COLD-FORMED STEEL RESEARCH CONSORTIUM

FastFloor Residential Testing Report

H.L. Caswell V, S. Torabian, B.W. Schafer

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COLD-FORMED STEEL RESEARCH CONSORTIUM

REPORT SERIES

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CFSRC Information

The Cold-Formed Steel Research Consortium (CFSRC) is a multi-institute consortium of university researchers dedicated to providing world-leading research that enables structural engineers and manufacturers to realize the full potential of structures utilizing cold-formed steel. More information can be found at www.cfsrc.org. All CFSRC reports are hosted permanently by the Johns Hopkins University library in the DSpace collection: https://jscholarship.library.jhu.edu/handle/1774.2/40427.

SDII Information

The Steel Diaphragm Innovation Initiative (SDII) is a collaborative dedicated to innovation in steel building systems. SDII was formed in 2015 to address specific challenges in concrete-filled and bare steel deck diaphragm systems and SDII's first phase concluded in 2021. In 2022 SDII expanded (Phase II) to broadly address innovation in steel building systems with a specific focus on the performance of steel floor systems. Updates on the activities of SDII are provided at https://steeli.org.

Acknowledgment

The team would like to acknowledge the American Institute of Steel Construction and the Steel Deck Institute for providing funding for this project. In addition, we provide thanks specifically to Thomas Sputo, Devin Huber and Larry Kruth for their oversight and advice on the work. We would also like to thank David Attanucci and DACS Inc. for donating and shipping the steel deck and USG for donating the structural panels and structural panel fasteners. Finally, we would like to acknowledge all the students in the Thin-walled Structures Group at JHU that helped transport and assemble the specimens. This testing would not have been possible without the contributions from all these people and organizations.

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1. Introduction

The goal of FastFloor Residential project is to create a new floor system that is lightweight, fast to construct and nonproprietary. FastFloor Residential strives to achieve this by using 3 in. deep steel deck of 18 gauge that is fastened back-to-back to create a cellular deck, as shown in Figure 1. The cellular deck is then topped with ¾ in thick cementitious (structural) panel that is screwed to the steel deck.

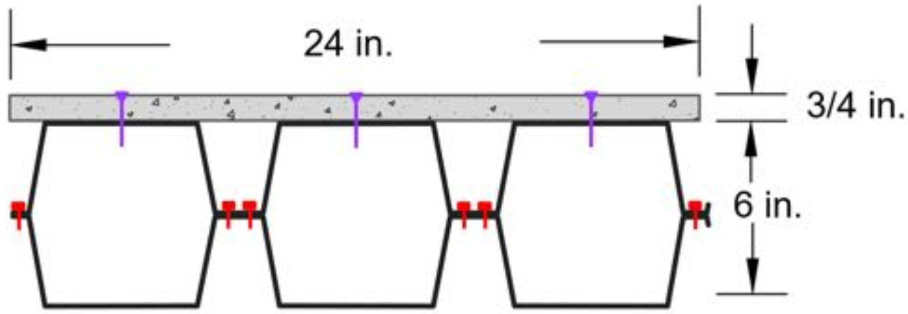


Figure 1. FastFloor Residential cross-section

A series of physical four-point bending tests were conducted in the Thin-Walled Structures Lab at Johns Hopkins University, As seen in Figure 2. The goal of the testing is to understand the behavior of the composite action between the steel deck and cementitious panel, identify the failure modes, and evaluate the strength and stiffness of the composite floor system.

Figure 2. Four-point bending lab setup

2. Background

There exists a competitive market for residential floor systems. It is important to understand how the FastFloor Residential prototype fits in comparison to these systems. Thus, a brief literature review of available systems is provided. Three proprietary systems, and three nonproprietary systems, are discussed including composite steel deck, dovetail steel deck, cold-formed steel joists, Ecospan, iSpan, and Hollow-core Plank to provide a sample of the current market.

Composite metal deck [1], as shown in Figure 3, is nonproprietary and common system particularly for longer spans. It consists of steel deck, shear studs, mesh reinforcement, and concrete. This provides a shallow deck profile but can provide up to a two-hour fire rating. For spans shorter than 14 ft, no shoring is required, making this system convenient, although, there is wet concrete needed that takes time to set on site and may involve multiple trades and the system is relatively heavy

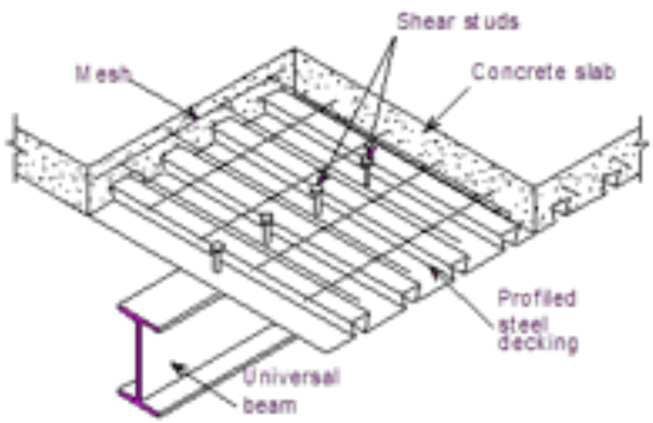


Figure 3. Composite deck rendering [2]

Dovetail deck [3], as shown in Figure 4, is also a common nonproprietary floor system. This system is optimized for slightly longer spans with the composite dovetail metal deck system providing up to 20 ft. Dovetail deck also provides a shallow floor system, but it commonly requires a thicker slab of at least 6 in. to satisfy acoustical requirements. The thicker slab helps provides this system with a fire rating of up to three hours.

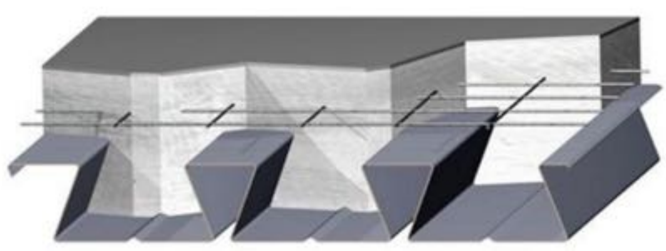


Figure 4. Dovetail deck rendering [4]

Cold-formed steel joists [5], as seen in Figure 5, is another nonproprietary floor ideal for spans around 20 ft or less, but has a very different design. It does not require any wet concrete allowing for quick construction, but is a deeper system, commonly greater than 8 in., typically with 12 in. to 24 in. spacing. Oriented strand board or cementitious panels are used as topping on the joists. A one-hour or two-hour fire rating can be achieved based on final detailing. Blocking against the webs is sometimes required to prevent torsion in the joists. The system is relatively light, but requires more steel to achieve its stiffness compared with some other solutions, and this can in some cases be costly.

Figure 5. Cold-formed steel joists rendering [6]

Ecospan [7], as shown in Figure 6, is a longer span proprietary system by Vulcraft. This open web steel joist-based system is a more complex solution that has between 10 in. and 30 in. depths depending on spans and load. The open web steel joists are spaced up to 60 in. apart. The system uses a composite deck in addition to the joists, and Ecospan uses proprietary Shearflex screws rather than conventional shear studs to provide the composite action between the concrete and steel deck. This system has up to a three-hour fire rating.

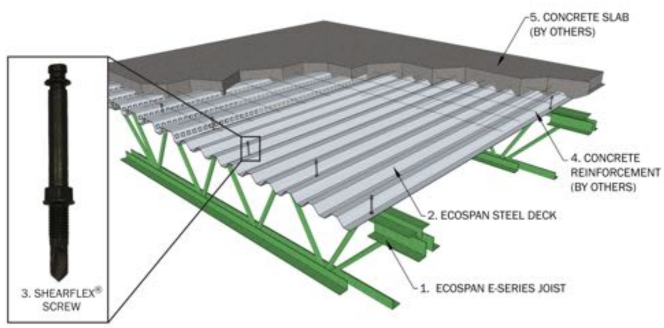


Figure 6. Ecospan rendering [7]

iSpan [8], as seen in Figure 7, is a proprietary version of cold-formed steel joists for fast floor assembly. The joists have I-shaped cross-sections for greater torsional stability and longitudinal stiffeners for improved strength-to-weight when compared with standard cold-formed steel C-sections. The iSpan joists are typically spaced between 12 in. and 24 in., but the system can achieve larger spans of up to 28 ft. With typical detailing, iSpan has a one-hour fire rating.

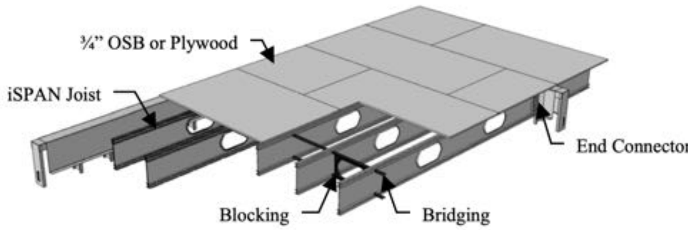


Figure 7. iSpan rendering [8]

Hollow-Core Plank [9], as seen in Figure 8, is a shallow proprietary concrete modular floor system for spans between 11 ft to 46 ft. This system has more concrete than the other systems investigated which makes this system heavier, but also able to provide a fire rating of up to four-hours. This system is precast meaning that no wet concrete is required on site, but grout to fill voids between panels is required. A cast in place topping can be added to improve capacity and acoustics. (Which may also be done with many of the other systems detailed here as well).

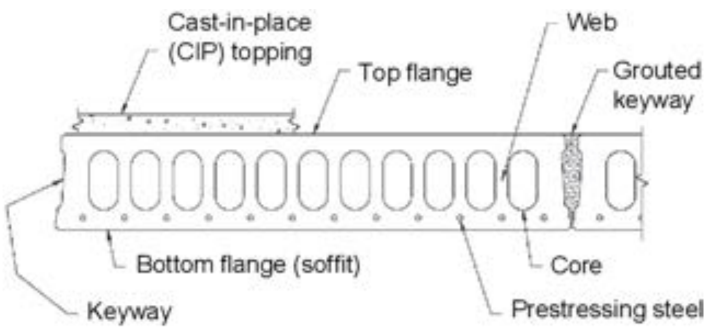


Figure 8. Hollow-Core Plank rendering [10]

Taken together, one can see that a rich array of different solutions exist for providing acceptable floor systems at spans and loads consistent with residential needs. All solutions must provide acceptable vibration, fire, and acoustic performance in addition to structural considerations. In general, solutions may be separated into those that use wet concrete and those that do not, and between heavy and light systems. For the exercise herein, we are focused on dry systems that are relatively light – and examining simple innovations that may provide a solution under those constraints.

3. Test Matrix and Specimens

Twelve specimens were tested in flexure with six unique configurations. All specimens used #12-14×3/4 Hilti fasteners for the deck-to-deck connections and #8 × 1-5/8" Grabber® fasteners for the deck-to-cementitious panel connection. For the cementitious floor panels 4 ft × 8 ft × ¾" structo-crete® structural panels were provided by USG. The 18 gage (43 mil) 3 in. deep N-deck was provided by DACS Inc. The unique configurations consist of tests with and without cementitious panels as well various combinations of fastener spacings to achieve various amounts of composite action between the materials, as summarized in Table 1. When referring to the panel, partially composite refers to a 12 in. panel fastener spacing and fully composite refers to a 6 in. fastener spacing. When referring to the deck, partially composite refers to an 8 in. deck fastener spacing and fully composite refers to a 4 in. fastener spacing. For all specimens, the deck fasteners have 1 ft at each end with 2 in. spacing to prevent the top and bottom decks from pulling apart due to the large support reaction forces. The layout of the fasteners is provided in Figure 9.

Table 1. Test matrix

Name	Deck Fastener Spacing (in.)	Panel Fastener Spacing (in.)	Quantity (#)
FC[1] Deck	4	--	2
FC Deck + PC[2] Panel	4	12	2
FC Deck + FC Panel	4	6	2
PC Deck	8	--	2
PC Deck + PC Panel	8	12	2
PC Deck + FC Panel	8	6	2

[1] Fully Composite
[2] Partially Composite

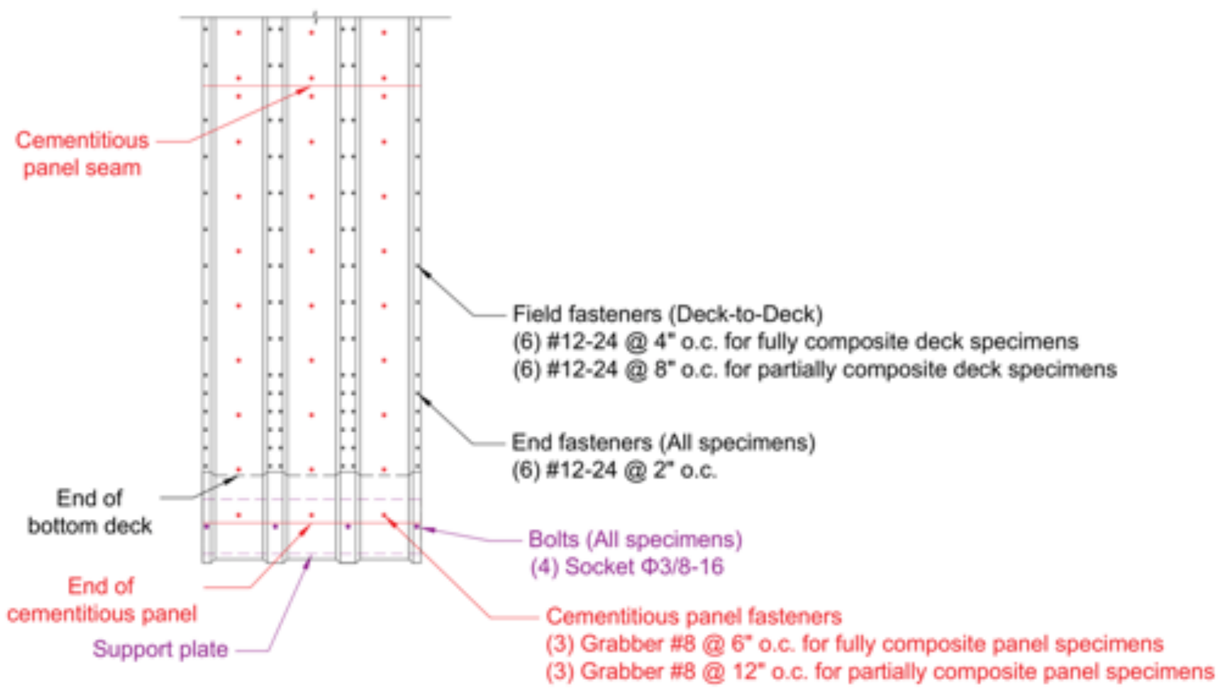


Figure 9. Layout of the fasteners to the deck

Stiffeners made from 600S137-68 lipped channels and 2×4's were added under the load points to ensure the specimen would not fail locally due to web crippling, as shown in Figures 10 and 11.

Figure 10. Stiffeners placed inside the specimen during fabrication

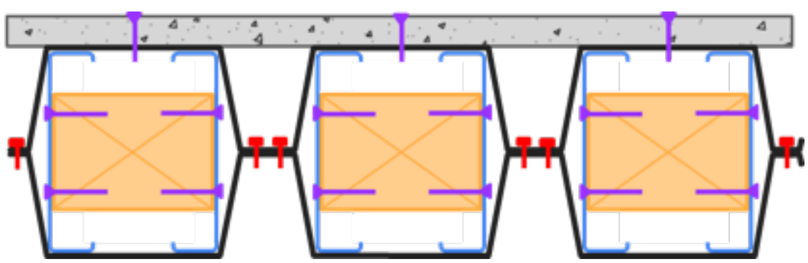


Figure 11. Stiffeners inside the full specimen

4. Test Setup

The four point bending tests were conducted on pin-roller end support conditions by using one 2 in. diameter free roller and one 2 in. diameter roller blocked with wood at the specimen ends. The specimens themselves had a longer top deck than the bottom deck to accommodate the support connections at each end. The top deck has ¾ in. bearing plates at the end support rollers that were wide enough to prevent web crippling. The end supports were affixed to the ground 16 ft apart with a primary W8x35 beam to distribute the load to two load points, as shown in Figures 12 and 13. The actuator was connected to a loading frame with swivel joints on both ends of the actuator. This allowed the specimen to deform freely. The loadcell was connected between the actuator and the swivel joint that attached to the main spreader beam. The spreader beam applies the point loads 5 ft apart from each other, with two fixed rollers to transfer the load from the primary loading beam to two HSS 6×6×5/8 secondary spreader beams. To prevent friction between the specimen and secondary spreader beams, 1/8 in. thick Teflon plates were put between the bottom face of the secondary HSS spreaders and the top face of the specimen.

