In bridge design, the advantages of using continuous spans over a series of simple spans are well understood. Continuous systems in composite construction are often achieved by joining prefabricated members spanning between piers with a cast-in-place reinforced concrete deck providing inter-span continuity.

Earlier studies\(^1,2\) have established the feasibility of using long slender prestressed concrete prisms (tension elements) in place of conventional deformed bars to develop continuity.

More recently, using specimens containing a single prestressed concrete tension element, the authors\(^3\) have established that the fatigue strength of such specimens under repeated loading was \(0.7 \, P_{cr}\), where \(P_{cr}\) is the cracking load of the tension element.

Investigation by Bishara, Mason, and
Synopsis

This paper describes the static and fatigue behavior of composite T-beams using a combination of prestressed concrete tension elements and reinforcing bars as the tension reinforcement.

Static tests indicated similar behavior of two continuous beams, one being designed with moment redistribution at ultimate load and the other without moment redistribution. Repeated load tests were performed on six composite beams.

Test results indicated that by augmenting multiple tension elements with steel reinforcing bars, the endurance limit of the composite beams was increased by nearly 43 percent. It was also shown that, under repeated load tests, the maximum crack width of the beam exclusively reinforced with tension elements was approximately one-half of that of the similar beam reinforced with conventional bars only.

Almeida revealed that for a given moment capacity, the use of a combination of prestressed concrete tension elements and conventional reinforcing bars (in the ratio of 3 to 1) provided optimum control of visible cracking and increased overall flexural rigidity of reinforced concrete beams.

A similar study carried out in Japan also indicated that composite beams using prestressed concrete tension elements as reinforcement behaved almost like fully prestressed concrete beams, as opposed to conventional reinforced concrete beams, insofar as crack control and deflection characteristics were concerned.

In 1969, a three-span continuous bridge over a railway was constructed in Burnaby, British Columbia, using precast prestressed concrete "rods" as continuity and crack control reinforce-
Table 1. Scope of test program.

<table>
<thead>
<tr>
<th>Test Series</th>
<th>Specimen Number</th>
<th>Specimen Reinforcement</th>
<th>Tension Element Used*</th>
<th>Date of Casting</th>
<th>Test Setup</th>
<th>Type of Loading</th>
<th>M**&lt;sub&gt;applied&lt;/sub&gt;</th>
<th>Date of Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>A1</td>
<td>Two #5 and three tension elements 7/16-in. strand</td>
<td>II</td>
<td>2/1/73</td>
<td>Continuous</td>
<td>Static</td>
<td>0.00</td>
<td>5/10/73</td>
</tr>
<tr>
<td></td>
<td>A2</td>
<td></td>
<td>II</td>
<td>4/5/73</td>
<td>Simple</td>
<td>Repeated</td>
<td>1.63</td>
<td>10/10/73</td>
</tr>
<tr>
<td></td>
<td>A3</td>
<td></td>
<td>V</td>
<td>5/25/73</td>
<td>Simple</td>
<td>Repeated</td>
<td>0.95</td>
<td>10/24/73</td>
</tr>
<tr>
<td></td>
<td>A4</td>
<td></td>
<td>V</td>
<td>6/15/73</td>
<td>Simple</td>
<td>Repeated</td>
<td>1.25</td>
<td>11/9/73</td>
</tr>
<tr>
<td>B</td>
<td>B1</td>
<td>Two #5 and three tension elements 3/8-in. strand</td>
<td>III</td>
<td>2/1/73</td>
<td>Continuous</td>
<td>Static</td>
<td>0.00</td>
<td>5/16/73</td>
</tr>
<tr>
<td></td>
<td>B2</td>
<td></td>
<td>III</td>
<td>4/5/73</td>
<td>Continuous</td>
<td>Repeated</td>
<td>0.77</td>
<td>7/13/73</td>
</tr>
<tr>
<td></td>
<td>B3</td>
<td></td>
<td>IV</td>
<td>5/25/73</td>
<td>Continuous</td>
<td>Repeated</td>
<td>1.07</td>
<td>8/31/73</td>
</tr>
<tr>
<td></td>
<td>B4</td>
<td></td>
<td>IV</td>
<td>6/15/73</td>
<td>Simple</td>
<td>Repeated</td>
<td>1.11</td>
<td>11/16/73</td>
</tr>
<tr>
<td>C</td>
<td>C1</td>
<td>Two tension elements 3/8-in. and two tension elements 7/16-in. strand</td>
<td>I</td>
<td>12/18/73</td>
<td>Simple</td>
<td>Repeated</td>
<td>0.85</td>
<td>1/11/74</td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>Two #5 and two #7</td>
<td>--</td>
<td>12/18/73</td>
<td>Simple</td>
<td>Repeated</td>
<td>0.85</td>
<td>2/15/74</td>
</tr>
</tbody>
</table>

*See Table 2.

