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STRENGTH OF TENSION LAP SPLICES IN POLYMER CEMENT CONCRETE

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Final Report   8 May 1978

A thesis submitted to the University of Colorado, Boulder in partial fulfillment of the requirements for the degree of Master of Science.

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IN POLYMER CEMENT CONCRETE

by

William F. Vanaskie

B.S., United States Military Academy, 1969

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For the past 30 years an area of major concern to engineers has been the strength of spliced reinforcement in reinforced concrete construction. This investigation used an epoxy emulsion to form a polymer-portland-cement concrete (PPCC), which was cast around tension lap splices in reinforced concrete beams. The beams, with varying length of lap splice, were tested with the splice located in a constant moment zone. The results showed that the strength of the splice can be increased by as much as 50 percent in ultimate load carrying capacity by use of the PPCC. Calculations also

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indicate that the required development length of the reinforcing steel
in the PPCC is approximately one-half of that in ordinary concrete.
These conclusions do not reflect, however, the physical properties of
the material. Although the test beams acted favorably, the tensile
strength of the PPCC at age aeven days was 602 psi (42.3 kg/cm^2), which
was only a 3 percent increase over the cement concrete. However,
the average bond stress was calculated to have an average increase of
200 psi over the cement concrete. The PPCC exhibited a failure pattern
which allowed the ductility of the member, even at short splice lengths,
to be superior to that of the control beams. The investigation indicates
that this type of construction can readily be adapted to field use
without additional costs in labor and with a savings in time and steel
over other types of polymer modified concrete.

$(42.3 \text{ Kg/sq cm}$

This Thesis for the Master of Science Degree by

William F. Vanaskie

has been approved for the

Department of

Civil, Environmental, and Architectural Engineering

by

James Chinn
———————————————————
James Chinn

Robert I. Carr
———————————————————
Robert I. Carr

Date *May 8, 1978*

Vanaskie, William F. (M.S., Civil Engineering)

Strength of Tension Lap Splices in Polymer Cement Concrete

Thesis directed by Professor James Chinn

The 1971 American Concrete Institute Building Code places
severe requirements on the splicing of deformed bars in
reinforced concrete construction for certain types of tension
splices. These requirements emphasize the complexity of the
problem of the development length and bond strength determination
in such construction. Extensive research has been performed on
the many variables affecting the strength of these splices.
Generally, however, these variables result from or affect the
physical properties of the material (usually Portland cement
concrete) surrounding the splice. Consequently, a reasonable
hypothesis is that the material surrounding the splice essentailly
controls the strength of the splice. It seems plausible to
assume that, if the physical properties of the material were
altered, a change in the strength of the splice could be
expected.

The purpose of this investigation was to analyze the validity
and application of this hypothesis by substituting a Polymer
Cement Concrete (PCC) in the area surrounding the splice. This
investigation is supplementary to the thesis presented to the
University of Colorado by William E. Benedict in May 1977.

The investigative program consisted of the comparative
testing of six sets of test beams, with varying lengths of lap,
in which all splices were located in a constant moment section.

Concrete compressive strength, and concrete and Polymer Cement Concrete tensile strengths were determined from companion test cylinders.

Analysis of the test results led to the following conclusions:

(1) The Polymer Cement Concrete used has no appreciable increase in tensile strength over that of Portland Cement Concrete at age eight days. This result is consistent with previously published data on Polymer Cement Concretes.

(2) Beams with a block of the PCC cast around the lap splice exhibited a greater load carrying capacity, for equal lengths of lap, than did the conventional reinforced concrete beams.

(3) Considerable savings in the length of the lap required to reach the yield strength of the steel can be realized by using the PCC in the splice zone. However, this savings is not as great as that found by Benedict using a non water-based epoxy system in a Polymer Concrete.

This abstract is approved as to form and content.

Signed *James Chinn*

Faculty member in charge of thesis.

ACKNOWLEDGEMENTS

The completion of this thesis marks the end of a wonderfully pleasant and rewarding segment of my life, and means that I must leave all those whom I have grown to know and love and admire here in Colorado. I would like to publicly express my gratitude to some of those who have helped me complete my graduate studies.

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There are no words to express my feelings for the love, and patience, and understanding which my family has always given me. So, to my wife, Kathleen, our son, Stephen, and our daughter, Ann Marie, I give my love and my life.

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INTRODUCTION

1.1 <u>Objective and Scope of Investigation</u>. This investigation
sought to determine the relative strengths of tension lap splices
cast in a Polymer Cement Concrete and similar splices cast in
conventional concrete. In order to perform such an analysis, six
sets of tests were conducted on beams cast from Portland cement
and Polymer Cement Concretes with varying lengths of lap. The
particular polymer used was an epoxy emulsion. Companion test
cylinders were cast and tested to determine the physical properties
and relative strengths of the hardened concretes.

An examination of results of bond and splice tests conducted
within the last 30 years reveals that no concise theory on the
effect of bond stress and development length on the strength of
spliced reinforcement is available. Development of such a theory
is hindered by the fact that so many variables have some sort of
influence on the behavior of reinforcing bars under the action of
bond stress. Prior studies have indicated that factors affecting
the strength of lapped splices of deformed bars include: length of
bars, spacing of splices, amount of cover, and the concrete strength.
Hence, this investigation set out to study the relative effectiveness
of replacing the material in the area of the splice with one whose
overall properties differ from those of Portland cement concrete.
The scope was further limited to the investigation of the effect of

a Polymer Cement Concrete (PCC) on the development length of reinforcement and no attempt was made to delve in detail into the physical property variations of the PCC and the Portland cement concrete.

1.2 <u>Previous Splice Tests</u>. The objective in testing tension lap splices is to determine the lap length required to develop a given stress in the reinforcing steel. The splice transfers the tensile force from one bar to another by bond between the bar and the surrounding concrete and resulting stresses in the concrete. Fig. 1 depicts such a transfer of forces.

Code limitations on bond stress have reflected concern both about bond strength and about bond slip. Strength is necessary to transfer stress out of the reinforcing bars, and excessive slip can result in wide cracks and premature flexural failure.

Bond strength of smooth (undeformed) bars is due initially to adhesion of the cement paste to the bar surface then, after slipping occurs, to friction. A bond failure consists of the bar pulling out of the hole it forms in the concrete. Bond strength of deformed bars is due initially to adhesion of the cement paste, then to friction plus bearing of the bar deformations against the concrete. Bars with shallow, widely spaced deformations often pulled through the concrete by shearing the concrete along a cylindrical surface surrounding the deformations. Sometimes, however, these bars would cause longitudinal splitting of the surrounding concrete before pull-through could take place. This splitting occurred along planes through the bar and was caused by the radially outward component of bearing stress of the bar deformations
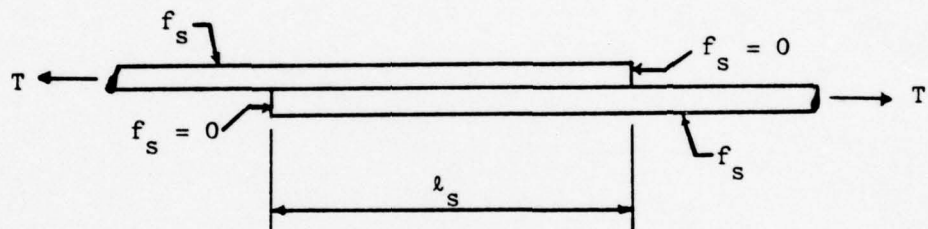
Fig. 1. Stress transfer and development length
in a splice.

on the surrounding concrete (see Fig. 2). The effect is similar to applying a fluid pressure to the sides of the hole in the concrete formed by the bar.

Prior to 1947, no general standard existed for reinforcing bar deformations. In that year, ASTM A305, "Tentative Specifications for the Deformations of Deformed Steel Bars for Concrete Reinforcement," was issued. Bars meeting the requirements of this specification proved to have bond qualities considerably superior to those of previously available commercial bars. In bond tests, failures were due to longitudinal splitting unless fairly large volumes of concrete surrounded each bar or splice or the concrete was reinforced with constraining hoops or spirals.

In early bond and splice tests, failures were by pull out rather than splitting, and researchers seemed to be concerned mainly with load-slip behavior rather than ultimate strength. In some of their research reports, they reported only mode of failure and not ultimate bond stress. Embedment lengths and strengths of concrete were of concern, but the amount of concrete around the bar or splice, related to cover and bar or splice spacing in beams, was not considered important. Little, if any, thought was given to the fact that bars in structures had much less cover and were much closer spaced than bars in test specimens.

Code values for allowable bond stress were set at an average of stresses in tests for a given loaded-end slip (such as 0.01 in.), divided by a factor of safety. It was thought that this would limit the width of cracks which might form under flexure.

After failures in tests became predominantly splitting failures, it took quite some time before it was generally realized
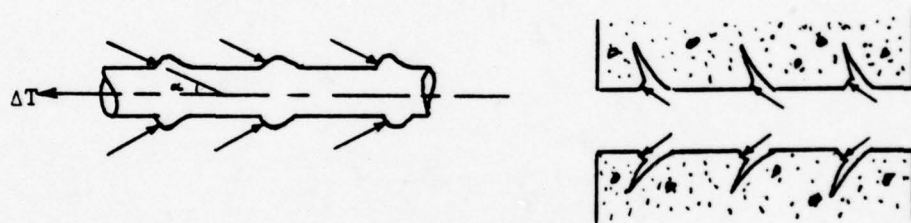
(a) Bond force on bar.   (b) Reaction on concrete.

Fig. 2. Forces between deformed bar and concrete.

that bars in structures had much less surrounding concrete per bar or splice and that splitting in structures could occur at considerably lower bond stresses than the ultimate bond stresses found in tests. In some tests, splitting failures were produced before bars pulled out, but the fact that this would happen at even lower stresses in a structure does not seem to have occurred to the researchers. They simply added more concrete around the bars in subsequent tests or added hoops or spirals.

Kluge and Tuma (17) tested lapped splices in beams as early at 1945. However, their tests were conducted on a single spliced bar flanked by two continuous bars in beams which were 13 or 14 bar diameters wide. This obscured the actual failure conditions, as the splitting was limited to that produced by the single splice, and the distribution of tensile force between the spliced bar and the continuous bars was uncertain. Their tests were, nonetheless, representative of the effect of staggered splices in a constant moment section. Their tests produced essentially the same results for spaced as well as for contact splices.

In 1951, Walker (30) reported on bond tests on pullout specimens which were meant to simulate spliced bars. In addition to having a rather large cross section (8 inch square), his specimens placed the concrete in compression rather than in tension. His ultimate bond stresses were thus higher than might be expected from a beam test with concrete around the splice in tension. Spaced and contact splices produced nearly identical results.

In 1952, Chamberlin (10) also reported on pullout tests of spliced reinforcement. Although his specimens were tested in tension, the large size of the concrete blocks (6" x 6" for #4 bars and 9" x 9" for #6 bars) and the use of spiral reinforcement effectively prevented or alleviated any splitting failures. Bond strengths obtained were too high relative to those which could be expected in structures in which splitting was not restrained.

Up until this time, splitting of the concrete was not considered to be linked to its bond capacity. Walker and Chamberlin took great pains to ensure that splitting would not occur just so they could arrive at "realistic" bond strength determinations.

In 1954, Chinn, Ferguson, and Thompson (12) implemented a pilot study into the strength of lapped splices in tension. Unlike in previous tests, their study made no specific attempts to eliminate splitting type failures and was designed to include the effects of thickness of cover, length of lap, stirrups, and concrete strength. Conclusions regarding these and other variables were thus obtained. They also found that all of the test beams failed by splitting of the concrete, usually at relatively low and unsatisfactory capacities when compared to pullout tests.

Their tests were followed in 1958 by another series of tests by Chamberlin (11). This time he employed test beams with varying lengths of lap and verified the earlier findings of Chinn, Ferguson, and Thompson.

These tests furnished convincing evidence that the bond capacity of steel in concrete was definitely linked to the

splitting behavior of the concrete. Appropriate changes in the ACI Building Code were forthcoming.

In the early 1960's a series of tests was conducted at the University of Colorado by Skillen (26), Sanders (25), and Judd (16). The tests by Skillen and Sanders were designed mainly to check the adequacy of the proposed 1963 ACI Code, and they added little to existing knowledge of bond behavior and development length. Judd attempted to develop steel strength by employing combinations of lap length and hooks, however, no appreciable decrease in bar lengths resulted.

The premise that the development length and thus the splice strength is limited by the splitting strength of the surrounding concrete was tested by Benedict (9) in 1977. He surrounded the splice with a Polymer Concrete (PC), specifically an epoxy concrete, and found the development length for #6 Grade 60 bars was reduced to four inches. He attributed this reduction directly to the increase in splitting strength of the material (the PC) surrounding the splice. The tensile splitting strength of the PC was approximately three times that of Portland cement concrete. However, he also found the material to be very brittle. Failure of his test specimens occurred suddenly, with almost no observable plastic deformation.

1.3 <u>American Concrete Institute Building Code Requirements</u>. Bond stress and splice length requirements contained in the ACI Building Code since 1947 also provide a sample of the thinking of "experts" prevalent at the time of their respective publication.

A comparison of the Code provisions will reveal a relaxation of bond stress requirements, then increased interest and more severe requirements on splices.

Allowable bond stress in the 1947 Code was 0.05 f_c' with a maximum of 200 psi. Splice length was governed only by the requirement to transfer the stress between bars by shear and bond. These provisions were written for deformed bars which would be unacceptable by today's standards and reflect the belief that bond failure was a pullout failure and slip was the measure of useable bond strength. For the most common combination of $f_c' = 3000$ psi and allowable steel stress, $f_s = 20,000$ psi, splices had to be lapped 33-1/3 bar diameters.

The ACI recognized the increased effectiveness of the newly standardized ASTM A305 high bond bars in its 1951 Code. In this revision bond stress was increased to 0.10 f_c' with a maximum of 350 psi. Splice length was still governed by bond, as in the 1947 Code, but the length for $f_c' = 3000$ psi and $f_s = 20,000$ psi was cut in half to 16-2/3 bar diameters by the doubled allowable bond stress.

The 1956 Code maintained the allowable bond stress of the 1951 Code but concern about splices was shown by the addition of a lap length requirement of 24 bar diameters but not less than 12 inches. It was still specified that allowable bond stresses were not to be exceeded.

Major changes were made in bond and splice provisions in the 1963 Code. Ultimate strength design was included in the main body of the Code and was given equal status with working stress

design. The terms "bond failure or splitting, flexural bond stress, embedment length, and anchorage or development bond stress" were used. Permissible flexural bond stress for ultimate strength design was set at $9.5 \ (f'_c)^{\frac{1}{2}}/d_b \leq 800$ psi for other than "top" bars, conforming to ASTM A305. In this relation d_b is the nominal bar diameter. For bars larger than $d_b = 84.21/(f'_c)^{\frac{1}{2}}$, the formula controls, and for bars smaller than that, the 800 psi controls.

The Force which can be transferred out of a bar, per unit length, is the bond stress times the perimeter, or $u\pi d_b$. For larger bars, this is $[9.5 \ (f'_c)^{\frac{1}{2}}/d_b \times \pi d_b] = 9.5\pi (f'_c)^{\frac{1}{2}}$, which is independent of bar size. This is a splitting provision because the splitting force per unit length is a function of the change in bar force per unit length and is not a function of the size of the bar. The 800 psi permissible stress is based on the assumption that smaller bars pull out and do not cause splitting failure.

The 1963 Code permitted flexural bond stress to be ignored if the anchorage bond did not exceed 0.8 of the permissible flexural bond.

Concern was shown for strength of splices in the 1963 Code by providing that the bar stress be transferred "from bar to bar without exceeding three-fourths of the permissible bond values" and that lap lengths should not be less than 24, 30, and 36 bar diameters for specified yield strengths of 40,000, 50,000, and 60,000 psi, respectively. Additionally, the lengths of contact splices spaced closer than 12 bar diameters or located closer than six inches or six bar diameters from an edge had to be increased 20 percent.

By the 1963 Code, the length required to develop the yield stress of a bar was

$$\ell_d = \frac{f_y d_b}{4\,u_u} = \frac{f_y d_b}{4} \times \frac{d_b}{9.5\sqrt{f'_c}}$$

or

$$\ell_d = 0.0335\, A_b\, f_y/(f'_c)^{1/2} \geq [f_y d_b/4(800) = 0.0008125\, f_y d_b]$$

in which $A_b = \pi d_b^2/4$ = area of bar.

For closely spaced splices, the length had to be $\ell_s = 1.2\ell_d/0.075$. This is $\ell_s = 0.0536\, A_b f_y (f'_c)^{1/2} \geq 0.005\, f_y d_b$ with $24d_b$, $30d_b$, and $36d_b$ minimum for f_y = 40,000, 5C,000, and 60,000 psi respectively.

In the 1971 Code, bond provisions were recast in terms of required development length. Further concern was shown for closely spaced bars by setting development lengths equal, essentially, to those required by the 1963 Code times the 1.2 factor for closely spaced splices. These could be reduced by 0.8 for bars spaced at least six inches on center, returning that case to approximately the same as the 1963 Code. The basic development lengths given were

$$\ell_d = 0.04\, A_b\, f_y/(f'_c)^{1/2} \geq 0.0004\, f_y d_b\ .$$

Splice requirements were made more conservative by requiring that the splice length be 1.3 ℓ_d to 2.0 ℓ_d, depending on whether splices occurred in regions of high or low steel stress and the fraction of the bars spliced in a required lap length. For all bars spliced at the same location and high steel stress,

$$\ell_s = 1.7\, \ell_d = 0.068\, A_b\, f_y/(f'_c)^{1/2} \geq 0.00068\, f_y d_b\ .$$

It should be noted that, although the 1963 and 1971 Code provisions did take splice spacing into consideration, the amount of cover over the bar has yet to be taken into account.

The Code provisions on bond and splices are summarized for a #6 Grade 60 bar, f'_c = 3000 psi and f_y = 40,000 and 50,000 psi in Table I. It is assumed all bars are spliced at the same section and are highly stressed and closely spaced.

1.4 <u>Polymer Modified Concrete</u>. Reinforced concrete construction has become more and more extensive as its adaptability to new requirements has reached seemingly endless proportions. However, more extensive use of concrete has been regulated by its inherent undesirable properties. Chief among these are its low tensile strength, its susceptability to deterioration due to changes in temperature and environment (moisture) conditions due to its absorption capacity, and its low resistance to chemically reactive agents. Efforts to improve these detrimental characteristics have resulted in an increased use of polymers in concrete.

