Strength of Composite Slabs

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by

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SUMMARY

This paper describes results to date of a current research program at Virginia Polytechnic Institute and State University under the sponsorship of the Steel Deck Institute. Full scale multi-span tests of composite floor systems are the basis of the experimental program. A primary objective of the research is to assess the strength of steel deck reinforced concrete floor slabs that are constructed to simulate actual field conditions, with respect to details at the intermediate supports and at end spans. In particular the influence of adjacent spans and typical pour stop details are considered.

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1.0 INTRODUCTION

Cold-formed steel deck has been a part of floor systems in buildings since the late 1940's. Initially, the deck was used strictly as a stay-in-place, or permanent, form. Not long after the first uses, engineers recognized the potential for utilizing steel deck as tensile reinforcement, thus improving the efficiency of the floor systems.

As the desire to use the deck as reinforcement became greater, so did the need to perform design calculations. Predicted strengths based on ultimate strength reinforced concrete theory did not agree with laboratory tests of the slab elements. Continued attempts to develop analytical methods, which are not dependent on experimental testing, have thus far not been completely successful.

Instead, the current design standard in the United States is based on a testing program that produces data from which statistical coefficients are obtained (*Specifications* 1984). These coefficients are then used, along with design parameters, to arrive at design live loads. This method resulted from an extensive research program at Iowa State University that was initiated by the American Iron and Steel Institute (Porter and Ekberg 1978). The approach, in similar form, is used in the European and Canadian design documents.

The experimental configuration in the U.S. standard is a single span, single panel width specimen. This arrangement, while convenient for the testing agency, has several details, which do not accurately reflect field conditions. One such detail is the lack of proper representation of end span and adjacent span details. Due to the lack of end restraint, which would typically be present in constructed floor systems, the predominant limit state in past laboratory tests is shear bond. This limit state is characterized by a breakdown of the bond between the steel deck and concrete in the shear span. The concrete is then essentially free to slip relative to the deck. Pour-stop, or closure angle details, and adjacent spans have a significant influence on inhibiting or preventing the shear bond limit state.

2.0 OBJECTIVE AND SCOPE

The primary objective of this study is to determine the influence of typical field details on the strength of composite floor systems. An additional objective is to evaluate the applicability of using traditional reinforced concrete models to predict the strength and stiffness of composite floor systems. To achieve this objective, a series of full-scale tests are being preformed and evaluated with such models. To date six tests have been completed and the program is ongoing.

Several specimen configurations are being evaluated. A three span setup permits the influence of adjacent spans on the strength of the center span to be studied. Or, the end spans can be tested to evaluate different pour stop details. To date, two center span tests and four end span tests have been completed. Four end detail configurations have been studied thus far. These are a cold-formed angle with no return lip, a hot rolled angle, a cold-formed angle with a return lip, and a cold-formed angle with a return lip and shear studs.

3.0 BACKGROUND INFORMATION

This section gives a brief historical background of the research performed in the U.S. that pertains to composite slabs. It should not be viewed as a complete literature survey, since significant European, Canadian and Australian publications are not included in the review.

3.1 History

In the second half of the nineteenth century, engineers began using reinforced concrete. One drawback of this material was the requirement of expensive and cumbersome formwork which, in many cases, was unsalvageable. For beams, columns and wall panels this problem was solved with precasting techniques and reusable form work. However, the problem remained for floor systems.

The H.H. Robertson Company started production, in 1938, of permanent steel deck form work which was known as the "keystone beam." The keystone beam was a cellular floor system used primarily in low rise buildings. Even though the steel deck form work was permanent it was not considered or designed to act compositely with the floor system.

Realizing the possibility of utilizing permanent steel form work as part of the required reinforcement, the Granco Steel Products company combined the deck with wires welded transverse to the ribs. This allowed transfer of the horizontal shear between concrete and deck and provided composite action. Due to the use of brittle high strength steel in the deck and wires, the ductility of the floor system was a concern. Even with this as a potential drawback, the product, which became known as "Cofar", gained acceptance and significantly reduced the cost of concrete floor systems. In the late 1950's and early 1960's the welded wire used as a horizontal shear transfer device was replaced with embossments and indentations within the deck.