**M**<sub>applied</sub> = Maximum applied moment under repeated load, including dead load effect.

M<sub>cr</sub> = Theoretical moment corresponding to cracking of tension element.

Each of the 30-ft long rods consisted of a single 1/2-in. strand tensioned to 25,000 lbs and encased in a 3 x 2½ in., 6000-psi concrete rectangle. A compression of 3000 psi was developed in the "rod."

As intended, the rods tended to close up any tension cracks in the deck slab due to live loadings, and the deck showed no signs of tension cracking in the negative moment areas.

**Objective**

This study is an extension of the previous work<sup>3</sup> to examine serviceability and post-cracking behavior under repeated cyclic loads of both simple and continuous members using combinations of tension elements and reinforcing bars.

Specifically, the study dealt with the cracking behavior, endurance limit, ultimate load capacity and amount of moment redistribution of continuous composite beams designed with and without 20 percent moment redistribution.

**Scope**

Ten specimens, classified into three series, were tested under two different types of loading as indicated in Table 1. Each specimen was designed as a two-span continuous composite T-beam.

Specimens of Series A were designed without consideration of moment redistribution at ultimate load, while those of Series B were designed with an assumed 20 percent moment redistribution.

Specimens of Series C were designed for an equal ultimate moment capacity as those of Series B, but used two different types of continuity reinforcement.

Specimen C1 was reinforced with tension elements exclusively, while Specimen C2 was reinforced with only reinforcing bars, so as to provide a comparison of the effectiveness between the two types of reinforcement for crack control.

In addition to the ten specimens for static and repeated load tests, two long
prestressed tension elements were fabricated to demonstrate the practicality of handling, transporting, and sectioning of the long elements. A description of these various aspects is presented in the Appendix.

**Experimental Program**

**Test specimens**

Each of the ten specimens was designed as a two-span continuous composite T-beam. The composite beams were fabricated from two precast prestressed concrete 8 x 10-in. stems with continuity over the central support being developed through a combination of tension elements and conventional reinforcing bars in a 5 x 42-in. flange, as shown in Fig. 1(a).

In addition to the natural bond between the stem and the flange, positive shear transfer was achieved through use of #3 bars at 7.5-in. spacing, extending out of the stem into the flange.

A 5-in. thick diaphragm reinforced with #4 bars was cast at the central support section connecting the two precast prestressed stems also shown in Fig. 1(b).

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**Fig. 1(a). Cross section of composite T-beam.**

**Fig. 1(b). Test set-ups.**
Fabrication of prestressed stems

Details of the precast prestressed concrete stem are shown in Fig. 2. The beam stems were fabricated by Arnold Stone Company, Greensboro, North Carolina, in a 220-ft prestressing bed.

Continuous wooden forms were used with 8 x 10-in. end blocks at 15-ft intervals. The 1/2-in. strands were initially tensioned to 28,000 lbs per tendon.

The properties of the strand as furnished by the manufacturer are as follows:

- Yield strength at 1 percent extension: 239 ksi
- Ultimate strength: 270 ksi
- Modulus of elasticity, $E_g$: 29,000 ksi

Normal weight concrete was used for the stems. The concrete mix proportions per cubic yard were as follows:

- Cement (Type III): 705 lbs
- Sand: 1254 lbs
- Coarse aggregate: 1630 lbs
- Water: 37 1/2 gal

The fine aggregate was well graded sand with a fineness modulus of 3.0. The concrete was vibrated externally and the top surface rough finished by wooden float. After normal air drying for 4 hrs, the castings were covered with heavy duty plastic sheets and cured by steam with temperature increased at the rate of 0.5 deg F/min up to 150 deg and held for approximately 8 hrs before being turned off.

Forms were removed and prestress transferred about 45 hrs after casting, when the concrete strength had reached at least 5000 psi. However, the total curing time for Casting I (see Table 2) was only 20 hrs, which is the normal production cycle of the prestressing plant.