The initial use of polymers in concrete was a method whereby precast, cured, conventional concrete was impregnated with a monomer which was allowed to thoroughly saturate the concrete and fill all the voids to a desired depth. The monomer was then polymerized (solidified) in place. This process has found some limited use in bridge deck construction and repair and in areas where the concrete is subjected to extreme stresses, as in spillways and stilling basins of dams. Tremendous improvements in the structural and durability properties of the treated concrete have been realized by this method (7). Although Polymer Impregnated

TABLE I

COMPARISON OF CODE REQUIREMENTS

Code	u_{allow}	$f_y = 40,000$ psi		$f_y = 50,000$ psi	
		ℓ_d(in.)	ℓ_s(in.)	ℓ_d(in.)	ℓ_s(in.)
1947	$0.05\ f_c' \leq 200$ psi	25.0^a	25.0^a	25.0^a	25.0^a
1951	$0.10\ f_c' \leq 350$ psi	12.5^b	12.5^b	12.5^b	12.5^b
1956	$0.10\ f_c' \leq 350$ psi	12.5^b	18.0^c	12.5^b	18.0^c
1963	$9.5\ (f_c')^{\frac{1}{2}}/d_b$**	10.8^d	18.0^e	13.5^d	21.6^e
1971	------	12.9^f	21.9^g	16.1^f	27.4^g

Note: ** = Permissible bond stress for ultimate strength design.

Letters a – g indicate the governing Code provisions as indicated below:

$a = 33\text{-}1/3\ d_b$

$b = 16\text{-}2/3\ d_b$

$c = 16\text{-}2/3\ d_b \geq 24\ d_b$

$d = 0.0335\ A_b\ f_y\ /\ (f_c')^{\frac{1}{2}}$

$e = 0.0536\ A_b\ f_y\ /\ (f_c')^{\frac{1}{2}} \geq 24\ d_b$

$f = 0.04\ A_b\ f_y\ /\ (f_c')^{\frac{1}{2}} \geq 0.0004\ f_y d_b$

$g = 1.7\ \ell_d$ (as given by f)

Concrete (PIC) exhibits vastly superior properties when compared to conventional concrete, it is nonetheless more brittle. The same tensile crack formations found in ordinary concrete also occur in a PIC element reinforced with steel bars (7).

The advantages in the overall strength properties of PIC have literally forced its increased use. However, it does require a precast element and sophisticated, and sometimes cumbersome, equipment, and it is not readily suited for general field use. The ability to cast a material with similar properties in place was the incentive to develop a mixture of cement paste and aggregate to which a monomer would be added prior to casting. The result was a Polymer Cement Concrete (PCC). Early work in this area incorporated either rubber latices or polymer emulsions into the concrete mix. The results were all too often disappointing, producing only modest improvement in strength and durability (7). The limited success can be attributed to the fact that organic materials are incompatible with an aqueous system and may in fact interfere with the cement hydration process (7). Recent research has produced an epoxy cement concrete system which attained an increase in strength of 90 percent with an appropriate mix design (27). However, special care had to be taken to limit the water/cement ratio while the epoxy resin was added. High or low resin/cement ratios result in a concrete whose strengths are no better than, and possibly even worse than, those of Portland cement concrete.

1.5 Epoxy. Epoxies are a family of synthetic resins with a wide range of viscosities. Essentially formulated by a combination

of Epechlorhydrin and Bisphenol, they were initially used as adhesives, electrical insulation, and protective coatings. Early uses of epoxies in the construction industry, in the early 1950's, made use of their adhesive properties. The rapid curing, toughness, strength, and superior chemical resistance properties, however, made them ideally suited for use in concrete construction, repair, and maintenance. This potential for use in the concrete construction industry fostered a remarkable progress in their commercial and technical development. As the development progressed, so too did their use and potential applications. Their performance in coatings, floor toppings, and overlays, grouts, and patching compounds made them a valuable companion product for concrete.

The basic epoxy resin most widely used in the construction industry is an amber-colored liquid with a viscosity similar to that of heavy motor oil. The resin will remain a liquid indefinitely until mixed with a chemically reactive hardening agent. The two parts of the epoxy system (the resin and the hardener) must be well mixed to produce the desirable properties. Improper mixing will result in weak spots within the molecular structure of the mixture. A single system is not suitable for all applications, thus, without exception, epoxy systems must be specifically formulated for particular applications.

Generally, a well mixed system will reach its initial set within a few hours and when thoroughly cured will reach the state of a hard, infusable solid. Concrete formulated with an aggregate bound with an epoxy system as the binder, i.e., an epoxy or polymer concrete, would seem to result in a vastly superior material to

ordinary concrete. Indeed, Benedict (9) found many superior characteristics. However, he also found the material to be extremely brittle.

The epoxy compound used in the current investigation is designed to be added as a monomer to a Portland cement mixture which, when cured, yields a Polymer Cement Concrete. The formulation, described in Chapter II, contains an emulsifier which makes it compatible with an aqueous system. This epoxy was donated for use in this investigation by its formulator, PROTEX Industries, Inc., of Denver, Colorado.

CHAPTER II

TEST SPECIMENS AND TESTING PROCEDURES

The object of the proposed research was to determine the relative strength of lapped splices in a Polymer Cement Concrete (PCC) versus the strength in regular Portland cement concrete (hereafter referred to as just "concrete").

The scope of the investigation had to be limited due to time, financial, and equipment constraints, so it was decided to test only one bar size, #6. Six sets of beams seemed a reasonable number, each set consisting of a concrete beam and a companion beam with PCC surrounding the splice. The only difference between beams in a set would be the PCC replacing the concrete around the splice. The variable to be tested was the length of lap.

A casting and testing schedule was set up to cast a set of beams every seven days and test each beam at an age of eight days.

2.1 Design of Test Specimens. In designing test specimens, the decision was made to investigate the worst case of a splice of #6 bars. The #6 bar was chosen because it had exhibited bond problems in past tests (12) and because it results in reasonable size beam specimens.

The worst case of a splice in a beam is one in which all bars are spliced at the same section in a constant moment zone, and the splices are spaced the minimum clear distance apart. By Sections 7.4.1 and 7.4.5 of ACI 318-71, this distance is one inch for #6

bars. Two splices per beam were selected to avoid dissymmetry which could result in torsion. Rather than use a normal two-bars-in-one-layer beam with 1½" side cover, specimens were designed to duplicate the interior portion of a multibars-in-one-layer beam (Fig.3). Width of specimens was therefore set at four bar diameters (4 x 3/4" = 3") plus one inch clear between splices plus two half-inch clear distances at the sides (2 x ½" = 1") for a total of five inches.

The worst case of cover over the bars occurs when no stirrups are present, and this is a cover of 1½" for beams. Overall beam depth was chosen somewhat arbitrarily as eight inches, resulting in an effective depth, d, of 8" - (1½" cover + 3/8" half bar diameter) equal to 6-1/8". The specimen cross section chosen is shown in Fig. 4.

The splice length for the first test set was chosen as 12 inches because it is approximately 16-2/3 bar diameters, a length once erroneously thought to be sufficient to develop a yield stress of 50 ksi in a reinforced concrete beam. At the time the present tests were planned, it was speculated that the PCC beam might be capable of developing the yield stress of the Grade 60 bars used.

The splice length for the second test set was set at eight inches because it was to be cast before the beams of the first set had been tested, and it was desirable to have the second series weaker than the first series.

The splice lengths for the remaining sets were chosen after examining the test results obtained prior to the casting of each series. At the end of the testing program, additional companion
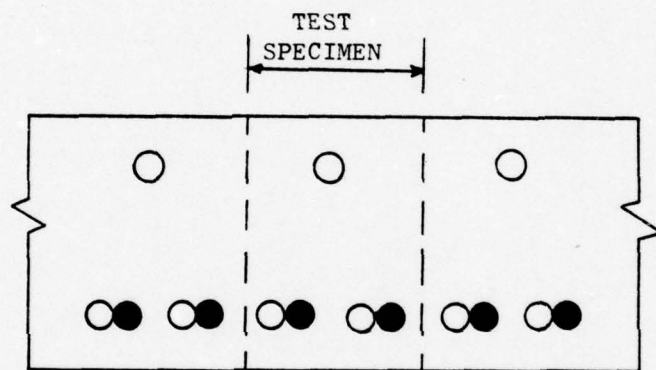
Fig. 3. Repeating section showing specimen design.
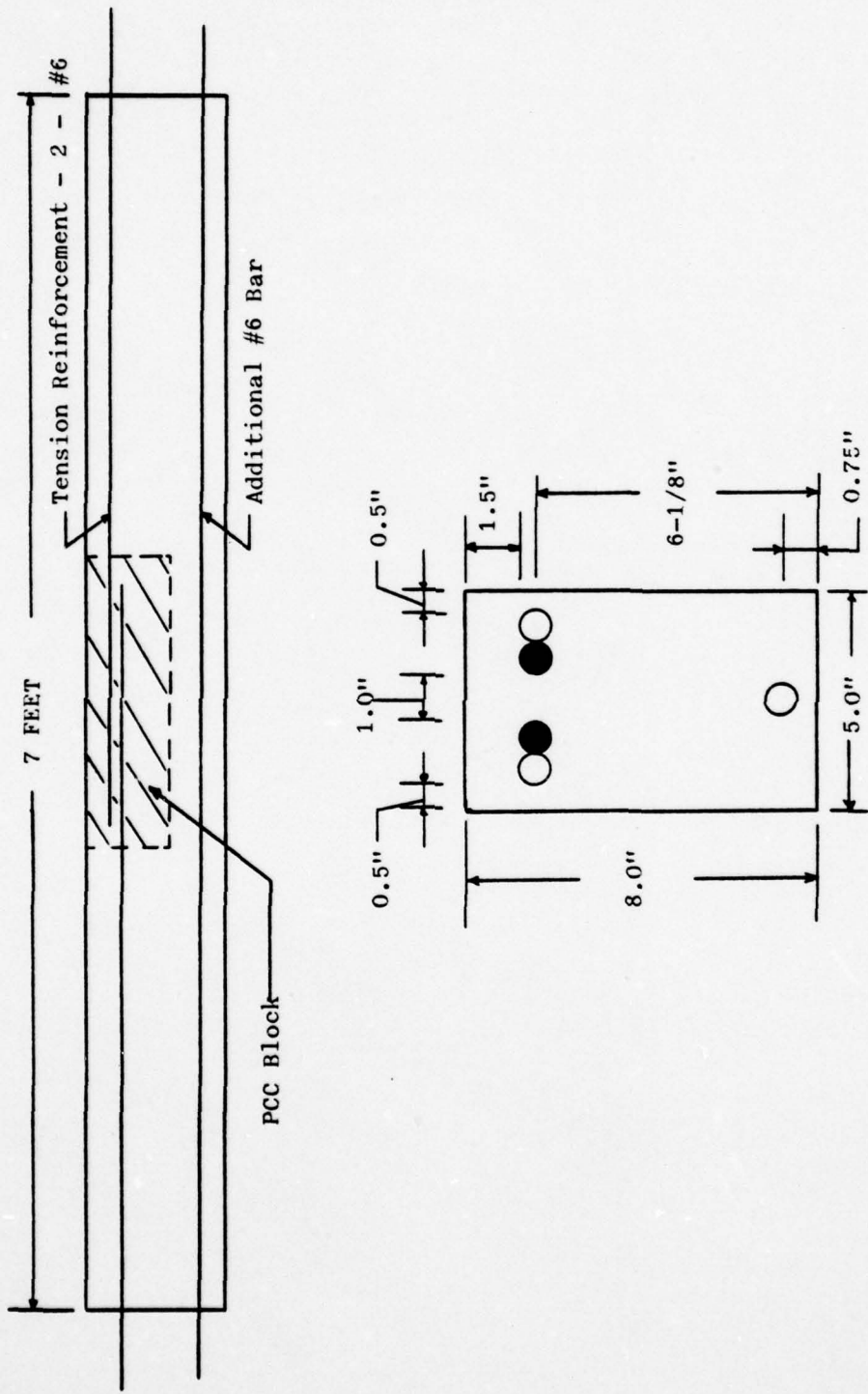
Fig. 4. Beam dimensions and geometrical constants and reinforcement location.

concrete and PCC beams had been cast with lap lengths of 16, 12, 14, and 6 inches. The 12 inch lap of the first test set was repeated in the fourth test set because the first set of results was suspect. The overall length of the specimens was set at 7'0" so that standard eight foot long plywood sheets could be used for beam forms. This made it necessary to include shear reinforcement in the beams to avoid shear failures during testing.

One #6 bar was included in the top of each test beam to take care of any tension which might accidentally develop during moving and handling the beam. It was not needed as compression steel.

Beam designations, dimensions, and lap lengths are given in Table II.

The depth of the PCC block was chosen as half the effective depth ($\frac{1}{2}$ x 6-1/8") plus bottom cover (1$\frac{1}{2}$") plus one-half bar diameter ($\frac{1}{2}$ x 3/4"), rounded off to the nearest inch, or five inches. In order not to have a potential plane of weakness at the end of the splice, the PCC block was extended 2$\frac{1}{2}$" beyond the splice end.

2.2 Companion Test Cylinders. For each test set the following test cylinders were cast to determine material properties:

(1) 3 - 6 x 12 inch concrete compression cylinders,

(2) 3 - 3 x 6 inch concrete tensile splitting test cylinders,

(3) 3 - 3 x 6 inch PCC tensile splitting test cylinders.

2.3 Materials. Concrete mix proportions are listed in Appendix B. Concrete materials used were:

(1) Aggregate — The fine and coarse aggregates were obtained from a local producer. The aggregate was blended to meet ASTM

TABLE II

TEST BEAM DETAILS

Test Beam Designation	Nominal Cross Section Dimensions (in.)			Lap Length ℓ_s (in.)
	b	h	d	
T-1-C	5	8	6-1/8	12.0
T-1-P	5	8	6-1/8	12.0
T-2-C	5	8	6-1/8	8.0
T-2-P	5	8	6-1/8	8.0
T-3-C	5	8	6-1/8	16.0
T-3-P	5	8	6-1/8	16.0
T-4-C	5	8	6-1/8	12.0
T-4-P	5	8	6-1/8	12.0
T-5-C	5	8	6-1/8	14.0
T-5-P	5	8	6-1/8	14.0
T-6-C	5	8	6-1/8	6.0
T-6-P	5	8	6-1/8	6.0

Specification C-33, "Standard Specification for Concrete Aggregate,"
for each test set. A slight variation in these specifications was
necessary for test sets five and six (see Appendix B).

(2) Water — City water from a laboratory faucet was used
throughout the test program.

(3) Cement — The cement used was a Portland Type I,
produced by the Martin Marietta Company. Cement was from two
different production lots; however, no appreciable difference was
observed on properties of the hardened concrete made from the two
cements.

(4) Epoxy — The epoxy used in the formulation of the PCC
was a two component system marketed by PROTEX Industries, Inc.,
Denver, Colorado, under the name Probond Epoxy ET-180 (Emulsified).
Specific terminology and constants of the system are listed
below:

Part A — Diglycidyl ether of bisphenol A with an
emulsifier added. The physical constants, listed
as typical by the manufacturer, are:

viscosity @ 25°C	13,000 cps
wt/gal	9.63 lbs
wt/epoxide	190 lbs

Part B — Polymeric amido-amine (Poly Amide type).
The physical constants, listed as typical by the
manufacturer, are:

viscosity	25,000 cps
wt/gal	8.15 lbs
equivalent wt.	116 lbs

Physical properties of the hardened system are not available for this particular emulsified epoxy, but they would be expected to be slightly lower than the properties of the non-water based epoxy system consisting of the two parts listed above but without the emulsifier. Typical properties for that system, at age 14 days, include:

Compressive strength	10,300 psi
Ultimate flexural strength	13,000 psi
Tensile strength	8,300 psi
Tensile elongation	4.6 %

The mix design for the PCC is given in Appendix B.

(5) Reinforcement — Steel reinforcement for all specimens consisted of #6 Grade 60 bars manufactured by Border Steel Mills, Inc., El Paso, Texas. Average yield stress of the bars tested was 65 ksi, and modulus of elasticity was approximately 29,000 ksi. A typical stress-strain curve for these bars is shown in Fig. 5. Web reinforcement was fabricated from welded wire fabric, style designation WWF 6 x 12 - W7.5 x W7.5, provided by Stanley Structures, Denver, Colorado.

2.4 Formwork. All six sets of test beams were cast utilizing one set of forms. The forms were constructed of 3/4 inch plywood with inside dimensions 5 inches x 8 inches x 7 feet long. In order to ensure that the forms would last throughout the entire testing program, each part was covered with up to four coats of shellac, varnished and, additionally, prior to each casting, was waxed with a commercial paste wax. In the area of the splice,
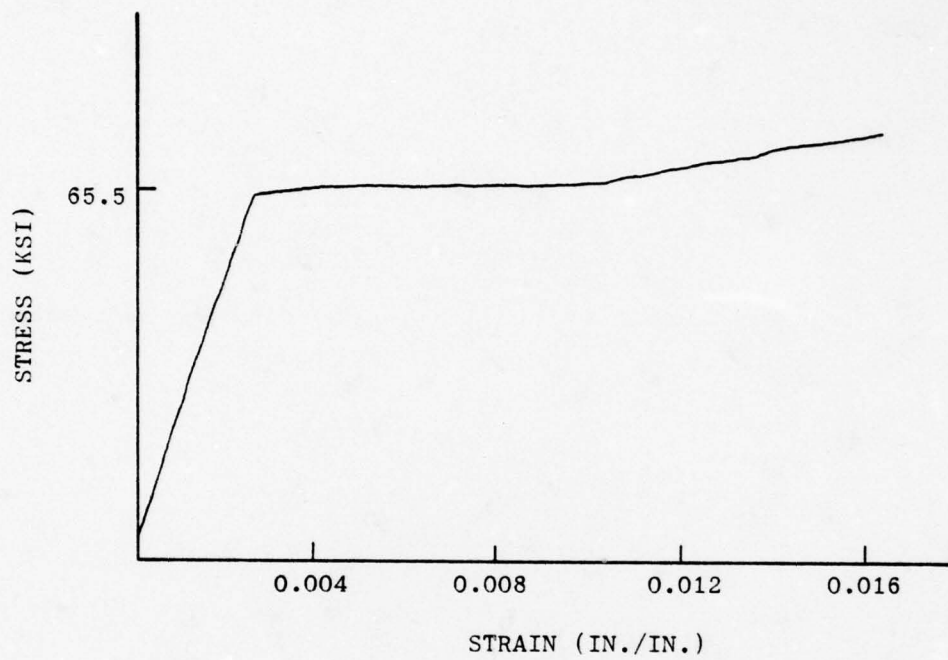
Fig. 5.   Stress-strain curve for reinforcing steel.

plastic sheets were taped to the sides and bottom of the forms to prevent adherence of the epoxy in the PCC. This technique proved very effective and added little additional effort or time to the overall formwork preparation. Temporary end forms, also constructed of 3/4 inch plywood and covered with plastic, were used to contain the PCC block until its initial set (Fig. 6). The temporary end forms were removed prior to placing the Portland cement concrete.

