:** 22.56 in
- **Deck thickness 2, t₂:** 14.89 in
- **Fastener:** #12, 1.9, 3/4" (No, threads/in, length/in)

## Loading protocol details:

- **Protocol:** FemA
- **Load Rate:** 0.05/0.01 in/sec
- **Max Displacement, \( \Delta_m \):** 1.5 in

## Test Notes:

**Actuator initial force:**

- 100 lbs

**Actuator initial displacement:**

- 0.3 in

**Actuator movement:**

- 35 in

**Test Notes:**

- Initial test at 100 lbs, 1.5 in.
- Load at 0.05 in/sec, 1.5 in max.
- Failure observed at 0.4 in.

## End of test Notes:

- **Max Force:** 100 lbs
- **Min Force:** 0 lbs
- **Failure Mode:**

- "Test in 0.05 in (P) cycle, a few 0.56 in (P) cycle, test end"
# UMass Connections testing program

## Test Details:
- **Test ID:** 20-22-14-12-R2
- **Date:** 07/13/23
- **Time:** 4:10 PM

## Specimen Details:
- **Deck thickness 1, t₁:** 22 in
- **Deck thickness 2, t₂:** 14 in
- **Fastener:** # 18, 14, 3/4
  - [No, threads/in, length (in)]

## Loading protocol details:
- **Protocol:** FEMA
- **Load Rate:** 0.005/0.01 in/sec
- **Max Displacement, Δₘ:** 1.50 in, 17.160 in

## Test Notes:
- **Actuator initial force:** lbs
- **Actuator initial displacement:** lbs

---

End of test Notes:
- **Max force:** lbs
- **Min Force:** lbs
- **Failure Mode:** Bearing in plies

---

A 0.75" (P) load carrying cap was lost, FH #1A all chek way but made no contact, 0.013" in 0.75" (P) pop of the #1 F2 completed last in 0.007" (P) in clu
# UMass Connections testing program

## Test Details:

- **Test ID:** 21_22_14_12_R3
- **Date:** 07/14/03
- **Time:** 11:55 am

## Specimen Details:

- **Deck thickness 1, \( t_1 \):** 2.25 in
- **Deck thickness 2, \( t_2 \):** 1.38 in
- **Fastener:** #12, 1/4, 3/4 [No, threads/in, length (in)]

## Loading protocol details:

- **Protocol:** FEMA 461
- **Load Rate:** 0.005 / 0.01 in/sec
- **Max Displacement, \( \Delta_m \):** 1.50 in / 10 sec

## Test Notes:

- **Actuator initial force:** lbs
  - **Actuator initial displacement:** in
  - 0.051 "Ply sep (P) → 0.001 " (P) Fit no longer flush → washing near F1, F2
  - 0.001 " (P) cyclic - F1 pop through observed in - 0.139 " (P) cycle
  - 0.01 " (P) cycle → Fit bearing into ply 1 observed in - 0.195 " (P) cycle → Fit bearing into ply 2 observed in - 0.195 " (P) cycle → Fit bearing into ply 2 observed in - 0.195 " (P) cycle → Fit bearing into ply 2 observed in - 0.195 " (P) cycle

## End of test Notes:

- **Max force:** lbs
- **Min Force:** lbs
- **Failure Mode:** Bearing of Ply followed by lint of swarf
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<th>UMass Connections testing program</th>
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<tr>
<td><strong>Test Details:</strong></td>
</tr>
<tr>
<td>Test ID: 2.2.24.14-12 -R1</td>
</tr>
<tr>
<td>Date: 3/14/23</td>
</tr>
<tr>
<td>Time: 12:30 PM</td>
</tr>
</tbody>
</table>

| Specimen Details:                |
| Deck thickness 1, t₁: 2.489 in   |
| Deck thickness 2, t₂: 1.48 in     |
| Fastener: #12, 1/4, 3/4 in       |

| Loading protocol details:        |
| Protocol: Ferma 481               |
| Load Rate: 0.055/0.01 in/sec      |
| Max Displacement, Δₘ: 1.50" (0 A)yps |

| Test Notes:                      |
| Actuator initial force: - 630.45 lbs |
| Actuator initial displacement: - 0.001" in |