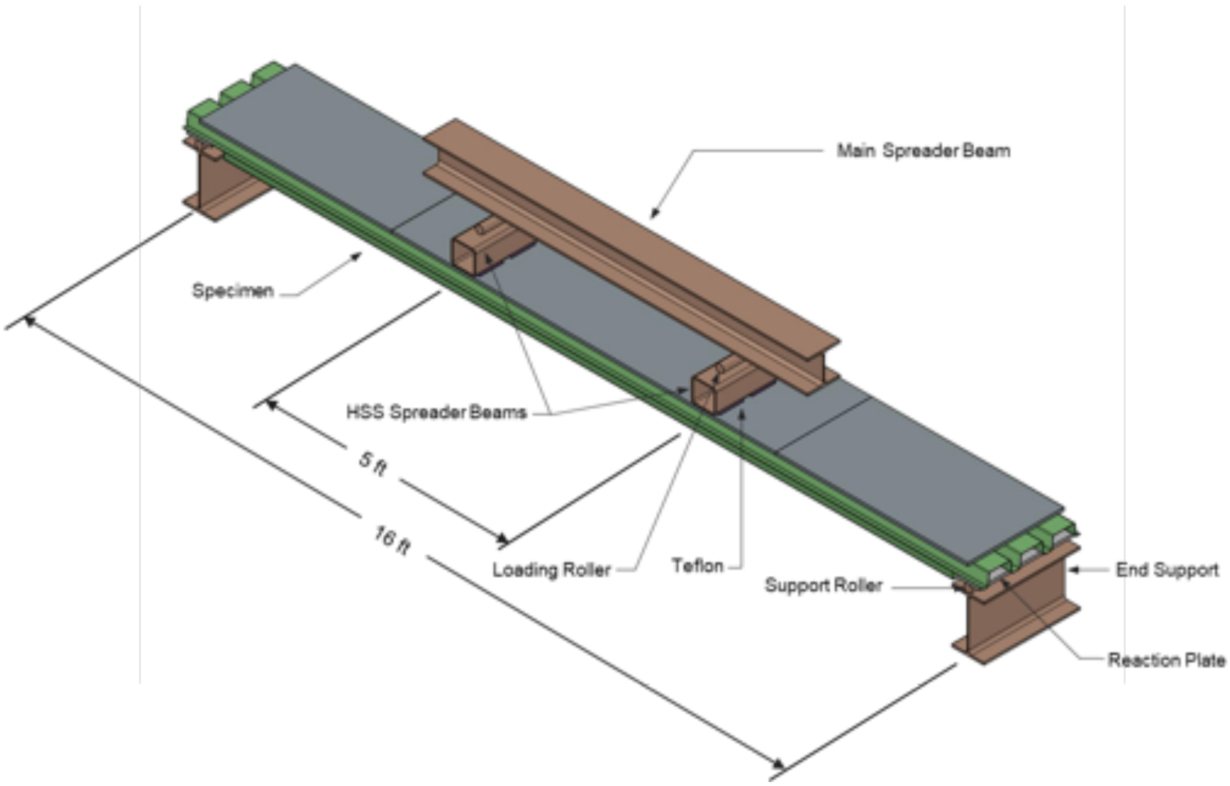
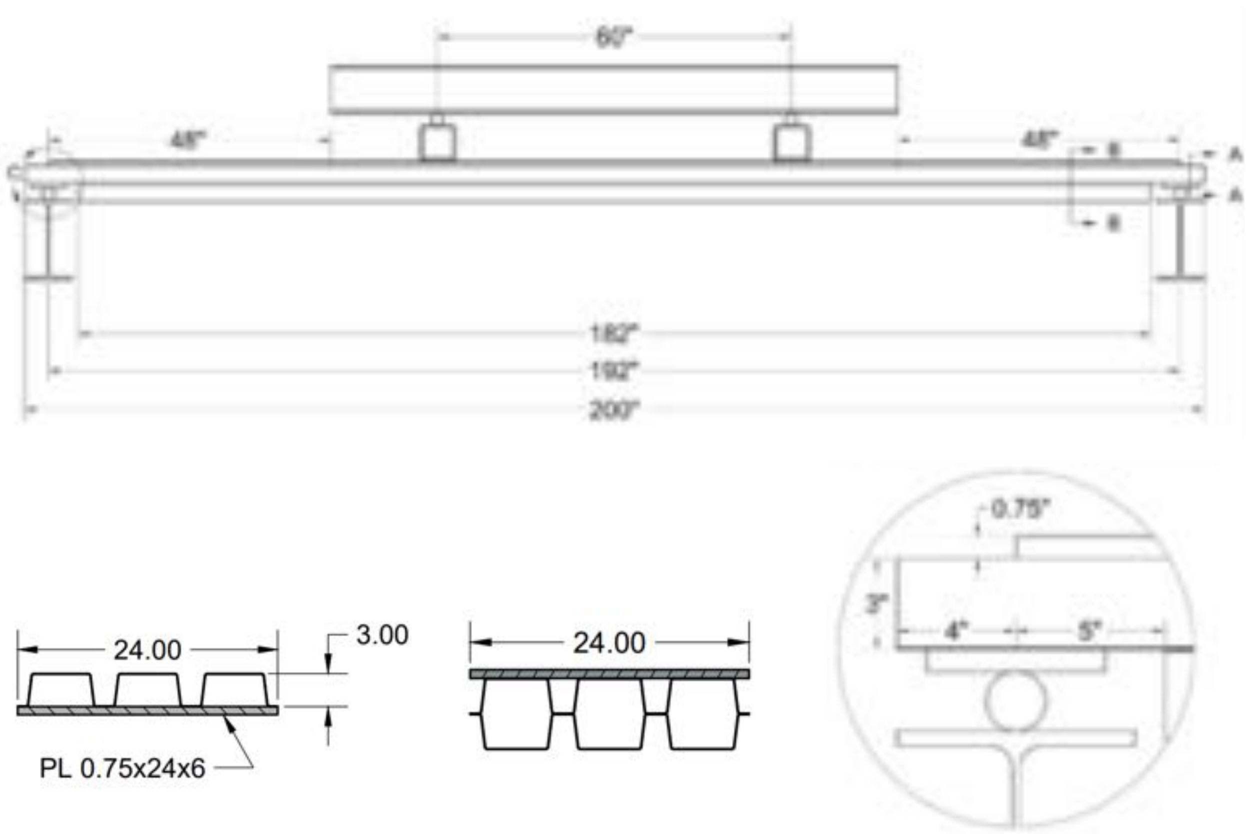


Figure 12. Lab setup schematic - Isometric view



Section A-A Section B-B Detail C

Figure 13. Lab setup schematic - Elevation view

5. Instrumentation and loading procedure

Six position transducers were used to measure deflection of the specimen with one at each side of the specimen at the midspan of the specimen and under both load points (PT1-PT6). The slip between the top and bottom deck and the slip between the deck and the structural panel was measured on both sides of the specimen with four additional position transducers (PT7-PT10). The location of the PT's can be seen in Figures 14 and 15.

The applied load in the tests were controlled using an MTS 407 controller. A displacement loading procedure was used at a rate of 0.0018 in/sec. The data acquisition system was a National Instruments NI cDAQ-9174. Timelapse photos were taken every 10 seconds using a Canon EOS Digital Rebel XS camera.

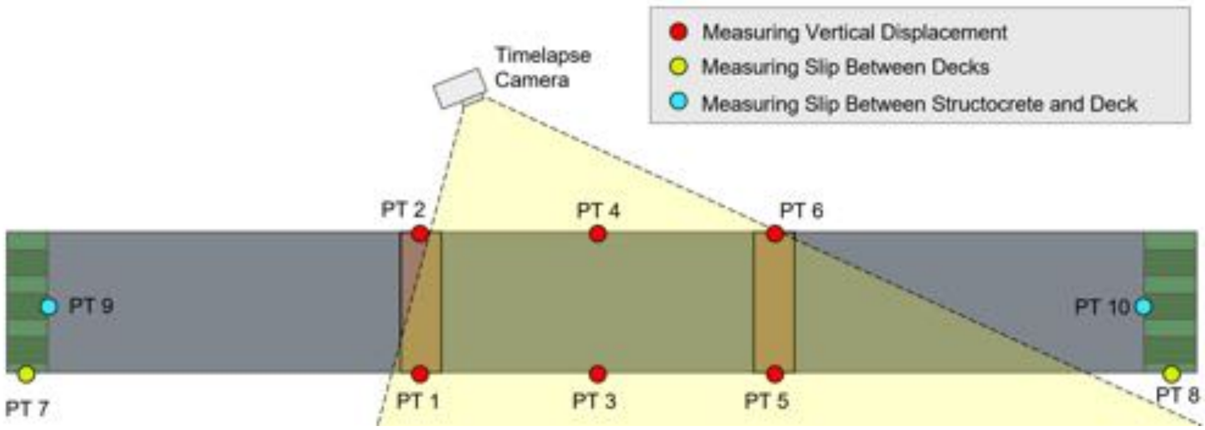


Figure 14. Position transducer sensor layout

Figure 15. Position transducer locations

6. Material Properties

Six coupon tests were conducted per ASTM A370-21, as shown in Figure 16.

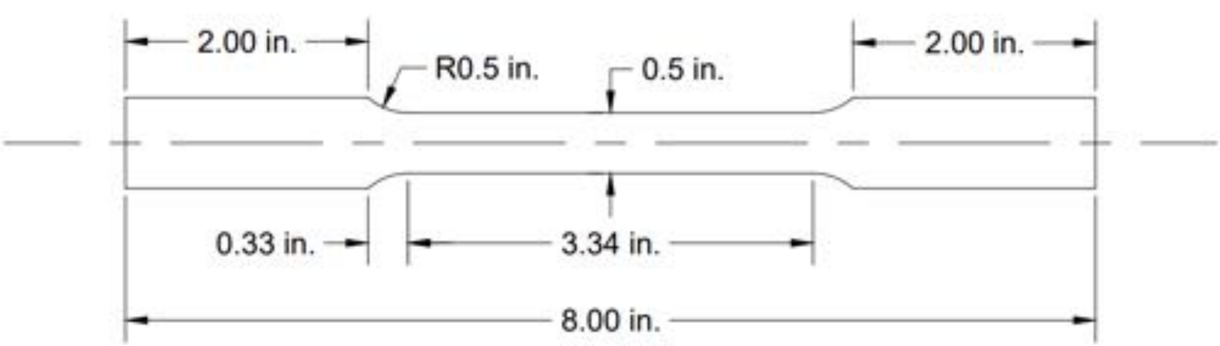


Figure 16. A370-21 coupon dimensions

The tests were conducted using an MTS Criterion Model 43 tensile testing rig using a load rate of 0.001 in/sec. The samples were taken from the ends of the first test specimen to determine the material properties of the steel deck, as provided in Figure 17.

Figure 17. Tensile test of deck coupon

An extensometer with a 1 in. gage length was used for all tests. The extensometer was removed just before the specimen reached 20% strain, therefore ultimate and fracture strain was calculated using the overall displacement of the MTS crosshead. These results are summarized below in Table 2, where the average F_y was determined using the 2% offset method to be 58 ksi. The stress strain curves from this test are also shown in Figure 18.

Table 2. Coupon test results

Coupon (#)	w (in.)	t (in.)	F_y (ksi)	F_u (ksi)	ε_y (%)	ε_u (%)	$\varepsilon_{Fracture}$ (%)
1	0.502	0.0458	57.87	71.97	0.45	16.7	28.3
2	0.501	0.0461	58.73	74.03	0.47	16.4	25.8
3	0.499	0.0461	58.14	72.14	0.45	16.6	26.1
4	0.501	0.0474	57.65	71.45	0.45	16.6	26.7
5	0.498	0.0475	56.79	71.36	0.46	16.1	23.4
6	0.502	0.0464	59.00	73.46	0.50	16.2	24.8
Mean	0.501	0.0465	58.03	72.40	0.46	16.5	25.9
COV (%)	0.29	1.55	1.37	1.52	3.76	1.39	6.52

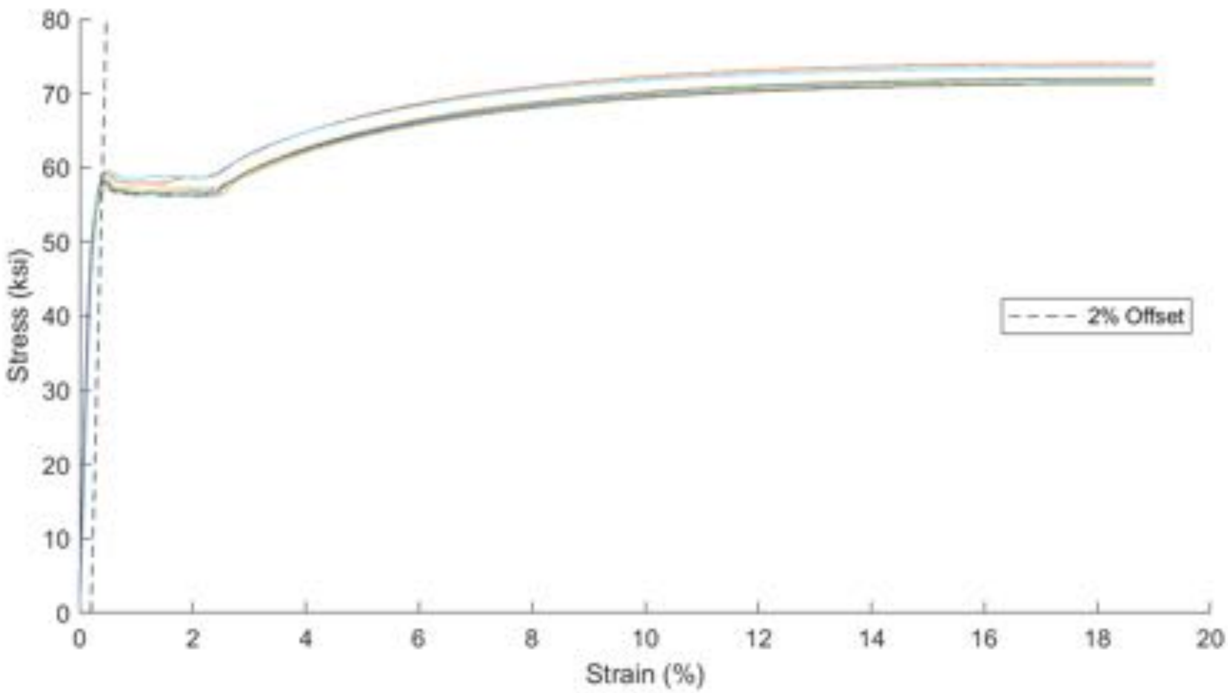


Figure 18. Stress-strain plot of the coupon tests

7. Test Results

The overall load and deflection of the specimens is provided in Figure 19 based on actuator displacements and load. A more acurate load and deflection curve, utilizing the position transducers, is provided in Figure 20, but only for the ten test specimens where the position transducers were properly in place and recorded. In some tests the position transducers registered their maximum deflection (stroked out) prior to peak load which results in Figure 20 appearing to show drops in load with no change in deflection when the specimen in reality, continued to deflect around the 5 in. mark.

The first test of the bare steel specimen with the fully composite deck to deck fastener spacing as well as the the first test of the fully composite panel and fully composite deck specimen were shakedown tests for this study and were conducted before the position transduceres were setup and therefore only have data for the load and displacement of the crosshead. The peak loads are reported in Table 3.

