CONCRETE BRIEFS

Notes from Field and Office

Tests of T-Beams with Precast Webs and Cast-in-Place Flanges

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PRECAST REINFORCED CONCRETE BEAMS are used in building and bridge construction in conjunction with cast-in-place concrete slabs. Economic considerations indicate the desirability of composite action between the precast and the cast-in-place elements. Composite behavior, with the slab serving as the flange of T-beams, results in horizontal shearing stresses at the construction joints. Various methods of joint treatment have been specified to obtain adequate joint strength and thus insure that the components behave compositely. Doweling, use of shear keys, and roughening of the joint surface have been used in joint construction, but the information available regarding the strength of these joints is insufficient to establish design criteria.

Tests were made as long ago as 1914 by Johnson and Nichols.[1] The results of recent tests conducted by Hanson[2] were published subsequent to the manufacture of the beams considered herein; reference to Hanson's tests will be made in another section of this paper. At the time of writing, a study was being made at the University of Wisconsin under the direction of G. W. Washa and sponsored by the Reinforced Concrete Research Council and the Wisconsin State Highway Department.

A pilot test program of limited scope was undertaken to study the effect of three joint treatment methods and the problems of instrumentation. Six composite beams and two monolithic beams were tested. Two levels of horizontal shearing stress at the joint were produced by varying the width of the contact surface between the web and flange.

TEST SPECIMENS

The test specimens were under-reinforced since this is the general case in practice. The design of the specimens was based on the use of concrete with an ultimate compressive strength of 3000 psi, and intermediate grade steel reinforcement. To limit the variables, the specimens were designed to have the neutral surface along the joint; this was done by assuming a modular ratio of 10. Later, based on the experimentally determined modulus of elasticity of the concrete, the location of the neutral surface was computed as ¼ in. above the joint.

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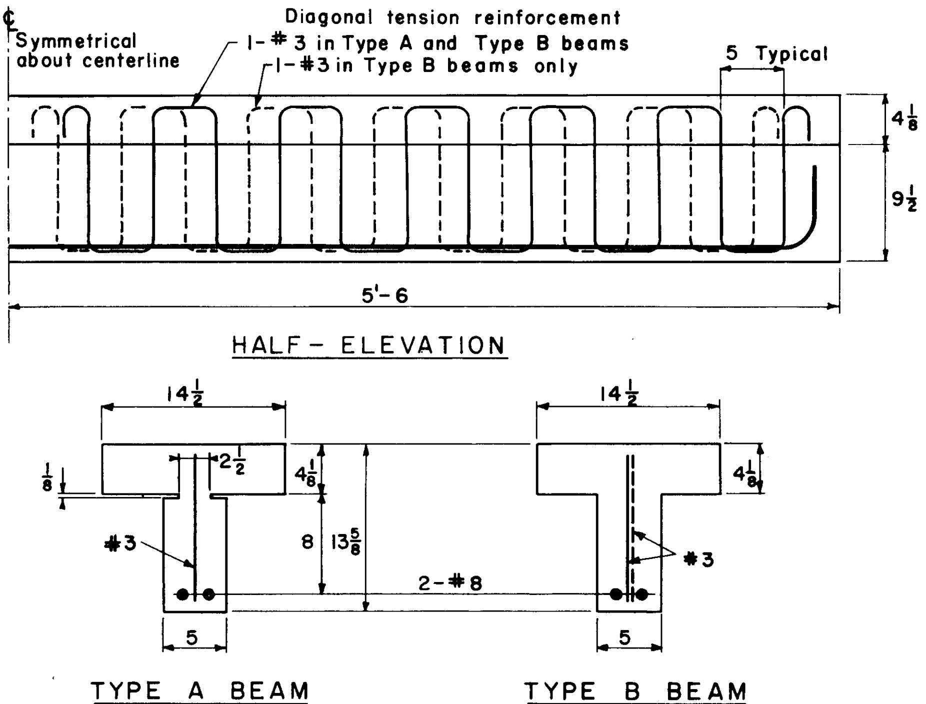

Fig. 1—Details of specimens

It was decided to make the horizontal shearing stress at design load a test variable and, accordingly, two types of T-beams were constructed as shown in Fig. 1. Both types of specimen had a 5 in. web width, but Type A beams had the width reduced to 2½ in. at the joint by using rubber strips attached to the web top prior to casting the flange concrete. These were removed after curing. Specimens of either type differed only in the method used to join the flange and web. To keep the percentage of steel across the joints of Types A and B beams constant, twice as much diagonal tension reinforcement was used for the Type B specimens. The vertical legs of the diagonal tension reinforcement were 5 in. on centers in Type A beams and 2½ in. on centers in Type B beams.

