Flexure, Shear and Torsion Tests on Prestressed Concrete I-Beam

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This paper presents the results of a series of full-scale tests performed to evaluate design procedures, determine ultimate strengths, and answer other design questions concerning prestressed concrete I-beams. To provide this information several beams both with and without compositely acting roadway slabs were tested in moment, shear, and torsion.

Data from these tests are recorded and discussed and some conclusions are drawn. These tests will provide useful information for bridge designers and also serve as a basis for planning further investigations and tests of prestressed concrete I-beams.

•EARLY IN 1959 the Texas Highway Department recognized the need to investigate and evaluate design methods and criteria currently in use by the department in designing prestressed concrete I-shaped beams. Some of the design methods and criteria used in designing these prestressed concrete beams were theoretical in nature and not completely substantiated by experimental evidence. Also, some of the methods used by the Texas Highway Department were not in general use by other State highway departments. This series of tests was performed to verify design methods and criteria and to provide new information on prestressed concrete I-beams.

Current design practice of the Texas Highway Department was to design prestressed concrete beams with composite slabs using the same modulus of elasticity for both slab concrete and beam concrete. The beams were considered to act compositely with the slabs without the presence of shear keys in the top of the beams. The evaluation of the soundness of these design assumptions was one of the objectives of this investigation.

At the time these tests were undertaken there was no well-established method for calculating ultimate moment and shear capacities of prestressed concrete I-beams with compositely acting slabs. Also unknown was the effect of occasional overloads on such members and whether these overloads would produce permanent and excessive deflections in the member. Identification of these ultimate moment and shear capacities and new knowledge concerning overloading were further objectives of the investigation.

The final concern of this investigation was the reaction of a prestressed concrete I-shaped beam to a torsional load. Because little information was available on this subject, tests were conducted to provide new data that might indicate the allowable value of resisting moment and angular rotation that these beams were capable of withstanding.

EXPERIMENTAL CRITERIA

Beams

The beams tested were of I-shaped cross-section and of the type designated as C-beams by the Texas Highway Department. Roadway slabs were cast on two test beams to provide a composite section for investigation under shear and flexure loadings. One of these beams was a standard pretensioned beam, and the other was a nonbonded post-tensioned beam. The pretensioned beam was not loaded to complete flexural failure so that shear tests could be made on the ends of the beam. To determine the effect of end blocks on the beam, one end of the beam was cast with an end block and the other without. Percentage of stirrup reinforcing steel was the same for both ends of the beam.

Two beams were tested as noncomposite sections; one beam in bending and the other beam in torsion. The beam tested in torsion was a beam that had been rejected from a bridge project due to excessive honeycombing at one location in the beam.

Design

The elastic theory was used for prestressed concrete members under design loads at working stresses. Assumptions were made that (a) strains will vary linearly over the depth of the member throughout the entire load range; (b) before cracking of the concrete, stress is linearly proportional to strain; (c) after cracking of the concrete, tension in the concrete will be neglected; and (d) for the composite section, the modulus of elasticity used was that calculated for the beam concrete.

Temporary allowable stresses in the concrete before losses are 0.60 f'ci in compression and $6\sqrt{f'ci}$ in tension where f'ci is the compressive strength of the concrete at initial prestressing. Minimum allowable f'ci = 4,000 psi. Allowable stresses at design load, and after losses have occurred, are 0.40 f'c in compression and zero in tension where f'c is the compressive strength of the concrete at 28 days.

Prestressed members were designed for a computed ultimate load capacity of not less than 1.5 dead loads plus 2.5 live loads including impact. Beams were designed to act compositely with the roadway slab so that failure of the steel rather than of the concrete would occur at ultimate loading.

Section Properties

Pretensioned members conformed to the beam properties shown in Figure 1, which also shows a composite slab cast integrally with the beams. Prestressing forces are applied with 45⅜-in. diameter stress relieved strands at 14,000 lb per strand. Figure 2 shows the configuration of the post-tensioned beam with a composite slab. This beam was designed with the same assumptions and allowables as the pretensioned beams and differs only by the change in type of stressing. The post-tension tendons A, B, and C are 13¼-in. diameter stress relieved wires, and tendons D, E, and F are 10¼-in.

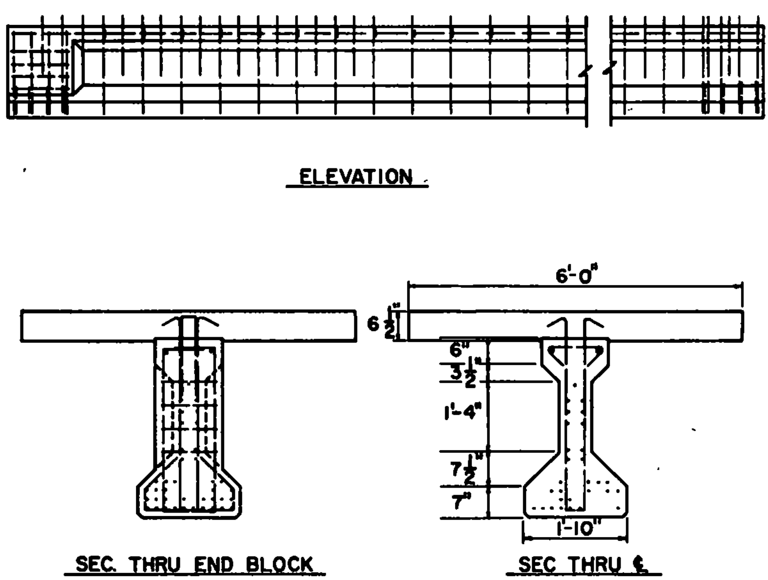

Figure 1. Pretensioned test beam details.

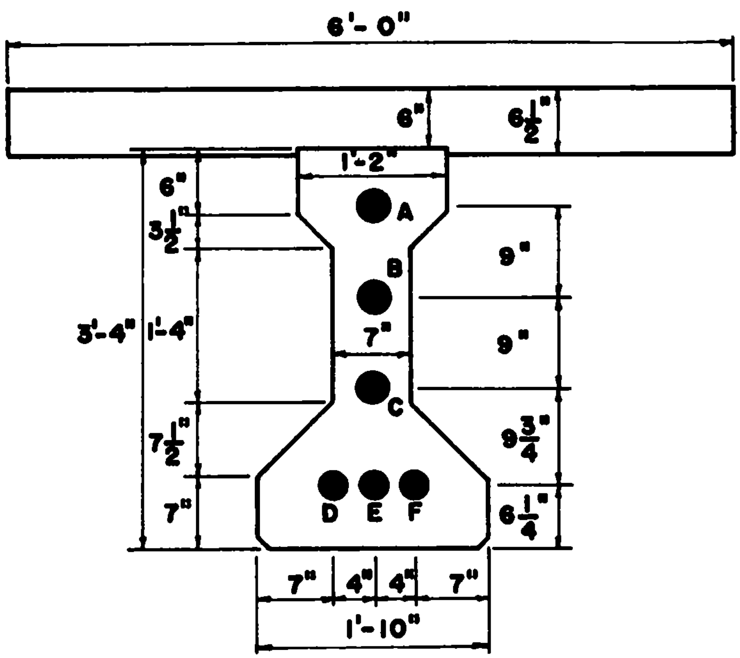

Figure 2. Post-tensioned test beam details.

diameter stress relieved wires. These wires were not grouted in their conduits. This beam was designed to produce the same initial and final design stresses as those produced by the pretensioned beams.

Concrete

Specifications for beams require Texas Highway Department Class F concrete having a compressive strength of 5,000 psi in 28 days. Maximum size of the coarse aggregate used was 1 in. A cement factor of 6.50, a water factor of 5.25, and a workability factor of 0.82 was used. Measured average slump per batch was 3 in. Specifications for slab concrete require Texas Highway Department Class A concrete having a compressive strength of 3,000 psi in 28 days.

Steel

Prestressing steel was ⅜-in. diameter stress-relieved seven-wire strands with a minimum specified ultimate strength of 250,000 psi. Post-tensioning tendons were made of ¼-in. diameter stress-relieved steel wire with a minimum specified ultimate strength of 240,000 psi. Plain reinforcing steel for stirrups, etc., was intermediate grade billet steel.

TESTING EQUIPMENT

Permanent testing facilities were constructed at Texas Highway Department District 14 Headquarters in Austin to accommodate the testing of these beams and for use in any future testing programs that might be undertaken. A concrete slab 8 ft wide, 12 in. thick, and 102 ft long was cast to form a smooth-working surface. Two movable concrete abutments served to support the beams while testing. A steel frame built of 12BP53 steel members straddled the testing bed to provide a loading yoke for the jacking systems. The legs of the loading yoke were sunk 10 ft deep into 24-in. diameter shafts drilled 20 ft through layers of shale and Austin chalk. Ten 1⅛-in. diameter reinforcing bars were welded to the loading frame legs and extended into the 3-ft diameter

bells on the bottom of the concrete-filled drilled shafts. At appropriate distances from the loading yoke, 18-in. diameter drilled shafts were located under the test bed to provide support for the abutments under the high reaction loads required in the shear tests.

Load was applied to the beam by one of two jacking systems. The first jacking system consisted of two 100-ton capacity hydraulic jacks connected in series to provide a total capacity of 200 tons. High pressure lines connected the jacks to a manual pumping unit on which a pressure gage and a relief valve were mounted. This system was calibrated in a verified hydraulic testing press before use in the tests. The second jacking system used a single 150-ton capacity jack with the same pumping system.

