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INNOVATIVE RESIDENTIAL FLOOR CONSTRUCTION:

HORIZONTAL DIAPHRAGM VALUES FOR COLD-FORMED STEEL FRAMING

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Prepared for

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Disclaimer

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1.0 Introduction

For centuries home builders in the United States have made wood their material of choice because of its satisfactory performance, abundant supply, and relatively low cost. However, over the past several years, lumber prices have experienced unpredictable price fluctuations that affect affordability. Builders have also voiced concerns with lumber quality. Consequently, builders and other providers of affordable housing are seeking functional and competitive alternative building materials and methods.

Use of cold-formed steel framing in the residential market has increased over the past several years because of previous cooperative efforts by HUD and industry to implement cost-effective alternative materials and methods. However, its use is still very limited, partly because steel is just beginning a process of being integrated into the conventional framing systems of homes. Properly focusing this process is crucial to the reasonable use of this technology in the home building industry.

Cold-formed steel is particularly suitable for residential floor framing systems. Conventional floor systems are usually constructed of more expensive, older growth lumber to meet the loading and span requirements. In addition, floor systems are often excluded from energy efficiency considerations since they are mostly contained within the building thermal envelope. Therefore, cold-formed steel (CFS) floor framing represents an effective utilization of this material within the context of traditional home building practice in the United States. However, there are some technical barriers that unnecessarily limit the use CFS residential floor construction such as the lack of horizontal diaphragm shear values used to design for wind and earthquake forces.

This report establishes appropriate design values for CFS horizontal diaphragms in support of safe and affordable housing design. Very few, if any, horizontal diaphragm tests have been performed on CFS floor systems. The need for diaphragm shear values has been primarily identified in instances where buildings are required to be engineered for higher risk regions of the United States (i.e. hurricane-prone coastlines and earthquake-prone areas). Furthermore, the steel joists used to construct the floor diaphragms contain large pre-formed holes to support an effort to develop a highly functional CFS floor system that meets both engineering and certain constructability requirements such as the routing of ductwork and large sanitary drain pipes. These innovative CFS joists provide similar structural performance in resisting gravity loads to those without holes as confirmed in a separate study [1].

2.0 Background and Literature Review

Function of a Horizontal Diaphragm

When used in light-frame construction such as homes, cold-formed steel (CFS) floor joists are often overlaid with plywood or oriented-strand-board (OSB) sheathing. The sheathing serves the primary function of a subfloor surface while transmitting live, dead, and construction loads into the structure. Roof sheathing performs a similar function in roof construction. Since the sheathing is typically fastened (with screws, pins, etc.) to the steel joists, it also forms shear-resistant system known as a “horizontal diaphragm”, which is used to resist in-plane forces arising from wind or earthquake loads. The diaphragm’s ability to resist in-plane loads is dependent on its stiffness and ultimate shear strength.

A diaphragm is a horizontal structural assembly that acts in a manner analogous to a deep beam, where the panels act as a “web” resisting shear and the diaphragm edge members perform the function of “flanges” to resist bending stresses. These edge members are commonly called chords in diaphragm design. Cold-formed steel floor assemblies are relatively new shear resisting systems for residential buildings in the United States. However, the use of plywood or oriented-strand-board (OSB) as shear-resistance material is not new, and has been widely accepted in the design of wood-framed roof and floor diaphragms for housing.

Review of Existing Test Data

A literature review of similar work was performed prior to testing. Little information pertaining to the design values of CFS horizontal diaphragms was found in the literature. Extensive data exist for roof diaphragms constructed with purlins and steel decks [2][3][4][5]. Shear values for vertical diaphragms (shear walls) can also be readily found [6][7].

The use of structural wood panel sheathing as a shear-resistant material for wood floors has been well established in previous testing by the American Plywood Association (APA) [8]. APA developed empirical and mechanics-based equations to predict the strength, stiffness, and deflections of wood-frame diaphragms [8]. In addition to the research done at APA, structural wood panel diaphragms have been tested at other laboratories including the Forest Products Laboratory (FPL) and Oregon State University. Although dated, a comprehensive listing of wood and plywood diaphragm tests has been published by the American Society of Civil Engineers [9].

Table 1 summarizes the design unit shear values (in pounds per foot) for unblocked horizontal diaphragms using structural wood panels and dimensional lumber. Unblocked diaphragms refer to situations where blocking is not provided at sheathing joints perpendicular to the floor joists.

Table 1
Design Unit Shear Values for Unblocked Horizontal Diaphragms Using Structural Wood Panels and Douglas-Fir Framing [8]

Plywood Grade	Common Nail Size or Screw Size	Fastener Spacing[1]	Minimum Panel Thickness (in.)	Maximum Joist Spacing (in.)	Minimum Nominal Size of Framing Member (in.)	Design Unit Shear Value (plf)	Load Factor[2]
Structural I	10d	6/12	15/32	24	2x	285	3.90
C-D, C-C, and other grades	10d	6/12	15/32	24	2x	255	4.57
			19/32	24	2x	285	3.89
APA Rated STURD-I-FLOOR	10d	6/12	19/32	24	2x	285	4.68
			15/32	24	2x	255	5.12
APA Rated STURD-I-FLOOR	8d	6/12	15/32	24	2x	240	4.66
Structural I C-D 32/16	8d	6/12	1/2	24	2x10	240	5.83
Structural I C-D 48/24	10d	6/6	3/4	48	3x12	320	3.94
2-4-1 T&G	8d	6/6	3/4	48	Double 2x8	320	3.55
Structural I C-D 48/24	#10 Screw	6/12	3/4	48	Steel Truss	190	3.79

For SI: 1 inch = 25.4 mm, 1 plf = 1.488 Kg/m
[1] The first number in the schedule refers to panel edge fastener spacing (inches) along panel edges supported by joists, and the second number refers to fastener spacing (inches) along framing members not at panel edges.
[2] The load factor is determined by dividing the ultimate tested shear value by the design shear value [8].

As can be seen in Table 1, the design shear values for wood diaphragms have somewhat inconsistent and conservative load factors (i.e., safety factor).

Review of Design Procedures

The use of diaphragm action to resist seismic and wind loads is common practice in the design of residential construction. A typical residential floor diaphragm comprises of a number of component elements including floor sheathing, primary supporting members (joists), secondary supporting members (braces), shear connectors, and a variety of fasteners (nails, pneumatic fasteners, screws, etc.). The shear capacity of diaphragms depends mostly on the strength and spacing of the individual fasteners that connect the sheathing to the floor or roof framing.

Design professionals typically model floor diaphragms as deep horizontal beams that carry in-plane forces to stiffened points in the structure (i.e. shear walls). Plywood or oriented-strand-board (OSB) sheathing acts as the web of the beam and the edge framing acts as the flanges of the beam. This diaphragm action provides lateral stability to the structure. To simplify design, the web is assumed to resist only shear forces and the flanges are assumed to resist only flexural forces. The stiffness and ultimate strength of a floor diaphragm must be established by a designer. Prediction of these quantities can be accomplished using any of the following three approaches:

1. Approximate Design — This approach is based on an assumed internal force distribution within the diaphragm where both strength and flexibility may be calculated using the principles of mechanics, fastener strength values, and plywood shear values. Bryan's design method provides a good prediction of the design shear of steel cladded diaphragms fastened on four sides [2]. The American Plywood Association (APA) developed a design method for plywood sheathed horizontal wood diaphragms [8].

2. Finite Element Analysis — A complete diaphragm is simulated by an assemblage of finite elements (beam elements for joists, fastener elements for screws or pins, and plate elements for sheathing). Lawrence [3] and Nilson [4] provide detailed information on the analysis of diaphragms with light-gauge steel cladding.

3. Empirical Design — This approach requires full-scale testing of the floor diaphragm in which load-deflection curves are determined for each type of diaphragm assembly.

There are several papers and articles written describing the first two approaches [2][3][4][5]; however, limited information on tested assemblies and appropriate test methods is found in support of the third approach or for verification of other design methods. The American Iron and Steel Institute (AISI) has recommended unified procedures for testing steel clad diaphragms [5]. However, the AISI recommended procedure only addresses steel decks (such as roof decks) and corrugated steel diaphragms and does not necessarily apply to CFS floor diaphragms as used in residential construction.

Engineering mechanics-based expressions for determining load-deflection characteristics are limited to rather simple cases having well defined boundary conditions. They are further limited by assuming that the deep beam analogy actually represents the strength and stiffness properties of these systems. Therefore, most mechanics-based methods will require some empirical validation and "tuning" to produce efficient design.

3.0 Experimental Approach

Materials

The floor diaphragm test specimens were constructed using materials and methods appropriate for residential construction in accordance with the *Prescriptive Method for Residential Cold-Formed Steel Framing* [10]. All steel materials conformed to the dimensional and minimum specified tensile strength requirements of Table 2. In accordance with the objective of this study, the CFS joists also included large pre-formed holes as shown in Figure 1. The dimensions of the formed holes in the joist are shown in Figures 2 and 3. The tensile strength was verified by tensile tests in accordance with ASTM A370 [11]. Base steel thickness was measured in accordance with ASTM A90 [12]. Mechanical properties were based on coupons cut longitudinally from the center of the web from three samples of each joist size and thickness used in fabricating the floor assembly test specimens.

Table 2
Floor Joist Dimensions and Minimum Specified Tensile Strength

Nominal Joist Size	SSMA Designation[1]	Minimum Tensile Strength (psi)	Minimum Thickness (in.)	Web Size (in.)	Flange Size[2] (in.)	Hole Depth[3] (in.)	Hole Width[4] (in.)	Hole Radius (in.)
2 x 8 x 43	800S162-43	33,000	0.043	8	1.625	4.25	7.00	2.207
2 x 12 x 54	1200S162-54	50,000	0.068	12	1.625	6.25	9.00	3.207

For SI: 1 inch = 25.4 mm, 1 psi = 6.9 KPa.
[1] The designation system used by the Steel Stud Manufacturers' Association (SSMA).
[2] All joist flanges have ½-inch (13 mm) return lip.
[3] A hole depth is the dimension of the hole measured across the depth of the joist.
[4] A hole width is the dimension of the hole measured along the length of the joist.

