An Introduction to Composite Steel Joists – Research and Behavior

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INTRODUCTION

History of Composite Steel Joists

The first experimental research conducted in the U.S. utilizing open-web steel joists as part of a composite joist system was carried out in the 1960's [Lembeck, 1965; Wang and Kaley, 1967]. Composite action in the earlier testing program was achieved by inverting and lowering the top chord angles so that the webs extended above the top chord into the concrete slab. Additional shear connection was created by the use of 1/2 in. (12.7 mm) diameter filler rods welded to the top chord between the panel points. The tests were compared to conventional joists with the same theoretical design load and the results showed that the composite steel joists were stiffer, having about a 20 percent reduction in deflection at the design load. The composite joists also attained an ultimate moment approximately 14 percent higher than the conventional joists that were tested. In the later experimental program, composite action was achieved by providing a longitudinal shear key along the one-piece top chord of the joists. Supplemental shear connection was provided in some of the tests by adding continuous metal chairs into the top chord that were shaped like a bulb. In both research projects, the results indicated that it was possible to achieve composite action in open-web steel joist construction.

Tide and Galambos [1970] performed tests on five composite steel joists that used 3/8 in. (9.5 mm) diameter x 2 in. (51 mm) long shear studs welded to the joist top chords. A 3 in. (76 mm) concrete slab was cast over each of the joist specimens. The main purpose of the research was to investigate the degree of composite action that could be obtained by studying the stud shear connector behavior in a composite system comprised of open-web steel joists, a cast-in-place concrete slab, and mechanical shear connectors holding the two together. The researchers varied the type and size of the joist top and bottom chords as well as the number and location of stud shear connectors. The web members were over-designed in all the test specimens to ensure there would be no web failures in the experiments.

It wasn't until the mid 1980's that there was a renewed interest in composite design using open-web steel joists. At the University of Minnesota, Leon and Curry [1987 and Curry, 1988] reported on the testing of two full-scale, 36 ft. (10.97 m) long span composite steel joists to failure. Each test specimen was constructed with 2 in. (51 mm) composite steel deck, 3/4 in. (19 mm) diameter headed shear studs, and normal weight concrete with a nominal strength of 4 ksi (27.6 MPa). Alsamsam [1988] tested another two full-scale specimens to failure. The major result of the four tests was that the composite beam model could be used to predict the ultimate moment capacity of composite steel joists.

The next series of two full-scale tests were conducted at the University of Nebraska - Lincoln by Patras and Azizinimini [1991]. The composite steel joists were 36 feet (10.97 m) long with a nominal depth of 12 inches (305 mm). Top and bottom chords of both specimens consisted of two equal leg angles welded back to back. Web members consisted of equal leg angles placed on the outside of the chords. Vulcraft's 2VLI 20 gauge, galvanized deck supported the 4 inch (102 mm) total concrete slab. Shear connectors, 3/4 inch (19 mm) diameter x 3 ½ inches (89 mm) long after welding, were welded through the metal deck to the steel joist top chord angles. Light weight concrete was utilized for both specimens. Test specimen CH-1 was designed for a nominal strength of 3 ksi (20.7 MPa) while CH-2 was designed for a nominal strength of 12 ksi (82.8 MPa). Crushing of the concrete adjacent to the shanks of the "Weak" position shear studs in CH-1 was observed while there was no noticeable concrete crushing in CH-2 in the vicinity of the shear studs. Test results also showed that the higher strength concrete in CH-2 exhibited a higher stiffness as expected. Ultimate load-carrying capacities were accurately predicted for both test specimens.

For over 35 years, similar research was ongoing in Canada to determine whether composite steel joist construction was feasible. Azmi [1972] conducted six tests on composite joists with 50 ft. (15.24 m) spans. In addition to the testing, a design model was developed that showed good correlation with the experimental data. The model was based on three levels of shear connection: Under-connected, balanced, and over-connected which related the stud shear strength to the tensile yield force in the bottom chord of the joist. The research continued with Fahmy [1974] who developed a finite difference method to analyze the behavior of composite steel joists in both the elastic and inelastic regime that considered two different methods for shear connection, puddle welds and shear studs. The method was verified and showed good agreement with the test results of Azmi as well as with two additional experiments conducted as part of this research. The numerical method was later refined by Robinson and Fahmy [1978] for the case of partial composite action between the open-web steel joists and the concrete slab.

A fundamental reference on the design and construction of composite floor systems was published by Chien and Richie [1984]. Contained in the work was a chapter devoted to the design of open-web steel joists. The design criteria was based on ultimate strength methods, but also addressed the issues of serviceability and connections.

