LATERAL RESPONSE OF COLD-FORMED STEEL DIAPHRAGMS WITH VARIABLE SHEATHING

Hernan Castaneda
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LATERAL RESPONSE OF COLD-FORMED STEEL DIAPHRAGMS WITH VARIABLE SHEATHING

A Dissertation Presented

by

HERNAN CASTANEDA

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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LATERAL RESPONSE OF COLD-FORMED STEEL DIAPHRAGMS WITH VARIABLE SHEATHING

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ABSTRACT

LATERAL RESPONSE OF COLD-FORMED STEEL DIAPHRAGMS WITH VARIABLE SHEATHING

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Cold-formed steel (CFS) framed buildings and subsystems have demonstrated safe and reliable performance while under lateral and seismic loads. Combined with their structural efficiency, this makes CFS-framed buildings popular choice even in high seismic zones. However, due to the complexity in response of CFS members and their interaction in subsystem and system levels, more research needs to be conducted to better understand the overstrength in the system, as well as the contribution of non-structural components to the overall response of the building. The lateral force resisting system (LFRS) of CFS framed buildings consist primarily of shear walls and diaphragms. The current AISI S400 North American Standard for Seismic Design of CFS framed systems includes only the design of shear walls and diaphragms sheathed with structural wood panels (AISI 2015).

In this design document in combination with provisions in ASCE 7-22, the design of shear walls is limited to seismic demand loads up to a 6-story building. The larger CFS-NHERI effort, of which this work is a part, seeks to expand this to 10 stories. The CFS-NHERI project culminates in a 10-story test on the shake table at UCSD UHPOST. To isolate
diaphragm behavior, which can be convoluted in a full-scale building test. The UMass effort examines the performance of diaphragms under lateral load, sheathed with novel sheathing. The best performing of these specimens will be tested also on the shake table, planned for 2023. A total of eight 10 ft by 15 ft CFS diaphragm specimens with variable sheathing on two CFS framing systems were tested following the cantilever test method. For this, a test rig was designed, fabricated, and installed in the structural laboratory to allow in-plane loading of diaphragms. The framing systems are comprised of 54 mils floor joist spaced by 2 ft and 97 mils joist spaced by 4 ft. Specimen sheathings included Oriented Strand Boards (OSB), steel deck, and a new dual structural skin system: structural Fiber Cement Boards (FCB) fastened to steel deck. Specimens were tested under monotonic and cyclic loading. The cyclic loading protocol was adopted from FEMA 461 (FEMA 2007). Results of this work provide a unique characterization of the lateral response of a CFS diaphragm sheathed with the dual skin system in which its behavior and strength are unknown, as well as a detailed progression of failure and failure mechanism in the diaphragms sheathed with form deck. Ultimately, a comparison with current design methods and design recommendations are provided.
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CHAPTER 1
INTRODUCTION

1.1 Background

In the construction industry, there are primarily two types of structural steel: hot-rolled steel and cold-formed steel (CFS). Hot-rolled steel shapes are formed at elevated temperatures while CFS shapes are formed at room temperatures by roll forming thin steel sheets, as shown in Figure 1.1.1. Both steels have similar, if not identical material properties.

Figure 1.1: Roll-forming for cold-formed steel shapes (adopted from: nucor.com, tophomas.com)

Common CFS shapes that are used in the construction industry are: C shapes with or without lips (a lipped and unlipped channels, respectively), Z shapes and steel deck panels. However, CFS shapes can be formed into a wide range of different geometric configurations as shown in Figure 1.2. Some of the benefits of using CFS framing include high structural efficiency strength and stiffness from optimized cross-sections, durable,
non-combustibility, economy in transportation, and quick and easy erection and installation of CFS components via screw fastened connections.

CFS has been used successfully for years in a variety of nonstructural systems, such as in partition walls or ceilings of residential and commercial buildings. However, with advances understanding of CFS system behavior, its use has been expanded into high seismic regions and more complex systems. A major structural application of CFS is in the framing of low to mid-rise residential and commercial buildings. A typical CFS framed building consists of light-frame construction in which the vertical and horizontal structural elements are formed via a repetitive framing system, as shown in Figure 1.3.
The framing members are typically spaced at 16 in. or 24 in. on center. In CFS framed buildings, the lateral force resisting system (LFRS) consists primarily of shear walls and diaphragms. Diaphragms are the floor or roof systems which are detailed to transfer lateral loads from wind or earthquakes to the vertical elements, such as the shear walls. Although extensive experimental and numerical works have been conducted on the lateral response of shear walls, the seismic response of CFS diaphragms has been studied for less, in comparison. Therefore, the need to conduct more research to better understand the lateral behavior of CFS diaphragms is evident to improve design guidelines for safer CFS structures.

1.2 Research Objectives

The main objective of this research is to evaluate the lateral response of CFS framed diaphragms under monotonic and cyclic loading following the cantilever test method. In addition, the work seeks to understand the effect of various floor sheathing materials;
including OSB, steel deck, and a new dual skin system: fiber cement boar (FCB) on top of steel deck. Joist spacing is also varied, between 2 ft and 4 ft. Finally, we aim to compare test results with current design methods to provide design recommendations.

1.3 Scope of Study

Current design methods for CFS framed buildings are limited in height to 65 ft, per ASCE 7-22. In a significant effort to extend CFS practice for buildings up to 10 stories, the CFS-NHERI project (CFSRC 2022) was conceived to examine the full-scale response of a 10-story building under simulated seismic loading using the Large High Performance Outdoor Shake Table (LH-POST) at the University of California, San Diego (UCSD). The project is additionally comprised of a set of experiments in which shear walls, diaphragm systems, and connections are tested to provide benchmarks of the full-scale building.

To expand the understanding of the lateral behavior of CFS floor diaphragms, a test-setup was designed, fabricated and installed in the Brack Structural Testing Laboratory at the University of Massachusetts Amherst to conduct cantilever diaphragm tests as a benchmark of the CFS-NHERI diaphragm experimental test program at the shake table at UCSD. A total of eight 10 ft by 15 ft diaphragms configurations were tested under monotonic and cyclic loading. The design of the specimens was based on the design of a 10-story CFS archetype building. Specimens were sheathed with OSB, steel deck and a new dual skin system. Two floor joist spacings were considered. In addition, this research contains a detail progression of failure mechanisms of steel deck diaphragms, as well as a comparison of the test results with the current design codes and a proposed design method for the new sheathing system.
1.4 Thesis Outline

This thesis is organized into six chapters as follows:

- **Chapter 1: Introduction**
  This chapter provides a general overview of the research topic, as well as the research objectives and the scope of this study.

- **Chapter 2: Literature Review**
  This chapter identifies the different components of a CFS diaphragm and the floor-to-wall framing systems in CFS framed buildings. In addition, it summarizes the literature that provide the foundation for this thesis and the recent contributions in this field.

- **Chapter 3: Specimen Design**
  This chapter contains a summary of the CFS-NHERI archetype building design which serves as the basis for the floor specimen design. Design and details of the test specimens are provided in this section. Diaphragm sheathed with OSB and steel deck followed current design methods and a new proposed design method for the dual skin system is introduced.

- **Chapter 4: Experimental Test Program**
  This chapter includes the design of the test-rig and test setup. Information of the test matrix and the description of each specimen are provided here, as well as details of the fabrication and construction of the specimens. In addition, layout of the instrumentation that is used on the test rig and specimen are presented here. Finally, the loading protocols that were employed are summarized in this chapter.
• Chapter 5: Experimental Results and Observations

This chapter presents the test results and discusses the lateral response of the CFS diaphragms. A detailed summary of the physical damage and progression of failure that was observed at different stages during the test is provided in this chapter. This chapter also discusses a comparison between specimens and the effects on the overall lateral response of the systems. In addition, comparison with the current design methods is discussed here.

• Chapter 6: Conclusions and Recommendations

This chapter contains design recommendations for the construction and design of CFS diaphragms. It also presents considerations for future work and summarizes the major findings of this research on the lateral response of CFS diaphragms with variable sheathing.
CHAPTER 2
LITERATURE REVIEW

2.1 Floor-to-Wall Framing Systems

In CFS construction there are three common framing systems, shown in Figure: platform framing, balloon framing, and ledger framing. In platform framing, the floor joist system sits on the top track of the wall studs. Subsequent stories bear on the sheathed floor as illustrated in Figure (a). In balloon framing, wall studs run continuously through each floor level and the floor joist system is fastened to the interior face of the wall studs as shown in Figure (b). Finally, in ledger framing, the floor joist system is connected to the top of the wall studs via a ledger and connected via clip angles. The sheathed floor is extended to the top track of the wall studs, and subsequent stories are framed on the sheathed floor as illustrated in Figure (c).

Figure 2.1: Types of framing systems in CFS construction
Traditionally, buildings are framed using platform construction and less frequently via balloon framing (Madsen et al. 2016). However, ledger framing is currently the dominant framing system in low-to-mid-rise CFS light framed construction in high seismic regions in North America due to its many benefits (Nakata et al. 2012). In ledger framing, floor joist spacing is independent of wall stud spacing as shown in Figure 2. This allows for structural and architectural flexibility. The ledger beam collects load from the floor joists and transfers it to the wall studs. While, in platform framing, floor joists are required to be aligned with wall studs as stipulated in AISI S240-15 (AISI 2015).

![Diagram of floor joist spacing and wall stud spacing in ledger framing](image)

Figure 2.2: Example of floor joist spacing and wall studs spacing in ledger framing

Another advantage of using ledger framing is that in multi-story buildings, the axial load in wall studs increases with the number of levels. In platform framing, where walls bear directly on joists, joist cross-section stability becomes the governing limit state, see Figure 2. (Ayhan et al. 2015). Note that in ledger framing, CFS joists must be used for the floor framing, while platform framing allows the use of non-CFS floor framing in combination with CFS wall framing. Construction method – stick-built or panelized – can
also influence floor framing system. Platform construction may be more conducive to panelized construction as floor panels can be placed on top of walls, whereas ledger framing is more conducive to stick-built construction, where joists are dropped into place one-by-one (Madsen et al. 2016).

Figure 2.3: Wall-to-floor connections in platform and ledger framing, highlighting axial compression on the joist caused in platform framing

2.2 Cold-Formed Steel Diaphragms and Shear Walls

A typical cold-formed steel (CFS) floor diaphragm consists of a frame of equally-spaced CFS floor joists sheathed with structural panels as shown in Figure . Transverse elements can be installed to provide additional lateral resistance on the overall diaphragm response (Xu et al. 2018). Plywood and oriented strand board (OSB) are commonly used as the floor sheathing material. However, with the advantages of steel deck over OSB (less material, non-combustibility, and reduced in the construction costs) the use of steel deck on top of CFS frames has been increasingly specified (Sputo 2019).
The design of diaphragms is based on analysis of its in-plane shear strength and stiffness. In addition, its contribution in resisting lateral loads (e.g., wind and earthquakes) depends on its relative stiffness compared with the shear walls. Diaphragms can be defined as flexible, rigid, or semi-rigid when comparing the maximum diaphragm deflection to the average inter-story drift (ASCE 7). Current design codes for the seismic design of CFS diaphragms sheathed with wood structural panels (AISI S400, AISI S100) are based on limited experimental work and most of this work that has been done is based on diaphragms sheathed with plywood. Lum (LGSEA 1998) provided allowable design strength values for CFS sheathed with plywood based on an analytical method given by Tissell and Elliott (2004) for estimating the diaphragm strength by multiplying individual fastener strength and factors based on fastener spacing and type of sheathing. These values are included in Table F2.4-1 of the design code AISI S400-15 (2015).

In 1999 the National Association of Home Builders Research Center (NAHBRC 1999) conducted an experimental test program of CFS diaphragms sheathed with structural
wood panels as is shown Figure , and what is believed to be the first experimental test for CFS diaphragms. A total of four 12 ft by 24 ft unblocked diaphragms were tested under monotonic loading. The wood sheathing was 23/32 in. thick and was fastened to the CFS framed by #8 screws and spaced by 6 in. at the edge panel and 12 in. in the field. Two floors joist sections with pre-formed holes were used: 800S162-43 and 1200S162-54, and joist were spaced by 2 ft. Results showed that prediction of the shear capacity of the diaphragm can be estimated by multiplying the individual screw connection shear capacity with the number of fasteners along one end of the diaphragm.

![Load-Deflection Curve - 8" Joists](image)

**Figure 2.5: NAHBRC test result and sheathing failure (NAHBRC 1999)**

Giving the limited experimental characterization on the lateral response of CFS framed diaphragms and the structural similarities between shear walls and diaphragms, research on wood-framed diaphragms and shear walls had given insight on how CFS diaphragms respond. In 1998 Cheung et al. (Cheung et al. 1998) conducted a quasi-static experimental test program using an 8 ft by 8 ft wood framed wall and sheathed with 1/2 in. thick plywood to verify an analytical model of wood diaphragms developed by Itani and Cheung (1984). In the experimental program, the effect of nail spacing, the modulus of
elasticity of the constituent materials, and the joist stiffness were investigated. The major parameters affecting the load-displacement response were found to be the nail spacing and the connection properties between the sheathing and the frame members. Dolan and Madsen (1992) performed an experimental test program of timber shear walls under monotonic and cyclic loading. The test results showed the important influence of the nail connection between the sheathing and the framing system. Bott (2005) studied the stiffness of wood diaphragms to delineate whether a diaphragm is classified as rigid or flexible. In addition, the importance of the diaphragm-to-wall connection was demonstrated by Brignola et al. (2012) through an experimental test program of timber diaphragms under monotonic and cyclic loading.

The Steel Deck Institute (SDI), based on extensive research, testing and analysis on steel roof and floor deck diaphragms; e.g. Luttrell (1967, 1981), Easley (1977), Nunna (2011), Porter and Easterling (1988), Avci and Easterling (2002), Avci et al. (2004), among others, included the design of steel deck diaphragms on CFS framing into the Diaphragm Design Manual 04 (SDI 2015) which meets the requirements of the AISI S310 (2016) and AISI S100 (2016). In addition, the SDI released the first edition of Steel Deck on Cold-formed Steel Framing Design Manual (SDI 2017). Recently the National Earthquake Hazards Reduction Program (NEHRP) has included recommendations for the seismic design of CFS deck diaphragm and adopting requirements of the AISI S400, AISI S310, AISI S100 and ASCE 7 design codes (NEHRP 2020).

In 1967, Luttrell investigated the shear behavior of light-gage steel diaphragms for roof decking and wall sheathing. About 70 specimens were tested to identify the influence of deck length, width, diaphragm size, fastener types, perimeter members stiffness,
material thickness and material properties. In addition, specimens were tested under monotonic and reversed loading. Figure shows the test setup. Tests results provided what it could be a first insight of the behavior of this type of diaphragms and showed that the shear strength increased with number of fasteners at the side-lap, deck thickness, deck width and material strength. This work formed the basis of the 1981 SDI diaphragm design manual (Luttrell, 1981).