Three pairs of specimens, each pair consisting of a Type A and a Type B beam, had their flanges cast to the webs after the latter had been cured; the fourth pair was cast monolithically. The joint treatments used are listed in Table 1. The roughened joint was obtained by forcing the edge of a wood block into the top of the still-plastic web concrete. This produced transverse V-shaped grooves approximately ¾ in. deep and 1½ in. on center. After curing, the smoothly troweled tops of the webs of Beams 5 and 6 were coated with epoxy adhesive and the flange concrete placed immediately thereafter.

The average age of the beam specimens at testing was 5 months. Ultimate compressive strength of standard cylinders of web and flange concrete averaged 4400 psi. The average yield point of the reinforcing bars was 50,000 psi.

TABLE I — JOINT TREATMENTS OF BEAMS

Beam	Type	Joint finish
1	A	Troweled smooth
2	B	Troweled smooth
3	A	Roughened surface
4	B	Roughened surface
5	A	Troweled smooth, adhesive applied
6	B	Troweled smooth, adhesive applied
7	A	Monolithic beam
8	B	Monolithic beam

LOADING AND INSTRUMENTATION

Load was applied to the specimens at two points, as shown in Fig. 2, by a hydraulic system. The 14 kip design load corresponds to horizontal shearing stresses of 260 psi in Type A beams and 130 psi in Type B beams as computed by the elastic theory.

For each beam the load was applied in 10 equal increments to 14 kips. The load was then released in two equal increments. Finally, the load was increased until the beam collapsed, using 2.8-kip increments. The initial loading caused flexural cracking of the section and thereby tended to eliminate discontinuous response during the final loading. Vertical deflection, strains and so-called slip measurements were made after each increment of load until a marked increase in the deflection rate forced removal of the deflection dial gages. The loading was then continued until the beam collapsed.

Deflections were determined at seven stations along the span by dial gages reading directly to 0.001 in. Strains were measured by SR-4 electrical resistance gages having a 2½ in. gage length. These gages were cemented to the specimen surface at the center and at one of the quarter points of the span. The pattern of strain gages at both locations is shown in Fig. 2.