The expansionism of the early 1960's produced an increased demand for the new composite formwork. To meet this charge, several manufactures introduced their version of steel deck. No design standards existed therefore individual manufactures had to verify that their designs were adequate. This was often done by performing numerous laboratory tests. Bryl (1967) made three critical observations as a result of manufacturer tests: (1) if no shear transfer devices are used a sudden failure occurs, (2) if shear transfer devices are present large deformations occurred and the load carrying capacity increased dramatically, and (3) the slab could be analyzed as uncracked with respect to bending, bond stresses, and permissible load on shear devices.

In order to gain a better understanding of the behavior of composite floor systems, in 1967 the American Iron and Steel Institute initiated an extensive research project at Iowa State University (ISU). The purpose of this study was to analyze the behavioral characteristics and develop a design standard for composite steel deck reinforced concrete (SDRC) floor systems. At approximately the same time, independent studies with similar objectives began at West Virginia University (Prasannan and Luttrell 1984).

The outcome of the ISU project is documented in several thesis, reports and papers (Ekberg and Schuster 1968; Porter and Ekberg 1971, 1972, 1976, 1977; Schuster 1972). Several hundred single panel tests were conducted on what has now become the standard test setup. One result from the study is the classification and description of limit states. These are shear bond which is the breakdown of adhesion bond between deck and concrete when first cracking occurs, under-reinforced flexure which occurs when the steel deck and concrete have adequate bonding and yielding of the steel deck occurs, and over-reinforced flexure which occurs when the bond is adequate and the concrete crushes before full yielding of the deck. Porter and Ekberg (1978) reported that, of the three modes, by far the governing mode of failure was shear bond. This failure generally starts with the formation of a crack under the applied load point, followed closely by the loss of bond between the load point and support reaction. With this information Porter and Ekberg presented the final form of the empirical equation for shear bond failure, which was initially developed by Śchuster (1976).

The equation given by:

$$V_u = \phi \left[\frac{12d}{S} \frac{m\rho d}{L'} + k\sqrt{f'_c} + \frac{\gamma w_1 L}{2} \right]$$

where,

V_u = design shear strength
ϕ = capacity reduction factor
S = spacing of shear transferring devices, in.
d = distance from extreme compressive fiber to centroid of deck, in.
m, k = slope and ordinate intercept of regression line developed from laboratory test program.
γ = shoring reaction factor
L' = shear length (distance from load to support),ft.
L = clear span length, ft.
w_1 = slab dead load, psf
ρ = reinforcement ratio
f'c= concrete compressive strength, psi.

This expression is dependent on a laboratory test program, from which the coefficients m and k are determined. Aspects of the floor system that this approach does not consider include typical boundary conditions that exist in the field, which confine the concrete and limit end slip. The ISU method is the basis for the U.S. design standard (*Specifications* 1984).

Prasannan and Luttrell (1984) developed an approach for the strength determination of composite slabs. The approach is based on a statistical evaluation of previously obtained test data. Regression analyses were performed with various slab and deck properties being the independent variables, and the theoretical moment capacity being the dependent variable. This method has undergone modification and refinement and will be part of the next edition of the ASCE Standard.

The method is attractive to steel deck manufacturers because it gives them a way to predict the performance of a potential new deck profile and embossment pattern, without having to go to the expense of fabricating new rolling stands to roll the profile and perform numerous tests in advance. If the profile is developed and manufactured confirmatory testing would then be performed. The method is fundamentally based on the test setup that is used in the ASCE Standard, therefore it does not reflect the influence of the typical end span details.

The equation is given by,

$$M_t = \frac{K\,M_{et}}{C_s}$$

where,

$K = \dfrac{k_3}{k_1 + k_2} \le 1.0$

M_{et} = Moment based upon first yield of extreme deck fibers, k-in / cell
k_1, k_2 = design variables to account for deck type variation and slab depth.
k_3 = design variable for number of deck flutes
C_s = cell spacing

4.0 EXPERIMENTAL PROGRAM

This section of the paper describes the test setup, testing procedure and results. The test identification is of the form SDI-*i*-*j*, where *i* indicates the slab number and *j* indicates the test number for the particular slab.

4.1 Test Setup

4.1.1 General

In all tests, a three span setup was used. For a given slab, either the center span was loaded or the two end spans were loaded. Figure 1 shows a schematic of the test setup for a center span test. The length of each span was eight feet (2.44 meters) center to center of supports and the total width was six feet (1.83 meters). Concrete was placed five inches (127 mm) deep, measured from the bottom of the deck to the top of the slab. The steel deck used was a 2 in. (50.8 mm), 20 gage galvanized trapezoidal section with web embossments. All of the tests except SDI-3-1 used steel deck with a nominal yield stress of 33 ksi. (248.22 MPa). SDI-3-1 used steel deck with a nominal yield stress of 80 ksi (551.6 MPa). No negative moment reinforcement or shrinkage and temperature steel was provided. The concrete was covered and kept moist for seven days and then allowed to air cure. Form-work along the edges was removed after seven days. Air temperature was not allowed to drop below 65° F (1 C) for the duration of the cure period.