Fabrication of tension elements

Tension elements were of 2 x 3-in. rectangular section with a single 7-wire ten-
Two types were made using 7/16-in. and 3/8-in. diameter strand tensioned initially to 21,700 and 16,100 lbs, respectively.

The tension elements were manufactured simultaneously with the precast stems using the same concrete mix, casting and curing procedures. A continuous wooden form was used with 2 x 3-in. end blocks at 12-ft intervals. Details are shown in Fig. 2.

Tendon force was determined by gage pressure of the hydraulic system and checked by strand elongation. Properties of the strands were the same as given before for the stem.

In addition to 12-ft sections, two specimens, 30 and 40 ft each, were cast to investigate lifting, handling and sectioning (with saw) techniques.

For each of the first five castings listed in Table 2, eighteen 6-in. cylinders were cast and cured under the same condition as the prestressed stems and tension elements. The cylinders were tested at 3 days, 7 days, 28 days, 3 months, 6 months, and 1 year. Fig. 3 shows the variation of concrete strength with age.

It is noted that after 6 months curing, the concrete virtually reached its potential strength. However, the concrete strength of the first casting was considerably lower than those of other castings, due to the shorter initial curing period as noted before.

**Table 2. Casting schedule.**

<table>
<thead>
<tr>
<th>Designation of Casting</th>
<th>Date of Casting</th>
<th>No. of 8 x 10 in. Stem</th>
<th>No. of Tension Elements 2 in. x 3 in. 12 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>9/13/72</td>
<td>8</td>
<td>6 (7/16 in. strand)</td>
</tr>
<tr>
<td>II</td>
<td>9/30/72</td>
<td>8</td>
<td>6 (7/16 in. strand)</td>
</tr>
<tr>
<td>III</td>
<td>10/5/72</td>
<td>6</td>
<td>6 (3/8 in. strand)</td>
</tr>
<tr>
<td>IV</td>
<td>10/10/72</td>
<td>---</td>
<td>6 (3/8 in. strand)</td>
</tr>
<tr>
<td>V</td>
<td>10/15/72</td>
<td>---</td>
<td>7 (7/16 in. strand)</td>
</tr>
<tr>
<td>VI</td>
<td>10/20/72</td>
<td>---</td>
<td>4 (3/8 in. strand)</td>
</tr>
<tr>
<td>VII</td>
<td>10/25/72</td>
<td>---</td>
<td>2 (7/16 in. strand)*</td>
</tr>
</tbody>
</table>

*One 30 ft long and one 40 ft long.

**Fig. 3. Compressive strength of concrete used for tension elements and stems.**

**Fabrication of composite beams**

Three series of beams were cast. All were composite T-beams consisting of precast stems and laboratory cast flanges. Following erection of formwork, three tension elements and two #5 Grade reinforcing bars were positioned and secured with centroid 2½ in. below the top of the flange.

U-stirrups of #3 bars, 7½ in. on centers, extended from the stem into the flange. Eight L-shaped rods with threaded end were appropriately spaced and secured in pairs with threaded end protruding out
of the flange to provide a means for lifting and moving.

In each beam, two SR-4 strain gages were attached to the reinforcing bars at a location near the central support. Lead wires were soldered and waterproofing compound applied to encase and protect the strain gage.

To determine periodically the total loss of prestress in the tension element, one tension element from each casting, except Castings VI and VII, was instrumented with mechanical strain gage points, and strain measurement was made at different time intervals.

Ingredients per cubic yard of ready mixed concrete used in the flange were:

- Cement ............ 611 lbs
- Sand ............ 1850 lbs
- Gravel ............ 1107 lbs
- Water ............ 34 gallons
- Admixture .......... 4 oz

The concrete was vibrated internally with a needle vibrator and trowel finished at the surface. Care was exercised not to disturb the position of the tension elements during this process.

Six hrs after casting, the concrete was covered with a heavy duty plastic sheet and kept moist for 15 days. Forms were removed after the concrete had cured for 21 days.

Eight concrete cylinders were prepared simultaneously with the beams to monitor compressive strength of the concrete. The cylinders were cured and tested under conditions identical to those of the beams.

**Test procedure**

**Static test**—The setup for the static test is shown in Figs. 1(b) and 4. Load was applied by a pair of 60-ton capacity hydraulic jacks located at 7.65 ft on each side of the central support, which rested on a semi-spherical head.