![Test setup](image)

**Figure 2.6:** Test setup for light-gage diaphragms studied in Luttrell (1967)

Easley (1977) determined that the shear stiffness of the diaphragms depends greatly on the layout and number of fasteners that are used to connect steel deck to the framing members. Porter and Easterling (1988), as part of large experimental test program on concrete filled steel deck diaphragms, have guided the design of composite diaphragms and that is reflected in the current design provisions (AISI 2016, SDI 2015). Avci et al. (2004) tested five full-scale cantilever diaphragms and it was found that panel edge conditions dictate the strength and stiffness of the diaphragm in addition to the structural connectors and side-lap connections. Furthermore, Avci and Easterling (2002) examined
the behavior of web-cripping of steel deck profiles. A total of 78 tests were conducted on deck sections at Virginia Polytechnic Institute and State University. Specimens were subjected to one end flange loading. The study resulted in the development and calibration of the web crippling coefficients for the unfastened and fastened multi-web deck sections. Nunna (2011) validated the equations to determine the panel buckling of corrugated steel diaphragms based on historic diaphragm buckling test data.

As mentioned previously, CFS shear walls have been investigated thoroughly. Because shear walls and diaphragms are both lateral force resisting systems which resist loads primary by shear, findings from shear wall testing can provide insights into anticipated diaphragm behavior. Serrette et al. (1996, 1997, 2002) experimentally predicted the lateral strength of CFS shear walls sheathed with OSB which is adopted in the current design provisions in the North American Standard. Zhao (2002), Chen (2004), Blais (2006), Branston et al. (2006), Hikita (2006), and Boudreault et al. (2007) have explored the lateral response of CFS shear walls sheathed with OSB and plywood as part of large experimental test program at McGill University. The effect of shear wall aspect ratio, fastener spacing and thickness of CFS framed members were investigated to provide an analytical method to determine the shear strength and stiffness, and the seismic force modification factors ($R_d$, $R_o$) for CFS wood sheathed shear walls. Liu et al. (2012, 2014) in an experimental test program at Johns Hopkins University, investigated the effect of stud thickness and grade, presence of ledger at the top of the wall, and gypsum sheathing at the interior of the wall to expand the understanding of the lateral response of CFS shear walls.

With the advantage of using other sheathing materials instead of wood for higher strength of CFS shear walls, Yu (2010) examined the lateral response of CFS shear walls
sheathed with steel sheet, in which the impact of the wall aspect ratio, steel sheet thickness, and the fastener spacing of the sheathing connections were investigated. DaBreo et al. (2014) improved the chord stud design in the CFS frame of the wall due to the eccentric loading resulting from sheathing only one face of the wall. Santos and Rogers (2017) and Briere and Rogers (2017) also attempted to minimize the impact of the load eccentricity by sheathing both sides of the wall as well as by using a single steel sheet sandwiched between wall studs. Furthermore, shear walls sheathed with steel deck have been experimentally investigated under lateral and gravity load by Fülöp and Dubina (2004), and Zhang et al. (2017). Higher strength and stiffness walls have been also identified by Mohebbi et al. (2016) when FCB and gypsum sheathings were employed in the walls.

In an effort to analyze the behavior of CFS diaphragms and its contribution to the overall seismic response of the CFS structure, a two-story full-scale cold-formed steel ledger framed building was tested as part of system and subsystem seismic testing program in the CFS-NEES project, as shown in Figure (Peterman 2016a, 2016b). The results showed that nonstructural elements of the building may contribute to the lateral load-resisting system of the building along with the main lateral load resisting system such as shear walls. In addition, the CFS-NEES project showed that floor and roof diaphragms behaved as semi-rigid diaphragms (closer to rigid diaphragms) while being designed as flexible diaphragms based on current design codes.
The CFS-NEES project has motivated an experimental test program on the seismic in-plane response of CFS floor and roof diaphragms sheathed with OSB at McGill University to expand the understanding of the behavior of CFS diaphragms (Nikolaidou et al. 2015 and Latreille 2016). The test program was comprised of two phases; in phase one a total of ten 12 ft by 20 ft CFS diaphragm specimens were tested while in phase two a total of six specimens were tested. All tests were conducted following the cantilever test method under monotonic and cyclic loading, as illustrated in Figure 2.7. Effects on the sheathing screw size, panel edge blocking, floor joist orientation, fastener spacing, and presence of transverse elements were investigated. In addition, effects of non-structural elements on the lateral response were explored. Results showed that full edge blocking and the use of larger screw diameter increased the shear strength and stiffness of the diaphragm. In addition, floor joists orientation with respect to the loading direction had a minimal effect on the overall diaphragm response. Fastener spacing at the perimeter of the diaphragm and non-structural components showed a significant impact on the overall
diaphragm response. Furthermore, a non-linear finite element model (FEM) under monotonic and reverse cyclic loading of wood sheathed CFS diaphragm was developed using ABAQUS as shown in Figure (Chatterjee et al. 2017). The computational model captured the monotonic peak strength in comparison with the experimental results but had difficulties of capturing the response beyond the peak load. The reverse cyclic loading acted as an upper bound and lower bound to the experimental results based on the contact formulation between the edges of the sheathing panels.

Figure 2.8: Frame of strap-blocked specimen and monotonic deformation (Latreille 2016)

Figure 2.9: FEM wood sheathed CFS floor diaphragm and reverse cyclic response (Chatterjee et al. 2017)
In more recent works, Xu et al. (2019) investigated the response of a CFS diaphragm sheathed with a gypsum-based self-leveling underlayment (GSU) on top of a form deck through a cyclic load testing and numerical simulation. Two 12 ft by 12 ft diaphragms were tested, see Figure 1. Test results showed that the primary failure mode of the specimens was shear failure of the self-drilling screws that connected the steel deck to CFS framed. Ultimately, the numerical simulation showed that spacing of the screws connecting the steel deck to the joists at the floor perimeter can significantly impact the diaphragm response. Baldassino et al. (2021) experimentally investigated the behavior of CFS diaphragm sheathed with gypsum boards and light concrete slab on top of steel deck. In total six 16 ft by 15 ft specimens were tested under monotonic and cyclic loading. Two CFS framing systems: CFS truss beams and back-to-back studs were employed as is shown in Figure 2. Test results showed that concrete-filled diaphragms had the highest stiffness and strength. In addition, the CFS framed with back-to-back joists exhibited a greater stiffness than the truss beams.

![Figure 2.10: Experimental test setup (Xu et al. 2019)](image-url)
2.3 CFS-NHERI Project

Given the overstrength on the experimental lateral performance of CFS framed buildings; the CFS-NEES 2-story building (Peterman et al. 2016a,b) and the CFS-HUD 6-story building (Hutchinson et al. 2021), has motivated the CFS-NHERI project in which a full-scale 10-story CFS framed building would be tested at the outdoor shake table at UCSD (CFSRC 2022). Sub-system level (shear walls and diaphragms) and component level (fastener connections) provide benchmarks for the full-scale shake table tests.

As part of the CFS-NHERI project, Singh et al. (2021, 2022) investigated the behavior of higher strength CFS shear walls sheathed with steel sheets. Full-scale shear walls were placed in-line with gravity walls and tested under a sequence of increasing amplitude earthquake motions, and monotonic loading. The effect of wall finishes, shear wall Type I and Type II, symmetric and unsymmetric walls, and openings on the wall were
examined. In addition, Zhang and Schafer (2020), and Derveni et al. (2020, 2021) have conducted experimental testing for CFS-to-steel sheets connections, and CFS-to-OSB and steel-gypsum composite board connections respectively. Both with the main goal to characterize the fastener shear behavior under monotonic and cyclic loading as a tool to develop finite element models of CFS systems. Finally, the study of CFS diaphragms will be tested in 2023-2024 at the shake table at UCSD in which the study presented serves experimental springboard connecting lateral to seismic response.

As the result of the limited literature on the lateral response of CFS diaphragms, there is an evident need to address this to improve prediction capabilities for CFS diaphragms and assist professional engineers in the construction of safer and more efficient CFS structures. This and the CFS-NHERI project have motivated the experimental test program presented herein to expand the characterization of the lateral response of CFS diaphragms and provide a toolset for designing to practicing engineers.
CHAPTER 3
SPECIMEN DESIGN

As part of the CFS-NHERI project and the experimental test program of CFS diaphragms in a shake table test at the University of California in San Diego, a total of eight floor diaphragm configurations were tested at the Brack Structural Testing Laboratory at the University of Massachusetts Amherst. The CFS archetype building by Torabian et al. (2016) was used to design the diaphragm specimens. This chapter summarizes the archetype building diaphragm lateral design and the design of the diaphragm specimens.

3.1 Diaphragm Design Force

The archetype building by Torabian et al. (2016) is a mid-rise CFS framed building in which the floor is framed to the walls via ledger framing. The floor plan of this archetype building is shown in Figure and it has an overall dimension of 116 ft by 48 ft. The building was considered to be located in Southern California, and site class D. The seismic design done in accordance with the provisions in ASCE 7-16 (2016). As a result, the following seismic design parameters were adopted: spectral acceleration at short periods, $S_s=1.39g$, spectral acceleration at a period of 1 second, $S_1=0.57g$, and design spectral accelerations, $S_{DS} = 0.927g$ and $S_{D1} = 0.57g$. The effective seismic weight of the 10-story building was estimated based on the weights of the roof, floor and exterior walls as shown in Table 3.1 and Table 3.2.
Figure 3.1: Archetype building floor plan and elevation (from Torabian et al. 2016)

Table 3.1: Dead and live load unit weights

<table>
<thead>
<tr>
<th>Component</th>
<th>Weight (psf)</th>
<th>Component</th>
<th>Weight (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof Roofing + re-roof</td>
<td>5.0</td>
<td>Roofing + re-roof</td>
<td>1.0</td>
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<tr>
<td>Sheathing</td>
<td>2.5</td>
<td>Sheathing</td>
<td>14.0</td>
</tr>
<tr>
<td>Trusses</td>
<td>3.0</td>
<td>Trusses</td>
<td>2.8</td>
</tr>
<tr>
<td>Insulation + sprinklers</td>
<td>2.5</td>
<td>Insulation + sprinkles</td>
<td>4.0</td>
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<tr>
<td>2 layers gypsum + misc</td>
<td>7.0</td>
<td>2 layers gypsum + misc</td>
<td>8.2</td>
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<tr>
<td>Dead Load</td>
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<td>Dead Load</td>
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<tr>
<td>Live Load</td>
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<td>Live Load</td>
<td>40.0</td>
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Table 3.2: Effective seismic weight of the 10-story building

<table>
<thead>
<tr>
<th>Level</th>
<th>Height of each floor (ft)</th>
<th>System</th>
<th>Unit weight (psf)</th>
<th>Area (ft²)</th>
<th>Weight (kips)</th>
<th>Story weight (kips)</th>
<th>Unit weight (psf)</th>
<th>Story weight (kips)</th>
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<td>105.8</td>
<td></td>
<td>20</td>
<td></td>
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<td></td>
<td></td>
<td>Ext wall</td>
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<td>1350</td>
<td>20.3</td>
<td>158.5</td>
<td>-</td>
<td>105.8</td>
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<td></td>
<td></td>
<td>Int wall</td>
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<td>2644</td>
<td>26.4</td>
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<td>-</td>
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<tr>
<td></td>
<td></td>
<td>Roof top MEP</td>
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<td>-</td>
<td>6.0</td>
<td></td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Floor</td>
<td>9.44</td>
<td>Floor</td>
<td>30</td>
<td>5288</td>
<td>158.6</td>
<td></td>
<td>40</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Ext wall</td>
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<td>211.5</td>
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<td></td>
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<td>Int wall</td>
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<td>5288</td>
<td>52.9</td>
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<td>-</td>
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</table>

The diaphragm is analyzed in the long and short direction of the archetype building as a continuous beam on multiple supports (flexible diaphragm) and loaded with a uniform distributed unit load in Torabian et al (2016) and as shown in Figure . The resulting maximum shear from the continuous beam model was used to design the floor specimens. The advantage of the unit load analysis is that the results can be scaled to the associated demand load on the diaphragm. Results of this method for the 10-story building are presented in Table 3.3. \( F_p \) is the lateral seismic force induced at each level and was calculated per ASCE 7-16 Chapter 12 (2016); \( w \) is the distributed seismic force per width of the floor plan; \( V_{max} \) and \( M_{max} \) are the scaled maximum shear and moment from the beam analysis, and \( V_u \) is the demand shear in the diaphragm.
Figure 3.2: Simplified diaphragm analysis for a unit load (Torabian et al. 2016)

Table 3.3: Diaphragm design loads for the 10-story building

<table>
<thead>
<tr>
<th>Long direction (116 ft)</th>
<th>Fp (lb)</th>
<th>w (lb/ft)</th>
<th>Vmax (lb)</th>
<th>Mmax (lb-ft)</th>
<th>Vu (lb/ft)</th>
<th>Short direction (48 ft)</th>
<th>Fp (lb)</th>
<th>w (lb/ft)</th>
<th>Vmax (lb)</th>
<th>Mmax (lb-ft)</th>
<th>Vu (lb/ft)</th>
</tr>
</thead>
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<td>366</td>
<td>5868</td>
<td>28002</td>
<td>122</td>
<td>Roof</td>
<td>42466</td>
<td>885</td>
<td>11298</td>
<td>38378</td>
<td>97</td>
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<td>559</td>
<td>8962</td>
<td>42764</td>
<td>187</td>
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<td>64853</td>
<td>1351</td>
<td>17254</td>
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<td>149</td>
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<td>61202</td>
<td>528</td>
<td>8458</td>
<td>40357</td>
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<td>38020</td>
<td>166</td>
<td>7th</td>
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<tr>
<td>5th</td>
<td>50664</td>
<td>437</td>
<td>7001</td>
<td>33408</td>
<td>146</td>
<td>5th</td>
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<td>13479</td>
<td>45788</td>
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<tr>
<td>4th</td>
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<td>47809</td>
<td>996</td>
<td>12719</td>
<td>43208</td>
<td>110</td>
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</table>
3.2 Design of Diaphragm Specimen Sheathed with OSB

The shear capacity of the specimen sheathed with OSB was calculated in accordance with AISI S400 Chapter F Table F2.4-1 (2015). The table provides design shear strength values for a limited number of diaphragm configurations sheathed with wood structural panels. 23/32 in. OSB panels are provided for the test specimen and the capacity was conservatively determined by a linear interpolation using the capacities for unblocked diaphragm sheathed with 7/16 in. and 15/32 in. thick structural wood panel. The nominal shear strength for 23/32 in. thick sheathing on an unblocked diaphragm with screws spaced 6 in. at the edges and 12 in. in the field is 955 lb/ft.

3.3 Design of Diaphragm Specimens Sheathed with Steel Deck

SDI released the first edition of Steel Deck on Cold-formed Steel Framing Design Manual (SDI 2017) which meets the requirements of the AISI S310 (2016) and AISI S100 (2016). The recommendations in the SDI design guide were followed to design the steel deck diaphragm specimens tested herein. The diaphragm design is controlled by either the contributions of the shear capacity of the sheathing connections or panel buckling. Sheathing connections are comprised of support, edge, and side-lap connections. Individual fastener connection shear strength is determined per AISI S100 Chapter J (2016). Two deck profiles are used, 24 gage-9/16" (2.5" pitch-35") for joists spaced at 2 ft and 24 gage-1” (4.5” pitch-27") for joists spaced at 4 ft. Support connection layouts were defined based on the deck profile as is shown in Figure. These patterns are consistent throughout the length of the specimen. Side-lap and edge connections are spaced by 8 in and meet all spacing limits stipulated in AISI S310 (2016).
The analysis to determine the shear strength of the diaphragm was conducted by considering the distribution of the reaction forces at each of the sheathing connections in a deck panel and using equations in Chapter D of AISI S310 (2016). Each screw connection is limited by its shear strength. Forces at the support connections are considered to linearly vary from the side-lap to the neutral axis of the panel, as illustrated in Figure 3.3. Three limit states are calculated for the shear strength controlled by connections, \( S_{nc} \), \( S_{ni} \), and \( S_{ne} \). \( S_{nc} \) is the shear strength controlled by connections at the corners of interior panels or edge panels, \( S_{ni} \) is the shear strength controlled by connections at interior panels or edge panels, and \( S_{ne} \) is the shear strength controlled by connections along the edge in an edge panel. The shear strength controlled by panel buckling was calculated for each configuration and it was found that it does not control the design. The shear capacity for 24 gage-9/16” deck fastened to 54 mil CFS framed with joists spaced at 2 ft is 1.67 kips/ft and for 24 gage-1” deck fastened to 97 mil CFS framed with joists spaced at 4 ft is 1.35 kips/ft. Calculations are presented in Appendix A.