2.5 <u>Mixing and Casting Procedures</u>. Mixing of the cement and PCC concretes and casting the test beams and cylinders generally followed the sequence outlined below:

(1) All test beams were cast with tension steel in the bottom. The four #6 bars, which served as tension reinforcement, were positioned in the forms which had been prepared as described above. Plastic bar supports were used to hold the bars at the proper level. Each splice was tied twice with soft wire ties providing a contact splice of the designated lap length (see Fig. 6). In the control beam, a single #6 bar was then positioned, and the web reinforcement was positioned and tied in place. In the PCC beam the additional #6 bar and the web reinforcement were left out until later.

(2) The temporary end forms were positioned in the PCC beam for casting a block of polymer cement concrete five inches wide by five inches high with a length equal to the length of the splice plus 2½ inches on each end.

(3) All aggregate, cement, epoxy, and water were weighed and set aside for mixing. The PCC was mechanically mixed using a half-inch electric drill operating at approximately 600 rpm. A special beater (Fig. 7) was fabricated to ensure proper stirring of the PCC mixture.
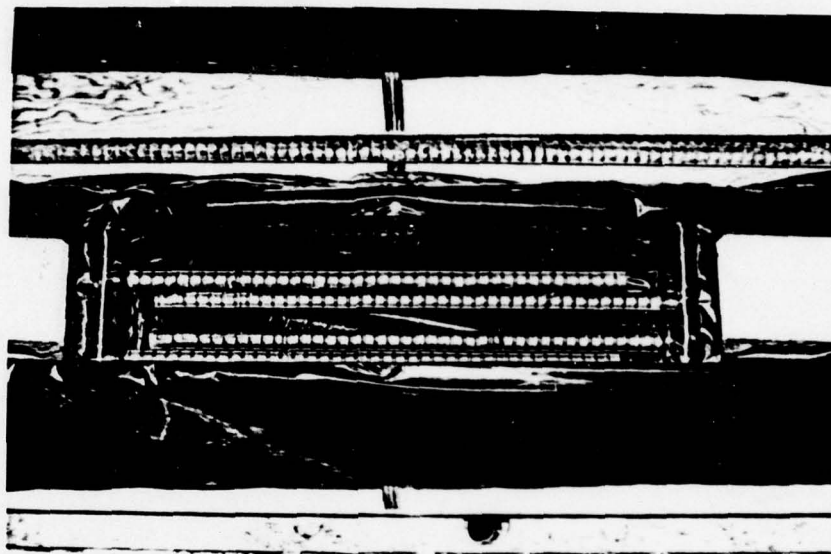
Fig. 6. Plastic sheets and temporary end forms
in place in PCC beam.
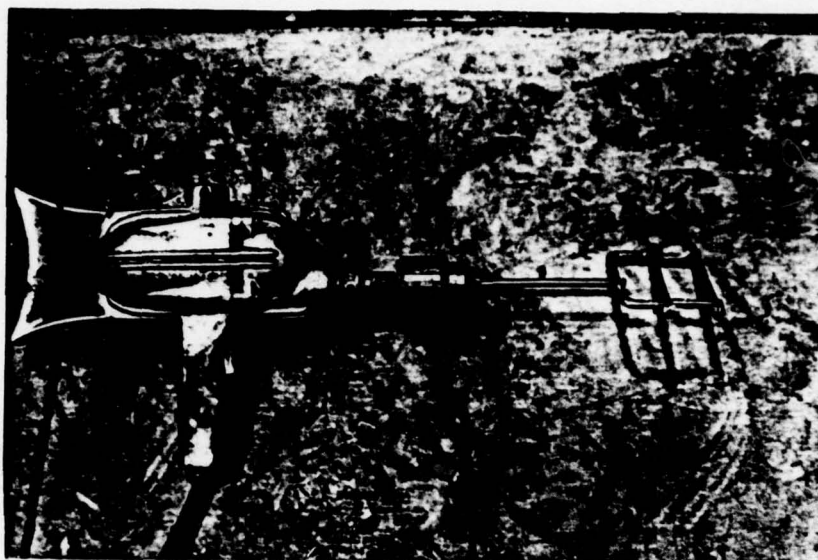
(a) Drill and Beater
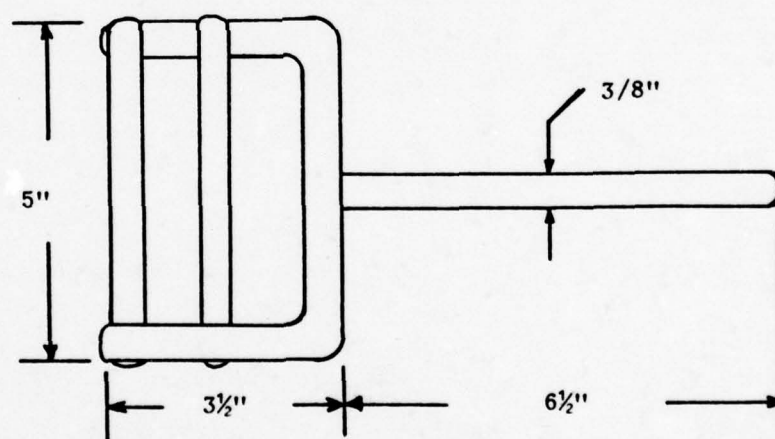
(b) Beater Dimensions

Fig. 7. Drill with special beater and beater dimensions.

Mixing of the epoxy system must produce a uniform, homogeneous mix. To accomplish this, the components of the epoxy system (Part A and Part B) were placed in a drum measuring 18 inches in diameter and 13½ inches deep and stirred well with the beater for two to three minutes until a consistent blend was obtained. The blended epoxy was then allowed to sit for five minutes. After this settling period the water was added in increments and blended in until the mix reached a uniform, taffy-like consistency. The cement was then added. It was introduced into the mix slowly, while stirring continued, to avoid balling and to ensure that each particle was coated with the epoxy. The coarse aggregate was added and mixed well, followed by the fine aggregate, which was added slowly. This order of addition helped prevent the mix from balling up and ensured that all particles were well coated and that the mix was well blended. The total mixing time was approximately 40 minutes.

(4) The PCC was then hand placed into the prepared forms. The particular mix used resulted in a very stiff, sticky mixture. It was this characteristic that necessitated hand placement of the PCC into the forms to prevent honeycombing. The block surrounding the lap splice was placed in three equal layers to the desired depth of five inches. Each layer was rodded extensively and internally vibrated with a Model L vibrator produced by the Viber Company. Despite the extensive rodding and vibrating, no segregation and little or no bleeding was observed in the mix. Three 3 x 6 inch cylinders were cast at the same time. The cylinders were cast in prepared metal forms according to ASTM Method C-192, "Standard

Method of Making and Curing Test Specimens in the Laboratory." The molds were well greased to prevent the epoxy from bonding to the metal surfaces, and each mold was encased in a plastic bag to prevent leakage of the epoxy. This last precaution proved to be unnecessary as no evidence of any leakage of the epoxy was observed during the test program.

(5) While the PCC block was allowed to start to set (approximately 30 minutes) the ingredients of the concrete, which had previously been weighed and set aside, were mixed in a rotating-drum, electrically-powered mixer. The mixer was a tilting-drum Essick, Model 62BE with a six cubic foot capacity.

(6) The temporary end forms on the PCC block were removed, the additional #6 top bar was positioned and the web reinforcement was set and tied in place.

(7) The concrete was placed in both beams in three equal layers. Since the PCC block had not yet reached its final set the surface was still tacky and bonded well with the fresh concrete. Each layer of the concrete was rodded and internally vibrated to produce a homogeneous concrete free from honeycombing. Three 3 x 6 inch and three 6 x 12 inch cylinders were also cast from the concrete batch in prepared metal forms according to ASTM Method C-192.

(8) Both beams and all the cylinders were covered with a sheet of plastic and allowed to cure for 24 hours in their forms. Forms were removed on the second day, and all specimens were placed in a 100% humidity room. The normal curing cycle lasted seven days. Specimens were removed from the curing room approximately 24 hours

prior to testing on the eighth day. Special curing procedures were adopted for the PCC cylinders and beams after the first test set. Representatives of the PROTEX Company thought it was possible that the specimens for the first test had not properly cured due to the 100% humidity. The condition was considered remote, but nonetheless, it may have explained the rather poor results. Therefore, the cylinders for the remaining tests were cured for varying periods of time under moist and air cured conditions according to the schedule in Table III. The curing cycles for the PCC beams for test sets two and three were as follows:

T-2 -- The beam was removed from the 100% humidity room on the sixth day and allowed to cure for two days in the ambient conditions of the laboratory prior to testing.

T-3 -- The beam was removed from the curing room on the fifth day. The concrete ends of the beam outside the splice zone were covered with moist cloths and wrapped with plastic sheets while the PCC block was opened to the air. The beam was allowed to cure in this condition for two days.

The beams for the ramaining three test sets were cured in a manner identical to that described for test set number three, except that the curing cycle lasted six days once the beams were removed from the forms.

2.6 <u>Testing Procedure</u>. All specimens for each set were tested on the same day. Tests for the compressive and tensile strengths were generally conducted prior to the testing of the beams.

TABLE III

TEST CYLINDER CURING CYCLES

		Number of Days	
Test Series	Cylinder Number	Moist Cured	Air Cured
T-2	P1	6	1
	P2	6	1
	P3	7	0
T-3	P1	5	2
	P2	4	3
	P3	5	2
T-4	P1	0	7
	P2	0	7
	P3	0	7
T-5	P1	0	7
	P2	0	7
	P3	0	7
T-6	P1	7	0
	P2	7	0
	P3	7	0
	P4	0	7
	P5	0	7
	P6	0	7

2.6.1 <u>Cylinders</u>. Prior to testing, the actual dimensions of
each of the cylinders were measured to the nearest 0.01 inch.
Testing of all cylinders was conducted in a 300 kip capacity
Southwark-Baldwin-Emery Universal Testing Machine. The compressive
strength of the concrete was determined according to ASTM Method
C39-72, "Standard Method of Test for Compressive Strength of
Cylindrical Concrete Specimens." These cylinders were capped on
both ends with Cylcap, a commercial sulfur-fire clay capping
compound, prior to testing. The relative tensile strengths of
the concrete and the PCC were determined by means of split
cylinder tests conducted according to ASTM Method C496-71,
"Standard Method of Test for Splitting Tensile Strength of
Cylindrical Concrete Specimens."

2.6.2 <u>Test Beams</u>. The actual dimensions of each of the
beams at the critical sections were measured to the nearest 0.01
inch prior to testing. The beams were then "whitewashed" with a
thin plaster of Paris solution to aid in the detection of cracks.
The testing arrangement for the beams was as shown in Fig. 8. All
testing was conducted on the University of Colorado Structural
Test Floor. The beams were tested with the tension side up to
permit more accurate inspection of the tension zone. The load was
applied by means of an hydraulic hand pump (Templeton, Kenly and
Company) which was connected by a manifold to two 60 kip capacity
hydraulic jacks manufactured by the Sims Engineering Company. A
100 kip capacity BLH C2P1 load cell was fitted in series with the
west hydraulic jack then electrically connected to a load dial
which indicated load on the jack. This arrangement necessitated
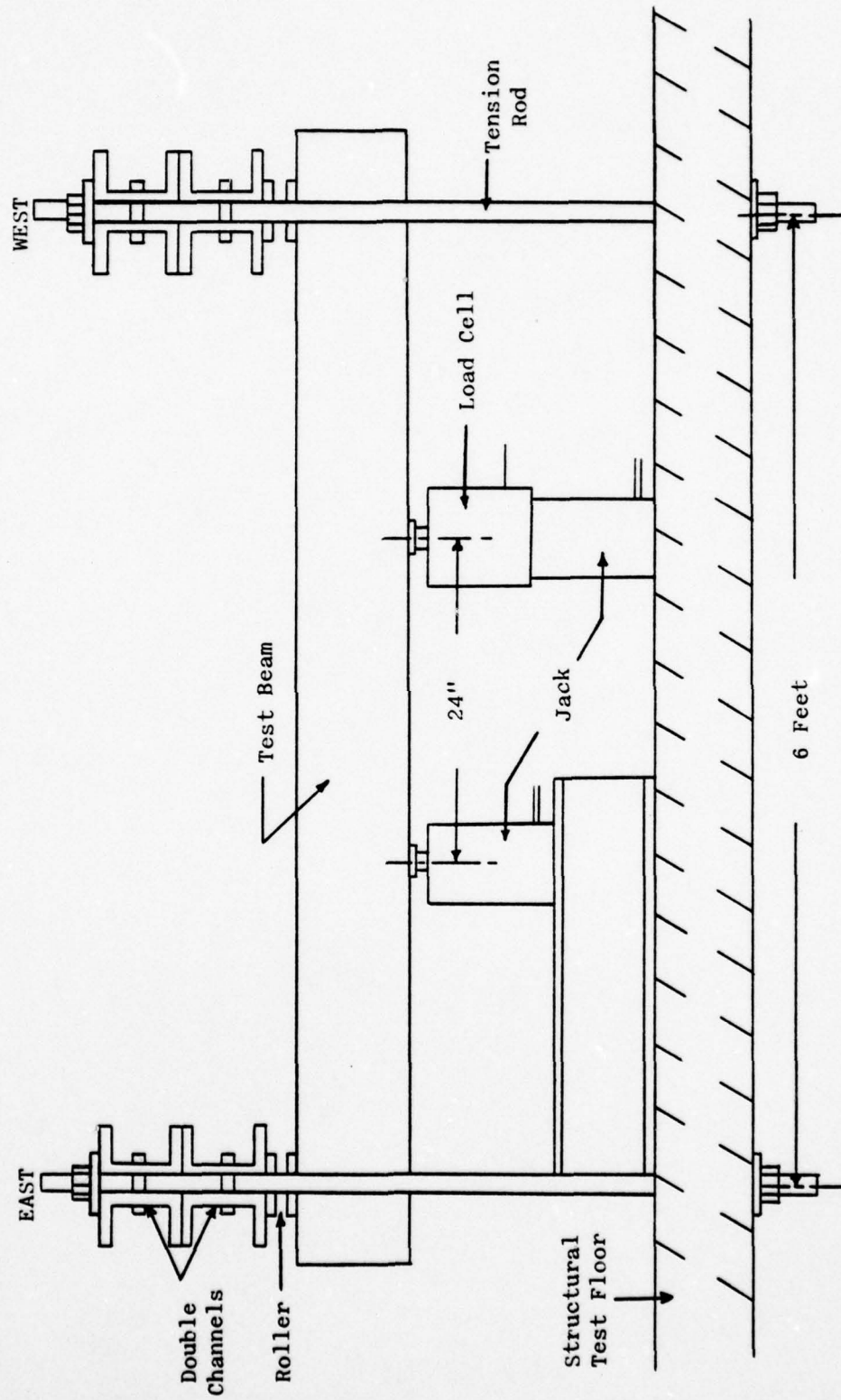
Fig. 8. Graphical test arrangement.

placing the east hydraulic jack on the top flange of a section of wide flange beam. Reactions for the six foot span were provided by connecting an arrangement of steel channels to the structural test floor by means of four 1-1/8 inch diameter steel tie rods. Free rotation of the test specimens was effected by means of rollers inserted between the top surface of the beam and the steel channels. Deflection readings were taken for the last three test sets by means of a deflection dial gauge placed underneath the specimen at midspan.

The arrangement of the apparatus for a typical test is shown in Fig. 9.
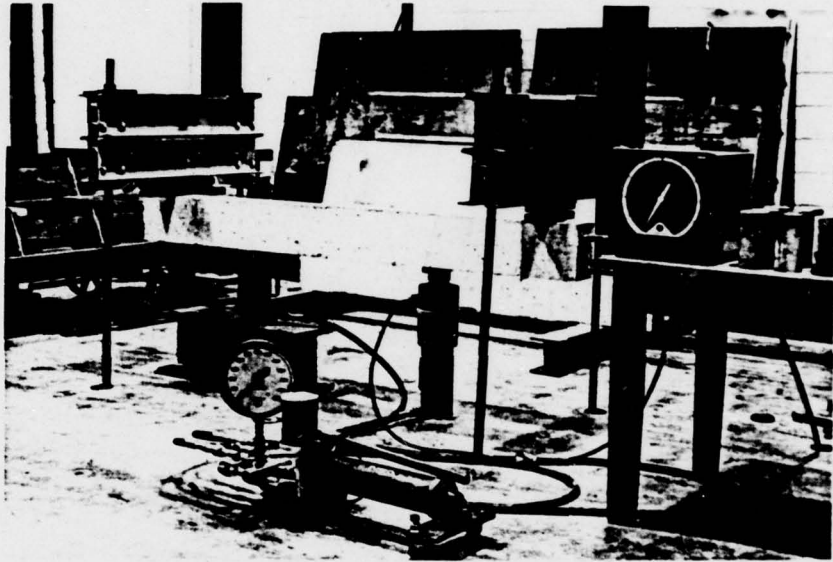
Fig. 9. Actual test arrangement.

CHAPTER III

DESCRIPTION OF TESTS AND TEST RESULTS

Results of the various tests performed and quantities calculated from them are presented in the following paragraphs.

3.1 <u>Compressive Strength Tests</u>. Compression test results for all cylinders are presented in Table IV. The strength ranged from 4475 psi to 5825 psi, and the average was 5090 psi considering 17 of the 18 cylinders tested. One cylinder was not properly capped, so its strength was not included in calculating the average.

3.2 <u>Split-Cylinder Tests</u>. As stated earlier, tensile strengths were determined using 3 x 6 inch cylinders for the split-cylinder tests. Test results for all cylinders are presented in Table V. The tensile strengths of the concrete cylinders ranged from 400 psi to 710 psi, and the average was 590 psi. The tensile strengths of the PCC cylinders ranged from 515 psi to 670 psi, and the average was 602 psi, excluding the results from test set #1.

3.3 <u>Test Beams</u>. None of the concrete control beams were expected to reach loads which would cause yielding of the steel reinforcing bars. It was anticipated that, due to the relative dimensions of the cover (½ inch on the sides and 1½ inches on the bottom), these beams would fail by side splitting of the concrete in the splice zone at stresses well below the 60 ksi specified yield stress (22). Typical failure modes are shown in Fig. 10.

TABLE IV

COMPRESSION TESTS -- CONCRETE CYLINDERS

Test Series	Cylinder Number	Length (in.)	Diameter (in.)	Cross-Sectional Area (sq. in.)	Maximum Load (lbs.)	Compressive Strength f'_c	
						(psi)	(average)
T-1	1	12.00	6.20	30.19	159,000	5270	
	2*	12.00	6.20	30.19	143,600	4760	
	3	11.98	6.24	30.58	150,300	4910	(5090)
T-2	1	12.05	6.20	30.19	153,000	5070	
	2	11.95	6.22	30.39	136,000	4475	
	3	11.98	6.20	30.19	167,500	5550	(5032)
T-3	1	11.95	6.25	30.68	154,500	5040	
	2	12.00	6.20	30.19	157,000	5200	
	3	12.05	6.22	30.39	177,000	5825	(5355)
T-4	1	12.00	6.21	30.29	154,000	5085	
	2	12.00	6.26	30.78	152,000	4940	
	3	12.00	6.20	30.19	155,000	5135	(5053)
T-5	1	12.06	6.28	30.97	160,250	5170	
	2	12.03	6.26	30.78	150,000	4870	
	3	12.07	6.23	30.48	156,000	5220	(5053)
T-6	1	11.95	6.20	30.19	154,500	5120	
	2	11.95	6.20	30.19	147,500	4890	
	3	12.00	6.30	31.17	151,500	4860	(4957)

*Poorly capped cylinder; not averaged in test results.