INSTRUMENTATION

Stresses in the composite pretensioned beam were estimated by measuring strains, determining Young's Modulus of representative specimens of the material under strain, and calculating the stresses from the relation between these data. Strains were measured (a) electrically by means of bonded wire strain gages, and (b) mechanically by means of a 20-in. Berry strain gage.

For the electric strain gages the SR-4, Type A-9 gage was selected. Concrete is recognized as a heterogeneous mixture of discrete particles. The largest particles, in this particular case, were about 1 in. in maximum dimension, which is as large as or larger than the usual electric strain gage. The A-9 type with its 6-in. gage length extends over a sufficient distance to be affected by a statistically representative portion of the material under test, yet is small when compared to the size of the beam.

For the moment test on the composite section, the gages were arranged in longitudinal lines on both sides of the beam and in transverse belts around the beam called stations. Two lines of gages extend along the top of the floor slab immediately above the upper flange of the beam, 5 in. to each side of the longitudinal centerline, and two lines were similarly placed along the underside of the lower flange. All of the previously mentioned gages were oriented with their major axes parallel to the longitudinal centerline of the beam. At the quarter points and at midspan, gages were placed on the top and bottom of the roadway slab at the center of the overhang on both sides. These gages had their major axes perpendicular to the longitudinal centerline. The arrangement of the gages is shown in Figure 3.

In the case of the shear test on the end of the beam having a conventional end block, the gages were arranged as shown in Figure 4. In the delta strain rosettes on either side of the beam, 7 ft 9 in. from the end, one gage was vertical and the other two placed so as to complete an equilateral triangle. This orientation was chosen in anticipation

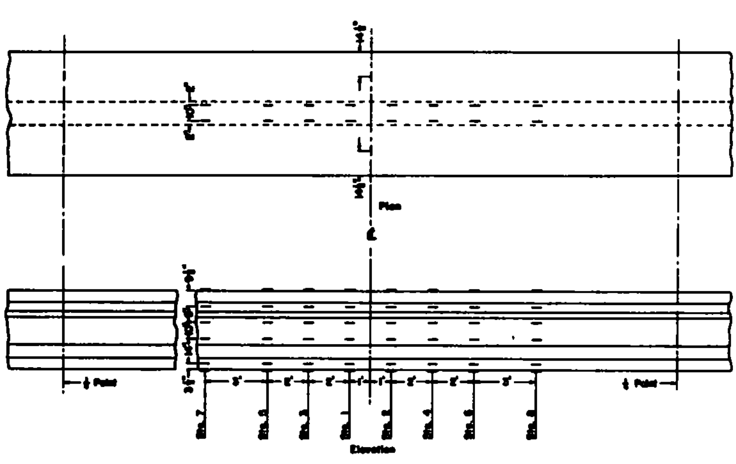

Figure 3. Strain gage locations, composite moment test, pretensioned beam.

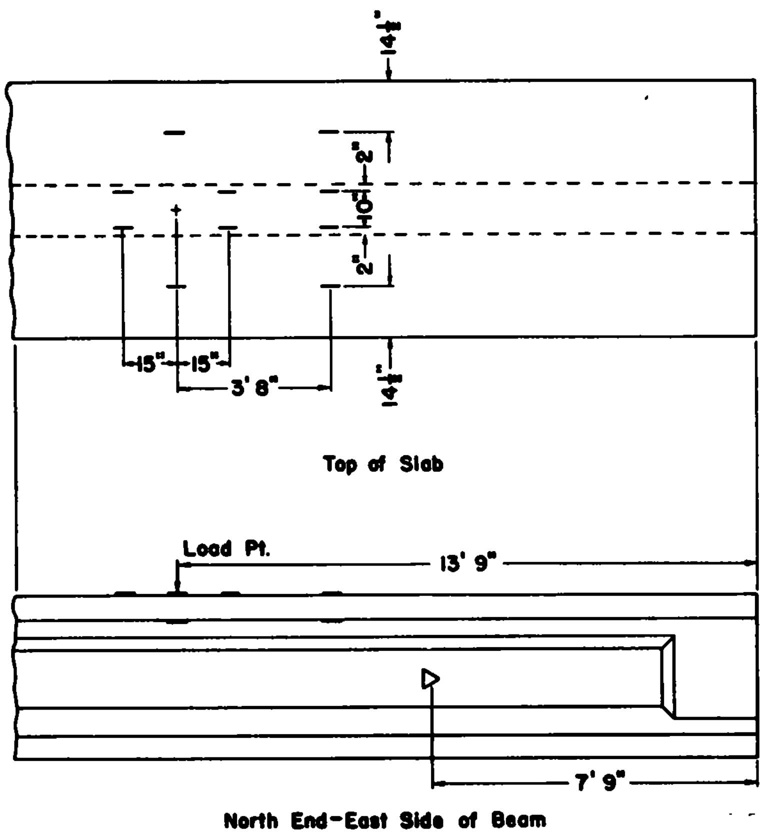

Figure 4. Strain gage locations, composite shear test, pretensioned beam.

that the direction of the major principal stress would be very nearly parallel to one of them. The strain gages on the top and underside of the floor slab had their major axes parallel to the longitudinal centerline of the beam. No strain gages were placed on the opposite end of the beam for the shear test at that point.

In the noncomposite moment test (i.e., on the beam without a roadway slab), the gages were placed in the pattern used for the composite beam test except that only four belts of gages were used, those in Stations 1, 2, 3, and 4 as shown in Figure 3. Also, those longitudinal gages previously located on the slab were now mounted directly on top of the beam.

The compensating or dummy gages were attached to concrete cylinders made from the same concrete batch used in casting the beams. These cylinders were cured with the beams and kept with them during each move.

To apply the gages to the beam, the locations were marked and the necessary area ground smooth with an offset grinder. A dark smear left by the adhesive in the abrasive disc interfered with the adhesion of epoxy cement to the concrete and final surface preparation was by sand blast. After sand blasting each position, a smooth thin layer of Armstrong A-1 epoxy cement was applied to a 1- by 10-in. area. When this layer had hardened, a pressure sensitive cement was used to fasten the gages to the epoxy layer. Gage leads were cemented to the epoxy layer with small additional quantities of epoxy cement. After the lead wires had been soldered to the gage leads, several coats of Di-Jell waterproofing wax were applied to each installation, extending well up on the insulation of the lead wires. The layer of epoxy, purposely kept very thin, served to insulate the gages from the beam, and the wax layer provided protection from the weather. All lead wires were cut to a 23 ft length. The cables contained eight

No. 22 AWG stranded, insulated copper wires. Color coding was helpful in following connections. The opposite end of each wire was soldered to a banana jack in a plexiglass switching panel. Eight wires with banana plugs soldered to both ends were used to connect the switching panel to the measuring bridge.

The bridge circuit used to measure the strain indicated by the gages was built especially for this application. It was patterned after a circuit published by Matlock and Ripperger (1) and is shown schematically in Figure 5. Essentially, it is a Kelvin double bridge with dual leads to the active and dummy gages, which serve to reduce or eliminate temperature effects in the lead wires, and with voltage splitters of selected resistance at the battery corners to reduce the error caused by small differences in contact resistance at the switching panel. The equations describing the reduction of error are discussed by Matlock and Thompson (2). In this adaptation resistances R_1 and R_2, R_3 and R_4, R_5 and R_6, R_B and R_D are selected pairs of nominal 300-ohm strain gages whose actual measured resistances are equal to the nearest 0.01 ohm. These pairs of gages were cemented to 1½- by ⅛-in. mild steel straps supported in a vertical position. R_7, R_8, and R_9 are short connectors of extremely low resistance. The helical slide wire consisted of 112 ft of No. 12 B & S Manganin wire having a resistance of 0.043 ohms per ft wound in 73 turns about a grooved Lucite cylinder 6 in. in diameter and 19 in. in length. The sliding contact was mounted so that it could be placed on any desired turn of wire. Consequently, no more than one complete revolution of the cylinder was required for balancing and it was possible to make fixed connections to the ends of the coil, rather than brush contacts. A sensitive Brown Electronik null indicator connected as shown in the diagram completed the bridge. Everything inside of the dashed line in the drawing was mounted in a box with a hinged lid.

Calibration of the slide wire was accomplished by the following procedure. A-9 strain gages from the lot to be used in the project were connected into the active and dummy positions of the bridge. The dummy was shunted with a 20,000-ohm fixed resistor; the slide wire contact was placed near one end of the coil and the bridge was

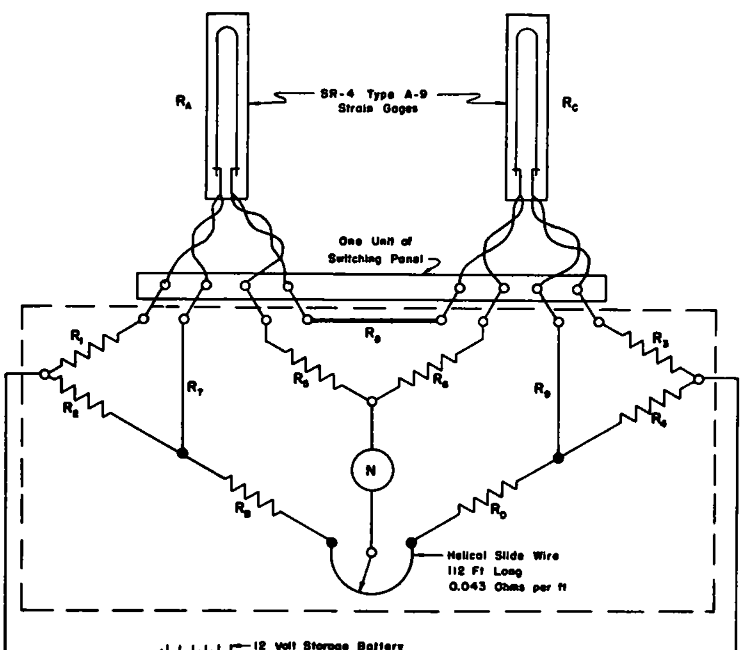

Figure 5. Bridge circuit used for measuring strains.

balanced by means of a variable decade resistance (Dekabox) shunted across the active arm. A mark was made on the slide wire cylinder. Then the variable resistance was changed by a calculated amount that resulted in a change of unit resistance of the active bridge arm equivalent to 25μ in. per in. of strain. The bridge was balanced by moving the slide wire contact and a new point on the cylinder was marked 25. Many repetitions of this procedure served to calibrate and mark the entire slide wire coil. A distance of approximately 2½ in. on the wire was equivalent to 25μ in. per in. of strain. A vernier scale fastened to the contact arm close to the coil made it possible to read strain to the nearest 1μ in. per in.

