A Composite Girder System for Joist Supported Slabs

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In current practice composite action between the concrete slab and the girder is not taken advantage of when framing a floor using standard open web joists and continuous metal decking. The reason is the girder is depressed (usually 2½ in.) to receive the joist seat and therefore is not in direct contact with the concrete slab. Since the girder supporting the joists is noncomposite, the economic and other advantages of composite design are not realized. Also, because the girder is not framed flush with the joists, its bottom flange is usually a critical element in setting floor-to-floor heights and clearances for duct work.

This paper describes a novel composite girder system to be used in conjunction with joist supported slab systems. As shown in Fig. 1, at the heart of the system are connections, such as tees, which are welded to the top flange of the girder intermittently between the joist locations. (These pieces are analogous to stubs in a stub-girder system). The tops of these connections are flush with the bottom of the slab and thus provide a surface to which standard shear connectors can be attached. The former gap between the bottom of deck and top of girder can now be utilized structurally.

LOAD TEST OF THE GIRDER SYSTEM

A load test was conducted at the facilities of The Berlin Steel Construction Company, Berlin, Conn. A 20-ft full-scale girder assembly was constructed to simulate, as closely as practical, actual field conditions (Fig. 2). The steel beam was a W14x22 section connected to the columns with Type 2 double angle connections. The stub connectors were pieces of WT2.5x8. The typical tee was 12 in. long and the two end connectors were 18 in. long. The tees were welded to the top flange of the girder using several stitch weld patterns, with the end connectors receiving the most weld and the connector at midspan having the least. A36 steel was used. The shear connectors were standard ¾-in. dia., headed studs manually welded to the tees with ¼-in. fillet welds.