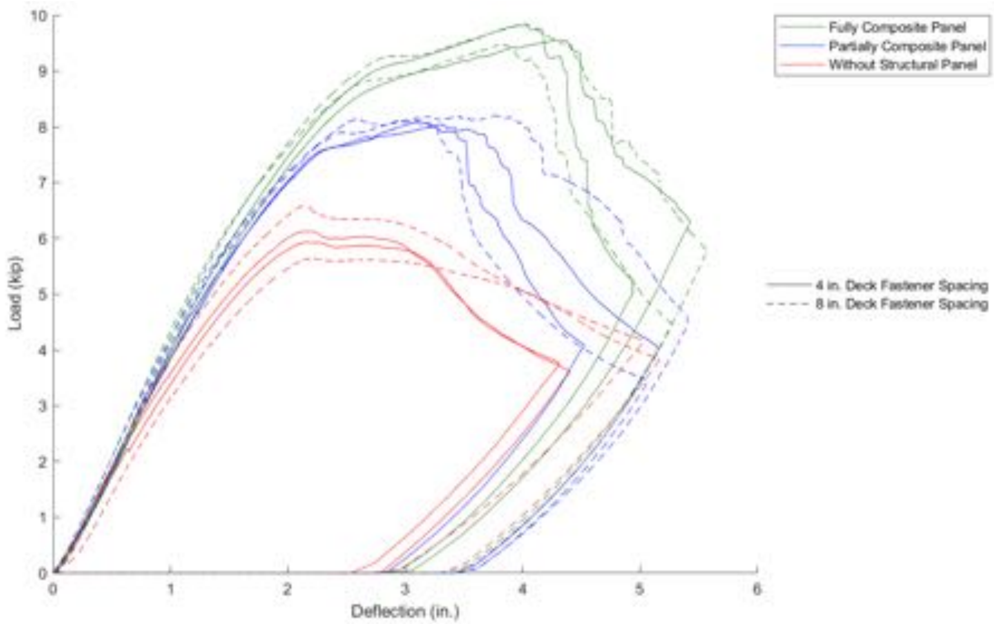


Figure 19. Load-displacement plots based on the actuator loadcell and cross head displacement

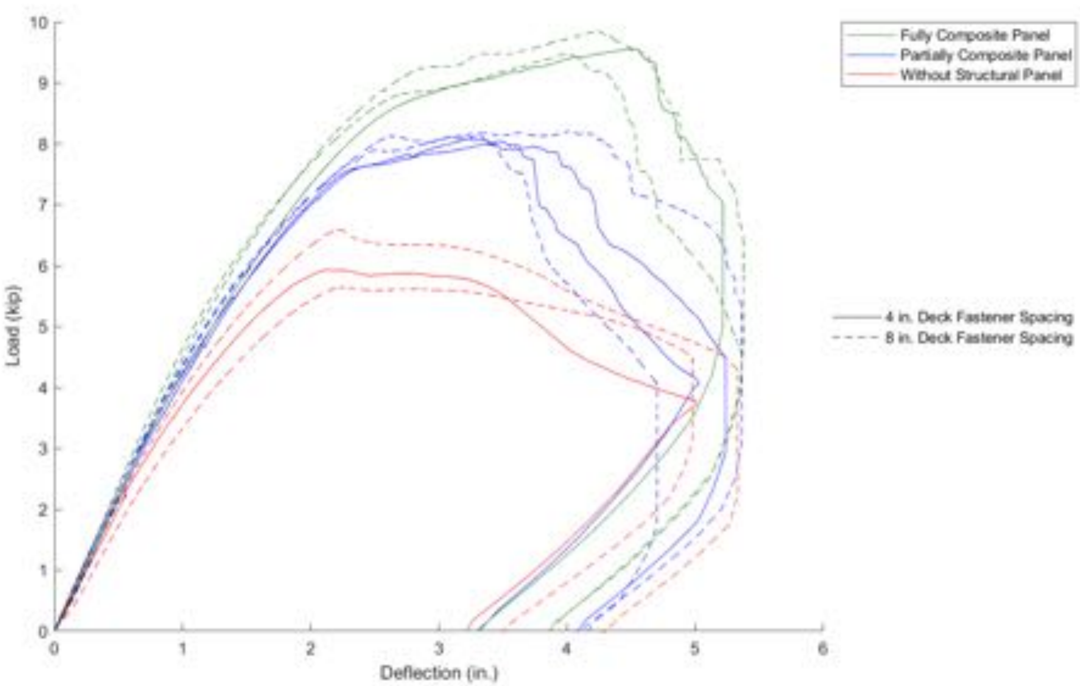


Figure 20. Load-displacement plots based on position transducers at midspan

Figure 21 provides the moment rotation plots for the specimens. Here the rotation is the chord angle at the support, as shown in Figure 22 and the moment is from Equation 1. The peak moments for all tested specimens are reported in Table 3.

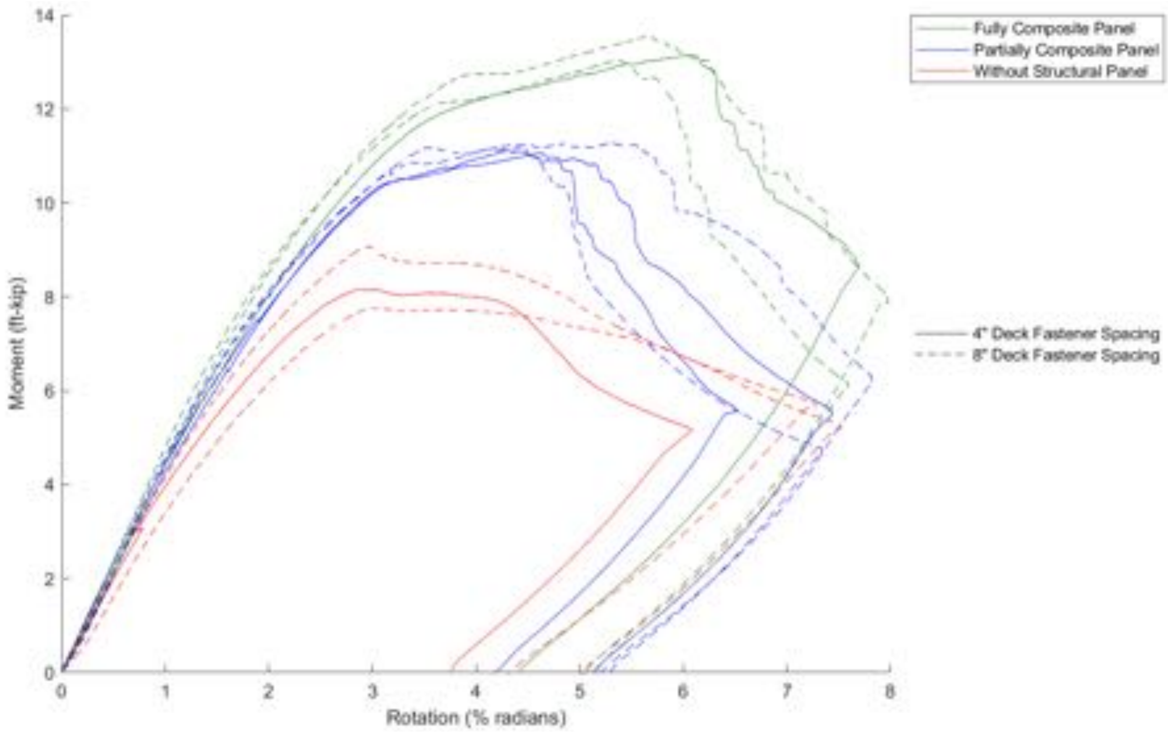


Figure 21. Moment rotation plot

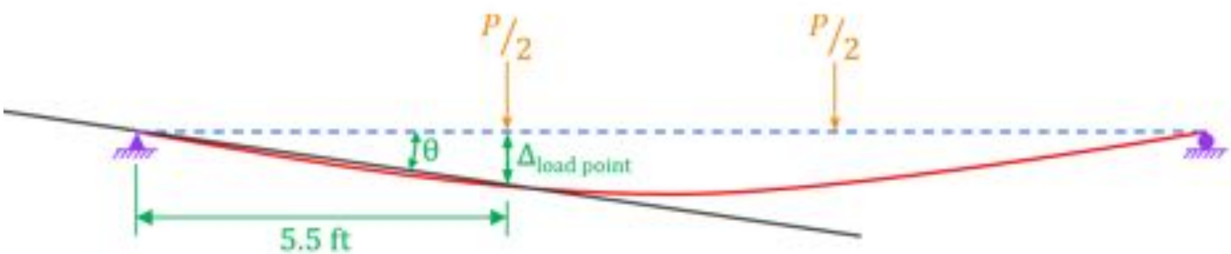


Figure 22. Rotation measured based on average of all position transducers at the load points

$$M = \left(\frac{P}{2}\right) \cdot \frac{5.5\ \text{ft}}{2} \qquad \text{Equation 1}$$

Adding cementious (structural) panels had a clear impact on the specimen capacity and stiffness, while the spacing of the deck-to-deck fasteners had only a negligible effect on the specimen capacity. Note, the baseline bare steel deck specimen with the lowest capacity was slightly warped before testing and therefore did not completely lay flat on the rollers throughout the test, potentially leading to some reduction in gross strength.

The capacity and stiffness of the specimens are summarized in Tables 3 and 4, respectively. The experimental EI is calculated from the position transducers at the midspan of the specimens. The Analytical EI is calculated using 29500 ksi for the modulus of elasticity of the steel and 747 ksi for the modulus of elasticity of the cementitious panel, as detailed in Appendix 2.

Table 3. Four-point bending ultimate capacity

Specimen Name	Load, P_{ult} (kip)	Moment, M_{ult} (kip-ft)	Avg. M_{ult}, $M_{ult,avg}$ (kip-ft)	M/M_{bare}	M/M_y*
FC[1] Bare Deck	6.13 5.93	8.43 8.16	8.30	1.00	0.28
FC Deck + PC[2] Panel	8.05 8.09	11.06 11.13	11.10	1.34	0.33
FC Deck + FC Panel	9.82 9.56	13.50 13.14	13.32	1.60	0.40
PC Bare Deck	5.64 6.59	7.75 9.06	8.41	1.00	0.28
PC Deck + PC Panel	8.14 8.21	11.19 11.29	11.24	1.34	0.34
PC Deck + FC Panel	9.48 9.85	13.03 13.54	13.29	1.58	0.40

* M_y = 30.1 kip-ft and 33.5 kip-ft for the bare deck and full specimen, respectively – Both calculated in Appendix 2

[1] Fully Composite

[2] Partially Composite

Table 4. Stiffness results

Specimen Name	Analytical EI - Fully Composite ($10^5 \times$ kip-in^2)	Experimental EI - at 40% P_{max} ($10^5 \times$ kip-in^2)	Ratio of Analytical EI
FC[1] Bare Deck	5.47	-- 5.25	-- 0.96
FC Deck + PC[2] Panel	6.84	5.70 5.45	0.83 0.80
FC Deck + FC Panel	6.84	-- 5.52	-- 0.81
PC Bare Deck	5.47	4.46 5.36	0.82 0.98
PC Deck + PC Panel	6.84	5.85 5.67	0.86 0.83
PC Deck + FC Panel	6.84	6.06 5.78	0.89 0.84

[1] Fully Composite

[2] Partially Composite

8. Test observation

The bare steel deck specimens and the specimens with the cementitious panel added to the top deck acted substantially different from one another. Using timelapse photos synced with the data collection, the behavior can be understood in alignment with the overall load-displacements response.

The behavior of the bare steel deck specimens were as follows. The load and deflection acted faily linearly until buckling in the top deck formed mechanisms and plasticized, as shown in Figure 23. The deflection then continued with a fairly constant load until the plastic mechanism formed in the bottom deck. Once the bottom deck began to contribute to the plastic mechanism, the specimens continued to deflect with significant drop in load, as shown in Figure 24.

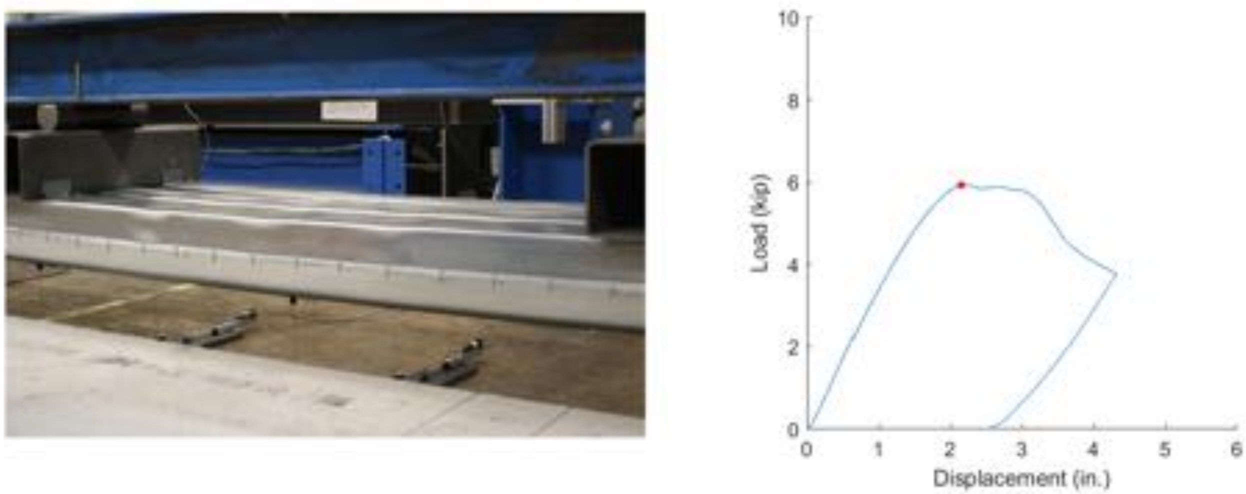


Figure 23. Initial buckling and yielding in the top deck of a bare steel deck specimen

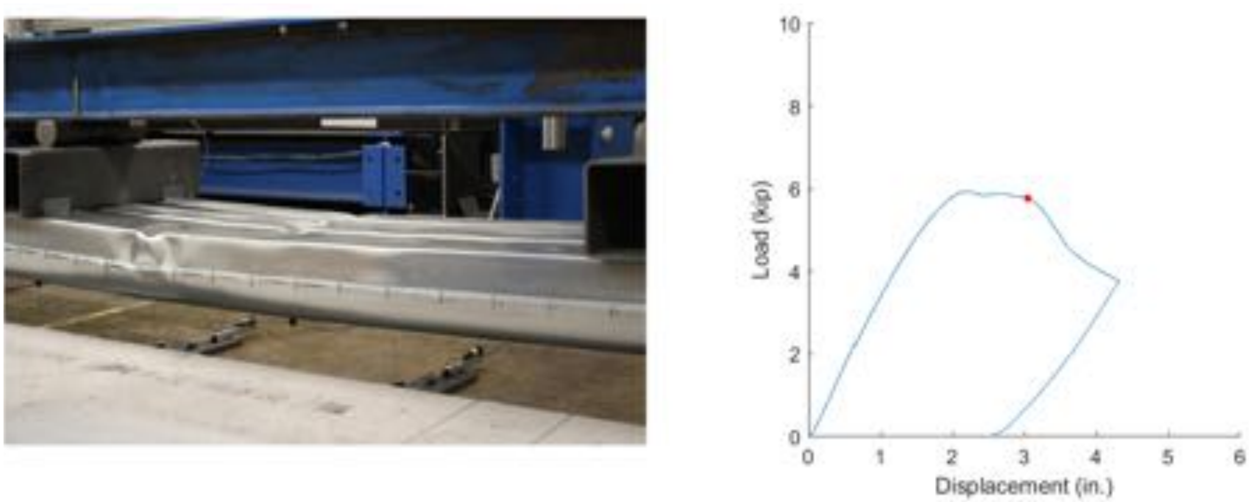


Figure 24. Substandial mechanisms in top deck and Initial buckling and yielding in the bottom deck of a bare steel deck specimen

The behavior of the specimens that included the cementirous (structural) panels was slightly different. The load and deflection again acted fairly linearly until buckling in the top deck beagn to form plastic mechanisms, as shown in Figure 25. The deflection then continued, with a reduced stiffness, but with an increase in capacity. Presumably, the cementitious panel was only fully engaged once the steel mechanisms formed and the majorty of compression had to be taken by the panel. This behavior continued until fasteners connecting the cementitious panel to the steel deck began to experience shear failure, as shown in Figure 26. Once the bottom deck contributed to plastification, the specimens continued to drop load, but not as quickly, as shown in Figure 27. This part of the failure is more ductile than the shear failure of the fasteners.