In use, the bridge was balanced for each gage with no load applied to the beam and the balance point recorded. When the gages were strained a new balance point was determined for each one and the difference between initial and subsequent readings indicated the strain produced by the loading. If strain gages having a gage factor different from that factor used in this calibration are employed, the readings must be corrected by the ratio of the two gage factors.

In the moment test of the composite beam full advantage was not taken of the dual lead feature because single leads connected the gages to the switching panel. Dual leads connected the panel to the bridge. Because there was a dummy gage and all leads were of the same material, length, and exposure, the temperature compensation was adequate. Matlock and Ripperger (1) point out additional precision could have been realized by reversing the battery connections and gage leads on every reading and taking an average of the four values of strain thus obtained.

In the mechanical measurement of strains, reference points for the 20-in. Berry gage were on steel discs (⅛-in. slices from a No. 5 reinforcing bar) that were bonded to the beam at 10-in. intervals with Armstrong A-1 epoxy cement. Thus, it was possible to set overlapping 20-in. gage lengths with the standard punching template. The punch marks were then deepened with a No. 54 drill. There were gage lines containing twelve such points on both sides of the beam at midheight of the web and along the lower flanges 6 in. above the bottom. These lines extended 55 in. left and right of the center of the beam. Strains were measured after every increment of load was applied. This system of measurement was intended to insure that test data should be available even though damage might cause the electric strain gage circuits to become inoperative or unreliable.

Wood scales made from 2-ft lengths of folding rules were fastened in vertical and horizontal positions at the bearing points, center, and quarter points of each beam. These scales were nailed to blocks and braces that had been set in epoxy cement and could be read accurately to $\frac{1}{100}$ ft. A precise level stationed about 50 ft from the beam was used to read vertical movements, and a theodolite stationed about 50 ft beyond one end of the beam was used to read horizontal movements.

To measure deformations in the first torsion test, a large protractor type scale having a radius of 19 in. was fastened to the top of each end of the beam. The smallest division of this scale was 15 min. A heavy pointer was fastened to the center of the scale on a loose-fitting pin. As the beam was twisted, the scale moved; but gravity kept the pointer vertical so that the difference in initial and final readings indicated the angle of deformation. It was apparent in the course of the first test that greater precision in measuring the angle of deformation was desirable, so another device was constructed for the second test. A horizontal arm extending equally to each side of center was fastened to the bottom of the beam at each end. Two-ft sections of these mentioned wooden scales were hung on loose pins at the outer ends of the cross pieces, exactly 3.438 ft left and right of center of the beam. Metal weights fastened to the bottom of these free-swinging scales kept them in a vertical position. Initial rod readings were taken on each of these scales with a precise level at no load and at each increment of load. The length of these gage arms (3.438 ft) was selected so that when one-half of the algebraic sum of the rod deflections is 0.060 ft, the angle of rotation of the beam is 1°.

Bourdon gages (Carver) of appropriate capacity were connected into the oil pressure lines to the hydraulic jacks. Each jack and gage combination was calibrated in a press that had been verified by precision instruments. Large-scale calibration curves were used to convert hydraulic pressure to load in pounds.

Portable radio sets were used to coordinate the activities of the several people involved in the tests. They were particularly useful when possible danger from flying fragments made it advisable to work behind protective barricades.

EXPERIMENTAL INVESTIGATIONS

Composite Moment Test

A standard pretensioned beam was placed on the testing bed for investigation under moment loading. The beam was seated on a 6- by 18- by ¾-in. 70 durometer neoprene bearing pad as is usual in bridge construction. A compositely acting slab 6½ in. thick and 6 ft wide was cast on this beam with all formwork being supported by the beam. The over-all beam length was 59 ft 6½ in. with a center-to-center-of-bearing length of 58 ft 10½ in. The dead load of the slab caused a ⁵⁄₁₆-in. deflection in the beam. Longitudinal strains and horizontal and vertical deflections were recorded at predetermined increments of load. Standard cylinder tests were made of beam and slab concrete samples taken at the time of pour. Beam cylinders had an average compressive strength of 8,923 psi and the slab cylinders had a compressive strength of 5,215 psi. Calculated modulus of elasticity of the prestressed concrete was 4.91×10^6 psi.

AASHO H20-S16 design load for this span length and beam spacing was found equivalent to a concentrated load of 41.7 kips including impact. Based on the composite section, initial cracking was calculated to occur at a load of 76 kips or dead load plus 1.82 live loads. Ultimate design according to the AASHO Specification used is 1.5 DL + 2.5 (LL+I) or 193.65^k. For comparison with final concentrated loads, only an ultimate load of .5 DL + 2.5 (LL+I) or 134.1^k is used as one dead load is already acting on the beam. Load in increments of 8 kips was applied at midspan of the beam until a total load of 65 kips was reached. No cracking of the beam or slab was noted. Load increments were then reduced to 4 kips and the total load was increased to 76 kips. In the loading increment from 76 kips to 80 kips initial tensile cracking in the bottom of the beam was observed. Vertical deflection at this load was 0.028 ft. Tensile strain on the bottom of the beam was 260μ in. per in. Compressive strain at the top of the slab reached 193μ in. per in. Eighty kips of load corresponds to 1.92 live loads.

The first loading cycle was continued in increments of 4 kips until a total of 130 kips, or dead load plus 3.12 live loads, was reached. Tensile strain measurements were no longer reliable due to the multitude of flexural cracks that had penetrated up into the top flange of the beam. Flexural cracking had been induced over a region approximately 8 ft on either side of the loading point. Compressive strain in the top of the slab had now reached 520μ in. per in. Vertical deflection at midspan had reached 0.180 ft. At this point the load was released and a check on initial deflection readings showed no residual deflection left in the beam.

Three loading cycles were employed to complete this test due to the excessive time required to record the readings of the 108 strain gages plus the deflection readings. In the first cycle of loading, compressive strains in the slab were very large, whereas on subsequent loadings the relaxation of strain differentials between the beam and slab and redistribution of strains within the slab, showed smaller strains. Figure 6 shows load vs strain, with the average of strains measured at Stations 3 and 4.

The second loading cycle began and the load was quickly brought up to the 130-kip load which marked the termination of the first loading cycle. To facilitate the plotting of a smooth curve a few measurements of strains and deflections were made in this range. Load increments of 4 kips was then applied until a total load of 160 kips or 3.84 live loads was on the beam. Vertical deflection at midspan had reached 0.457 ft. Compressive strain in the slab now reached 928μ in per in. Total load was increased to 164 kips at which point flexural cracking in the bottom of the slab was noted. Flexural cracking had progressed outward to each quarter point of the beam. Cracks had extended the complete depth of the beam in the area 3 ft to either side of the loading point. The load was released and a residual deflection of 0.046 ft was recorded.

Loading for the third cycle was begun and the total load brought up to 168 kips or 4.03 live loads or 1.25 times ultimate loading. Vertical deflection reached 0.552 ft with a measured compressive strain of $1,057\mu$ in. per in. Flexural cracking of the slab

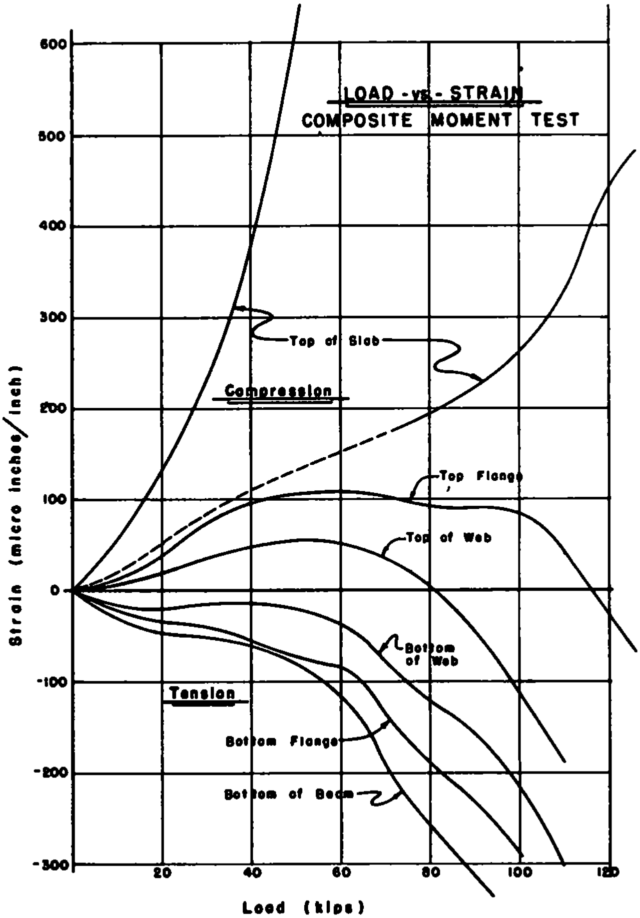

Figure 6. Load vs average strain, composite moment test, pretensioned beam.