Figure 3.3: Support connections layouts
3.4 Design of Diaphragms Sheathed with a Dual Skin System

The dual skin system is a new proposed sheathing system, and its capacity and behavior are unknown and not addressed in modern specifications. FCB is attached on top of the steel deck via screw fasteners spaced by the deck pitch and along the deck flutes spaced by 6 in at the edge and 12 in over the field. In a first attempt for predicting the strength of this new system, Torabian (2020) proposed a model in which one possible implication of the FCB panel fastened on top of the steel deck is that the side-lap slip between deck panels could be restrained as shown in Figure 3.4. In addition, end warping at the end of the steel panel is prevented as illustrated in Figure 3.4. Eq. 3.4.1 determines the diaphragm shear stiffness proposed by Luttrell (1981) in the first edition of the Steel Deck Institute Diaphragm Design Manual (SDI DDM-01). This equation considers the shear deformation of the steel deck ($\Delta_s$), the distortion deformation ($\Delta_d$), the discrete fastener deformation at the side-lap ($\Delta_c$), and the deformation at the outer edges of the system ($\Delta_e$). In Eq. 3.4.2 the potential benefits of the dual skin system on the diaphragm shear stiffness are represented assuming that $\Delta_d$ and $\Delta_c$ are small, and therefore neglected, which leads to
a higher shear stiffness (Torabian 2020). To determine the diaphragm shear stiffness of the dual skin system, Eq. 3.4.3 is used. Note that $\Delta_s$ involves the shear deformation of the steel deck and the FCB.

$$G_{deck}' = \frac{Pa/l}{\Delta_s + \Delta_d + \Delta_c + \Delta_e}$$  \hspace{1cm} \text{Eq. 3.4.1}$$

$$G_{dual\ skin}' = \frac{Pa/l}{\Delta_s + \Delta_e}$$  \hspace{1cm} \text{Eq. 3.4.2}$$

$$G_{dual\ skin}' = \frac{1}{\frac{2E_s t_s}{1 + \nu_s} + \frac{E_c t_c}{1 + \nu_c} + \frac{25}{a (2\alpha_1 + n_p \alpha_2 + n_e)}}$$  \hspace{1cm} \text{Eq. 3.4.3}$$
Where,

- $E_s$ = elastic modulus of steel deck
- $t_s$ = thickness steel deck
- $\nu_s$ = Poisson’s ratio steel deck
- $d$ = steel deck pitch
- $s$ = developed flute width per Eq. D2.1-2 in Chapter D of AISI S310 (2016)
- $E_c$ = elastic modulus of FCB
- $t_c$ = thickness of FCB
- $\nu_c$ = Poisson’s ratio FCB
- $l$ = diaphragm length
- $a$ = diaphragm depth
- $S_f$ = structural fastener flexibility per section D5.2 in AISI S310 (2016)
- $\alpha_1$ = end fastener distribution factor across diaphragm depth, $a$
  \[
  \alpha_1 = \frac{\sum x_e}{a}
  \]
- $X_e$ = distance from diaphragm center line to any fastener at the end support
- $\alpha_2$ = support fastener distribution similar to $\alpha_1$
  \[
  \alpha_2 = \frac{\sum x_p}{a}
  \]
- $n_p$ = number of interior supports along diaphragm depth, $a$
- $n_e$ = number of edge connections between cross supports

The shear strength of the dual skin system is taken as the minimum value of the shear strength controlled by the end fastener, $S_{ne}$ (Eq. 3.4.4), corner fastener buckling, $S_{nl}$ (Eq. 3.4.5), and corner fastener, $S_{nc}$ (Eq. 3.4.6) (Torabian 2020). Using these equations,
the shear strength of the dual skin diaphragm with a 2 ft joist spacing is 2.94 kips/ft and for joist spaced by 4 ft is 3.18 kips/ft.

\[ S_{ne} = (2\alpha_1 + n_p\alpha_2 + n_e) \frac{Q_f}{l} \]  
Eq. 3.4.4

\[ S_{ni} = [2A(\lambda - 1) + B] \frac{Q_f}{l} \]  
Eq. 3.4.5

\[ S_{nc} = Q_f \sqrt{\frac{N^2B^2}{l^2N^2 + B^2}} \]  
Eq. 3.4.6

Where,

\( Q_f \) = fastener shear strength per section J4.3 in AISI S100 (2016)

\( A \) = number support screws at side-lap at deck ends

\( \lambda \) = connection strength reduction factor per Eq. D1-4a in AISI S310 (2016)

\( B \) = factor defining screw interaction per Eq. D1-5 in AISI S310 (2016)

\( N \) = number of screws into support per ft along deck ends
CHAPTER 4
EXPERIMENTAL TEST PROGRAM

As part of the experimental test program for CFS diaphragms at the shake table at UCSD in the CFS-NHERI project, eight unblocked floor diaphragm configurations were tested following a cantilever test method at the Brack Structural Laboratory at the University of Massachusetts Amherst to expand the understanding of the lateral response of CFS floor diaphragms sheathed with OSB, steel form deck, and a new dual skin system consisting of Fiber Cement Board (FCB) fastened on top of the form deck, in which the capacity and behavior of these dual skin systems is unknown.

4.1 Test Rig

The test apparatus used in this research was design to allow application of in-plane forces distributed along the perimeter of diaphragm specimens. Figure shows the test rig configuration for the cantilever diaphragm test program. The test rig consists of a steel frame comprised of four built-up C shape beams and pin connected. One end of the frame is fixed to the floor, comprising the fixed reaction at one end of the cantilever. The other end, which is connected to a hydraulic actuator, can move freely along the loading plane. The fixed beam as indicated in Figure is connected to the strong floor via two steel supports, spaced by 10 ft. The loading beam is shown as the free beam (see Figure ) which is supported at each end by two roller supports as illustrated in Figure. The rollers at the top and the bottom of the beam were designed to prevent any vertical displacement and to withstand a maximum uplift force of 10 kips.
Load is provided by a hydraulic MTS actuator attached at the midpoint of the loading beam. The actuator has a tensile capacity of 146 kips and 100 kips of compressive capacity. Two transverse beams are used to provide a connection support to the ends of the
floor sheathing. The test frame has the capabilities of accommodate 15 ft by 15 ft specimens and 10 ft by 15 ft specimens. To accommodate a 10 ft by 15 ft specimen, the fixed beam slides in 5 ft over the transverse beams. Supports are connected directly to the strong floor at the tie-down location of the floor. Each tie-down location has a capacity up to 200 kips. Components of the test frame were designed based on the provisions of AISC 360-16 (AISC 2017). In addition, ABAQUS was used as a supplementary tool for the analysis. The rig components were designed according to Allowable Strength Design (ASD) to ensure elasticity under a maximum load of 146 kips. Transverse stiffeners were installed along all beams of the test frame to provide flexural and torsional resistance. A total of four additional holes were made with a magnetic drill on the web of the loading beam, as illustrated in Figure 4.3, to ensure a symmetric connection point with the fixed beam when fastening the specimen. Figure 4.3 illustrates the fabrication and installation of 1/4 in. steel shims to prevent any contact of self-drilling screws from the test specimen with the web of the loading and fixed beam.

Figure 4.3: Additional holes in the web of the loading beam
4.2 Description of Test Specimens

Design of the test specimens was discussed in Chapter 3. A total of eight 10 ft x 15 ft CFS floor diaphragms were tested under monotonic and cyclic load. Table 4.1 shows the test matrix. The floor diaphragm specimens consist of a series of equally-spaced floor joist framed to a ledger beam via clip angle connections. Two floor joist spacings are considered, 24 in. (2 ft) and 48 in. (4 ft) as illustrated in Figure 4.4. In addition, two different floor joist thickness were employed, 54 mils and 97 mils (1 mils = 1/1000 in). Floor joists are lipped channels, 12 in. deep and 2 in. wide (1200S200). Ledgers are unlipped channels which are 12 in. deep and 2 in. wide (1200T200) and match the joist thickness. Because ledgers cap the floor joists, they are sized to accommodate out-to-out joist dimensions. Diaphragms were sheathed with OSB, steel form deck and FCB on top of the form deck. Specimen names reflect the test configurations. For example, “M-97S-48-Deck”, is a specimen under monotonic load with a 97 mils joist spaced 48 in and sheathed with steel.
deck. The first letter of the specimen name represents the loading condition, “M” for monotonic load and “C” for cyclic load. The second term indicates the thickness of the floor joist; the third, joist spacing; and the last term, the floor sheathing. “Dual” represents the dual structural skin system: FCB on top of steel deck (FCB+Deck) and “Bare” corresponds to the CFS framed system with not sheathing.

Table 4.1: Cantilever diaphragm test matrix

<table>
<thead>
<tr>
<th>Specimen Name</th>
<th>CFS Joist</th>
<th>Joist center-to-center Spacing (in.)</th>
<th>Sheathing</th>
<th>Steel Deck</th>
<th>Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>M-54S-24-Bare</td>
<td>1200S200-54</td>
<td>24</td>
<td>-</td>
<td>-</td>
<td>Monotonic</td>
</tr>
<tr>
<td>C-54S-24-OSB</td>
<td>1200S200-54</td>
<td>24</td>
<td>23/32” OSB</td>
<td>-</td>
<td>Cyclic</td>
</tr>
<tr>
<td>M-54S-24-Deck</td>
<td>1200S200-54</td>
<td>24</td>
<td>-</td>
<td>24 gage-9/16”(2.5” pitch-35”)</td>
<td>Monotonic</td>
</tr>
<tr>
<td>C-54S-24-Deck</td>
<td>1200S200-54</td>
<td>24</td>
<td>-</td>
<td>24 gage-9/16”(2.5” pitch-35”)</td>
<td>Cyclic</td>
</tr>
<tr>
<td>C-54S-24-Dual</td>
<td>1200S200-54</td>
<td>24</td>
<td>¾” FCB</td>
<td>24 gage-9/16”(2.5” pitch-35”)</td>
<td>Cyclic</td>
</tr>
<tr>
<td>M-97S-48-Deck</td>
<td>1200S200-97</td>
<td>48</td>
<td>-</td>
<td>24 gage-1”(4.5” pitch-27”)</td>
<td>Monotonic</td>
</tr>
<tr>
<td>C-97S-48-Deck</td>
<td>1200S200-97</td>
<td>48</td>
<td>-</td>
<td>24 gage-1”(4.5” pitch-27”)</td>
<td>Cyclic</td>
</tr>
<tr>
<td>C-97S-48-Dual</td>
<td>1200S200-97</td>
<td>48</td>
<td>¾” FCB</td>
<td>24 gage-1”(4.5” pitch-27”)</td>
<td>Cyclic</td>
</tr>
</tbody>
</table>

Note: CFS joist notation per AISI (2016)

Figure 4.5: Floor joist spacing; (a) 24 in o.c.; (b) 48 in o.c.
4.2.1 M-54S-24-Bare and C-54S-24-OSB

Specimen M-54S-24-Bare was the CFS framed of specimen C-54S-24-OSB as shown in Figure (a). The main purpose of this configuration was to evaluate any contribution to the shear strength of the system attributed to the steel frame of the diaphragm without the floor sheathing. This specimen was conservatively tested under monotonic loading (within the elastic region) since it was the bare frame of Specimen C-54S-24-OSB. Figure (b) shows the specimen sheathed with 23/32 in. OSB panels. The tongue-and-groove of the panels were placed perpendicular to the joist direction and #10 self-drilling screws were used to connect the OSB panels to the steel frame as illustrated in Figure .

Figure 4.6: (a) specimen: M-54S-24-Bare; (b) specimen: C-54S-24-OSB
4.2.2 M-54S-24-Deck and C-54S-24-Deck

Specimen M-54S-24-Deck is shown in Figure 4.7. Open web joists with stiffened holes were used for the framing and the steel frame was sheathed with a 9/16 in form deck. Specimen C-54S-24-Deck used the same CFS frame from the specimen M-54S-24-Deck. For this case, fastener connections of the new sheathing to the steel frame were shifted by 1/2 in. from the existing hole in the steel frame. The intention of these specimens was to assess the shear strength of the diaphragm controlled by the strength of the sheathing connections per AISI-S310 (AISI 2016). Figure 4.7 illustrates the sheathing connections, which are comprised of a support connection, an edge connection, and a side-lap connection. Note that the sheathing connections follow the nomenclature used in the DDM04 Design Diaphragm Manual (SDI 2015). The support connections were #12 self-
drilling screws for attaching the steel deck to the top flange of the floor joist. Edge connections were also #12 screws for connecting the edge of an exterior deck panel to the top flange of the ledger. And the side-lap connections used #10 screws to connect the edges of two interior deck panels.

Figure 4.8: Steel bare frame and frame with steel deck sheathing; specimens: M-54S-24-Deck and C-54S-24-Deck
Figure 4.9: Steel deck fastener layout; specimens: M-54S-24-Deck and C-54S-24-Deck

4.2.3 C-54S-24-Dual

Specimen C-54S-24-Dual is similar to specimen C-54S-24-Deck, with the exception that FCB was fastened on top of the steel deck creating the dual skin system. In this specimen, half of the length of the floor joists had stiffened holes and they were installed alternating to minimize any effect of the asymmetric joist as shown in Figure (a). The main purpose of this specimen was to evaluate the performance of the new dual skin system (Figure (b)), in which the capacity and behavior of these dual skin systems is unknown. The tongue-and-groove joint of the FCB was placed perpendicular to the side-lap direction and #10 self-drilling screws were used to connect the FCB to the steel deck as illustrated in Figure.
Figure 4.10: Steel bare frame and frame with dual skin sheathing; specimen: C-54S-24-Dual

Figure 4.11: FCB sheathing-to-steel deck screw connections; specimen: C-54S-24-Dual
4.2.4 M-97S-48-Deck and C-97S-48-Deck

Figure shows at the top the CFS frame of specimen M-97S-48-Deck and at the bottom shows the specimen sheathed with 1 in. form deck. Similar to the specimens 54S-24-Deck, the CFS frame was reused, and the fastener connections of the next sheathing were shifted by 1/2 in. from the existing hole in the steel frame. The intention of these specimens was to assess the shear strength of the diaphragm when doubling the floor joist spacing and thickness of the CFS frame. All sheathing connections are illustrated in Figure 4.12.