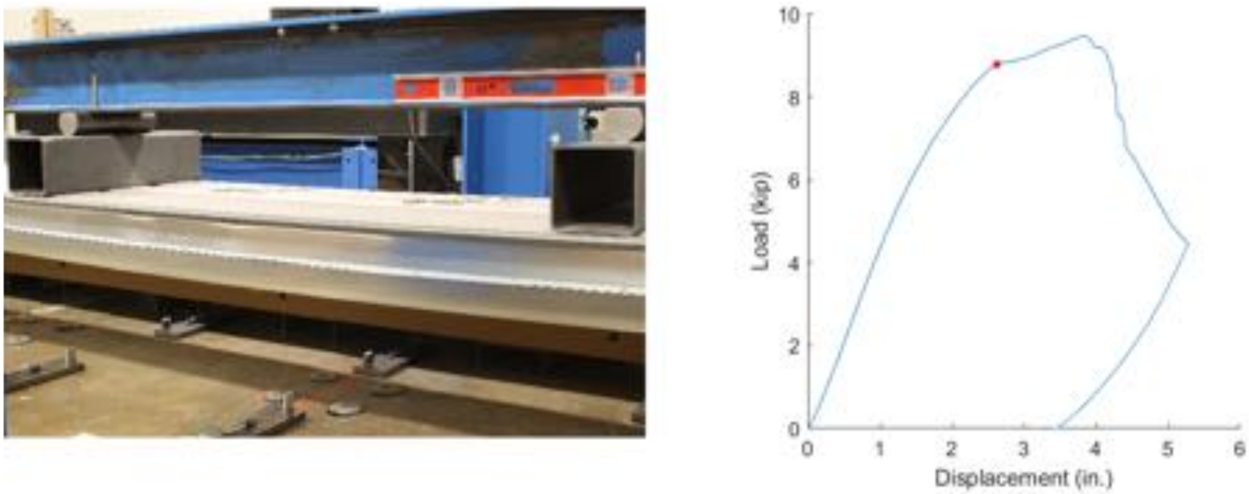


Figure 25. Initial buckling and yielding in the top deck of a specimen with cementitous panel

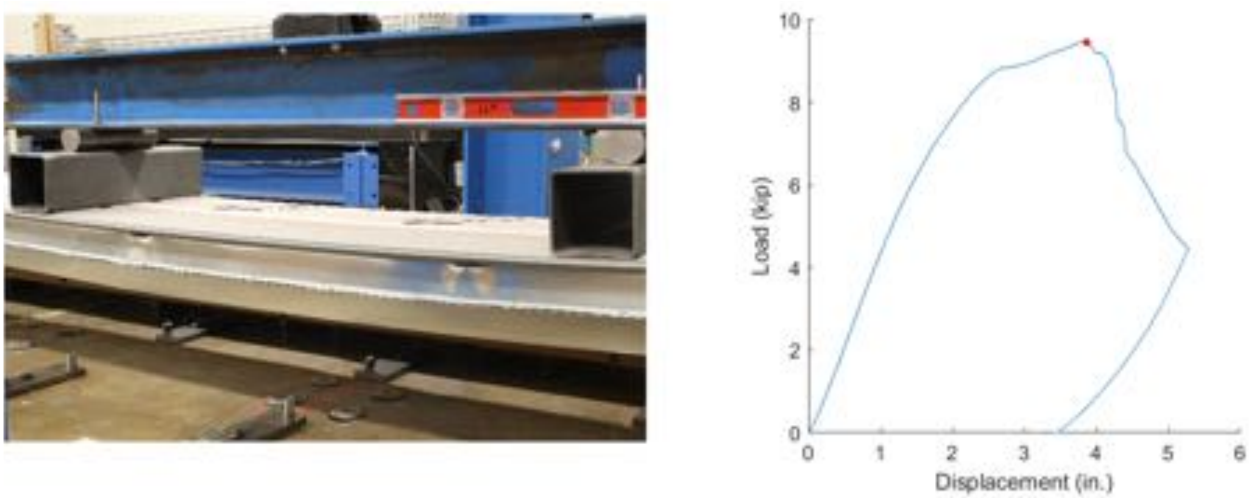


Figure 26. Panel fasteners begin failing in shear as peak load is reached

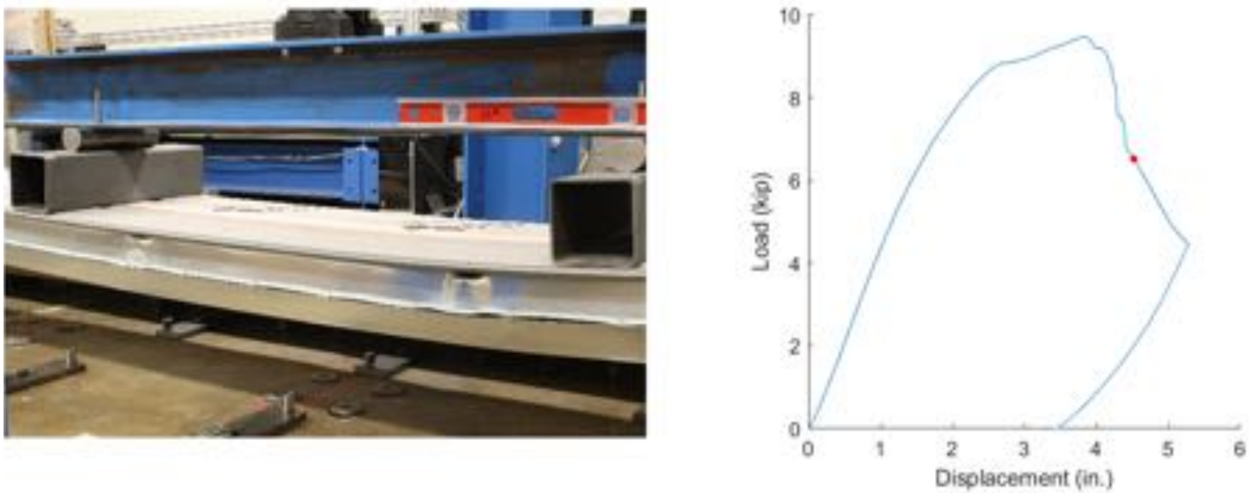


Figure 27. Yeilding in the bottom deck of a specimen during descending branch of structural response

All the specimens at their ultimate capacity are provided in Figures 28 and 29. All the specimens at final test condition are provided in Figures 30 and 31.

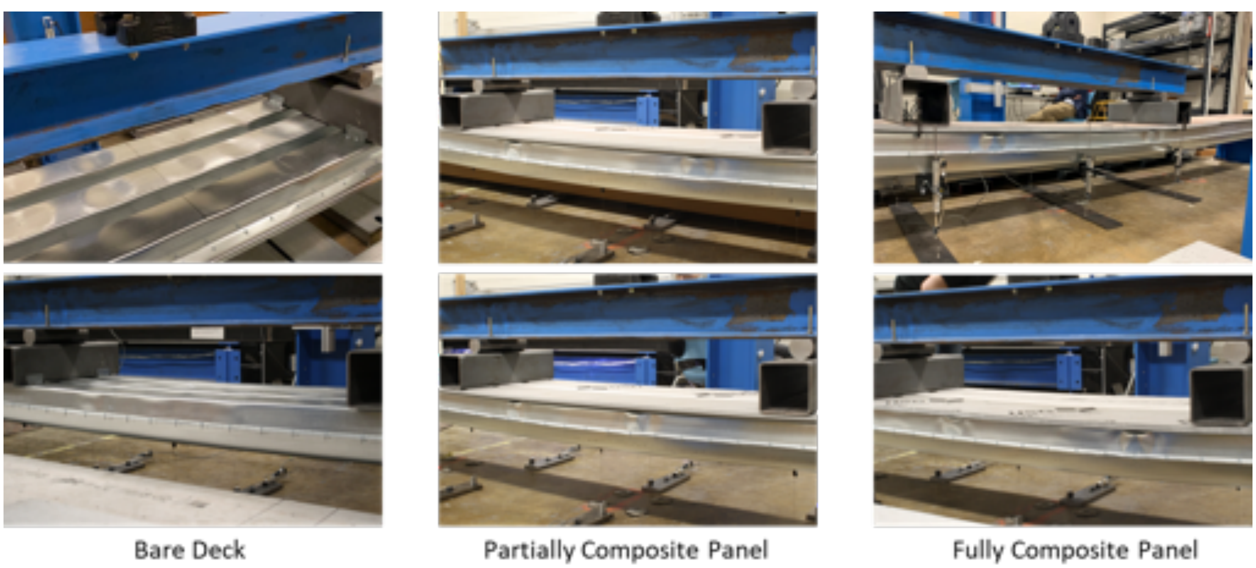


Figure 28. Ultimate capacity of all tests with fully composite deck-to-deck connections

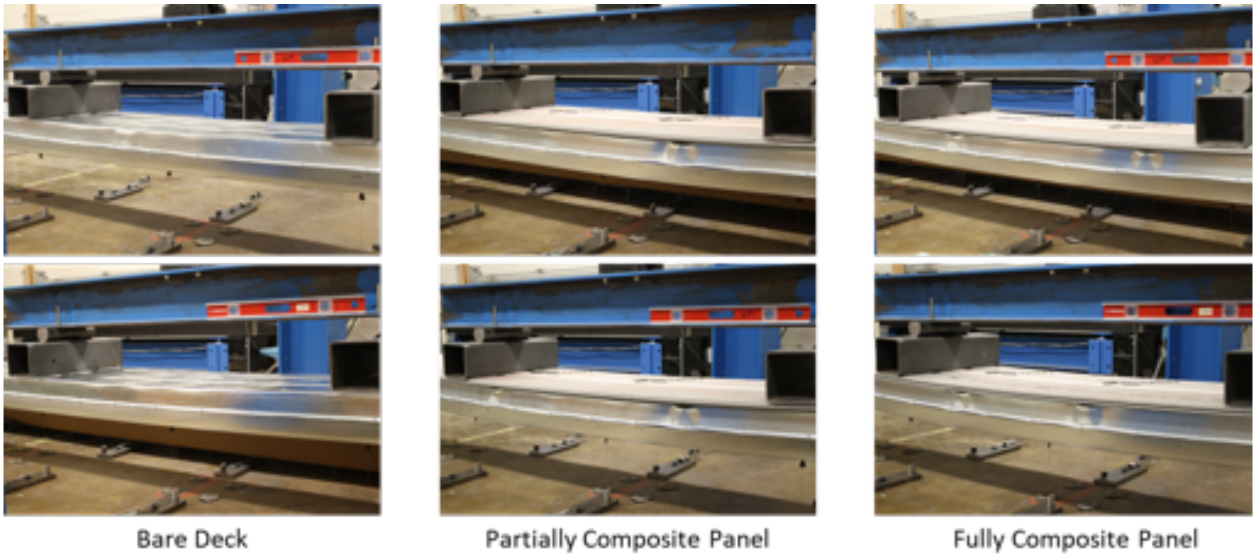


Figure 29. Ultimate capacity of all tests with partially composite deck-to-deck connections

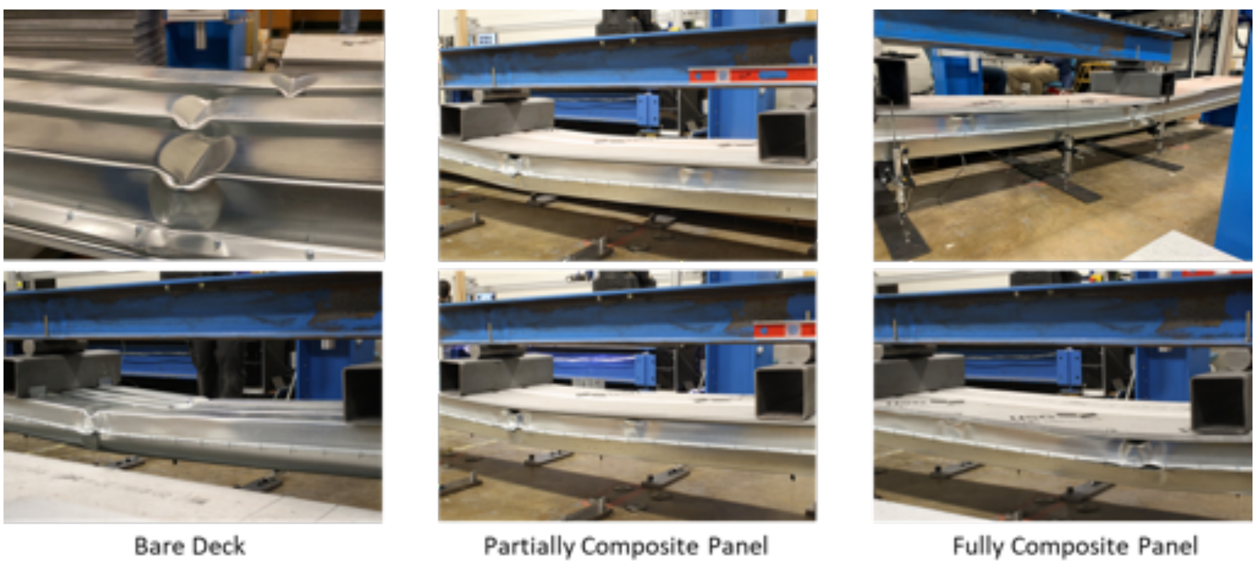


Figure 30. Final condition of all tests with fully composite deck-to-deck connections

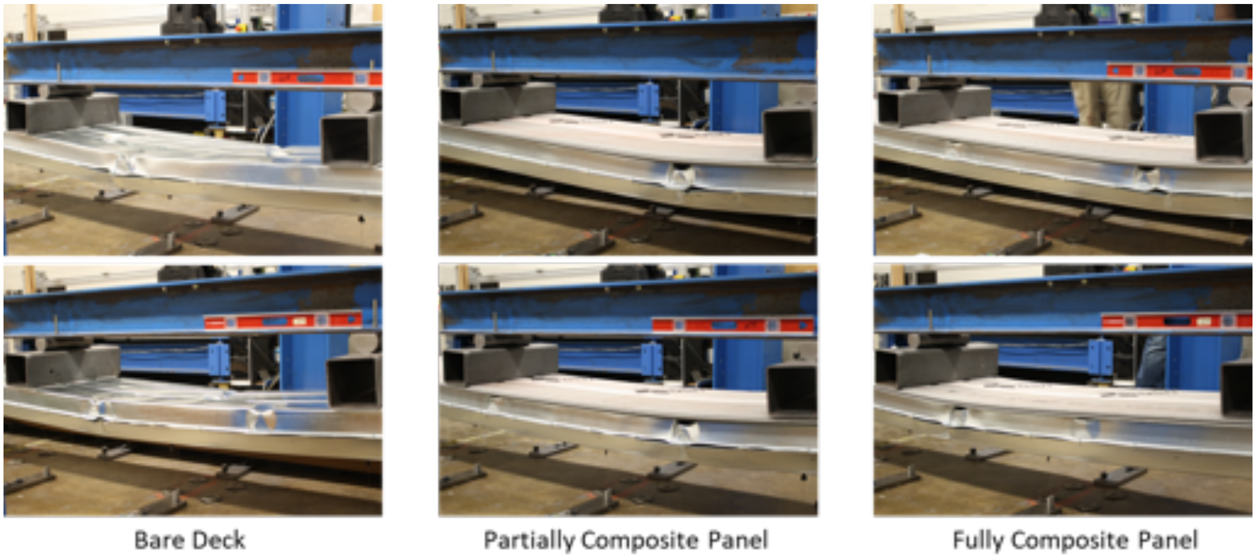


Figure 31. Final condition of all tests with partially composite deck-to-deck connections

For the tests with cementitious panels attached to the top deck. The shear failure of the fasteners was apparent after the test. The fasteners typically failed on one side of each specimen's primary buckling location, as shown in Figure 32. These failures occurred with loud popping sounds. The fasteners caused uplift on the structural panels on the side where the shear failures occurred, as shown in Figure 33.

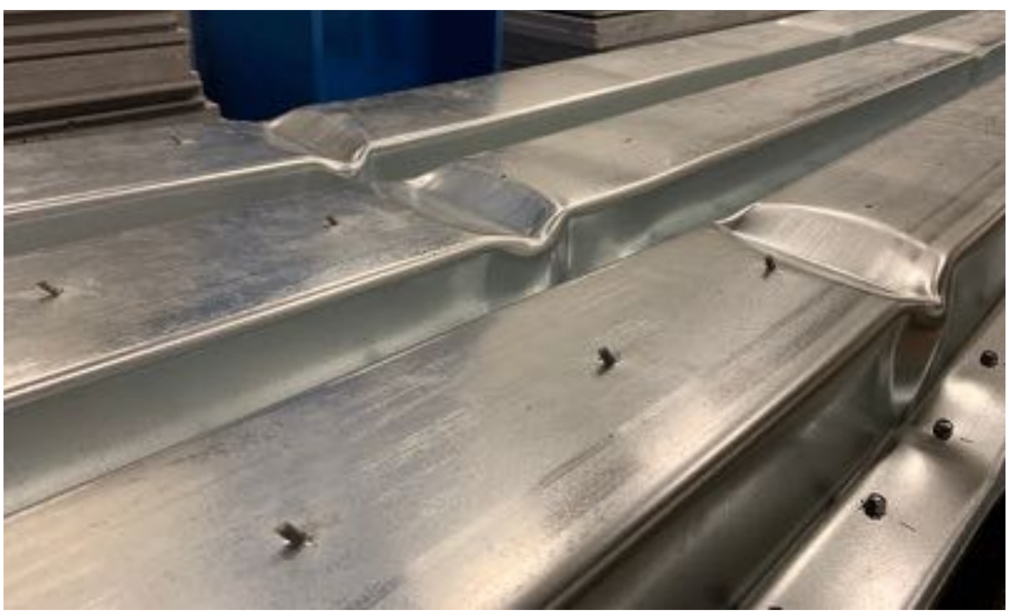


Figure 32. Shear failure of the panel fasteners

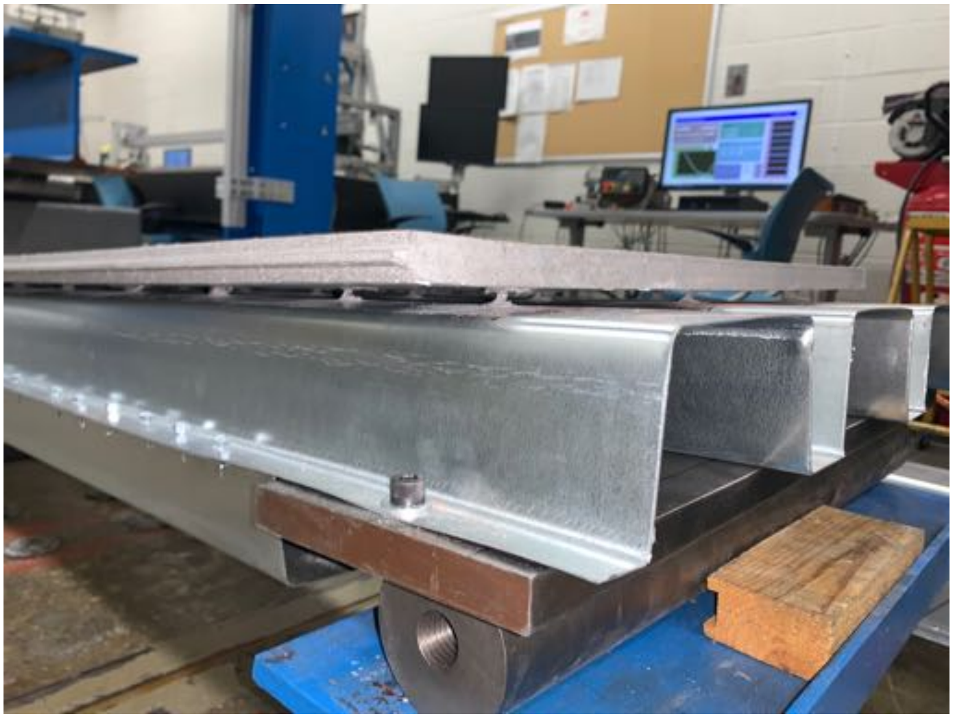


Figure 33. Uplift of structural panel due to shear failure of the panel fasteners

9. Position Transducer Sensor Data

The first fully composite (FC) deck specimen and the first FC Deck + FC Panel specimen, were shakedown tests and did not have position transducer sensors in place. All other specimens' sensor data are deatiled below in Figures 34 to 43.

