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Twelfth International Specialty Conference on Cold-Formed Steel Structures
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FURTHER STUDIES OF COMPOSITE SLAB STRENGTH

Angela S. Terry[1] and W. Samuel Easterling[2]

SUMMARY

The results to date of a research program focusing on the strength of composite slabs are described. Full-scale experimental slab tests are compared to strengths calculated using the Steel Deck Institute *Composite Deck Design Handbook*. Based on the comparisons, recommendations are made for modifications to the calculation procedures.

INTRODUCTION

A research project, sponsored by the Steel Deck Institute (SDI) at Virginia Tech, was completed in 1990 in which the principal objective was to show that the strength and stiffness of steel-deck-reinforced concrete floor systems can be predicted with traditional reinforced concrete models, if typical field details are considered. These details include the consideration of interior spans, common pour-stop details and the use of headed shear studs. The results of the research indicated that indeed the simple reinforced concrete models are good indicators of the lower bound strength and elastic stiffness of composite slab systems (Easterling and Young 1992).

The Virginia Tech research was then combined with other information (Luttrell and Prassanan 1986; Slutter 1975; *Standards for* 1993) to produce the SDI *Composite Deck Design Handbook* (CDDH) (Heagler, et al. 1991). There are two distinct design procedures given in the CDDH; one for use if shear studs are present on the beams and the other for use if shear studs are not used. Additionally, partial composite action is considered for those cases in which there are insufficient shear studs present to provide 100% anchorage to the deck.

A continuation of the research at Virginia Tech, sponsored by the SDI and the American Iron and Steel Institute (AISI), is currently underway. The principal objective of the current research is to generate additional test data that will confirm the general application of simple reinforced concrete models for determining the strength and stiffness of composite floor systems. Another goal is to use the additional data to refine the design rationale presented in CDDH such that a single unified method can be developed for all degrees of anchorage, regardless if the anchorage is provided by welds or studs. The experimental portion of the program is nearing completion at the time of this writing. Results of the program to date, comparisons to the CDDH methods and suggested revisions to the methods are presented in this paper.

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EXPERIMENTAL PROGRAM

TEST PARAMETERS

Eight three-span composite floor slabs were constructed. At the time of this writing seven have been tested. A total of nineteen tests were performed. The two end spans were tested on slabs one and seven. All three spans were tested on slabs two through six, with the center span tested last. Only the center span will be tested on slab eight. Variables in the eight castings were deck thickness, 0.036 in. (0.9 mm) or 0.048 in. (1.2 mm); rib height, 2 in. (51 mm) or 3 in. (76 mm); slab thickness, 4.5 in. (115 mm) or 5.5 in. (140 mm); span length, 9 ft. (2.75 m) or 10 ft. (3.05 m); and number and type of anchorage over supports, studs or welds. End restraint from pour stops and deck continuity over supports were also investigated. The steel yield stress and the concrete compressive strength varied from specimen to specimen. All specimens were 6 ft. (1.83 m) wide and all of the deck utilized had a galvanized coating.

The test designation for specimen one, span one was SDI-2/20-4-9. The 2/20 provides information about the steel deck (rib height / gage). For example, the steel deck used in the first composite floor was 2 inches deep with a 20 gage thickness. The 4 indicates the type of anchorage over the supports of the span. A number indicates the number of studs. In this test four studs provided anchorage over the supports. A P designation indicates the presence of puddle (arc spot) welds over the supports. All puddle weld visible diameters were 0.75 in. (19 mm), and the welds were placed approximately every 12 in. (305 mm). A PX designation indicates puddle welds as well as butted joints, i.e., the deck was not continuous over the interior supports. The additional number on the P and PX designations, for example, P1 and PX1, is the span number. The last number of each test designation is the span length, center-to-center of supports. For example, the span length for the first test was nine feet. Table 1 summarizes The details of each floor system are summarized in Table 1 and the support details illustrated in Figures 1 and 2.

Each specimen was constructed similarly. The deck was cut to the appropriate length, or lengths if there were butted joints over the supports. Strain gages were attached on the underside of the deck sheets at several locations. The sheets were then placed on the supports, and the seams were aligned. The two panels were fastened together by button punching on 18-in. (460 mm) centers. The deck was positioned on the supports and attached with either shear studs or puddle welds. Pour stops were screwed to the deck and wire mesh (WWF 6x6-W1.4 x W1.4) was placed inside the form and allowed to rest on the top flange of the deck.