Strain gages were placed on the bottom side of the deck at the middle of each of the three spans. To measure strain variation on the cross section, gages were placed on the bottom flange, the top flange, and except for test number SDI-3-1 on the web. In addition, with the exception of test numbers SDI-1-1, SDI-2-1, SDI-2-2, strain gages were placed on the bottom flange at one foot (305 mm) intervals along the entire tested span. Deflection transducers were placed at midspan and at the quarter points of the span being loaded. Additionally, transducers were placed at midspan of the two spans that were not being loaded. Dial gages were placed at the ends of the specimens to measure slip between the frame and the end of the slab.

All instruments were zeroed prior to the application of the spreader beam system. The first load point consisted of the weight of the spreader beams and associated plates and pads. Subsequent loading was applied with a hydraulic cylinder connected to the test frame. Load was measured by a load cell at this location. The point load of the cylinder was distributed by the spreader beam system which distributes the load to the slab as two line loads transverse to the span. The line loads were located 30 inches (764 mm) from the middle of the supports for the span being loaded.

In the following discussion, the hot rolled angle reference is a L5x5x1/4 (L127mm x 127mm x 6.35mm), the cold-formed angle without a return lip is a L5x5x0.048 (L127mm x 127mm x 1.22mm), and the cold-formed angle with return lip is the same as above except with a one inch (25 mm) lip along the top edge, turned into the slab at a 45° angle. All angles were attached to the support members by one inch (25 mm) welds placed at one foot (305 mm) intervals along the toe of the attached leg. Intermittent tack welds were placed as needed along the heel of the angles to prevent distortion of the member during the welding process. Figure 2 shows the various end span details.

4.1.2 SDI-1-1, SDI-2-1, SDI-2-2, SDI-4-1, and SDI-4-2

Specimen configuration for these tests consisted of the general setup with two panels connected, by crimping, at approximately 10 in. (250 mm) intervals to form the six foot (2.44 meter) width. Steel deck with a measured yield stress of 40 ksi (275.8 MPa) and a ultimate strength of 59 ksi (406.8 MPa) was used for these tests. The area of steel was 0.5213 square inches per foot width (1103.5 mm^2 per meter of width) and the moment of inertia of the deck was 0.409 in^4 per foot width (558,521.9 mm^4 per meter width).

SDI-1-1 was a center span test, with the boundary conditions of adjacent spans on each end. SDI-2-1 was an end span test, with the boundary conditions of an adjacent span on one end and a hot rolled angle on the other end. SDI-2-2 was an end span test, with the boundary conditions of an adjacent slab on one end and a cold-formed angle without a return lip on the other end. SDI-4-1 was an end span test, with the boundary conditions of an adjacent slab on one end and a cold-formed angle with a return lip and shear studs on the other end. SDI-4-2 was an end span test, with the boundary conditions of an adjacent span on one end and a cold-formed angle with a return lip on the other end.

4.1.3 SDI-3-1

Test SDI-3-1 consisted of three, equal, triple panel width, continuous spans and was a center span test. The steel deck for this test had a measured yield stress of 90 ksi (620 MPa) and a tensile strength of 94 ksi (648 MPa). The area of steel was 0.5281 square inches per foot width (1117.8 mm^2 per meter of width) and the moment of inertia of the deck was 0.399 in^4 per foot width (544,866.1 mm^4 per meter width).

SDI-3-1 was a center span test, with the boundary conditions of adjacent spans on each end.

4.2 Test Results and Observations

For the SDI-2 series the strain gages were monitored, during the placement of the concrete. An average strain of 120 micro strain was recorded at the bottom flange of the center span and 290 micro strain at the bottom of the two end spans. For the SDI-4 series, an average strain of 80 micro strain was recorded at the bottom flange of the center span and 264 micro strain at the bottom of the two end spans.

4.2.1 SDI-1-1

The concrete compressive strength on the day of the test was 4330 psi (29.9 MPa).