Oriented strand board (OSB) structural sheathing conformed to U.S. Product Standard PS 2 [13]. The OSB sheathing was APA rated "Sturdi-floor" with a 23/32-inch (18.3 mm) thickness and tongue and grove joints.

Figure 1 - Joist with Large, Pre-formed holes

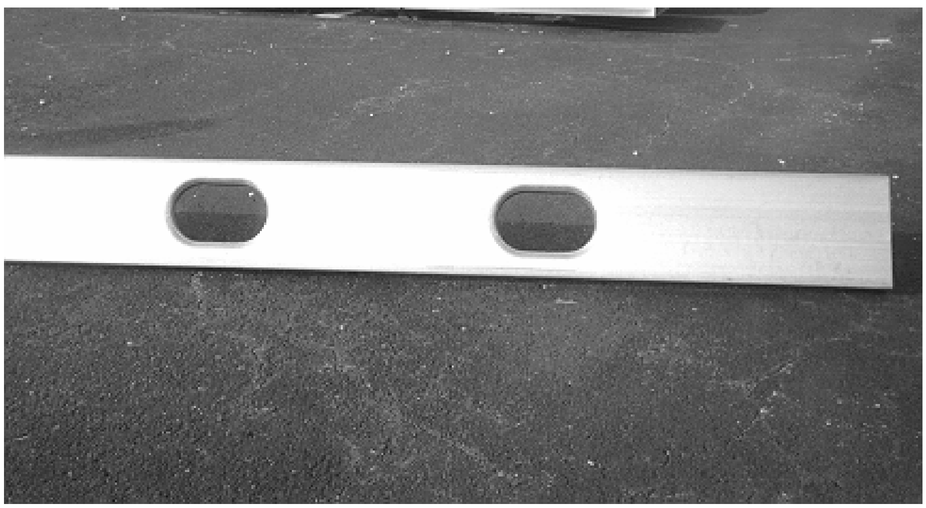
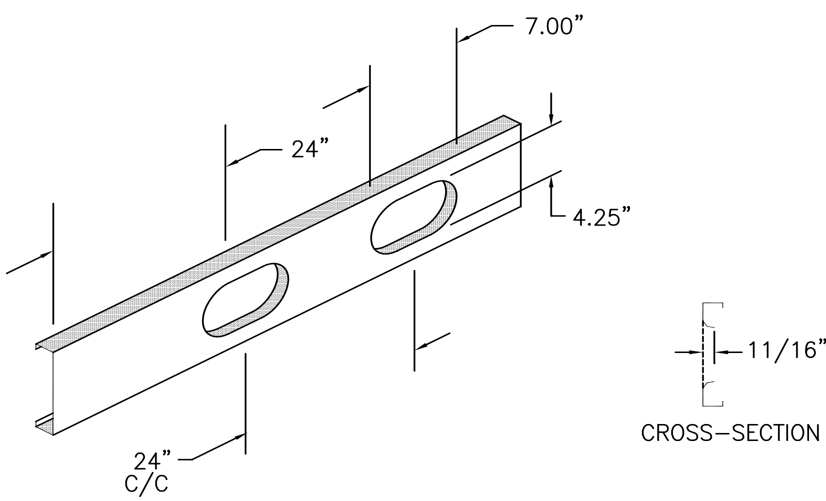


Figure 2 - Web Hole for 8-inch Joist



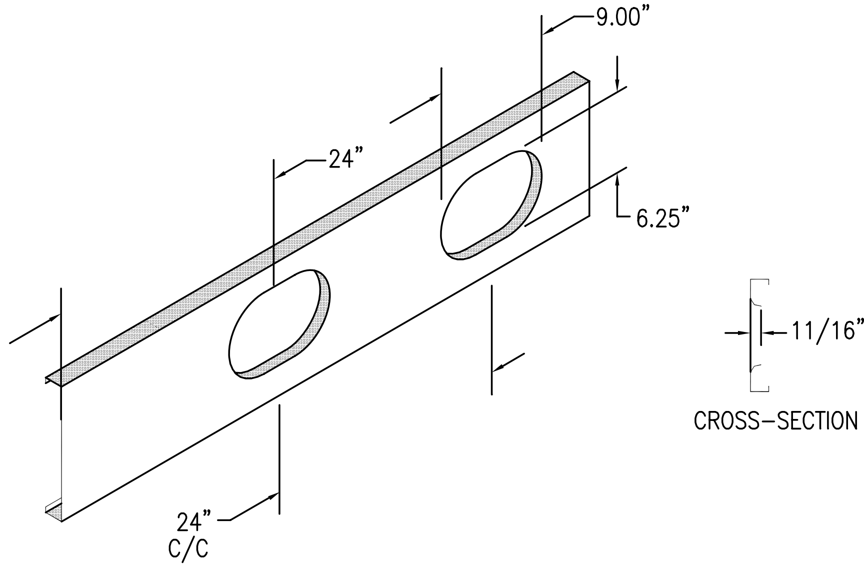


Figure 3 - Web Hole for 10- and 12-inch Joists

Fasteners

In all tests, the sheathing was fastened to steel joists with 1-1/4 in. (32 mm) long, #8 self-drilling, tapping screws with a bugle head diameter of 0.292 in. (7.4 mm). Perimeter steel tracks were fastened to steel joists through the flanges (one screw per flange) using #8 self-drilling, tapping screws with pan-heads. Web stiffeners were installed at all joist bearing locations. A web stiffener was fastened to each end of each joist with four #8 self-drilling, tapping screws. Where required, steel tracks were spliced as shown in Figure 4. All screws protruded through steel framing members with a minimum of three exposed threads.

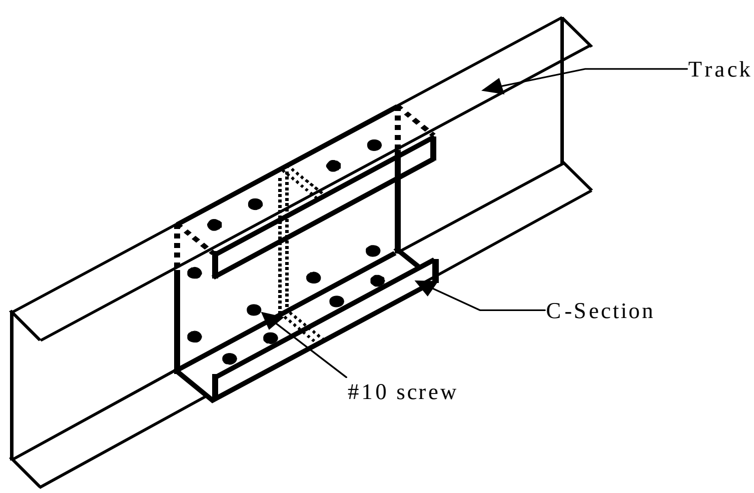


Figure 4 - Track Splice Detail

Floor Diaphragm Fabrication and Test Setup

All floor diaphragms were 12-foot wide x 24-foot long (3.6 m x 7.2 m). Typical floor diaphragm construction and details are shown in Figures 5 and 6 (refer to Figures 2 and 3 for individual floor joist dimensions). Two floor diaphragms were constructed with 800S162-43 (2 x 8 x 43 mil; 51 mm x 203 mm x 1.09 mm) joists and two were constructed with 1200S162-54 (2 x 12 x 54 mil; 51 mm x 305 mm x 1.37 mm) joists. Each diaphragm assembly consisted of 13 joists spaced at 24-inches (610 mm) on-center, 12-foot (3.6 m) long each. OSB sheathing panels were staggered as in common practice. Sheathing screws were spaced 6-inches (152 mm) on-center at panel edges and 12-inches (305 mm) on-center at intermediate supports. The sheathing panel edges transverse to the joists were not blocked.

Each floor diaphragm was supported along the 12-foot (3.6 m) sides as shown in Figure 7 (See also photos in Appendix C). The end joists were fastened to supporting steel tracks with #8 screws (except diaphragm test FD12-54-1 that used #10 screws) at 6-inches (152-mm) on center as shown in Figure 5. An I-beam provided a vertical support at mid-span of each diaphragm specimen. All diaphragm assemblies were loaded using a hydraulic cylinder and spreader beams as shown in Figure 8. The hydraulic cylinder applied the load to the center of an I-beam that transferred equal loads at its reactions to two shorter I-beams. Each short I-beam received a point load at its center of gravity and transferred this load to two equal loads at the diaphragm assembly. This setup resulted in a diaphragm assembly subjected to four equally spaced concentrated loads of equal magnitudes. Ball bearing steel wheels were installed under the ends of all I-beams. The wheels permitted free lateral movement of the beams in the direction of the applied load.