Each composite floor was cast with a normal weight, 3,000 psi (21 MPa) mix concrete, vibrated, and screeded. Steel deck strains and displacements were recorded during casting. Each composite floor was covered with plastic and moist cured for seven days. On the seventh day the plastic and the pour stops were removed, with the exception of specimens three and four. The pour stop on the end of span three of both specimens had a return lip into the slab and was left on during testing to evaluate end restrain capability. Concrete cylinders were tested every seven days. Concrete strain gages were attached to the top surface of the slab after it had cured. Each composite floor was tested after a minimum of 21 days provided the concrete strength had reached 3,000 psi (21 Mpa).

Table 1. Specimen Details

Specimen Number	Test Designation	Span	End Details	Support Anchorage
1	SDI-2/20-4-9	end	1 ft. cantilever	4 studs
	SDI-2/20-5-9	end	1 ft. cantilever	5 studs
2	SDI-2/20-2-9	end	1 ft. cantilever	2 studs
	SDI-2/20-23-9	center	N/A	2 studs/3 studs
	SDI-2/20-3-9	end	1 ft. cantilever	3 studs
3	SDI-2/20-PI-9	end	1 ft. cantilever	arc spot welds
	SDI-2/20-P2-9	center	N/A	arc spot welds
	SDI-2/20-P3-9	end	angle with lip	arc spot welds
4	SDI-2/20-PX1-9	end	1 ft. cantilever, int. sup. deck joint	arc spot welds
	SDI-2/20-PX2-9	center	deck joints	arc spot welds
	SDI-2/20-PX3-9	end	angle with lib, int. sup. deck joint	arc spot welds
5	SDI-2/18-3-9	end	1 ft. cantilever	3 studs
	SDI-2/18-35-9	center	N/A	3 studs/5 studs
	SDI-2/18-5-9	end	1 ft. cantilever	5 studs
6	SDI-3/20-3-10	end	1 ft. cantilever	3 studs
	SDI-3/20-35-10	center	N/A	3 studs/5 studs
	SDI-3/20-5-10	end	1 ft. cantilever	5 studs
7	SDI-3/20-PX1-10	end	1 ft. cantilever, int. sup. deck joint	arc spot welds
	SDI-2/18-PX3-9	end	1 ft. cantilever, int. sup. deck joint	arc spot welds
8	SDI-3/20-33-10	center	N/A	3 studs

INSTRUMENTATION

Each specimen was instrumented with strain gages on the steel deck and the concrete. Transducers were used to measure vertical displacement. Potentiometers or dial gages were used to measure end slip. A pressure transducer was used to measure the load applied to the specimen.

Strain gages were placed on the underside of the steel deck in four major groups. A series of gages was placed nine inches inside the centerline of both the interior and exterior supports of all end spans tested. A second series of gages was placed at the location of maximum moment, which was calculated assuming a three span configuration with the load placed only on the span under consideration. The last series of gages was placed along the span at one foot intervals. In each series of gages along a cross section of the deck, gages measured strain in the top flange and the bottom flange of the deck. At the exterior support and location of maximum moment, strain was also measured in the web of the deck. Strain gages were placed on the top of the cured composite floor to measure the compressive strains in the concrete. Two gages were placed at the location of maximum moment of each span tested.

Two transducers were used to measure the vertical displacement at midspan of the loaded span. Potentiometers or dial gages were used to measure the horizontal slip between the steel deck and the concrete at the end of the specimen during an end span