The loading program proceeded by beginning at the first load point as described above. After this, the load was increased in approximately one kip (4.44 kN) increments until it became necessary to proceed in increments of displacement (approximately 22.2 kips (98.75 kN)). Load was then applied in midspan displacement increments of 0.05 in. (1.25 mm) change in midspan deflection. Loading continued until two inches (50.8 mm) of deflection was recorded. At this point, the test was stopped and unloaded.

Cracking over the supports was observed at a moment of 58.5 k-in. (6.6 kN-m). At a moment of 220.5 k-in. (25 kN-m), cracking under the spreader beams occurred but no slip at the ends of the slab was measured. Separation of the deck and concrete between the spreader beams was observed at a moment of 315 k-in. (35.6 kN-m). The concrete and deck were in contact between the spreader beams and the support members. The maximum applied moment was 400.5 k-in. (45.25 kN-m) with no measured end slip occurring. A plot of the moment verses deflection, shown in Figure 3, reveals that there is a gradual change in the slope of the curve and a long plateau of yielding of the steel deck.

4.2.2 SDI-2-1

The compressive strength of the concrete for this slab was 4700 psi (32.5 MPa).

The loading sequence for this test was similar to SDI-1-1 except that the load was increased in three kips (13.3 kN) increments until a load of 22.3 kips (99.2 kN) was reached, at which point midspan deflection increments of 0.05 in. (1.25 mm) were used. Loading was terminated when two inches (50.8 mm) of deflection was recorded. No slip between the deck and concrete occurred until after ultimate load.

Cracking over the supports was present before the loading process began. At a moment of 184.5 k-in. (20.8 kN-m) separation of the concrete and hot rolled angle occurred. At a moment of 195 k-in. (22 kN-m) cracking under the load points occurred. At a moment of 363 k-in. (41 kN-m) separation of the deck and concrete between the spreader beams was observed. At several points during the loading process, the slab was unloaded and then reloaded. The different stiffness values of each unloading can be seen in the plot of moment verses displacement (Figure 4). At the loading point that caused cracking under the loads points, an appreciable change in stiffness can be seen on the graph.

Observation of the end detail during the loading showed that the concrete and deck were rotating about the inside edge of the top flange of the outer support member and thus the end of the concrete was riding up the angle. A comparison of Figures 3 and 4 indicate that the behavior observed in SDI-2-1 is considerably less ductile than that of SDI-1-1. In the post ultimate range for SDI-2-1, the deck tore around the puddle welds at the end of the span and slip between the deck and concrete occurred.

4.2.3 SDI-2-2

Concrete compressive strength for this test was 4700 ksi (32.5 MPa).

The loading sequence was the same as for SDI-2-1 with the exception that no unloading occurred. The transition from load control to displacement control occurred at a load of 24.3 kips (108.1 kN). No slip between the deck and concrete occurred until after ultimate load.

Cracking over the supports was present before the loading process began. At a moment of 237 k-in. (26.8 kN) cracking under the spreader beams occurred without a significant drop in load, as can be observed from Figure 5. An ultimate moment of 364.5 k-in. (41.2 kN-m) was reached with a corresponding midspan displacement of 0.91 in. (23.2 mm). After ultimate load, at a moment of 337.5 k-in. (38.1 kN-m), separation of the pour-stop and concrete occurred suddenly. Similar to SDI-2-1, this test was less ductile than the center span test. Also, as in SDI-2-1, the deck ripped out around the puddle welds and slip between the deck and concrete occurred in the post ultimate range. The concrete was again rotating about the inside edge of the top flange of the support member thus causing the end of the concrete to ride up and bend the cold-formed angle.

4.2.4 SDI-3-1

Compressive strength of the concrete for this test was 4400 psi (30.4 MPa).

For this test the strain gages were monitored, during the placement of the concrete, and an average strain of 64 micro strain was recorded at the bottom flange of the center span and 150 micro strain at the bottom of the two end spans.

The loading sequence was similar to that of previous tests. Two unloading cycles were performed at selected points along the loading path, as can be seen in Figure 6. Unloading number one occurred at a moment of 148.5 k-in. (16.78 kN-m) to obtain an approximate uncracked stiffness and the second unloading occurred at a moment of 270.0 k-in. (30.51 kN-m). No appreciable slip of the deck and concrete occurred after ultimate load had been reached. The specimen was loaded to an ultimate moment of 802.5 k-in. (90.7 kN-m).

