Composite Action of Concrete Slab and Open Web Joist (Without the Use of Shear Connectors)

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THE ADVANTAGE OF composite action of concrete slab and steel beam in increasing load carrying capacity and structural stiffness has been well recognized. The American Institute of Steel Construction in its 1963 Specification[1] has given the explicit methods of analysis. However, it restricts the composite action to only fully encased beams or beams with properly designed shear connectors. In the case of concrete slab and steel open web joist, limited study of composite action has been made.

Edward S. Klausner, of the K-System, Inc., of New York City, has devised a floor system, identified in this paper as "K-composite",* as shown in Fig. 1 (Specimens 2 and 4), where three special features are provided:

1. A dovetail trough is provided at the top flange of the joist to receive the concrete.
2. At the ends of the joists, the slab is haunched down for a distance to encase the entire top flange.
3. The end seats of the joists are welded to the supporting girders.

The first feature is to restrain the concrete from separating from the steel. The dovetail is a wedge shape and is reinforced by properly spaced wire chairs.

The second feature provides a strong end anchorage at the point where horizontal shear is largest.

The third feature enables the joist seats to act as shear connectors for the supporting girder. Together with the haunched down slab, this provides composite action of the steel girder and concrete slab.

The experimental and analytical work of this paper have a two-fold purpose:

1. To compare the stresses and deflections under incremental load of the following three systems: Joist alone, joist with concrete slab poured over corrugated sheet steel form, and the K-composite system.
2. To present a rational design procedure of the K-composite system.

TEST PROGRAM

Four specimens were erected at the Graduate Center of Polytechnic Institute of Brooklyn. Each specimen consisted of two joists—Republic Steel Corporation 10J2 modified. Exact chord sizes varied slightly and are recorded in Fig. 1 together with all the essential details. All joists had ½-in. dia. interior web members and 9/16-in. dia. end web members. A steel frame was erected to support the specimens as shown in Figs. 2 and 3.

Specimen 1 was for the purpose of testing the performance of the joists alone. The bottom chord was heavier than the top chord and the top chord had a dovetailed trough.

Specimen 2 used the joists of Specimen 1, except that a 2½-in. thick concrete slab, constructed according to the K-composite system, was added to test for composite action.

Specimen 3 had a 2½-in. thick slab poured on a corrugated steel form, which in turn was plug-welded to the top chord (using standard washers) of the joist.

Specimen 4 was identical to Specimen 2, except that chord sizes were slightly different and the metal wire chairs in the trough of the top chord were eliminated.

The locations for instrumentation are shown in Fig. 4. At mid-span and quarter-points of the top and bottom chords of all the joists and at the bottom of the concrete slabs, strain gages (Baldwin-Lima, SR-4) were attached.

Deflection gages (Federal, 0.001 increments) were also attached at the mid-span and quarter-points of the bottom chords.

Loading was provided by pouring water into a wooden tank built with a detached bottom (see Fig. 2),

