BOND IN PRESTRESSED CONCRETE

PROGRESS REPORT NO. 1

ULTIMATE FLEXURAL BOND

IN

CONCRETE BEAMS PRE-TENSIONED WITH HIGH-STRENGTH STRAND

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ABSTRACT

This study is an investigation of the bond characteristics of \( \frac{3}{8} \)-in. 270 K prestressing strand. The primary objective was to determine the effect of embedment length on the axial stress necessary to produce general bond slip. The principal variable in this investigation was the embedment length of the strand.

The results of thirteen tests conducted on twelve beams pretensioned with \( \frac{3}{8} \)-in. 270 K strand are presented and compared with the test results obtained by Hanson and Kaar in tests conducted on beams pre-tensioned with \( \frac{1}{2} \)-in. conventional strand. The observed critical length for the \( \frac{3}{8} \)-in. 270 K strand was found to be 80 inches, as compared with 135 inches reported by Hanson and Kaar for the conventional strand. In general, it was found that higher stresses were required to cause general bond slip with the 270 K strand, than with the conventional strand.

An analytical concept has been developed, but a comparison with the experimental results is not included, mainly due to the lack of sufficient information on the development of friction, mechanical action, and the coefficient of creep in concrete.
1. **INTRODUCTION**

The usefulness of pre-tensioned concrete structural members depends largely upon the bond developed between the prestressing steel and the concrete. The prestressing steel in pre-tensioned members serves a dual purpose. Initially, the steel is used to develop a compressive prestress in the concrete. Later, when the member is subjected to loading, the steel works with the concrete in resisting the applied load. Prior to the development of tensile cracks, the contribution of the steel is small. However, after the cracks have formed, the steel is essentially responsible for resistance to the internal tensile forces. In order to efficiently utilize the tensile strength of the steel, interaction of the two materials should exist, enabling the concrete and steel to work together in developing the prestressing force, and in resisting the applied external loads.

In the fabrication of a pre-tensioned prestressed concrete beam the steel is first tensioned to the desired stress level. Concrete is then placed in the form. After the concrete has reached a specified strength, the steel is released, and bond is responsible for transferring the force from the steel to the concrete. The bond developed in this transfer is commonly referred to as the **prestress transfer bond**, and the length required for the complete transfer of prestress force is referred to as **transfer length** or **anchorage length**. Knowledge
of the transfer bond is important in determining stresses in the early life of the member, and later, in determining the ability of the member to develop anchorage up to the ultimate strength.

When the beam is subjected to loading, the prestressing steel serves a function similar to that of ordinary reinforcing steel in reinforced concrete. As a result, additional bond stresses are developed between the steel and the concrete. These bond stresses are called flexural bond stresses. Flexural bond is insignificant as long as flexural cracking does not occur. However, the development of cracking results in greatly increased flexural bond stresses which change continually with changes in the crack pattern. As the ultimate load is approached, the region of high flexural bond stress moves nearer to the ends of the member. When this region reaches the end of the prestress transfer zone, general bond slip occurs. Normally, with 7-wire strand as the prestressing element, failure does not occur at this point. Mechanical resistance provided by the helical shape of the strand will permit some additional load before failure occurs. Therefore, in line with the general philosophy of prestressed concrete design that "ultimate failure should be governed by elongation of the prestressing steel rather than by shear, bond, or concrete compression," a full knowledge of the characteristics of bond between concrete and 7-wire strand at a high level of stress is necessary in order to design and predict the overall behavior of pre-tensioned, prestressed concrete members.

At the present time there is no specific theory available for predicting the bond performance of pre-tensioned beams. In the past some
important contributions have been made in the study of this problem. In 1954, Janney\(^{(5)}\) reported a study of the nature of bond in pre-tensioned prestressed concrete. Four sizes of prestressing wire and one size of prestressing strand (5/16-in.) were used in studying both transfer bond and flexural bond characteristics. The principal variables were diameter, surface condition, and degree of initial pre-tensioning of the steel. A variation in anchorage length was noted for wires of different diameters, and it was found that surface condition was also a major factor. Based on an elastic analysis of the deformations of the test specimens after release of the pre-tensioned steel, it was suggested that prestress transfer bond is largely a result of friction between the concrete and steel. Beam tests indicated that reliable values for flexural bond stress after cracking cannot be obtained by the expression

\[
u = \frac{V}{\Sigma \delta Jd}\]

In 1956, Niels Thorsen\(^{(11)}\) showed that bond stresses in the end zones of a pre-tensioned flexural member differ from the bond stresses in the interior region. It was pointed out that an increase in bond stress within the transfer length would result first in slip of the strand, and eventually in failure of the member.