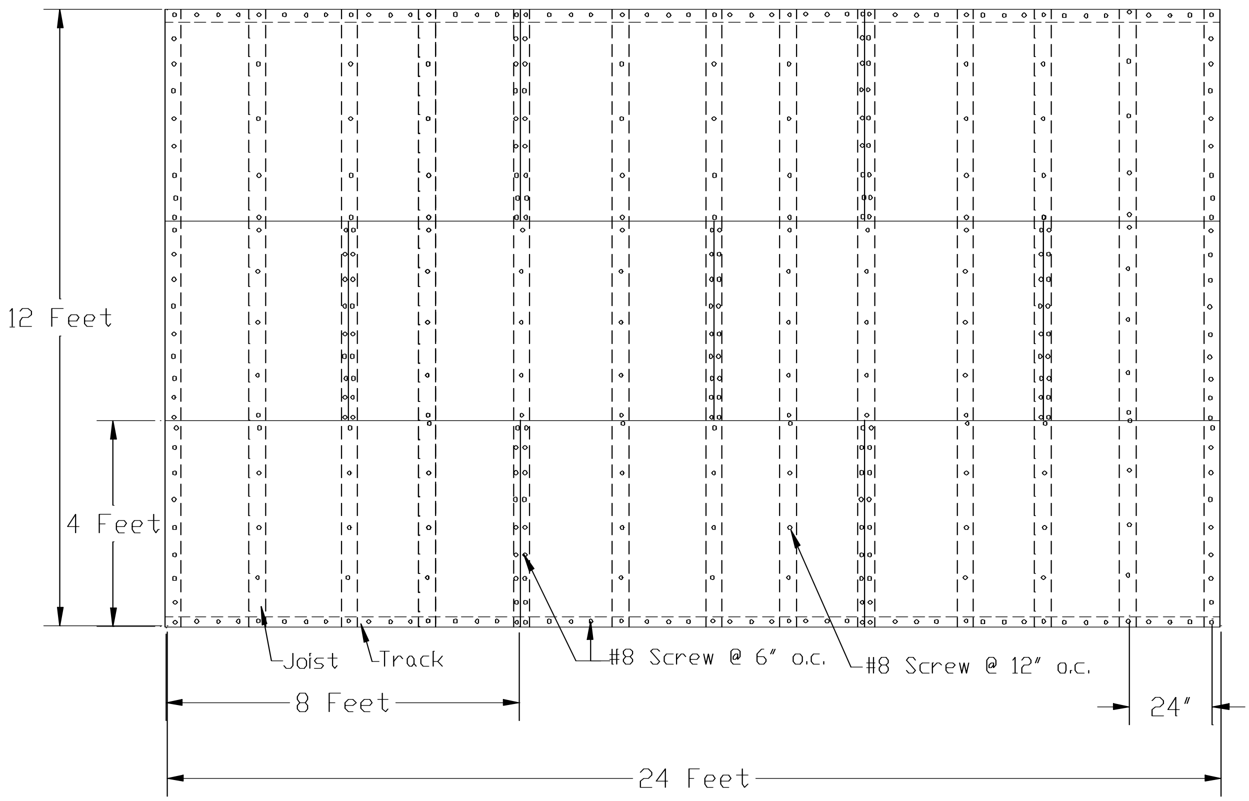


Figure 5 - Floor Diaphragm Sheathing Connection Detail

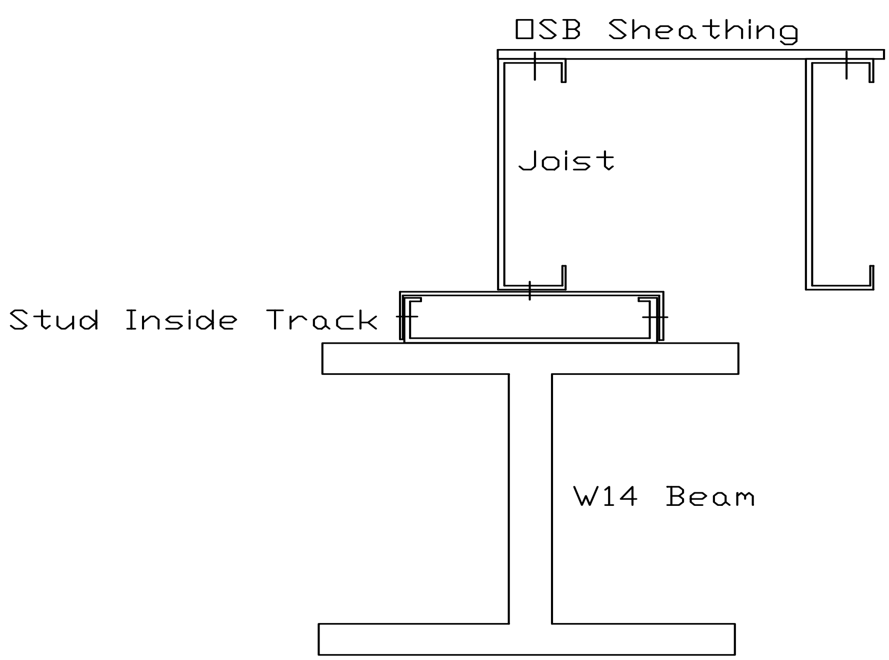


Figure 6 - Floor Diaphragm Mounting Detail

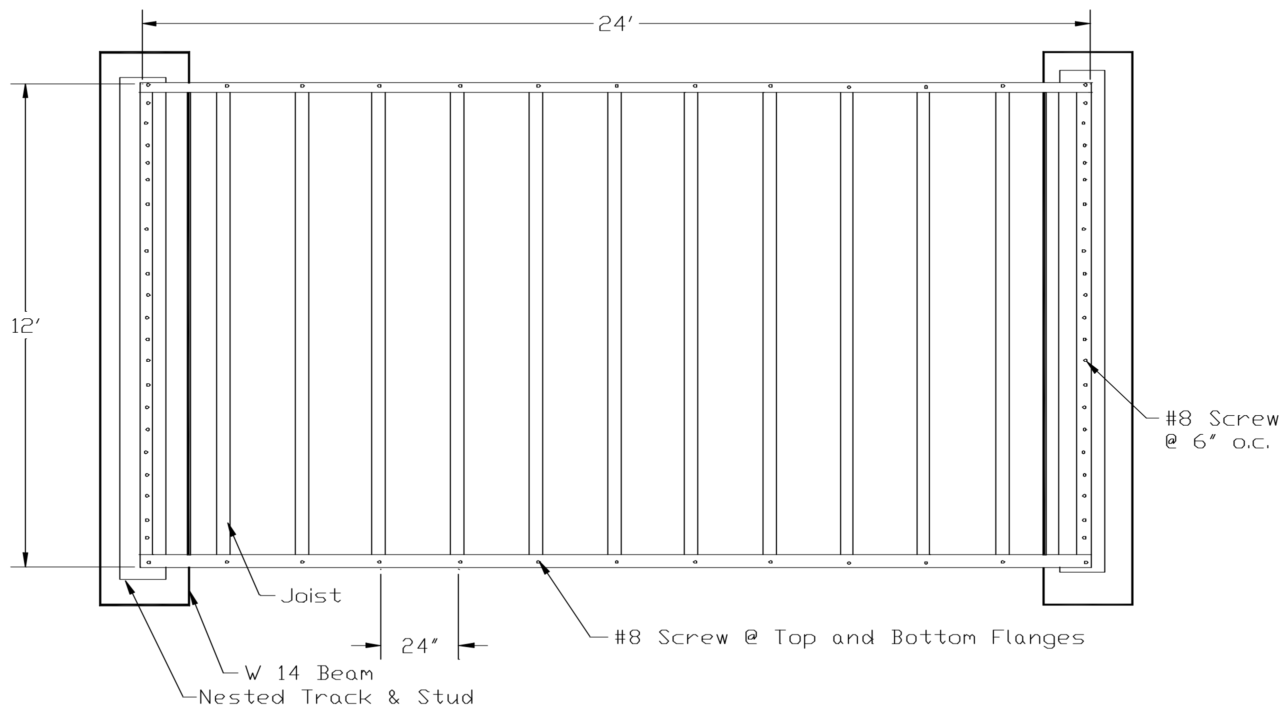


Figure 7 – Floor Diaphragm Supporting Detail

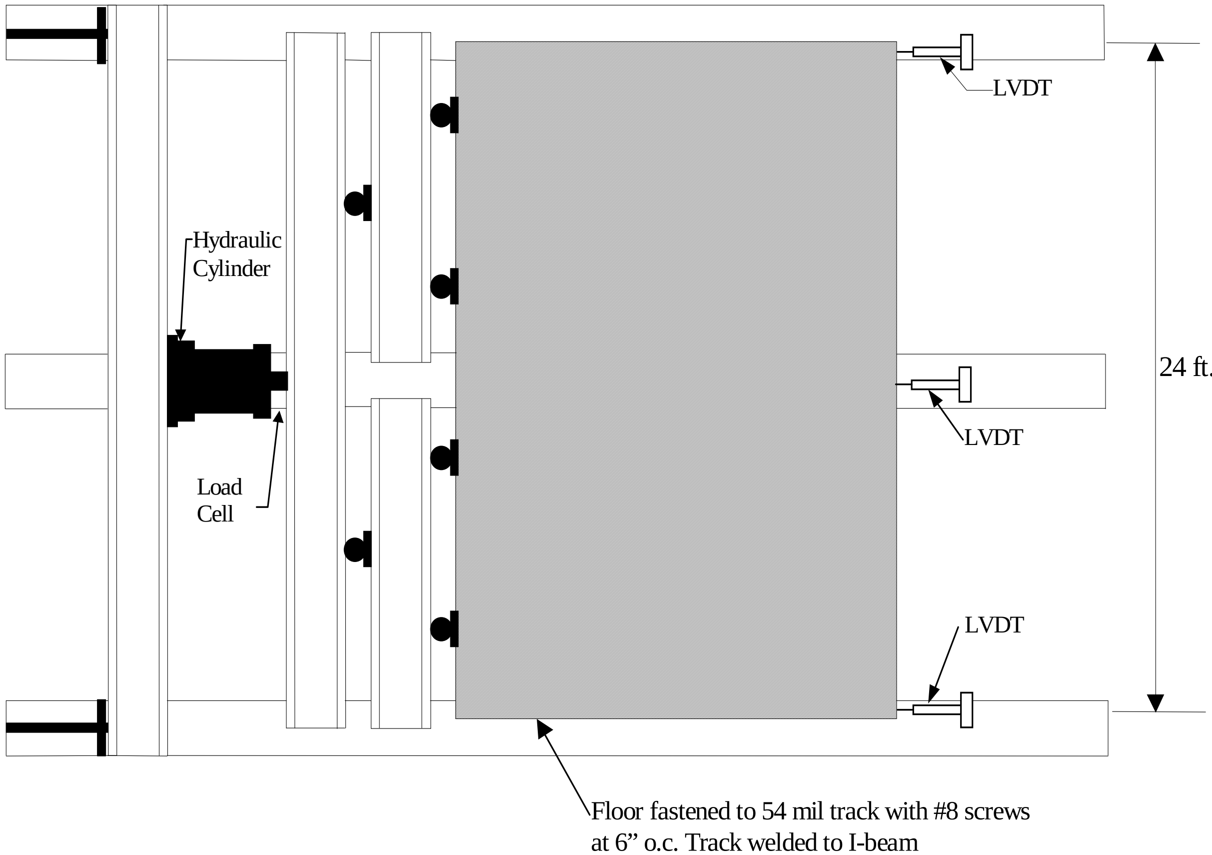


Figure 8 - Floor Diaphragm Test Setup

Horizontal Diaphragm Test Procedure

Each diaphragm was tested by applying four equal concentrated loads spaced at 58-inches (1.45 m) on center through a hydraulic cylinder and spreader beams as previously shown in Figure 8. The same duration and sequence was used for each of the four tests. The sequence of loading was in accordance with Figure 9. The loading sequence followed the APA recommended test sequence with the exception of the number of cycles [8]. APA test protocol calls for eight test cycles before loading the diaphragm to failure. Two cycles were considered adequate for these tests.