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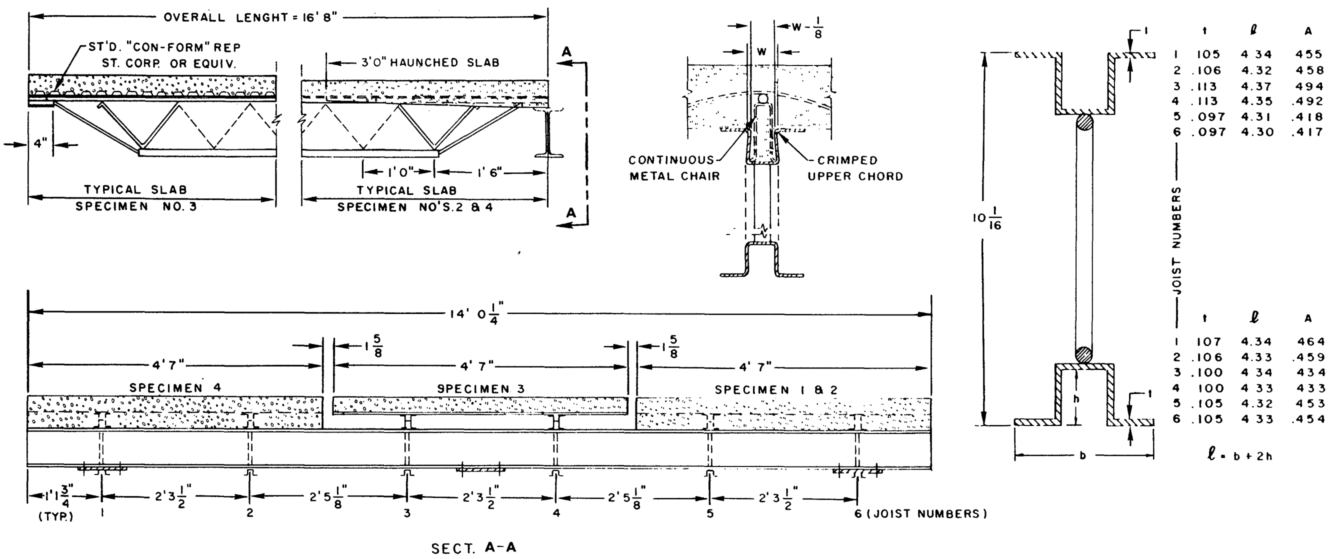
* Patent applications have been made for features of this system.



SPECIMEN NO.	3 .	4	2
GENERAL CONSTRUCTION	PERMANENT CORRUGATED SHEET CONCRETE SLAB ON OPEN WEB JOISTS	REMOVABLE PLYWOOD FORM CONCRETE SLAB ON OPEN WEB JOISTS - TYPICAL OF K-SYSTEMS INC. 101 PARK AVE NEW YORK 17, N Y	
JOISTS	STANDARD 10J2 REF REPUBLIC STEEL CORP	10J2 WITH UPPER AND LOWER CHORD MEMBERS REVERSED REF. DETAIL AT RIGHT FOR SPEC. NO 2	
BRIDGING	1/2" RODS 1/3 SPAN - BOTH UPPER & LOWER	NONE	
REINFORCEMENT	STANDARD 4x4 MESH WITHOUT CHAIRS		ST'D 4 4 MESH WITH CHAIRS

NOTES:

SPEC NO 2
1. MESH TO REST ON CHAIRS
2. 3/4" MIN COVER AT BOTTOM

SPEC NO 3
1. CORRUGATED SHEETS SHALL BE WELDED TO JOISTS USING WELD WASHERS - 25 WELDS FOR EACH 100 SQ FT OF SLAB AS MIN
2. FOLLOW MANUFACTURING SPECIFICATION FOR JOINTS
3. MESH TO BE ARRANGED BY HAND DURING POURING
4. 3/4" MIN COVER THROUGHOUT