TABLE V

SPLIT-CYLINDER TESTS -- CONCRETE AND PCC CYLINDERS

Test Series	Cylinder Number	Length (in.)	Diameter (in.)	Ultimate Load (lbs.)	Tensile Strength (psi)	(average)	Comments
T-1	C1	5.97	2.97	18,175	655		
	C2	6.00	2.98	17,200	610		
	C3	5.99	2.98	19,575	700	(655)	
	P1	6.00	3.00	11,450	405*		old mix design
	P2	6.02	3.03	11,925	415*		old mix design
	P3	5.98	3.02	11,840	420*	(413)	old mix design
T-2	C1	5.96	2.99	15,625	560		
	C2	5.87	3.03	17,200	615		
	C3	5.96	3.01	15,450	550	(575)	
	P1	6.00	2.98	9,100	325*		heated for 2 hrs. prior to test
	P2	5.96	2.98	15,175	545		6 days moist cured
	P3	5.96	2.98	14,350	515	(530)	7 days moist cured
T-3	C1	6.05	2.97	14,750	525		
	C2	6.07	2.97	11,350	400		only ½ cylinder split
	C3	6.07	2.95	12,225	435	(453)	only ½ cylinder split
	P1	5.98	2.97	16,550	595		5 days moist cured
	P2	5.97	2.96	16,250	585		4 days moist cured
	P3	5.98	2.95	15,000	540	(573)	5 days moist cured
T-4	C1	6.00	2.96	16,300	585		
	C2	5.95	2.95	11,800	430*		unusual fracture
	C3	6.00	2.96	16,350	585	(585)	

TABLE V (Continued)

Test Series	Cylinder Number	Length (in.)	Diameter (in.)	Ultimate Load (lbs.)	Tensile Strength (psi)	(average)	Comments
T-4	P1	5.96	2.96	17,500	630		0 days moist cured
	P2	6.00	2.97	16,050	575		0 days moist cured
	P3	6.00	3.00	17,375	615	(607)	0 days moist cured
T-5	C1	5.98	2.96	14,250	515*		poorly cast specimen
	C2	5.96	2.95	17,200	625		
	C3	6.00	2.97	18,750	670	(648)	
	P1	5.97	2.95	18,500	670		0 days moist cured
	P2	5.98	2.95	18,000	650		0 days moist cured
	P3	5.98	2.95	16,500	595	(638)	0 days moist cured
T-6	C1	5.97	3.00	15,100	535		
	C2	5.97	2.95	17,625	640		
	C3	5.97	2.95	19,600	710	(628)	
	P1	5.95	2.95	17,750	645		7 days moist cured
	P2	6.00	3.00	18,000	635		7 days moist cured
	P3	6.01	3.00	17,250	610	(630)	7 days moist cured
	P4	5.95	2.95	17,500	635		0 days moist cured
	P5	6.00	2.95	17,250	620		0 days moist cured
	P6	6.00	2.95	18,000	650	(635)	0 days moist cured

*Not averaged in test results.

NOTE: The mix design for the PCC was changed following the first test. Subsequent test specimens used the mix design as listed in Appendix B.

All the PCC cylinders for Test Series 3 were heated to a temperature of $110^{o}F$ on the day prior to testing.
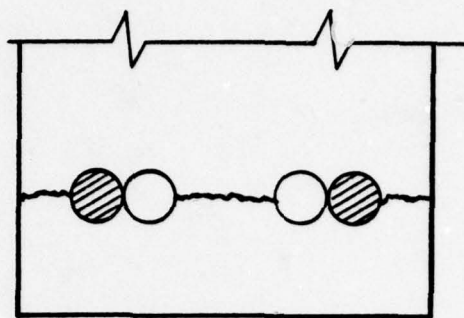
$$C_b > C_s$$

Side split failure.
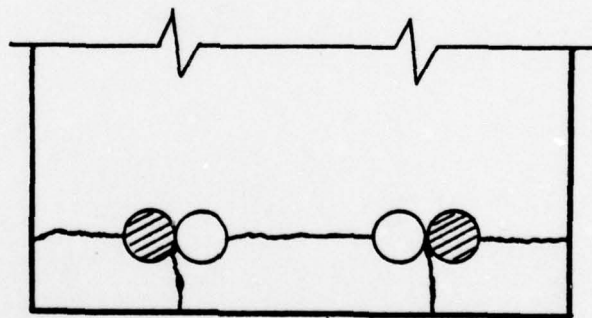
$$C_s > C_b$$

Face and side split failure.

Fig. 10. Typical failure modes.

The beams were loaded incrementally, and photographs were taken periodically to show crack progression and to record failure modes. Cracks were mapped and tick marks were made at the ends and labeled with the value of the load at each increment, and a record was maintained of significant developments. The deflection of the beams in the last three test sets was also recorded. A complete listing of these records is presented in Appendix C. Summaries of these test results are presented below:

(1)   T-1-C: This first concrete control beam was fabricated with a lap length of 16-2/3 bar diameters or 12 inches and tested with a constant moment zone of 36 inches. The beam was loaded in 1 kip per jack increments until the first flexural cracks appeared on the top surface at a value of 3 kip per jack (kpj). These cracks appeared directly above the loading jacks and did not progress vertically down the sides of the beam as might be expected. Loading continued at 0.250 kpj increments. At a load of 3.25 kpj, flexure cracks were observed in the vicinity of the ends of the splices. Diagonal tension cracks were first observed at a load of 5 kpj. Two additional flexural cracks on the top surface of the beam within the splice zone were observed at a load of 5.5 kpj. Failure of the beam occurred at a load of 8.5 kpj by side splitting of the concrete in the splice zone as expected. At no time up until failure occurred was horizontal cracking in evidence at the level of the reinforcement.

(2)   T-1-P: PCC beam T-1-P was the companion to T-1-C. It had a 12 inch lap length but it had a block of PCC encasing the

splices and running for 2½ inches beyond each end of the splice.
Load was positioned to produce a three foot constant moment section.
One flexural crack was observed directly above each of the loading
jacks at a load of 4.0 kpj. These did progress vertically down
the sides of the beam for approximately one inch. The first
cracks in the PCC block occurred at a load of 4.5 kpj in the
vicinity of the ends of the splice, but they were restricted to
the top surface only. These cracks extended vertically down the
sides of the beam approximately one inch at a load of 5.5 kpj.
Diagonal tension cracks were observed at a load of 7.0 kpj.
Failure occurred at a load of 8.0 kpj by side splitting of the PCC
in the splice zone. The failure was sudden, with only slight
longitudinal cracking in the plane of reinforcement in evidence
prior to failure.

(3) T-2-C: Concrete beam T-2-C was cast with a 10-2/3 bar
diameter, or eight inch, lap length and was tested with a 24 inch
constant moment zone. Initial flexural cracks over the loading
jacks were observed at a load of 2.5 kpj. These cracks were
restricted to the top surface only and did not extend vertically
down the sides of the beam. At a load of 3.0 kpj, flexural
cracks were observed in the vicinity of the ends of the splice.
These cracks did extend one inch vertically down the sides of
the beam. At this load the initial flexural cracks had extended
vertically down the sides of the beam. Longitudinal splitting in
the splice zone at the level of the reinforcement was first
observed at a load of 4.5 kpj. As a result of this cracking, the
load relaxed to a value of 4.3 kpj. Side split failure of the

beam occurred while an attempt was being made to reload the beam to 4.5 kpj. The failure load was recorded as 4.5 kpj.

(4) T-2-P: PCC beam T-2-P was also cast with a splice length of eight inches and tested in a 24 inch constant moment zone. Initial flexural cracks occurred directly above the loading jacks at a load of 2.0 kpj but did not extend vertically down the sides of the beam. Vertical extension of these cracks down the sides of the beam was first observed at a load of 3.0 kpj. The first cracks in the PCC were observed at the ends of the splice at a load of 4.5 kpj. They extended only slightly down the sides of the beam. Longitudinal splitting in the splice zone was observed at a load of 4.6 kpj. At 4.7 kpj the longitudinal cracks in the splice zone were observed to widen and extend horizontally along the splice length. As a result of this cracking, the load dropped to a value of 3.75 kpj. During an attempt to reload the beam to 4.7 kpj, crack progression in the splice zone caused side split failure of the beam at a load of 4.2 kpj. The progression of the cracking just prior to failure was slow and clearly visible to the eye. It can best be described as plastic flow. The failure load was recorded as 4.7 kpj.

(5) T-3-C: A lap length of 21-1/3 bar diameters, or 16 inches, was employed in concrete test beam T-3-C. The beam was loaded with a 24 inch constant moment zone. (Note: All subsequent test beams were loaded with a 24 inch constant moment zone.) At a load of 2.0 kpj flexural cracks occurred across the top surface of the beam directly above the loading jacks and extended up to

two inches vertically down the sides of the beam. At a load of 2.5 kpj flexural cracks were observed in the vicinity of the ends of the splice. An additional flexural crack was observed in the center of the splice zone at a load of 3.5 kpj. At a load of 5.25 kpj some face splitting was observed on the top surface of the beam and centered between the two splices. Longitudinal splitting in the splice zone on the sides of the beam occurred at a load of 5.5 kpj. Diagonal tension cracks were first observed near the reactions at a load of 6.0 kpj. More longitudinal splitting in the splice zone was observed at a load of 6.25 kpj. At a load of 6.5 kpj failure occurred by side splitting of the concrete in the splice zone; however, face splitting was also evident along most of the splice length on the top surface of the beam.

(6) T-3-P: PCC test beam T-3-P was the companion to beam T-3-C. Flexural cracks occurred above the loading jacks at a load of 2.0 kpj. The first cracks in the PCC were observed at a load of 5.0 kpj at the ends of the splices. Diagonal tension cracks were first noted at a load of 6.5 kpj. At a load of 7.5 kpj an additional flexure crack occurred in the PCC near the center of the splice length. This was followed by a loud "pop", but no new cracks in the PCC, at a load of 7.75 kpj. Longitudinal splitting of the sides of the beam in the splice zone began at a load of 8.25 kpj. At a load of 9.0 kpj diagonal tension cracks and bond splitting were observed along the tensile reinforcement plane in the shear spans. Failure occurred at a load of 10.0 kpj by side splitting of the PCC in the splice zone following the slow,

visible extension of the longitudinal cracks along the sides of the beam. The failure was also accompanied by a loud "pop" from the area of the splice.

(7) T-4-C: A 16 bar diameter, 12 inch, lap length was repeated in concrete beam T-4-C because the constant moment zone dimensions had been changed from 36 inches in test set #1 to 24 inches in all subsequent test sets. First flexural cracks were observed over the loading jacks at a load of 2.0 kpj. The normal progression of flexural cracking of the concrete occurred throughout the loading. Cracking at the ends of the splices was first noted at a load of 3.0 kpj. Longitudinal splitting in the splice zone was observed at a load of 5.25 kpj. Side split failure of the beam occurred at a load of 5.5 kpj.

(8) T-4-P: This PCC beam was the companion to concrete beam T-4-C. Initial flexural cracking began at a load of 2.0 kpj, but only over the west loading jack. Flexural cracking over the east loading jack occurred at a load of 3.0 kpj. Cracking continued in what had become a normal progression until a load of 4.75 kpj at which load a single flexural crack occurred at the concrete/PCC interface on the east end of the PCC block. Flexural cracks at the ends of the splice were first noticed at a load of 5.0 kpj. At a load of 6.0 kpj small longitudinal cracks on the sides of the beam in the splice zone were observed. At a load of 7.25 kpj these cracks developed short diagonal companion cracks. Failure occurred at a load of 7.75 kpj following the slow, visible progression of the longitudinal and their companion diagonal cracks on the sides of the beam in the splice zone.

(9) T-5-C: An 18-2/3 bar diameter (14 inches) splice length
was employed in concrete beam T-5-C. The normal crack progression
began with flexural cracking over the loading jacks at a load of
2.0 kpj. Longitudinal splitting in the splice zone began at a
load of 3.5 kpj. Progression of the familiar cracking pattern
continued until the beam failed by side splitting of the concrete
in the splice zone at a load of 5.75 kpj.

(10) T-5-P: PCC test beam T-5-P was cast with a 14 inch
splice length and was the companion to T-5-C. Initial flexural
cracks were noted at a load of 3.0 kpj, again, located directly
over the loading jacks. The first cracks in the PCC occurred at
a load of 5.0 kpj and were located at the ends of the splice.
These cracks were continuous across the top surface and extended
only a short distance vertically down the sides of the beam.
Very small longitudinal cracks in the PCC in the splice zone were
first noted at a load of 8.25 kpj. Failure occurred at a load of
8.75 kpj. As occurred in the previous test sets, failure was
preceded by the slow, visible extension of the small longitudinal
cracks in the splice zone. Unlike in earlier test sets, however,
the failure mode progression began with longitudinal cracks which
were much smaller, and the continuation of the crack progression
lasted much longer (approximately four minutes).

(11) T-6-C: A splice length of eight bar diameters, six
inches, was tested in concrete beam T-6-C. Flexural cracking over
the loading jacks began at a load of 2.0 kpj, and flexural cracks
at the ends of the splice were observed at a load of 3.0 kpj.
Failure occurred suddenly at a load of 3.875 kpj by side splitting

of the concrete in the splice zone. At no time up until failure occurred was longitudinal splitting evident within the splice zone.

(12) T-6-P: This beam was the companion to beam T-6-C. Initial cracking in the beam occurred at the concrete/PCC interface at a load of 2.0 kpj. Flexural cracking over the loading jacks did not occur until a load of 2.5 kpj. Flexural cracking in the PCC in the splice zone began with one crack at the west end of the splice at a load of 4.75 kpj. This was followed by a similar crack at the east end of the splice at a load of 5.0 kpj. Failure occurred at a load of 5.25 kpj following the common slow, visible side splitting of the PCC in the splice zone.

Failure loads recorded from the load cell gauge were used to compute the actual bending moment in the beam at failure. With this value, the force in the steel and the steel stress at failure were computed as:

$$T = \frac{M}{(jd)} \tag{3-1}$$

and,

$$f_s = \frac{T}{A_s} \tag{3-2}$$

wherein,

T = the tensile force in the steel,

M = actual bending moment in the beam,

jd = the distance from the centroid of the tension steel to the line of action of the resultant of compressive forces,

f_s = stress in the steel reinforcing bars, and

A_s = area of the tension steel (0.88 sq. in.).

The value of j was calculated to be 0.85. This calculation is presented in Appendix D.

The calculated average bond stress for each of the beams at failure was then determined from the following relation:

$$u = \frac{T}{\Sigma_o \ell_s} \qquad (3-3)$$

wherein,

$\quad u$ = the average bond stress,

$\quad \Sigma_o$ = the perimeter of the tension reinforcement, and

$\quad \ell_s$ = actual splice length.

Beam test results and computed moments and stresses are given in Table VI.

TABLE VI

TEST BEAM RESULTS

Specimen Designation	Splice Length (in.)	Ultimate Load (kips)	Ultimate Moment (kip-ft)	Steel Stress f_s (calc) (ksi)	Average Bond Stress u (calc) (psi)	Type of Failure
T-1-C[a]	12.0	8.500	12.75	33.43	521.8	Side split
T-1-P[b]	12.0	8.000	12.00	31.47	491.7	Side split
T-2-C	8.0	4.500	9.00	23.60	553.1	Side split
T-2-P[c]	8.0	4.700	9.40	25.65	577.7	Visible progression; side split
T-3-C	16.0	6.500	13.00	34.09	399.4	Modified face and side split
T-3-P[d]	16.0	10.000	20.00	52.45	614.6	Side split
T-4-C	12.0	5.500	11.00	28.85	450.7	Side split
T-4-P	12.0	7.750	15.50	40.65	635.1	Side split
T-5-C	14.0	5.750	11.50	30.16	403.9	Side split
T-5-P	14.0	8.750	17.50	45.89	614.6	Side split
T-6-C	6.0	3.875	7.75	20.32	635.0	Side split
T-6-P	6.0	5.250	10.50	27.53	860.0	Side split

[a]Test Series 1 used a shear span of 18 inches, all others used a 24 inch shear span.

[b]Mix design for the PCC altered after the first test.

[c]Test beam was heated to a surface temperature of 110°F for two hours prior to testing.

[d]Test beam was heated to a surface temperature of 110°F on the day prior to testing.

CHAPTER IV

ANALYSIS AND DISCUSSION OF RESULTS

Test results are discussed and analyzed in the paragraphs
which follow.

4.1 <u>Compression Tests of Concrete</u>. The eight-day compressive
strengths, f_c', of the 6 x 12 inch cylinders ranged from 4475 psi
to 5825 psi, with an overall average of 5090 psi. This is a bit
higher than the approximately 4500 psi strength used by other
researchers (12), but it is still an acceptable level of strength
often obtained in job-cast concrete.

4.2 <u>Tensile-Splitting Tests of Concrete</u>. The eight-day
splitting strengths, f_{sp}, of the 3 x 6 inch concrete cylinders
ranged from 400 psi to 710 psi, with an average of 590 psi. This
is 8.27 $(f_c')^{1/2} = 5090$ psi. If the f_c' from the 6 x 12 is converted
to the f_c' for 3 x 6 inch cylinders using the conversion factor of
1.06 recommended by Troxell and Davis (28), one obtains $f_c' = 5395$
psi and f_{sp} is then 8.03 $(f_c')^{1/2}_3 \times 6 = 5395$ psi.

Although ACI 318-71 implies that $f_{sp} = 6.7\sqrt{f_c'}$ for normal
weight concrete, it is said that the tensile strength is a more
variable property than the compressive strength and should range
from 10 - 15% of it (13). The values of the previous paragraph
fall within this range.

4.3 <u>Tensile-Splitting Tests of PCC</u>. The eight-day tensile
splitting strengths, f_{sp}, of the 3 x 6 inch PCC cylinders ranged

from 515 psi to 670 psi, with an average of 602 psi. This is only three percent higher than the average for the concrete, or it is essentially the same. This result was disappointing, and various efforts were made to increase the tensile strengths of the PCC cylinders as discussed in Chapter II.