Figure 7. Load vs deflection, composite moment test, pretensioned beam.

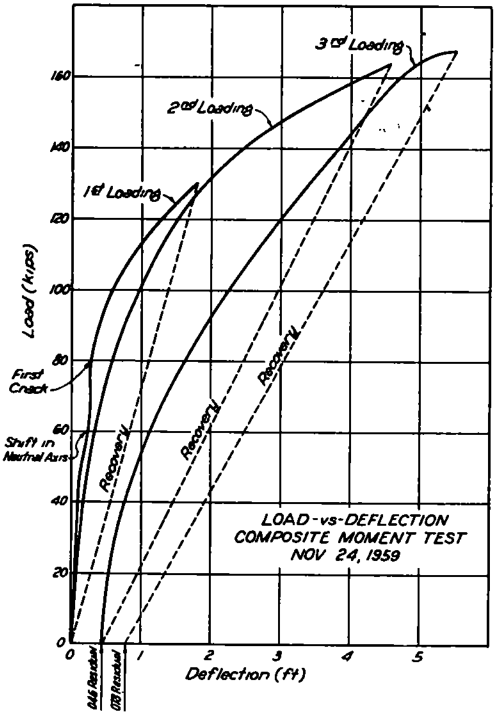

Figure 8. Cracking pattern, composite moment test, pretensioned beam.

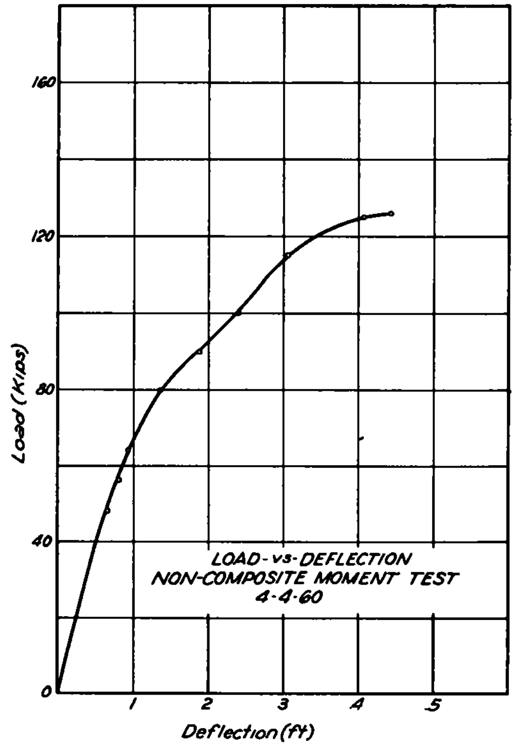

Figure 9. Load vs deflection, noncomposite moment test, pretensioned beam.

penetrated approximately 2½ in. Flexural cracking of the beam under the loading point was extensive, as shown in Figure 7. Figure 8 shows a load vs deflection.

At this point it was decided to discontinue loading to keep the beam intact for use in other tests. On release of the load, the beam showed a total residual deflection of 0.078 ft.

Noncomposite Moment Test

For the noncomposite moment test a standard pretensioned beam was used without a slab cast on the beam. The over-all beam length was 59 ft 6½ in. with a distance center to center of bearing of 58 ft 10½ in. Longitudinal strains along with vertical and horizontal deflections were measured. Standard cylinder tests were made of beam concrete samples taken at time of casting of the beam. Compressive strength of the concrete at the time of testing was 9,126 psi. Calculated cracking load was 67^k.

Load increments of 16 kips each were placed on the beam until a total load of 64 kips was on the beam. Deflection reached 0.095 ft under this load as shown in Figure 9, a plot of load vs deflection for this test. Tensile strains on the bottom of the beam showed 480μ in. per in. of strain induced with compressive strains on the top of the beam of 600μ in. per in. Figure 10 is a plot of load vs average strains measured at Stations 3 and 4. Loading continued to 80 kips where initial flexural cracking was observed in eleven places. Vertical deflection was 0.137 ft with a compressive strain of 925μ in. per in. and a tensile strain of 940μ in. per in. Later observation of the plotted strain indicated that minute cracking could have been present around a load of 64 kips, and a measured tensile strain of 495μ in. per in., without having been detected. At 100 kips, compressive strains reached 1,575μ in. per in. on the top of the beam. Recorded deflection at midspan was 0.239 ft. Flexural cracks had increased in number and length

until cracks were present approximately 7 ft to either side of the loading point and had extended high into the web. Continued loading to 125 kips produced the beginning compressive failure in the top of the beam. At 127 kips, complete collapse of the beam occurred. Deflection at 125 kips reached 0.409 ft, and measured compressive strain was 1,809 μ in. per in. Figures 11, 12, and 13 show the extent of failure in the beam.

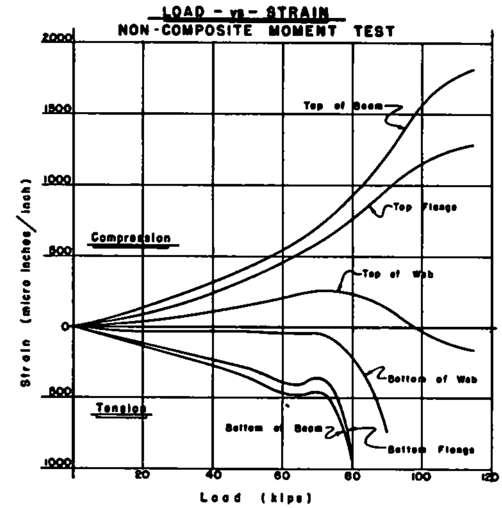

Figure 10. Load vs average strain, noncomposite moment test, pretensioned beam.

Composite Shear Tests

Because of the limited number of beams available, the beam used for the composite moment test was also used for the shear tests. This beam was cast without an end block on one end. Flexural cracks were present in the center portion of the beam, but were not expected to be of major importance other than possibly resulting in larger deflections. However, these cracks did influence the choice for location of a loading point for the shear tests. If the load point was located from 1D to 4D, D being the depth of the beam, away from the reaction then a shear compression type of failure would be expected to occur. Loading in a range from 4D to 7D should result in unrestrained diagonal tension failure. Unfortunately, the presence of the flexural cracks forced the loading point into the shear compression failure range. Loading point for both shear tests was located 13 ft 5 in., or 3.5 D, from the reaction. Strain measurements were made only on the section that had the end block. For both tests the vertical and horizontal deflections were recorded. Hydraulic jacks supplied the load to the sections.

Figure 11. Beam failure, noncomposite moment test, pretensioned beam.

Figure 12. Beam failure, noncomposite moment test, pretensioned beam.

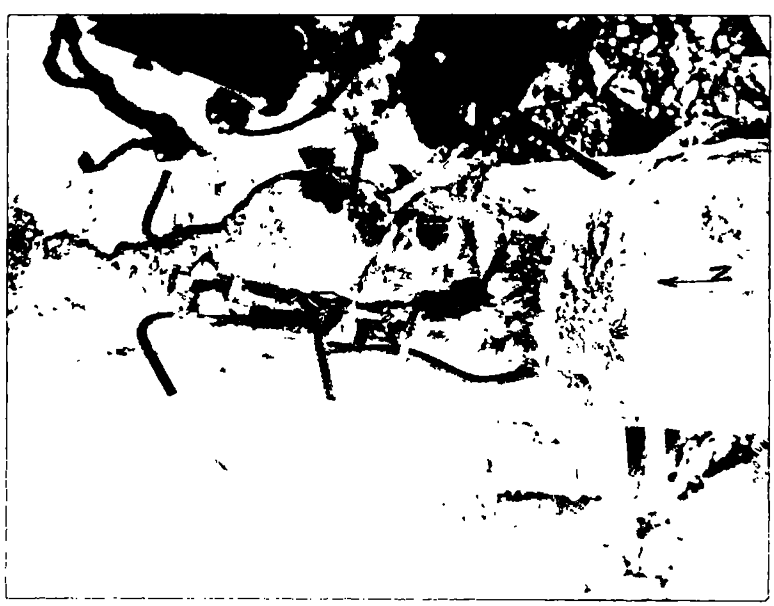

Figure 13. Beam failure, noncomposite moment test, pretensioned beam.

Test Without End Block.—Load increments of 20 kips each were placed on the beam until a total load of 140 kips was reached. Deflection reached 0.093 ft at the load point. In the loading increment from 140 to 160 kips, flexural cracking occurred under the loading point and extended high into the web. Increments of load were reduced to 10 kips and the total load brought up to 200 kips. In this loading range only flexural cracking was observed. Figure 14 shows the deflection under the load point had increased to 0.210 ft. This load of 200 kips was held on the beam for about 5 to 10 min while the crack pattern was being marked. Just before loading was resumed there was a sharp report and two diagonal tension cracks appeared between the loading point and the near reaction. These cracks were at an approximate angle of 25° to the bottom of the beam and extended through the web and into the upper and lower flanges. Only a slight increase of 0.002 ft in deflection was noted. Two short cracks, inclined at about the same angle, appeared and were contained in the web above the bearing point in the region the end block normally occupies. Loading continued to 210 kips where two additional diagonal cracks appeared nearer the loading point and extended through the bottom of the beam. Loading continued to 230 kips with a recorded deflection of 0.310 ft. Additional flexural cracks appeared and the initial diagonal tension cracks lengthened. Due to darkness, loading was discontinued and the load released with a residual deflection of 0.027 ft noted.