SPEC NO 4 SAME AS SPECIMEN NO 3 WITHOUT CHAIRS

Fig. 1. Description of test specimens

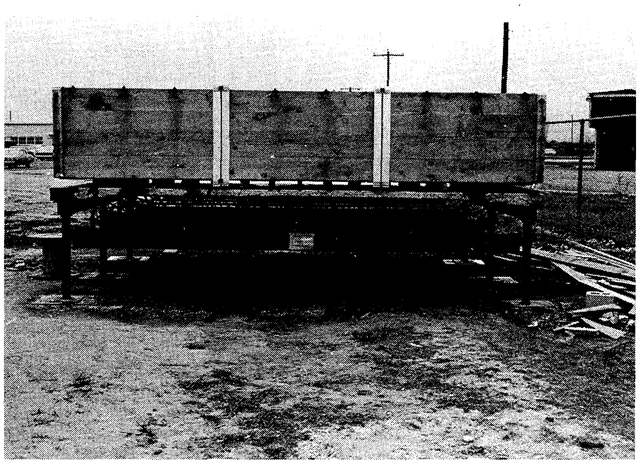


Fig. 2. Test set-up

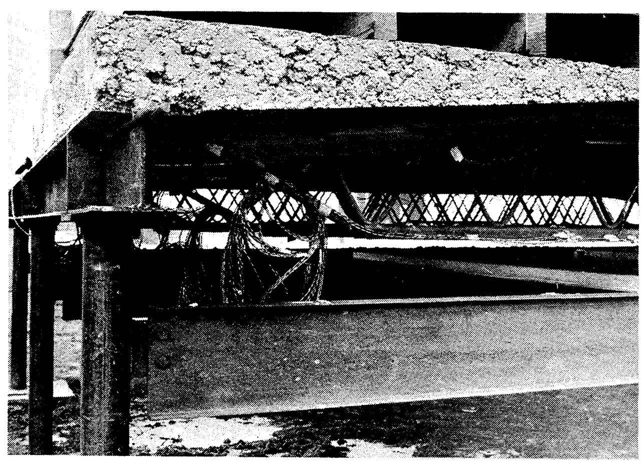


Fig. 3. View of haunched slab at end of joist (K-composite)

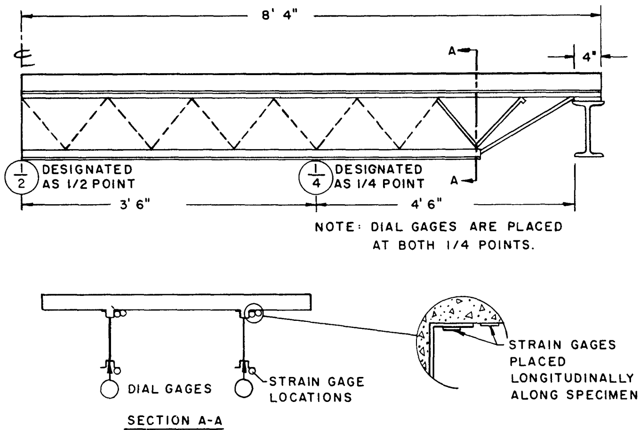


Fig. 4. Locations of instrumentation

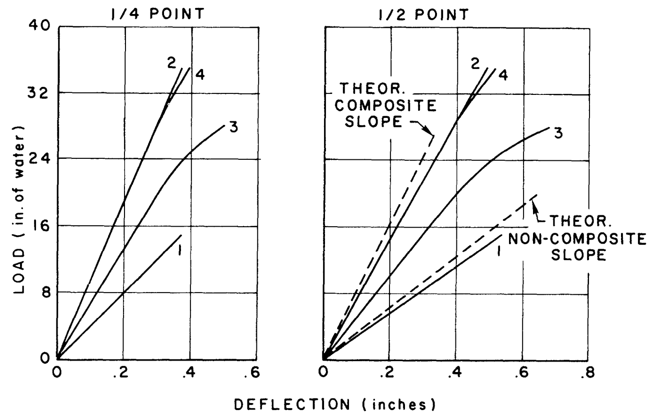


Fig. 5. Joist deflections, ¼ point and ½ point

and lined with plastic lining which was oversized to allow water to fill all the corners. Although the tank bottom was nailed to the wall at the ends (not at the sides), the effect on uniform load distribution is considered negligible. The outside dimensions of the tank were 4′-8′′ x 16′-4′′ x 3′-0′′ high and the tank wall consisted of 2′′ x 10′′ Douglas fir planks.

As water was poured into the tank (at the rate of about 1 in. height of water per five minutes) both strain gages and deflection gages were read at the interval of 3 in. of water. These readings are plotted as the load-deflection and load-strain curves shown in Figs. 5 through 9. The numbers assigned to the curves correspond to the specimen numbers. The load scale presented for the curves is in inches of water. Each inch of water corresponds to 5.2 psf of uniformly distributed load.

For clarity of presentation, the average readings at mid-span and quarter-points of the two joists are used in plotting the curves.

High-early strength concrete had been ordered. However, the three concrete cylinders tested after seven days show a strength of 1400 psi. Fortunately, the concrete stresses during these tests were extremely low, hence the poor quality of concrete was acceptable.

It should be noted that the test load is in addition to the weight of the concrete slab and the loading tank.

Upon completion of the test on Specimen 2 a sustained load of 156 psf (30 in. of water) was applied for a period of 48 hours. No appreciable increase of mid-span deflection was observed.