Cracking over the supports occurred at a moment of 162.0 k-in. (18.3 kN-m). At a moment of 260.0 k-in. (29.5 kN-m) cracking under the load points occurred. Some cracking between the two loading points occurred at a moment of 717.0 k-in. (81 kN-m). The test was stopped when a midspan deflection of two inches (50.8 mm) was recorded. Observations at the end of the test indicated that the deck was ripping out around the puddle welds over the supports.

4.2.5 SDI-4-1

The compressive strength of the concrete for this test was 4600 psi (31.8 MPa).

The loading sequence for SDI-4-1 was similar to those of previous tests. Four unloading cycles were performed during the course of the test, as can be seen in Figure 7. The first at a moment of 127.5 k-in. (14.4 kN-m). to obtain an approximate uncracked stiffness. The subsequent unloading cycles were made at moments of 247.5 k-in. (27.96 kN-m), 393 k-in. (44.4 kN-m) and 426 k-in. (48.1 kN-m). The last being made in the post-ultimate range.

Cracking over the support occurred at a moment of 58.5 k-in. (6.6 kN-m). Cracking occurred under the loading closest to the cold-formed angle detail at a moment of 231 k-in. (26.1 kN-m). After ultimate load was reached, cracking over the shear studs occurred and a subsequent drop in load occurred. An ultimate moment of 444 k-in. (50.2 kN-m) was reached. A post test inspection revealed that in general the shear studs and deck were still attached to the support members.

4.2.6 SDI-4-2

Compressive strength of the concrete was 4600 psi (31.8 MPa).

Two unloadings cycles were performed during this test, as can be seen in Figure 8. The first was at a moment of 200 k-in. (22.6 kN-m) to obtain a cracked stiffness and the second after a peak moment of 234 k-in. (26.4 kN-m).

Immediately upon loading an end slip was recorded. Cracking over the support occurred at a moment of 178.5 k-in. (20.2 kN-m). Note that this cracking occurred at a higher moment than in the previous tests. Cracking under the load points occurred at a moment of 244.5 k-in. (27.6 kN-m). One should also note that the maximum load that was reached was less than in previous tests that had similar configuration.

After the test, the concrete slab, deck, and end details were all closely examined in order to find an explanation for the results. Upon lifting the slab off of the cold-formed angle support end, it was discovered that the welding had been ineffective. Approximately 50% of the puddle welds did not fuse properly with the base metal.

5.0 STRENGTH AND STIFFNESS FORMULATIONS

Comparisons between the test results and predicted strengths are made. Two limit states are used to predict the strength. These are ultimate strength based on reinforced concrete theory and first yield of the extreme fiber of the deck. The ultimate strength moment has the slab dead load removed. In calculating the first yield moment, the strain induced in the bottom fibers of the deck due to the concrete placement was considered. Values for both limit states are shown on the applied moment vs. deflection plots (Figures 3-8). In each case the strength values are determined by considering the slab to be simply supported.

Two elastic stiffness lines are shown on each plot. The stiffer of the two lines is based on the uncracked moment of inertia and the other is based on an average of the cracked and uncracked moments of inertia. In all cases the calculations are based on simply supported boundary conditions, with the section transformed to an equivalent concrete member. The ASCE Standard (*Specifications* 1984) suggests using the average moment of inertia for calculating deflections.

6.0 KEY OBSERVATIONS

In each test, except for the SDI-4 series, the experimental capacity exceeds the predicted strength corresponding to first yield, but does not reach the predicted ultimate strength. This behavior is generally indicative of partial composite action. In SDI 4-1 the ultimate strength moment is nominally obtained. In SDI 4-2 the yield moment is surpassed, but due to faulty welds, not to the degree that was observed in other tests that had a similar configuration.

The details of the end of the span clearly influence the behavior of a particular specimen. In terms of efficiency, the case that exhibits the best behavior is that in which shear studs and a cold-formed angle with a return lip are used.

Comparisons between the test results and the calculated stiffness values show, that in the range of loading which extends up to approximately 70% of the yield moment, the slabs are stiffer than would be predicted with the simple model. Therefore, serviceability checks could be made with acceptable accuracy using the simple approach employed herein.

7.0 FUTURE RESEARCH

The research reported herein is ongoing. Additional tests are planned, which will consider design variables not addressed in the completed tests. Further analytical studies are to be conducted, which will include the assessment of the end anchorage forces that are required to develop the yield moment in the composite slab.