The design load of each diaphragm assembly was estimated at approximately 8,000 lb. (36 kN) (see Appendix A). One-half increments of the estimated design load were applied to each tested assembly. Each increment was applied and held for ten minutes after which loads and deflections were recorded (load was held constant). The loading sequence of Figure 9 was continued until twice the estimated design load was reached. The load was released after twice the design load was reached, and any residual deflections were recorded after ten minutes at zero applied load. The loading was repeated and the floor diaphragm was loaded with load increments equal to the estimated design load. After the third load increment (3 x estimated design load) the load was continued until failure.

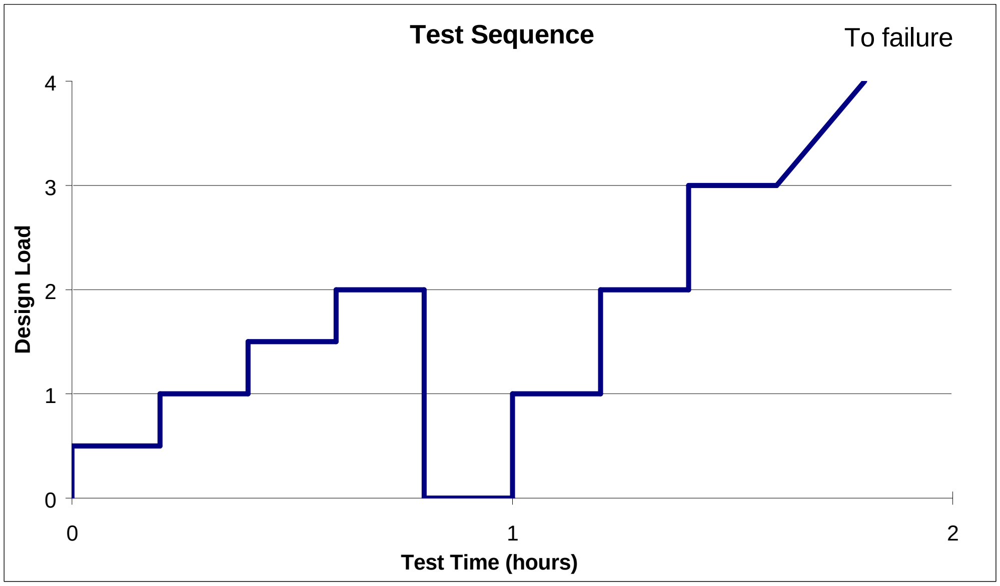


Figure 9 - Load Application Versus Time for all Diaphragm Tests

Individual Fastener Test Procedure

The load-slip response of #8 screws connecting the OSB sheathing to the steel joists (54 mil thickness) was determined by testing. All screws tested were #8 self-drilling tapping screws with bugle heads. Five tests were conducted for each configuration in accordance with ASTM D1761 [14] as shown in Table 3 and Figure 10. Each test utilized a 2-inch wide x 12-inch long (51 mm x 305 mm) strip of 23/32-inch (18 mm) thick OSB fastened to a 54 mil (1.38 mm) steel joist with one # 8 screw. OSB strips were cut parallel and perpendicular to the longer edge of the 4-foot x 8-foot (1.2 m x 2.4 m) sheets. All screws protruded through the steel joists a minimum of 3/8-inch (9.5 mm) with a minimum of three exposed threads. A deflection gauge was installed under the OSB strip to measure the screw slip by measuring the relative displacement between the OSB and the steel joist specimen. A load rate of 0.20-inch (5 mm) per minute was used.

Table 3
No. 8 Screw Test Plan

No. of Tests	Load Application	Screw Edge Distance (in.)
5	Parallel to Grain	2
5	Perpendicular to Grain	2
5	Parallel to Grain	1/2
5	Perpendicular to Grain	1/2
5	Parallel to Grain	3/8
5	Perpendicular to Grain	3/8

For SI: 1 inch = 25.4 mm

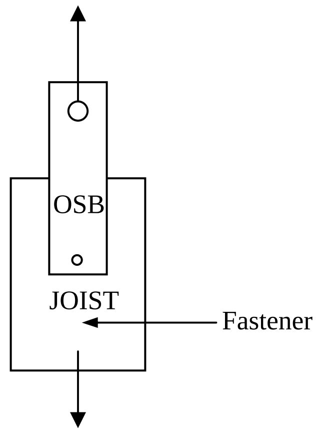


Figure 10 - No. 8 Screw Test Detail (Single Shear)

4.0 Test Results

Tensile Coupon Tests

The mechanical properties of the steels used for the horizontal diaphragm specimens were established by standard tensile coupon tests. Three coupons were cut from the web element of each CFS joist type, and prepared in accordance with ASTM A370 [11]. Uncoated steel thicknesses for all coupon samples were measured in accordance with ASTM A90 [12]. Table 4 lists the average joist dimensions, average tensile test data for yield point (F_y), average ultimate tensile strength (F_u), average uncoated steel thickness (t) and average percent elongation in 2-inch (51 mm) gage length and ½-inch (13 mm) gage length.

Table 4
Physical and Mechanical Properties of Test Specimens

Nominal Joist Size[1]	SSMA Member Designation	Web Size (in.)	Flange Size (in.)	Lip Size (in.)	Yield Point (ksi)		Tensile Strength (ksi)		Uncoated Thickness (in.)		Elongation[2] (percent)	
					Mean	COV	Mean	COV	Mean	COV	2-in.	1/2-in.
2 x 8 x 43	800S162-43	8	1.50	0.5	42.9	0.053	50	0.012	0.0444	0.0542	25	38
2 x 12 x 54	1200S162-54	12	1.50	0.5	53.5	0.060	70	0.006	0.0542	0.0597	20	40

For SI: 1 inch = 25.4 mm, 1 ksi = 6.9 MPa
[1]Table provides mean values based on three tests.
[2] Percent elongation in 2-inch gage length and ½-inch gauge length.

Fastener Tests

Table 5 shows the results of the fastener tests. The parallel-to-grain loading condition exhibited a trend of slightly lower shear capacity than the perpendicular-to-grain condition.

Table 5
No. 8 Screw Ultimate Capacity and Slip[1]

Load Application	Screw Edge Distance (in.)	Ultimate Capacity (lb.)		Slip @ Ultimate Capacity (in.)		Failure Mode
		Mean	COV	Mean	COV	
Parallel to Grain	2	689	0.027	0.346	0.021	Screw tore through OSB
Perpendicular to Grain	2	716	0.024	0.280	0.019	Screw sheared off
Parallel to Grain	1/2	595	0.025	0.285	0.016	Screw tore through OSB
Perpendicular to Grain	1/2	612	0.015	0.298	0.021	OSB cracked at screw location
Parallel to Grain	3/8	497	0.028	0.311	0.024	Screw tore through OSB
Perpendicular to Grain	3/8	532	0.022	0.239	0.018	Screw tore through OSB
Mean		607		0.293		
COV		0.14		0.121		

For SI: 1 inch = 25.4 mm, 1 lb. = 4.5 N

[1] Mean and COV reported on the basis of 5 test repetitions for each configuration.

Diaphragm Tests

Tables 6 and 7 summarize test results for the four floor diaphragms tested. Actual load-deflection curves for the tests are included in Appendix B in addition to smoothed curves using a moving average of the data. For all tests, diaphragm failures occurred when screws pulled through the OSB sheathing in the highly stressed end region of the diaphragm (see photos of failed diaphragms in Appendix C). An occasional screw failed in shear. There was no evidence of shear failure in the OSB in any of the tests.

Table 6
Summary of Tested Horizontal Shear Capacity

Diaphragm Test Designation	Nominal Joist Size [1]	SSMA Member Designation[1]	Min. Panel Thick. (in.)	Screw Spacing [2] (in.)	Peak Load (lb.)	Ultimate Unit Shear Capacity [3] (plf)	Ultimate Unit Shear Capacity/2.5 (plf)
FD12-54-1	2x12x54	1200S162-54	23/32	6/12	28,817	1,201	480
FD12-54-2	2x12x54	1200S162-54	23/32	6/12	30,715	1,280	512
FD8-43-1	2x8x43	800S162-43	23/32	6/12	29,148	1,215	486
FD8-43-2	2x8x43	800S162-43	23/32	6/12	28,611	1,192	477
Mean[4]					29,323	1,222	489
COV[4]					0.028	0.048	0.033

For SI: 1 inch = 25.4 mm, l lb. = 4.5 N, 1 plf = 1.488 Kg/m
[1] Refer to Table 2 for actual joist dimensions.
[2] Screw spacing is 6-inches on-center at supported panel edges and 12-inches on-center at intermediate supports.
[3] Ultimate unit shear is calculated as ½(peak load)/(diaphragm width).
[4] The mean and COV include all tests because the failure was controlled by fastener tearing through sheathing irrespective of joist size and thickness.