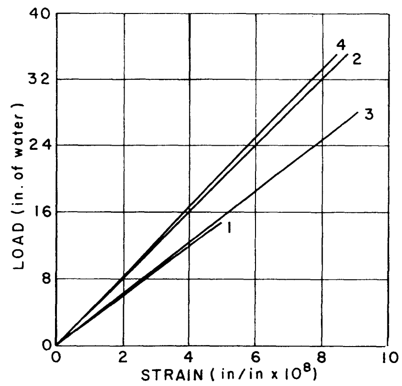


Fig. 6. Lower chords, ½ point (all tension)

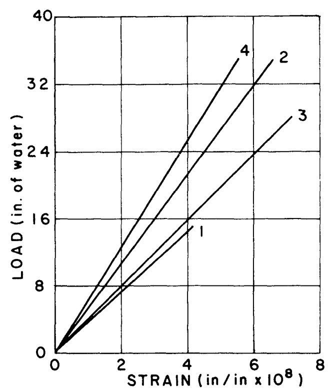


Fig. 7. Lower chords, ¼ point (all tension)

ANALYTICAL SOLUTIONS

Beam Solution (Fig. 10)—The beam solution is based on the following assumptions:

1. The joist and slab work together as a composite beam. (Assume $n = E_s/E_c = 10$.)
2. The compression face of the beam consists of the upper chord of the joist and the concrete slab depending on the position of the neutral axis.
3. Tension is carried by the lower chord of the joist.
4. The web does not contribute bending strength.

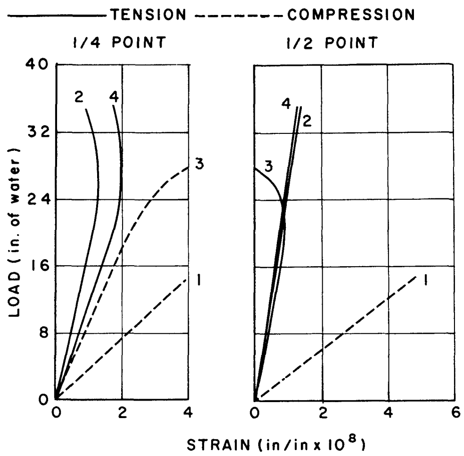


Fig. 8. Upper chords, ¼ point and ½ point

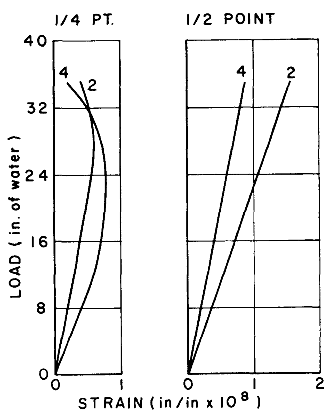


Fig. 9. Underside of concrete slab (all tension)

Neutral axis:

	A	y (from center of joist)	Ay
Top chord	0.455	4.544	2.07
Bottom chord	0.458	4.544	− 2.08
Slab $(2.5 \times 24)/n$	6.000	6.281	37.68
	6.913		37.67

$$\therefore \bar{y} = \frac{37.67}{6.913} = 5.45 \text{ in. above center of joist}$$

Moment of inertia:

$$0.074 + 0.455 \times 0.906^2 = 0.45$$
$$0.074 + 0.458 \times 9.994^2 = 45.72$$
$$\frac{2.08^3 \times 24}{10} \times 1/3 = 7.22$$
$$53.39$$

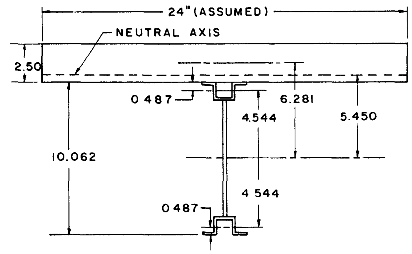


Figure 10

Maximum stress for 100 psf (lower chord):

$$f_{\max} = \frac{M_{\max} C}{I} = \frac{2.292 \times 100 \times 16^2 + 10.481 \times 12}{8 \times 53.39}$$
$$= 17{,}320 \text{ psi (tension)}$$

The top chord will be subjected to a small tensile force.