8.0 SUMMARY AND CONCLUSIONS

The results of six full scale composite slab tests are reported. Comparisons between the test results and predicted strengths based on conventional reinforced concrete theory are made. Results from elastic stiffness calculations are shown, which are based on an uncracked moment of inertia and on an average of the uncracked plus the cracked moments of inertia.

For the tests considered herein, the range of test variables is not all inclusive. However, based on the data obtained from the research project to date, one may conclude that a predictive strength for composite slabs that is based upon the onset of yielding in the extreme tension fiber appears to be reasonable. In order for the yield moment to be used, proper attention to the details at the end of the slab is essential. Further, deflection predictions based on an average moment of inertia are reasonable within the elastic range of behavior. One should recall that all of the calculations are based on simply supported boundary conditions, as is typical in design when no negative moment reinforcement is provided.

This study is significant in that the approach to determining the strength of composite slabs used represents a departure from past work, which has focused on the limit state of shear bond. The shear bond mode of behavior is only a consideration at slab locations where there is a free end. Results of this study indicate that proper detailing at the slab ends can effectively prevent the end slip associated with shear bond. Upon refinement, this should permit a design limit state, such as yielding of the extreme tension fibers of the deck, that would be independent of laboratory testing.

9.0 ACKNOWLEDGEMENTS

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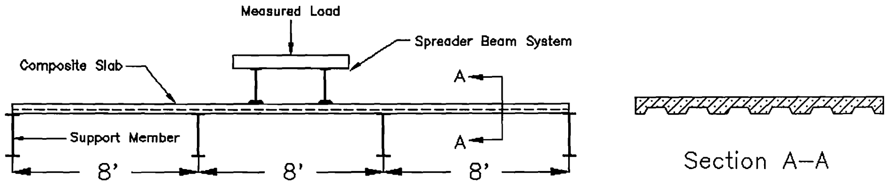

Figure 1. Schematic of Test Setup (center span test)