Although the tests are nominally symmetric, failure localizes and all of the tests show one shear span slipping slightly more than the other shear span, with respect to the the deck-to-deck slip. The deck-to-deck slip was measured by PT sensors 7 and 8, as labeled in Figure 14, by clamping the sensor to the top deck and magnetically attaching the extention rod to the bottom deck. Once peak load was reached, the deck-to-deck slip stopped increasing as the weakest link was no longer deck-to-deck movement but rather at the plastic mechanisms where failure was occuring. For specimens with cementitious (structural) panels, The deck-to-panel slip was measured by PT sensors 9 and 10, also labeled in Figure 14, by clamping the sensor to the top deck and the extention rod was kept in place aginst the end of the cementitious (structural) using a rubberband. The deck-to-panel slip was also asymmetric with one shear span side measuring no further slip after peak load while the other would continue to slip. This continued slip always happened on the side of the specimens that the panel fasteners failed on. The slip of one side of the deck can best be observed in Figures 44 to 53. These figures also show that the panels slip much more than the decks throughout the tests.

It should be noted that the partially composite (PC) Bare Deck – Test 1 specimen was slightly warped and when the load reached 2.77 kips, the ends of the specimen quickly warped the other way, which had little to no impact on the displacement PT sensors, but impacted the deck-to-deck slip PT sensors substantially.

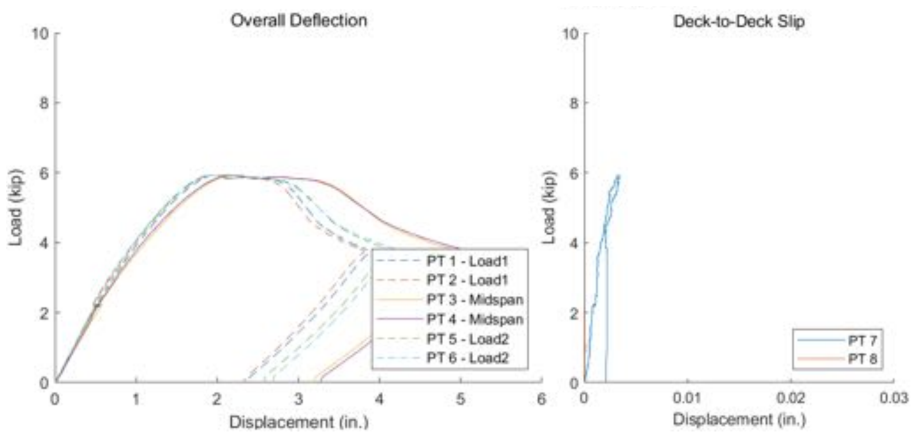


Figure 34. FC Bare Deck - Test 2 - Position Transducer Data (PT data not available for Test 1)

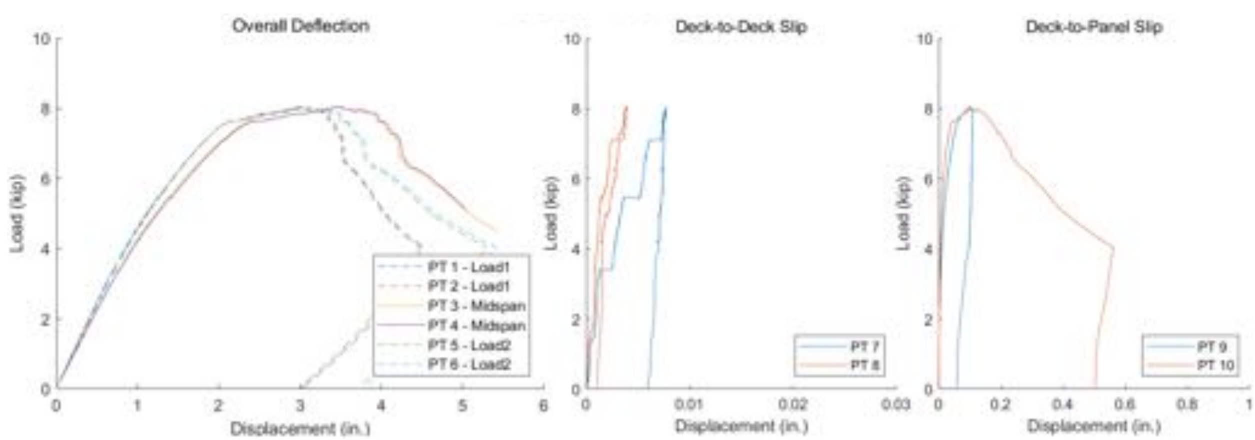


Figure 35. FC Deck + PC Panel - Test 1 - Position Transducer Data

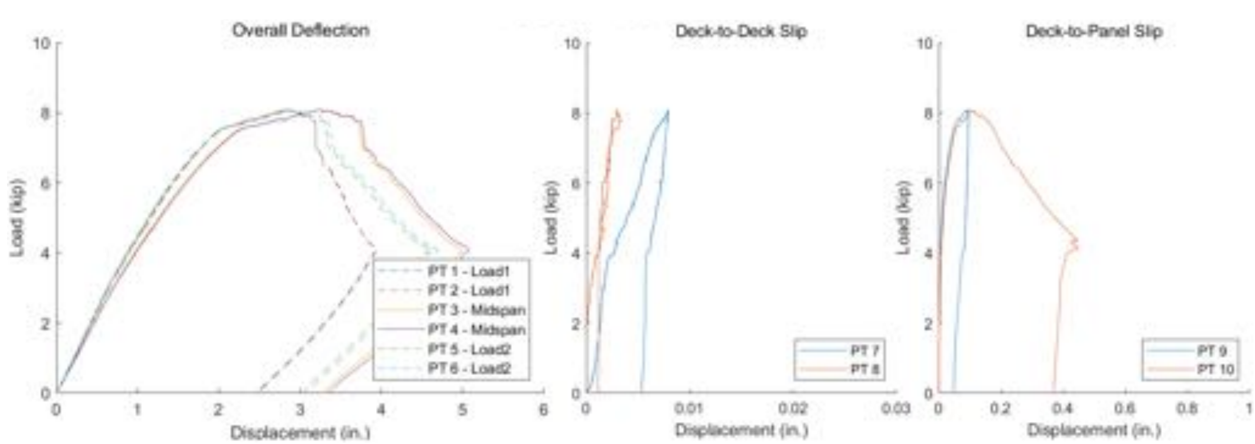


Figure 36. FC Deck + PC Panel - Test 2 - Position Transducer Data

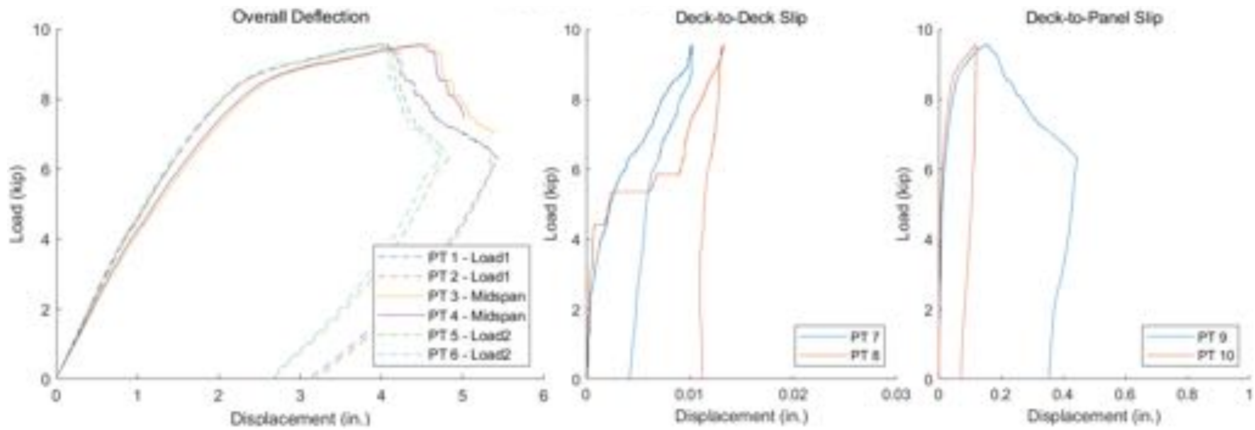


Figure 37. FC Deck + FC Panel - Test 2 - Position Transducer Data (PT data not available for Test 1)