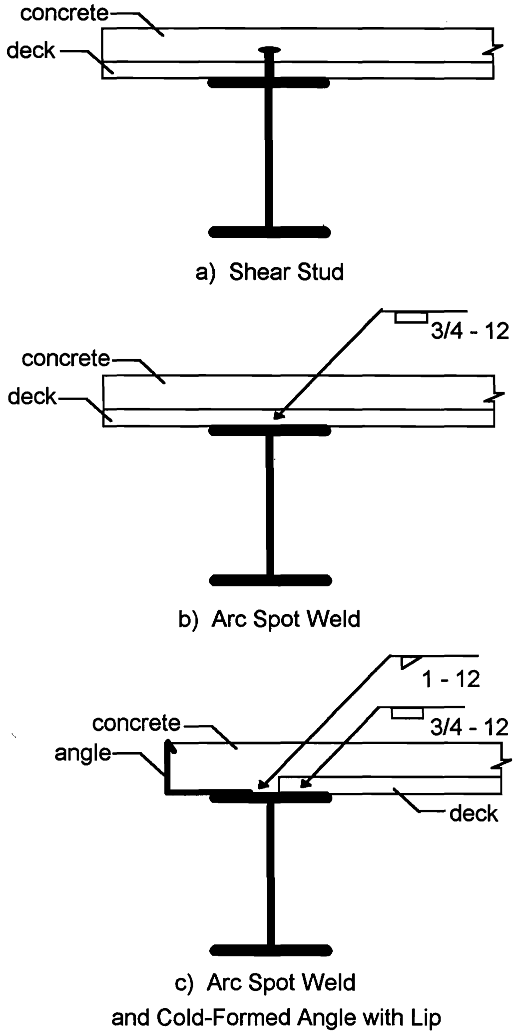
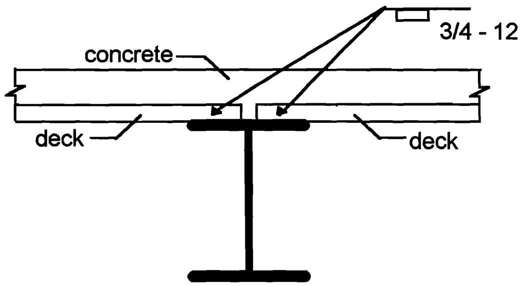
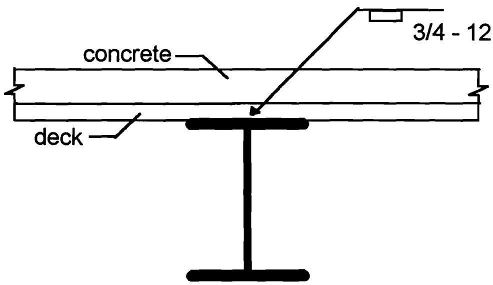


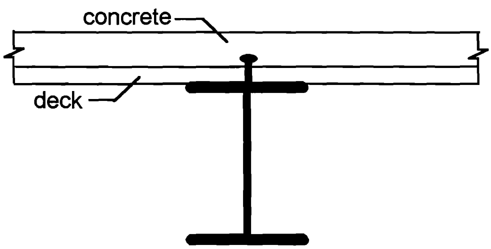
Figure 1 End Span Details--Exterior Support



a) Deck Joint with Arc Spot Weld



b) Continuous Deck with Arc Spot Weld



c) Continuous Deck with Shear Stud

Figure 2 End Span Details--Interior Support test. Measurements of end slip were taken at several locations along the specimen cross section, including both top and bottom flanges of the deck.

Uniform load was applied using an airbag, as will be described in the next section. The air bag was designed with two valves, one for the input of air and the other for measuring the pressure in the bag. A calibrated pressure transducer was connected to this second valve and the pressure was recorded by a data acquisition system.

TEST SETUP

The test setup, illustrated schematically in Figure 3, consisted of two W21x68 column frames, bolted to the laboratory floor outside the supports of the span being tested. Two W12x26 beams were bolted horizontally between the columns, parallel to the composite floor. A rubber press bag with a 6 ft. x 10 ft. (1.83 m x 3.05 m) bearing surface was placed on the slab. Sheets of 3/4-in. (19.1 mm) plywood were placed on top of the bag. Two holes in the plywood allowed access to the valves in the bag. For nine foot spans, five W8x24 beams were bolted to the bottoms of the W12x26 beams, perpendicular to the composite floor. The frame was extended to seven perpendicular beams for ten foot spans. The regulated air source and the pressure transducer were attached to the valves in the bag. All instrumentation was then connected to a data acquisition system.