Consider joists alone:

$$\bar{y} = \frac{-0.01}{0.913} = -0.011$$

$$\therefore I = 0.455 \times 4.555^2 + 0.458 \times 4.533^2 + 0.148$$
$$= 9.53 + 9.39 + 0.148 = 19.068 \text{ in.}^4$$

$\therefore f_{\max}$ in bottom chord

$$= \frac{2.292 \times 100 \times 16^2 \times 5.02 \times 12}{8 \times 19.068}$$
$$= 22{,}500 \text{ psi (tension)}$$

It should be noted that the upper chord would be critical for this solution.

$$f_{\max} \text{ in top chord} = 22{,}500 \times \frac{5.042}{5.020}$$
$$= 22{,}600 \text{ psi (compression)}$$

Considering a factor of safety of 1.65 and a yield stress of 36,000 psi (basis for allowable loads recommended by the manufacturer), the allowable load for the joists alone would be as follows:

$$\text{allowable stress} = \frac{36{,}000}{1.65} = 21{,}800 \text{ psi}$$

$$\therefore \text{allowable load} = \frac{21{,}800}{22{,}600} \times 100 = 96.5 \text{ lbs/ft}^2$$

for the condition that the top chord is laterally braced.

Bond Stress:

With the same factors applied, the allowable load for the composite beam would be as follows:

$$\text{allowable load} = \frac{21{,}800}{17{,}320} \times 100 = 126.0 \text{ lbs/ft}^2$$

which is 30.5 percent above the noncomposite joist.

The maximum bond stress under the allowable load between the slab and the joist for the composite beam is

$$u = \frac{VQ}{Ib}$$

Where V is the end shear, Q is the static moment of the transformed area of the slab about the neutral axis, I is the moment of inertia, and b is the width of the contact surface.

By substituting the numerical values:

$$u = \frac{126.0 \times 2.292 \times 8 \times (2.08 \times 24) \times 1.04}{53.39 \times 4.34 \times 10} = 51.8 \text{ psi}$$

This bond stress is considered tolerable for composite construction without mechanical connections.[2]

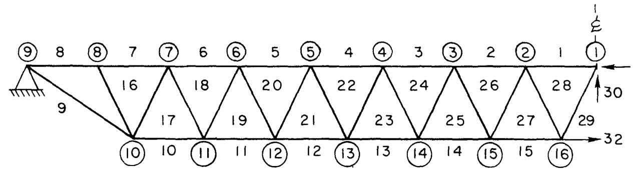


Figure 11

Truss Solution (Fig. 11)—Two truss solutions were computed—one considering the joists alone, the other considering the upper chord members to consist of the upper chord of the joist and the slab.

The analysis of the trussed sections was accomplished on a previously established computer program,[3] using the average area values for the joist chord members.

Since the truss is symmetrical, only one half is considered. Figure 10 indicates the joint and member numbering system.

The necessary data for each analysis are listed in Tables 1 and 2 with their solutions. The uniformly distributed load of 100 psf is considered to act at the upper chord panel points as concentrated loads.

Table 1. Member Forces (Determined by Truss Solution)