The two end supports were on roller bearings. One 100-kip and two 25-kip capacity load cells were used to measure the support reactions at the central and end supports, respectively.

The strain in the reinforcing bars was measured by a strain indicator through a switching-balancing unit. The midspan deflections of the continuous beam were measured by two mechanical dial gages, reading to the nearest 0.001 in. After each load increment, the width of the major crack was measured by a 50X power micrometer microscope.

Following preliminary checks for proper seating of the specimen and operation of the instrumentation, the load was applied in increments of 10 kips up to failure; and corresponding deflections, strain gage readings, load cell readings, and crack widths were recorded. In addition, the development and location of the cracks were traced.

**Repeated load tests**—Repeated cyclic load was applied by a 50-kip capacity ram of a hydro-electronic closed-loop testing
system as shown in Fig. 5.

For Specimens B2 and B3, which were tested as continuous beams, a steel beam W 12 x 81 was used to distribute the ram load to the specimen at 7½ ft on each side of the central support as shown in Fig. 1(b).

The load was applied at a frequency of 1 cycle per sec. Table 1 shows the maximum applied moment for each specimen. The minimum applied moment in each case was 2.8 ft-kips for the continuous beam and 2.5 ft-kips for the simple beam. A limit switch was mounted on the loading ram to prevent any accidental overload and to control the maximum stroke of the ram.

The repeated loading was periodically stopped in order to conduct intermediate static tests to determine the behavior of the specimen which included measurements of strains, crack widths, deflections and reactions. The maximum static load was limited to the peak magnitude of the repeated load.

At the conclusion of the repeated loading, if the specimen had not suffered a fatigue failure as evidenced by a virtually complete loss of stiffness, it was tested to destruction under static load. For each of the static tests, the applied load and the support reactions were plotted by three X-Y recorders.

Likewise, the load-strain curves were plotted by an X-Y recorder. The midspan deflections were also measured by dial gages as described previously.

For all other specimens, the above test procedures were followed, except that the specimens were tested as simple beams of inverted T-section with the hydraulic ram applying a single concentrated load at the midspan, as shown in Fig. 1(b).

Test Results

Loss of prestress

As noted previously, one tension element from each of the first five castings listed in Table 2 was selected as control specimen to determine the total loss of prestress due to elastic shortening, shrinkage and creep. Three sets of strain gage points were embedded in the concrete, one at 2 ft from each end and one at midpoint of the tension element.

A typical plot of change in strain as a function of time is shown in Fig. 6. It can be seen that the loss of prestress occurred mostly in the first 100 days after casting.

Based on the average of three strain

<table>
<thead>
<tr>
<th>Casting</th>
<th>( f'_e ) (psi) at transfer</th>
<th>Strand Force (lbs)</th>
<th>Percent Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>4000</td>
<td>21,700</td>
<td>9795</td>
</tr>
<tr>
<td>II</td>
<td>5000</td>
<td>21,700</td>
<td>9500</td>
</tr>
<tr>
<td>III</td>
<td>5000</td>
<td>16,100</td>
<td>5710</td>
</tr>
<tr>
<td>IV</td>
<td>5000</td>
<td>16,100</td>
<td>4540</td>
</tr>
<tr>
<td>V</td>
<td>5600</td>
<td>21,700</td>
<td>7056</td>
</tr>
</tbody>
</table>
measurements, the total loss of prestress was calculated for each group of specimens and are summarized in Table 3.

It should be noted that the total loss of prestress is much higher than the value ordinarily encountered in prestressed concrete designs. This was expected since the tension element was subjected to a fairly high initial prestress.

**Theoretical cracking and ultimate strengths**

When the composite beam is subjected to bending, initial cracking would develop in the concrete surrounding the tension elements and reinforcing bars when the flexural tension exceeds the rupture modulus of the concrete.