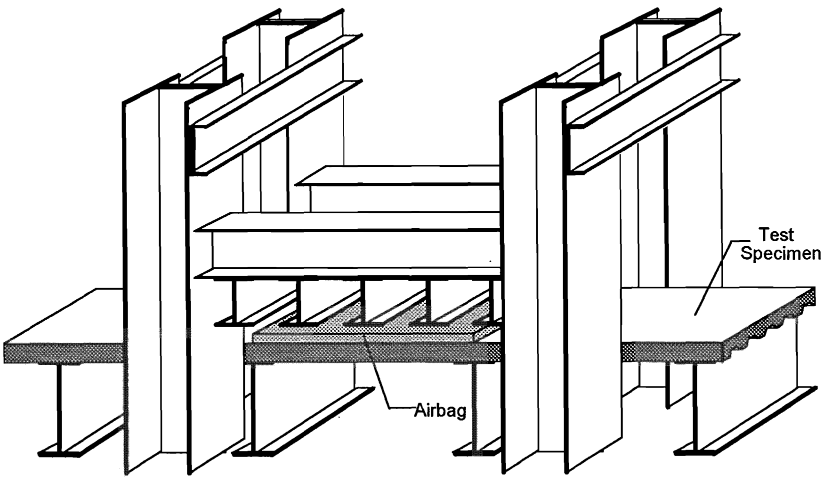


Figure 3. Experimental Setup

TEST PROCEDURE

The test procedure was the same for all tests. The test span was first preloaded to 0.35 psi (2.4 kPa) to seat the structure and insure that all instrumentation was functioning properly. The bag was then emptied through a valve in the input line.

The span was loaded in 0.35 psi (2.4 kPa) increments at a rate of approximately 0.3 psi (2.1 kPa) per minute. At each load increment the air flow was stopped, and the system was allowed to stabilize for two minutes before any measurements were recorded. This process continued until cracks appeared over the interior supports and a plot of load versus displacement showed that the system would have some permanent set when unloaded. The bag was again emptied.

The span was loaded again in 0.35 psi (2.4 kPa) increments. Pressure, steel and concrete strains, midspan deflection, and end slip were recorded. Any cracks along the sides of the span were noted at each load increment. Loading was stopped between increments and measurements were taken if significant slip, debonding, or cracking occurred before the next load level was reached. As the midspan displacement increased in later stages of the test, displacement increments were used instead of load increments. Measurements were taken at 0.5-in. (13 mm) displacement increments. If a plot of load versus displacement showed that the maximum load had been reached and that further loading only increased the midspan displacement with a decrease in load resistance, the input valve was shut and the bag was emptied.

After the test frame and bag were removed, cracks on the surface of the floor were noted. Areas where the steel deck had debonded from the concrete were estimated by tapping the bottom of the floor system. During the removal of the floor system, the steel deck surrounding a shear stud was examined for buckling.

RESULTS

A similar series of events occurred during each test. The first visible effect of the applied load was the formation of a transverse crack in the concrete over the interior supports. Subsequent load caused the formation of transverse cracks in the positive bending region. These were vertical cracks, typically described by and associated with flexural cracking in reinforced concrete slabs. With increased load the deck began to debond from the concrete near the location of the cracks. Debonding was often accompanied by an increase in the steel deck strain and sometimes a sudden drop in load. As the load continued to increase new transverse cracks formed near the location of maximum moment and existing cracks propagated through the depth of the concrete slab. The bottom flange of the steel deck in the positive bending region yielded, and midspan displacement increased significantly. Longitudinal cracks formed in the concrete over the deck seam connecting the two panels. Near the end of the test, midspan displacement and end slip increased significantly with only slight increases in load. The test was stopped when the maximum load had been reached and the midspan displacement was three inches or more.

Load versus displacement curves for specimens with shear studs for anchorage showed a gradual increase in displacement with increased load as illustrated in Figure 4. On the other hand, specimens with puddle welds for anchorage over the supports had irregular load versus displacement curves as illustrated in Figure 5. Peaks and plateaus mark sudden drops in load with increased displacement that accompany debonding, loss of deck anchorage, etc.

Test results are summarized in Table 2. A few general observations can be made. The strengths of specimens with shear studs for deck anchorage were higher than the strengths of specimens without shear studs. The highest loads were obtained with the 2-in. (51 mm), 18 gage deck profile. The next highest loads were obtained with the 3-in. (76 mm), 20 gage deck profile. The 2-in. (51 mm), 20 gage deck profile supported the lowest maximum load of the three decks. The strengths increased somewhat in proportion to the cross-sectional area of the deck. The bottom deck flange in the positive bending region yielded in every specimen. The strain in the deck at the exterior supports was below the yield strain in the specimens without studs, but above the yield strain in the specimens with studs.

The largest end slips occurred in the specimens without shear studs for anchorage, specimens three and seven. This is expected because the mechanical interlocking ability of the embossments alone does not compare to the ability of shear studs to resist slip. The midspan displacements at maximum load for the specimens with deck joints, specimens four and seven, were smaller than the displacements of specimens with continuous deck over the supports. These specimens were less ductile than specimens with continuous deck.