In 1957, Nordby and Venuti\(^{(9)}\) conducted a series of tests on pre-tensioned beams prestressed with 3/8-in. strands. Conventional and expanded shale aggregate concrete were used in the beams. The results indicated that embedment length was the governing factor against failure rather than bond stress as computed by conventional equations. An equation in the form of

\[
U_u = \frac{3f_A s}{4\pi DL_e}
\]

was given to compute the average bond
stress at the time of failure.

In 1958, a report by Dinsmore, Deutsch and Montemayor(2) summarized a study of both transfer bond and flexural bond characteristics in test specimens pre-tensioned with 7/16-in. strands. It was concluded that friction was the major factor in the development of ultimate bond strength.

In 1959 Hanson and Kaar(3) reported a comprehensive study of flexural bond in beams pre-tensioned with 7-wire strand. Three sizes of strand (1/4, 3/8, and 1/2-in.) were used. The principal variables were the embedment length and the diameter of the strands, although the effects of other variables such as surface condition of the strands, concrete strength, and percentage of steel were investigated. It was concluded that a general bond slip occurs in a pre-tensioned beam when the region of high flexural bond stress reaches the stress transfer zone. Also, it was found that strand size and embedment length have a considerable influence on the value of the average bond stresses at which general bond slip will occur in a flexural member.

A study recently underway at the University of Illinois was reported by Anderson, Rider, and Sozen in 1964.(1) This study involved investigations of both anchorage bond and flexural bond in pre-tensioned members. Pull-out test specimens were designed to simulate (1) end-block conditions, and (2) the tensile region of a beam. In addition, several beams with non-prestressed strand were tested. Three types of strand were used: 7/16-in. 7-wire round-wire strand, 7/16-in. 7-wire rectangular-wire strand, and 1/4-in. 3-wire rectangular-wire strand. Unit-bond-slip curves are presented, and it is emphasized that additional axial stress
can be developed in a strand after initial bond slip takes place. Lengths required for anchorage and embedment are also given.

In recent years, the trend in American practice related to pre-tensioned prestressed concrete members has been directed toward the use of 7-wire high-strength strands with larger diameter. In the early 1950's, the ¼-in. size was commonly used. But since then, extensive use has been made of 3/8, 7/16, and 1/2-in. strands.

Recently, a new high-strength strand was developed and made available for general use. The new strand, commonly referred to as 270 K strand, is fabricated and tested in accordance with the requirements of ASTM designation A416-64, except that the specified minimum ultimate strength is 270 ksi, rather than the 250 ksi of the conventional strand. Also, the cross-sectional areas of the nominal sizes are slightly greater than those same nominal sizes in the conventional strand. To date, only one study has been completed, involving the new strand. This study, reported by H. K. Preston(10), was a very limited study of transfer bond. Therefore, there is an immediate need for information regarding bond characteristics of the 270 K strand. The recognition of this need was responsible for the planning of a testing program aimed at the development of information on the flexural bond characteristics of members pre-tensioned with this strand.

Following is a report of the study which was conducted at Lehigh University, in the Fritz Engineering Laboratory, Department of Civil Engineering.
2. OBJECTIVE AND SCOPE

1.1 OBJECTIVE

The principal objectives of this investigation are, (1) the development of information on the flexural bond characteristics of prestressed concrete beams pre-tensioned with \( \frac{1}{2} \)-in. 270 K, 7-wire strand, and (2) comparison of this information with the results obtained by Hanson and Kaar\(^3\) for beams prestressed with 250 ksi strands. An additional objective is the development of an analytical concept for bond failure in flexural members.

1.2 SCOPE

The bond performance of a pre-tensioned concrete member may depend upon a number of factors such as: (1) size of the strand, (2) ultimate strength of the strand, (3) surface condition of the strand, (4) concrete strength at the time of prestressing, (5) rate of release of the strands, (6) spacing of the strands, and (7) steel percentage. In the previous investigation by Hanson and Kaar, the relative effects of several of the variables were set forth. Essentially, it was found that the significant variables were strand size and embedment length. Therefore, in this investigation the test specimens had the following characteristics.