Figure 4.12: Bare frame and sheathed specimen with steel deck; specimens: M-97S-48-Deck and C-97S-48-Deck
4.2.5 C-97S-48-Dual

Figure shows the Specimen C-97S-48-Dual which is similar to the specimen 97S-48-Deck but with the addition of FCB on top of the steel deck. This specimen aims to assess the strength and behavior of the dual skin system on a steel frame with heavier joists, but at a larger spacing. Figure illustrates the fastener layout of the FCB-tosteel deck connections that uses #10 self-drilling screws.
Figure 4.14: Steel frame with dual skin sheathing; specimen: C-97S-48-Dual

Figure 4.15: FCB sheathing-to-steel deck screw connections; specimen: C-97S-48-Dual
### 4.3 Specimen Fabrication

Table summaries a list of all components used in the construction of the test specimens. Because screws contribute significantly to the lateral-load response of CFS diaphragms, Table contains a list of all self-drilling screws used for all connections. In total four different screws were used as shown in Figure.

**Table 4.2: Summary of components used in the floor diaphragms**

<table>
<thead>
<tr>
<th>Component</th>
<th>Length (in)</th>
<th>Depth (in)</th>
<th>Width (in)</th>
<th>Thickness (mils)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joist</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1200S200-54</td>
<td>120</td>
<td>12</td>
<td>2</td>
<td>54</td>
</tr>
<tr>
<td>1200S200-97</td>
<td>120</td>
<td>12</td>
<td>2</td>
<td>97</td>
</tr>
<tr>
<td>Ledger</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1200T200-54</td>
<td>168</td>
<td>12</td>
<td>2</td>
<td>54</td>
</tr>
<tr>
<td>1200T200-97</td>
<td>168</td>
<td>12</td>
<td>2</td>
<td>97</td>
</tr>
<tr>
<td>Clip-angle</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>L3½X3½-54</td>
<td>11</td>
<td>-</td>
<td>-</td>
<td>54</td>
</tr>
<tr>
<td>1200S200-54</td>
<td>12</td>
<td>-</td>
<td>-</td>
<td>54</td>
</tr>
<tr>
<td>End-angle</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>L3½X3½-54</td>
<td>120</td>
<td>-</td>
<td>-</td>
<td>54</td>
</tr>
<tr>
<td>L3X3-97</td>
<td>120</td>
<td>-</td>
<td>-</td>
<td>97</td>
</tr>
<tr>
<td>Sheathing</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>OSB</td>
<td>4’X8’X23/32”</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FCB</td>
<td>4’X8’X3/4”</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deck</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>24 gage-9/16&quot;(2.5&quot; pitch-35&quot;)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>24 gage-1&quot;(4.5&quot; pitch-27&quot;)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table 4.3: Summary of self-drilling screws used to connect components**

<table>
<thead>
<tr>
<th>Connection</th>
<th>Self-Drilling Screw</th>
<th>Length (in)</th>
<th>Screw Head</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joist-to-clip-angle</td>
<td>#12</td>
<td>3/4</td>
<td>Hex-Washer</td>
</tr>
<tr>
<td>Ledger-to-clip-angle</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deck-to-steel frame</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deck side-lap</td>
<td>#10</td>
<td>3/4</td>
<td></td>
</tr>
<tr>
<td>FCB-to-deck</td>
<td>#10</td>
<td>1-1/2</td>
<td>Wafer</td>
</tr>
<tr>
<td>OSB-to-steel frame</td>
<td>#10</td>
<td>1-7/16</td>
<td></td>
</tr>
</tbody>
</table>
The construction of all specimens started with the cut of CFS studs to the length of the members. To cut the length of any component, a bandsaw was used as shown in Figure (a). Given the limited available material, ledgers were obtained from cutting the lip of one of the flanges of a stud section (see Figure (b)), and from removing its opposite flange (see Figure (c)) to ensure a “perfect” fit for the floor joist-to-ledger connection. The obtained ledger sections were 7 ft long and a 6 in depth stud section was used to splice the end of two members in the middle, as shown in Figure . The spliced member matched the thickness of the ledger, and #12 self-drilling screws were used for fastening the web of the ledger to the web of the spliced member.

Following this, the ledgers were bolted to the loading and fixed beam of the test rig via 3/4 in. structural bolts. With the ledger in place and level, all remaining locations or connecting bolts were marked with a transfer punch and the ledgers were dismounted. Using the magnetic drill, holes were drilled at the marked locations. After all the marked locations were drilled, the ledgers were again installed and bolted to the test rig, see Figure . The bolts were installed snug-tight using a socket wrench.

Figure 4.16: Self drilling screws
Figure 4.17: (a) cutting to the length of the members with a bandsaw; (b) cutting the lip of a stud section with a jigsaw; (c) cutting the flange

Figure 4.18: Ledger splice
Figure 4.19: Ledger drilling holes with a magnetic drill and ledger-to-test rig connections

With the ledgers bolted to the loading and fixed beam of the rig, clip-angles were aligned and clamped on the interior face of the ledger web. The clip-angles were spaced accordingly with the corresponding floor joist spacing. Ledgers were de-attached from the rig and clip angles were fastened with six #12 screws spaced by 2 in. The screws were fastened from the back of the ledger web so the steel shims could prevent any contact of the screws with the test rig. After clip angles were fastened, the ledgers were again installed and bolted to the test rig, see Figure.
Figure 4.20: Clip angle-to-ledger connection

The ends of the floor sheathing were connected to the transverse beams of the test rig via two steel end-angles that matched the thickness of the floor joist. The end-angles for the specimens were prepared in a similar way as the ledgers, except for the spliced at the middle of the component. After the end-angles were marked and drilled, the angles were installed snug-tight with the socket wrench, as shown in Figure .
After the ledgers and end-angles were in place, all the floor joists were clamped to the clip-angle and to the top flange of the ledger. Clip-angles were marked and fastened to the web of the joist via six #12 self-drilling screws and spaced by 2 in., as shown in Figure (a). Open web joist were stiffened up at the ends with a 12 in. long 1200S200-54 stud section, which was used at the same time as a clip-angle connection. The stud section was fastened back-to-back to the joist (see Figure (b)) and the flange of the stud was fastened to the web of the ledger as shown in Figure (c). Given the limited area for fastening the screws on the open web of the joist, it was conservatively decided to have two rows of six #12 self-drilling screws, top and bottom, and spaced by 2 in. In addition, two fasteners spaced by 2 in. were fastened at the sides of the bigger opening in the web of the joist.

Figure 4.21: Specimen end-angle-to-rig connection
Once the CFS frame was constructed in the test rig, floor sheathings were prepared. For steel deck sheeting, panels were cut to the length using metal shears as illustrated in Figure 4.22 (a). OSB and FCB were cut by using a circular saw as shown in Figure 4.22 (b) and (c) respectively.

Figure 4.22: Clip angle-to-floor joist connection; (a) clip angle; (b) and (c) stiffener for open web joist

Figure 4.23: Cutting sheathing material; (a) metal shears for the steel deck; electric circular saw for the (b) OSB and (c) FCB
Finally, the floor sheathing was placed on top of the CFS frame and fastened following the layouts presented earlier in sections 4.2.1 through 4.2.5. OSB was installed perpendicular to the joist direction and staggered by minimum 2 ft, see Figure (a). Steel deck was also installed perpendicular to the joists. Both, OSB and steel deck, started at one side of the test rig with full width panels and ended at the opposite side with a shorter width panel. Figure (b) shows the installation of FCB on top of steel deck. Panels were installed perpendicular to the side-lap of the steel deck and staggered by minimum 2 ft. Similar to the OSB and the steel deck, full width panels started at one side of the rig and ended with a reduced width panel.

Figure 4.24: Floor sheathing installation; (a) OSB; (b) steel deck and FCB
4.4 Instrumentation

Force and displacement from the MTS actuator were obtained from a built-in load cell and linear variable differential transformer (LVDT) respectively. In conformance with AISI S907 (AISI 2017), four linear potentiometers were placed at opposite corners of the test rig, two at each corner; X1, Y1, X2 and Y2 in Figure 4.25, to measure in-plane displacement of the loading and fixed beams. In addition, two linear potentiometers (sensors Z1 and Z2 in Figure 4.25) were installed at the top of the loading and fixed beam to measure any out-of-plane displacement. A total of nine linear potentiometers (sensors A1 through C3) were placed at the top of the specimens to measure out of the plane displacements of the floor sheathing as illustrated in Figure 4.25. All sensors were attached to rigid built-up unistrut frames that were fixed to the strong floor of the laboratory.

Figure 4.25: Location of linear potentiometers to the rig
All linear potentiometers were calibrated in an Instron test machine in which a displacement protocol was created to record the output voltage of the sensor every quarter of an inch. Figure shows the setup for the calibrations of the sensors. Sensors were powered with 10 volts by an external source of power and voltage readings were obtain with a digital voltage meter. All sensors were wired to a data acquisition system in addition to the output.
signals for force and displacement of the MTS controller for the hydraulic actuator. Finally, experimental raw data was recorded in a computer using LabVIEW, as is shown in Figure 4.28.

Figure 4.27: Sensor calibration

Figure 4.28: LabView interface to record experimental raw data
4.5 Monotonic and Cyclic Loading Protocol

A constant load rate, in displacement control, of 0.01 in/sec was implemented for all monotonic and cyclic loading. The cyclic loading protocol was adopted from the Interim Testing Protocols for Determining the Seismic Performance Characteristics of Structural and Nonstructural Components, FEMA 461 (FEMA 2007). The FEMA protocol consists of three regions of loading: initiation cycles, trailing cycles, and primary cycles as illustrated in Figure 4.29. A loading cycle is completed when the frame restores to its initial position after being displaced to a target displacement amplitude, $\Delta$, in the positive and negative direction of loading.

![Figure 4.29: FEMA 461 loading protocol (FEMA 2007)](image)

The first six loading cycles are the initiation cycles, that are executed at small amplitudes in which the lowest damage state is first observed. A primary cycle is a single loading cycle that is larger than its preceding cycle and is followed by a trailing cycle. The trailing cycles have an amplitude equal to 1.4 times the amplitude of their preceding...
primary cycle. $\Delta_m$, is the targeted maximum deformation amplitude at which the most severe damage level is expected to initiate. At least 26 cycles must be completed prior to end of a test. If the specimen has not initiated the most severe damage state at $\Delta_m$, the amplitude increases further by a constant increment equal to $0.3\Delta_m$. $\Delta_m$ was conservatively taken equal to 1 in and determined from the monotonic experimental tests at McGill University (Nikolaidou et al. 2016). Table summarizes the amplitude values adopted for the loading protocol.

<table>
<thead>
<tr>
<th>Cycle No.</th>
<th>$\Delta$ (in.)</th>
<th>Cycle No.</th>
<th>$\Delta$ (in.)</th>
<th>Cycle No.</th>
<th>$\Delta$ (in.)</th>
<th>Cycle No.</th>
<th>$\Delta$ (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-2</td>
<td>0.018</td>
<td>15-16</td>
<td>0.186</td>
<td>29-30</td>
<td>1.600</td>
<td>43-44</td>
<td>3.700</td>
</tr>
<tr>
<td>3-4</td>
<td>0.025</td>
<td>17-18</td>
<td>0.260</td>
<td>31-32</td>
<td>1.900</td>
<td>45-46</td>
<td>4.000</td>
</tr>
<tr>
<td>5-6</td>
<td>0.035</td>
<td>19-20</td>
<td>0.364</td>
<td>33-34</td>
<td>2.200</td>
<td>47-48</td>
<td>4.300</td>
</tr>
<tr>
<td>7-8</td>
<td>0.048</td>
<td>21-22</td>
<td>0.510</td>
<td>35-36</td>
<td>2.500</td>
<td>49-50</td>
<td>4.600</td>
</tr>
<tr>
<td>9-10</td>
<td>0.068</td>
<td>23-24</td>
<td>0.714</td>
<td>37-38</td>
<td>2.800</td>
<td>51-52</td>
<td>4.900</td>
</tr>
<tr>
<td>11-12</td>
<td>0.095</td>
<td>25-26</td>
<td>1.000</td>
<td>39-40</td>
<td>3.100</td>
<td></td>
<td></td>
</tr>
<tr>
<td>13-14</td>
<td>0.133</td>
<td>27-28</td>
<td>1.300</td>
<td>41-42</td>
<td>3.400</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: 1) $\Delta_m = 1$ in.
2) Load rate = 0.01 in/sec
CHAPTER 5
EXPERIMENTAL RESULTS AND OBSERVATIONS

5.1 Force-Displacement Results

All test results are shown in section 5.1.1 through section 5.1.4. The monotonic and cyclic force-displacement results for specimens sheathed with steel deck are plotted in the same figure. Table summarizes the maximum diaphragm shear strength and the initial elastic stiffness of each specimen. The elastic stiffness, $K_e$, was determined as a secant stiffness calculated at the 40% of the maximum strength, $P_{\text{max}}$, accordingly to AISI S907 provisions for determining the strength and stiffness of CFS diaphragms following a cantilever test method (AISI 2017). Net diaphragm displacement for each specimen was calculated using Eq. 5.7 that is adopted from AISI S907.

$$\Delta = X_1 - [X_2 + (Y_1 + Y_2)a/b]$$  \hspace{1cm} \text{Eq. 5.7}

Where, $X_1$ and $Y_1$ are the orthogonal in-plane displacements of the loading beam (see Figure ), $X_2$ and $Y_2$ consider any displacement of the fixed beam (see Figure ), $a$ is the length of the diaphragm perpendicular to the load and $b$ is the depth parallel to the load. Appendix B contains data from all sensors.
### Table 5.1: Maximum diaphragm strength and elastic stiffness

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$P_{max}$ (kips)</th>
<th>$\Delta_{max}$ (in)</th>
<th>$P_{40}$ (kips)</th>
<th>$\Delta_{40}$ (in)</th>
<th>$K_e$ (kips/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M-54S-24-Bare*</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>C-54S-24-OSB</td>
<td>10.29</td>
<td>1.90</td>
<td>4.12</td>
<td>0.28</td>
<td>14.75</td>
</tr>
<tr>
<td>M-54S-24-Deck</td>
<td>28.90</td>
<td>2.34</td>
<td>11.55</td>
<td>0.28</td>
<td>41.07</td>
</tr>
<tr>
<td>C-54S-24-Deck</td>
<td>24.86</td>
<td>1.56</td>
<td>9.95</td>
<td>0.19</td>
<td>51.11</td>
</tr>
<tr>
<td>C-54S-24-Dual</td>
<td>28.03</td>
<td>1.90</td>
<td>11.22</td>
<td>0.22</td>
<td>50.56</td>
</tr>
<tr>
<td>M-97S-48-Deck</td>
<td>25.53</td>
<td>2.06</td>
<td>10.23</td>
<td>0.23</td>
<td>44.34</td>
</tr>
<tr>
<td>C-97S-48-Deck</td>
<td>21.45</td>
<td>1.56</td>
<td>8.58</td>
<td>0.20</td>
<td>42.49</td>
</tr>
<tr>
<td>C-97S-48-Dual</td>
<td>38.82</td>
<td>1.60</td>
<td>15.53</td>
<td>0.27</td>
<td>57.52</td>
</tr>
</tbody>
</table>

* Negligible strength and stiffness

### 5.1.1 Diaphragm Bare Frame System

The test was conducted under monotonic loading to a target displacement less than 1.7 in. to ensure that the specimen remained in the elastic region (Nikolaidou et al. 2016), since the steel frame was reused for the specimen sheathed with OSB and no damage in frame was desired. The test was ended when the actuator reached a displacement equal to 1.3 in. as is shown in Figure . This test was used to test the setup, but also showed that the contribution of the steel frame to diaphragm lateral resistance is negligible. After the test, all the components and connections were checked, and no damage was observed.
5.1.2 Diaphragm Sheathed with OSB

Figure shows the test results for the specimen sheathed with OSB. This specimen was tested under cyclic loading. During the loading of specimen, wood crushing was heard first when the specimen was reaching 0.5 in. of displacement. With increasing displacement, damage of the sheathing connections also increased, leading to separation of the panels, as illustrated in the right of Figure. This separation of the panels was observed before the peak strength. Post peak, the relative displacement among panels was increasing with the damage of the sheathing connections, until the point at which the panels started bearing and crushing on the test rig, as shown in the left of Figure. From the test results, this bearing started at a displacement equal to 3.1 in. and a plateau is observed after this point (see Figure). After the test was completed, failure of the sheathing connections was
recorded. Figure 5.2 shows a schematic representation of three fastener failure modes for all the sheathing connections in the specimen: Edge tear-out, pull-through the OSB, and pull-out of the steel frame.