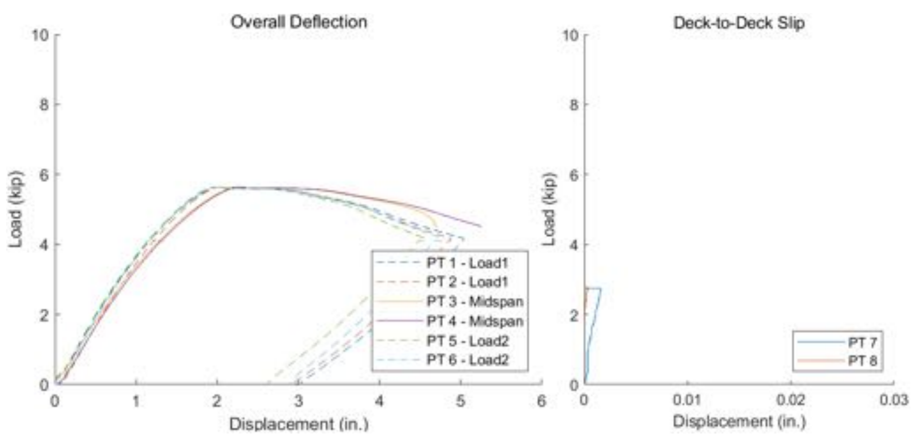


Figure 38. PC Bare Deck – Test 1 - Position Transducer Data

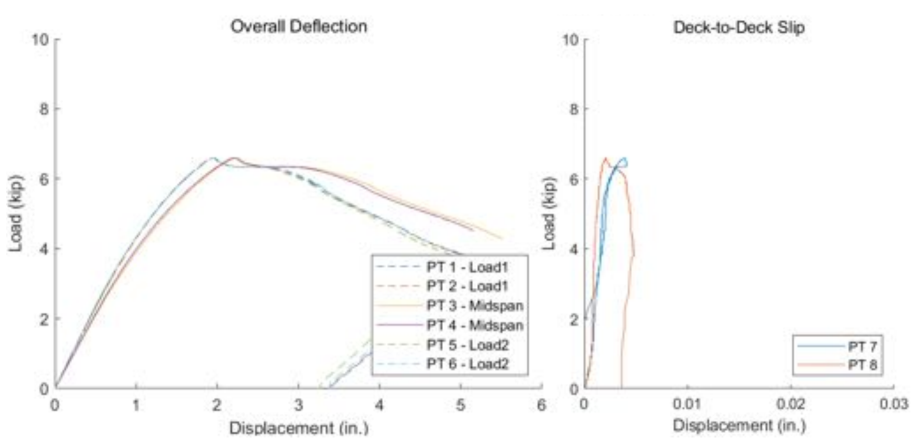


Figure 39. PC Bare Deck - Test 2 - Position Transducer Data

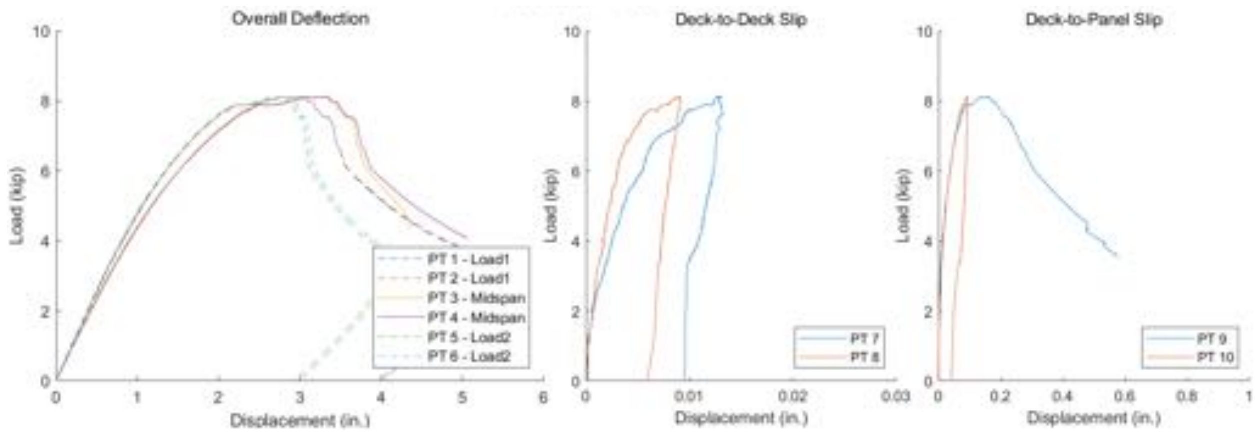


Figure 40. PC Deck + PC Panel - Test 1 - Position Transducer Data

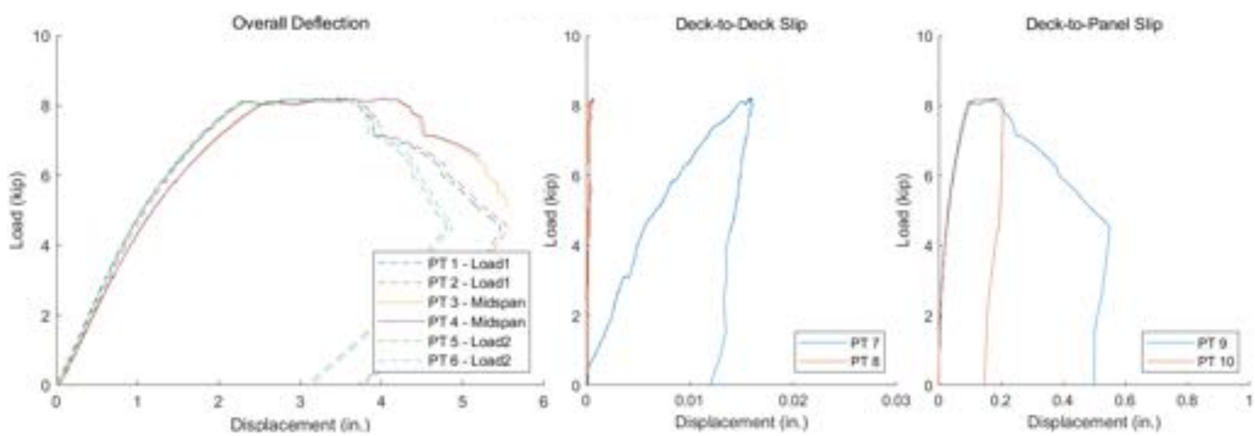


Figure 41. PC Deck + PC Panel - Test 2 - Position Transducer Data

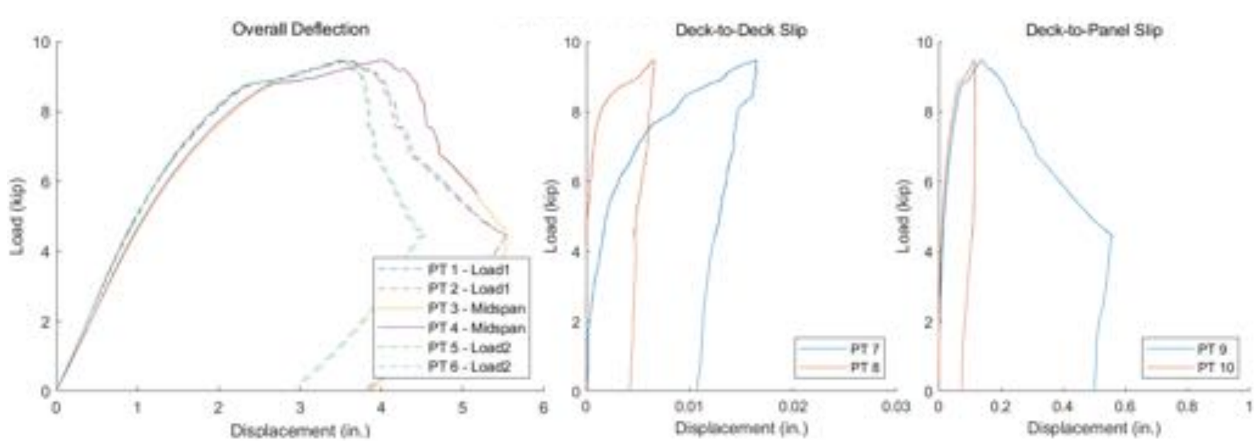


Figure 42. PC Deck + FC Panel - Test 1 - Position Transducer Data

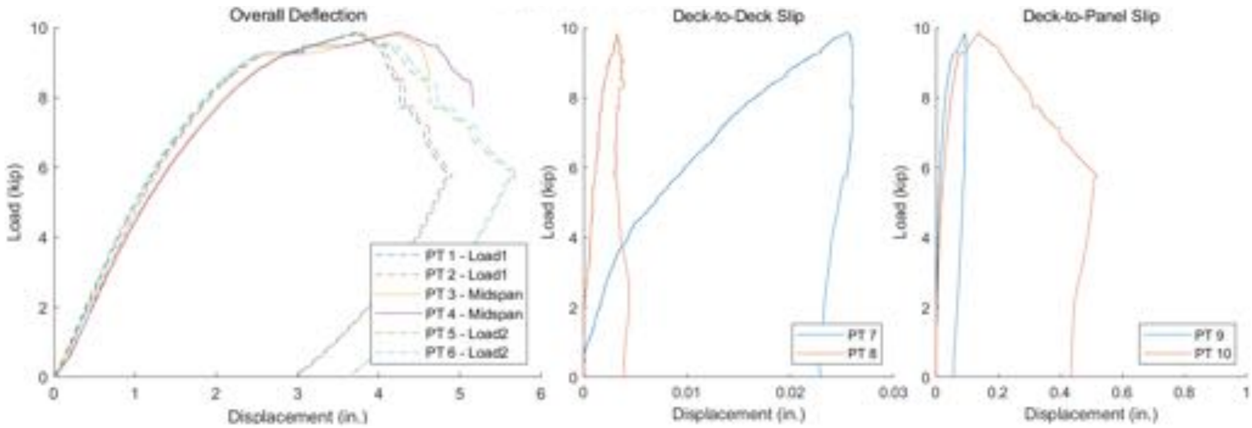


Figure 43. PC Deck + FC Panel - Test 2 - Position Transducer Data

The magnitude of the deck-to-panel slip is also much greater than the deck-to-panel slip.

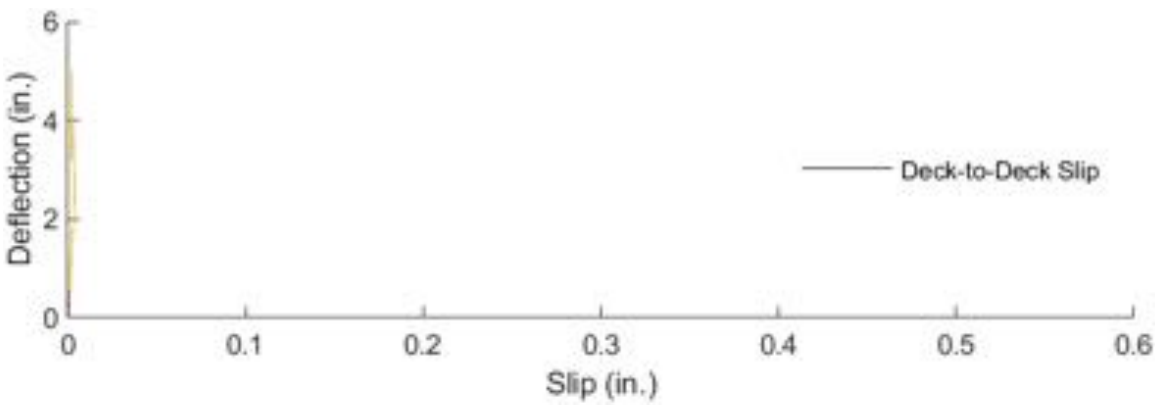


Figure 44. FC Bare Deck - Test 2 – Deck-to-deck slip (PT data not available for Test 1)

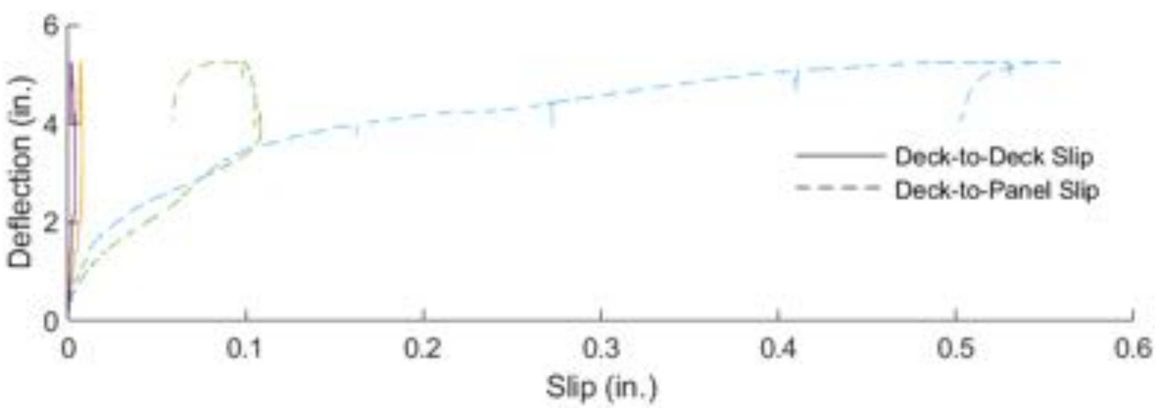


Figure 45. FC Deck + PC Panel - Test 1 - Deck-to-deck and deck-to-panel slip

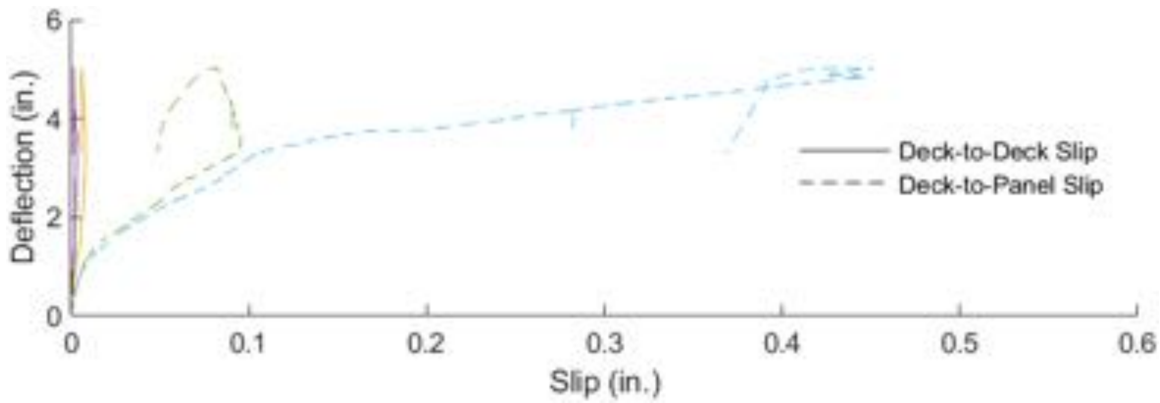


Figure 46. FC Deck + PC Panel - Test 2 - Deck-to-deck and deck-to-panel slip

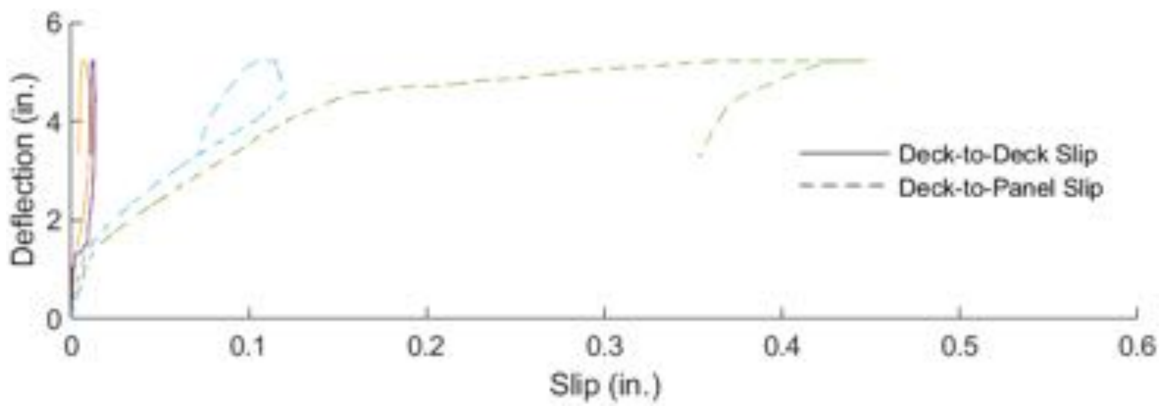


Figure 47. FC Deck + FC Panel - Test 2 - Deck-to-deck and deck-to-panel slip (PT data not available for Test 1)