It was expected that the strength of the PCC mix would be lower than the 8000 psi listed as typical for the neat system since the system would be extended by the mix water and "contaminated" with the aggregate. The formulator estimated that the ultimate tensile strength of the PCC would be in the 1000 - 1500 psi range. Various opinions have been expressed as to the cause of the supposed strength "loss", but none have been proven to be true or false (18). A supplementary investigation (29) on the PCC used in this investigation found only limited strength gain past age seven days and determined the modulus of elasticity to be approximately 1.16×10^6 psi. Consultation with experts in the concrete technology field at the U. S. Army Corps of Engineers Waterways Experimental Station (18) indicated that the results obtained in this investigation are consistent with numerous tests conducted at that facility and seemed typical of PCC results in general. Further, it has been determined that these results are consistent with other researchers in this area (7). Correlation is particularly good with the results of test series CV conducted by Sun, Nawy, and Sauer (27). That test series was run on a mix with a water/cement ratio of 0.25 and a resin/cement ratio of 0.422, and it resulted in an average seven-day tensile strength of 675 psi. The current investigation used a PCC mix with a water/cement ratio

of 0.27 and a resin/cement ratio of 0.541 which resulted in an average seven-day tensile strength of 602 psi. Differences in the chemical composition of the two epoxy systems used could account for this slight difference.

4.4 Concrete Test Beams. The performance and splice strengths of the reinforced concrete control beams were as anticipated. All beams failed by side splitting of the concrete as expected. One beam, T-3-C, also exhibited some face splitting, although it was not consistent with normal face split crack progression. It should be noted that this beam had the longest lap length tested, 16 inches, and thus had the lowest average bond stress, but the cover and the splice spacing were the same as for all other beams. Crack progression was mapped and recorded during each test as stated in Chapter III. The crack progression showed little, if any, deviation from the normal progression described by ACI Committee 408 for tension lap splices in a constant moment section (4). This crack progression is outlined below:

(1) Cracks first occur in the vicinity of the ends of the splice and progress down the sides of the beam (assuming tension side up).

(2) Horizontal splitting of the concrete proceeds from these end cracks toward the center of the splice along the reinforcement plane.

(3) Supplementary flexure and splitting cracks may form between the end cracks.

(4) Failure occurs suddenly, sometimes explosively, as the remaining 20 - 40 percent of the splice length concrete cracks and splits.

This sequence was generally observed in all of the control beams. It is illustrated for test beam T-4-C in the series of photographs in Fig. 11 through Fig. 14. The resulting failure modes for the other control beams are presented in Fig. 15 - 19. The face splitting in test beam T-3-C is shown in Fig. 20. Notice that only one face crack is evident, almost centered between the two vertical planes of the splices. Normal progression of cracks in a face split failure would have included a crack on the face of the beam over each splice (see Fig. 10).

As anticipated, the steel stress in the concrete control beams never exceeded the yield stress. The bond strengths found in these tests is accurately predicated by an equation recently proposed by Orangun, Jirsa and Breen (21). The equation relates concrete strength, cover, bar diameter, and length of lap to the average ultimate bond stress. The relation is:

$$u/\sqrt{f'_c} = 1.2 + 3.0 \; C/d_b + 50.0 \; d_b/\ell_s \qquad (4\text{-}1)$$

wherein,

u = the average ultimate bond stress in splices in a constant moment region (psi),

C = the smaller of the clear bottom cover, C_b, or half the clear spacing between bars or splices, C_s,

d_b = bar diameter,

ℓ_s = the splice length, and

f'_c = the concrete compressive strength.
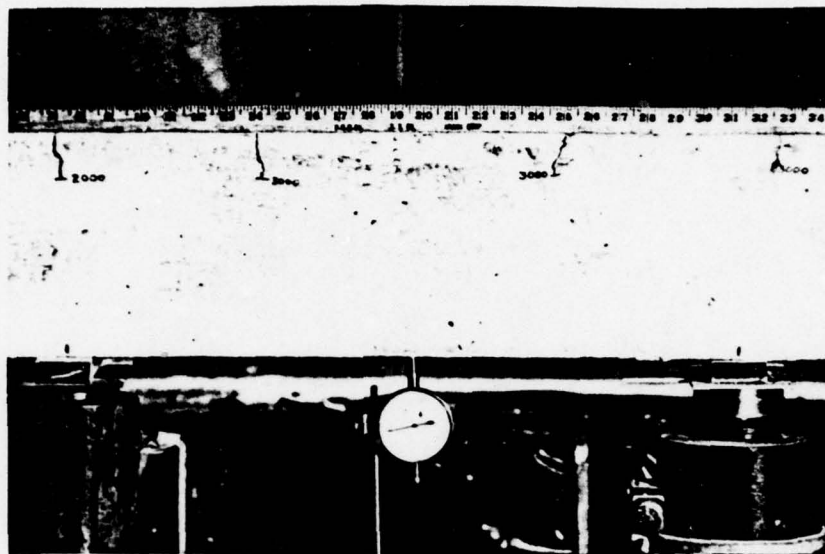
Fig. 11. Premature flexure cracks at the ends of the splice (14 and 25 inch marks). Beam T-4-C at a load of 3.0 kpj with l_s = 12 inches.
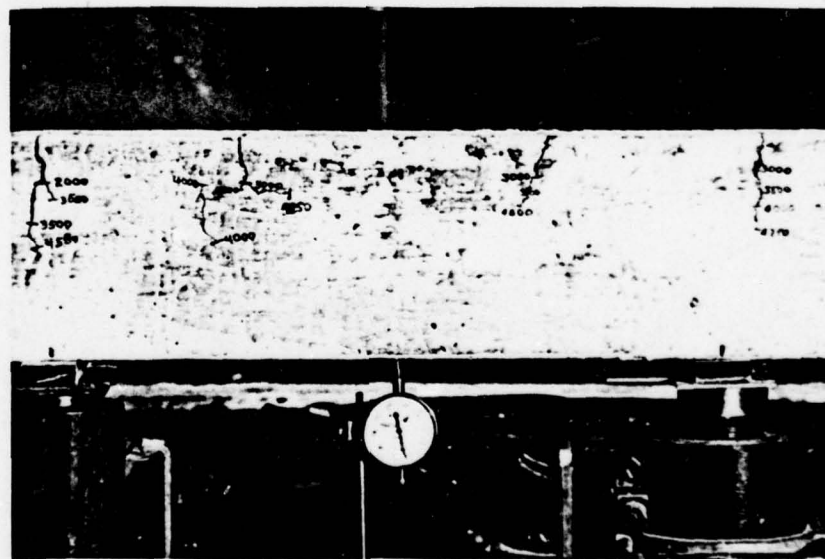
Fig. 12. Horizontal splitting of the concrete along the reinforcement plane. Beam T-4-C at a load of 5.25 kpj.

Fig. 13. Lateral cracking of concrete extends the
full length of the splice zone. Beam
T-4-C at a load of 5.50 kpj.

Fig. 14. Close-up view of failure of Beam T-4-C.
Ultimate load is 5.50 kpj.
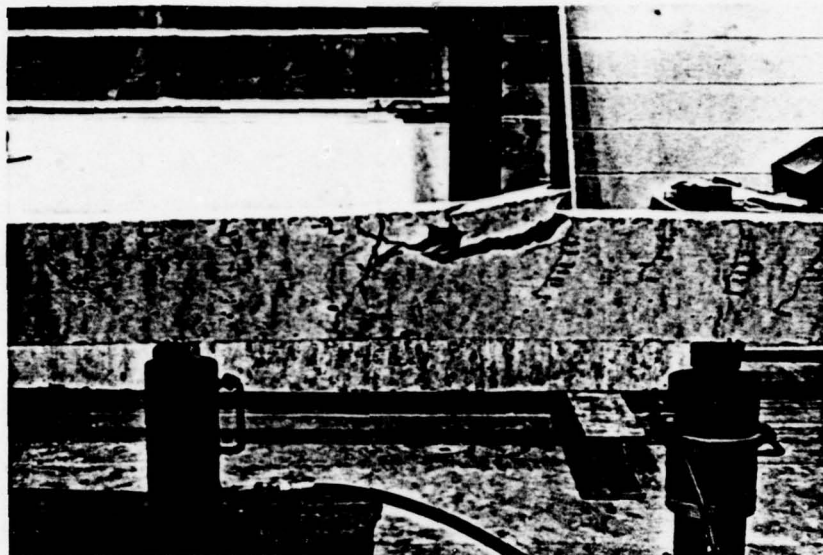
Fig. 15. Failure of Beam T-1-C. Splice length = 12 inches. Ultimate load = 8.5 kpj.
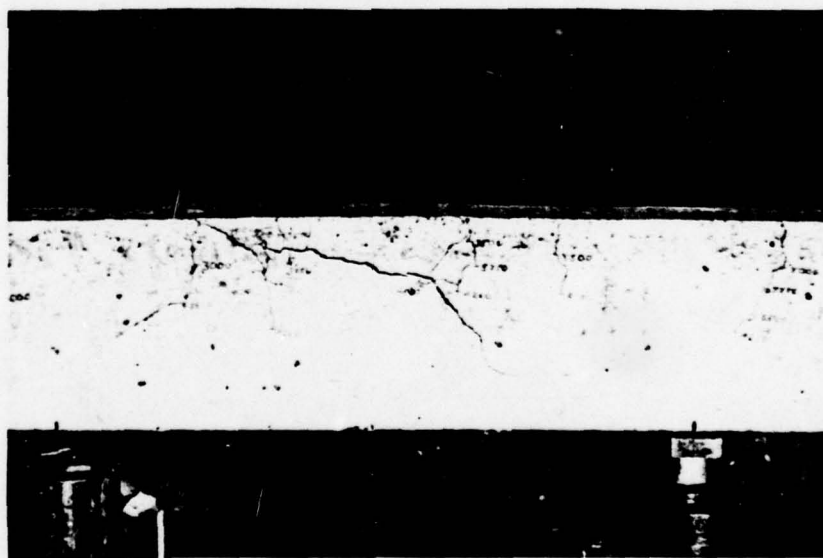
Fig. 16. Failure of T-2-C. Splice length = 8 inches. Ultimate load = 4.5 kpj.

Fig. 17. Failure of T-3-C. Splice length = 16 inches.
Ultimate load = 6.5 kpj. Note the supplementary
flexure cracks near the third points of the
splice length.

Fig. 18. Failure of T-5-C. Splice length = 14
inches. Ultimate load = 5.75 kpj.
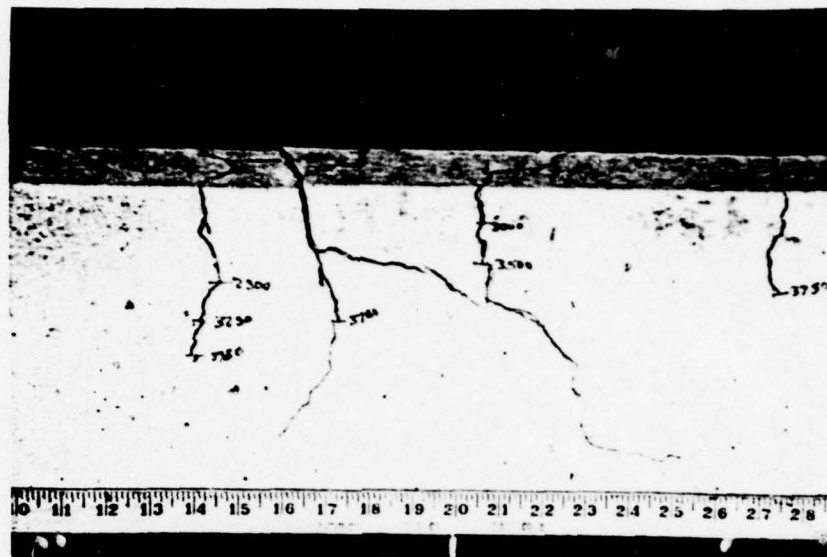
Fig. 19. Failure of T-6-C. Splice length = 6
          inches. Ultimate load = 3.875 kpj.
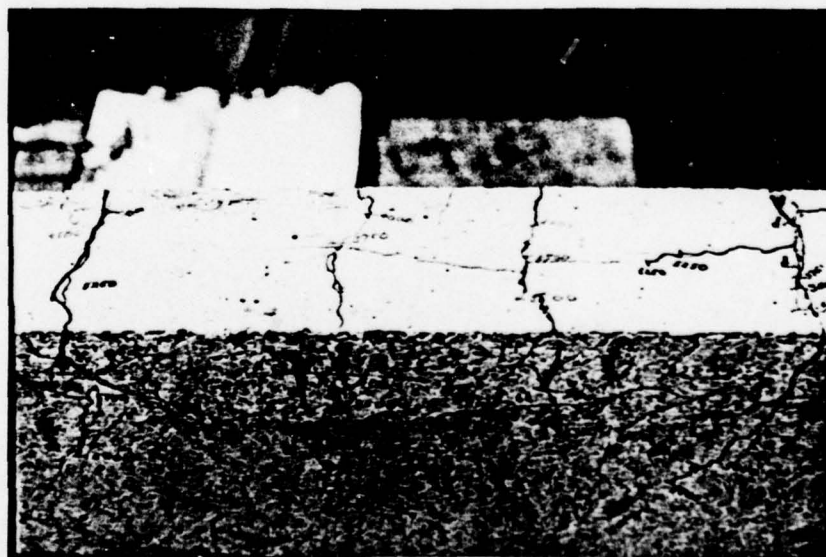
Fig. 20. Modified face split failure of Beam T-3-C.
          Note the single face crack running the
          length of the splice and nearly centered
          between the splices.

The test results for average ultimate bond stress calculated from equation 3-3 showed excellent correlation with equation 4-1. Calculated values are presented in Table VII.
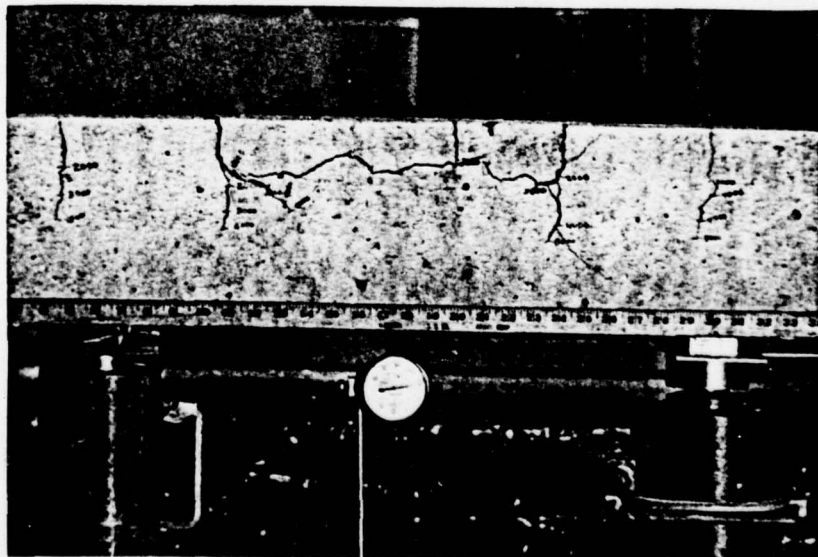
Equation 4-1 can be modified to predict the development length (critical lap length) of the reinforcement steel. If we let $f_s = f_y$ in the relation

$$u = \frac{f_s d_b}{4 \ell_d} \; ,$$

we can write equation 4-1 as

$$\frac{f_y d_b}{4 \ell_d (f_c')^{\frac{1}{2}}} = 1.2 + 3.0 \; C/d_b + 50.0 \; d_b/\ell_d \; ,$$

which yields

$$\ell_d = \frac{d_b \left[\frac{f_y}{4(f_c')^{\frac{1}{2}}} - 50.0 \right]}{(1.2 + 3.0 \; C/d_b)} \; . \tag{4-2}$$

Using f_y = 60,000 psi and d_b = 0.75 inch for this investigation, and letting f_c' = 5100 psi and C/d_b = 2/3, the predicted development length can be calculated as ℓ_d = 37.5 inches.

Values for the steel stress, f_s, calculated from the test results, are plotted against the actual lap length in Fig. 21. By assuming the results can be extrapolated linearly to other lap lengths, we predict from the graph a development length of approximately 36.1 inches for f_y = 60 ksi. This is excellent agreement with the calculated value of 37.5 inches using equation 4-2.

4.5 <u>PCC Test Beams</u>. The objective of this study was to determine the strengths of tension lap splices in Polymer Cement

TABLE VII

COMPARISON OF CALCULATED BOND STRESS AND TEST VALUES

Test Beam	ℓ_s (in.)	f_s (test) (ksi)	u_t (test) (psi)	f'_c (psi)	$u_t/\sqrt{f'_c}$	$u*/\sqrt{f'_c}$	$u_c/\sqrt{f'_c}$	$u*/u_t$	u_c/u_t
T-1-C	12	33.4	521.8	5090	7.31	6.70	6.33	.916	.866
T-2-C	8	23.6	553.1	5032	7.80	8.35	7.89	1.070	1.011
T-3-C	16	34.0	398.4	5353	5.45	5.86	5.54	1.075	1.016
T-4-C	12	28.8	450.7	5050	6.33	6.70	6.33	1.058	1.000
T-5-C	14	30.1	403.2	5053	5.67	6.22	5.88	1.097	1.037
T-6-C	6	20.3	634.4	4957	9.01	10.01	9.45	1.110	1.048

NOTE: $u*/\sqrt{f'_c} = 1.22 + 3.23\ C/d_b + 53.0\ d_b/\ell_s$ (best fit equation)

$u_c/\sqrt{f'_c} = 1.20 + 3.0\ C/d_b + 50.0\ d_b/\ell_s$ (simplified equation)
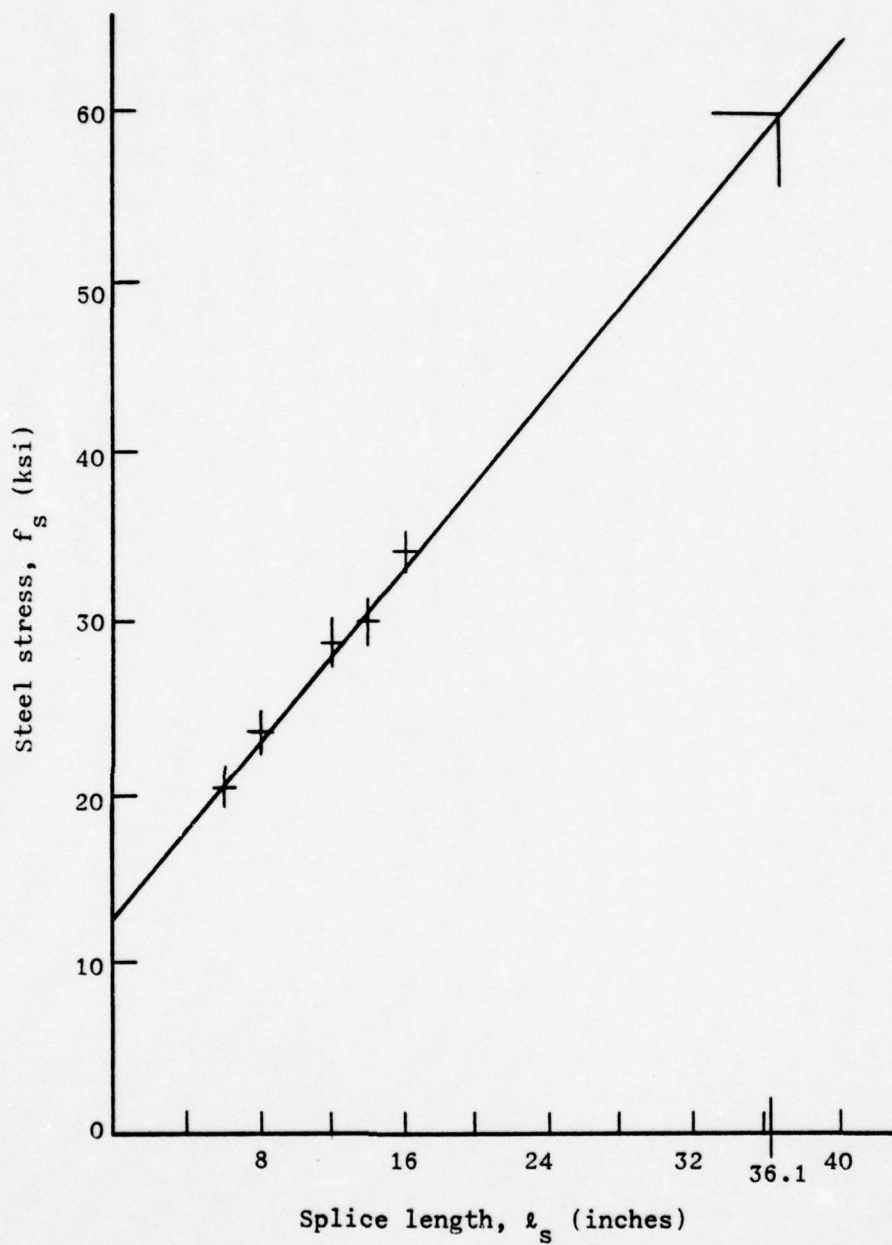
Fig. 21. Steel stress versus actual lap length in
the concrete beams.