![Force-displacement response of specimen C-54S-24-OSB](image)

**Figure 5.2:** Force-displacement response of specimen C-54S-24-OSB

![OSB sheathing bearing on test rig (left); OSB separation of the panels (right)](image)

**Figure 5.3:** OSB sheathing bearing on test rig (left); OSB separation of the panels (right)
Figure 5.4: Failure modes of OSB-to-steel frame screw connections

Figure illustrates the damage induced to the steel frame of the specimen. Local lip buckling on some of the joist was observed. The location of this damage corresponded to a seam and it can be attributed to the resultant reaction force caused by the compression zone in the steel frame and the tension zone in the OSB of the specimen. Furthermore, all floor joists presented with permanent lateral torsional deformation.
5.1.3 Diaphragm Sheathed with Steel Deck

Test results for specimens M-54S-24-Deck and C-54S-24-Deck are presented in Figure . Given limited available material, the CFS steel frame was reused for both, monotonic and cyclic test in the specimens 54S-24-Deck and 97S-48-Deck. M-54S-24-Deck was the first specimen tested. The specimen was conservatively loaded to avoid any damage on the steel frame. After the test started and tilting of the side-lap connections was observed, the test was paused and unloaded (see Figure ). Then the specimen was loaded again until a plateau in the force-displacement response, after which the test was stopped. The steel deck was removed, and a new deck was installed shifting by 0.5 in. the screw connections to the steel frame for the specimen C-54S-24-Deck. At the beginning of the cyclic loading of C-54S-24-Deck, the relative “zero” position of the actuator was resetting to the end of a trailing cycle and the response was shifting towards the negative quadrant. The test was stopped, and the loading protocol was fixed. When the test began again, there
was a residual strength of 3kips. In Figure 5.6 for this test there is a difference on the strength in the positive quadrant and the negative quadrant. That difference is an indication that some damage occurred during the first loading. Similar to the OSB specimen, test was considered over when the deck panel started bearing on the rig, a plateau at the end of the response was observed as shown in Figure 5.6.

![Force-displacement response of specimens M-54S-24-Deck and C-54S-24-Deck](image)

Figure 5.6: Force-displacement response of specimens M-54S-24-Deck and C-54S-24-Deck
Failure modes of the steel deck-to-steel frame connections are shown in Figure 5.7 for specimen M-54S-24-Deck and in Figure 5.8 for specimen C-54S-24-Deck. In the monotonic test the damage that was observed in the connections was less severe than in the cyclic test (compare Figure 5.7 and Figure 5.8). A more detailed explanation of the failure propagation is explained in section 5.3 of this chapter.
No failure in the steel frame was observed after the monotonic test. However, after the cyclic test, buckling of the clip-angle connection was identified as shown in Figure 5.8.

Figure 5.8: Failure modes of steel deck-to-steel frame screw connections; specimen M-54S-24-Deck

Figure 5.9: Failure modes of steel deck-to-steel frame screw connections; specimen C-54S-24-Deck

Figure 6.5: Failure modes of steel deck-to-steel frame screw connections; specimen C-54S-24-Deck

Figure 6.6: Failure modes of steel deck-to-steel frame screw connections; specimen C-54S-24-Deck
addition, permanent deformation at the ends of the end-angles and flexural deformation of the floor joist were noticeable, see Figure.

Figure 5.10: Steel frame damage of specimen C-54S-24-Deck

Figure shows the test results for specimens M-97S-48-Deck and C-97S-48-Deck. Specimens C-97S-48-Deck was conducted first and after the test was completed, the steel deck was replaced with a new deck and new connections to the steel frame were shifted 0.5 in. from the existing connection hole in the frame. Similar to specimen C-54S-24-Deck, the test was considered finished when the steel deck started bearing on the rig. This bearing point occurred at the same displacement for both, monotonic and cyclic as shown in Figure, when a plateau starts at the end of the monotonic test. Failure mode of the sheathing connections are represented in Figure and Figure for specimen M-97S-48-Deck and C-97S-48-Deck respectively. For the 97 mil steel frame, no damage was observed in any of the members.
Figure 5.11: Force-displacement response of specimens M-97S-48-Deck and C-97S-48-Deck

Figure 5.12: Failure modes of steel deck-to-steel frame screw connections; specimen M-97S-48-Deck
5.1.4 Diaphragm Sheathed with Dual Skin System

Force-displacement results for specimen C-54S-24-Dual and C-97S-48-Dual are shown in Figure 5.13 and Figure 5.14 respectively. In specimen C-97S-48-Dual, edge tear-out of the connections was evident as is shown in the left of Figure 5.13. Both specimens C-54S-24-Dual and C-97S-48-Dual did not meet minimum required edge distance for the fastener connection of FCB-to-steel deck given the limitation of the deck flute width that was smaller than 1 in. in both specimens. Some relative displacement among the panels was observed, but less severed than in the OSB panels. The test was ended when the dual skin system started bearing on the test rig as shown in the right of Figure 5.13.
Figure 5.14: Force-displacement response of specimen C-54S-24-Dual

Figure 5.15: Force-displacement response of specimen C-97S-48-Dual
Failure modes of the FCB-to-steel deck connections are represented in Figure 5.16 and the failure modes of the deck-to-steel frame connections are shown in Figure 5.17. Damage was observed in all the clip angle connections at both the loading and fixed side of the specimen as illustrated in Figure 5.18. It is believed that this damage on the clip angle connections is represented in the force-displacement results, see Figure 5.19 and Figure 5.20, with the change of slope in the response right before the specimen reached the targeted displacement in a cycle before and after peak strength. That change in stiffness can be attributed to the end of the joist bearing against the web of the ledger.
Figure 5.17: Failure modes of FCB-to-steel deck screw connections; specimen C-97S-48-Dual

Figure 5.18: Failure modes of steel deck-to-steel frame screw connections; specimen C-97S-48-Dual
In the specimen C-54S-24-Dual no edge tear-out was observed but some relative movement among the FCBs was identified as shown in Figure 5.19. This specimen had more flexibility on the sheathing connections given the thinner thickness and test was ended after post-peak when the dual skin system started bearing on the rig and the FCB started crushing as represented on the right of Figure 5.20.

Figure 5.19: Clip-angle and end-angle damage of specimen C-97S-48-Dual

Figure 5.20: FCB relative displacement (left); Dual skin bearing on test rig (right)
After the test was concluded, identification of the failure modes in the sheathing connections was performed as shown in Figure 5.21 and Figure 5.22. Near to the corners of the specimen, fastener pull-out from the steel deck was identified.

Figure 5.21: Failure modes of FCB-to-steel deck screw connections; specimen C-54S-24-Dual

Figure 5.22: Failure modes of steel deck-to-steel frame screw connections; specimen C-54S-24-Dual
Similar to specimen C-97S-48-Dual, damage at the clip-angle connections was identified in addition to permanent deformation at the ends of the end-angles, as shown in Figure.

![Buckling Clip-Angle](image1.png) ![Deformation End-Angle](image2.png)

Figure 5.23: Clip-angle and end-angle damage of specimen C-54S-24-Dual

### 5.2 Relative Performance of The Specimens

Figure shows the backbone curve comparison between all diaphragm specimens results under cyclic loading. The backbone curve is a simplified characterization of the test results that provides the shear behavior of the CFS diaphragm for future reduced-order computational modeling efforts. The construction of the backbone curve follows the maximum strength corresponding to each primary cycle at every increment of the load displacement. Backbone curves terminate at the cycle before the floor sheathing started to bear on the test rig during the loading of the specimens. Appendix C contains individual backbone curves for all specimens.
Figure 5.24: Backbone curve comparison between specimens

Peak strength of the cyclic test results of specimens 54S-24-Deck and 97S-48-Deck were 14% and 16% lower than in their respective monotonic tests. This is attributed to additional flexibility in the screw connections due to the nature of the cyclic loading. 97S-48-Deck specimens showed a reduction in strength of 12% and 14% in comparison to specimens 54S-24-Deck for monotonic and cyclic loading respectively, and an increment of 7% in stiffness in the monotonic test and a decreased of 17% in the cyclic test. This reduction in strength and stiffness are expected since the overall lateral response of steel deck diaphragm is directly affected by the strength of the sheathing connections and specimens with larger joist spacings can significantly reduce the number of sheathing
connections, leading to a reduction in strength. The strongest and stiffer specimens were the diaphragms sheathed with the dual skin system. Specimen C-97S-48-Dual had 26% more stiffness and 45% more strength compared to the same specimen without FCB on top of the steel deck. Specimen C-54S-24-Dual had 11% more strength compared to the same specimen without FCB on top of the steel deck, and contribution in stiffness can be neglectable. Specimens with the dual skin system also showed a significant decrease in damage of the side lap connections and support connections of deck fastener connections to the CFS frame. Observed damage is shown in Figure and Figure for the 97 mil specimen, and Figure and Figure for the 54 mil specimen. However, because the 97 mil dual skin specimen experienced higher loads, the edge connections were also more highly loaded, resulting in bearing damage versus incipient tiling and bearing. A potential benefit of the dual skin system was to reduce the effect of floor joist spacing on the specimen sheathed just with steel deck. Specimen C-97S-48-Dual had 28% and 11% more strength and stiffness compared to specimen C-54S-24-Dual. Finally, specimen sheathed with OSB presented the lowest strength and stiffness among all other specimens. It had a reduction in strength and stiffness of 73% and 74% compared to specimen C-97S-48-Dual which was the most beneficial specimen to the contribution in the shear capacity of the diaphragm.

5.3 Progression of Failure for Diaphragms Sheathed with Steel Deck

For all the specimens tested with steel deck sheathing, it was observed that tilting of the side-lap connections occurred first, followed by bearing of the support connections along the side-laps which includes the corner fastener of interior panels. Figure shows an example of this behavior. Screws heads of the side-lap connections are shown tilted, while
the screw head of the support connection is flush. The steel deck demonstrates localized damage due to fastener bearing. Once the damage of the connections along the side-lap directions increases, more relative displacement is allowed between the panels and load to the support connections increases, propagating damage from the side lap toward the center line of the panel (the neutral axis of the panel). This behavior is represented in Figure . When the first row of fasteners from both sides of the side-lap direction are being damaged, the edge connections start showing damage as well.

Figure 5.25: Example of damage progression; first tilting, then bearing
Figure 5.26: Propagation of damage for the support connections

Figure 5.26 and Figure 5.27 show a sequence of observations through the monotonic and cyclic test for both 54S-24-Deck and 97S-48-Deck specimens. It also includes the three limit states from the contribution to shear strength of the sheathing connections. Descriptions and the sequence of the test observations are found in Table 5.1 and Table 5.2 respectively. Note that the predicted strength of the diaphragm sheathed with steel deck is controlled by the failure of the corner fasteners at interior or edge panels ($S_{nc}$) and this was observed after the tilting of the side-lap connections in all specimens tested in this experimental program. Another important observation is that tilting of side-lap connections started happening at the predicted limit state $S_{ni}$ (strength controlled by connections at interior panels or edge panels) in both monotonic and cyclic tests of both specimens. While the maximum shear strength value of the monotonic tests was closer to the predicted limit state $S_{nc}$ (strength controlled by connections along the edge in an edge panel).
Figure 5.27: Test observation points and predicted strength limit states; specimens 54S-24-Deck

Figure 5.28: Test observation points and predicted strength limit states; specimens 97S-48-Deck
Table 5.2: Observed progression of failure for specimens 54S-24-Deck

<table>
<thead>
<tr>
<th>Loading</th>
<th>Point</th>
<th>Force (kips)</th>
<th>Displacement (in)</th>
<th>Test Observation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monotonic</td>
<td>MP1</td>
<td>20.20</td>
<td>0.73</td>
<td>Incipient tilting side-lap connections</td>
</tr>
<tr>
<td></td>
<td>MP2</td>
<td>23.60</td>
<td>1.09</td>
<td>Incipient bearing support connections along the side-lap</td>
</tr>
<tr>
<td></td>
<td>MP3</td>
<td>28.22</td>
<td>1.88</td>
<td>Incipient bearing edge connections</td>
</tr>
<tr>
<td>Cyclic</td>
<td>CP1</td>
<td>-13.69</td>
<td>-0.53</td>
<td>Incipient tilting side-lap connections</td>
</tr>
<tr>
<td></td>
<td>CP2</td>
<td>16.27</td>
<td>0.47</td>
<td>Incipient tilting side-lap connections</td>
</tr>
<tr>
<td></td>
<td>CP3</td>
<td>20.00</td>
<td>0.67</td>
<td>Tilting side-lap connections</td>
</tr>
<tr>
<td></td>
<td>CP4</td>
<td>-16.04</td>
<td>-0.73</td>
<td>Incipient bearing support connections along the side-lap</td>
</tr>
<tr>
<td></td>
<td>CP5</td>
<td>23.12</td>
<td>0.96</td>
<td>Incipient tilting/bearing edge connections</td>
</tr>
<tr>
<td></td>
<td>CP6</td>
<td>-18.88</td>
<td>-1.02</td>
<td>+ bearing support connections along the side-lap</td>
</tr>
<tr>
<td></td>
<td>CP7</td>
<td>3.95</td>
<td>0.06</td>
<td>+ bearing support connections at the edge connections</td>
</tr>
<tr>
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<td>1.26</td>
<td>Incipient tilting/bearing end-angle connections from side-lap</td>
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<td>CP9</td>
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<td>-1.31</td>
<td>++ bearing support connections along the side-lap</td>
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<tr>
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<td>Incipient bearing support connections from side-lap</td>
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<td>CP11</td>
<td>22.15</td>
<td>1.26</td>
<td>++ bearing/tilting edge connections</td>
</tr>
<tr>
<td></td>
<td>CP12</td>
<td>-3.43</td>
<td>-0.41</td>
<td>++ bearing/tilting end-angle connections from side-lap</td>
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<td>CP13</td>
<td>24.86</td>
<td>1.56</td>
<td>Peak</td>
</tr>
<tr>
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<td>CP14</td>
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<td>0.01</td>
<td>+ bearing support connections from side-lap</td>
</tr>
<tr>
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<td>-1.60</td>
<td>+++ bearing/tilting edge connections</td>
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<td>CP19</td>
<td>9.25</td>
<td>1.69</td>
<td>+ popping out edge connections</td>
</tr>
<tr>
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<td>CP20</td>
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</tr>
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<td>CP21</td>
<td>15.39</td>
<td>2.46</td>
<td>Deck bearing on the rig at the corners</td>
</tr>
</tbody>
</table>
Table 5.3: Observed progression of failure for specimens 97S-48-Deck