Input Data					Computer Output	
Member	Near Joint	Far Joint	Area (in.^{2})	Area (in.^{2})	Joists Only Force (kips)	Composite Force (kips)
1	1	2	0.4175	6.4175	−9.46	−7.90
2	2	3	0.4175	6 4175	−9.16	−7.65
3	3	4	0.4175	6.4175	−8.56	−7.16
4	4	5	0.4175	6.4175	−7.67	−6.41
5	5	6	0.4175	6.4175	−6.48	−5.41
6	6	7	0.4175	6 4175	−4 99	−4.17
7	7	8	0.4175	6.4175	−3.20	−2.68
8	8	9	0.4175	6.4175	−3.35	−2.80
9	9	10	0.2485	0.2485	3.75	3.27
10	10	11	0.4535	0.4535	4.17	3.48
11	11	12	0.4535	0.4535	5.81	4.85
12	12	13	0.4535	0.4535	7.51	5.97
13	13	14	0.4535	0.4535	8.19	6.84
14	14	15	0.4535	0.4535	8.94	7.47
15	15	16	0.4535	0.4535	9.38	7.84
16	8	10	0.1963	0.1963	−0.27	−0.26
17	7	10	0.1963	0.1963	−1.76	−1.68
18	7	11	0.1963	0.1963	1.49	1.42
19	6	11	0.1963	0.1963	−1.49	−1.42
20	6	12	0.1963	0.1963	1.22	1.16
21	5	12	0.1963	0.1963	−1.22	1.16
22	5	13	0.1963	0.1963	0.95	0.90
23	4	13	0.1963	0.1963	−0.95	−0.90
24	4	14	0.1963	0.1963	0.68	0.64
25	3	14	0.1963	0.1963	−0.68	−0.64
26	3	15	0.1963	0.1963	0.41	0.39
27	2	15	0.1963	0.1963	−0.41	−0.39
28	2	16	0.1963	0.1963	0.14	0.13
29	1	16	0.1963	0.1963	−0.14	−0.13
30					1.69	1.69
31					9.53	7.96
32					9.53	7.96

Table 2. Deflections (Determined by Truss Solution)

Input Data			Computer Output			
			Joists Only		Composite	
Joint	x (in.)	y (in.)	Hor. Disp. (in.)	Vert. Disp. (in.)	Hor. Disp. (in.)	Vert. Disp. (in.)
1	0	0	0	−0.623	0	−0.233
2	−12	0	0.906	−0.616	0.493	−0.232
3	−24	0	0.178	−0.586	0.970	−0.222
4	−36	0	0.260	−0.532	0.142	−0.203
5	−48	0	0.334	−0.458	0.182	−0.175
6	−60	0	0.396	−0.365	0.215	−0.140
7	−72	0	0.444	−0.258	0.241	−0.987
8	−84	0	0.475	−0.143	0.258	−0.570
9	−96	0	0.507	0	0.275	0
10	−78	−9.096	0.385	−0.199	−0.322	−0.756
11	−66	−9.096	0.348	−0.313	−0.291	−0.119
12	−54	−9.096	0.297	−0.414	−0.248	−0.157
13	−42	−9.096	0.234	−0.498	−0.195	−0.189
14	−30	−9.096	0.162	−0.562	−0.130	−0.212
15	−18	−9.096	0.828	−0.604	−0.692	−0.227
16	−6	−9.096	0	−0.622	0	−0.233

DISCUSSION OF RESULTS

From the first analytical solution, it is seen that the neutral axis of the composite joist is located within the concrete slab. Hence, if the section were to act compositely, the upper flange of the joists would be in tension. As shown in the curves of Fig. 8, the upper chords of the joists in Specimens 2 and 4 show tension throughout the load range.

At mid-span of these specimens the curves continue linearly as would be expected. The quarter-point strains show a tendency toward compression, indicating a shift of the neutral axis. Yielding is ruled out due to the low stresses throughout the structure. Another possible cause of a redistribution of this nature would be a loss of bond between the two materials, causing a loss of composite action. However, this possibility can be eliminated since the deflections remain linear. The deflections would show a considerable change in slope if this were the case.

The concrete strains adjacent to these locations show similar effects. Curves illustrating the similarity of strains in the upper flange of the joists and the adjacent concrete in Specimens 2 and 4 are shown in Figs. 8 and 9.

Specimen 3 (corrugated sheet construction) acts completely differently. The upper flange of the joist at the quarter-point shows compression immediately and picks up sharply after the plug welds begin to fail, as shown in Fig. 8. A tendency toward composite action is shown at the mid-span upper flange on Specimen 3, but it fails to persist. This indicates that the plug welds serve as shear connectors, producing composite action at low levels in comparison to that of the K-composite specimens.

From the analytical solution as a truss (comparable to the beam solution as indicated by maximum stresses), it is seen that the ratio of maximum stress and deflection between the composite and the noncomposite section would be 83.5 percent ($7.96 \times 100/9.53$) and 37.5 percent ($0.233 \times 100/0.622$) respectively. Therefore, deflection would be a good indication of how the section was functioning.