(1) One size of 270 K strand (\( \frac{1}{2} \)-in.) was used in all specimens. Since the ratio of surface perimeter to cross-sectional area decreases as the strand size is increased, bond is most critical in
members prestressed with the largest size of strand. Since the \( \frac{1}{2} \)-in. size is the largest size in common usage, it was concluded that the results from the tests would represent the extreme case. Also, the strand used was rust-free, since earlier investigations indicated that bond is less critical when the strands are rusted. Likewise, it was felt that the rust-free strands would represent the extreme case.

(2) The initial prestress stress was constant for all specimens. The stress was 70% of the specified minimum ultimate which, for the 270 K strand, was 189 ksi.

(3) The center-to-center spacing of strands was set at 2 inches for all specimens. This spacing represents the minimum allowable for \( \frac{1}{2} \)-in. strand as specified by the current ACI Building Code (ACI 318-63). Therefore, it was felt that this spacing would represent the most critical case.

(4) Two specifications governed the ultimate compressive strength of the concrete. The first was that the strength must be at least 4500 psi at release of prestressing steel, and the second, that the strength must be no more than 6000 psi at test.

(5) The main variable introduced was the embedment length, which is defined as the distance from the end of the specimen to the location of the maximum bending moment. In the test specimens, the embedment length was varied from 36 to 96 inches.

These characteristics were specified in order that the test specimens would represent flexural members designed with respect to the currently specified requirements regarding concrete strength, prestress stresses in both the strand and concrete, and spacing of the strand. In addition, the \( \frac{1}{2} \)-in. rust-free strand was used to produce the most critical condition relative to bond failure.
3. TEST SPECIMENS

3.1 DESCRIPTION OF THE TEST SPECIMENS

The specimens were designed as under-reinforced beams such that the loading would produce flexural failure. In line with the main objective of the project, the tests were to indicate (1) whether or not general bond slip would occur prior to flexural failure, or (2) if general bond slip did occur with a given embedment length, to determine the maximum strand stress which could be developed at the interior end of that length.

A rectangular cross-section with a width of $7\frac{1}{2}$ inches and a depth of 12 inches was selected for the specimens. The width of $7\frac{1}{2}$ inches was chosen to satisfy the minimum spacing of 2 inches and to allow minimum cover requirement as specified by the current ACI Building Code (ACI 318-63). The depth of the section and the location of the strands were chosen such that the maximum allowable prestress stresses in the concrete were not exceeded. Four $\frac{1}{2}$-in., 7-wire, 270 K strands were used for prestressing the beams. Three strands were located near the bottom with the minimum cover of $1\frac{1}{2}$ inches required by the Code. The other strand was placed near the top to produce the desired pre-stress stress distribution. This arrangement produced equal tensile stresses in each of the bottom strands. The initial tensile stress in each strand was 189,000 psi, which is 70 percent of the specified ultimate strength ($f'_s = 270,000$ psi) of the strand. No. 3 deformed bars were used for shear reinforcement with, 6-in. spacing, starting 3 inches
from the end of each shear span. For complete details of the test specimens, see Fig. 1. The length of the specimens was varied from 8 feet to 23 feet with an embedment length variation of 36 inches to 96 inches. The actual values are given in Table 1.

The test section was reviewed to determine the probable ultimate load characteristics. The ultimate strength was determined in accordance with the method presented by Hognestad, Hanson, and McHenry (6). The stress distribution at the ultimate moment capacity of the section is given in Fig. 2. The ultimate moment, $M_u$, is given by the expression:

$$M_u = f_{su} A_s (d - k_2 c)$$

The tensile force in the steel and the compressive force in the concrete are given by:

$$T = f_{su} A_s \quad \text{and} \quad C = k_1 k_3 f'bc$$

Since $T = C$, the depth of the compression block is given by:

$$c = \frac{f_{su} A_s}{k_1 k_3 f'b}$$

From the geometry of the unit strains:

$$\frac{c}{d} = \frac{F_{eu}}{\varepsilon_{su} + F_{eu} - (\varepsilon_{se} + \varepsilon_{ce})}$$