<table>
<thead>
<tr>
<th>Loading Point</th>
<th>Force (kips)</th>
<th>Displacement (in)</th>
<th>Test Observation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monotonic</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MP1</td>
<td>20.20</td>
<td>0.73</td>
<td>Incipient tilting side-lap connections</td>
</tr>
<tr>
<td>MP2</td>
<td>23.60</td>
<td>1.09</td>
<td>Incipient bearing support connections along the side-lap</td>
</tr>
<tr>
<td>MP3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MP4</td>
<td></td>
<td></td>
<td>Incipient bearing edge connections</td>
</tr>
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<td>MP5</td>
<td>28.22</td>
<td>1.88</td>
<td>Incipient deck bearing on the rig</td>
</tr>
<tr>
<td>Cyclic</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>CP1</td>
<td>-13.69</td>
<td>-0.53</td>
<td>Incipient tilting side-lap connections</td>
</tr>
<tr>
<td>CP2</td>
<td>16.27</td>
<td>0.47</td>
<td>Incipient bearing support connections along the side-lap</td>
</tr>
<tr>
<td>CP3</td>
<td>20.00</td>
<td>0.67</td>
<td>Incipient bearing corner fastener at interior panels</td>
</tr>
<tr>
<td>CP4</td>
<td>-16.04</td>
<td>-0.73</td>
<td>Incipient bearing support connections from the side-lap</td>
</tr>
<tr>
<td>CP5</td>
<td>23.12</td>
<td>0.96</td>
<td>Peak</td>
</tr>
<tr>
<td>CP6</td>
<td>-18.88</td>
<td>-1.02</td>
<td>Incipient tilting/bearing end-angle connections from side-lap</td>
</tr>
<tr>
<td>CP7</td>
<td>3.95</td>
<td>0.06</td>
<td>Incipient bearing edge connections at loading side</td>
</tr>
<tr>
<td>CP8</td>
<td>24.44</td>
<td>1.26</td>
<td>Incipient bearing edge connections at fixed side</td>
</tr>
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<td>-1.31</td>
<td>+ bearing support connections from the side-lap</td>
</tr>
<tr>
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<td>3.95</td>
<td>-0.14</td>
<td>Incipient edge tear-out end-angle connections from side-lap</td>
</tr>
<tr>
<td>CP11</td>
<td>22.15</td>
<td>1.26</td>
<td>Incipient popping out side-lap connections</td>
</tr>
<tr>
<td>CP12</td>
<td>-3.43</td>
<td>-0.41</td>
<td>+ bearing/tilting end-angle connections from side-lap</td>
</tr>
<tr>
<td>CP13</td>
<td>24.86</td>
<td>1.56</td>
<td>Deck bearing on the rig at the corners</td>
</tr>
<tr>
<td>CP14</td>
<td>-3.02</td>
<td>0.01</td>
<td>Deck bearing on the rig at the corners</td>
</tr>
</tbody>
</table>

5.4 Comparison to Current Design Methods

Figure through Figure compare the test results with the predicted diaphragm strengths and Table summarizes the strength comparison. Experimental-to-predicted strength ratios lower than one represent an overestimation of the predicted strength. Values larger than one represent an underestimation of the predicted strength. Note that the predicted strength of the specimen sheathed with OSB was conservatively taken from a linear interpolation since the AISI S400 provisions do not include strength values for 23/32 in. thick OSB panels. However, this interpolation gave a predicted strength 7% lower than the experimental result as is shown in Figure.
Table 5.4: Comparison experimental vs predicted strengths

<table>
<thead>
<tr>
<th>Specimen</th>
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<th>$V_{n, AISI}$ (kips/ft)</th>
<th>$V_{exp}/V_{n, AISI}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-54S-24-OSB</td>
<td>1.03</td>
<td>0.96 (a)</td>
<td>1.08</td>
</tr>
<tr>
<td>M-54S-24-Deck</td>
<td>2.89</td>
<td>1.67 (b)</td>
<td>1.73</td>
</tr>
<tr>
<td>C-54S-24-Deck</td>
<td>2.49</td>
<td>1.67 (b)</td>
<td>1.49</td>
</tr>
<tr>
<td>C-54S-24-Dual</td>
<td>2.80</td>
<td>2.94 (c)</td>
<td>0.95</td>
</tr>
<tr>
<td>M-97S-48-Deck</td>
<td>2.55</td>
<td>1.35 (b)</td>
<td>1.89</td>
</tr>
<tr>
<td>C-97S-48-Deck</td>
<td>2.15</td>
<td>1.35 (b)</td>
<td>1.59</td>
</tr>
<tr>
<td>C-97S-48-Dual</td>
<td>3.88</td>
<td>3.18 (c)</td>
<td>1.22</td>
</tr>
</tbody>
</table>

(a) Predicted strength per AISI S400; linear interpolation
(b) Predicted strength per AISI S310
(c) Predicted strength per Torabian (2020)

Figure 5.29: Shear strength vs. predicted strength for specimen C-54S-24-OSB

All predicted strengths values for specimens sheathed with steel deck were conservative as is shown in Figure and Figure. The nominal design strength is controlled
by the minimum of the limit states given by the strength of the sheathing connections: $S_{nc}$ (strength controlled by connections at corners of interior panels or edge panels), $S_{ni}$ (strength controlled by connections at interior panels or edge panels) and $S_{ne}$ (strength controlled by connections along the edge in an edge panel), as shown in Figure and Figure. Predicted strength can be as low as 47% of the experimental strength. Ultimately, proposed design methods by Torabian (2020) for the new dual skin system showed an overestimation of 5% for specimen C-54S-24-Dual and an underestimation of 18% for specimen C-97S-48-Dual as is shown in Figure and Figure respectively.

![Graph](image)

**Figure 5.30:** Shear strength vs. predicted strength for specimens M-54S-24-Deck and C-54S-24-Deck
Figure 5.31: Shear strength vs. predicted strength for specimens M-97S-48-Deck and C-97S-48-Deck

Figure 5.32: Shear strength vs. predicted strength for specimen C-54S-24-Dual
5.5 Out-of-Plane Displacement Results

Figure illustrates the locations of the out-of-plane linear potentiometers located on top of the sheathing in all the specimens. Axial forces of the floor joist in the CFS framed of the diaphragm are expected to undergo under compression on one side of the diaphragm and in tension on the opposite side depending on the load direction that is applied from the actuator as is shown in Figure.
Out-of-plane results are presented from Figure 5.34 to Figure 5.35 for all specimens except for the specimen sheathed with OSB due to loss of the data. In general, the response showed an upward displacement (-δ) of the sheathing under the compression zone and a downward
displacement ($\delta$) for the tension zone. In the middle of the diaphragm the displacements were near to zero. In addition, looking at all the responses from the sensors showed an asymmetric response towards the positive loading direction.

![Figure 5.36: Out-of-plane sheathing deformation, specimen: M-54S-24-Deck](image)

For specimen M-54S-24-Deck the steel deck deformed upward under the compression zone as shown in the response of the sensors A1, A2 and A3 in Figure 5.36. At the peak shear strength, A3 moved 0.166 in., A1 0.160 in., and A2 slipped down from the top of the deck flute before reaching the peak. Out-of-plane deformation at the middle of the diaphragm (see B1, B2, B3) was near zero. At peak, sensor B1 moved 0.068 in., B2
0.023 in., and B3 0.016 in. Deck deformed downward under the tension zone of the CFS framing, as shown in the response of sensors C1, C2 and C3. At the peak strength, C3 moved 0.074 in., C2 0.055 in., and C1 0.045 in. Sensors located near to the fixed end of the test rig showed the maximum out-plane displacements.

Figure 5.37: Out-of-plane sheathing deformation, specimen: C-54S-24-Deck

For specimen C-54S-24-Deck the steel deck deformed upward under the compression zone of the CFS framing as shown in the response of the sensors A1, A2 and A3 in a positive load direction, and sensors C1, C2 and C3 in a negative load direction (see Figure ). At the peak shear strength in the positive direction, A1 moved 0.157 in., A3 0.116

88
in., and A2 slipped down from the top of the deck flute before reaching the peak. In the negative direction, C3 moved 0.095 in., C2 0.060 in., and C1 slipped down from the top of the deck before peak. Out-of-plane deformation at the middle of the diaphragm was near to zero (see B1, B2, B3). At peak in the positive load direction, sensor B1 moved 0.044 in., B2 0.024 in., and B3 0.018 in. At peak in the negative load direction, sensor B1 slipped down before peak, sensor B2 moved 0.008 in. and B3 0.007 in. Deck deformed downward under the tension zone as shown in the response of the sensors C1, C2 and C3 in the positive load direction, and sensors A1, A2 and A3 in the negative load direction (Figure). At the peak strength in the positive direction, C3 moved 0.061 in., C2 0.031 in., and C1 slipped down from the top of the deck before peak. A2 slipped down from the top of the deck flute before peak, A1 moved 0.138 in. and A3 moved 0.096 in. Sensors located at opposite corners, A1 and C3, showed the maximum out-plane displacements.
In specimen C-54S-24-Dual, the dual skin system deformed upward under the compression zone as illustrated in the response of sensors A1, A2 and A3 in a positive load direction, and sensors C1, C2 and C3 in a negative load direction (Figure 5.38). At the maximum shear strength in the positive direction, A1 moved 0.164 in., A3 0.155 in., and A2 0.153 in. In the negative direction, C3 moved 0.111 in., C2 0.096 in., and C1 0.072 in. Out-of-plane deformation at the middle of the diaphragm (see B1, B2, B3) was near to zero. At peak in the positive load direction, sensor B1 moved 0.052 in., B2 0.025 in., and B3 0.009 in. At peak in the negative load direction, sensor B1 moved 0.019 in., B2 0.014 in., and B3 0.011 in. The dual skin system deformed downward under the tension zone of compression.
the CFS framing, as shown in the response of sensors C1, C2 and C3 in the positive load direction, and sensors A1, A2 and A3 in the negative load direction (Figure 5.39). At the maximum shear strength in the positive direction, C3 moved 0.015 in., C2 0.006 in., and C1 0.021 in. A1 moved 0.049 in., A3 0.043 in., and A2 0.012 in. Sensors located at opposite corners, A1 and C3, showed the maximum out-plane displacements. The dual skin system reduced in average the downward deformation of the steel deck under the tension zone by 69% compared to the specimen without the FCB on top of the steel deck.

Figure 5.39: Out-of-plane sheathing deformation, specimen: M-97S-48-Deck
In specimen M-97S-48-Deck, the sheathing deformed upward under the compression zone of the CFS framing, as shown in the response of the sensors A1, A2 and A3 in Figure. At the maximum shear strength, A3 moved 0.129 in., A1 0.107 in., and A2 0.108 in. Out-of-plane deformation at the middle of the diaphragm (response of sensors B1, B2, B3) was near to zero. At peak, sensor B3 moved 0.017 in., B2 0.013 in., and B1 0.004 in. Deck deformed downward under the tension zone of the CFS framing, as shown in the response of sensors C1, C2 and C3 (see Figure). At the maximum strength, C3 moved 0.066 in., C2 0.046 in., and C1 0.023 in. Sensors located near to the fixed end of the test rig showed the maximum out-of-plane displacements. On average, out-plane displacement in specimen M-97S-48-Deck was 30% lower compared to specimen M-54S-24-Deck. Note that the deck profile employed for the 4 ft spaced joist was deeper and with a shorter width than the deck for the 2 ft spaced joist.
In the response of specimen C-97S-48-Deck, the steel deck deformed upward under the compression zone of the CFS framing, as shown in the response of the sensors A1, A2 and A3 in a positive load direction, and sensors C1, C2 and C3 in a negative load direction in Figure 5.40. At the maximum shear strength in the positive direction, A1 moved 0.129 in., A2 0.122 in., and A3 0.103 in. In the negative direction, C3 moved 0.056 in., C2 0.050 in., and C1 0.046 in. Out-of-plane deformation at the middle of the diaphragm, (see B1, B2 B3) was near to zero. At the maximum strength in the positive load direction, sensor B1 moved 0.056 in., B2 0.040 in., and B3 0.007 in. In the negative load direction, sensor B1 slipped down before peak, B2 moved 0.003 in., and B3 0.028 in. Deck deformed downward.
under the tension zone of the CFS framing, as shown in the response of sensors C1, C2 and C3 in the positive load direction, and sensors A1, A2 and A3 in the negative load direction (see Figure ). At the peak strength in the positive direction, C3 moved 0.068 in., C2 0.031 in., and C1 0.017 in. A1 moved 0.049 in., A2 0.054 in. and A3 moved 0.080 in. Sensors located at opposite corners, A1 and C3, showed the maximum out-of-plane displacements.

Figure 5.41: Out-of-plane sheathing deformation, specimen: C-97S-48-Dual

In specimen C-97S-48-Dual, the dual skin system deformed upward under the compression zone of the CFS framing, as shown in the response of sensors A1, A2 and A3 for a positive load direction, and sensors C1, C2 and C3 for a negative load direction, see
Figure. At the maximum shear strength in the positive direction, A1 moved 0.172 in., A2 0.171 in., and A3 0.207 in. In the negative direction, C3 moved 0.124 in., C2 0.105 in., and C1 0.086 in. Out-of-plane deformation at the middle of the diaphragm, (see B1, B2 B3 in Figure) was near to zero. At peak in the positive load direction, sensor B1 moved 0.080 in., B2 0.065 in., and B3 0.047 in. At peak in the negative load direction, sensor B1 moved 0.005 in., B2 0.004 in., and B3 0.015 in. Dual deformed downward under the tension zone of the CFS framing, as shown in the response of sensors C1, C2 and C3 in the positive load direction, and sensors A1, A2 and A3 in the negative load direction (see Figure). At the maximum shear strength in the positive direction, C3 moved 0.120 in., C2 0.087 in., and C1 0.063 in. A1 moved 0.069 in., A2 0.081 in., and A3 0.105 in. Sensors located near to the fixed end, A3 and C3, showed the maximum out-plane displacements. The dual skin system increased in average the out-plane deformation by 49% compared to the specimen without the FCB on top the steel deck.
CHAPTER 6
CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary and Conclusions

In an effort to expand the understanding of the lateral response of CFS diaphragms with variable sheathing in CFS ledger framed buildings, and as a benchmark of an experimental test program at the outdoor shake table at UCSD in the CFS-NHERI project, a total of eight 10 ft by 15 ft diaphragms were tested under monotonic and cyclic loading at the Brack Structural Testing Laboratory at the University of Massachusetts Amherst. To do so, an experimental test rig with capabilities to fit 15 ft by 15 ft and 10 ft by 15 ft specimens was design, fabricated, and installed in the structures laboratory. The rig follows a cantilever test approach to characterize the general response of diaphragms and their in-plane strength and stiffness. Two CFS framing systems were employed: 54 mils floor joist (lipped channel) spaced at 2 ft and framed to a 54 mils ledger beam (un-lipped channel) via clip-angle connections, and 97 mils floor joist spaced by 4 ft and framed to a 97 mils ledger beam. Three sheathing materials were used: OSB, steel form deck and a new structural dual skin system: FCB fastened on top of the form deck. Test results showed that the bare CFS framed system, absent of floor sheathing, did not contribute to the shear resistance of the diaphragm. As a result, floor sheathing is necessary to develop any shear resistance. Floor diaphragm sheathed with 23/32 in. OSB developed a maximum shear strength of 1.03 kips/ft. Diaphragm lost strength and stiffness with the failure of the sheathing connections, allowing relative displacement and separation of the panels. Failure modes of the OSB-to-CFS frame connections were tilting and tear-out of the screws at the
edge of the panels, pull-out of the screw from the steel frame and fastener pull-trough the OSB. Lip local buckling was observed on the floor joist at the location of the seams. Diaphragms sheathed with steel form deck developed their maximum shear strength in the monotonic loading, 2.89 kips/ft and 2.55 kips/ft for the 54 mils and 97 mils CFS framed respectively. While in their cyclic loading there was a reduction in strength of 14% and 16%. However, there was an increased in stiffness of 20% for 54 mils specimen and a reduction of 4% for 97 mils. 97 mils specimen showed a reduction in strength of 12% and 14% in comparison with the 54 mils specimen for both monotonic and cyclic loading respectively. While stiffness increased by 7% and decreased by 17%. Deck diaphragm shear strength and stiffness increased by 59% and 71% for 54 mils steel framed, and 52% and 65% for 97 mils in comparison with the OSB specimen. Response of deck diaphragms was controlled by the failure of the sheathing connections, deck-to-steel frame. Tilting of the side-lap connections was observed first, followed by bearing of the support connections along the side-lap and then bearing at the edge connections and support connections from the side-lap. Shear strength and stiffness decreased after failure of the edge connections. The most beneficial configuration for the lateral response of the diaphragm was the 97 mils steel framed sheathed with the new dual skin system. The maximum developed shear strength was 3.88 kips/ft, 73% and 74% higher than OSB in strength and stiffness, and 45% and 26% higher than just sheathed with steel deck. In comparison with the dual skin system on top of the 54 mils steel framed, shear strength increased by 28% and stiffness by 11%. For 54 mils dual skin system the maximum shear strength was 2.80 kips/ft. Fastener edge tear-out, screw tilting and fastener pull-out of steel deck were the observed failure modes of the FCB-to-steel deck connections. In the same system, damage of the steel deck-to-
steel frame connections was reduced for the side-lap connections and support connections. While for the edge connections was more severe. In addition, buckling of the clip-angle was observed in both framing systems.