- Warning: A dock near F1 & F2 obs in 0.0039" cycle -
- Deck crack seen in 0.0036" cycle -
- Finally failed at 0.051" CP -
- Warning program increases until inc in ACF
- Bearing in F1 & F2 in bolt hole obs at 0.1" CP cycle -
- Chop in K 0.05" into 0.139" CP cycle -
- F1 PL bearing in F1 cycle prematurely
- M5.2020-191" CP (bolt width damaged) -
- Tailing also obs in 0.05 CP -
- Cap dog, obsv at 0.073" (P) vs 0.193" (P) cycle -
- Wply ref finish -
- Large chop in cap F1 in 0.273" cycle -
- End crack pull through no inc -
- Also obsv at 0.243" CP (SP) -
- F1 pulling through 0.05 CP in 0.55 CP cycle -
- F1 PL bearing int. FLPS -
- F1 pulled over in 0.5 CP cycle -

| End of test Notes:               |
| Max force:                        |
| Min Force:                        |
| Failure Mode: Pull over F2 & F2 Bear in F1 & F2. |

#1 has pulled out of hole into 0.2 CP cycle - Both F1 & F2 not concluded.
# UMass Connections testing program

## Test Details:
- **Test ID:** 24-24-14-12-R2
- **Date:** 7/14/03
- **Time:** 2:40 PM

## Specimen Details:
- **Deck thickness 1, t₁:** 2.499 in
- **Deck thickness 2, t₂:** 1.996 in
- **Fastener:** #12, 1/4, 3/4 [No, threads/in, length (in)]

## Loading protocol details:
- **Protocol:** FEMAG 61
- **Load Rate:** 0.025/0.01 in/sec
- **Max Displacement, Δm:** 0.50 in

## Test Notes:
- **Actuator initial force:** \(-40\) kips
- **Actuator initial displacement:** \(-0.003\) in
- **Testing of Pyrex in 1/8 in thick aluminum:** 0.019" (P), Ply was observed to be 0.026" (P) in 1 cycle → 0.026" (P) in 1 cycle
- **Load dropped 1/2" (P) due to bearing failure:** 0.026" in 0.154" (P) → Fastener
- **Load dropped 1/2" (P) due to bearing failure:** 0.026" in 0.154" (P) → Fastener
- **Load dropped 1/2" (P) due to bearing failure:** 0.026" in 0.154" (P) → Fastener
- **Load dropped 1/2" (P) due to bearing failure:** 0.026" in 0.154" (P) → Fastener
- **Load dropped 1/2" (P) due to bearing failure:** 0.026" in 0.154" (P) → Fastener

## End of test Notes:
- **Max force:** lbs
- **Min Force:** lbs
- **Failure Mode:** Bearing progrinding wear occurs until Pyrex pulls
- "Fastener"
UMass Connections testing program

### Test Details:
- **Test ID:** 25 24 14 12 R3
- **Date:** 04/14/23
- **Time:** 3:45 pm

### Specimen Details:
- Deck thickness 1, $t_1$: 0.25 in
- Deck thickness 2, $t_2$: 0.50 in
- Fastener: #11/14 x 3/4 [No, threads/in, length (in)]

### Loading protocol details:
- **Protocol:** FEM A 461
- **Load Rate:** 0.005/0.01 in/sec
- **Max Displacement, $A_m$:** 0.50 in

### Test Notes:
- **Actuator initial force:** 175 lbs
- **Actuator initial displacement:** 0.005 (2.03 mm)

- Fly up "deck warp" near F1 & F2. Start in 0.026" (P) cycle o clock ends (begin in 0.031" (P) → F2. Note hole heading obv in F1 + F2 (0.01" (P), do not notice). Heading in 0.017" (P) → Lower L's overs in 0.035" (P)

- F19 F2 tilting @ 0.094 in (P) → Initial heading clearly visible in 0.035" (P)