More recent research in Canada has dealt with the effect of concrete shrinkage on the behavior of composite steel joists [Kennedy and Brattland, 1992]. The authors tested two full-scale 38 ft. (11.58 m) specimens to failure, one at 65 days and the other at 85 days. It was found that the majority of the shrinkage occurred in the first 30 days. The failure loads that the specimens attained closely matched predictions based on an ultimate strength method with only the bottom chord in tension.

By far the most extensive research program has been carried out at Virginia Polytechnic and State University. This research, sponsored by Nucor Corporation starting in 1991 [Gibbings and Easterling, 1991], [Nguyen, Gibbings, Easterling, and Murray, 1992], [Sublett and Easterling, 1992], [Lyons, Easterling, and Murray, 1994], [Roddenberry, Easterling, and Murray, 2000 and 2002] and most recently [Avci and Easterling, 2003], have all examined the behavior of composite steel joists. The projects have concentrated on the ultimate strength of the composite joists and the development of better prediction models or the behavior and strength of the stud shear connectors as part of the overall composite system. Table 1 provides a summary of the major research projects conducted in the U.S. on composite steel joists over the past 20 years.

TABLE 1 SUMMARY OF COMPOSITE STEEL JOIST RESEARCH PROJECTS

Test No.	Joist Depth (in.)	Joist Span (ft.)	Top Chord	Bottom Chord	Deck Type	Slab Thickness Above Deck (in.)	f_c' (ksi)	γ_c (pcf)	Slab Width (in.)	Studs (No. – Dia.)
CJ-1[1]	21	36	2L-3x3x.227	2L-3.5x3.5x.344	2VLI	4	4.86	145	96	37-3/4
CJ-2[1]	21	36	2L-3x3x.227	2L-3.5x3.5x.344	2VLI	4	4.11	145	96	37-3/4
CJ-3[1]	8	24	2L-2x2x.187	2L-3.0x3.0x.250	1.5VL	2.5	3.97	145	72	26-5/8
CJ-4[1]	20	22.5	2L-1.75x1.75x.188	2L-2.5x2.5x.250	1.5VL	2.5	3.97	145	67	24-5/8
CH-1[2]	12	36	2L-2x2x.250	2L-3x3x.313	2VLI	1.625	4.31	120	48	22-3/4
CH-2[2]	12	36	2L-2x2x.250	2l-3x3x.313	2VLI	2	10.97	150	48	22-3/4
CLH-1[3]	36	56	2L-2.5x2.5x.313	2L-3.5x3.5x.313	3VLI	3	4.43	145	102	22-3/4
CLH-2[3]	36	56	2L-2.5x2.5x.313	2L-3.5x3.5x.313	3VLI	3	4.18	145	102	38-3/4
CLH-3[3]	16	40	2L-3.5x3.5x.313	2L-5x5x.438	3VLI	3	4.0	145	81	66-3/4
CLH-4[3]	16	40	2L-3.5x3.5x.313	2L-5x5x.438	3VLI	3	3.1	145	81	44-3/4
CLH-5[3]	34	40	2L-3.5x3.5x.313	2L-3.5x3.5x.313	3VLI	3	5.86	145	81	22-3/4
CLH-6[3]	14	40	2L-3x3x.313	2L-4x4x.438	2VLI	3	4.43	145	81	36-3/4
CLH-7[3]	20	40	2L-3x3x.313	2L-4x4x.438	2VLI	3	5.72	145	81	36-3/4
CLH-8[3]	20	40	2L-3x3x.313	2L-4x4x.438	2VLI	3	5.38	145	81	36-3/4
CLH-9[3]	32	40	2L-3x3x.313	2L-3x3x.313	2VLI	4	3.17	145	81	22-3/4
CLH-10[3]	32	48	2L-3x3x.313	2L-3.5x3.5x.287	2VLI	2.5	3.35	145	81	32-3/4
CLH-11[3]	16	40	2L-3x3x.313	2L-4x4x.438	2VLI	2.5	2.90	110	81	40-3/4
CSJ-5[3]	12	30	ST-2x3.85	2L-2.5x2.5x.212	1.5VL	2	4.40	145	40	12-3/4
CSJ-6[3]	18	30	2L-1.5x1.5x.123	2L-2x2x.163	1.0C	3	4.2	145	40	22-1/2
CSJ-7[3]	18	30	2L-1.5x1.5x.123	2L-2x2x.163	1.0C	3	3.6	145	40	12-1/2

[1]Test performed at University of Minnesota, Minneapolis, MN (Alsamsam, 1988), (Curry, 1988)
[2]Test performed at University of Nebraska-Lincoln, Lincoln, NE (Patras and Azizinimini, 1991)
[3]Test performed at Virginia Polytechnic Institute and State University, Blacksburg, VA (Nguyen, Gibbings, Easterling, and Murray, 1992), (Gibbings and Easterling, 1991), (Lauer, Gibbings, Easterling, and Murray, 1996)

COMPOSITE MODEL

Research performed on composite joists indicates that the moment capacity can be calculated utilizing a simple model similar to one used for composite construction as shown in Figure 1. The tension force in the bottom chord to reach its yield strength, $A_{bc}F_y$, is balanced by an equal compression force in the concrete slab of $0.85f'_c\, a\, b_e$.