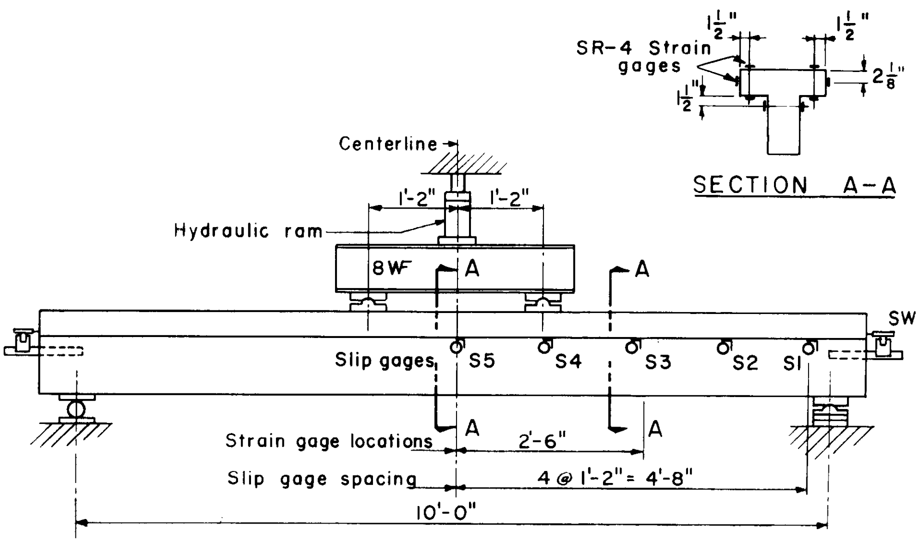

Fig. 2—Loading and instrumentation

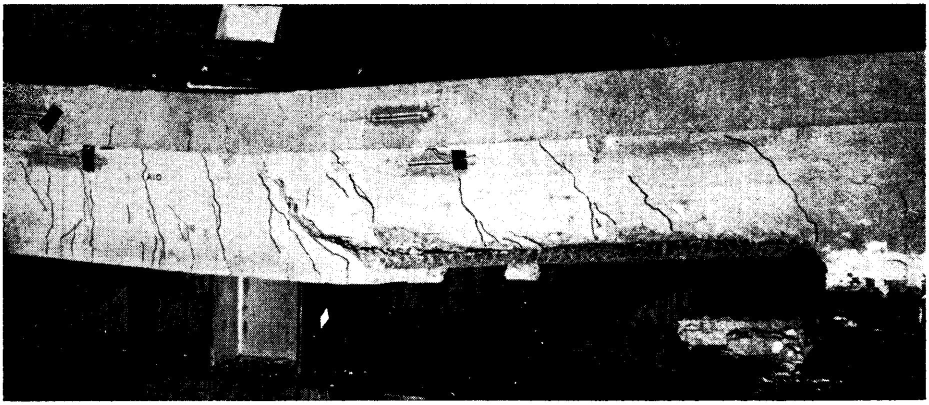

Fig. 3—Typical shear-tension failure

Five small dial gages, reading directly to 0.0001 in. were used to measure relative longitudinal movement between web and flange. The gage locations are shown in Fig. 2. The dial gages support consisted of a ¾ in. square by ¼ in. thick aluminum block cemented to the beam web about 1 in. below the joint. It had a drilled hole to receive the dial gage post bracket which was secured by a setscrew. The dial gage plunger bore on a piece of aluminum angle cemented to the underside of the beam flange. Larger dial gages, also reading to 0.0001 in. were mounted at the ends of the beams as indicated in Fig. 2.

TEST RESULTS

Primary failure, caused by yielding of the longitudinal reinforcement, occurred at about a 35-kip load and was evidenced by a sharply increased rate of deflection. Except for Beam 3, no flange cracking was observed before primary failure although web cracks reached the construction joint at loads just slightly larger than the 14 kip design load. All beams were able to sustain additional loading after primary failure but the accompanying deflection and cracking were excessive.

Secondary failure was of the shear-tension type, illustrated in Fig. 3. The peak load carrying capacity of the beam was reached just after a large diagonal web crack had propagated horizontally, at the level of the longitudinal steel, to the reaction. The concrete below the horizontal crack tore off, destroying the bond with the reinforcing steel and in some cases causing an explosive failure of the concrete surrounding the hook. In one of the beams a shear-compression type failure occurred before the hook failed.

Had conventional U-type stirrups been used as diagonal tension reinforcement, secondary shear-tension failure might not have occurred. The diagonal tension reinforcement was not welded to, or wrapped around, the main reinforcement. Thus, as the diagonal crack began to widen, the vertical component of the motion could be resisted only by the longitudinal steel. This resulted in vertical tensile forces in the concrete on the plane of the longitudinal steel, causing horizontal cracking. If this phenomenon had been prevented, it is possible that the peak load capacity of some of the specimens might have been increased. This is, however, of little importance since the primary failure load had been reached previously.

In Beam 1, which had a smooth joint finish, there occurred separation of flange and web. Since this was a Type A specimen with a reduced joint width, it was not possible to determine the

TABLE 2—MAXIMUM MOVEMENTS BETWEEN FLANGE AND WEB DURING INITIAL LOADING

Beam	Type	Maximum movement at 14 kips initial loading	Maximum residual movement after initial loading
1	A	0.0080	0.0041
3	A	0.0013	0.0003
5	A	0.0003	0.0001
7	A	0.0012	0.0006
2	B	0.0005	0.0004
4	B	0.0003	0.0002
6	B	0.0003	0.0001
8	B	0.0005	0.0003

†Values shown are not necessarily at the same gage location for any beam.

full extent of the separation by inspection. It is believed, however, that it extended for a major portion of the shear span. This was the only beam in which separation of flange and web was evident. The behavior of Beams 5 and 6, both having smoothly finished joints with adhesive, differed from that of the other specimens in that they exhibited less cracking at all loads up to failure.