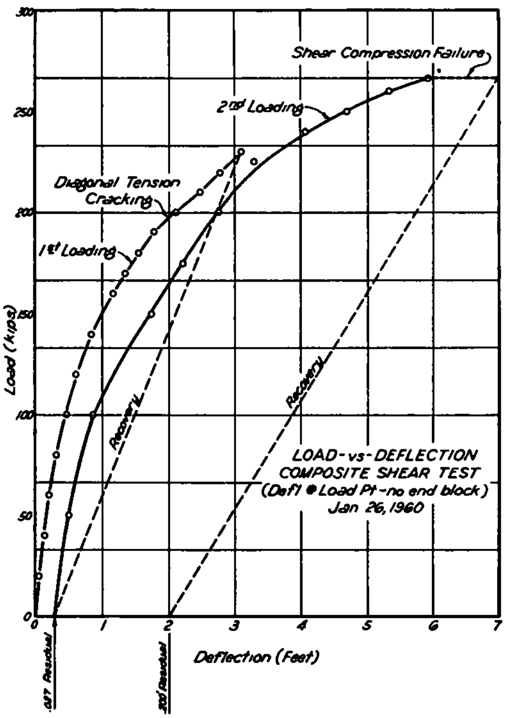

Figure 14. Load vs deflection, composite shear test.

Loading for the final cycle began with large increments of load being applied until a total load of 250 kips was placed on the section. Diagonal tension cracks turned in the lower flange and were parallel to the bottom of the beam. Slight cracking was noted in the lower surface of the slab. Deflection under the loading point had reached 0.590 ft. At 266.25 kips, continued deflection with no increase in load was noted. Loading was discontinued at this point. With this load maintained on the beam, a radiograph gamma ray exposure was made of the stirrups intersecting the most critical diagonal tension cracks. No evidence of fracture of the stirrups was noted. A final deflection of 0.700 ft was recorded.

Initial diagonal tension failure occurred at an equivalent design load of 1 dead load plus 6.54 live loads including impact.

Final inspection of the beam showed a longitudinal split in the bottom of the beam approximately 2 ft long and located 4½ ft from the loading point toward the near reaction: Figures 15 and 16 show the final cracking pattern of the beam, and Figure 17 shows the longitudinal and transverse cracking of the slab.

Test With End Block.—For the second test it was decided to measure strains in the slab and in the web region where diagonal tension failure was expected to occur. Figure 4 shows the location of the electrical strain gages used. Vertical and horizontal deflections were recorded at the center of bearings and at the loading point.

Load in 20 kip increments was applied until initial flexural cracking under the load point occurred at 160 kips. Compressive strains in the slab reached 275 μin. per in. with a vertical deflection of 0.129 ft. Figure 18 shows a plot of load vs deflection. Principle strain components of the web in the horizontal direction measured 276 μ in.

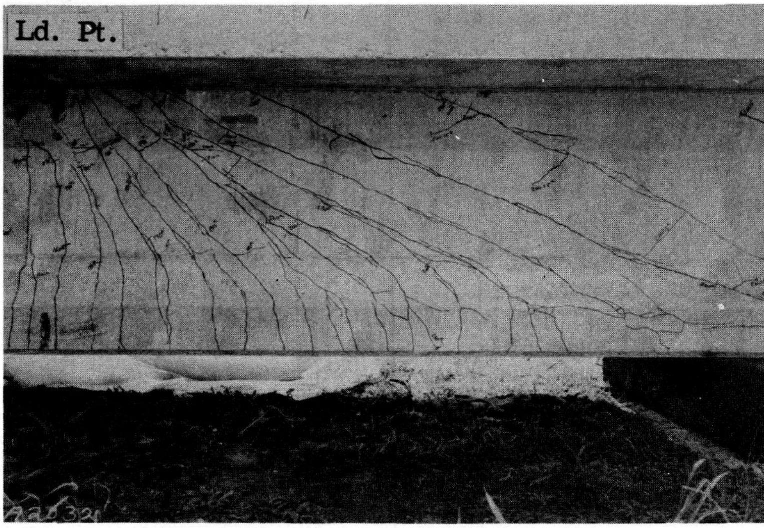

Figure 15. Cracking pattern, composite shear test, pretensioned beam.

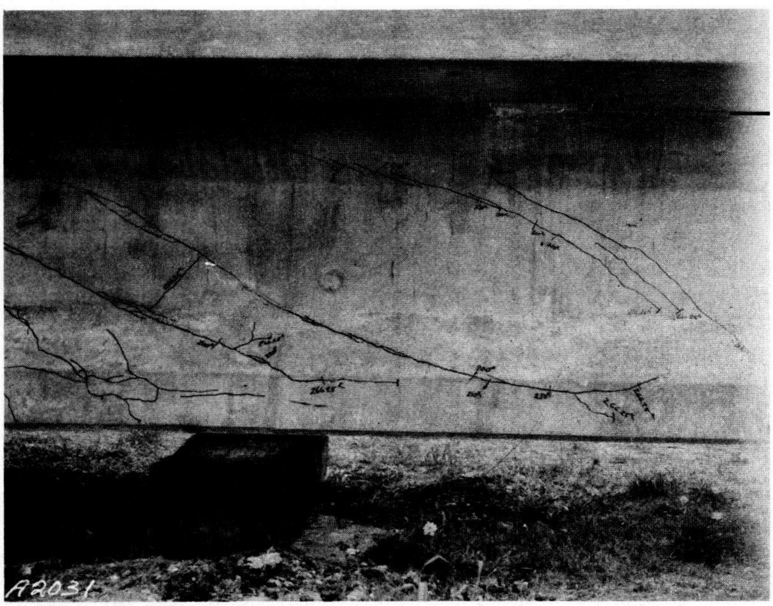

Figure 16. Cracking pattern, composite shear test, pretensioned beam.

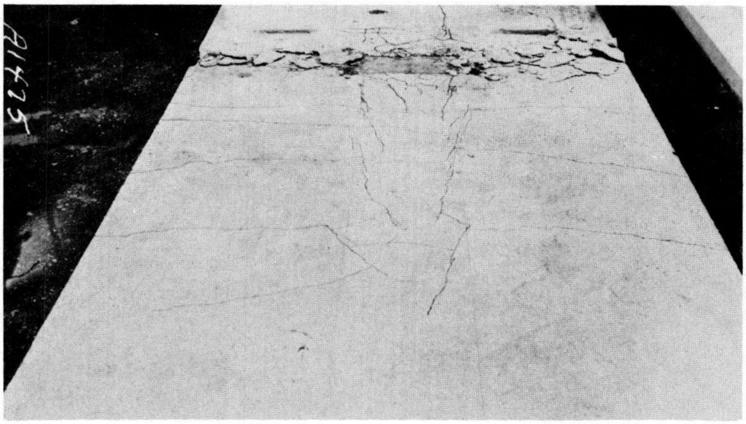

Figure 17. Slab-cracking pattern, composite shear test, pretensioned beam.

per in. with 66 μ in. per in. measured in the vertical direction. At 190 kips, compressive strains in the slab reached 491 μ in. per in. with a vertical deflection of 0.205 ft. Diagonal tension failure occurred before rosette strains in the web could be read. Flexural cracking had extended high into the upper flange of the beam. Three diagonal tension cracks occurred and were inclined at an angle of approximately 25° from the bottom of the beam. No cracks appeared above the bearing point as occurred in the test without the end block. Loading continued up to 240 kips where initial flexural cracking in the bottom of the slab was noted. Flexural cracking continued near the loading point with a small increase in length of the diagonal tension cracks. Loading continued to 260.45 kips where compression failure of the slab occurred. Deflection at this point reached 0.607 ft. Compressive strains in the slab had reached 1,801 μin. per in.

Initial diagonal tension cracks occurred at an equivalent design load of 1 dead load plus 4.56 live loads including impact. Shear compression failure occurred at 1 dead load plus 6.25 live loads, including impact. As in the previous test, longitudinal cracking in the bottom of the beam was noted. Figures 19 and 20 show the final cracking pattern of the beam, and Figure 21 shows longitudinal cracking of the slab.

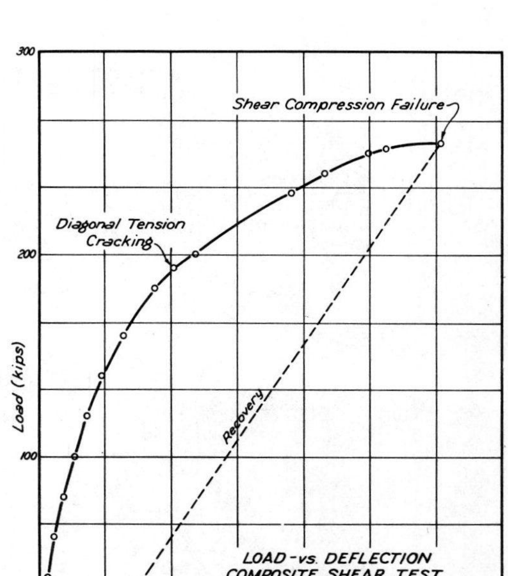

Figure 18. Load vs deflection, composite shear test, pretensioned beam.