With the centroid of the joist top chord being very close to the centroid of the concrete compression area, one has a very small gain in moment capacity if sufficient shear studs are provided to develop the top chord. It is far more efficient to provide only sufficient shear connection to fully develop the joist bottom chord. For this reason, the potential tension force in the top chord is not included in the composite joist moment capacity.

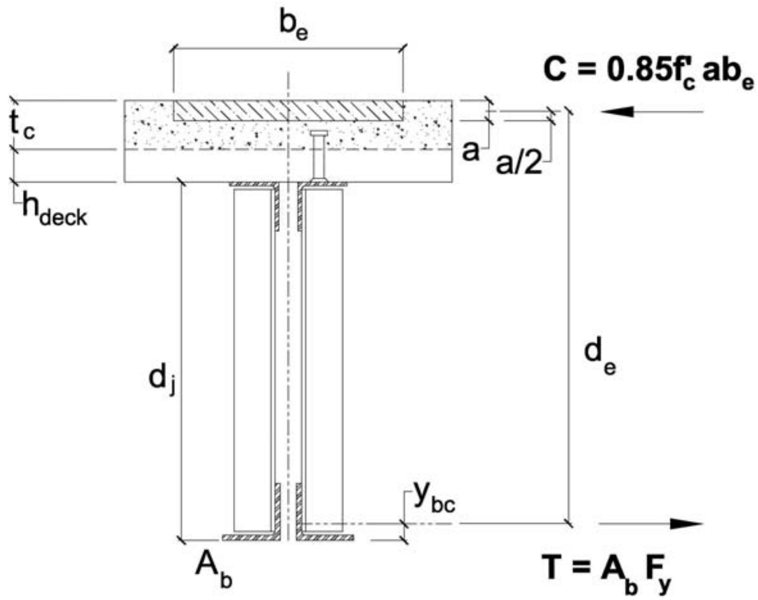

FIGURE 1 COMPOSITE JOIST MODEL

BOTTOM CHORD BEHAVIOR

As load is first applied to a composite joist, the bottom chord elastically strains. Generally, the SJI **CJ**-Series joists [SJI, 2007] have an allowable total composite load that stresses the bottom chord to approximately 60% of the load necessary to yield the bottom chord. Removal of any of the load prior to the bottom chord reaching yield merely results in a linear elastic reduction in the bottom chord strain. As additional load is once again applied to the joist, the bottom chord will eventually reach its yield strength and then continue to elongate inelastically. If the load is reduced after the bottom chord is in the inelastic region, this will result in a linear elastic reduction in the member strain that would be parallel to the elastic strain that occurred during loading. A permanent deflection will be seen when the composite live load is reduced. As can be seen from Figure 2 and from load –deflection curves, composite joists can be designed to behave in a ductile manner.

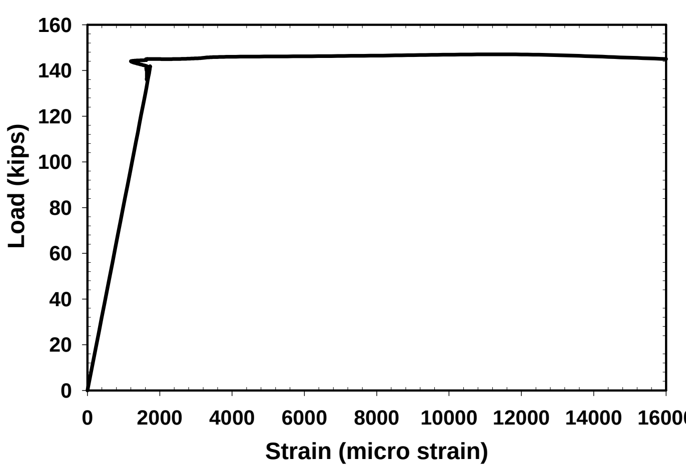

FIGURE 2 LOAD VERSUS STRAIN FOR A TYPICAL COMPOSITE JOIST BOTTOM CHORD

TOP CHORD BEHAVIOR

When the metal deck, bridging, and concrete slab are applied to a composite joist, the joist is non-composite until such time that the concrete has reached sufficient strength to transfer load from the joist top chord through the shear stud connectors into the concrete slab. Prior to this point in time the joist acts identically to a non-composite joist as shown in the initial portion of the loading curve shown in Figure 3.