Table 7
Summary of Measured Deflections[1]

Diaphragm Test Designation	Nominal Joist Size [2]	SSMA Designation[2]	Panel Thick. (in.)	Screw Spacing [3] (in.)	Measured Deflection @ Ultimate Capacity [4] (in.)	Measured Deflection @ Ultimate Capacity/2.5 (in.)
FD12-54-1	2x12x54	1200S162-54	23/32	6/12	2.03	0.45
FD12-54-2	2x12x54	1200S162-54	23/32	6/12	2.35	0.46
FD8-43-1	2x8x43	800S162-43	23/32	6/12	2.36	0.54
FD8-43-2	2x8x43	800S162-43	23/32	6/12	2.22	0.51
Mean[4]					2.24	0.49
COV[4]					0.059	0.087

For SI: 1 inch = 25.4 mm, l lb. = 4.5 N, 1 plf = 1.488 Kg/m
[1] Values are based on a minimum #8 self-drilling, tapping screw with a minimum 0.292-inch diameter bugle head.
[2] Refer to Table 2 for actual joist dimensions.
[3] Screw spacing is 6-inches on-center at supported panel edges and 12-inches on-center at intermediate supports.
[4] Deflections taken at floor mid-span.
[5] The mean and COV include all tests because the failure was controlled by fastener tearing through sheathing irrespective of joist size and thickness.

Floor Diaphragm FD12-54-1 (12’ x 24’, 12” joists, 54 mil steel thickness, 6/12 screw pattern)

The floor diaphragm exhibited noticeable bending deformation at mid-span at approximately 22,000 lbs. (99 kN) load. This deformation was characterized by separation of the middle OSB sheets from each other (about 1/2-inch (13 mm) along the 4-foot (1.2 m) edge of the OSB) on the tensile side of the diaphragm. The separation of the OSB sheets was symmetrical about the center of the floor. The tensile bending load was redistributed to the outer edges, where the chords (i.e. tracks) were located. The load continued to build up to a peak load of 28,817 lbs. (129.68 kN) when the screws suddenly tore through the OSB along one of the supported ends of the floor diaphragm. A few screws were sheared-off in this region. Coincidentally, screws connecting the end joist to the track failed by tearing through the OSB.

Floor Diaphragm FD12-54-2 (12’ x 24’, 12” joists, 54 mil steel thickness 6/12 screw pattern)

The floor diaphragm exhibited noticeable bending deformation at mid-span at approximately 23,000 lbs. (103.5 kN) load. This deformation was characterized by separation of the middle OSB sheets from each other (about ½-inch (13 mm) along the 4-foot (1.2 m) edge of the OSB) on the tensile side of the diaphragm. The separation of the OSB sheets was symmetrical about the center of the floor. The tensile bending load was redistributed to the outer edges, where the chords (i.e. tracks) were located. The load continued to build up to a peak load of 30,715 lbs. (138.22 kN) when the screws suddenly tore through the OSB along one of the supported ends of the floor diaphragm. Coincidental with the shear failure of the end region screws, the edge of the perimeter track was separated from the end joist and the screw head was sheared-off. Some of the sheathing screws along the failed edge, closest to the load application, had their heads sheared-off.

Floor Diaphragm FD8-43-1 (12’ x 24’, 8” joists, 43 mil steel thickness, 6/12 screw pattern)

Similar to the two previous tests, the floor diaphragm exhibited noticeable bending deformation at mid-span at approximately 20,500 lbs. (92.25 kN) load. This deformation was characterized by separation of the middle OSB sheets from each other (about ½-inch (13 mm) along the 4-foot (1.2 m) edge of the OSB) on the tensile side of the diaphragm. The separation of the OSB sheets was symmetrical about the center of the floor. The tensile bending load was redistributed to the outer edges, where the chords (i.e. tracks) were located. The load continued to build up to a peak load of 29,148 lbs. (131.17 kN) when the screws suddenly tore through the OSB along one of the supported ends of the floor diaphragm. Coincidental with the shear failure of the end region screws, both perimeter tracks were separated from the end joists (along the failed diaphragm end) and the screw heads were sheared-off. Approximately one third of the screws closest to the loading point (along the failed end of the diaphragm) were pulled through the OSB sheathing. One third of the screws at the reaction edge (along the failed end of the diaphragm) were sheared- off. The middle one third of the screws tore out through the sheathing.

Floor Diaphragm FD8-43-2   (12’ x 24’, 8” joists, 43 mil steel thickness, 6/12 screw pattern)

Similar to the three previous tests, the floor diaphragm exhibited noticeable bending deformation at mid-span at approximately 21,000 lbs. (94.5 kN) load. This deformation was characterized by separation of the middle OSB sheets from each other (about 1/2-inch (13 mm) along the 4-foot (1.2 m) edge of the OSB) on the tensile side of the diaphragm. The separation of the OSB sheets was symmetrical about the center of the floor. The tensile bending load was redistributed to the outer edges, where the chords (i.e. tracks) were located. The load continued to build up to a peak load of 28,611 lbs. (128.75 kN) when the OSB panels were separated from the joist along one of the supported ends of the floor diaphragm. Coincidental with the shear failure of the end region screws, both perimeter tracks were separated from the end joists (along the failed diaphragm end) and the screw heads were sheared-off. Approximately one third of the screws closest to the loading point (along the failed end of the diaphragm) were pulled through the OSB sheathing. One third of the screws at the reaction edge (along the failed end of the diaphragm) were sheared- off. The middle third of the screws tore out through the sheathing.

5.0 Discussion

Fastener Tests

Table 8 shows the #8 screw slip values at the ultimate capacity and at the ultimate capacity divided by a factor of safety of 2.5. The screw test data for the 3/8-inch and 1/2-inch (9.5 mm and 13 mm) were used because they are representative of the diaphragm conditions.

Table 8
Load-Slip Response of No. 8 Screw Connecting OSB to CFS Joists[1]

Screw Edge Distance (in.)	Mean Ultimate Load (lb.)	Mean Joint Slip @ Ultimate load (in.)	Mean Ultimate Load /2.5 (lb.)	Mean Joist Slip @ Ultimate Load / 2.5 (in.)
1/2	595	0.285	238	0.051
1/2	612	0.298	245	0.058
3/8	497	0.311	199	0.048
3/8	532	0.239	213	0.050
Average	559	0.283	224	0.052
COV	0.096	0.111	0.096	0.084

For SI: 1 inch = 25.4 mm, 1 lb. = 4.5 N
[1] OSB thickness is 23/32-inches (18.3 mm) and steel thickness is 54 mil (1.37 mm)

Diaphragm Tests

Only the diaphragm action of the floor sheathing was considered in the tests. Ceiling finish materials were not included so that the test results would not be dependent on particular ceiling finish. The addition of shear resistant ceiling material (such as gypsum board) would have likely increased the diaphragm stiffness and strength under monotonic loading. The additive shear resistance of assemblies fabricated with different or dissimilar materials has been shown during previous testing of shear walls [6].

The size and thickness of the CFS floor joists had an insignificant impact on the ultimate shear capacity of the floor diaphragms. The only difference was that FD8-43-1 and FD8-43-2 floor diaphragms showed signs of OSB panel joint separation at lower loads (about 10-20% lower) than FD12-54-1 and FD12-54-2. All floor diaphragm failures were abrupt in nature.

Table 9 gives recommended design unit shear values for CFS horizontal diaphragms for the conditions tested. A safety factor of 2.5 (or resistance factor of 0.55) is recommended to be consistent with CFS shear walls currently recognized in Volume II of the Uniform Building Code [7]. Table 10 shows the nominal load per fastener based on the ultimate shear load of the floor diaphragm.

Table 9
Recommended Allowable Unit Shear Values for Unblocked
CFS Diaphragms Sheathed with Structural Wood Panels[1]

Screw Size[2]	Edge Fastener Spacing	Intermediate Fastener Spacing	Minimum Sheathing Thickness[3]	Allowable Unit Shear Capacity[4] (plf)
#8	6”	12”	23/32”	489

For SI: 1 inch = 25.4 mm, 1 plf = 1.488 Kg/m
[1] Values apply to 16 gauge (54 mil) and 18-gauge (43 mil) steel floor joists at a maximum spacing of 24-inches on-center.
[2] Minimum head diameter of 0.29 in.
[3] Applies to APA rated structural panel sheathing only.
[4] Average ultimate shear capacity divided by a factor of safety of 2.5.

Table 10
Fastener Load

Floor Diaphragm Average Ultimate Shear Load (plf)	Average Ultimate Load per Perimeter Screw[1] (lb.)	Average Ultimate Load per Individual Screw[2] (lb.)
1,222	543	559

For SI: 1 plf = 15 N/m, 1 lb. = 4.5 N
[1] An average of 2.25 screws per foot is used to account for actual number of end panel screws (1,222/2.25 = 543 lbs.)
[2] Based on individual fastener test data (Table 5) with 3/8-inch and 1/2-inch edge distance.

The diaphragm mid-span deflection can be estimated using the deflection equation in [15] with a modified screw slip coefficient, as shown below in Equation 1. The calculation of the modified coefficient of the deflection portion of the equation due screw slip can be found in Appendix A. The screw slip coefficient is based on individual fastener and diaphragm test results. Predicted and actual deflections for the diaphragm tests are shown in Table 11 (with screw joint slippage, e_n, obtained from Table 12).

$$\Delta = \frac{5VL^3}{8EAb} + \frac{VL}{4Gt} + 0.23Le_n + \frac{\sum(\Delta_c X)}{2b} \qquad \text{[Equation 1]}$$

Where

V = maximum unit shear in the direction under consideration plf (kg/m)
L = diaphragm length, ft (m)
b = diaphragm width, ft (m)
A = net area of track cross section, in.^{2} (mm^2)
E = elastic modulus of joist, psi (MPa)
G = modulus of rigidity of sheathing, psi (MPa)
t = effective thickness of sheathing for shear, in. (mm)
e_n = screw joint slippage at load per screw on perimeter of interior panel, in. (mm) (refer to Table 12 for e_n values. These values are obtained from Figure B9)
$\sum(\Delta_c X)$ = sum of individual joist splice slip values on both sides of the diaphragm, each multiplied by its distance (ft) to the nearest support.
Δ = calculated deflection at mid-point of diaphragm, in. (mm).