The deflection of a noncomposite section is established from the data generated by Specimen 1 (joists w/o concrete). Since this specimen consists of the joists used for Specimen 2 (K-composite), comparison of Specimens 1 and 2 would develop evidence of the composite capabilities of the K-composite construction. Comparing the maximum deflection (mid-span) as a ratio of Specimen 2 to Specimen 1, a ratio of 39.8 percent exists. This shows excellent agreement with the theoretical ratio of 37.5 percent if the system were composite, indicating composite action does exist.

Although the theoretical solutions do not apply directly to Specimen 3 (corrugated sheet construction), similar comparisons of deflections can be made to establish the composite capabilities of this construction. The mid-span deflections in Fig. 5 are curved and indicate loss of composite action at the lower load.

A summary table of test results is shown in Table 3.

CONCLUSIONS

As demonstrated in the Discussion of Results, composite action exists in the K-composite system of reinforced concrete slab and open web steel joist construction. This system acts compositely and therefore reduces

Table 3. Summary of Test Results & Comparison with Theoretical Analysis

Speci-men No.	Description	Maximum Test Load[a] (psf)	Maximum Stress Bottom Chord (psi)	Maximum Deflection Mid-Span (in.)	Maximum Equivalent Stress @ 100 psf	Maximum Equivalent Deflection @ 100 psf	Theoretical Solution @ 100 psf (Truss) Maximum Stress	Maximum Deflection
1	Joists w/o concrete	78.0	14,360	0.533	18,410	0.683	21,010	0.622
2	K-composite System	176.5	25,380	0.480	14,380	0.272	17,550	0.233
3	Corrugated sheet construction	145.5	26,390	0.667	18,140	0.458	...	...
4	K-composite system w/o chairs	176.5	24,500	0.508	13,880	0.288	...	...

	Stress		Deflection	
	Theoretical	Actual	Theoretical	Actual
Effects of composite action	$\frac{17,550}{21,010} = 83.5$	$\frac{14,380}{18,410} = 78.1$	$\frac{0.233}{0.622} = 37.5$	$\frac{272}{683} = 39.8$

[a] *Excluding loading fixture—approximately 7 psf.*

deflections and member stresses appreciably. Comparison of data illustrates that mid-span deflections were reduced to 39.8 percent and the maximum lower chord stresses were reduced to 78.1 percent of a comparable noncomposite structure.

Based on the results of these tests, corrugated sheet construction exhibits some composite action up to a lower load level, depending on the strength of the plug welds between the welding washers and the top flange of the joist.

Bond between the concrete slab and the trough shaped top chord of the joists in the K-composite system appears reliable under the test load. The encasement of the top chord at the end of the joist enhances the bond where the horizontal shear is greatest. A conservative estimate of the average bond stress can be computed as indicated below.

The stress in the bottom chord under the highest applied load is 25,380 psi, which corresponds to a force of 25,380 $\times$ 0.4535 or 11,510 lbs. Disregarding the small tensile force in the top chord, an average bond can be obtained by dividing this force by the contact area between the concrete and the top chord from the end of the joist to the center of the span, or

$$\mu \text{ avg.} = \frac{11.510}{8.33 \times 12 \times 4.33} = 26.6 \text{ psi}$$

If the bond stress distribution is assumed to vary triangularly in the same manner as the shear, then the maximum bond at the end will be 53.2 psi, which is still tolerable since the flange is fully encased in the concrete.

In addition to the above discussion, a composite action exists between the supporting steel girder at the ends of the joist and the haunched slab. The design of the supporting girder as a composite beam presents no departure from the regular composite beam construction. The shear value of each seat as a shear connector has been verified by a load test conducted by the Manufacturing Division of the Republic Steel Corporation at Youngstown, Ohio.[4] The ultimate value approaches 65 kips. A safe value based on the strength of the weld on the seat may be used.

The analysis of the K-composite system construction may be made either based on the beam solution or on the truss solution, since the results are essentially the same. As shown in the computations, the maximum chord stresses are 17,320 psi and 17,550 psi, respectively.

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