Under increasing bending moment,

<table>
<thead>
<tr>
<th>Test Series</th>
<th>Specimen Number</th>
<th>Cracking Moment, ft-kips</th>
<th>Ultimate Moment, ft-kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>A1</td>
<td>55</td>
<td>114.2</td>
</tr>
<tr>
<td></td>
<td>A2</td>
<td>55</td>
<td>116.7</td>
</tr>
<tr>
<td></td>
<td>A3</td>
<td>63.3</td>
<td>115</td>
</tr>
<tr>
<td></td>
<td>A4</td>
<td>63.3</td>
<td>115.2</td>
</tr>
<tr>
<td>B</td>
<td>B1</td>
<td>47.3</td>
<td>93.6</td>
</tr>
<tr>
<td></td>
<td>B2</td>
<td>50.8</td>
<td>95.2</td>
</tr>
<tr>
<td></td>
<td>B3</td>
<td>47.3</td>
<td>94</td>
</tr>
<tr>
<td></td>
<td>B4</td>
<td>50.8</td>
<td>94.3</td>
</tr>
<tr>
<td>C</td>
<td>C1</td>
<td>54.3</td>
<td>94.8</td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>42.4</td>
<td>86.3</td>
</tr>
</tbody>
</table>
the precompression in the tension element is gradually reduced and, finally, cracking of the tension elements would occur when the tensile strength of the concrete for the tension element is exceeded. The response of the composite beam to bending after cracking is therefore similar to that of a conventional reinforced concrete beam.

To determine the moment causing cracking in the tension element, the elastic theory with transformed cracked section can be used taking into account the loss of prestress and the concrete tensile strength of the tension element.

Likewise, the ultimate strength of a composite section containing tension elements can be determined according to the well known ultimate flexure strength theory. Detailed analyses have been presented elsewhere, and the results are summarized in Table 4.

**Moment redistribution**

Specimens A1 and B1 were tested under static load up to failure as two-span continuous beams. Measured reactions corresponding to the applied load are plotted in Figs. 7 and 8. The deviation of the measured reaction from the theoretical value according to the elastic theory indicates clearly the phenomenon of moment redistribution.

Using the average value of the two measured end reactions, the negative moment at the central support was computed from statics and plotted against the applied load in Figs. 9 and 10. These load-moment relationships show clearly the progressive increase in the moment redistribution as the load was increased.

At the instant of cracking of the tension element, the computed cracking moment at the central support based on the measured reaction was 56.5 and 45 ft-kips for Specimens A1 and B1, respectively.

These values compare closely with the theoretical results given above. When failure occurred, there was a maximum moment redistribution of 34 percent for A1 and 36 percent for B1.

It should be noted that the moment

---

**Fig. 9. Redistribution of negative moment in Specimen A1.**

**Fig. 10. Redistribution of negative moment in Specimen B1.**
capacity at the central support for B1 was, by design, roughly 20 percent less than that for Specimen A1. However, the prestressed stem provided an excess moment capacity of the midspan section. Accordingly, a larger amount of moment redistribution was obtained for both specimens.

The computed load carrying capacity of Specimen A1 was 117.5 kips, assuming the ultimate moment of the section was fully developed at midspan as well as at the central support. This was slightly less than the actual failure load of 124.6 kips carried by Specimen A1. Similarly, the predicted load carrying capacity of Specimen B1 was 109 kips as compared to its actual failure load of 117.4 kips.

Both Specimens B2 and B3 were subjected to a repeated load of 37 kips and 50 kips, respectively. According to the elastic theory, the maximum moment at the central support due to the applied load plus the dead load would be 60.5 ft-kips for B2 and 78.8 ft-kips for B3. However, Figs. 11 and 12 clearly show the effect of moment redistribution.

Measured end reactions progressively increased in magnitude after the beams were subjected to an increasing number of cycles of repeated load. Because of the moment redistribution, the actual applied moment at the central support (including the dead load effect) was 39.5 and 50.4 ft-kips for Specimen B2 and B3, respectively. These values are only 0.77 and 1.07 times the respective theoretical cracking moment of the section for Specimens B2 and B3 discussed before.

**Cracking behavior**

The location and growth of cracks were monitored during most tests. In the static tests, as increasing load was applied, a transverse crack developed on the tension face on one side of the diaphragm. Other cracks were observed to develop as loading progressed; however, the first one was the widest and was graphed as a function of load for Specimens A2 and B1 as shown in Fig. 13.

The use of higher tensioned elements in Specimen A2 should account for the lesser rate of increase in crack width compared to Specimen B1. The departure from linearity of the curve would indicate stiffness loss due to cracking of the tension elements.
The growth in crack widths due to repeated cyclic loading is well demonstrated in Figs. 14 and 15. Specimens C1 and C2 were subjected to cyclic loading with peak load equivalent to 80 percent of the cracking load of C1.