Table 11
Predicted vs. Calculated Diaphragm Deflection

	Deflection @ Peak Load (in.)	Deflection @ 55% of Ultimate Capacity (in.)	Deflection @ 40% of Ultimate Capacity (in.)
Actual	2.24	0.59	0.49
Using Equation 1[1]	2.13	0.71	0.52

For SI: 1 inch = 25.4 mm.
[1] Based on Equation 1 with e_n from Table 12 for #8 screw and 23/32-inch (18.3 mm) thick OSB.

Table 12
Screw Joint Slippage (e_n) Values

e_n @ Peak Load (in.)	e_n @ 55% of Ultimate Capacity (in.)	e_n @ 40% of Ultimate Capacity (in.)
0.28	0.077	0.052

For SI: 1 inch = 25.4 mm.

6.0 Conclusions

This report provides test data and design values for typical CFS floor diaphragms applicable to residential construction. In addition, a deflection equation is provided for CFS horizontal diaphragms based on that currently used for wood diaphragms. The following conclusions are based on the findings of this study:

- The size and/or thickness of the CFS joists tested did not significantly influence diaphragm stiffness and strength.
- The CFS diaphragms failed by the screws pulling through the sheathing at the supported edges along the ends of the diaphragms.
- The individual screw shear capacity can be used to reasonably predict the diaphragm's shear capacity based on the fastener spacing along the diaphragm ends. The prediction can be accomplished by multiplying the individual fastener's shear capacity by the number of fasteners along one of the ends of a rectangular floor diaphragm.

7.0 References

[1] "*Innovative Residential Floor Construction: Structural Evaluation of Steel Joists with Pre-Formed Web Openings*," Prepared for the US Department of HUD, the National Association of Home Builders, and Dietrich Industries, Inc. by the NAHB Research Center, Inc. Upper Marlboro, MD. March 1999.

[2] Bryan, E. R., "*The Stressed Skin Design of Steel Buildings*," Constrado Monographs, Crosby Lockwood Staples, London, England, 1972.

[3] Lawrence, S. J., and Sved, G., "*A Finite Element Analysis of Clad Structures*," Conference on Metal Structures Research and its Applications, Sydney, Australia, November 1972.

[4] Nilson, A. H. "*Analysis of Light Gauge Steel Shear Diaphragms*," Second Specialty Conference on Cold-Formed Steel Structures, University of Missouri, Rolla, MO. October 1973, pp 325-363.

[5] "*Design of Light Gauge Steel Diaphragms*" American Iron and Steel Institute, Washington, DC, 1967.

[6] "*Shear Wall Design Guide*", 1998 Edition. American Iron and Steel Institute (AISI), Washington, DC. February 1998.

[7] "*Uniform Building Code,*" International Conference of Building Officials (ICBO). Whittier, CA. 1997.

[8] Tissell, J. R., "*Plywood Diaphragms*" Research Report 138, American Plywood Association (APA), Tacoma, Washington, September 1993.

[9] Carney, J.M. "*Bibliography on Wood and Plywood Diaphragms*," Journal of the Structural Division. Proceedings of the American Society of Civil Engineers, Vol. 101, No. ST 11. November 1975.

[10] *Prescriptive Method for Residential Cold-Formed Steel Framing*. Second Edition. Prepared by the NAHB Research Center, Inc., for the American Iron and Steel Institute (AISI), the U.S. Department of Housing and Urban Development (HUD) and the National Association of Home Builders (NAHB). Upper Marlboro, MD. 1997.

[11] ASTM A 370-96 "*Standard Test Methods and Definitions for Mechanical Testing of Steel Products*," American Society for Testing and Materials (ASTM), West Conshohocken, PA. 1996.

[12] ASTM A 90/A90M-93 "*Standard Test Method of Weight [Mass] of Coating on Iron and Steel Articles with Zinc or Zinc-Alloy Coatings*," American Society for Testing and Materials (ASTM), West Conshohocken, PA. 1993.

[13] NIST PS 2-92 "*Performance Standard for Wood-Based Structural-Use Panels*," U.S. Department of Commerce/Technology Administration, National Institute of Standards and Technology. Washington D.C. December 1992.

[14] ASTM D 1761-88 "*Standard Test Methods for Mechanical Fasteners in Wood*," American Society for Testing and Materials (ASTM), West Conshohocken, PA. Reapproved 1995.

[15] "*Design and Fabrication of All-Plywood Beams*," Form No. H815E. American Plywood Association (APA). Tacoma, WA. September 1995.

APPENDIX A

SAMPLE CALCULATIONS

Estimated Diaphragm Design Shear Load

Because of the absence of data to predict horizontal diaphragm capacity with CFS joists and structural wood sheathing, shear wall design values were used as a point of reference. The AISI *Shear Wall Design Guide* [1] does not contain shear data for walls sheathed with 23/32-inch (0.72 mm) OSB. Therefore, data for walls sheathed with 15/32-inch (0.47 mm) plywood was used to estimate the expected design load for the floor diaphragms tested in this study.

V = 904/2.5 = 362 plf. (Table 5 of [1], 15/32 plywood, 0.054 in. framing thickness, 6/12 screw spacing)

Total Design Load = 362 plf x 12 ft. x 2 = 8,688 lb.

Diaphragm Deflection Using APA's Method [2][3]:

$$\frac{5VL^3}{8EAb} + \frac{VL}{4Gt} + 0.188Le_n + \frac{\sum(\Delta_c X)}{2b} = \Delta$$

Deflection due to track splice slip, assume 0

Deflection due to screw slip

Shear deflection

Bending deflection

For SI:

$$\Delta = \frac{52VL^3}{EAb} + \frac{VL}{4Gt} + 0.614Le_n + \frac{\sum(\Delta_c X)}{2b}$$

where,

V = maximum unit shear in the direction under consideration = 1,222 plf (1,818 kg/m)

L = diaphragm length = 24 ft (7.2 m)

b = diaphragm width = 12 ft (3.6 m)

A = net area of track cross section = 0.7993 in.^{2} (516 mm^2) {calculated per [4]}

E = elastic modulus of joist = 29,500,000 psi (203,395 MPa), [4]

G = modulus of rigidity of sheathing = 73,000 psi (503.3 MPa) {23/32" plywood, Table 7 of [5]}

t = effective thickness of sheathing for shear = 0.739 in. (18.8 mm) {Table 1 of [3]}

e_n = screw joint slippage at load per screw on perimeter of interior panel = 0.28 in. (7.11 mm) at ultimate load (Table 8 of main body of report)

$\sum(\Delta_c X)$ = sum of individual joist splice slip values on both sides of the diaphragm, each multiplied by its distance in feet (mm) to the nearest support

Δ = calculated deflection at mid-point of diaphragm, in. (mm)

A = t(w + 2f + 2r) {Section 3 of [4]}

The parameters "w" and "f" are the flat portions of the web and flange, respectively, and "r" is the length of arc (corner radius) as follows:

w = 12 – (2x0.09375 + 2x0.05418) = 11.70 in.
f = 1.5 – 0.09375 – 0.05418/2 = 1.379 in.
r = π(0.09375)/2 = 0.147 in.

Gross area = A_G = 0.7993 in.^{2}

$$\frac{5VL^3}{8EAb} = \frac{5x1222x24^3}{8x29{,}500{,}000x0.7993x12} = 0.037 \text{ ft} = 0.448 \text{ in.}$$

$$\frac{VL}{4Gt} = \frac{1222x24}{4x73{,}000x0.739} = 0.136 \text{ in.}$$

Solve the deflection equation for the constant (χ):

Δ = 0.444" + 0.136" + (χ)(24')(0.28") + 0 = 2.24 in.

Where e_n = 0.28 in. (Table 8 of main body of report)
2.24 in. is the actual measured deflection at ultimate load (Table 7 of main body of report)

χ = 0.247

Therefore, the deflection due to screw slip at ultimate load is 0.247Le_n and the total deflection is:

$$\Delta = \frac{5VL^3}{8EAb} + \frac{VL}{4Gt} + 0.247Le_n + \frac{\sum(\Delta_c X)}{2b}$$

Calculate the constant in the screw slip deflection equation at design load (1222 plf/2.5 = 489 plf)

$$\frac{5VL^3}{8EAb} = \frac{5x489x24^3}{8x29{,}500{,}000x0.7993x12} = 0.0149 \text{ ft.} = 0.179 \text{ in.}$$

$$\frac{VL}{4Gt} = \frac{489x24}{4x73{,}000x0.739} = 0.0544 \text{ in.}$$

Δ = 0.179" + 0.0544" + (χ)(24')(0.052") = 0.49 in.

Where e_n = 0.052 in. (Table 8 of main body of report)

0.49 in. is the actual measured deflection at design load (Table 7 of main body of report)

χ = 0.206

Therefore, the deflection due to screw slip at design load is $0.206Le_n$ and the total diaphragm deflection is:

$$\Delta = \frac{5VL^3}{8EAb} + \frac{VL}{4Gt} + 0.206Le_n + \frac{\sum(\Delta_c X)}{2b}$$

Use the average of the screw slip at ultimate load and design load to estimate the deflection of the diaphragm:

$$\Delta = \frac{5VL^3}{8EAb} + \frac{VL}{4Gt} + 0.23Le_n + \frac{\sum(\Delta_c X)}{2b}$$

APPENDIX A REFERENCES

[1] "*Shear Wall Design Guide*", 1998 Edition. American Iron and Steel Institute (AISI), Washington, DC. February 1998.

[2] Tissell, J. R., "*Plywood Diaphragms*" Research Report 138, American Plywood Association (APA), Tacoma, Washington, September 1993.

[3] "*Design and Fabrication of All-Plywood Beams*," Form No. H815E. American Plywood Association (APA). Tacoma, WA. September 1995.