Composite Moment Test (Post-Tensioned Beam)

This post-tensioned beam with non-bonded tendons was not in the original testing schedule. However, as the beam was made available at the time the other beams were being tested it was felt that worthwhile knowledge could be gained from testing it also. A 6½-in. thick, 6-ft wide, compositely acting slab was cast

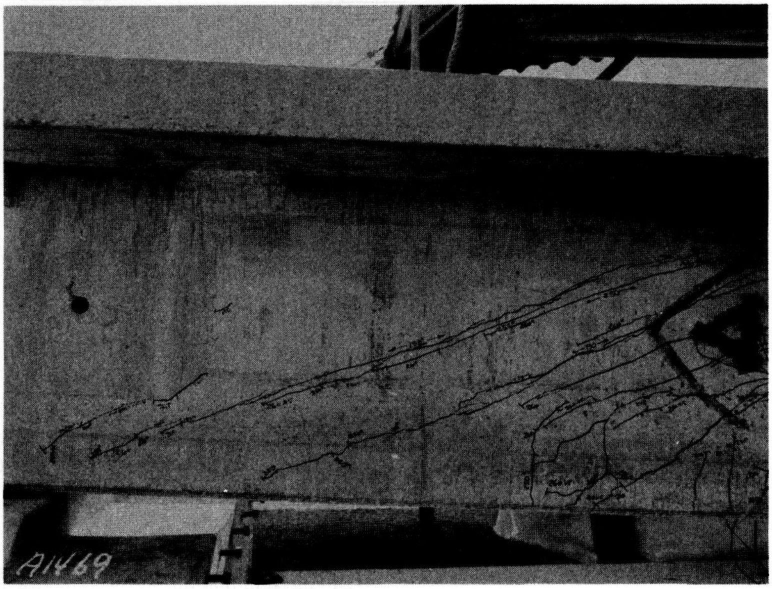

Figure 19. Cracking pattern, composite shear test, pretensioned beam.

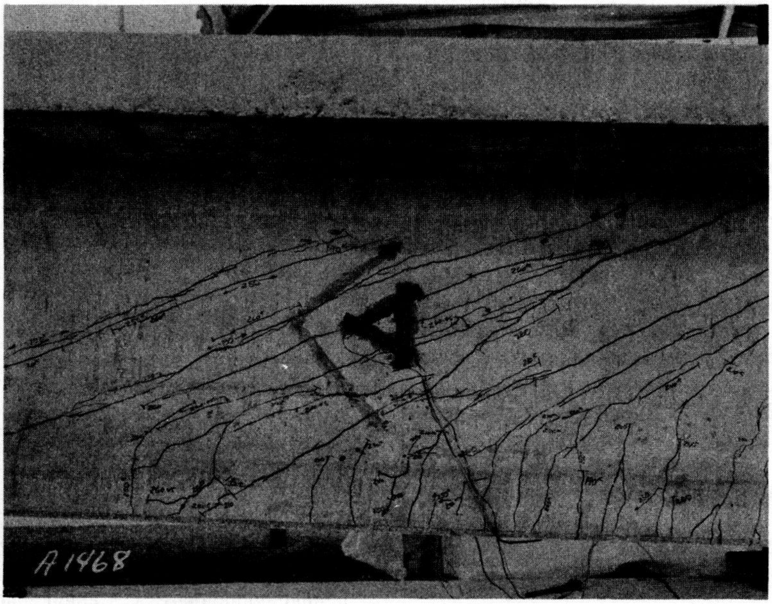

Figure 20. Cracking pattern, composite shear test, pretensioned beam.

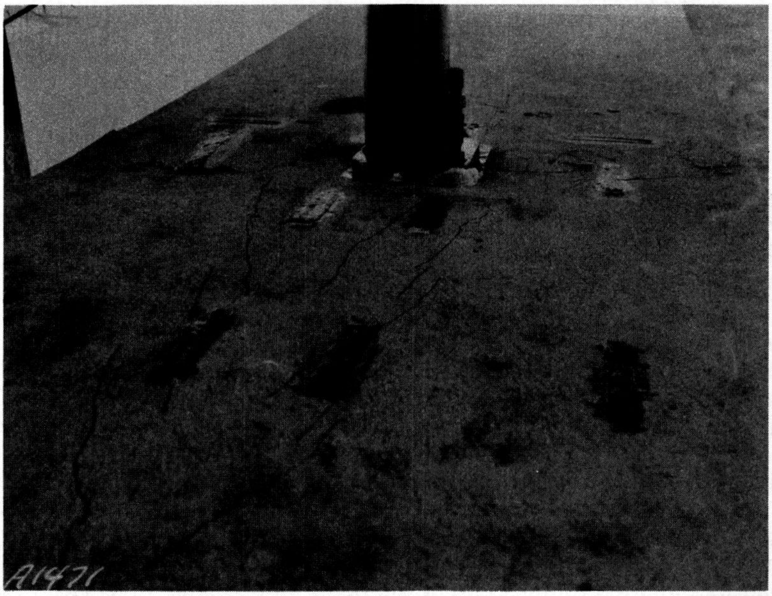

Figure 21. Slab-cracking pattern, composite shear test, pretensioned beam.

on the beam shown in Figure 2. Tendons for post-tensioning the beam were not grouted as is normally required. No strain measurements were made on this test. Over-all length of the beam was 59 ft 6½ in., with a center-to-center-of-bearing length equal to 58 ft 10½ in. Vertical and horizontal deflections were measured at bearing points, quarter points, and midspan. Load was applied at midspan and the load was brought up to 46 kips where initial flexural cracking ocurred. Figure 22 shows a plot of load vs deflection with a deflection of 0.025 ft at 46 kips. A single flexural crack extended up through the bottom flange into the web where it split into a Y-shape. Loading was continued in 10-kip increments up to 70 kips where first cracking on the underside of the slab was noted. Deflection had reached 0.417 ft. The single flexural crack through the bottom flange had opened to a width of approximately ⅜ in. No other flexural cracks through the bottom flange were initiated during the test.

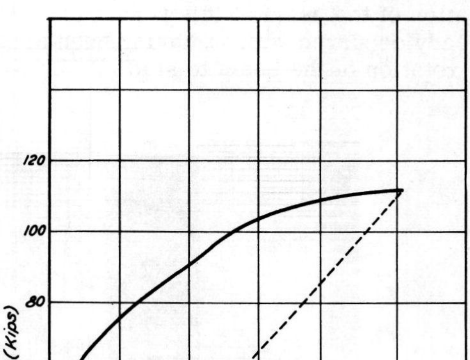

Figure 22. Load vs deflection, composite moment test, post-tensioned beam.

Loading continued to 80 kips where a network of cracks appeared on the underside of the slab with some cracks penetrating approximately 2½ to 3 in. into the slab. The initial flexural crack in the beam continued to widen. In the loading from 80 to 100 kips, no increase in length of cracks was noted. The flexural crack through the beam had opened to about 2½ in. at the bottom flange. Deflection increased rapidly with a small additional amount of load. At 100 kips, the deflection had reached 0.527 ft.

Between 100 and 112 kips, the deflection increased 0.608 ft to a total of 1.135 ft. In the loading from 100 to 110 kips, compressive failure in the top of the slab occurred with spalling of the slab approximately one-third the depth of the slab. Due to lack of jack travel and clearance between the beam bottom and the testing bed it was decided to discontinue loading of the beam. A quick estimate of the stresses in the tendons showed that with the maximum attainable deflection of 2 ft, only about 236,000 psi could be induced in the bottom tendons of the beam. Therefore, the load was released and the beam allowed to recover. After a short wait original deflection readings were checked and a residual deflection in the beam of 0.025 ft was noted. Figures 23 and 24 show the beam under the maximum loading.

Torsion Test

A standard pretensioned beam was scheduled to be tested in torsion as an aid in helping to establish maximum criteria for handling and hauling as well as supporting overhanging formwork and fluid concrete during superstructure construction. The beam used was 64 ft long with end blocks cast on each end. This beam had been cast for a bridge project, but was rejected due to excessive honeycombing at one end of the beam. Near one end of the beam five strands were visible in the bottom flange due to this condition, but in an effort to make use of every available test specimen, it was decided to use the beam as it was, realizing that failure probably would occur in this area.

No strain measurements were made. Only the angular rotation and load were recorded. Load was applied to each end of the beam in opposing directions through eccentrically loaded collars which encircled the beam and were bolted to tilting jigs mounted on concrete abutments. Length between centers of collars was 62 ft 2⅝ in. Figure 25 shows the beam in position for the test. Rotation was applied at both ends simultaneously in increments of $\frac{1}{2}^{\circ}$. Relatively soon after the test began, difficulties in obtaining accurate load readings in one pumping unit became apparent. Rotation of the beam was stopped and efforts were made to correct this difficulty. In this interim the constant torque applied to the beam caused initial cracking of the section. Angular rotation of the beam at this time was just under 2°. As the point of initial failure had already occurred without having been accurately recorded, it was decided to continue the rotation of the beam to study its behavior visually. The failure of the beam had been

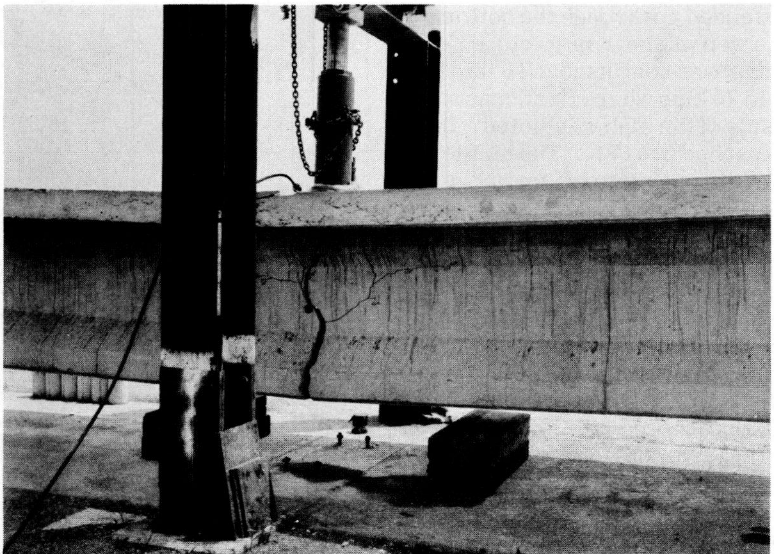

Figure 23. Post-tensioned beam under maximum load.