The slab, with a total thickness of 3 ½ in., was placed

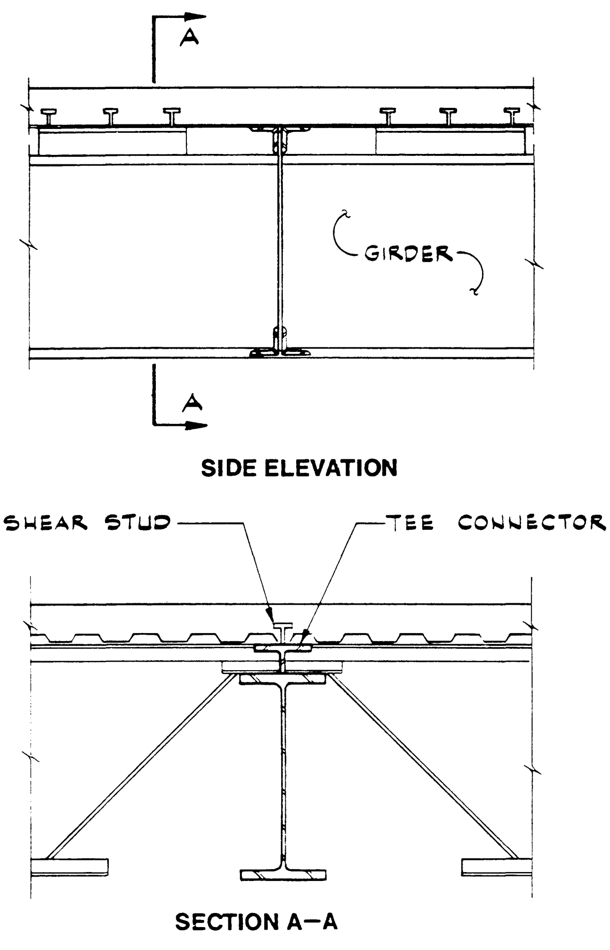

Fig. 1. Composite girder system

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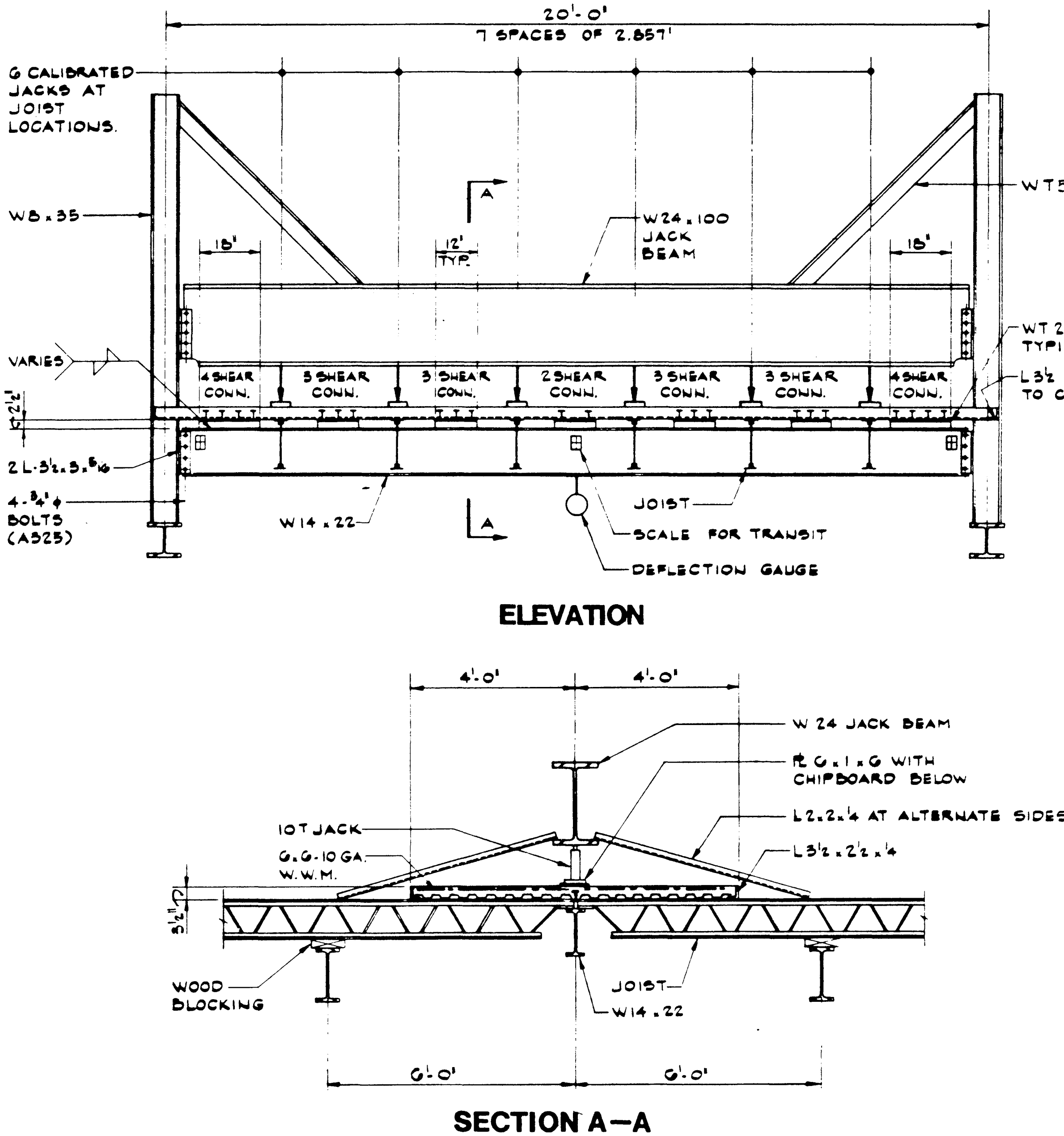


Fig. 2. Load test assembly

on 28-ga. uncoated metal deck with 9/16-in. high corrugations. Slab was minimally reinforced with 6x6 10/10 (W1.4) welded wire fabric. Normal weight concrete was used and, at the time of the test, had a tested compressive strength of 3,700 psi. The slab width was 8 ft due to test limitations. This was considerably wider than the effective width required in composite design. The W14 was unshored when the concrete was placed.

Joists were supported on wood blocks at the respective far ends to prevent any restraint. They were welded to the top flange of the W14, but were not connected to each other to assure a simple span condition. Pieces of 2½ in. tube were placed adjacent to each joist end to prevent any possibility of joist ends crushing during the overloading phase.