### 6.2 Design Recommendations

Currently in AISI S400 Table F2.4-1 (2015) does not provide a nominal shear strength value for 23/32 in. thick structural wood panels. In this research, a 23/32 in. OSB specimen was tested and its predicted strength was obtained by interpolation. The experimental to predicted strength ratio was 1.08. This linear interpolation method is adequate for strength prediction.

The shear design of diaphragms sheathed with steel deck was found to be conservative. Diaphragm design was controlled by the connections at corners of interior panels or edge panels, $S_{nc}$. Experimental to predicted strength ratios for 54 mils steel framed were 1.73 for monotonic and 1.49 for cyclic loading, and for 97 mils, 1.89 and 1.59 respectively. This demonstrated that in AISI S310 (2016) the predicted strength is underestimated. At the predicted limit state of connections at interior panels or edge panels, $S_{ni}$ matched with the first observed failure mode corresponding to tilting of the side-lap connections in both monotonic and cyclic loading. However, the specimens continued to gain strength, without loss of stiffness, signifying force redistribution capability which should be considered in design. The predicted limit state for connections along the edge in edge panels, $S_{ne}$ is closer (in average 6% of difference) to the maximum shear strength of monotonic loading but exceeds cyclic peak strength by 19% for 54 mils and 11% for 97 mils. It is recommended to not include the limit state $S_{ne}$ in design as corner fastener failure occurs later in the response, after $S_{ni}$ is attained. However, considering $S_{ni}$ as the controlling
strength can still lead to a conservative value. An alternative method to predict the shear strength of the diaphragm based on the test results presented herein is to determine the individual fastener shear capacity per AISI S100 (2016) and multiply the connection capacity by the number of fasteners in which damage was observed from the side-lap to the center line of deck panel, as shown in Figure 6.1 and Figure 6.2. However, the prediction to identify which fasteners will be damage needs to be studied. Example of this method is shown below.

For 97mils

Figure 6.1: Proposed force distribution for 97 mils specimen based on the observed damage of the sheathing connections
1. Screw shear strength is calculated per AISI S100-16 section J4.3

   For support and edge connections, $P_{n\#12} = 1.25 \text{ kips}$

   For side-lap connections, $P_{n\#10} = 0.61 \text{ kips}$

2. Shear strength of the diaphragm is calculated by multiplying the screw shear capacity with the number of fasteners in which damage was observed. In this case, 3 fasteners from the side-lap of the steel deck in the support connections (total of 6 supports), and all side-lap connections (total of 17 fasteners).

   $$\sum F_x = (18 \times 1.25) + (17 \times 0.61) - S_nL = 0$$

   $$S_nL = 32.87 \text{ kips}$$

   $$S_n = \frac{32.87 \text{ kips}}{15 \text{ ft}} = 2.19 \text{ kips/ft}$$

3. Experimental (cyclic) to predicted strength ratio

   $$\frac{2.15 \text{ kips/ft}}{2.19 \text{ kips/ft}} = 0.98$$
For 54 mils

Figure 6.2: Proposed force distribution for 54 mils specimen based on the observed damage of the sheathing connections

1. Screw shear strength is calculated per AISI S100-16 section J4.3

For support and edge connections, \( P_{n#12} = 1.25 \text{ kips} \)

For side-lap connections, \( P_{n#10} = 0.61 \text{ kips} \)
2. Shear strength of the diaphragm is calculated by multiplying the screw shear capacity with the number of fasteners in which damage was observed. In this case, 2 fasteners from the side-lap of the steel deck in the support connections (total of 10 supports), and all side-lap connections (total of 14 fasteners).

\[ \sum F_x = (20 \times 1.25) + (14 \times 0.61) - S_nL = 0 \]

\[ S_nL = 33.54 \text{ kips} \]

\[ S_n = \frac{33.54 \text{ kips}}{15 \text{ ft}} = 2.24 \text{ kips/ft} \]

3. Experimental (cyclic) to predicted strength ratio

\[ \frac{2.49 \text{ kips/ft}}{2.24 \text{ kips/ft}} = 1.11 \]

Using this method for steel deck diaphragms, the strength predictions are markedly improved, to within +/- 10%, compared with current predictive methods, which underestimated capacity by as much as 40%. However, determining fastener engagement remains a critical future work item to advance this methodology.

The proposed predicted method by Torabian (2020) presented experimental to predicted strength ratios of 0.95 for 54 mils steel framed and 1.22 for 97 mils. Indicating promise of both the dual skin system, and the predicted method. Results showed the potential for the new dual skin system for stronger and stiffer CFS diaphragms.
6.3 Future Work

The tests results from this experimental study will provide validation of and calibration for a high-fidelity FEM of CFS diaphragms, that can expand the understanding of the failure mechanism through the system. In addition, the effect of different structural parameters such as dimensions of CFS sections, fasteners configurations, and other sheathing materials on the lateral response can be explored. The gravity response of the systems tested herein needs to be investigated, as well as serviceability limits states, especially for vibration and acoustics.

More validation for the proposed method to predict the shear strength of diaphragms sheathed with steel deck is needed. It was found that the strength predictions were improved, to within +/- 10%, compared with current predictive methods, which underestimated capacity by as much as 40%. The method involves the number of fasteners that were observed to be damage from the side-lap to the center line of a deck panel. In the 97 mils framing 3 support fasteners from the side-lap are considered into the calculations, while for the 54 mils only two support fasteners are involved, in addition to the side-lap fasteners. However, determining fastener engagement remains a critical future work item to advance this methodology.

Shake table testing of isolated diaphragms is currently scheduled for 2023-2024. This work will dictate specimen structural system and provide key performance metrics to enable behavior predictions.
APPENDIX A

DESIGN CALCULATIONS FOR COLD-FORMED STEEL DIAPHRAGM

SHEATHED WITH STEEL DECK

This appendix presents the details of the design calculations used in the diaphragms sheathed with steel deck. Design provisions per AISI S310 and S100 (2016) were considered. In addition, calculations (excel sheet) to determine the shear strength of the dual skin system: fiber cement board (FCB) fastened on top of the steel deck are presented at the end of this appendix.

A.1 Design for Specimen 54S-24-Deck
1. Screw shear strength is calculated per AISI S100-16 section J4.3

\[ P_{n\#12} = 1.25 \text{ kips} \]
\[ P_{n\#10} = 0.61 \text{ kips} \]

Parameters to determine the screw shear strength,

\[ d_{\#12} = 0.216 \text{ in} \quad t_{deck} = 0.0239 \text{ in} \quad F_{udeck} = 90 \text{ ksi} \]
\[ d_{\#10} = 0.190 \text{ in} \quad t_{joist} = 0.0566 \text{ in} \quad F_{ujoist} = 65 \text{ ksi} \]

2. Connection strength reduction factor at corner fastener is calculated per AISI S310-16 section D1

\[ \lambda = 0.97 \]

Parameters to determine the connection strength reduction factor,

\[ D_d = 0.5625 \text{ in} \quad t_{deck} = 0.0239 \text{ in} \quad L_y = 2 \text{ ft} \]

3. \( S_n \) for interior panel

\[
\sum M_{C,L} = 0 = (2 \times 1.21w) + (8 \times 1.25w) + \left(10 \times 0.89 \left(\frac{5w}{7}\right)\right) \\
+ \left(10 \times 0.54 \left(\frac{3w}{7}\right)\right) + \left(10 \times 0.18 \left(\frac{w}{7}\right)\right) + (14 \times 0.61w) - S_n w L
\]
\[ S_n L = 29.89 \text{ kips} \]

\[
S_{ni} = \frac{29.89 \text{ kips}}{15 \text{ ft}} = 1.99 \text{ kips/ft}
\]

4. \( S_{nc} \) for interior panel

\[
S_{nc} = \sqrt{N^2 \beta^2 \frac{P_{nf}}{L^2 N^2 + \beta^2}} = \sqrt{\frac{(2.4^2)(24.06^2)}{(15^2)(2.4^2) + 24.06^2}} (1.25) = 1.68 \text{ kips/ft}
\]

\[ N = \frac{(1/2 + 1 + 1 + 1 + 1 + 1 + 1/2)/(35 \text{ in}/12^\prime\prime)}{35/8 \text{ Pattern}} = 2.4 \text{ fasteners per foot} \]

\[ L = 15 \text{ ft} \]

\[ \beta = n_s \alpha_s + 2n_p \alpha_p^2 + 4\alpha_s^2 = 24.06 \]

\[ n_s = 14 \text{ side-lap connections} \]

\[ \alpha_s = \frac{P_{ns}}{P_{nf}} = \frac{0.61 \text{ kips}}{1.25 \text{ kips}} = 0.49 \]

\[ n_p = 8 \text{ interior supports} \]

\[ \alpha_p^2 = \frac{1}{w^2} \sum x_p^2 = \frac{1}{35^2} (2.5^2 + 7.5^2 + 12.5^2 + 17.5^2 + 2.5^2 + 7.5^2 + 12.5^2 + 17.5^2) = \]
\[ \alpha_p^2 = 0.86 \]

\[ \alpha_e^2 = \alpha_p^2 = 0.86 \; ; \text{same fastener pattern at interior and exterior support} \]

Solving from calculated \( S_{nc} \)

\[ Q_v = \frac{1.68 \text{kips/ft}}{2A \text{fasteners/ft}} = 0.70 \text{kips/fastener} \]

\[ Q_N = \sqrt{1.25^2 - 0.70^2} = 1.04 \text{kips/fastener} \]

\[ Q_S = \alpha_s Q_N = 0.51 \text{kips} \]

\[ \sum M_{C,L} = 0 = (10 \times 1.04w) + \left( 10 \times 0.74 \left( \frac{5w}{7} \right) \right) + \left( 10 \times 0.45 \left( \frac{3w}{7} \right) \right) \]

\[ + \left( 10 \times 0.15 \left( \frac{w}{7} \right) \right) + (14 \times 0.51w) - S_n w L \]

\[ S_n L = 24.97 \text{kips} \]
$$S_{nc} = \frac{24.97 \text{ kips}}{15 \text{ ft}} = 1.67 \text{ kips/ft}$$

$$S_{nc} = 1.67 \text{ kips/ft}$$

5. $S_{ne}$ for exterior panel

$$\sum F_x = (10 \times 1.25) + (14 \times 1.25) + (10 \times 0.89) + (10 \times 0.54) + (10 \times 0.18) - S_{nL} = 0$$

$$S_{nL} = 46.10 \text{ kips}$$

$$S_{ne} = \frac{46.10 \text{ kips}}{15 \text{ ft}} = 3.07 \text{ kips/ft}$$

$$S_{ne} = 3.07 \text{ kips/ft}$$

Diaphragm shear strength is determined by the minimum strength of $S_{nc}$, $S_{ni}$, and $S_{ne}$. $S_{nc}$ governs the design for specimens M-54S-24-Deck and C-54S-24-Deck. The Diaphragm shear strength is equal to 1.67 kips/ft.
A.2 Design for Specimen 97S-48-Deck

![Diagram of the specimen design](image)

**Support Connection**: Pattern: 27/7
- Screws: #12

**Edge Connection**: Pattern: 8" o.c.
- Screws: #12

**Side-Lap Connection**: Pattern: 8" o.c.
- Screws: #10

**Dimensions**:
- **L**: 1.25
- **S**: 0.6
- **W**: 1.2
- **C.L.**: 1.125

**Pattern**:
- **27/7 Pattern**
- **C.L.**
- **1/2**
- **1**
- **1/2**

**Annotations**:
- **Load direction**
- **Jost direction**

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Screw shear strength is calculated per AISI S100-16 section J4.3

\[ P_{n\#12} = 1.25 \text{ kips} \]
\[ P_{n\#10} = 0.61 \text{ kips} \]

1. \( S_{ni} \) for interior panel

\[ S_{ni} = 1.46 \text{ kips/ft} \]

\[ S_{ni} = [2A(\lambda - 1) + \beta] \frac{P_{nf}}{L} = [2(0.90 - 1) + 17.69] \frac{1.25 \text{ kips}}{15 \text{ ft}} = 1.46 \text{ kips/ft} \]

\[ A = 1 \]
\[ \lambda = 0.90 \]
\[ \beta = n_s \alpha_s + 2n_p \alpha_p^2 + 4\alpha_e^2 = 17.69 \]

\[ n_s = 17 \text{ side-lap connections} \]
\[ \alpha_s = \frac{P_{ns}}{P_{nf}} = \frac{0.61 \text{ kips}}{1.25 \text{ kips}} = 0.49 \]

\[ n_p = 4 \text{ interior supports} \]
\[ \alpha_p^2 = \frac{1}{w^2} \sum x_p^2 = \frac{1}{27^2}(4.5^2 + 9^2 + 27^2 + 13.5^2 + 4.5^2 + 9^2 + 13.5^2) = 0.78 \]
\[ \alpha_e^2 = \alpha_p^2 = 0.78 \text{; same fastener pattern at interior and exterior support} \]

2. \( S_{nc} \) for interior panel

\[ S_{nc} = 1.35 \text{ kips/ft} \]
\[ S_{nc} = \sqrt{\frac{N^2 \beta^2}{L^2 N^2 + \beta^2}} \frac{P_{nf}}{P_{nf}} = \sqrt{\frac{(2.7^2)(17.69^2)}{(15^2)(2.7^2) + 17.69^2}(1.25)} = 1.35 \text{ kips/ft} \]

\[ N = (1/2 + 1 + 1 + 1 + 1 + 1/2)/(27 \text{ in./12''}) = 2.7 \text{ fasteners per foot} \]

\[ L = 15 \text{ ft} \]

\[ \beta = 17.69 \]