Concrete. The ultimate strength desired of the splice is that which would develop any desired stress in the steel reinforcing bars prior to failure of the splice-zone concrete. The most economical splice design, therefore, would be one which would use a lap length only sufficient to ensure the full stress in the steel could be realized, with possibly some slight reserve, prior to failure of the concrete. The development length for bars acting at yield stress is

$$\ell_d = \frac{f_y d_b}{4 \, u_u}$$

where u_u is the ultimate bond stress capacity of the concrete. This formula could not be applied to the PCC since its bond strength was not known, and therefore a development length in the PCC could not be predicted prior to testing. The determination of the development length of reinforcement in the PCC became an important by-product of this investigation.

Test results for the PCC beams are presented in Table VI. With the exception of test set #1, all PCC beams showed marked improvement in overall load carrying capacity over the concrete control beams. The results of test beam T-2-P were only 4.5 percent above the control beam, but this is directly attributable to the heating of the beam within two hours prior to testing (17). Although too much heat can have a detrimental effect on the epoxy, heat ensures complete curing. Test beam T-3-P was also heated, but only on the day prior to testing. It showed a 54 percent increase in the load carrying capacity over the control beam.

None of the PCC test beams reached a load which would cause yielding of the steel. However, the development length required to yield a #6 Grade 60 bar in the PCC can be predicted from the test results. Fig. 22 is a graph of the calculated steel stress at failure as a function of the actual lap length of the splice in the PCC beams. From this graph a development length of 19.6 inches is predicted for a #6 Grade 60 reinforcing bar.

As noted in Chapter III the manner in which the PCC beams failed demonstrated that the material was much less brittle than concrete. This fact is further demonstrated by the crack progression pattern exhibited by the PCC test beams. All of the failures were characterized by a slow visible progression of the horizontal splitting cracks along the reinforcing plane. Such a progression is illustrated in the series of photographs in Fig. 23 through Fig. 28. The photographs in Fig. 24 - 27 were taken in rapid succession while the beam was held at a constant applied load. Another characteristic of the crack progression is the slight diagonal direction of the side splitting cracks formed prior to failure. These cracks appeared at a regular interval throughout the splice length. Failure occurred when these cracks "grew" and intersected adjacent cracks, thus completing the splitting of the entire splice length. These cracks are shown on test beam T-3-P prior to failure in Fig. 29 and at failure in Fig. 30.

Crack pattern at failure loads for the other PCC test beams are shown in Fig. 31 through 34. Note the zipper-like effect of the diagonal splitting cracks along the reinforcement plane in each of the failures.
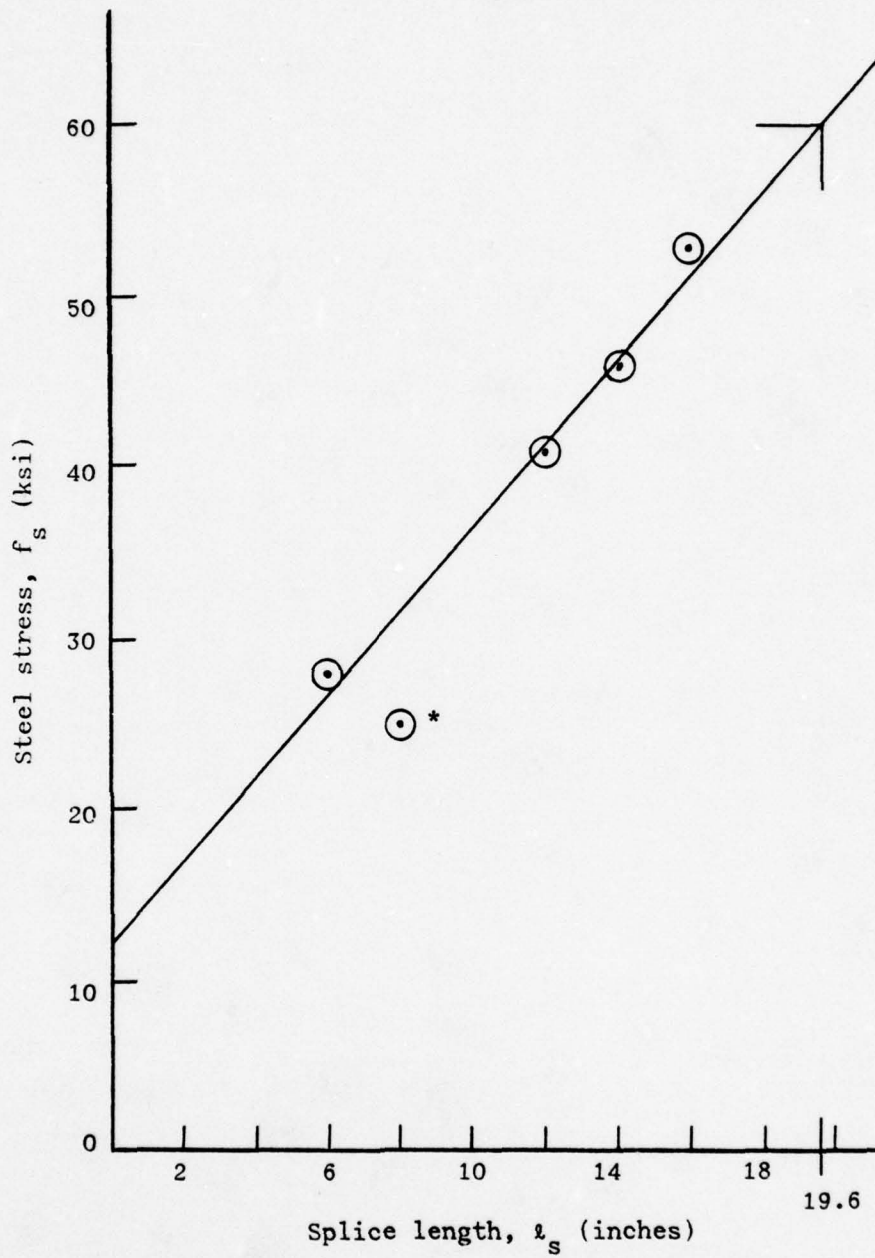
*Not considered in regression.

Fig. 22. Actual steel stress versus lap length in
the PCC beams.

Fig. 23. Beam T-2-P at a load of 4.6 kpj just
after horizontal cracking became
evident.
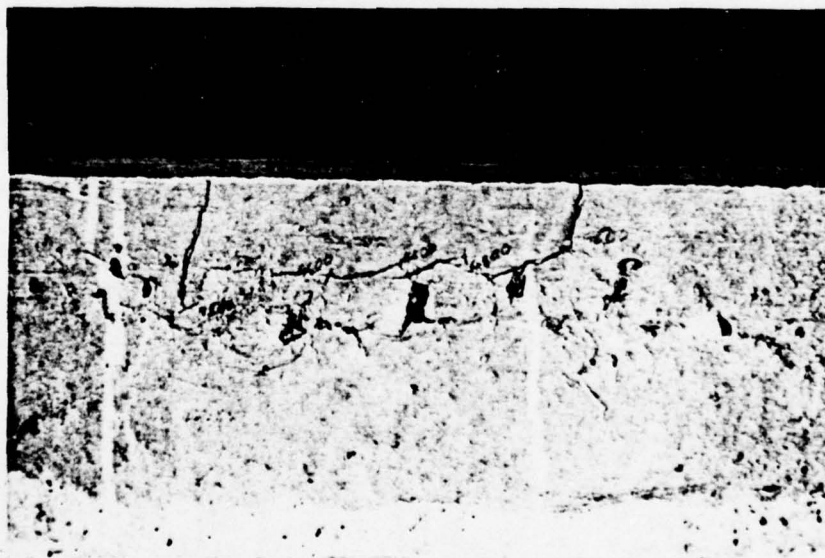
Fig. 24. This photograph was taken at an applied load
of 4.7 kpj. Note the extension of the
horizontal cracks.
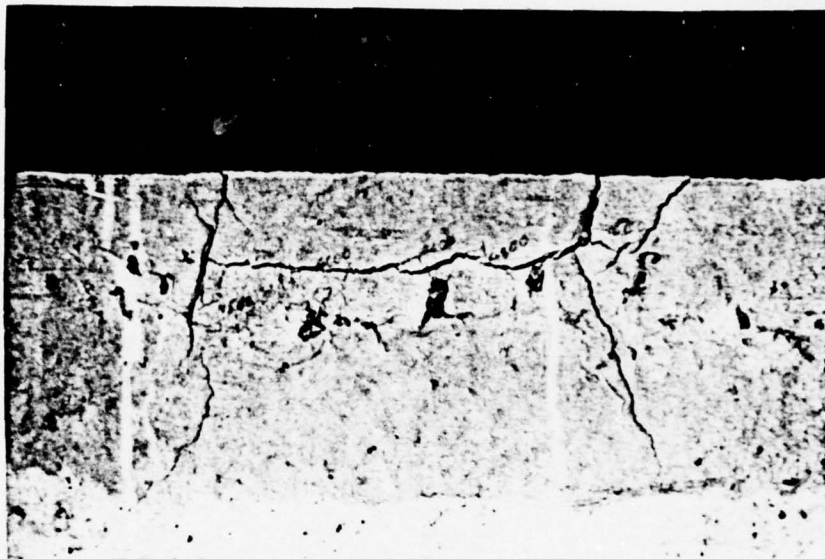
Fig. 25. Under a constant load, the beam reacts slowly enough
for these pictures. The horizontal cracking is
complete on the exterior of the splice zone.

Fig. 26. The beam continues to deform with the PCC material in
the splice zone visibly lifting from the beam. Note
the extension of the vertical cracking from the splice
zone to the compression steel.

Fig. 27. This is the final position of the crack formation for Beam T-2-P. The block of PCC which was lifted from the splice zone was not loose and could not be picked up from the beam.
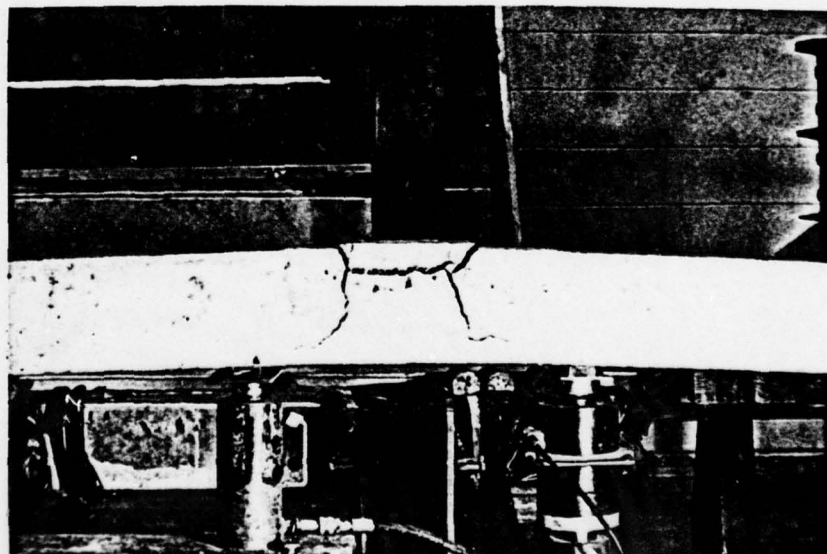
Fig. 28. View of the deflection of Beam T-2-P at failure. Ultimate load = 4.7 kpj. Splice length = 8 inches.
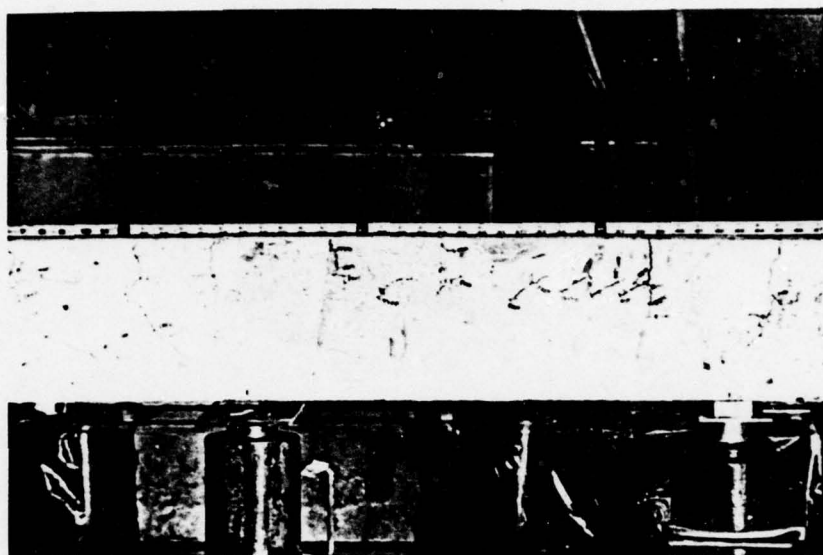
Fig. 29. A view of Beam T-3-P at a load of 9.75 kpj. Note the disposition and spacing of the side splitting cracks in the splice zone.
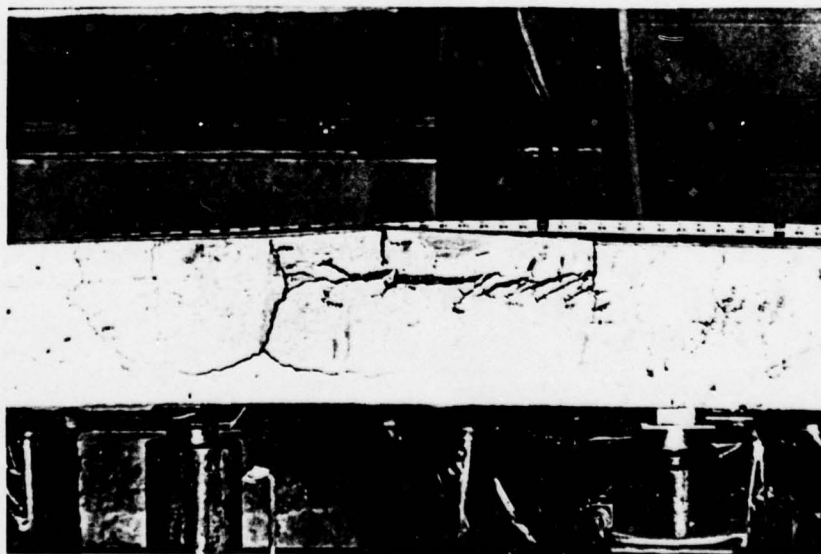
Fig. 30. Failure of Beam T-3-P. Ultimate load = 10.0 kpj. Splice length = 16 inches. Note the extension of the side splitting cracks and their zipper-like action on the PCC.
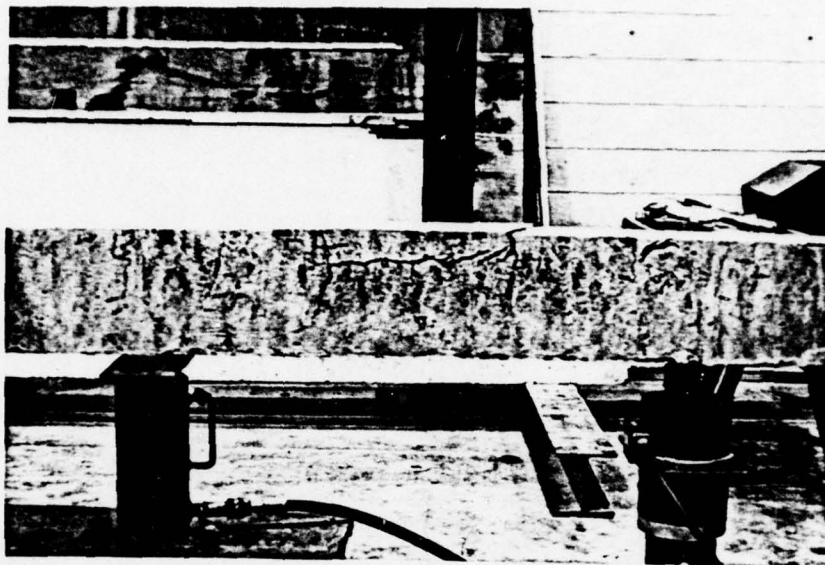
Fig. 31. Failure of Beam T-1-P. Ultimate load = 8.0 kpj.
Splice length = 12 inches.
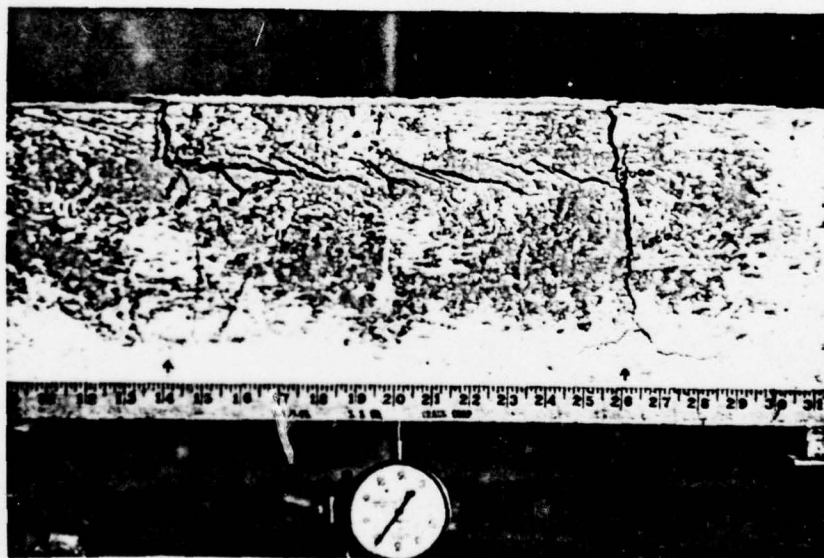
Fig. 32. Beam T-4-P at failure. Ultimate load = 7.75 kpj.
Splice length = 12 inches. Note the disposition
of the side splitting cracks.
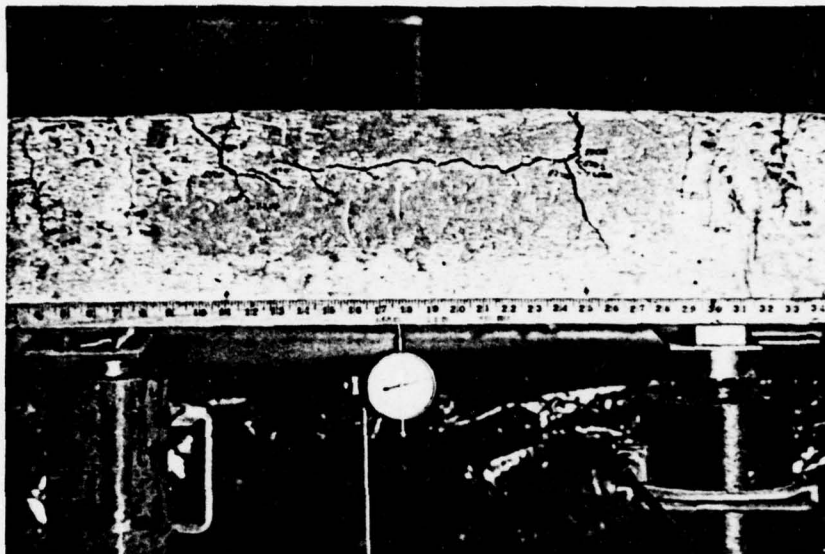
Fig. 33. Failure of Beam T-5-P. Ultimate
load = 8.75 kpj. Splice length =
14 inches.
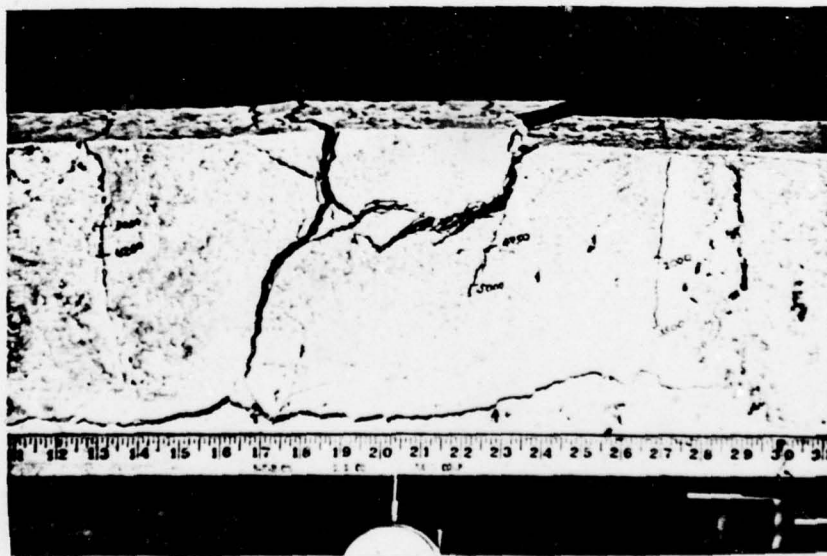
Fig. 34. Failure of Beam T-6-P. Ultimate
load = 5.25 kpj. Splice length =
6 inches.