[4] AISI "*Cold-Formed Steel Design Manual*," 1996 Edition. American Iron and Steel Institute (AISI), Washington, DC. June 1997.

[5] "*Plywood Design Specification*," American Plywood Association (APA). Tacoma, WA. January 1997.

APPENDIX B

LOAD-DEFLECTION CURVES

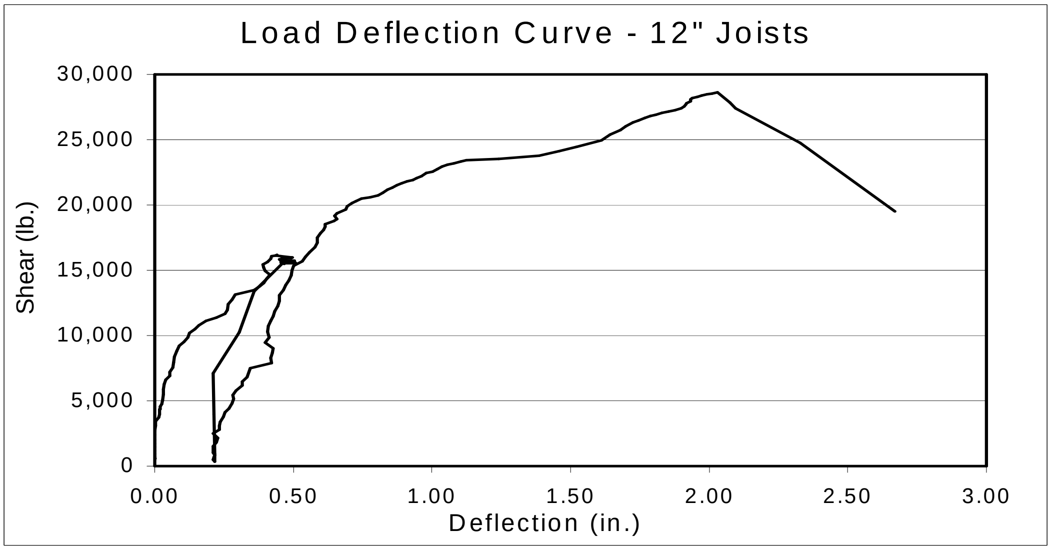


Figure B1 - Total Applied Load-Deflection Curve for FD12-54-1

Figure B2 - Shear Load-Deflection Curve for FD12-54-1

Figure B3 - Total Applied Load-Deflection Curve for FD12-54-2

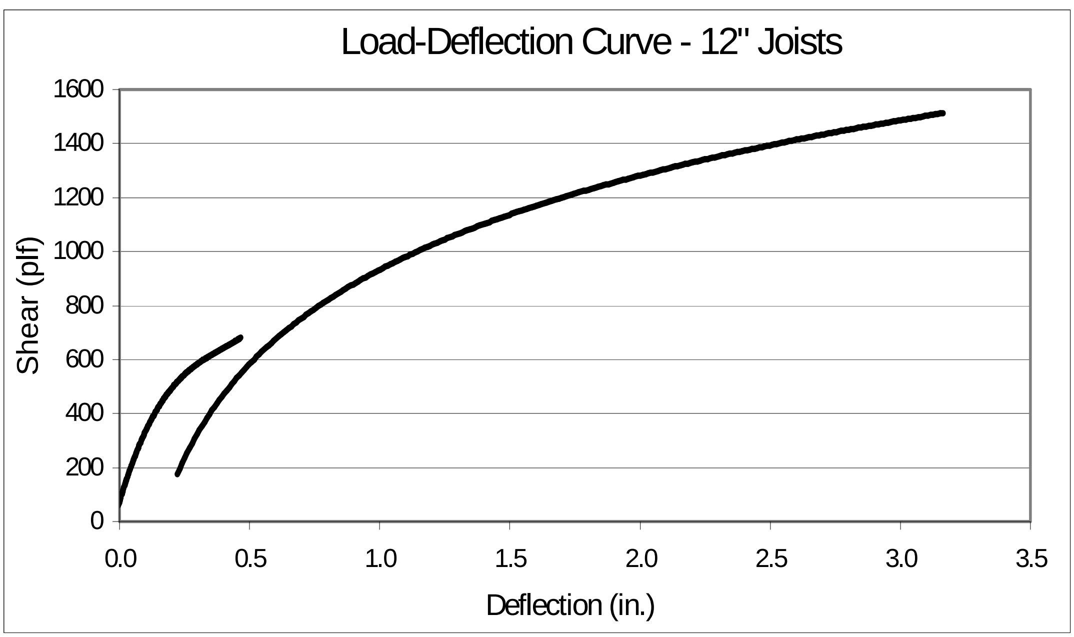


Figure B4 - Shear Load-Deflection Curve for FD12-54-2

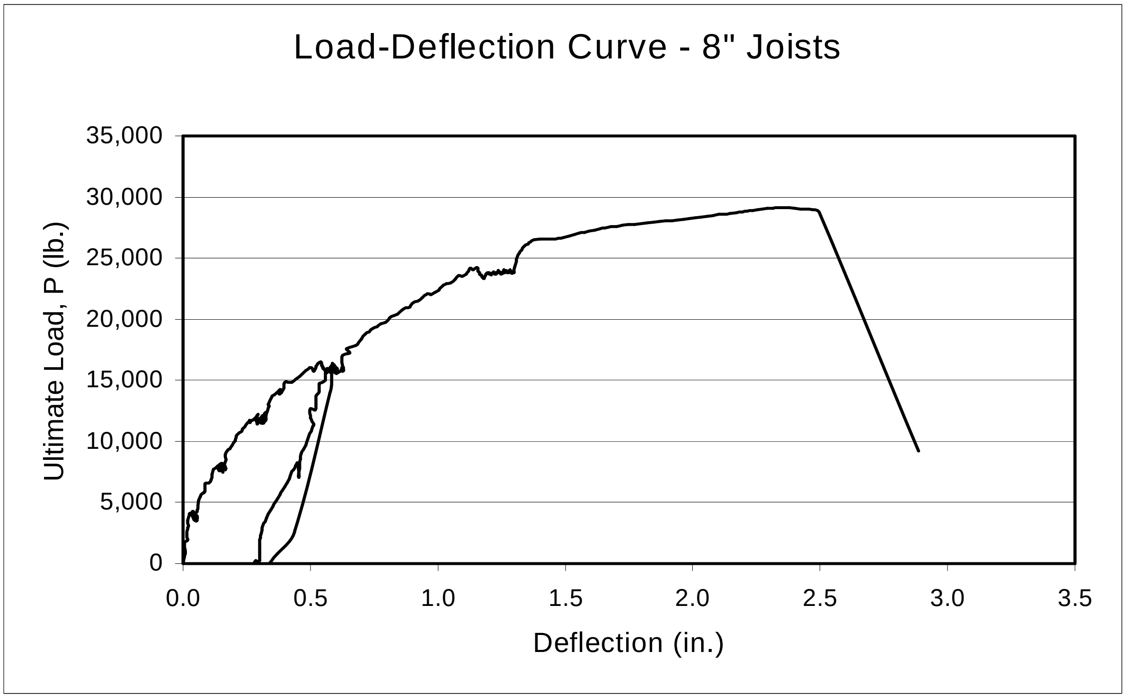


Figure B5 - Total Applied Load-Deflection Curve for FD8-43-1

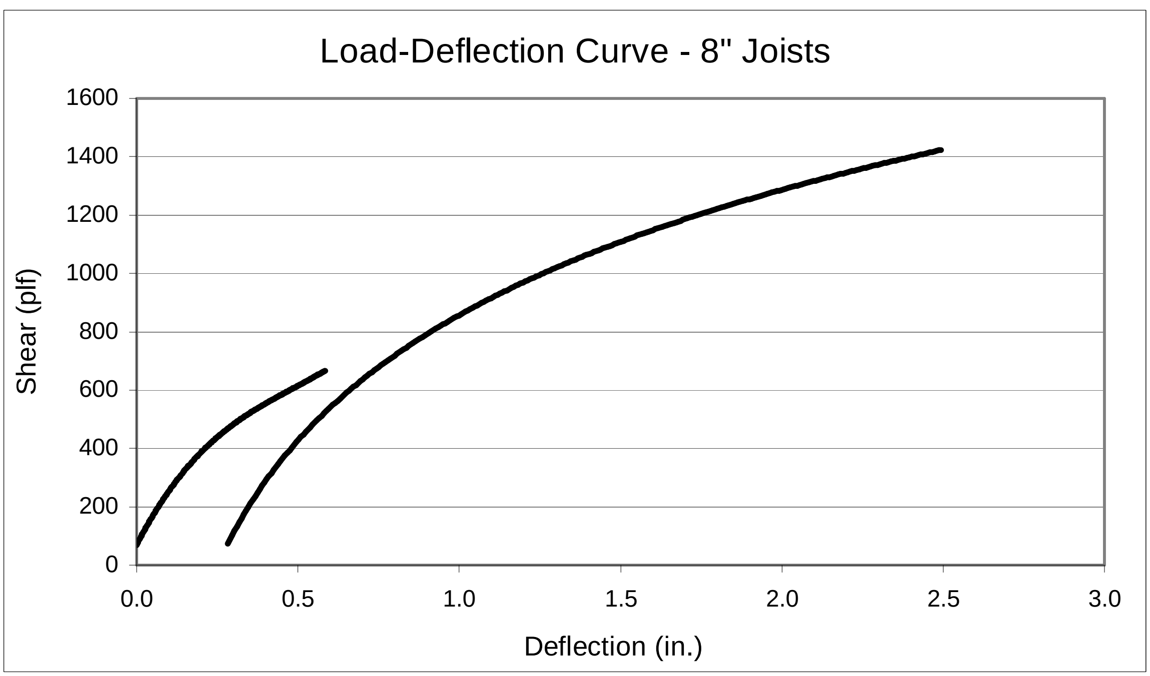


Figure B6 - Shear Load-Deflection Curve for FD8x43-1

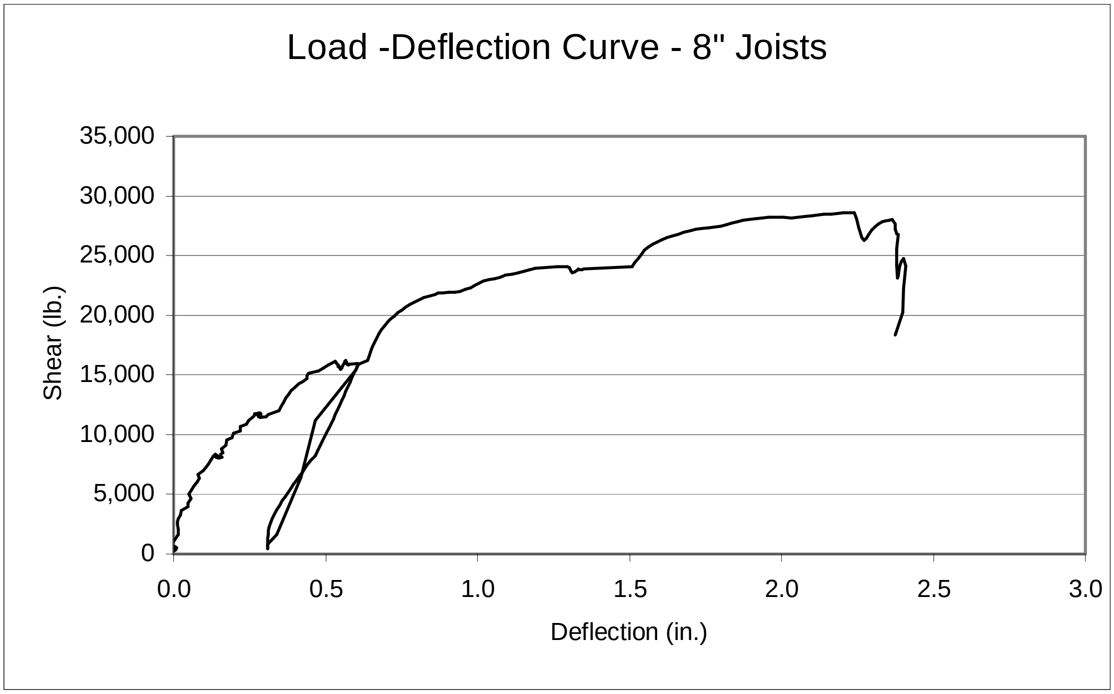


Figure B7 - Total Applied Load-Deflection Curve for FD8-43-2

Figure B8 - Shear Load-Deflection Curve for FD8-43-2

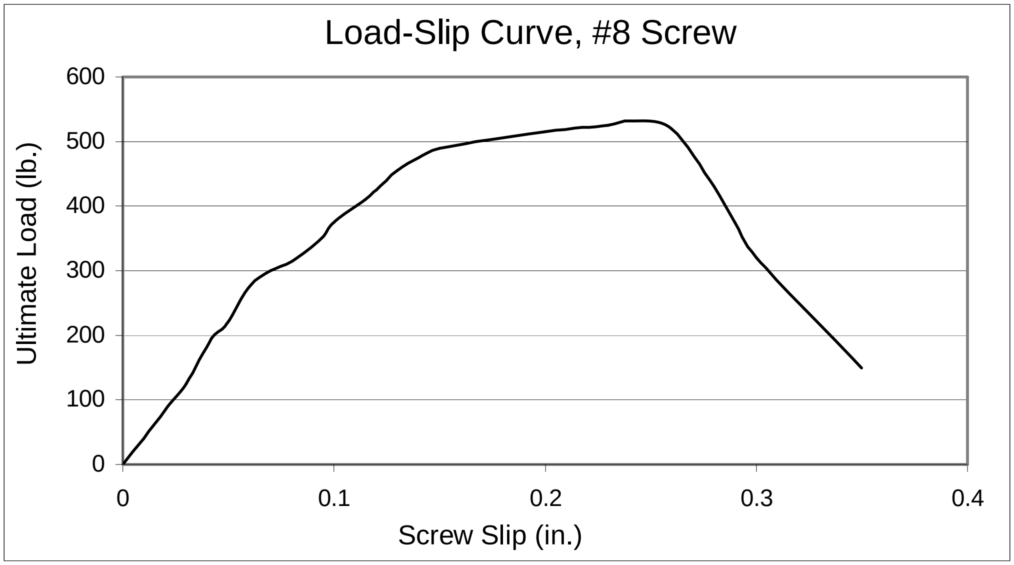


Figure B9 - Screw Slip Curve

APPENDIX C

TEST PHOTOGRAPHS

APPENDIX D

METRIC CONVERSION

Metric Conversion Factors

The following list provides the conversion relationship between U.S. customary units and the International System (SI) units. A complete guide to the SI system and its use can be found in ASTM E 380, Metric Practice.

To convert from	to	multiply by
Length		
inch (in.)	micrometer (µm)	25,400
inch (in.)	millimeter (mm)	25.4
inch (in.)	centimeter (cm)	2.54
inch (in.)	meter (m)	0.0254
foot (ft)	meter (m)	0.3048
yard (yd)	meter (m)	0.9144
mile (mi)	kilometer (km)	1.6
Area		
square foot (sq. ft)	square meter (sq. m)	0.0929
square inch (sq. in)	square centimeter (sq. cm)	6.452
square inch (sq. in.)	square meter (sq. m)	0.00064516
square yard (sq. yd)	square meter (sq. m)	0.8391
square mile (sq. mi)	square kilometer (sq. km)	2.6
Volume		
cubic inch (cu in.)	cubic centimeter (cu cm)	16.387064
cubic inch (cu in.)	cubic meter (cu m)	0.00001639
cubic foot (cu ft)	cubic meter (cu m)	0.02831685
cubic yard (cu yd)	cubic meter (cu m)	0.7645549
gallon (gal) Can. liquid	liter	4.546
gallon (gal) Can. liquid	cubic meter (cu m)	0.004546
gallon (gal) U.S. liquid*	liter	3.7854118