The load was applied by a series of 10-ton hydraulic jacks which were placed on top of the slab over the joist bearing points. The jacking system was calibrated to an accuracy of .5%. Deflections were recorded at midspan by a system of dial gages, and also independently with a transit. Ends of the W14 section were also monitored to record any movement or rotation. Since the test was conceived as a load test, it was not necessary to use strain gages on the test beam or slab, although the test beam

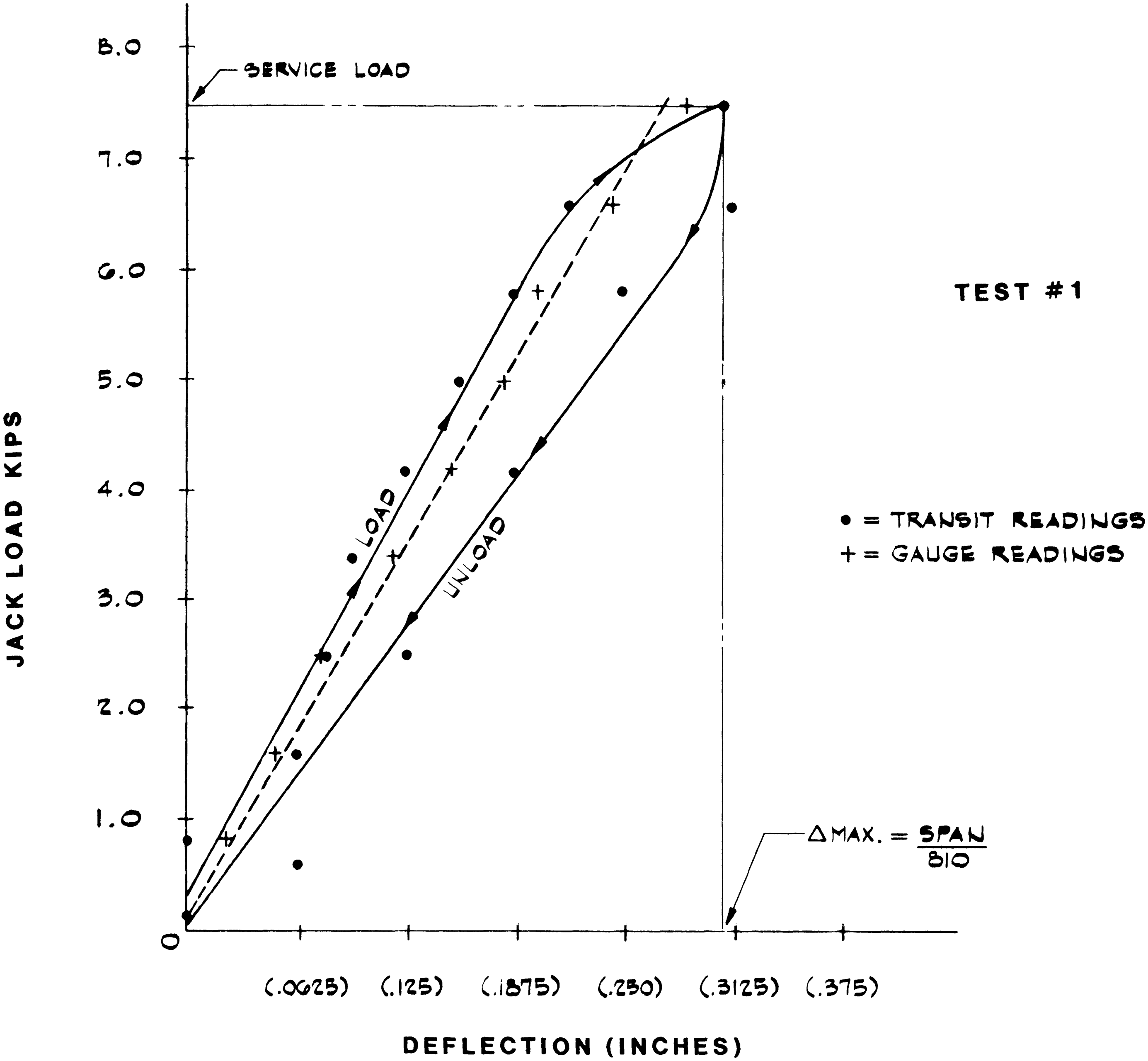


Fig. 3. Load-deflection curve, Test one

was whitewashed to aid in detecting any local yielding in the steel.

Two tests were run. In the first test, the load was applied in increments of 833 pounds, until the service load of 7.5 kips per load point was attained. (This load represented an equivalent design load adjusted to compensate for additional dead load of a continuous slab.) The load was held at this value for 30 min. and then decreased in increments to its original value.

The second test was used to load the beam to a maximum value of approximately twice the service load (unless sudden failure occurred before this goal was reached) and then reduced in increments similar to the first test. In the second test, the load increments took 40 min., and off-load increments took 15 minutes. To prevent the possibility of damage, dial gages were removed for the second test and deflections were recorded with only the transit.