4.6 <u>Comparison of the Concrete and PCC Test Beams</u>. In order
to determine the relative strength increase of the tension lap
splice in the PCC over ordinary concrete we must compare certain
of the test results. As had already been noted the PCC beams
showed marked increases in ultimate load carrying capacity over
identical beams of ordinary reinforced concrete. But the analysis
must go further than that. For ease in comparison, test data
presented in several tables has been consolidated in Table VIII.

For equal lengths of lap it can be seen that the PCC beam
results in a significantly greater bond stress. In fact,
disregarding test series one and two for reasons already indicated,
it can be seen that the average bond stress increases by an
average 209 psi in the PCC beams. This indicates that the PCC
possesses an increased bond stress capacity. A graphical
representation of the results (Fig. 35) clearly illustrates this
advantage. Although the PCC showed very little increase in
tensile strength over the concrete, the ratio of bond stress to
tensile strength, u/f_{sp}, in the PCC is almost uniformly greater
than the same ratio of concrete values.

Since the required development length and the bond stress are
reciprocal relations, a reduction in the required development
length is clearly indicated. Recall that the test results indicate
(Fig. 20) that a lap length of 36.1 inches is required to yield
the Grade 60 steel reinforcement in the concrete beam. Current
ACI provisions require a minimum lap length of 30.6 inches for a
#6 Grade 60 bar, while inserting the values of f_c', C and d_b of the
present tests in equation 4-2 gives a required lap length of 37.5

TABLE VIII

COMPARISON OF TEST RESULTS

Test Beam	ℓ_s (in.)	Ultimate Load (lbs.)	f_s (test) (ksi)	u (test) (psi)	f_{sp} (psi)	u/f_{sp}
T-1-P	12[a]	8000	31.47	491.7	413[b]	1.190
T-1-C	12[a]	8500	33.43	521.8	654	0.798
T-2-P	8	4700	24.65	577.7	528[c]	1.094
T-2-C	8	4500	23.60	553.1	573	0.965
T-3-P	16	10000	52.45	614.6	567	1.084
T-3-C	16	6500	34.09	399.4	525	0.761
T-4-P	12	7750	40.65	635.1	607	1.046
T-4-C	12	5500	28.85	450.7	585	0.770
T-5-P	14	8750	45.89	614.6	638	0.963
T-5-C	14	5750	30.16	403.9	601	0.672
T-6-P	6	5250	27.53	860.0	634	1.356
T-6-C	6	3875	20.32	635.0	628	1.011

Notes: [a]Test series one used a shear span of 18 inches, all others used a shear span of 24 inches.

[b]Mix design for the PCC was altered after this test.

[c]Beam was heated within two hours prior to testing.
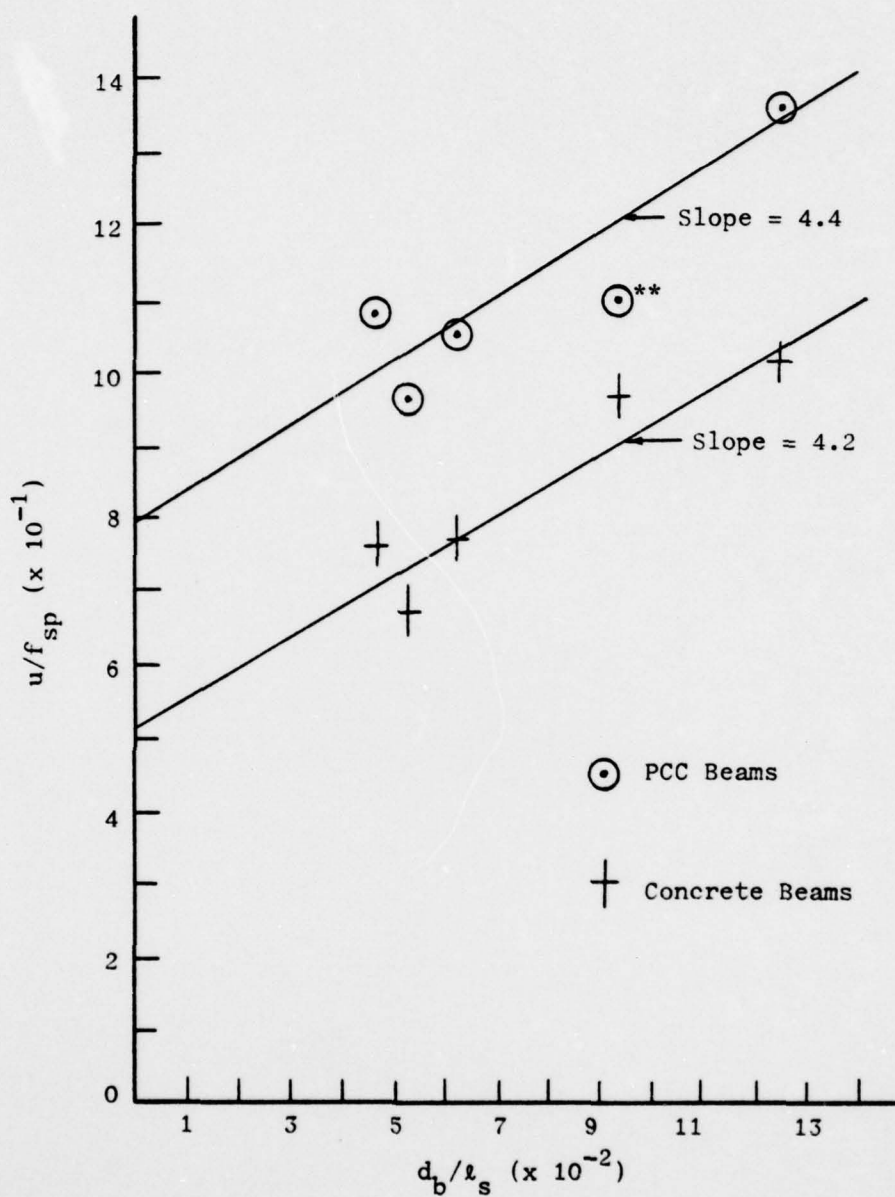
NOTE: **Test beam T-2-P, not considered in regression.

Fig. 35. Variation of u/f_{sp} with d_b/ℓ_s for all test beams.

inches. Comparing these with the predicted development length of 19.6 inches in the PCC shows, for beams similar to the test specimens, a savings of eleven inches of reinforcement per splice over the ACI provisions, while eighteen inches per splice can be saved over the development length by equation 4-2.

The reduction of the development length in the PCC is not accompanied by a loss of ductility as would be expected in normal reinforced concrete construction. All PCC specimens exhibited a ductility which is not typical of ordinary concrete. The slow crack progression, which was evident in each PCC beam at failure, is an example of the atypical physical properties of the PCC.

In a brittle material, such as concrete, we expect a high concentration of bond stress at the end of the splice as shown in Fig. 36. This stress then tapers off along the bar toward the other end of the splice. When the effects of the bond stress exceed the tensile strength of the concrete, splitting occurs along the plane of reinforcement. In a tension lap splice the tensile strength of the concrete is exhausted first at the ends of the splice (in the areas of high bond stress concentrations) and longitudinal splitting initiates here. As the stress is redistributed along the splice length, splitting progresses from the ends toward the center of the splice.

The failure mode of the PCC beams began with small longitudinal splitting cracks distributed along the entire splice length. Failure occurred as these cracks grew and intersected adjacent cracks, thus completely splitting the PCC in the splice zone.
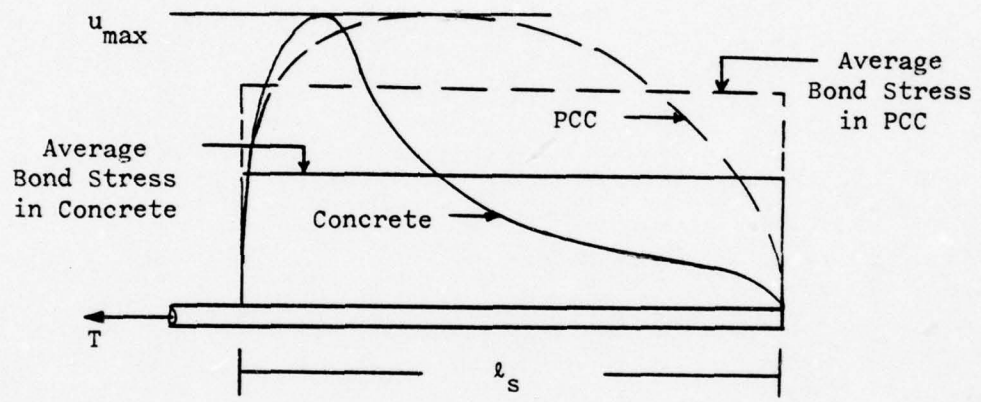
Fig. 36. Bond stress distribution.

Based upon the initial crack formation, we can postulate that the bond stress is more evenly distributed along the splice length in the PCC. This hypothesis is shown as a dashed line in Fig. 36.

Thus, since the tensile strengths of the two materials are essentially the same, the increased average bond stress capacity seemingly exhibited by the PCC does not mean that it has a higher ultimate bond stress but that it distributes the bond stress more evenly along the splice length. The increase in the average bond stress in the PCC over the concrete can be considered to be the difference in areas under the PCC and concrete curves in Fig. 36.

CHAPTER V

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

5.1 Summary. This investigation was performed to determine the relative strength increases of tension lap splices cast in a Polymer Cement Concrete over similar splices in conventional Portland cement concrete. In order to evaluate this relative strength, six sets of test beams were cast. Each set consisted of an ordinary reinforced concrete beam containing two tension splices and another beam, identical except that the concrete surrounding the splices was Polymer Cement Concrete. Companion test cylinders, used to determine concrete compressive strength and the split-cylinder tensile strength of both the PCC and the concrete, were also cast as part of each test set. The splice length varied from one test set to another, but each splice was tested in a constant moment zone.

Tests of the beam were performed on the University of Colorado structural test floor with the load being applied by means of two hydraulic jacks. Each beam was positioned, tension side up, on a simple span of six feet. Reactions were provided by a system of steel channels positioned normal to the span and secured to the floor by steel tie rods. Rollers, positioned under each reaction, prevented restraint of rotation of the ends of the test specimen during testing. Symmetrical two point loading was

applied and increased incrementally to permit mapping and recording of crack progression.

All beams failed by side splitting at steel stresses well below yield stress. Crack progression in the concrete beams followed a pattern which began with flexural cracks at the ends of the splice, followed by horizontal cracking of the concrete along the reinforcing plane. Additional flexural cracks also routinely appeared between the splice ends, with companion horizontal cracks along the reinforcing plane. Failure occurred suddenly as the remaining concrete split horizontally along the entire splice length.

Crack progression in the PCC beams differed slightly from that of the concrete beams. Following the appearance of the flexural cracks at the ends of the splice, longitudinal side splitting cracks appeared as in the concrete beams, but these were short, displaced diagonally, and spaced along the entire splice length, usually with no additional flexural cracks evident. Failure occurred when these diagonal cracks "grew", slowly and visibly, while under constant load, and connected with adjacent cracks, thus completing the splitting along the entire PCC splice length.

Material strengths and splice strengths were compared for each test series.

5.2 <u>Conclusions</u>. The data obtained from the test results, when analyzed, permitted the formulation of the following conclusions:

(1) The Polymer Cement Concrete used showed no appreciable increase in tensile strength over Portland cement concrete at age eight days. This result is consistent with previously published test reports on Polymer Cement Concretes.

(2) Although the tensile strengths of the two materials tested were very close, the PCC beams exhibited an increased splice strength and a considerably greater bond stress capacity, for equal lengths of lap, than did the concrete control beams.

(3) Considerable savings in the length of lap required to reach yield strength of the steel can be realized by using the Polymer Cement Concrete in the splice zone. This savings is even greater when compared with the development length predicted by the equation presented by Orangun, Jirsa and Breen.

(4) Beams with a block of PCC around the spliced reinforcement exhibit a greater ductility than similar concrete control beams.

(5) A review of the test results indicates that splice strength is greatly influenced by the material surrounding the splice.

5.3 Recommendations for Further Study. This investigation was only a limited study into the strength of tension lap splices in Polymer Cement Concrete, but the results could have far-reaching implications. Before practical application of the conclusions of this study can be implemented in field use, however, additional research is suggested. The following recommendations for further study are offered:

(1) Investigate a similar application of an epoxy which has been specifically formulated for use in bulky specimens

such as beams. Most systems are now formulated for use as toppings or coatings.

(2) Determine the curing time and conditions required for any type of polymer used in a similar application.

(3) Conduct a study of the effects of cover and bar diameter on the strength of spliced reinforcement in a PCC.

(4) Perform an analysis of the cost effectiveness of the use of an epoxy in an application similar to the current investigation.

(5) Determine the optimum dimensions of the PCC block with respect to bar diameter and lap length.

(6) Study the effect of the depth of the PCC block on the strength of the lapped splice.

(7) Conduct a complete study into the physical properties of the PCC to include the effect of temperature, age, and mixing procedure.
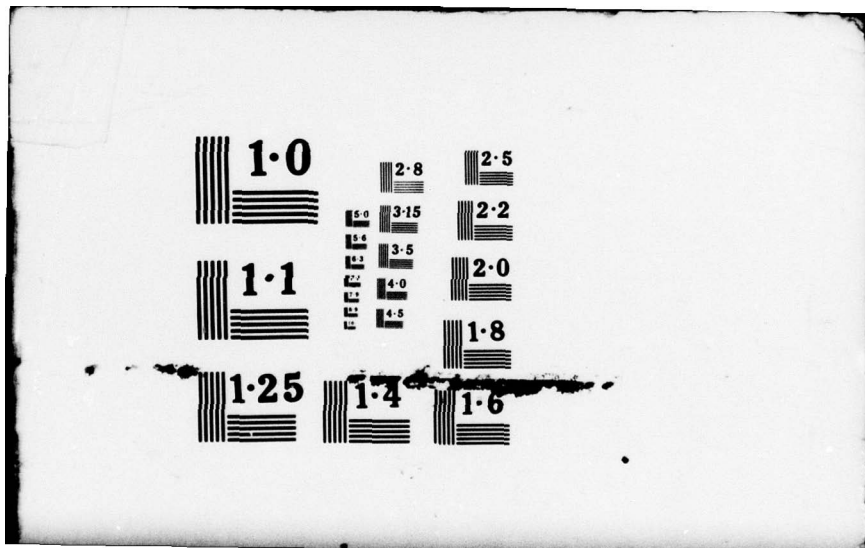
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APPENDIX A

NOTATION

A_b = cross sectional area of longitudinal steel bar, inch^2

A_s = area of longitudinal tension reinforcement, inch^2

A_s' = area of longitudinal compression reinforcement, inch^2

a = depth of equivalent rectangular stress block, inch

b = width of compression face of flexural member, inch

c = distance from the extreme compression fiber to the neutral axis, inch

C = the smaller of C_b or $(1/2)C_s$

C_b = clear bottom cover to main reinforcement, inch

C_c = compressive force in concrete, lbs or kips

C_s = clear spacing between bars or splices, inch

C_s' = compressive force in compression reinforcement, lbs or kips

d = distance from extreme compression fiber to centroid of tension reinforcement

d_b = diameter of main reinforcement

d' = distance from extreme compression fiber to centroid of compression reinforcement

E_c = modulus of elasticity of concrete, psi

E_s = modulus of elasticity of steel, psi

f_c' = specified or actual compressive strength of concrete, psi

f_s = calculated steel stress in reinforcement, psi

f_{sp} = tensile-splitting strength of concrete, psi

f_y = specified yield strength of longitudinal reinforcement, psi

j = ratio of distance between compressive and tensile force to the depth, d

ℓ_d = development length, inch

ℓ_s = splice length, inch

M = moment, ft-kips or in-kips

M_u = ultimate moment, ft-kips or in-kips

P = load, kips or lbs

p_u = ultimate load, kips or lbs

T = tensile force in reinforcement, lbs or kips

u = average bond stress, psi

u_c = calculated average bond stress, psi, using equation 4-1

u_t = average bond stress calculated from test results, psi

u_u = ultimate bond stress, psi

$u*$ = calculated average bond stress using best fit equation, psi

ϵ_c = strain in the extreme compression fiber of concrete

ϵ_s = strain in the tension reinforcement

ϵ_s' = strain in the compression reinforcement

β_1 = a factor equal to 0.80 for this investigation

Σ_o = perimeter of the tension reinforcement

APPENDIX B

MIX DESIGNS

Cement Concrete Mix Design

Cement: Martin Marietta brand, Type I, Portland Cement.