A better comparison is evidenced in Fig. 16 which graphs crack width variation of Specimens C1 and C2 with number of cycles of repeated load. The crack in Specimen C1, which includes tension elements, is much narrower than that in C2 which is designed with conventional steel.

**Load-deflection characteristics**

Load-deflection curves are a useful indicator of flexural stiffness of concrete members. The curves shown in Fig. 17 plot deflection at midspan of Specimen A1 as a two-span continuous member under static load.

Note that an increase in rate of deflection at around 55 kips is indicative of a decrease in stiffness due to cracking of the tension elements. The corresponding reaction is measured to be 10 kips, which indicates a support mo-
The theoretical cracking moment is calculated to be 55 ft-kips as previously mentioned. Similarly, the experimentally obtained cracking moment from observations of the load-deflection test of Specimen B1 is 45 ft-kips, and theoretical value is 47.3 ft-kips.

Typical load-deflection curves obtained at the beginning of and intermittently during repeated loading are shown in Fig. 18 for Specimen B2. The peak load was 37 kips. This corresponded to a moment of $1.23 \, M_{cr}$ for the section over the central support, calculated according to elastic theory.

In reality however, due to moment redistribution, the moment resulting due to this load was calculated from

Fig. 16. Comparison between crack width of Specimens C1 and C2.

Fig. 17. Load-deflection characteristics of Specimen A1 under static load test.

Fig. 18. Midspan deflection of Specimen B2 during repeated loading test.
measurements of reaction to be only 0.77 $M_{cr}$. Not surprisingly therefore, there was no fatigue failure or much evidence of its effects up to 1.5 million cycles, when the test was discontinued.

Fig. 19 shows a load-deflection curve for the final static test on Specimen B2 conducted to failure. The prestressing tendons had not yet ruptured as evidenced by an almost 50 percent recovery.

Several beams were tested as simply supported members in order to be able to apply higher bending moment. Typical load-deflection curves at midspan obtained from an X-Y plotter are shown in Fig. 20 where fatigue failure did not occur and in Fig. 21 where fatigue fail-

![Fig. 19. Midspan deflection of Specimen B2 after $1.5 \times 10^6$ cycles of loading.](image)

![Fig. 20. Load-deflection characteristics of Specimen A3 during repeated load test.](image)

![Fig. 21. Load-deflection characteristics of Specimen A4 during repeated loading test.](image)
Table 5. Summary of test results.

<table>
<thead>
<tr>
<th>Test Series</th>
<th>Specimen Number</th>
<th>( \frac{M^*}{M_{cr}} )</th>
<th>Number of Cycles of Loading</th>
<th>Mode of Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>A1</td>
<td>-</td>
<td>Not Applicable</td>
<td>Collapse</td>
</tr>
<tr>
<td></td>
<td>A2</td>
<td>1.63</td>
<td>5,590</td>
<td>Fatigue Failure</td>
</tr>
<tr>
<td></td>
<td>A3</td>
<td>0.95</td>
<td>Over 1,500,000</td>
<td>No Failure</td>
</tr>
<tr>
<td></td>
<td>A4</td>
<td>1.26</td>
<td>124,000</td>
<td>Fatigue Failure</td>
</tr>
<tr>
<td>B</td>
<td>B1</td>
<td>-</td>
<td>Not Applicable</td>
<td>Collapse</td>
</tr>
<tr>
<td></td>
<td>B2</td>
<td>0.77</td>
<td>Over 1,500,000</td>
<td>No Failure</td>
</tr>
<tr>
<td></td>
<td>B3</td>
<td>1.09</td>
<td>Over 750,000</td>
<td>No Failure</td>
</tr>
<tr>
<td></td>
<td>B4</td>
<td>1.11</td>
<td>1,418,000</td>
<td>Fatigue Failure</td>
</tr>
<tr>
<td>C</td>
<td>C1</td>
<td>0.8</td>
<td>Over 2,000,000</td>
<td>No Failure</td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>0.8 (Based on ( M ) of Spec. C1)</td>
<td>Over 2,000,000</td>
<td>No Failure</td>
</tr>
</tbody>
</table>

*See Table 1.

Data are graphed with respect to the number of cycles of load as shown in Figs. 22 and 23, respectively. Repeated load of peak magnitude equal to the cracking load of the specimen appears to be a critical value. Thus when repeated loading is higher than the cracking load, stiffness loss increases with number of cycles of load indicating progressive deterioration of the section.