DISCUSSION OF TEST RESULTS

The load deflection curve of the first test is shown in Fig. 3. At a working load of 7.5 kips per jack, the recorded deflection was 0.289 in., which corresponds to a span/depth ratio of 810. Ends of the beam were monitored and indicated the original elevations were maintained, except for a small amount of rotation in the end connection. When unloaded, the W14x22 returned to its original position, indicating the member behaved elastically. This agrees with the findings of related studies.[1,2,3]

In the second test, a load of 14.25 kips per jack was attained and, subsequently, released in increments. The load-deflection curve for this test is in Fig. 4. As the maximum value approached, diagonal cracks emanating from the corners of the columns were observed, and three edges of the slab curled upwards approximately 3⁄16 in. (Fig. 5), a result of the horizontal restraint at the columns. When unloaded, a residual deflection of 7⁄16 in. in the W14 was recorded, indicating the anticipated yielding. No separation between the slab and the tee connectors was noticed, indicating composite behavior was maintained throughout the test. Also, there were no other visible signs of local failure in the connectors. There was no cracking or flaking in the whitewash on the beam, indicating deformation was regular and not localized.

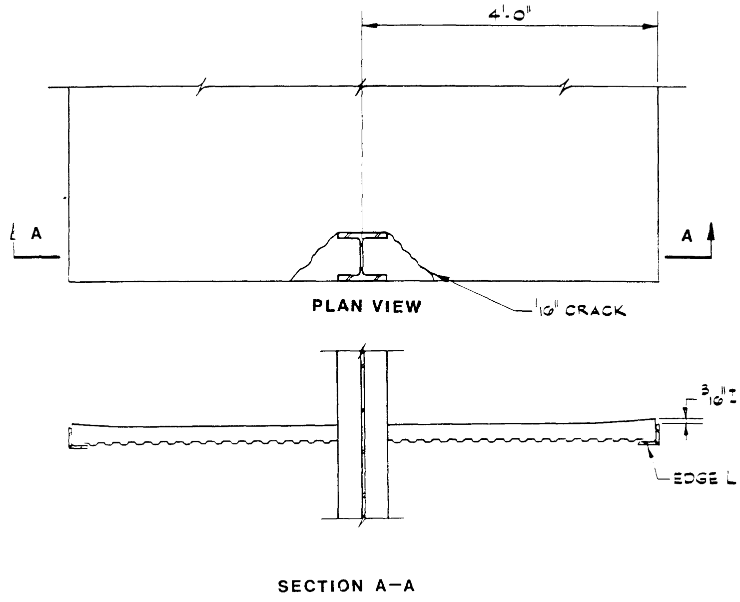


Fig. 5. Concrete failure, Test two

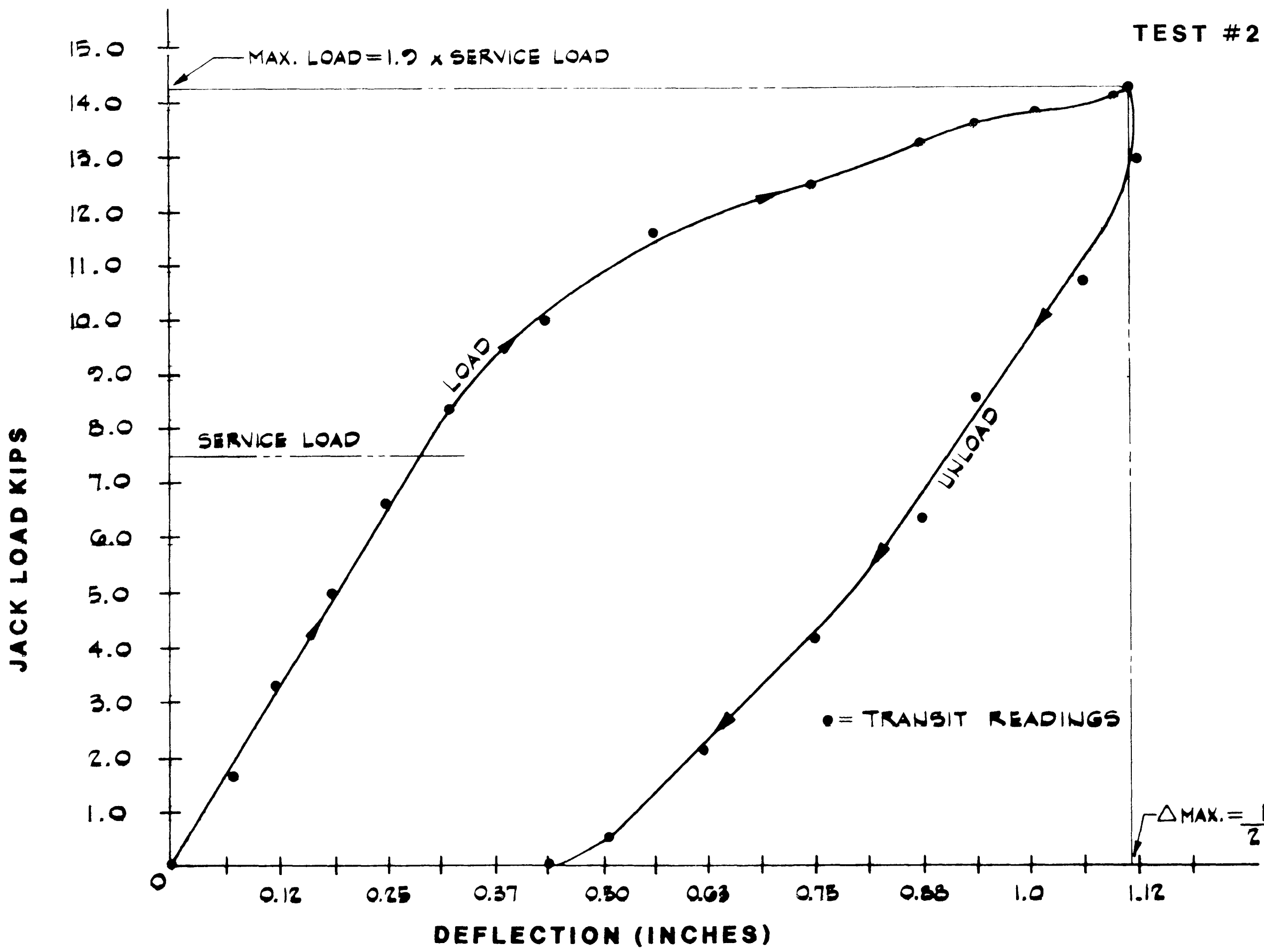


Fig. 4. Load-deflection curve, Test two

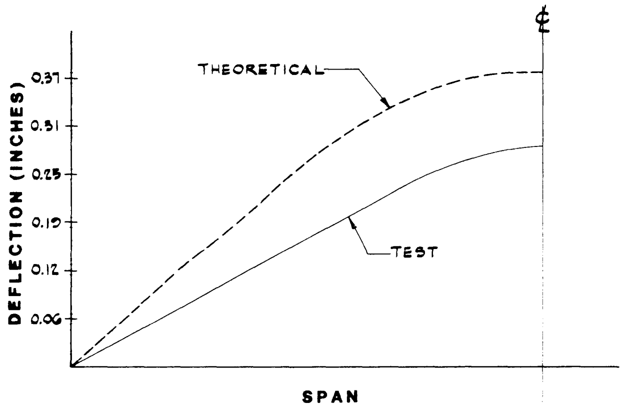


Fig. 6. Deflection comparison

A comparison of three 30-ft. girders is shown in Fig. 7. The composite system is shown in A, an equivalent non-composite substitute in B and a shallower non-composite girder used when duct work clearance is needed in C. In the example, cost of the installed beam was based on \$0.60/lb., the installed tee connectors at \$0.70/lb. and the shear connectors at \$1.50 ea. Deflections were based on a 50-psf live load. In addition to the lower cost of the composite girder system, a savings in other building components such as curtain wall can be realized.

Other beneficial features of the system are that it can be designed using unshored construction and can develop moment restraint by using the appropriate end connections. It is also flexible in that it does not require special fittings, techniques or training of workmen. The nature of the system is such that the work of applying connectors and studs can be divided in several combinations to suit various conditions of availability of material, equipment, labor and/or local regulations.