Deflection

Deflection measurements at the center line of the span, under initial loading, are shown in Fig. 4. Since the load-deflection curves were similar and, except for Beams 1 and 6, lay within a narrow band, only an envelope enclosing these curves is shown. The change in slope of the envelope at about a 7-kip was due to a decrease in stiffness resulting from cracking. For Beam 1 the load-deflection curve deviates from the envelope at approximately 5 kips and thereafter exhibits a flatter slope than the envelope. The curve for Beam 6 departs from the envelope at about a 7-kip load and thereafter exhibits a steeper slope than the envelope. The reasons for the increased stiffness of Beam 6 are unknown.

Fig. 5 shows load-deflection curves for the final loading. Since the measured deflections of several beams would overlap if plotted, individual points are not shown. A single curve has been drawn to represent the behavior of several beams where overlapping would occur. The deflection of a Type A beam is greater in every case than that of the corresponding Type B beam. In fact, the deflections of all the Type A beams are greater than any of the Type B beams at the higher loads. Type A beams, with only half as much web reinforcement as Type B beams, exhibited larger and more extensive cracks. Most likely this is the cause of the increased deflection at the higher loads. The curve for Beam 1 deviates from the other specimens from the beginning of loading

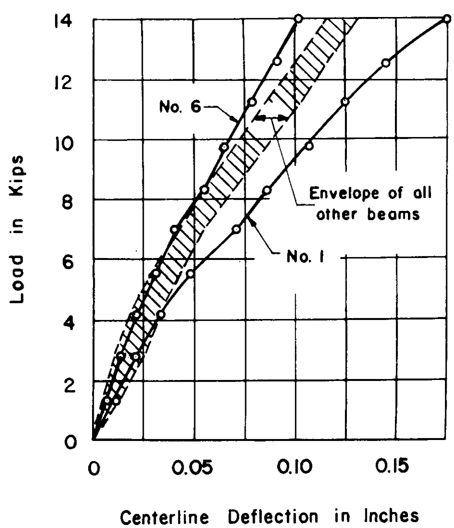

Fig. 4—Initial loading versus deflection

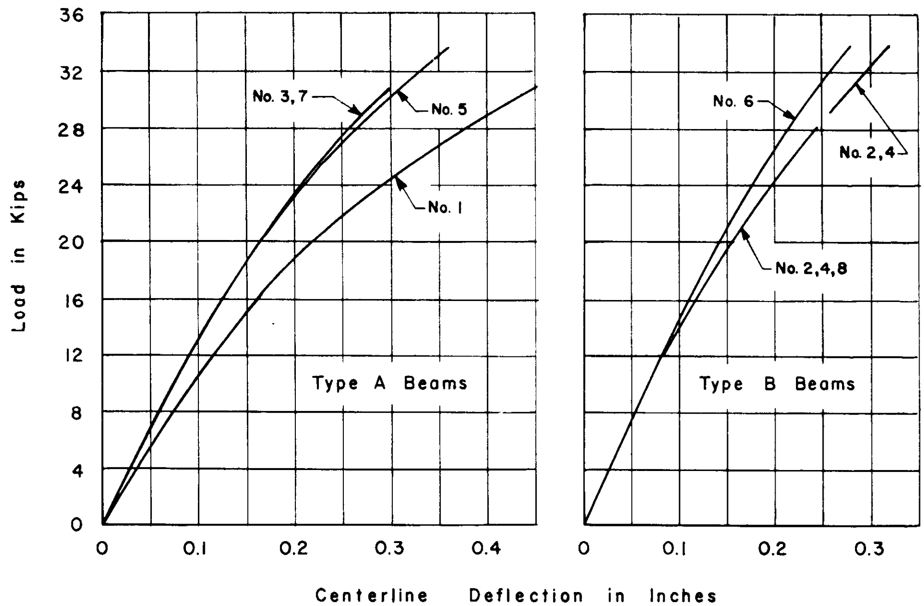

Fig. 5—Load-deflection curves for final loading

because of the apparent loss of composite action under the initial loading.