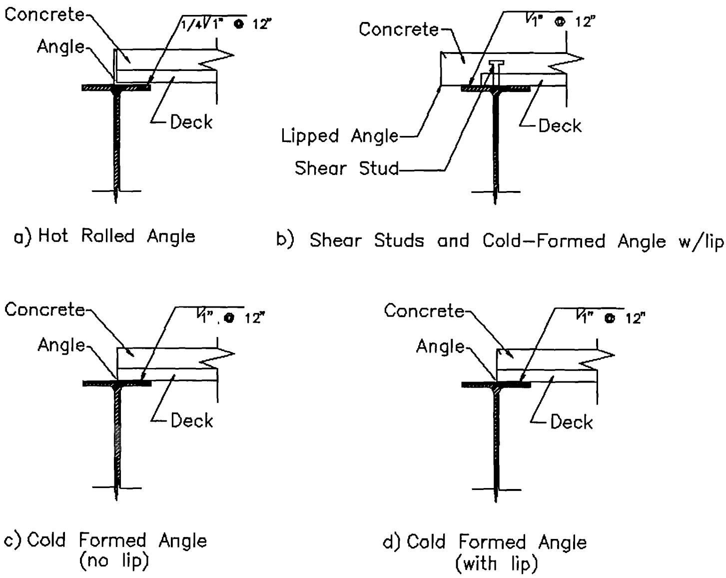

Figure 2. End Span Edge Details

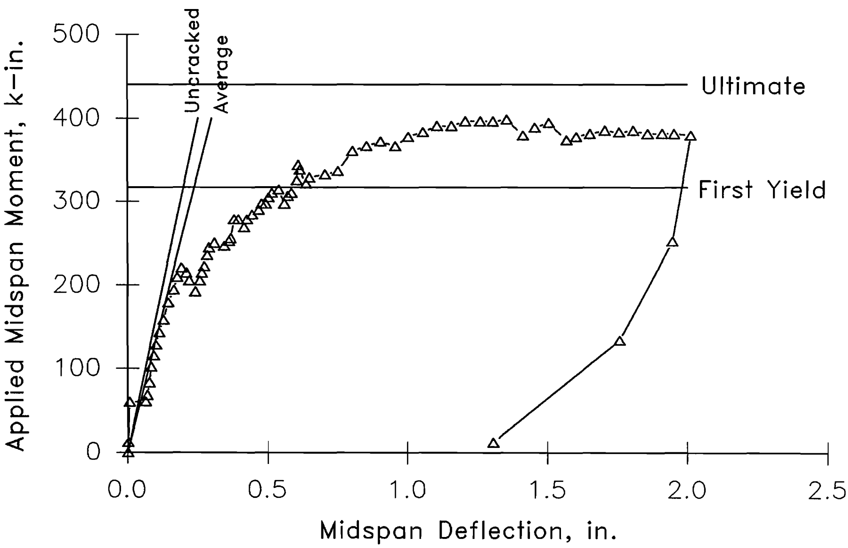

Figure 3. Applied Midspan Moment vs. Midspan Deflection : SDI-1-1

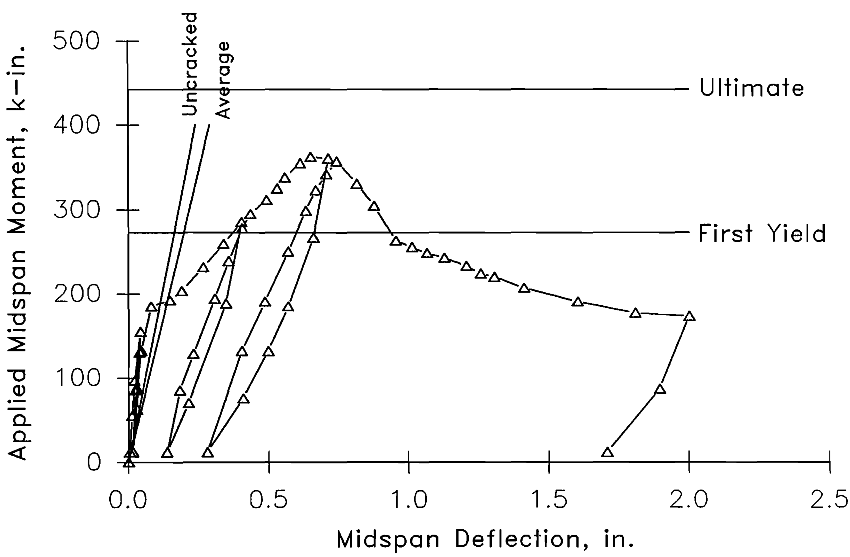

Figure 4. Applied Midspan Moment vs. Midspan Deflection : SDI-2-1

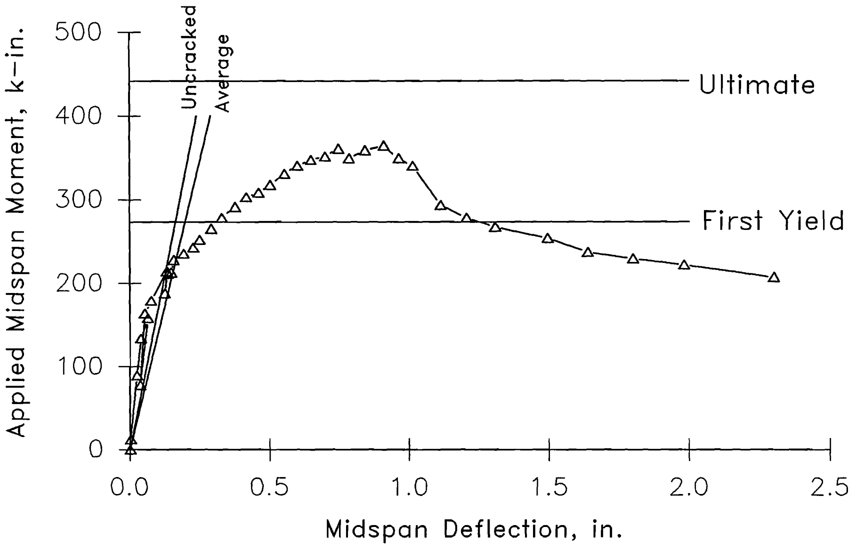