- 0.049" (P) head pre violence in 0.035" cycle → Substantial head break

- Drop blues & T. Obv in 0.035" cycle. F2. Note heading clearly visible in 0.536" cycle. F1 heading note

- Head break @ 0.12" in 0.035" cycle. F1 thread pull through note heard @ 0.12" in 0.385" cycle. F2 headed pull through now heard @ 0.12" in 0.385" cycle. F2 headed pull through now heard @ 0.12" in 0.385" cycle. F2 headed pull through now heard @ 0.12" in 0.385" cycle. F2 headed pull through now heard @ 0.12"

### End of test Notes:
- **Max force:** 150.5 lbs
- **Min Force:** 100 lbs
- **Failure Mode:** Bearing fly off

- 0.536" (P) capable almost a "pull through" load dish @ 0.35 in 0.536" (P). Sensor indicated @ 0.24" in 0.35" (P) → 0.25" (P) → 0.25" (P) → 0.25" (P) → 0.25" (P) → 0.25" (P)
# UMass Connections testing program

## Test Details:
- **Test ID:** 25_26_14_12_R1
- **Date:** 07/15/23
- **Time:** 10:20am

## Specimen Details:
- **Deck thickness 1, t₁:** 2.88 in
- **Deck thickness 2, t₂:** 1.8 in
- **Fastener:** #12, 1-1/2
  - [No, threads/in, length (in)]

## Loading protocol details:
- **Protocol:** FEMA 461
- **Load Rate:** 0.005/0.01 in/sec
- **Max Displacement, Δₘ:** 1.5” in 17.5 sec

## Test Notes:
- **Actuator initial force:** 3410.6 lbs
- **Actuator initial displacement:** in

> Ply sep & deck warpig obs. at 0.015” cycle (P) = warpig in with im. dispo. → F'Hole bonding sign obvs. in 0.037” (T) → lower k UNTW P obvs into 0.1” (T) → bonding & F'hole obvs in 0.35” (P)

> (K degradation was) more in 0.15’ cycle. → peak @ 0.19” lower than 0.134” (P) → FHI turing post obs in 0.195” (N) → FHI bonding into ply 2 obvs in 0.273” (N) → tension in deck warpig with in bonding

> Cap. in. obs in 0.383” (P) due to FHI bonding as pseudo 0.336” (P) FHI warpig towards cap. and pull over indication obs. in F12F2 @ 0.536 (P) 

> Large cap. failure in post obvs in 0.536 (P). → F1 pulled over @ 0.6” in 0.5” (P) F2 followed → test concluded.

## End of test Notes:
- **Max Force:** lbs
- **Min Force:** lbs
- **Failure Mode:** Bonding leading F1 & F2 pulling over.
# UMass Connections testing program

## Test Details:
- **Test ID:** 26-26-14-12-R2
- **Date:** 10/15/23
- **Time:** 11:40 am

## Specimen Details:
- **Deck thickness 1, t₁:** 2.699 in
- **Deck thickness 2, t₂:** 1.499 in
- **Fastener:** #10, 1/4-3/4
  - [No, threads/in, length (in)]

## Loading protocol details:
- **Protocol:** FEMA 461
- **Load Rate:** 0.005/0.01 in/sec
- **Max Displacement, Δm:** 1.50 in at slope

## Test Notes:

### Actuator initial force:
- **lbs**

### Actuator initial displacement:
- **in**
  - Flap up to 2 prior to testing above flyover a dock
  - Working area F2 prior to testing above of dock
  - Working area F2.1 above 0.139" (P)
  - Dock ends (flyover) above 0.656 (P)
  - Reduction in K (Pist) in 1.0" (P)
  - Cap reduction in 0.159 (P)
  - Plate bearing above 0.223 (P)
  - Plate peak cap. continues at girdler above 0.583 (P)
  - Plate peak cap. above 0.583 (P)
  - Plate peak cap. above 0.583 (P)
  - Plate peak cap. above 0.583 (P)
  - Plate peak cap. above 0.583 (P)
  - Plate peak cap. above 0.583 (P)