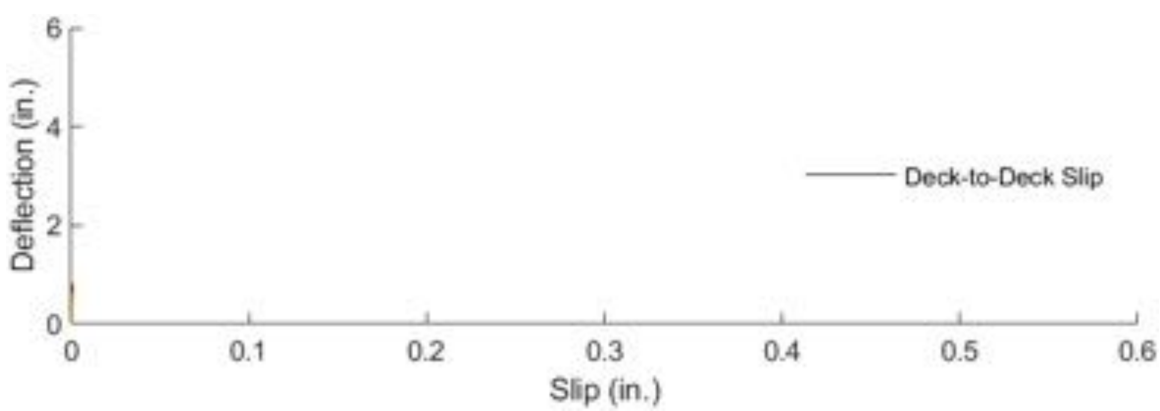


Figure 48. PC Bare Deck – Test 1 – Deck-to-deck slip

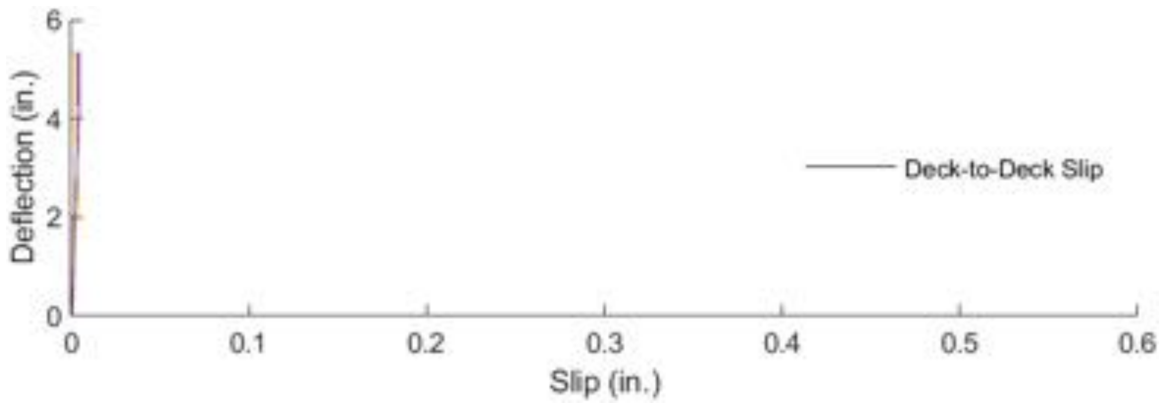


Figure 49. PC Bare Deck - Test 2 - Deck-to-deck slip

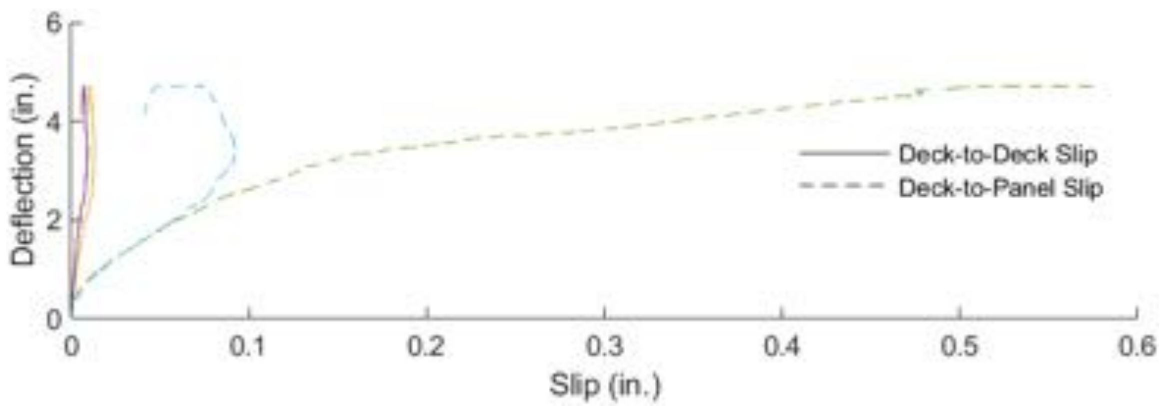


Figure 50. PC Deck + PC Panel - Test 1 - Deck-to-deck and deck-to-panel slip

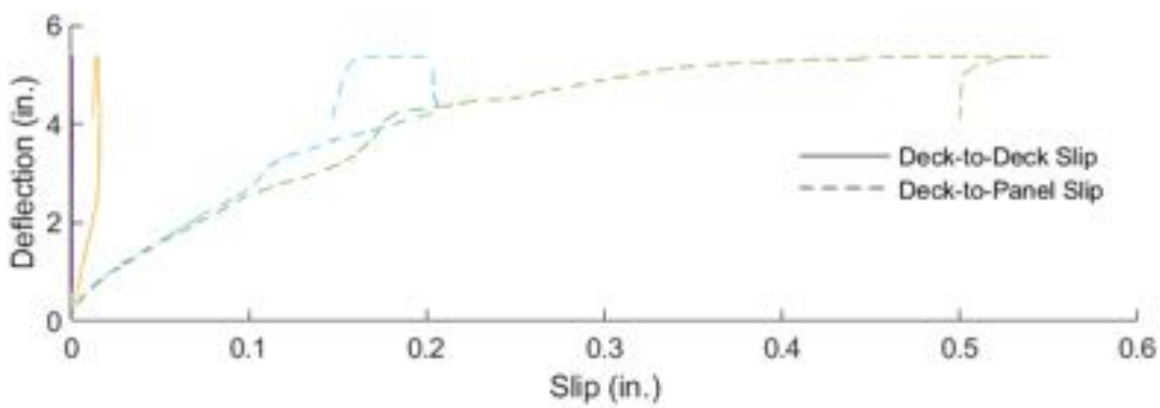


Figure 51. PC Deck + PC Panel - Test 2 - Deck-to-deck and deck-to-panel slip

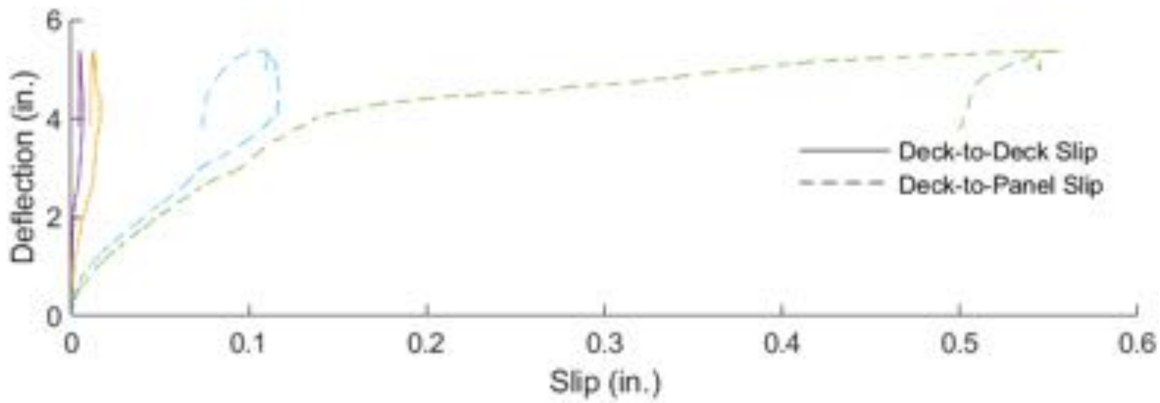


Figure 52. PC Deck + FC Panel - Test 1 - Deck-to-deck and deck-to-panel slip

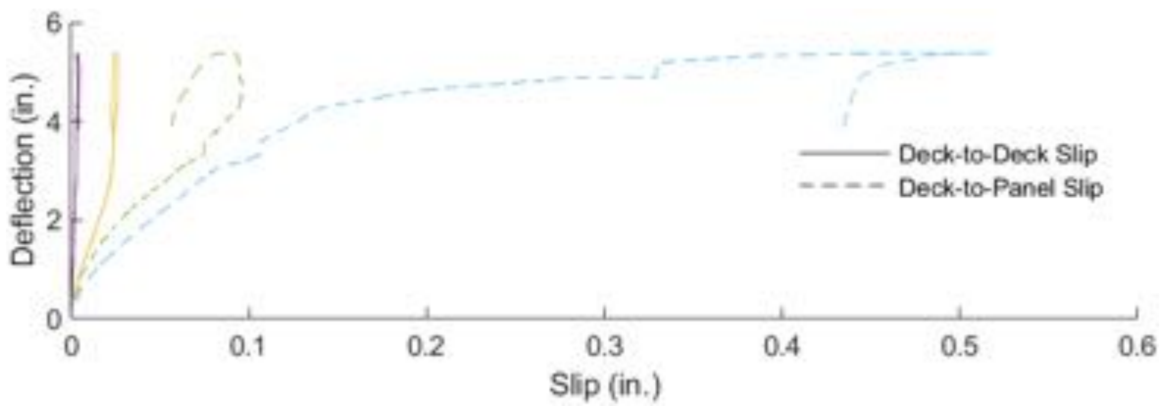


Figure 53. PC Deck + FC Panel - Test 2 – Deck-to-deck and deck-to-panel slip

10. Conclusions

The Fastfloor residential project explores a new type of modular floor system using non-proprietary materials while avoiding the use of any cast-in-place concrete. This prototype design uses two cold-formed steel profiled decks fastened together with self-drilling screws to create a cellular deck that is then topped with a cementitiouspanel. Through a series of 12 four-point bending tests, the impact of fully and partially composite deck-to-deck and deck-to- panel connections are explored. The deck-to-deck fastener spacing of 4 in. and 8 in. has only a marginal influence on the results – whiel the presence of the cementitous panel and the spacing of the panel-to-deck fasterns has a substantial influence. When the cementious panel is added stiffness increases marginally, on average xx% about the bare steel deck specimen, but strengh increases substantially. For panels fastened at 12 in. o.c. the strength increases xx% and for 6 in., on center yy% above the bare steel deck specimens. The system is able to undergo large rotations without significant degradation in strength. The primary limit state observed is buckling and yielding of the steel deck, followed by shear of the panel-to-deck fasteners at large deformations. Future work for the team includes development and comparison to design methods, examination of vibration and non-structural performance, further examination of the panel-to-deck fastener behavior and more.

11. References

[1] 3'' Composite Metal Deck, Panel Systems, Inc. Wood-bridge, VA, U.S.A.: PSI, n.d.

[2] Multi-Story Building Systems and Selection Criteria, New Millennium. Fort Wayne, IN, U.S.A.: New Millennium, 2021.

[3] Design of composite steel deck floors for fire, The free encyclopedia for UK steel construction information. SteelConstruction.info, Feb. 2021

[4] Dovetail Deck, Panel Systems, Inc. Woodbridge, VA, U.S.A.: PSI, n.d.

[5] Cold Formed Joists, Panel Systems, Inc. Woodbridge, VA, U.S.A.: PSI, n.d.

[6] How to Engineer Subfloors and Cold-Formed Steel Floor Joists for a Little Peace and Quiet, The Steel Network. Durham, NC, U.S.A.: TSN, 2022.

[7] Ecospan Composite Floor System, Design Manual. Vulcraft, 2018.

[8] Fox, D. M.; Schuster, R. M.; and Strickland, M. R.; ISpan a Light Steel Floor System. International Specialty Conference on Cold-Formed Steel Structures, 2006.

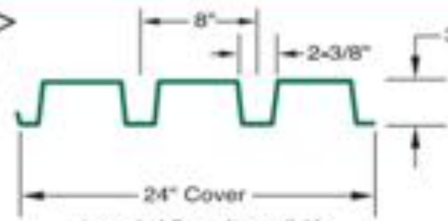
[9] Buettner, D. R.; and Becker, R. J.; PCI Manual for the Design of Hollow-core Slabs. PCI Hollow Core Slab Producers Committee, 1998.

[10] Ghosh, S.K.; Cleland, N.M.; and Naito, C.J.; Seismic design of precast concrete diaphragms. 2017.

12. Appendix-1: Data Sheets

TYPE "N" ROOF DECK *(LONG SPAN)*

FM APPROVED



Acoustical Type also available

Helpful Hint: Type "N" deck is used when the support spacing exceeds the recommended spacing for "B" type deck.

Type "N"

Type "N Acoustical" (long span perforated)

Section Properties (Fy=33 ksi)

Gage	Design Thickness	Weight (psf) Ptd	Weight (psf) Galv	Ip(In⁴)	In(In⁴)	Sp(In³)	Sn(In³)
22	.0295	2.01	2.05	0.6152	0.8158	0.3604	0.4129
20	.0358	2.58	2.65	0.7921	1.0216	0.4748	0.5311
18	.0474	3.20	3.40	1.1625	1.3695	0.7027	0.7502
16	.0598	4.10	4.25	1.5909	1.7448	0.9132	0.958
14	.0747	5.12	5.35	2.126	2.186	1.1704	1.2091
12	.1046	7.17	7.40	3.0732	3.0732	1.6874	1.6923

- Section properties calculated in accordance with AISI specifications

Acoustical Data: Type "N Acoustical"

Absorption Coefficients						NRC
125	250	500	1000	2000	4000	
.20	.30	.68	.81	.46	.38	.55

Gage	Span Cond	Max SDI Const Sp	Uniform Total Load in Pounds Per Square Foot (Dead and Live)										
			9'-0"	9'-6"	10'-0"	10'-6"	11'-0"	11'-6"	12'-0"	12'-6"	13'-0"	13'-6"	14'-0"
22	One	11'-3"	57	51	46	42	38	35	32				
20		12'-9"	76	68	61	55	49	44	40	36	33	31	
18		15'-5"	111	98	85	75	67	59	53	48	44	40	37
16		18'-0"	145	130	113	99	88	78	70	63	57	52	47
14		20'-10"	187	168	148	129	114	101	90	81	73	66	60
12		25'-0"				183	160	141	125	112	101	91	83
22	Two	13'-3"	70	62	56	51	46	42	39	36	33	31	
20		15'-0"	88	79	71	64	59	54	49	45	42	39	36
18		18'-2"	122	109	98	89	81	74	68	63	58	54	50
16		22'-0"	155	139	125	114	103	95	87	80	74	68	64
14		24'-7"	194	174	157	143	130	119	109	101	93	86	80
12		29'-6"				199	182	166	153	141	130	120	112
22	Three or More	13'-3"	87	78	70	64	58	53	49	45	42	38	36
20		15'-0"	110	99	89	81	73	67	62	57	52	49	45
18		18'-2"	152	136	123	112	102	93	85	79	73	67	62
16		22'-0"	194	174	157	142	129	118	109	100	93	86	80
14		24'-7"			197	178	163	149	137	126	116	108	100
12		29'-6"							191	176	162	151	140

Notes:
1. Load tables are calculated using section properties based on the steel design thickness shown in the Steel Deck Institute (SDI) design manual.
2. Loads shown in the shaded areas are governed by the live load deflection not in excess of 1/240 of the span. A dead load of 10 psf has been included.

DACS inc. 900 Port Centre Parkway, Portsmouth, VA 23704
Toll-Free: 866.286.6215 | Fax: 757.393.9461 | dacsinc.com

sdi member

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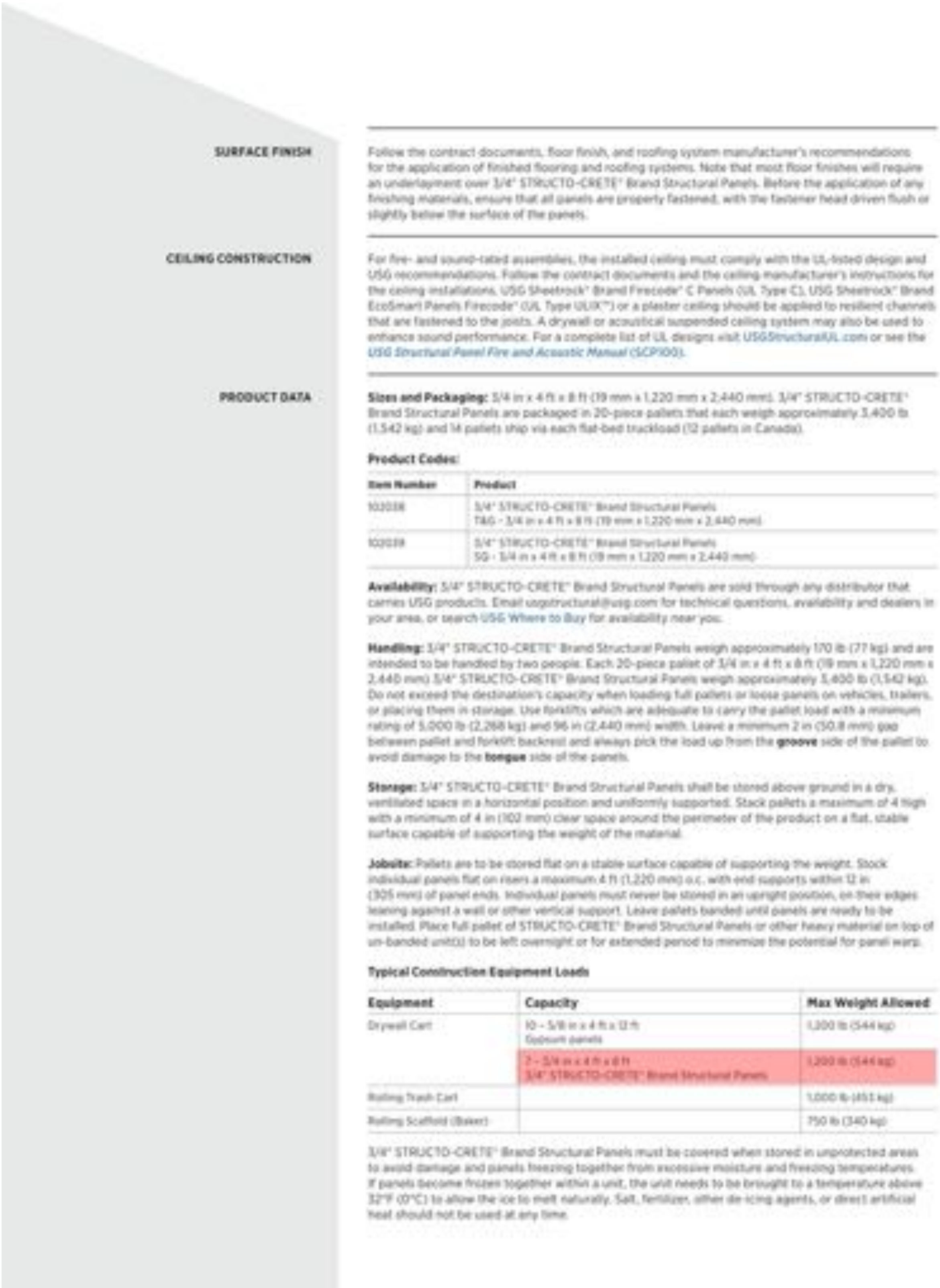
Figure A1-1. Steel deck datasheet



SURFACE FINISH

Follow the contract documents, floor finish, and roofing system manufacturer's recommendations for the application of finished flooring and roofing systems. Note that most floor finishes will require an underlayment over 3/4" STRUCTO-CRETE® Brand Structural Panels. Before the application of any finishing materials, ensure that all panels are properly fastened, with the fastener head driven flush or slightly below the surface of the panels.

CEILING CONSTRUCTION

For fire- and sound-rated assemblies, the installed ceiling must comply with the UL-listed design and USG recommendations. Follow the contract documents and the ceiling manufacturer's instructions for the ceiling installations. USG Sheetrock® Brand Firecode® C Panels (UL Type C), USG Sheetrock® Brand EcoSmart Panels Firecode® (UL Type ULIX™) or a plaster ceiling should be applied to resilient channels that are fastened to the joists. A drywall or acoustical suspended ceiling system may also be used to enhance sound performance. For a complete list of UL designs visit USGStructuralUL.com or see the *USG Structural Panel Fire and Acoustic Manual (SCP100).*

PRODUCT DATA

Sizes and Packaging: 3/4 in x 4 ft x 8 ft (19 mm x 1,220 mm x 2,440 mm). 3/4" STRUCTO-CRETE® Brand Structural Panels are packaged in 20-piece pallets that each weigh approximately 3,400 lb (1,542 kg) and 14 pallets ship via each flat-bed truckload (12 pallets in Canada).

Product Codes:

Item Number	Product
102038	3/4" STRUCTO-CRETE® Brand Structural Panels T&G - 3/4 in x 4 ft x 8 ft (19 mm x 1,220 mm x 2,440 mm)
102039	3/4" STRUCTO-CRETE® Brand Structural Panels SQ - 3/4 in x 4 ft x 8 ft (19 mm x 1,220 mm x 2,440 mm)

Availability: 3/4" STRUCTO-CRETE® Brand Structural Panels are sold through any distributor that carries USG products. Email usgstructural@usg.com for technical questions, availability and dealers in your area, or search USG Where to Buy for availability near you.

Handling: 3/4" STRUCTO-CRETE® Brand Structural Panels weigh approximately 170 lb (77 kg) and are intended to be handled by two people. Each 20-piece pallet of 3/4 in x 4 ft x 8 ft (19 mm x 1,220 mm x 2,440 mm) 3/4" STRUCTO-CRETE® Brand Structural Panels weigh approximately 3,400 lb (1,542 kg). Do not exceed the destination's capacity when loading full pallets or loose panels on vehicles, trailers, or placing them in storage. Use forklifts which are adequate to carry the pallet load with a minimum rating of 5,000 lb (2,268 kg) and 96 in (2,440 mm) width. Leave a minimum 2 in (50.8 mm) gap between pallet and forklift backrest and always pick the load up from the **groove** side of the pallet to avoid damage to the **tongue** side of the panels.

Storage: 3/4" STRUCTO-CRETE® Brand Structural Panels shall be stored above ground in a dry, ventilated space in a horizontal position and uniformly supported. Stack pallets a maximum of 4 high with a minimum of 4 in (102 mm) clear space around the perimeter of the product on a flat, stable surface capable of supporting the weight of the material.

Jobsite: Pallets are to be stored flat on a stable surface capable of supporting the weight. Stack individual panels flat on risers a maximum 4 ft (1,220 mm) o.c. with end supports within 12 in (305 mm) of panel ends. Individual panels must never be stored in an upright position, on their edges leaning against a wall or other vertical support. Leave pallets banded until panels are ready to be installed. Place full pallet of STRUCTO-CRETE® Brand Structural Panels or other heavy material on top of un-banded unit(s) to be left overnight or for extended period to minimize the potential for panel warp.

Typical Construction Equipment Loads

Equipment	Capacity	Max Weight Allowed
Drywall Cart	10 - 5/8 in x 4 ft x 12 ft Gypsum panels	1,200 lb (544 kg)
	7 - 3/4 in x 4 ft x 8 ft 3/4" STRUCTO-CRETE® Brand Structural Panels	1,200 lb (544 kg)
Rolling Trash Cart		1,000 lb (453 kg)
Rolling Scaffold (Baker)		750 lb (340 kg)

3/4" STRUCTO-CRETE® Brand Structural Panels must be covered when stored in unprotected areas to avoid damage and panels freezing together from excessive moisture and freezing temperatures. If panels become frozen together within a unit, the unit needs to be brought to a temperature above 32°F (0°C) to allow the ice to melt naturally. Salt, fertilizer, other de-icing agents, or direct artificial heat should not be used at any time.