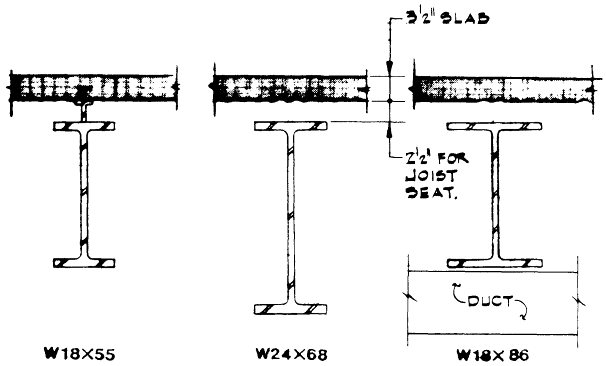
COMPARISON WITH THEORETICAL ANALYSIS

The girder was analyzed as a composite section using a Vierendeel approach, with the top chord being the transformed slab section and the connectors the rigid vertical members. Deflections recorded in the first test are compared to those obtained through analysis as in Fig. 6. The test values were stiffer than the theoretical values. The measured values were less than the analytical values, indicating the system behaved as a fully composite one. The higher (analytical) stress values at the ends of the slab were verified by cracks observed in these regions.

COMPARISON TO A CONVENTIONAL NON-COMPOSITE GIRDER

Several advantages of this girder system compared to a conventionally designed non-composite girder are:

1. A reduction in girder depth. In the 20- to 35-ft. range, typical reductions of 4 to 6 in. are possible, which result in a lower floor-to-floor height and, therefore, savings to the owner. Also, by coincidence, within this span range the bottom of the girder is approximately flush with the bottom of joists, which affords greater efficiency and flexibility for mechanical systems.
2. A savings in girder weight (in this test, the non-composite substitute for the W14x22 used would be a W18x35).
3. A stiffer system. The girder acts compositely with the slab and effects of common problems such as floor vibrations would be diminished.



	(A)	(B)	(C)
OVERALL DEPTH OF FLOOR STRUCTURE	24"	30"	24"
GIRDER WEIGHT	1650 LBS.	2040 LBS.	2580 LBS.
LIVE LOAD DEFLECTION	0.37 IN.	0.52 IN.	0.62 IN.
INSTALLED COST	40 STUDS - \$60 10 T CONNECTORS - \$84 1650 LBS. - \$990 \$1134	2040 LBS. = \$1224	2580 LBS. = \$1548
RELATIVE COST	1.00	1.08	1.36
RELATIVE DIFFERENCE IN FLOOR TO FLOOR HEIGHT	—	+6"	—

Fig. 7. Comparison of 30-ft girders

Photographs taken during erection of the test rig are shown in Figs. 8 to 10. Figure 8 shows the test beam with the connections before the metal deck was placed. Figure 9 shows the studs protruding through the deck before the concrete was poured and Fig. 10 shows the assembled rig.

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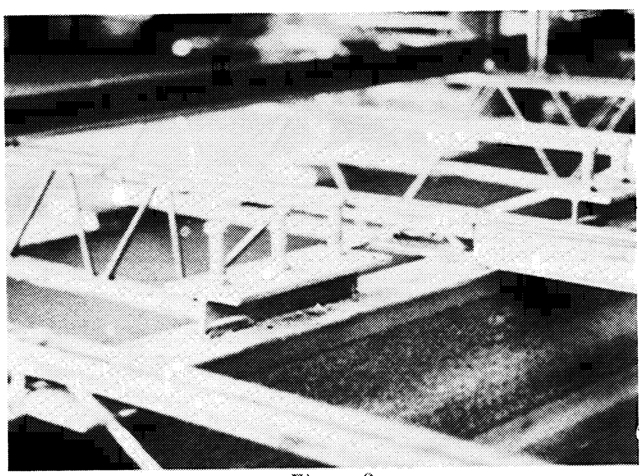

Figure 8

Figure 9

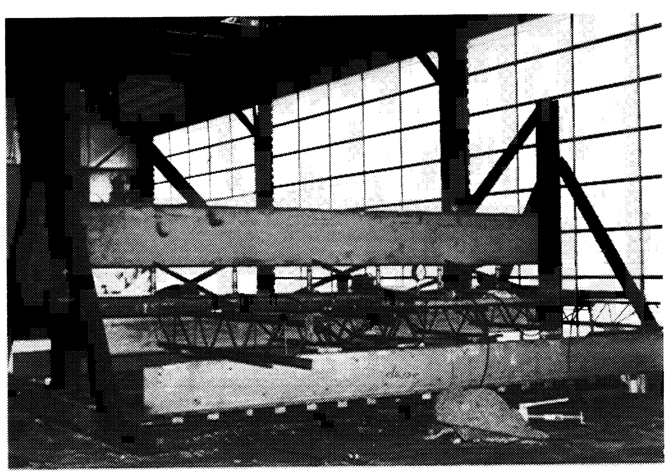

Figure 10