The cold-formed angle with the return lip significantly increased the strength of specimens three and four. The end restraint provided by the angle increased the capacity of SDI-2/20-P3-9 by 20 percent over the comparable span without the angle. Similarly for SDI-2/20-PX3-9, maximum capacity was increased 32 percent.

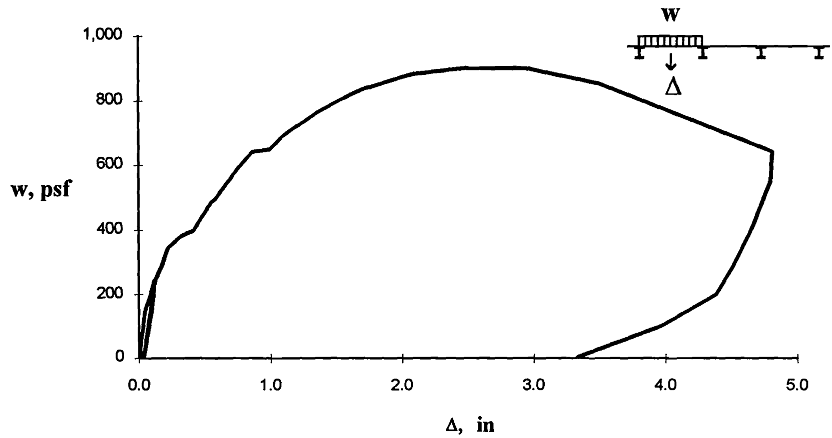


Figure 4. SDI-2/18-3-9 Load versus Displacement

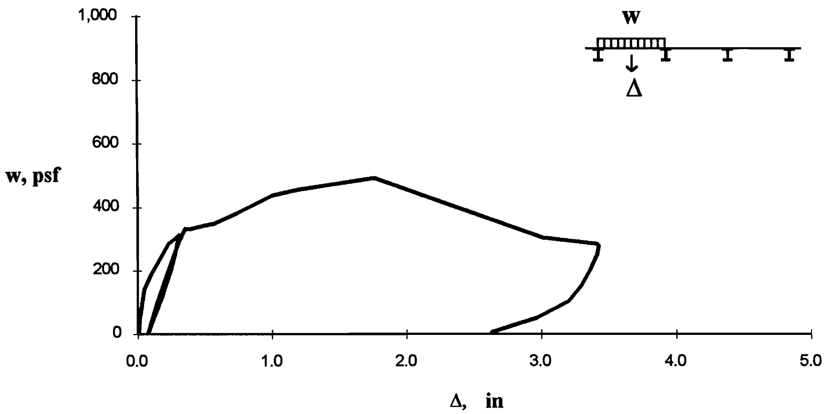


Figure 5. SDI-2/20-P1-9 Load versus Displacement

Table 2. Experimental Results

Specimen Number	Test Designation	f'c (psi)	Fy (ksi)	Maximum Load (psf)	Deflection at Max. Load (in)	End Slip at Max. Load (in)
1	SDI-2/20-4-9	3180	45	703	2.70	0.15
	SDI-2/20-5-9			729	2.61	0.11
2	SDI-2/20-2-9	5170	45	597	2.55	0.06
	SDI-2/20-23-9			598	3.17	N/A
	SDI-2/20-3-9			602	2.35	0.12
3	SDI-2/20-PI-9	3340	45	492	1.76	0.16
	SDI-2/20-P2-9			598	2.74	N/A
	SDI-2/20-P3-9			590	1.69	0.00
4	SDI-2/20-PX1-9	3770	45	369	1.59	0.10
	SDI-2/20-PX2-9			344	1.82	N/A
	SDI-2/20-PX3-9			488	1.77	0.00
5	SDI-2/18-3-9	5300	47	903	2.50	0.17
	SDI-2/18-35-9			891	3.10	N/A
	SDI-2/18-5-9			912	2.70	0.21
6	SDI-3/20-3-10	3750	50	743	2.58	0.12
	SDI-3/20-35-10			787	3.83	N/A
	SDI-3/20-5-10			891	3.49	0.12
7	SDI-3/20-PX1-10	3370	50	478	0.60	0.00
	SDI-2/18-PX3-9	3400	47	499	1.57	0.16