Figure A1-2. Structural panel datasheet 1

PRODUCT DATA

Sizes and Packaging: 3/4" x 4' x 8' (19mm x 1220mm x 2440mm) panels. Each panel weighs approximately 170 lbs. (77kg) and is intended to be handled by two people. USG Structural Panel Concrete Subfloors are packaged in 20 piece units.

Availability: USG Structural Panel Concrete Subfloors are sold through any USG distributor. Email usgstructural@usg.com for information on availability and a dealer in your area.

Storage: USG Structural Panel Concrete Subfloors are shipped in 20 piece units. Panels should be stored in a horizontal position and uniformly supported. Panels must be covered when stored in unprotected areas.

Excessive moisture and freezing temperatures may result in panels sticking together within the units. Therefore, care should be taken to ensure units of USG Structural Panel Concrete Subfloors are not exposed to excessive moisture, ice and snow. In the event that panels do become frozen together within a unit, the unit needs to be brought to a temperature above 32°F (0°C) to allow the ice to melt naturally. Salt, fertilizer or other de-icing agents should not be used at any time. Covering the units completely with tarps or similar coverings is an easy way to avoid panels freezing together.

Maintenance: USG Structural Panel Concrete Subfloors do not require any regular maintenance except to remove standing water and repair damage from abuse. Any cracked or broken panels should be replaced with sound USG Structural Panel Concrete Subfloor that are secured following the fastening schedule prescribed in the original installation documents. The replacement panels must be a minimum of 24" (610mm) wide and must span a minimum of two supports. If not, the replacement panel must be fully blocked on all sides. See *USG Structural Panel Concrete Subfloor Installation Guideline* (form SCP14) for additional information.

TEST DATA

Physical and Mechanical Properties	Test Standard	Approximate Values Standard (Metric)
Moment capacity (3/4" (19mm) thick panel)	ASTM C1185, Sec. 5	1,585 lb-in/ft (588 N-m/m)
Bending stiffness (3/4" (19mm) thick panel)	ASTM C1185, Sec. 5	515,000 lb-in²/ft ([illegible])
Concentrated load	ASTM E661	550 lbs (2.45 kN) static 0.108" (2.7mm) max. deflection @ 200 lbs (0.89 kN)
Fastener lateral resistance[a]	ASTM D1761, Sec. 10.2	> 210 lbf (0.93 kN) dry > 160 lbf (0.71 kN) wet
Density[b]	ASTM C1185	75 lbs./ft³ (1,201 kg/m³)
Weight at 3/4" (19mm) thickness	ASTM D1037	5.3 lbs./ft² (26 kg/m²)
pH value	ASTM D1293	10.5
Linear variation with change in moisture (25% to 90% relative humidity)	ASTM C1185, Sec. 8	<0.10 %
Thickness swell	ASTM D1037, B	max. 3.0 %
Freeze / thaw resistance	ASTM C1185	Passed (50 cycles)
Mold resistance	ASTM D3273 ASTM G21	10 0
Water absorption[c]	ASTM C1185, Sec. 5.2.3.1	<15.0 %
Noncombustibility	ASTM E136-12 (unmodified) CAN/ULC-S114	Passed Passed
Surface-burning characteristics (flame spread/smoke developed)	ASTM E84 CAN/ULC-S102	0/0
Long-term durability	ASTM C1185, Sec. 10	min. 75% retention of physical properties
Water durability	ASTM C1185, Sec. 5	min. 70% retention of physical properties

(a) Fastener lateral resistance measured with #8, 1-5/8" (41mm) Hi-Low screw.
(b) Density measured at equilibrium conditioning per Section 5.2.3.1., 28 days after manufacturing.
(c) Absorption measured from equilibrium conditioning followed by immersion in water for 48 hours.

Figure A1-3. Structural panel datasheet 2

Ultimate tensile strengths – pullover (tension), lb (kN)[1,2,3,4,5,6,7]

Screw designation	Washer or head diameter in.	Thickness of steel member in contact with the screw head, ga (in.)						
		22 (0.030)	20 (0.036)	18 (0.048)	16 (0.060)	14 (0.075)	12 (0.105)	10 (0.135)
Hex Washer Head (HWH)								
#8	0.335	675 (3.00)	815 (3.63)	1000 (4.45)	1000 (4.45)	1000 (4.45)	1000 (4.45)	1000 (4.45)
#10	0.399	805 (3.58)	970 (4.31)	1290 (5.74)	1370 (6.09)	1370 (6.09)	1370 (6.09)	1370 (6.09)
#12-14	0.415	835 (3.71)	1010 (4.49)	1340 (5.96)	1680 (7.47)	2100 (9.34)	2325 (10.34)	2325 (10.34)
#12-24	0.415	835 (3.71)	1010 (4.49)	1340 (5.96)	1680 (7.47)	2100 (9.34)	2940 (13.08)	3780 (16.81)
1/4 in.	0.500	1010 (4.49)	1220 (5.43)	1620 (7.21)	2030 (9.03)	2530 (11.25)	3540 (13.75)	4560 (20.28)
Phillips Pan Head (PPH)								
#7	0.303	615 (2.74)	735 (3.27)	980 (4.36)	1000 (4.45)	1000 (4.45)	1000 (4.45)	1000 (4.45)
#8	0.311	630 (2.80)	755 (3.36)	1000 (4.45)	1000 (4.45)	1000 (4.45)	1000 (4.45)	1000 (4.45)
#10	0.364	740 (3.29)	885 (3.94)	1180 (5.25)	1370 (6.09)	1370 (6.09)	1370 (6.09)	1370 (6.09)
Phillips Truss Head (PTH)								
#8	0.411	830 (3.69)	1000 (4.45)	1000 (4.45)	1000 (4.45)	1000 (4.45)	1000 (4.45)	1000 (4.45)
#10	0.433	875 (3.89)	1050 (4.67)	1390 (6.18)	1390 (6.18)	1390 (6.18)	1390 (6.18)	1390 (6.18)
Phillips Pancake Head (PPCH)								
#10, #12	0.409	830 (3.69)	995 (4.43)	1325 (5.89)	1370 (6.09)	1370 (6.09)	1370 (6.09)	1370 (6.09)
Phillips Flat Truss Head (PFTH)								
#10	0.364	740 (3.29)	885 (3.94)	1180 (5.25)	1475 (6.56)	1840 (8.18)	2170 (9.65)	2170 (9.65)

1 The lower of the ultimate pullout, pullover, and tension fastener strength of screw should be used for design.
2 Load values based upon calculations done in accordance with Section J4 of the AISI S100.
3 AISI S100 recommends a safety factor of 3.0 be applied for allowable strength design, a Φ factor of 0.5 be applied for LRFD design or a Φ factor of 0.4 be applied for LSD design.
4 ANSI/ASME standard screw head diameters were used in the calculations and are listed in the tables.
5 Phillips Bugle Head (PBH) and Phillips Wafer Head (PWH) styles are not covered by this table because they are not intended for attachment of steel to steel.
6 The load data in the table is based upon sheet steel with F_u = 45 ksi. For F_u = 55 ksi steel, multiply values by 1.22. For F_u = 65 ksi steel, multiply values by 1.44.
7 Refer to Section 3.4.2.5 for drilling capacities.

Nominal ultimate fastener strength of screw

Screw designation	Nominal diameter (in.)	Nominal fastener strength			
		Tension, P_{ts} lb (kN)[1]		Shear, P_{ss} lb (kN)[2,3]	
#6-20	0.138	1000	(4.45)	890	(3.96)
#7-18	0.151	1000	(4.45)	890	(3.96)
#8-18	0.164	1000	(4.45)	1170	(5.20)
#10-12	0.190	2170	(9.65)	1645	(7.32)
#10-16	0.190	1370	(6.09)	1215	(5.40)
#10-18	0.190	1390	(6.18)	1645	(7.32)
#12-14	0.216	2325	(10.34)	1880	(8.36)
#12-24	0.216	3900	(17.35)	2285	(10.16)
1/4 in.	0.250	4580	(20.37)	2440	(10.85)

1 The lower of the ultimate pullout, pullover, and tension fastener strength of screw should be used for design. The Pullout and Pullover tables in this section have already been adjusted where screw strength governs.
2 The lower of the ultimate shear fastener strength and shear bearing should be used for design. The Shear Bearing table in this section has already been adjusted where screw strength governs.
3 AISI S100 recommends a safety factor of 3.0 be applied for allowable strength design, a Φ factor of 0.5 be applied for LRFD design or a Φ factor of 0.4 be applied for LSD design.

Torsional strength[1,2]

Size	Min. torsional strength in-lb (Nm)	
6-20	24	(2.7)
7-18	38	(4.3)
8-18	42	(4.8)
10-12	61	(6.9)
10-16	61	(6.9)
10-18	61	(6.9)
10-24	65	(7.3)
12-14	92	(10.4)
12-24	100	(11.3)
1/4-14	150	(17.0)
1/4-20	156	(17.6)

1 Based on screw only. Does not consider base material limitations.
2 Values in table are ultimate torsional strengths. To obtain maximum setting torque, multiply values in table by 0.66.

Figure A1-4. Hilti fasteners datasheet

Fastener Drive Bits

Fastener selection is based on several criteria and will affect framing type, framing flange size, minimum end distance for fastener insertion, and driver/drive bit used. Only USG recommended fasteners should be used and must be inserted according to the fastener pattern specified.

Framing Type[1]	Fastener Manufacturer	Fastener Part Number	Fastener Description[3]	Drive Bit Part Number
33 mil (20 ga) CFS	Grabber Construction Products, Inc.	CMS0178JBWRG	#10 x 1-7/8" Flat Head DRIVALL® Pilot Point Self-Drilling Screw	T2178LN LOX® #2 (178 mm)[5]
43 mil (18 ga) CFS	Grabber Construction Products, Inc.	CGH8158LG	#8 x 1-5/8" Winged Flat WaferHead Self-Drilling Screw	T2178LN LOX® #2 (178 mm)[5]
54 mil (16 ga) – 97 mil (12 ga) CFS	Grabber Construction Products, Inc.	CGH8158LG	#8 x 1-5/8" Winged Flat WaferHead Self-Drilling Screw	T2178LN LOX® #2 (178 mm)[5]
	Simpson Strong-Tie Company, Inc.	CB3DQ158S	#8 x 1-5/8" Winged Self-Drilling Screw	BIT2SU #2 (Undersized sq.)
1/4" (6.4 mm) A36 HRS[3]	Grabber Construction Products, Inc.	CC12250LRG	#12 x 2-1/2", Winged Self-Drilling Screw	T3178LN LOX® #3 (178 mm)[6]
	Simpson Strong-Tie Company, Inc.	TB612S05	#12 x 2-3/8", Flat Head, Strong-Drive® TB Wood-to-Steel Screw	BIT3SU #3 (Undersized sq.)
	Muro North America, Inc.	RSM6430WFL-CY	M6.0 x 45 mm Winged Self-Drilling Screw	#3 SQ
1/8 in (3.2 mm) – 1/2 in (13 mm) A36 HRS	Aerosmith® Fastening	5324HPG	0.145 x 1-1/4" Helical PowerPin®	Tool Setting and Load will vary based on steel thickness and hardness
	DeWalt - Engineered by Powers, Inc.	50458-PWR	0.157 in x 1-1/4" CSI Spiral Drive Powder Actuated Pin	
	Hilti, Inc.	X-U 32 MX	0.157 in x 1-1/4" Knurled Shank Powder Actuated Fastener	
SPF Lumber	Grabber Construction Products, Inc.	C8200L2M	#8 x 2", Flat Head, Type 17, Nibs, GrabberGard®	T2178LN LOX® #2 (178 mm)[5]
	SENCO Brands, Inc.	GL24AABF	8d Ring Shank Nails[4]	N/A

Table Notes:

1. **CFS** = cold formed structural steel **HRS** = hot rolled structural steel **Lumber** = specific gravity 0.42 or greater.
   Gauge/thickness of steel, fastener end distance, and joist flange width is identified for each fastener and are minimums. Framing gauge, size, and type is determined by the engineer, architect, or design professional.
2. 33 mil (structural 20 ga) is for gravity loads only.
3. Any length of the same fastener is approved provided a minimum of 3 threads penetrate the steel framing.
4. SENCO 8d ring shank nails are manufactured with a length of 2-3/8 in., head diameter of 0.266 in. and a shank diameter of 0.113 in. Equivalent 8d ring shank nails meeting these dimensional requirements may be utilized when approved by the engineer or designer of record.
5. Grabber SuperDrive® 75 uses the 178mm LOX® #2 drive bit. They also offer a 3" LOX® #2 drive bit for hand held drill use, Part # 3002L.
6. Grabber SuperDrive® 75 uses the 178mm LOX® #3 drive bit. They also offer a 3" LOX® #3 drive bit for drill use, Part # 3003L.

General Note:
In accordance with PER-15067 (Subfloor) and PER-14076 (Roof Deck), the minimum screw pattern is 6 in. (153 mm) o.c. along the perimeter and 12 in. (305 mm) o.c. in the field of the panels. Refer to PER-15012 for Foundation Wall fastener schedules.

Made in Delavan, WI, USA

A qualified architect or engineer should review and approve calculations, framing, and fastener spacing for all projects.

PRODUCT INFORMATION
See usg.com for the most up-to-date product information.

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Figure A1-5. Grabber® fasteners datasheet

Appendix-2: Calculations

Analytical Stiffness Calcs 09/28/22	FastFloor Residential	Cas Caswell	1/2

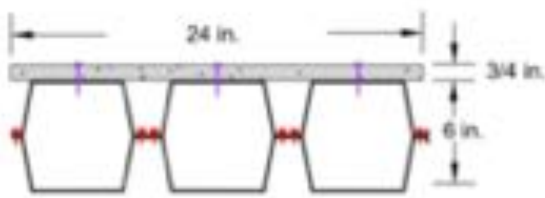
GIVEN:

Bare Specimen Info

- $A_s = 3.7421\ in^2$ From CUFSM5
- $I_s = 18.537\ in^4$ From CUFSM5
- $E_s = 29500$ ksi From CUFSM5
- $y_s = 3$ in - Neutral axis
- $F_y = 58.03$ ksi - From Coupon Tests

Panel Info

- $EI_{panel} = 315000\ \frac{lb * in^2}{ft}$ from Apendix 1 - Figure A1-3 (USG Documents)
- $b_{panel} = 24$ in
- $t_{panel} = 0.75$ in
- $y_{panel} = 6\ in + \frac{0.75\ in}{2} = 6.38$ in - Neutral axis



Analytical

$$I_{panel\ strip} = \frac{0.75\ in^3}{12} = 0.422\ \frac{in^4}{ft}$$

$$E_{panel} = \frac{EI_{panel}}{I_{panel\ strip}} = \frac{315000\ \frac{lb * in^2}{ft}}{0.422\ \frac{in^4}{ft}} = 747\ ksi$$

$$n = \frac{E_s}{E_{panel}} = \frac{29500\ ksi}{747\ ksi} = 39.5$$

Panel Transformed Properties

$$b_{panel,T} = \frac{b_{panel}}{n} = \frac{24\ in}{39.5} = 0.607\ in$$

$$A_{panel,T} = b_{panel,T} * t_{panel} = 0.607\ in * 0.75\ in = 0.456\ in^2$$

$$I_{panel,T} = \frac{b_{panel,T} * t_{panel}^{\ 3}}{12} = \frac{0.607\ in *\ 0.75\ in^3}{12} = 0.0214\ in^4$$

Total Properties

$$y_{bar,tot} = \frac{y_s * A_s + y_{panel} * A_{panel,T}}{A_s + A_{panel,T}} = \frac{3\ in * 3.74\ in^2 + 6.38\ in * 0.456\ in^2}{3.74\ in^2 + 0.456\ in^2}$$

$= 3.37\ in$

$I_{tot} = I_s + A_s * (y_s - y_{bar,tot})^2 + I_{panel,T} + A_{panel,T} * (y_{panel} - y_{bar,tot})^2 = 18.5\ in^4$
$+ 3.74\ in^2 * (3\ in - 3.37\ in)^2 + 0.0214\ in^4 + 0.456\ in^2 * (6.38\ in - 3.37\ in)^2 = 23.2\ in^4$

$EI_{bare} = I_s * E_s = 18.5\ in^4 * 29500\ ksi = 5.47E+05\ kip * in^2$

$EI_{tot} = I_{tot} * E_s = 23.2\ in^4 * 29500\ ksi = 6.84E+05\ kip * in^2$

$$M_{y,bare} = \frac{F_y * I_s}{y_s} = \frac{58.0\ ksi * 18.5\ in^4}{3\ in} = 29.9\ kip * ft$$

$$M_{y,tot} = \frac{F_y * I_{tot}}{y_{bar,tot}} = \frac{58.0\ ksi * 23.2\ in^4}{3.37\ in} = 33.3\ kip * ft$$