This is confirmed in Fig. 24 which plots the load versus number of cycles curve. The curve is characteristically a reverse "S" with the lower leg asymptotically approaching a value equal to \( P_{cr} \) which may be taken as the endurance limit.

The results thus indicate that by using a combination of reinforcing steel bars and prestressed tension elements, the endurance limit is increased to.

ure was experienced at 124,000 cycles of repeated loading with peak value of 1.26 \( P \).

Details of load-deflection curves from similar tests on four other beams are given elsewhere.\(^7\) It is observed from these tests that load-deflection curves are direct indicators of member flexural stiffness.

For repeated loadings of sufficient magnitude to produce fatigue failure within a million cycles, the loss of stiffness was discernable early in the loading history. However, for lower levels of loading, loss of stiffness with progressive repeated loading was negligible.

**Endurance limit**

Test results are summarized in Table 5. Stiffness variation as a function of applied repeated load and crack width data are graphed with respect to the number of cycles of load as shown in Figs. 22 and 23, respectively.

Repeated load of peak magnitude equal to the cracking load of the specimen appears to be a critical value. Thus when repeated loading is higher than the cracking load, stiffness loss increases with number of cycles of load indicating progressive deterioration of the section.

This is confirmed in Fig. 24 which plots the load versus number of cycles curve. The curve is characteristically a reverse "S" with the lower leg asymptotically approaching a value equal to \( P_{cr} \) which may be taken as the endurance limit.

The results thus indicate that by using a combination of reinforcing steel bars and prestressed tension elements, the endurance limit is increased to.
Fig. 22. Stiffness variation with different range of applied repeated load.

Fig. 23. Crack width variation with different range of applied repeated load.

Fig. 24. Load versus number of repetitions relationship.
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cision ele­cr

m e nt s.

The prestressed tension elements should be accompanied by an amount of non-prestressed reinforcement to obtain optimum results for crack control and fatigue strength.

On the basis of this investigation, the following conclusions may be drawn:

1. The use of prestressed concrete tension elements as continuity reinforcement in composite construction creates a superior section. The pre­stressing strands tend to close up the cracks in the slab, thus providing better protection of the reinforcement against corrosion.

2. Before failure, there was a considerable amount of moment redistribution, up to 35 percent, in the continuous beams tested under static load.

3. The ultimate load carrying capacities of the specimens were closely predicted by the theory.

4. When the force induced in the tension element due to flexure was less than its cracking load, no fatigue failure developed in the specimens tested under repeated load for well over one million cycles of loading.

5. The loss of stiffness of a specimen as measured by the slope of its load­deflection curves, was dependent on the magnitude of the repeated load. In general, the higher the magnitude of the repeated load, the greater the rate of progressive loss of stiffness. When the induced force in the tension element was less than the cracking load, the loss of stiffness was negligible.

6. Under comparable loading, i.e., 80 percent \(P_{cr}\), the beam reinforced with tension elements exclusively exhibited less deflection and almost one half of the crack width when compared with members using reinforcing bars only.

7. The specimens subjected to repeated load tests developed only a major crack at the critical section, whereas those subjected to static tests developed many smaller cracks evenly distributed.

**References**


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Appendix

Handling and Cutting of Tension Elements

One of the practical questions regarding prestressed concrete tension elements is whether the element could be mass produced and cut to desired length. If so, what would be the practical length limitation of the element that could be handled without special care at job site?

To provide some evidence to these questions, two long tension elements were fabricated, one being 30 ft and the other 40 ft in length. The latter was lifted easily and conveniently with two-point pickup by a mobile crane in the casting yard and encountered no difficulty in handling as shown in Fig. A.

The 40-ft long tension element was then shipped 70 miles to the laboratory and withstood well in transit without special care, along with other tension elements and beams.

The 30-ft long tension element was easily cut in the casting yard using a masonry saw into one 15-ft section and two 7½-ft sections. Due to bond transfer, the prestressing strand withdrew approximately ½-in. at each cut end. However, there was no apparent distress or loss of bond due to the cutting operation.

Fig. A. Handling of 40-ft long prestressed tension element by two-point pickup.

Discussion of this paper is invited. Please forward your discussion to PCI Headquarters by April 30, 1977.