COMPOSITE DECK DESIGN HANDBOOK STRENGTH CALCULATION PROCEDURE

The SDI CDDH considers two distinct cases, composite slabs with and without headed shear studs on the beams. For beam designs that do not use shear studs, the limit state is yielding of the deck bottom flange. Stresses from fresh concrete on the bare deck are added to live load stresses on the composite slab system; these stresses are limited to 0.60 times the yield stress but not allowed to exceed 36 ksi (250 MPa). The SDI method then increases the live load calculated by using this method by 10% if welded wire fabric, with an area of at least 0.0075 times the concrete area above the flutes, is present. The increase in live load provided by the welded wire fabric is based on test results. This stress additive method, also called general strain analysis, is covered in the 1993 ASCE Standard (*Standard for* 1993) and also is in the SDI specification. However, the 10% allowance for welded wire fabric and the 0.6 F_y stress limit have been newly added by the SDI.

If a sufficient number of shear studs are present, yielding of the deck cross section occurs and strength design procedures are used. (The required anchorage force is addressed later in this section). The method to determine the allowable live load is summarized in the following paragraphs.

The design moment per unit width of slab is determined by the familiar equation

$$\phi M_n = \phi A_s F_y \left(d - \frac{a}{2}\right) \tag{1}$$

where $\phi = 0.85$, M_n = nominal moment capacity, A_s = the deck area per unit width, F_y = specified minimum yield stress, which is not to exceed 60 ksi (415 MPa) in the calculation, d = distance from the top of the slab to the centroid of the steel deck and a = depth of the compressive stress block.

The uniform service live loads, W_L, are found by using the relation

$$\phi M_n = \left[1.6W_L + 1.2W_D\right] \ell^2 C \tag{2}$$

where W_D = the weight of the concrete, the deck and any superimposed dead load, ℓ = the clear span and C = the bending coefficient which for cases with simple supports and uniform loads is 0.125. The factors 1.6 and 1.2 are live and dead load factors respectively.

The required stud anchorage force per unit width is estimated by:

$$F = F_y\left(A_s - \frac{A_{webs}}{2} - A_{bf}\right) \tag{3}$$

where A_{webs} = the area of the deck webs per unit width and A_{bf} = the area of the deck bottom flanges per unit width.

The nominal stud strength, Q_n, is given by Eq. I5-1 in the American Institute of Steel Construction Specification (*Load and* 1993) and repeated here:

$$Q_n = 0.5A_{sc}\sqrt{f_c' E_c} \le A_{sc}F_u \tag{4}$$

where A_{sc} = cross sectional area of the shear stud, f'_c = specified compressive strength of concrete, E_c = modulus of elasticity of concrete and F_u = specified minimum tensile stress of the shear stud material. The influence of the steel deck on the stud strength must also be accounted for using the deck reduction factor given by equation I3-2 in the AISC specification.

If the anchorage force provided by the actual number of studs is less than the force needed to develop the full nominal moment, then the available nominal moment is reduced accordingly. The reduction factor, R, is

$$R = \frac{N_r Q_n}{F} \leq 1.0 \tag{5}$$

where N_r = number of studs per rib. The lower limit for the reduced strength is that resulting from the stress additive technique based on no studs present.

The reader should note that the strength check in the CDDH for the slabs with shear studs on the beams is based on a load and resistance factor format, while the remainder of the design checks are based on an allowable stress design approach. Philosophically, this mixing of concepts is not desirable, but it was deemed the best way to handle the composite slab flexural calculations in the SDI method (Heagler, et al. 1991). The choice of allowable stress design for the format resulted because of the prevalence of this approach, both in design offices and among steel deck manufacturers. A future edition of the *Composite Deck Design Handbook* will likely include a complete limit states format.

Comparison of Experimental Data with CDDH Procedure

One of the primary objectives of this project is to evaluate the methods in the SDI CDDH and determine if a single method can be established to cover all degrees of anchorage. Currently there are two methods, one for the composite slab with studs and one for the slab without studs. Therefore, the test results will be discussed in two groups, specimens with and without studs. A comparison of observed strengths and predicted strengths is given in Table 3 and illustrated in Figure 6. Strengths of the composite floors are given in terms of the maximum moment produced at midspan assuming simple supports. The observed test moment is given as M_t. The predicted first-yield moment is given as M_{et}, and the predicted moment based on the under-reinforced flexural strength of the section is given as M_n. (Note that all calculations are based on measured material properties for comparisons with test results.) The calculation of M_n assumes the entire cross section of the steel deck at the location of maximum moment has yielded. These calculations are described in the literature (*Standard for* ASCE 1993; Easterling and Young 1992).