Figure 5. Applied Midspan Moment vs. Midspan Deflection : SDI-2-2

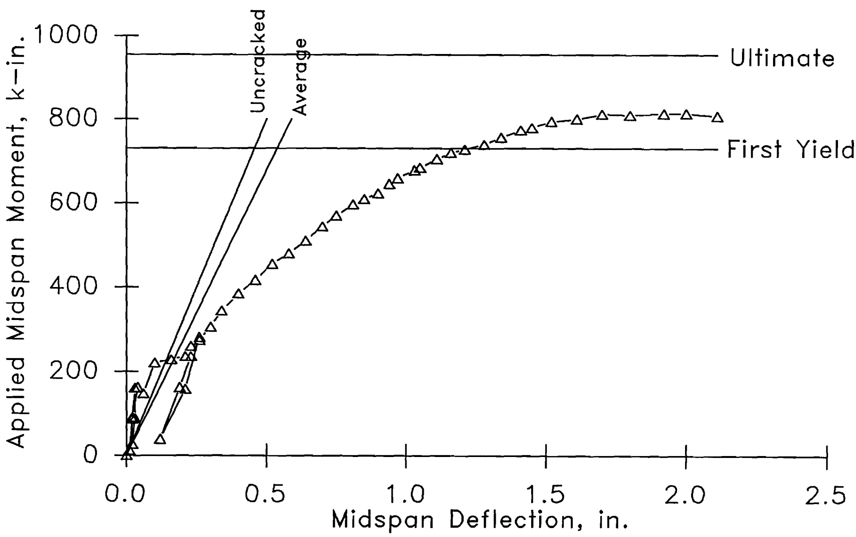

Figure 6. Applied Midspan Moment vs. Midspan Deflection : SDI-3-1

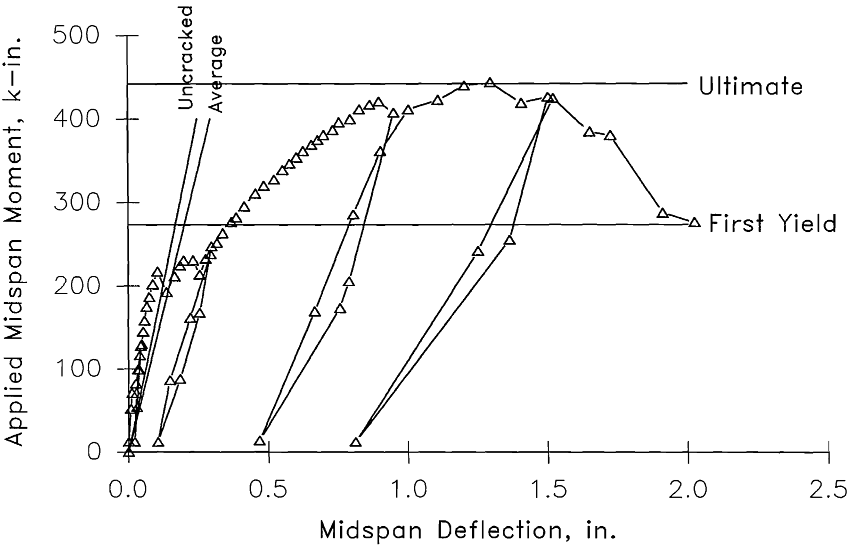

Figure 7. Applied Midspan Moment vs. Midspan Deflection : SDI-4-1

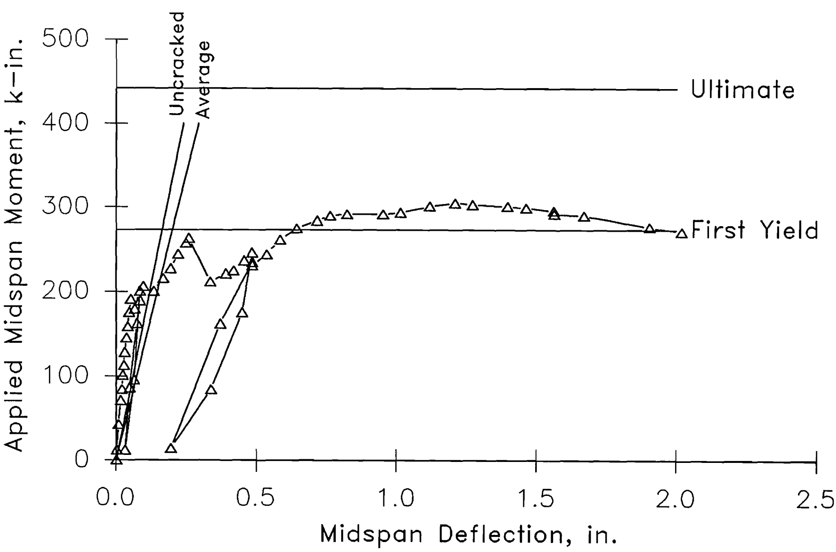

Figure 8. Applied Midspan Moment vs. Midspan Deflection : SDI-4-2