gallon (gal) U.S. liquid	cubic meter (cu m)	0.00378541
fluid ounce (fl oz)	milliliters (ml)	29.57353
fluid ounce (fl oz)	cubic meter (cu m)	0.00002957
Force		
kip (1000 lb.)	kilogram (kg)	453.6
kip (1000 lb.)	Newton (N)	4,448.222
pound (lb.)	kilogram (kg)	0.4535924
pound (lb.)	Newton (N)	4.448222
Stress or pressure		
kip/sq. inch (ksi)	megapascal (Mpa)	6.894757
kip/sq. inch (ksi)	kilogram/square centimeter (kg/sq. cm)	70.31
pound/sq. inch (psi)	kilogram/square centimeter (kg/sq. cm)	0.07031
pound/sq. inch (psi)	pascal (Pa) **	6,894.757
pound/sq. inch (psi)	megapascal (Mpa)	0.00689476
pound/sq. foot (psf)	kilogram/square meter (kg/sq. m)	4.8824
pound/sq. foot (psf)	pascal (Pa)	47.88

To convert from	to	multiply By
Mass (weight)		
pound (lb.) avoirdupois	kilogram (kg)	0.4535924
ton, 2000 lb.	kilogram (kg)	907.1848
grain	kilogram (kg)	0.0000648
Mass (weight) per length		
kip per linear foot (klf)	kilogram per meter (kg/m)	0.001488
pound per linear foot (plf)	kilogram per meter (kg/m)	1.488
Moment		
1 foot-pound (ft-lb.)	Newton-meter (N-m)	1.356
Mass per volume (density)		
pound per cubic foot (pcf)	kilogram per cubic meter (kg/cu m)	16.01846
pound per cubic yard (lb/cu yd)	kilogram per cubic meter (kg/cu m)	0.5933
Velocity		
mile per hour (mph)	kilometer per hour (km/hr)	1.60934
mile per hour (mph)	kilometer per second (km/sec)	0.44704
Temperature		
degree Fahrenheit (°F)	degree Celsius (°C)	$t_C = (t_F - 32)/1.8$
degree Fahrenheit (°F)	degree Kelvin (°K)	$t_K = (t_F + 59.7)/1.8$
degree Kelvin (°F)	degree Celsius (°C)	$t_C = (t_K - 32)/1.8$

* One U.S. gallon equals 0.8327 Canadian gallon
** A pascal equals 1000 Newton per square meter.

The prefixes and symbols below are commonly used to form names and symbols of the decimal multiples and submultiples of the SI units.

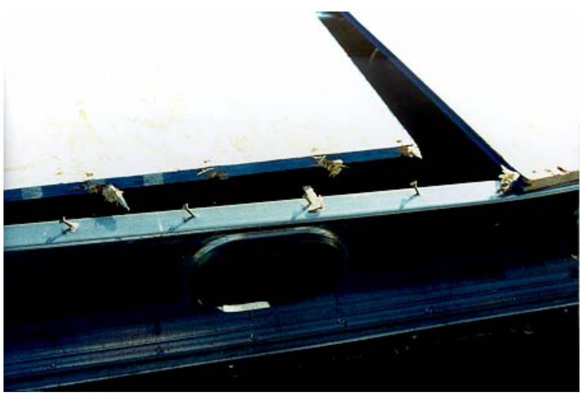
Multiplication Factor	Prefix	Symbol
$1,000,000,000 = 10^9$	giga	G
$1,000,000 = 10^6$	mega	M
$1,000 = 10^3$	kilo	k
$0.01 = 10^{-2}$	centi	c
$0.001 = 10^{-3}$	milli	m
$0.000001 = 10^{-6}$	micro	µ
$0.000000001 = 10^{-9}$	nano	n

DIAPHRAGM ASSEMBLY AND TEST SETUP

SHEATHING SEPARATION BEFORE FAILURE




DIAPHRAGM ASSEMBLY AT FAILURE



DIAPHRAGM ASSEMBLY AT FAILURE