Fine Aggregate: No. 100 to No. 4.

Coarse Aggregate: No. 4 to ½ inch.

Slump: 1 - 3 inches.

Water/Cement Ratio: 0.41.

Design Strength: 4500 psi.

Batch Yield: 6.3 cubic feet.

Material	% Absolute Volume	Proportions by Weight
Cement	15.16	1.00
Water	19.62	0.41
Fine Aggregate	31.42	1.75
Coarse Aggregate	33.90	1.84

Results: f'_c = 5089 psi (average); f_t = 583 psi (average).

Polymer Cement Concrete Mix Design

Polymer: PROTEX Industries brand, PROBOND Epoxy ET-180

        Emulsified.

Cement: Martin Marietta brand, Type I, Portland Cement.

Fine Aggregate: No. 100 to No. 4.

Coarge Aggregate: No. 4 to ½ inch.

Batch Yield: 0.43 cubic feet.

Material	% Absolute Volume	Proportions by Weight
Blended Epoxy	30.91	15.0
Cement	10.54	15.0
Water	-----	----
Fine Aggregate	28.34	34.3
Coarse Aggregate	30.21	35.7

Results: f_t = 602 psi (average).

NOTE: The amount of water is not listed above as it is not considered a part of the overall mix but rather an integral part of the emulsion system. The water content for the PCC was maintained at a water/cement ratio of 0.27. The optimum water/cement ratio as determined by the formulator is listed as the range 0.27 - 0.31.

Aggregate Blends

Coarse Aggregate:

Sieve Size	Mix Content (lbs/100 lbs)	% Passing
#4	10	10
3/8 inch	45	55
1/2 inch	45	100

Fine Aggregate:

Sieve Size	Mix Content (lbs/100 lbs)	% Passing	% Cumulative Retained
#4	10	100	0
#8	15	90	10
#16	30	75	25
#30	25	45	55
#50	20(10)*	20	80
#100	0(10)*	0(10)*	100(90)*
		Fineness Modulus:	2.70 (2.60)*

NOTE: *Denotes a change in the fine aggregate blend for test series #5 and #6.

APPENDIX C

BEAM TEST DATA

The record of the crack progression, load and deflection of each of the test beams obtained during testing is presented in this appendix. The deflection was measured at midspan with a deflection gauge from which readings were taken periodically. Deflection measurements were taken only for the last three test series.

TEST BEAM: T–1–C

Design Splice Length: 12 inches; Shear Span: 18 inches

Load per Jack (lbs)	Mid span Deflection (0.001 in.)	Comments
1,000	Not measured	
2,000		
3,000		Flexure cracks; directly over loading jacks (top only)
3,250		Flexure cracks; vicinity of ends of splices
3,500		Continuation of flexural cracks
3,750		
4,000		
4,250		
4,50C		
4,750		
5,000		Diagonal tension cracks initiated
5,250		
5,500		Additional flexural cracks in splice zone
5,750		
6,000		
6,250		
6,500		
6,750		
7,000		
7,250		
7,500		
7,750	Not measured	
8,000		
8,250		
8,500		FAILURE: Side split; no horizontal splitting along plane of reinforcement observed at 8,250 lbs.

TEST BEAM: T-1-P

Design Splice Length: 12 inches; Shear Span: 18 inches

Load per Jack (lbs)	Mid Span Deflection (0.001 in.)	Comments
1,000	Not measured	
2,000		
3,000		
4,000		Flexure cracks over loading jacks; some cracking down sides from the flexure cracks
4,500		First crack in the PCC; vicinity of the end of the splice, only on the top surface
5,000		Cracks on sides of the PCC block from the flexure crack
5,500		Side splitting cracks continue in the PCC
6,000		More side splitting cracks in the PCC
6,500		More side splitting cracks in the PCC
7,000		First shear cracks noted; diagonally displaced on the sides of the beam
7,500		Cracks progressing and widening
8,000		FAILURE: Side split; failure was sudden; only slight horizontal cracks on side of beam before failure

TEST BEAM: T-2-C

Design Splice Length: 8 inches; Shear Span: 24 inches

Load per Jack (lbs)	Mid Span Deflection (0.001 in.)	Comments
1,000	Not measured	
2,000		
2,500		Flexure cracks over loads
3,000		More flexure cracks; ends of splice
3,250		
3,500		
3,750		
4,000		Crack widening particularly over loads
4,250		
4,500		Lateral cracking in splice zone; load drops to 4,300. FAILURE: Side split; failure occurred while attempting to reload to 4,500

TEST BEAM: T-2-P

Design Splice Length: 8 inches; Shear Span: 24 inches

Load per Jack (lbs)	Mid Span Deflection (0.001 in.)	Comments
1,000	Not measured	
2,000		Flexure cracks over loading jacks
2,500		Flexure cracking continued
3,000		Flexure cracks progress down the sides of the beam
3,500		Flexure cracking continues
4,000		Flexure cracking continues, cracks widen and elongate
4,250		
4,500		First crack in the PCC; vicinity of the end of the splice
4,600		Horizontal cracking of the PCC in the area of the splice; vicinity of the plane of reinforcement
4,700		Great increase in the horizontal cracking; load drops to 3,750; cracks widen and "grow" slowly and visually; beam was reloaded to a load of 4.2 kpj, crack visibly grew. FAILURE: Horizontal cracking progressed until the beam failed in a side split failure mode

TEST BEAM: T-3-C

Design Splice Length: 16 inches; Shear Span: 24 inches

Load per Jack (lbs)	Mid Span Deflection (0.001 in.)	Comments
2,000	Not measured	Flexure cracks over loads; cracks continue down sides 1 inch – 2 inches
2,500		Flexure cracks cver ends of splice
3,000		
3,250		
3,500		Flexure cracks in center of splice zone
3,750		
4,000		
4,250		
4,500		
4,750		
5,000		
5,250		Face splitting between two splices
5,500		Horizontal cracking in splice zone along reinforcement plane
6,000		Diagonal tension cracks developing
6,250		Additional horizontal cracking in splice zone
6,500		FAILURE: Side split; face splitting evident for approximately 70% of splice length

TEST BEAM: T-3-P

Design Splice Length: 16 inches; Shear Span: 24 inches

Load per Jack (lbs)	Mid Span Deflection (0.001 in.)	Comments
2,000	Not measured	Small flexural cracks; vicinity of the loading jacks, top surface only
3,000		Flexure cracking continues down sides of beam 1 - 2 inches
4,000		
4,500		
5,000		First crack in the PCC; vicinity of the ends of the splice
5,500		
6,000		Flexure cracks in the PCC continue to develop, but no new cracks evident
6,250		
6,500		Diagonal tension cracks begin to develop
6,750		
7,000		More diagonal tension cracks forming; flexure cracks in the PCC steady
7,250		
7,500		Another crack in the PCC; almost centered between the ends of the splice length
7,750		A loud "pop" is heard; vicinity of the splice zone; no new cracks evident in the area of the splice
8,000		
8,250		Slight horizontal crack in the area of the splice
8,500		Cracks widening; more horizontal cracks in splice zone, but only very slight
8,750		
9,000		Bond cracks evident in concrete; vicinity of the supports and along the shear span

TEST BEAM: T-3-P (Continued)

Load per Jack (lbs)	Mid Span Deflection (0.001 in.)	Comments
9,250		Diagonal cracks develop in the PCC on the sides of the beam at the level of the plane of reinforcement
9,500		
9,750		Additional diagonal cracking in the splice zone
10,000		FAILURE: Side split; slow horizontal progression of cracks along the reinforcement plane, followed by a loud "pop" at failure

TEST BEAM: T-4-C

Design Splice Length: 12 inches; Shear Span: 24 inches

Load per Jack (lbs)	Mid Span Deflection (0.001 in.)	Comments
1,000		
2,000	.004	Flexure cracks over load
3,000	.061	
3,500	.188	Additional flexure cracks in splice zone
4,000	.215	
4,250	.228	
4,500	.242	Cracks widening
4,750	.255	
5,000	.261	
5,250	.285	Horizontal cracking in splice zone
5,500	.335	FAILURE: Side split; load drops to 1,100.
		Actual Splice Length: 12.05 in.
		Actual Bottom Cover: 1.62 in.

TEST BEAM: T-4-P

Design Splice Length: 12 inches; Shear Span: 24 inches

Load per Jack (lbs)	Mid Span Deflection (0.001 in.)	Comments
2,000	.015	Flexure cracks over the west loading jack only
3,000	.161	Flexure cracks over both loading jacks now
3,500	.186	
4,000	.201	
4,500	.230	More flexure cracks in the vicinity of the loading jacks
4,750	.243	First crack in the PCC; vicinity of the cement/PCC interface, only on the top surface
5,000	.257	Flexure cracks in the PCC; vicinity of the ends of the splice; cracks could be seen to "grow" while under constant load
5,250	.270	
5,500	.284	Small elongation of existing cracks
5,750	.297	
6,000	.311	Small horizontal cracks in the PCC block; along the plane of the reinforcement
6,250	.323	
6,500	.338	More flexure cracks observed; widening and elongation of existing cracks evident
6,750	.353	
7,000	.369	
7,250	.384	Diagonally displaced cracks develop in vicinity of the horizontal cracks
7,500	.403	Cracking continued; PCC cracks widening

TEST BEAM: T-4-P (Continued)

Load per Jack (lbs)	Mid Span Deflection (0.001 in.)	Comments
7,750	.425	FAILURE: Side split; failure mode could be followed visually as cracks widened and elongated without an increase in load. Deflection increased also; just prior to failure the gauge reading indicated .505 inch deflection at mid span
		Actual Splice Length: 12.0 in.
		Actual Bottom Cover: 1.60 in.

TEST BEAM: T-5-C

Design Splice Length: 14 inches; Shear Span: 24 inches

Load per Jack (lbs)	Mid Span Deflection (0.001 in.)	Comments
2,000	.206	Flexure cracks over loads
3,000	.253	Flexure cracks over ends of splice
3,500	.278	Horizontal cracking initiated
4,000	.298	
4,500	.318	
5,000	.340	
5,500	.362	
5,750	.376 – → .445	FAILURE: Side split
		Actual Splice Length: 14.20 in.
		Actual Bottom Cover: 1.60 in.

TEST BEAM: T-5-P

Design Splice Length: 14 inches; Shear Span: 24 inches

Load per Jack (lbs)	Mid Span Deflection (0.001 in.)	Comments
2,000	.011	
3,000	.161	Flexure cracks top surface only, over the east jack only
4,000	.206	Flexural cracks develop over the other loading point
4,500	.232	
5,000	.257	First crack in the PCC; vicinity of the ends of the splice
5,500	.282	Flexure cracks in the PCC continue down the sides of the beam
5,750	.300	
6,000	.316	
6,250	.329	Very little change in the crack progression
6,500	.341	
6,750	.354	
7,000	.369	
7,250	.383	
7,750	.408	Only slight change in the crack progression
8,250	.440	Vertical cracks in the PCC begin to turn slightly toward the center of the splice
8,750	.478	FAILURE: Side split; again the failure followed a slow, visual crack progression in the splice zone; no horizontal cracks were noted in the area of the splice before the failure mode began. With no increase in load, failure occurred within four minutes after load of 8,750 was applied.

Actual Splice Length: 13.92 in.

Actual Bottom Cover: 1.60 in.

TEST BEAM: T-6-C

Design Splice Length: 6 inches; Shear Span: 24 inches

Load per Jack (lbs)	Mid Span Deflection (0.001 in.)	Comments
2,000	.180	Flexure cracking begins
2,500	.209	
3,000	.234	Flexure cracks at end of splice zone
3,250	.251	
3,500	.265	
3,750	.280	
3,875	.355	FAILURE: Side split; no obvious horizontal cracking prior to failure
		Actual Splice Length: 5.98 in.
		Actual Bottom Cover: 1.64 in.

TEST BEAM: T-6-P

### Design Splice Length: 6 inches; Shear Span: 24 inches

Load per Jack (lbs)	Mid Span Deflection (0.001 in.)	Comments
2,000	.173	Initial cracks occur at the PCC/cement interface
2,500	.201	Flexure cracks over loading points
3,000	.226	Flexure cracks continue down sides of the beam
3,500	.254	
4,000	.279	
4,250	.295	
4,500	.309	
4,750	.327	First crack in the PCC; one only on the west end of the splice
5,000	----	Second crack in the PCC at the other end of the splice
5,250	.382	FAILURE: Side split; the now normal slow crack progression preceded actual failure of the splice with no increase in load
		Actual Splice Length: 6.2 in.
		Actual Bottom Cover: 1.65 in.

APPENDIX D

SAMPLE CALCULATIONS

Determination of the Ultimate Strength of the Control Beam Test

Specimen (31)
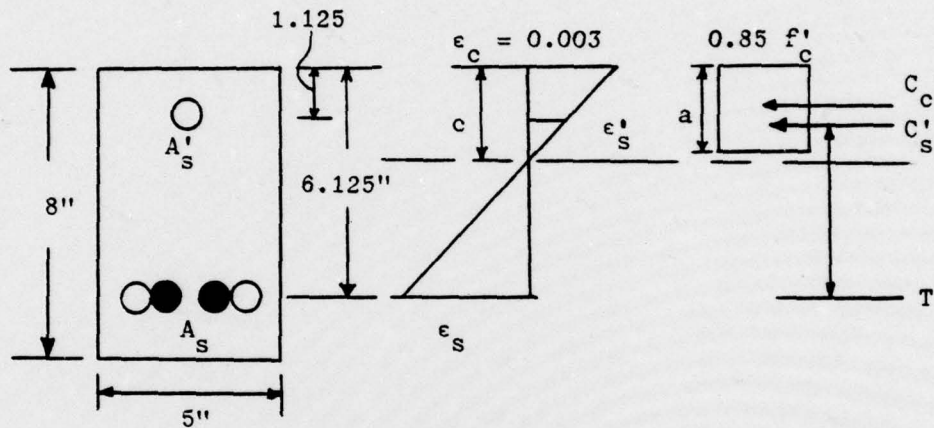
Given: b = 5.0 inches f'_c = 5100 psi

d = 6.125 inches f_y = 60,000 psi

d' = 1.125 inches β_1 = 0.80

A'_s = 1 - #6 = 0.44 inch^2

A_s = 2 - #6 = 0.88 inch^2

From the figure:

$$T = f_y A_s = 60 \ (0.88) = 52.8 \text{ kips}$$

$$C_c = 0.85 \ f'_c \ (b)(a) = 0.85 \ (5.1)(5)(0.80c) = 17.34 \ c \text{ kips}$$

$$C'_s = [29 \ (3/c)(c - 1.125) - (0.85)(5.1)] \ (0.44) =$$

$$38.28 \ (\frac{c - 1.125}{c}) - 1.907 \text{ kips}$$

If the compression steel does not yield we can determine the

location of the neutral axis by equating $(C_c + C'_s)$ to T. After

simplifying we have:

$$0 = 17.34c^2 - 16.43c - 43.07$$

Solving we can determine the distance from the extreme compression fiber to the neutral axis as

$$c = 2.12 \text{ inches}$$

Therefore,

$$a = \beta_1 c = 0.80 \ (2.12) = 1.7 \text{ inches}$$

Then,

$$C_c = 0.85 \ (5.1) \ (5) \ (1.7) = 36.85 \text{ kips}$$

$$\varepsilon_s' = (0.003)(2.12 - 1.125)/2.12 = 0.0014$$

$$\varepsilon_y = 60/29{,}000 = 0.00207 > \varepsilon_s'$$

$$C_s' = [29(3/2.12) \times (2.12 - 1.125) - (0.85)(5.1)] \ (0.44) =$$

$$16.06 \text{ kips}$$

Check,

$$C_c + C_s' = 36.85 + 16.06 = 52.91 \simeq T = 52.8 \text{ kips, ok.}$$

The ultimate moment is thus,

$$\overline{M}_u = C_c[6.125 - (1.7)/2] + C_s'(6.125 - 1.125) = 274.68 \text{ inch-kips}$$
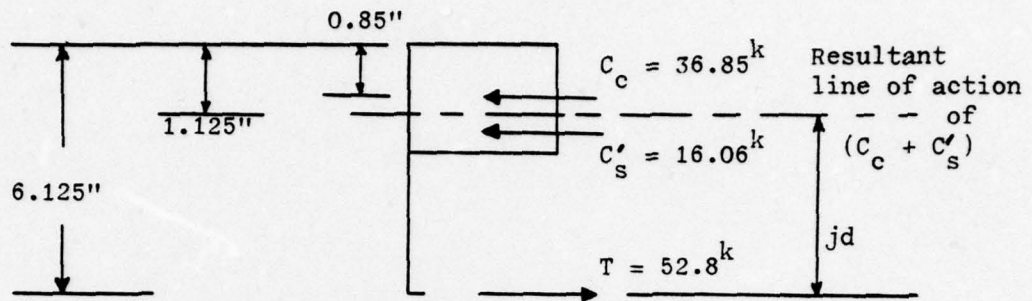
or,

$$\overline{M}_u = 22.89 \text{ kip-ft.}$$

The ultimate load, per jack, considering a shear span of two feet, is

$$P_u = 11.445 \text{ kips.}$$

Determination of "j" for Control Beam at Tension Failure

The compressive forces C_c and C_s' do not act along the same line. The location of the line of action of the resultant of these compressive forces can be determined by summing moments about the tensile force.

$$36.85(6.125 - 0.85) + 16.06(6.125 - 1.125) = (36.85 + 16.06)jd$$

Therefore,

$$jd = 5.192 \text{ inches}$$

or,

$$j = 5.192/6.125$$

$$\underline{j = 0.85}$$

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