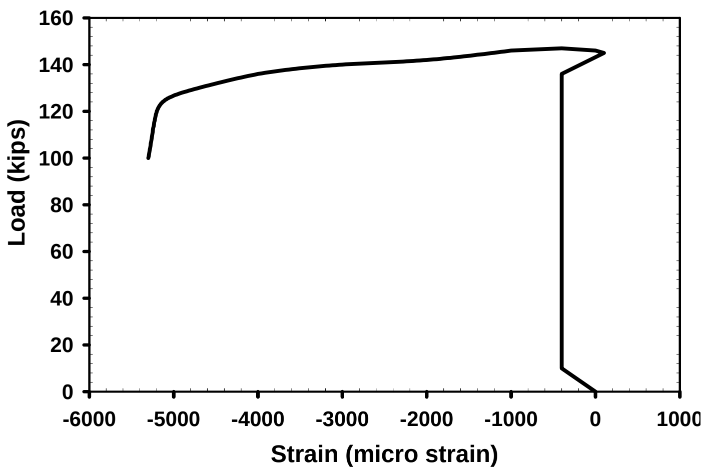

FIGURE 3 LOAD VERSUS STRAIN FOR TYPICAL COMPOSITE JOIST TOP CHORD

After the concrete has cured, additional load is transferred through the shear stud connectors to the concrete slab. The net result is an increase in the compression within the top chord that is very small. Once the bottom chord starts to yield, and if sufficient shear connectors are provided, the top chord will go into tension. Generally, sufficient shear connection is not provided to fully develop the top chord so this may not occur. Once the shear stud connections start to fail, the tension force that has developed within the top chord will start to reduce. Eventually, the top chord continues to develop more and more compression until the composite joist reaches its ultimate moment capacity.

WEB BEHAVIOR

Full-scale load tests have demonstrated that the webs of a composite joist behave in a similar manner to those found within a non-composite joist. Simply stated, the webs must be capable of transferring the vertical shear from the interior of the joist to the two end reactions. Prior to yielding of the bottom chord, the web members behave linearly elastic. Once the bottom chord starts to yield and shear connection starts to be lost, the forces in the web members reduce accordingly.

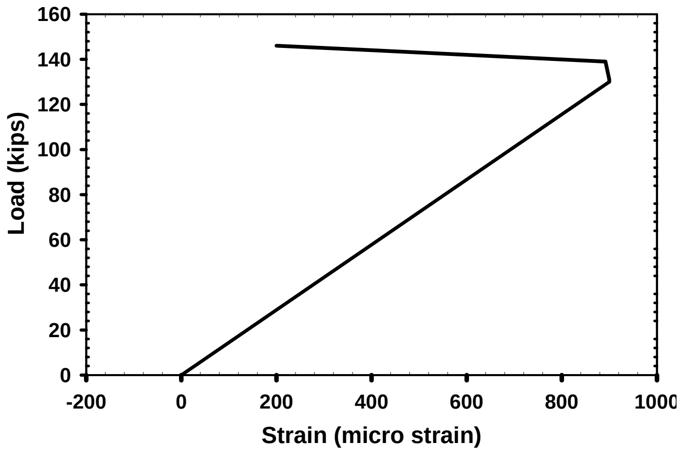

FIGURE 4 WEB BEHAVIOR

SHEAR STUD BEHAVIOR

One of things noted in the initial full-scale composite joist tests [Gibbings and Easterling, 1991] was that the location of the shear studs within the deck rib influenced the ultimate load-carrying capacity of the test specimens. When 2 or 3 inch (51 or 76 mm) composite deck is utilized, these deck profiles typically have a stiffening rib at the center of each bottom rib. This requires that the shear stud be located on either one side or the other of the deck stiffening rib. For shear studs located on the side of the deck stiffening rib closest to the center of the joist span (typically referred to as the "Weak" position) lower shear stud capacities were noted. Conversely, if the shear studs were located on the side of the deck stiffening rib closest to the ends of the joists (typically referred to as the "Strong" position) increases in shear stud capacity of approximately 20 – 30% could potentially be obtained [Roddenberry, Easterling, and Murray, 2002].

CONCLUSIONS

A significant amount of data has been accumulated from testing that has been performed on composite steel joists over the past 40 years. The results of these experimental and analytical research projects have been reviewed by the Steel Joist Institute's Composite Joist Committee and the committee determined that it is possible to accurately predict the load-carrying capacities and deflection behavior of composite steel joists. The committee also found that the ultimate strength models that were originally developed for composite wide flange beams are applicable to composite steel joists. Based on all the available information to date on the behavior of composite steel joists, the committee was able to develop the new Standard Specifications for Composite Steel Joists, **CJ**-Series that has been published by SJI in the First Edition 2007 Composite Steel Joist Catalog.

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