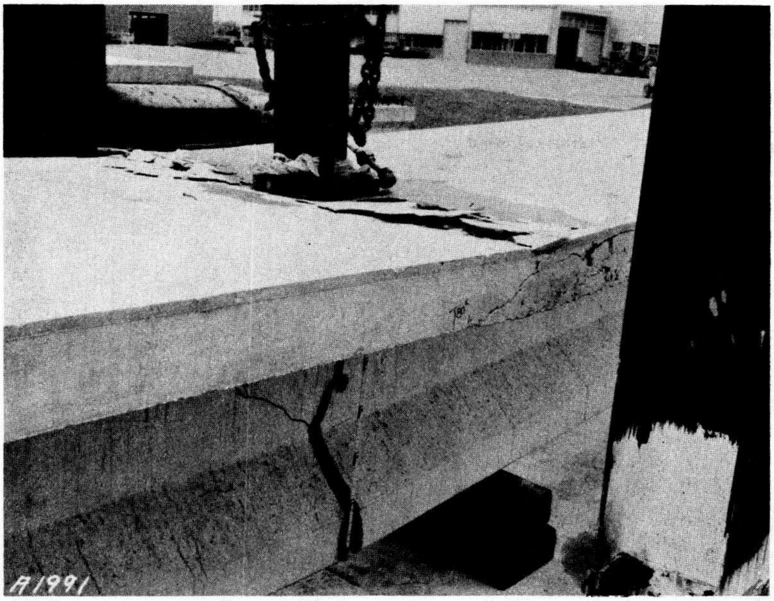

Figure 24. Post-tensioned beam under maximum load.

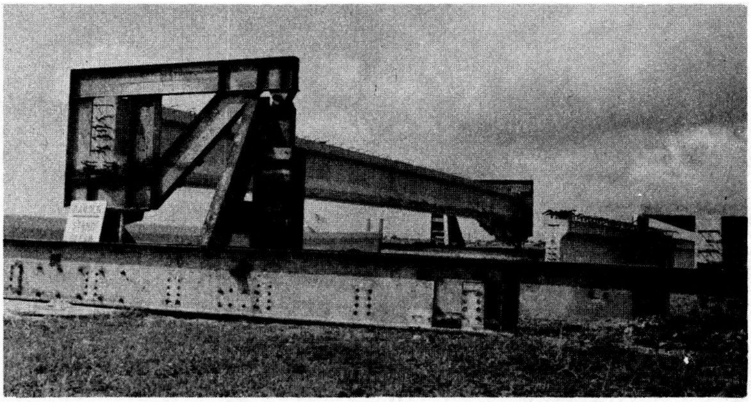

Figure 25. Pretensioned beam in position for torsion test.

confined to one end of the span and a thorough inspection of the remainder showed no outward signs of damage. It was then decided to cut off the damaged section and perform a new test on the intact section of the beam. The undamaged portion of the beam was about 50 ft long. For the second test the distance between centers of collars was 48 ft 5 in. The jacking systems were checked out and pressure gages permitting more accurate load readings were installed.

For the second test it was decided to rotate one end of the beam at a time and then only in increments of $\frac{1}{4}^{\circ}$. Again, only loads and angle of rotation were recorded. To coordinate the jacking operations, portable walkie-talkies were used to inform the men working the pumping units of the rate of rotation required for each increment of loading.

Torsion was applied to the beam and when the total angle of rotation reached 2° the first cracks were initiated. A total of 229.60 ft-kips of resisting moment was produced in the beam. Initial cracking of the beam occurred near the south end of the beam and

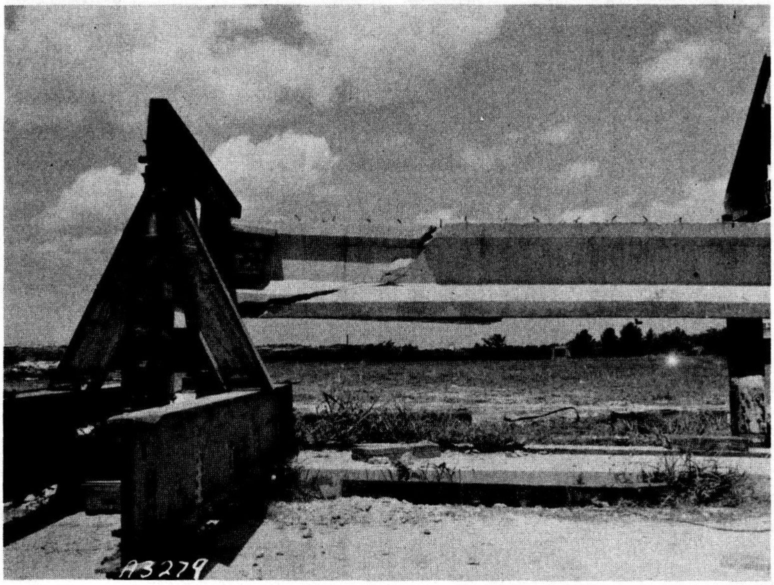

Figure 26. Beam failure, torsion test, pretensioned beam.

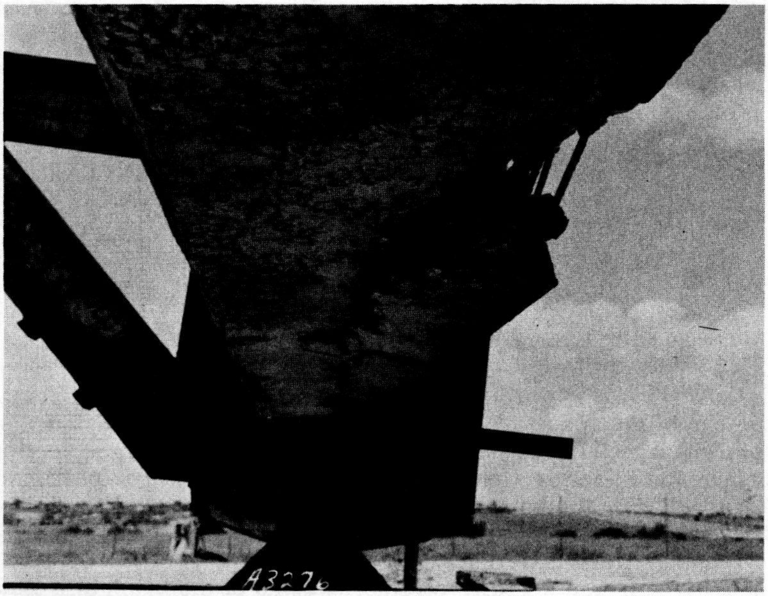

Figure 27. Beam failure, torsion test, pretensioned beam.

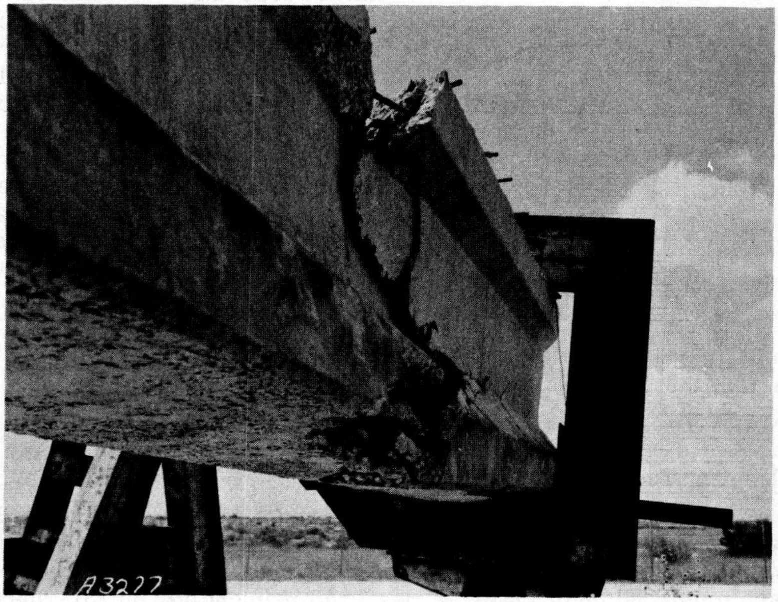

Figure 28. Beam failure, torsion test, pretensioned beam.