Slip measurements

In general, the largest relative movements between flange and web were measured by dial gages at Locations S2 and S3, which were midway between the reaction and one load point. Much smaller movements were recorded by the gages at Locations S1 and S4. Very small movements were measured at the center line and at the ends of the specimen.

Under the initial loading, Beam 1 had a maximum movement of about 0.008 in., which was much larger than that of any other specimen (see Table 2). Beams 3 and 7 had the next largest movements, about 0.0012 in., while the other beams averaged close to 0.0004 in. On removing the load, the residual movements were quite small, averaging about 0.0003 in. for all beams except Beam 1, which had a maximum residual movement of 0.0041 in.

Additional movements were measured on reloading. Fig. 6 shows the measurements for all beams under 14 and 28 kip loads. The gages were not reset after the initial 14-kip loading; consequently, due to the large residual movements which occurred, the range of some of the gages on Beam 1 was exceeded at loads below 28 kips. The largest movements measured by these gages before their range was exceeded, and the corresponding load, are indicated in Fig. 6. The uniformly large magnitude of movements in the shear span of Beam 1 justifies the belief that the horizontal crack at the joint extended along a major portion of the shear span.

The monolithic specimens, Beams 7 and 8, exhibited movements of the same magnitudes as the corresponding composite specimens. A monolithic beam would not be expected to slip at all; therefore, the movement measured was probably due to cracking. It is entirely possible, then, that for the composite beams the movements measured were not slip, or at least not slip alone. The small slip readings of Beams 5 and 6 might be due to the relatively small amount of cracking which they underwent.

While cracking might be the reason for most of the relative movement of Beams 2 through 8, it cannot account for the behavior of Beam 1. In this case it is definite that slip made up at least a part of the movement which was measured. Only Beam 1 registered appreciable movement at the ends of the span. Horizontal cracks at the ends of the beam and separation of the flange and web were visible indication that slip took place.

Strain measurements

Strain gages were used on the beams for two purposes. The first was to determine the position of the neutral axis. The second purpose was to aid in determining the load at which composite action was lost.

Strain gages were successfully used to accomplish the first purpose. At a 14-kip load the strain gage readings closely confirmed the computed location of the neutral axis based on the