Figure 6 is normalized to the predicted moment, M_n, which varies for each specimen according to deck geometry, steel yield strain, and concrete strength. The two dashed lines indicate the range of first-yield moment ratios for all specimens. The ratio of M_{et} to M_n varies between 0.58 and 0.68. Moment ratios are plotted versus the stud, or weld, reduction factor, R, for the test. Under the current SDI CDDH procedure the line from the origin to the intersection of M_n represents the strength of composite floors with less than 100% anchorage ($R \leq 1$).

For the specimens without studs a puddle weld strength, P_n, was calculated using Eqs. E2.2-1 through E2.2-4 in the American Iron and Steel Institute LRFD Specification (*Load and* 1991). The anchorage force provided by the puddle welds is less than the force needed to develop the moment, M_n. Therefore, a reduction factor, R, is computed similar to Eq. 5

$$R = \frac{N_r P_n}{F} \leq 1.0 \tag{6}$$

where N_r = number of welds per rib.

The test moment exceeded the calculated moment M_n in all the tests where studs were present. However, the entire cross section of the steel deck did not yield in any of the tests as is assumed in the computation of the nominal moment. This suggests that the shear studs provided some rotational restraint at the supports, and the assumption of simply supported boundary conditions was not completely accurate. The test moment for specimens without studs exceeded the first-yield moment in all cases. SDI-2/20-P3-9, which has end restraint from a cold-formed angle with the return lip, slightly exceeded the moment M_n as well.

Table 3. Comparison of Experimental and Analytical Results

Test Number	Test Designation	Mt (ft-k)	Met (ft-k)	Mn (ft-k)	Mt / Met	Mt / Mn
1	SDI-2/20-4-9	42.7	23.2	34.4	1.84	1.24
2	SDI-2/20-5-9	44.3	23.2	34.4	1.91	1.29
3	SDI-2/20-2-9	36.3	23.6	36.0	1.54	1.01
4	SDI-2/20-23-9	36.3	23.6	36.0	1.54	1.01
5	SDI-2/20-3-9	36.6	23.6	36.0	1.55	1.01
6	SDI-2/20-PI-9	29.9	22.9	34.6	1.30	0.86
7	SDI-2/20-P2-9	36.3	22.9	34.6	1.58	1.05
8	SDI-2/20-P3-9	35.8	22.9	34.6	1.56	1.04
9	SDI-2/20-PX1-9	22.4	21.8	35.1	1.03	0.64
10	SDI-2/20-PX2-9	20.9	21.8	35.1	0.96	0.60
11	SDI-2/20-PX3-9	29.7	21.8	35.1	1.36	0.85
12	SDI-2/18-3-9	54.9	32.2	49.2	1.70	1.11
13	SDI-2/18-35-9	54.1	32.2	49.2	1.68	1.10
14	SDI-2/18-5-9	55.4	32.2	49.2	1.72	1.13
15	SDI-3/20-3-10	55.7	31.7	48.3	1.76	1.15
16	SDI-3/20-35-10	59.0	31.7	48.3	1.86	1.22
17	SDI-3/20-5-10	66.8	31.7	48.3	2.11	1.38
18	SDI-3/20-PX1-10	35.9	27.9	47.7	1.29	0.75
19	SDI-2/18-PX3-9	30.3	29.3	46.5	1.03	0.65