3. \( S_{nc} \) for exterior panel

\[ S_{ne} = 2.42 \text{ kips/ft} \]

\[ S_{ne} = \left[ (2\alpha_1 + n_p\alpha_2)P_{nf} + n_eP_{nfs} \right]/L = \left[ (2(2) + 4(2))1.25 + 17(1.25) \right]/15 \]
\[ = 2.42 \text{ kips/ft} \]

\[ \alpha_1 = \frac{1}{w_e} \sum x_{ee} = \frac{1}{27}(4.5 + 9 + 13.5 + 4.5 + 9 + 13.5) = 2 \]

\[ \alpha_2 = \alpha_1 = 2; \text{ same fastener pattern at interior and exterior support} \]

Diaphragm shear strength is determined by the minimum strength of \( S_{nc}, S_{ni}, \) and \( S_{ne} \). \( S_{ne} \) governs the design for specimens M-97S-48-Deck and C-97S-48-Deck. The Diaphragm shear strength is equal to 1.35 kips/ft.
A.3 Design for Specimen 54S-24-Dual

1. Steel Deck

Deck

- **Yield Stress, $F_y$**: 80 ksi O.K. (c)
- **Tensile Strength, $F_u$**: 62 ksi O.K. (c)
- **Depth, $D_d$**: 0.5625 in O.K. (a)
- **Thickness, $t$**: 0.0239 in O.K. (b)
- **Top flat width, $t$**: 0.8125 in
- **Bottom flat width, $2e$**: 0.8125 in
- **Moment of Inertia, $I_z$**: 0.019 in$^4$/ft
- **Modulus of Elasticity, $E$**: 29500 ksi

- **Panel Length, $L$**: 15 ft
- **Cover width, $w$**: 35 in FCB, $w$ 15 ft
- **Pitch, $d$**: 2.5 in O.K. (d)
- **Web flat width, $w$**: 0.71 in

2. Floor Joist

CFS Joist

<table>
<thead>
<tr>
<th>Yield Stress, $F_y$</th>
<th>50 ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile Strength, $F_u$</td>
<td>65 ksi</td>
</tr>
<tr>
<td>Thickness, $t$</td>
<td>0.0566 in</td>
</tr>
</tbody>
</table>

- **Spacing, $L_w$**: 2 ft

3. Ledger Beam (Edge Support)

CFS Ledger

<table>
<thead>
<tr>
<th>Yield Stress, $F_y$</th>
<th>50 ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile Strength, $F_u$</td>
<td>65 ksi</td>
</tr>
<tr>
<td>Thickness, $t$</td>
<td>0.0556 in</td>
</tr>
</tbody>
</table>
### 4. Connections

**Support Connection**

#### 4.1. Interior Support

<table>
<thead>
<tr>
<th>Fastener Pattern</th>
<th>#12 Screw</th>
</tr>
</thead>
</table>

| Number of Screws | 8         |
| Diameter, $d$    | 0.216 in  |
| Shear Strength of screw, $P_{sxx}$ | 3.11 kips |
| Tensile Strength of screw, $P_{tst}$ | 4.98 kips |

#### 4.2. Exterior Support

<table>
<thead>
<tr>
<th>Fastener Pattern</th>
<th>35/8</th>
</tr>
</thead>
</table>

| Number of Screws | 8    |
| Diameter, $d$    | 0.216 in |
| Shear Strength of screw, $P_{sxx}$ | 3.11 kips |
| Tensile Strength of screw, $P_{tst}$ | 4.98 kips |

#### 4.3. Side-Lap Connection

<table>
<thead>
<tr>
<th>Fastener Pattern</th>
<th>#10 Screw</th>
</tr>
</thead>
</table>

| Diameter, $d$    | 0.190 in  |
| Shear Strength of screw, $P_{sxx}$ | 1.62 kips |

| Spacing (Between Supports) | 8 in |

#### 4.4. Edge Connection

<table>
<thead>
<tr>
<th>Fastener Pattern</th>
<th>#12 Screw</th>
</tr>
</thead>
</table>

| Diameter, $d$    | 0.216 in  |
| Shear Strength of screw, $P_{sxx}$ | 3.11 kips |

<p>| Spacing (Between Supports) | 8 in |</p>
<table>
<thead>
<tr>
<th>Parameters for $S_{pe}$</th>
<th>Description</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A$</td>
<td>1 Number support screws at side-lap at deck ends</td>
<td></td>
</tr>
<tr>
<td>$\lambda$</td>
<td>0.97 O.K. Connection strength reduction factor</td>
<td>Eq. (D1-4a)</td>
</tr>
<tr>
<td>$\beta$</td>
<td>79.09 Factor defining screw interaction</td>
<td>Eq. (D1-5)</td>
</tr>
<tr>
<td>$n_s$</td>
<td>14 Number of side-lap screws along the panel length, L</td>
<td></td>
</tr>
<tr>
<td>$\alpha_1$</td>
<td>1.00 Connection strength ratio (no side lap connection)</td>
<td></td>
</tr>
<tr>
<td>$n_p$</td>
<td>8 Number of interior supports</td>
<td></td>
</tr>
<tr>
<td>$\alpha_2$</td>
<td>3.25 Analogous section modulus interior support connection</td>
<td>Eq. (D1-7)</td>
</tr>
<tr>
<td>$\alpha_4$</td>
<td>3.25 Analogous section modulus exterior support connection</td>
<td>Eq. (D1-8)</td>
</tr>
<tr>
<td>$L$</td>
<td>15 ft Panel length</td>
<td>Eq. (D1-9)</td>
</tr>
<tr>
<td>$N$</td>
<td>2.4 screw/ft Number of screws into support per ft along deck ends</td>
<td></td>
</tr>
<tr>
<td>$\alpha_1$</td>
<td>9.50 Measure of exterior support fastener distribution at edge panel</td>
<td>Eq. (D1-10)</td>
</tr>
<tr>
<td>$\alpha_2$</td>
<td>9.50 Measure of interior support fastener distribution at edge panel</td>
<td>Eq. (D1-11)</td>
</tr>
<tr>
<td>$n_e$</td>
<td>14 Number of edge support connections along the panel length, L</td>
<td></td>
</tr>
<tr>
<td>$Q_s$</td>
<td>1.25 kips Support Connection Strength</td>
<td></td>
</tr>
</tbody>
</table>

**End Fastener Limitation**

$S_{ne} = 9.12$ klf

**Corner Fastener Buckling**

$S_{ni} = 6.61$ klf

**Corner Fastener Buckling**

$S_{nc} = 2.94$ klf

**5. Available Dual-Skin Diaphragm Strength**

$S_n = \min(S_{ne}, S_{ni}, S_{nc})$ *Torabian proposed method*

$S_n = \frac{S_n}{\Omega}$ ($\Omega = 2.5$)

$S_n = 2.94$ klf
A.4 Design for Specimen 97S-48-Dual

1. Steel Deck

Deck: 1" x 4-1/2" Form Deck (24 gage)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield Stress, $F_Y$</td>
<td>80 ksi</td>
<td>O.K.</td>
</tr>
<tr>
<td>Tensile Strength, $F_u$</td>
<td>62 ksi</td>
<td>O.K.</td>
</tr>
<tr>
<td>Depth, $D_d$</td>
<td>1 in</td>
<td>O.K.</td>
</tr>
<tr>
<td>Thickness, t</td>
<td>0.0239 in</td>
<td>O.K.</td>
</tr>
<tr>
<td>Top flat width, $t_f$</td>
<td>1.5 in</td>
<td></td>
</tr>
<tr>
<td>Bottom flat width, $t_e$</td>
<td>1.5 in</td>
<td></td>
</tr>
<tr>
<td>Moment of Inertia, $I_{yz}$</td>
<td>0.055 in$^4$/ft</td>
<td></td>
</tr>
<tr>
<td>Modulus of Elasticity, $E$</td>
<td>29500 ksi</td>
<td></td>
</tr>
<tr>
<td>Panel Length, $L$</td>
<td>15 ft</td>
<td></td>
</tr>
<tr>
<td>Cover width, $w$</td>
<td>27 in</td>
<td>FCB, w</td>
</tr>
<tr>
<td>Pitch, $d$</td>
<td>4.5 in</td>
<td>O.K.</td>
</tr>
<tr>
<td>Web flat width, $w$</td>
<td>1.25 in</td>
<td></td>
</tr>
</tbody>
</table>

2. Floor Joist

CFS Joist: 12005200-97

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield Stress, $F_Y$</td>
<td>50 ksi</td>
</tr>
<tr>
<td>Tensile Strength, $F_u$</td>
<td>65 ksi</td>
</tr>
<tr>
<td>Thickness, t</td>
<td>0.1017 in</td>
</tr>
<tr>
<td>Spacing, $L_s$</td>
<td>4 ft</td>
</tr>
</tbody>
</table>

3. Ledger Beam (Edge Support)

CFS Ledger: 12001200-97

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield Stress, $F_Y$</td>
<td>50 ksi</td>
</tr>
<tr>
<td>Tensile Strength, $F_u$</td>
<td>65 ksi</td>
</tr>
<tr>
<td>Thickness, t</td>
<td>0.1017 in</td>
</tr>
</tbody>
</table>
4. Connections

Support Connection

4.1. Interior Support

Fastener Pattern: #12 Screw

Number of Screws: 7
Diameter, d: 0.216 in
Shear Strength of screw, \( P_{s,ss} \): 3.11 kips
Tensile Strength of screw, \( P_{t,ss} \): 4.98 kips

4.2. Exterior Support

Fastener Pattern: #12 Screw

Number of Screws: 7
Diameter, d: 0.216 in
Shear Strength of screw, \( P_{s,ss} \): 3.11 kips
Tensile Strength of screw, \( P_{t,ss} \): 4.98 kips

4.3. Side-Lap Connection

Fastener Pattern: #10 Screw

Diameter, d: 0.190 in
Shear Strength of screw, \( P_{s,ss} \): 1.62 kips
Spacing (Between Supports): 8 in

4.4. Edge Connection

Fastener Pattern: #12 Screw

Diameter, d: 0.216 in
Shear Strength of screw, \( P_{s,ss} \): 3.11 kips
Spacing (Between Supports): 8 in
### Parameters for $S_{vc}$

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Formula</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A$</td>
<td>1 Number support screws at side-lap at deck ends</td>
<td>Eq. (D1-4a)</td>
</tr>
<tr>
<td>$\lambda$</td>
<td>0.89 O.K. Connection strength reduction factor</td>
<td>Eq. (D1-5)</td>
</tr>
<tr>
<td>$\beta$</td>
<td>60.05 Factor defining screw interaction</td>
<td>Eq. (D1-5)</td>
</tr>
<tr>
<td>$n_z$</td>
<td>17 Number of side-lap screws along the panel length, L</td>
<td>Eq. (D1-7)</td>
</tr>
<tr>
<td>$\alpha_z$</td>
<td>1.00 Connection strength ratio (no side lap connection)</td>
<td>Eq. (D1-7)</td>
</tr>
<tr>
<td>$n_a$</td>
<td>4 Number of interior supports</td>
<td>Eq. (D1-7)</td>
</tr>
<tr>
<td>$\alpha_s^2$</td>
<td>3.59 Analogous section modulus interior support connection</td>
<td>Eq. (D1-7)</td>
</tr>
<tr>
<td>$\alpha_e^2$</td>
<td>3.59 Analogous section modulus exterior support connection</td>
<td>Eq. (D1-7)</td>
</tr>
<tr>
<td>$L$</td>
<td>15 ft Panel length</td>
<td>Eq. (D1-7)</td>
</tr>
<tr>
<td>$N$</td>
<td>2.666667 screw/ft Number of screws into support per ft along deck ends</td>
<td>Eq. (D1-7)</td>
</tr>
<tr>
<td>$a_1$</td>
<td>10.50 Measure of exterior support fastener distribution at edge panel</td>
<td>Eq. (D1-10)</td>
</tr>
<tr>
<td>$a_2$</td>
<td>10.50 Measure of interior support fastener distribution at edge panel</td>
<td>Eq. (D1-11)</td>
</tr>
<tr>
<td>$n_e$</td>
<td>17 Number of edge support connections along the panel length, L</td>
<td>Eq. (D1-11)</td>
</tr>
<tr>
<td>$Q_e$</td>
<td>1.25 kips Support Connection Strength</td>
<td>Eq. (D1-11)</td>
</tr>
</tbody>
</table>

#### End Fastener Limitation

$S_{pe} = 6.69$ klf

#### Corner Fastener Buckling

$S_{pi} = 5.00$ klf

#### Corner Fastener Buckling

$S_{pc} = 3.18$ klf

### Available Dual-Skin Diaphragm Strength

$S_{d} = \min(S_{pe}, S_{pi}, S_{pc})$ *Torabian proposed method*

$S_d = S_n / \Omega$  ($\Omega = 2.5$)

$S_d = 3.18$ klf
APPENDIX B

SENSOR RESPONSE FOR ALL TEST SPECIMENS

This appendix presents the sensor response for all the test specimens. Note that data for the specimen sheathed with OSB was lost due to the space of the memory in the computer. Figures contain a photo illustrative to the specimen, basic information of the specimen, force-displacement curve, out-of-plane response, response of the sensors attached to the loading beam and response of the sensors attached to the fixed beam. For sensor schematics see section 5.5 in Chapter 5.
Figure B.1: Sensor response for specimen M-54S-24-Deck
Figure B.2: Sensor response for specimen C-54S-24-Deck
Specimen: C-54S-24-Dual
Loading: Cyclic
Floor joist: 1200S200-54
Joist spacing: 2 ft
Sheathing: 24 gage 9/16” form deck & 3/4” FCB
Sheathing connections:
- Support: #12 screws 35/8 pattern
- Side-lap: #10 screws 8” o.c.
- Edge: #12 screws 8” o.c.
- FCB: #10 6”/12” & 5”/20” pattern

Figure B.3: Sensor response for specimen C-54S-24-Dual
Figure B.4: Sensor response for specimen M-97S-48-Deck
Figure B.5: Sensor response for specimen C-97S-48-Deck
Figure B.6: Sensor response for specimen C-97S-48-Dual
Figure B.7: Sensor response for specimen C-54S-24-OSB

Specimen: C-54S-24-OSB
Loading: Cyclic
Floor joist: 1200S200-54
Joist spacing: 2 ft
Sheathing: 23/32” OSB
Sheathing connections:
- OSB: #10 screws 6”/12” pattern

No data
APPENDIX C

BACKBONE CURVE FOR CYCLIC TEST RESULTS

Figure C.1: Backbone curve of specimen C-54S-24-Deck
Figure C.2: Backbone curve of specimen C-54S-24-Dual
Figure C.3: Backbone curve of specimen C-97S-48-Deck
Figure C.4: Backbone curve of specimen C-97S-48-Dual
Figure C.5: Backbone curve of specimen C-54S-24-OSB
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Nunna, R.V. (2011). “Buckling of profiled steel diaphragms”, Center for Cold-Formed Steel Structures Library. 163.


Yu, C. (2010). “Shear resistance of cold-formed steel framed shear walls with 0.686 mm, 0.762 mm, and 0.838 mm steel sheet sheathing”, Engineering Structures, 32(6):1522–1529.


Zhao Y. (2002). “Cyclic performance of cold-formed steel stud shear walls”, Master dissertation, Department of Civil Engineering and Applied Mechanics, McGill University, Montreal, CA.