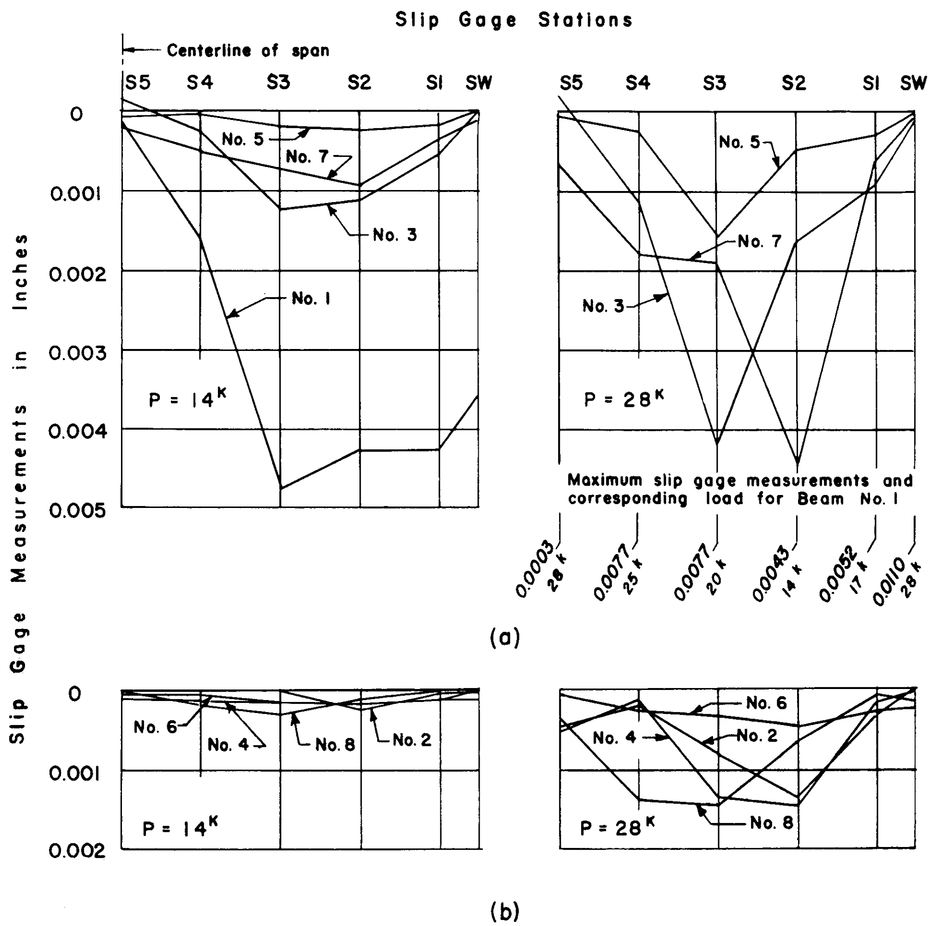

Fig. 6—Slip-gage measurements at points along span; (a) Type A beams; (b) Type B beams

experimentally determined modulus of elasticity of the concrete.

The use of strain gages for determining the initiation of noncomposite behavior met with only limited success. The significant gages for this purpose were at the bottom of the flange and near the top of the web. The observed strain distribution for Beam 1 followed the expected behavior for partially noncomposite beams at both the center line and the quarter point of the span. The noncomposite behavior indicated by the strain gage measurements is consistent with that demonstrated by the slip and deflection measurements. The only indication of noncomposite behavior in the other beams was the development of compressive strains at the gages on the webs of Beams 5 and 7. This indication of noncomposite behavior is not consistent with the other measured data for these beams. The results of the strain gages on the webs of all the beams are unreliable. The likelihood of cracks intersecting the gage is considerable since the gage length was 2½ in. Even if cracks were not to intersect the gage, the complex redistribution of stresses and strains in the beam webs due to cracking makes the interpretation of these strains difficult.

DISCUSSION

Studies of composite structural steel and concrete beams have used push-off tests to determine the capacities of various types of shear transfer devices. The convenience and relatively low cost of such specimens enabled investigators to study many variables. Hanson[2] adopted push-off tests for studying the construction joints of composite concrete-to-concrete beams. He concluded that push-off tests demonstrate characteristics of slip versus shear stress similar to those obtained from beam tests. The validity of Hanson's conclusion seems questionable to the authors for two reasons:

(1) The measurement of actual slip is difficult in beam tests and

(2) The effects of web cracking in a beam are not reproduced in a push-off test.

A fully composite beam would transmit the shearing forces at the joint with the same deformations as a monolithic beam. Referring to Fig. 6, note that, with the exception of Beam 1 which displayed definite noncomposite behavior, the measured movements of both the Types A and B beams were of the same magnitudes as for the corresponding monolithic beams. Since the behavior of the composite beams is so similar to that of the monolithic, it is likely that the measured movements are due to the same causes. If the possibility of slip in a monolithic beam is ruled out, then web cracking must account for these relative movements. Further note in Fig. 6 that the relative movements of the Type A beams, both composite and monolithic, are much larger than those of the corresponding Type B beams. The Type B beams, having twice as much web reinforcement, developed fewer and smaller cracks and therefore exhibited less relative movement between flange and web.

Even though Beam 1 developed large slips, it is not possible to determine how much of the measured movement was actual slip and how much of it was due to cracking. An estimate of the actual slip can be made by subtracting the movements of the monolithic beam from those of Beam 1, but since loss of composite action affects the cracking of the beam, this estimate is, at best, a rough one. Thus, while the determination of slip in push-off tests appears possible, its determination in beam tests is difficult.

Hanson suggests that the critical "slip" for composite action in his beam tests was 0.005 in. It is the authors' contention that his "slip" measurements include cracking. It is also their opinion that an attempt to develop design criteria based on permissible slip, as was done for structural steel and concrete composite construction is unrealistic.

ACKNOWLEDGMENT

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