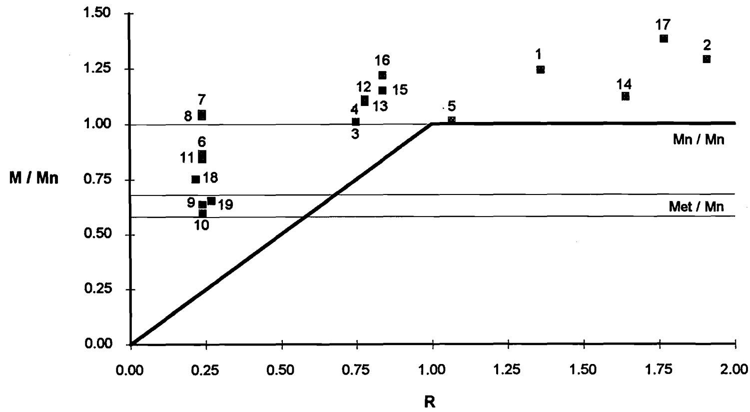


Figure 6. Comparison of Experimental and Analytical Results

RECOMMENDED MODIFICATIONS TO CDDH PROCEDURE

The results of this study support the use of simple reinforced concrete models for determining a safe design bending strength of composite floor systems. The results also suggest that a single unified method can be developed to predict the strength of composite floors regardless of the type or degree of anchorage. Figure 6 illustrates the current SDI approach to predicting the strength of floors with less than 100% anchorage. The reduction factor, R, applied to the nominal moment capacity, M_n, results in a line from the origin to the intersection of the nominal moment line at R = 1. The data from this study is well above this line. The computation of an approximate R value for decks with puddle welds (tests 6-8, 9-11, and 18-19) was discussed in the previous section.

Figure 7 illustrates the suggested modification to the CDDH method. Test results suggest that floors with studs that provide less than 100% anchorage and floors with puddle welds over the supports have a strength at least equal to the strength required to develop the first-yield moment of the slab. Therefore, the end of the line predicting the strength of slabs with less than 100% anchorage is shifted from the origin, or the point of zero moment, to the first-yield moment. In Figure 7 the two lines from the moment ratio axis to the intersection of the nominal moment line at R = 1 indicate a range for all specimens tested. Three tests fell outside this range (9, 10, and 19). These specimens had butted joints and puddle welds over the supports. If the strength reduction factor, ϕ = 0.50, (*Load and* 1991) is applied to the puddle weld strength, P_n, these data points shift to the left, and fall within or very close to the allowable range. However, it is recommended that M_{et} be used as the maximum strength for single span deck configurations.

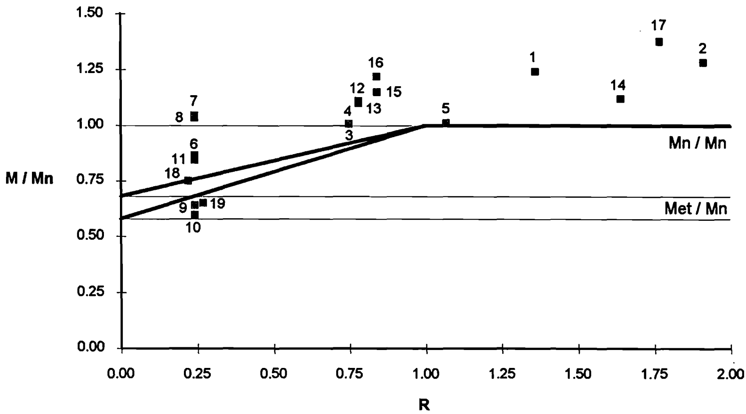


Figure 7. Comparison of Experimental and Modified Analytical Results

CONCLUSIONS

1. The strength of composite floors, regardless of the type of deck anchorage, can be calculated based on the under-reinforced flexural limit state. This strength must be adjusted for floors in which the end of the sheets are not sufficiently anchored to develop the required tensile force in the deck. The lower-bound strength is calculated based on first-yield of the extreme fibers of the deck.

2. Pour stops significantly increase the strength of composite floor systems. An analytical method of predicting the additional capacity has not yet been established, however, is the subject of continuing study.

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APPENDIX.--NOTATION

A_{sc}	=	area of shear stud
A_{webs}	=	area of deck webs per foot of width
a	=	depth of concrete compressive block
C	=	bending coefficient for positive moment
d	=	distance from top of slab to centroid of steel deck
E_c	=	concrete modulus of elasticity
F	=	required anchorage force per foot of width
F_u	=	specified minimum tensile stress of shear stud
F_y	=	specified minimum yield stress of deck
f'_c	=	specified compressive strength of concrete
l	=	clear span
M_{et}	=	predicted first-yield moment
M_n	=	predicted nominal moment (using nominal material properties), predicted maximum moment (using measured material properties)
M_t	=	observed test moment
N_r	=	number of studs or welds per rib
P_n	=	nominal puddle weld strength
Q_n	=	nominal shear stud strength
R	=	stud spacing reduction factor
W_D	=	weight of concrete, deck, and superimposed dead load
W_L	=	uniform service live load