### End of test Notes:
- **Max force:** lbs
- **Min Force:** lbs
- **Failure Mode:** Bearing in ply 1 leads to pull over & back

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- Thread pull enough warp cond. in 0.05 (P)
- Capacity lost
- Capacity lost in 1.05 (P)
- F2 pulled over @ 1.15 (P)
# UMass Connections testing program

<table>
<thead>
<tr>
<th>Test Details:</th>
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<tbody>
<tr>
<td>Test ID: 27-26-14-12-R3</td>
</tr>
<tr>
<td>Date: 2/15/23</td>
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<tr>
<td>Time: 3:30 pm</td>
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<table>
<thead>
<tr>
<th>Specimen Details:</th>
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<tbody>
<tr>
<td>Deck thickness 1, $t_1$: 2.682 in</td>
</tr>
<tr>
<td>Deck thickness 2, $t_2$: 14.82 in</td>
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<tr>
<td>Fastener: 17-14-13-19</td>
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<thead>
<tr>
<th>Loading protocol details:</th>
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<tbody>
<tr>
<td>Protocol: FEMA 461</td>
</tr>
<tr>
<td>Load Rate: 0.005/0.01 in/sec</td>
</tr>
<tr>
<td>Max Displacement, $A_m$: 1.50' in 17 steps</td>
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</tbody>
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<table>
<thead>
<tr>
<th>Test Notes:</th>
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<tbody>
<tr>
<td>Actuator initial force: -730.00 lbs</td>
</tr>
<tr>
<td>Actuator initial displacement: 0.00 in</td>
</tr>
<tr>
<td>Deck warp: around F1, F2 in 0.019&quot; cycle at warping in c 0.352c</td>
</tr>
<tr>
<td>Deck warp: around F1, F2 in 0.051&quot; (P) drop in K, fall obv in 0.061 cycle</td>
</tr>
<tr>
<td>Failure: Beam, 0&quot;/0&quot; (P) -&gt; F1 -&gt; Beam, 0&quot;/0&quot; (P)</td>
</tr>
<tr>
<td>Ply warp: 0.01&quot; (P) -&gt; C, fall obv in 0.019&quot; cycle (p)</td>
</tr>
<tr>
<td>Ply warp: 0.033&quot; cycle (P)</td>
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<tr>
<td>Degradation cond: in 0.27&quot; vs 0.58&quot; (P) cycle</td>
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</tbody>
</table>

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<thead>
<tr>
<th>End of test Notes:</th>
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<tbody>
<tr>
<td>Max force: 0.25 lbs</td>
</tr>
<tr>
<td>Min Force: 0.25 lbs</td>
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<tr>
<td>Failure Mode: F1 -&gt; Beam, 0&quot;/0&quot; cycle (P)</td>
</tr>
<tr>
<td>F2 -&gt; Beam, 0&quot;/0&quot; cycle (P)</td>
</tr>
</tbody>
</table>
| F1 pulled out, fail. Complete deb. Test concluded
C. Appendix C – Material testing results
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Figure C-29: Connections project coupon test results – Steel supports
D. Appendix D – Diaphragm test rig
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F. Appendix F – Detailed per-specimen experimental results of panel buckling tests
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Peak = 15.71 kips

AISI S310 capacity
Figure F-40: 36-7-R2: Actuator and Sensor displacement time data
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Figure F-43: 36-5-R1: Actuator and Sensor force-displacement results
Figure F-44: 36-5-R1: Actuator and Sensor displacement time data
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Peak = 13.22 kips
AISI S310 capacity
Figure F-46: 36-5-R2: Actuator and Sensor displacement time data
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Figure F-50: 36-4-R1: Actuator and Sensor displacement time data
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G. Appendix G – Estimation of $P_{nb}$
Figure G-55: 36-5-R3: Determination of $P_{nb}$
H. Appendix H – Detailed numerical result of panel buckling FEA simulations
Figure H-56: Force vs displacement results for parametric evaluation
Figure H-57: Force vs Mid-span out-of-plane displacement results for parametric evaluation
REFERENCES


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