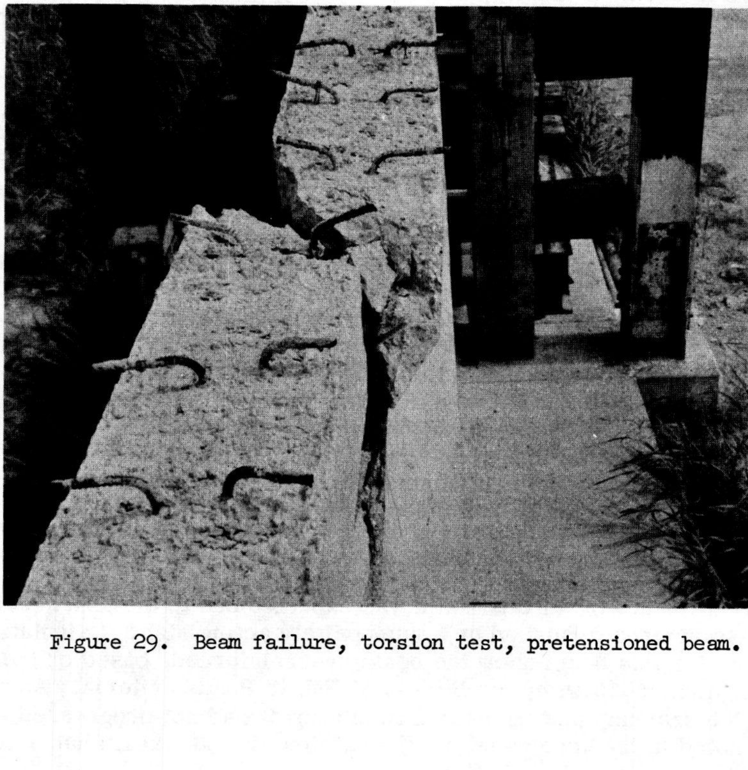

Figure 29. Beam failure, torsion test, pretensioned beam.

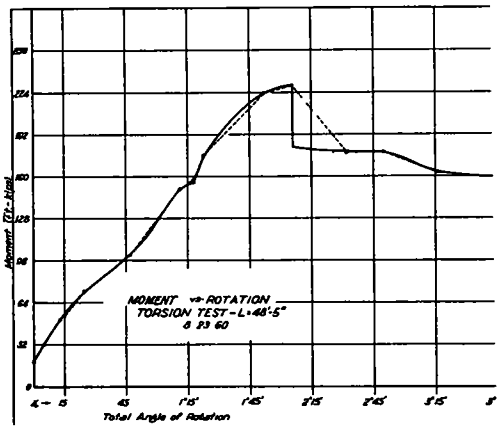

Figure 30. Resisting moment vs angle of rotation, torsion test, pretensioned beam.

appeared suddenly in the top and bottom flanges and inclined through the web of the beam. Resisting moment was reduced at initial cracking and continued to drop off as the angle of rotation was increased. Diagonal tension cracks appeared following the direction of principal tensile strains on either side of the beam. As rotation increased the main plane of failure widened with some spalling in the general area. No failure of stirrups was in evidence as cracking seemed to follow a plane along the side of the area enclosed by the stirrups. When a total of 12° of rotation was reached the test was stopped. Figures 26 through 29 show final cracking of the beam. Figure 30 is a partial plot of total applied moment vs angle of rotation.

ANALYSIS

Composite Moment Test (Pretensioned Beams)

Working as a composite section this beam showed a capacity equal to 1.25 times the required minimum ultimate moment produced by 1.5 DL + 2.5 (LL+I) as set forth by tentative AASHO specifications (3). Calculated ultimate moment based on a flanged section was 2,669 ft-kips as compared to a measured moment of 2,907 ft-kips or 1.09 times greater than the calculated value. Considering distribution to adjacent sections of a bridge it appears that the beam in actual service would be capable of an even higher ultimate capacity than that shown by the tests. Complete failure of the section could be safely assumed to occur as anticipated in that yielding of the prestressing steel would lead to collapse of the section. This region of strand yielding was barely reached in the test as was shown by the small residual deflection of 0.078 ft left in the beam after removal of the load. However, after the load had been released, a load equal to the design load was reapplied to the beam and the resulting additional deflection, above and beyond the residual deflection, was still within the allowable deflection of $1/800$ of the span length. Therefore, the beam, even in its state of having been damaged almost up to ultimate failure could have passed traffic over the bridge safely without collapsing.

Initial cracking of the beam, if deemed to be a point of control, may be estimated although never definitely fixed due to constant changing of the modulus of elasticity through time and loading. Stresses in the bottom of the beam due only to prestressing and dead loads are added to unit stresses calculated by taking the modulus of rupture in bending as $7.5\sqrt{f'c}$ to provide a stress at which first cracking should appear. In this case the estimated stress was about 687 psi or 275 μ in. per in. of strain as compared to 235 μ in. per in. of strain measured.

From the test results it can be seen that the design criteria used for this beam is more than adequate for the design situation produced by the beam span and spacing used. Possibly more accurate solutions of the calculated results could be obtained by determining equivalent areas for the slab concrete in respect to beam concrete, and then calculating stresses using the transformed section method.

Noncomposite Moment Test (Pretensioned Beam)

The compressive failure of this beam was expected due to the absence of an additional compressive area furnished by a compositely acting slab. Calculations of percentage of steel for this beam show the beam overreinforced, based on balanced beam steel percentage as set forth by the Bureau of Public Roads criteria. As noted in the test results, the cracking pattern near ultimate loads had not progressed as high in the beam as was noted in the underreinforced condition when the beam had a composite

slab acting integrally with the beam. Near the ultimate capacity of the beam the cracks ceased to extend themselves and finally a load was reached at which the strength of the concrete in the compressive zone was no longer strong enough to counterbalance the tensile forces acting in the steel. At this point the flexural capacity of the beam was reached and further loading resulted in compressive failure of the concrete.

Measured moment at failure was found to be 2,305 ft-kips as compared to a calculated moment of 2,126 ft-kips.

Because this beam was designed as a component part of a bridge acting integrally with the slab, this type of loading and failure would not be probable.

Composite Shear Tests (Pretensioned Beam)

Initial cracking in the lower load ranges was due to flexural stresses under the loading point. As very little shear is present directly under the loading point these initial cracks progressed vertically up into the beam. As flexural cracks appeared on either side of the loading point, the influence of shear stresses may be seen as the cracks start vertically in the flange and then begin to bend over, toward the loading point, in the web where shear stresses become larger. Within reasonable limits of loading these cracks in the beam will close if the load is released. In contrast to flexural cracks, the diagonal tension cracks appeared in the web suddenly and traversed the web quickly due to the presence of high shearing stresses. As these diagonal cracks reached the bottom flange they turned to run almost parallel to the bottom of the beam where the largest mass of prestressing steel was located. Removal of the load did not allow these cracks to close again as happened with the flexural cracks.

In both shear tests more than one initial diagonal crack appeared at the same time. The crack nearest to the load point should have been subjected to somewhat greater stresses than the others, but hardly enough to state that one crack was initiated slightly before the other. With continued loading flexural cracks occurred near the point where the diagonal cracks entered the bottom flange of the beam. As these new cracks progressed into the web where large shear stresses were present they quickly traversed the web and extended up into the top flange following a slope close to that made by diagonal cracks

All of the flexural and diagonal cracks progressed toward the loading point on top of the slab. In the latter stages of loading, flexural cracking had appeared on the underside of the slab. All of this cracking tended to reduce the available working compressive area of the section and, as in the noncomposite moment test, finally reached a point where the compressive forces could no longer balance the tensile forces, thus causing compressive failure in the slab.

Apparently the lack of an end block did not affect the structural capacities of the beam. Initial flexural, diagonal tension, and compression failure loads were nearly equal in both tests. From this it could be surmised that the exclusion of the end block would be permissible with the exception of those cases where higher stresses encountered at the ends of the beam might tend to cause splitting.

Composite Moment Test (Post-Tensioned Beam)

By not bonding the stressing tendons, these tendons were allowed to distribute the induced elongation throughout the entire length of the stressing steel. This elongation of the tendon essentially let the beam act as a nonreinforced member once the prestressing forces have been overcome. Only the reinforcing steel in the top of the beam flange and in the slab was available for any work and that was contained in the compressive zone of the section. Therefore, initial cracking of this "nonreinforced member" occurred under the loading point where the highest stresses in the concrete occurred. After the initial cracking of the beam no other flexural cracks were found in the bottom of the beam. This initial crack could only progress higher up into the beam and become wider as the load was increased because no transfer of stresses from the steel to the concrete was possible. Again compression failure in the slab occurred when the flexural capacity of the beam was exceeded.

Total deflection of this beam under load far surpassed that recorded for the preten-

sioned beam, with the residual deflection in the beam, after unloading, being approximately one-third as great.

Visual cracking of the beam occurred at 45 kips or 1DL + 1.1 (LL + I) as compared to 78 kips or 1DL + 1.87 (LL + I) for the composite moment test of the bonded pretensioned beam.

Torsional Tests

When subjected to torsional forces, prestressed concrete, like plain concrete, will fail when the ultimate tensile strength of the material is reached. The torsional strength of concrete is low, but failure may be delayed by the addition of web reinforcement that is placed to intercept the potential failure cracks and by imposing a compressive force on the beam. Although the web reinforcement may only add strength to a small degree, the addition of the prestressing force will greatly increase the torsional strength of a concrete member.

Due to the presence and spacing of vertical reinforcement the beam did not fail with the explosive type of failure with flying debris that is characteristic of nonreinforced prestressed concrete. Initial cracking was not extensive enough to cause the beam to be unable to support itself. Only after extensive damage had occurred did the beam sag down to rest on timber staging placed to arrest complete collapse of the beam.

Although the resisting moment of the beam was as adequate as expected, the angle of rotation before initial cracking occurred was smaller than anticipated. Initial cracking for both tests occurred at approximately the same total rotation and torsional moment, which does not follow the theory that torsional stiffness is a function of length. This result could have been caused by the original tested length being weak due to the excessive honeycombing in one end of the beam, and further, gage readings for the first test were not very reliable. Clearly this test shows the need for additional experimentation in the torsional behavior of I-section prestressed beams.

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