Several trials are usually necessary to establish the compatibility between the calculated $f_{su}$ and the values of $\varepsilon_{su}$ and $f_{su}$ obtained from the stress-strain curve of the strand.
The ultimate strength factors used in the calculation of the ultimate moment have been expressed as functions of the concrete strength. These functions, given by Hognestad, Hanson, and McHenry (4), are:

\[ k_1k_3 = \frac{3900 + 0.35 f'_c}{3200 + f'_c} \]

\[ k_2 = 0.50 - \frac{f'_c}{80,000} \]

\[ k_1 = 0.94 - \frac{f'_c}{26,000} \]

\[ k_3 = \frac{3900 + 0.35 f'_c}{3000 + 0.82 f'_c - \frac{f'_c}{26,000}} \]

\[ \varepsilon_u = 0.004 - \frac{f'_c}{6.5 \times 10^6} \]

These expressions, along with the concrete strength at the time of test given in Table 2, were used to compute the ultimate moment for each of the test specimens. The results are given in Table 3.

After several test specimens had been fabricated, load tests were begun. The results from the first few tests indicated that it would be desirable to develop a greater stress in the strand at the time when the ultimate moment was reached. Therefore, it was decided that slabs should be cast on the remaining specimens. At that time, three beams had been cast with a smooth top surface. After considering several alternatives for bonding the slab to these specimens, an epoxy compound was selected. All of the beams subsequently cast were fabricated with a roughened top surface and with the shear reinforcement extending above
the surface to provide an effective bond. The details of these cross-sections are shown in Fig. 1.

In the design of the test specimens, an estimate of the pre-stress losses was required. The factors which contribute to the loss are elastic shortening of the member, and shrinkage and creep of the concrete. The loss of stress in the steel due to elastic shortening is given by:

\[ \Delta f_s = n f_c \]

where \( f_c \) = concrete stress at the level of the prestressing steel.

According to Lin (8) the loss resulting from shrinkage may be taken as:

\[ \Delta f_s = \varepsilon_s E_s \]

where \( \varepsilon_s \) is equal to the unit strain resulting from shrinkage in the surrounding concrete. In this investigation a value of \( \varepsilon_s = 0.0003 \) was used.

Likewise the loss resulting from creep was computed by the expression:

\[ \Delta f_s = (C_c - 1) n f_c \]

A value of \( C_c = 3.0 \) was used in the computations. The computed prestress losses are given in Table 4, along with the average of the actual measured losses.
3.2 MATERIALS

Prestressing Steel

The prestressing strand used was $\frac{3}{8}$-in., 7-wire, uncoated, stress-relieved type 270 K strand. This type of strand is commercially fabricated and tested in accordance with the requirements of ASTM designation A416-64. An increase in the specified minimum ultimate strength from 250 to 270 ksi, together with a slight increase in area, had resulted in the type 270 K, which will develop an ultimate force approximately 15% greater than that of the conventional prestressing strand. The load-elongation curve for this strand is shown in Fig. 3. The physical properties are given in Fig. 4. Although several manufacturers now produce strand of this type, the strand used in all of the test specimens was produced by the John A. Roebling's Sons Division of the Colorado Fuel and Iron Corporation.

Shear Reinforcement

The shear reinforcement was fabricated from No. 3 deformed bars having a nominal yield stress $f_y = 50,000$ psi.

Concrete

Concrete strength was not a variable in this investigation. The mix was designed to yield a concrete with an ultimate strength $f'_c = 6000$ psi at an age of 21 days. The mix selected consisted of type III (high-early strength) portland cement, sand, and crushed limestone coarse aggregate (3/4-in. maximum). The proportions, by weight, of the cement-to-sand-to-coarse aggregate were 1.00:2.64:2.98. The concrete
was obtained from a local ready-mixed concrete supplier, and was delivered in two-cubic-yard batches. The slumps for all of the batches ranged between two and three inches.

Cylinder samples (6 x 12-in.) were used to determine the ultimate compressive strength of the concrete at the time of release of the prestress force, and at the time of test. Information for load-deformation curves was obtained with a compressometer. The tensile strength of the concrete at the time of test was obtained by splitting tensile strength tests. Three cylinders were tested in each determination. The strength properties are given in Table 2.

**Epoxy Resin**

The epoxy compound used to bond the slabs to the three specimens with the smooth top surface was based on the recommendations set forth in the report by Kriegh and Endelbrock (7). The compound used was type B, with the two-component system. The ingredients of system (1) were Epi-Rez 510, Alumina T-60, and Asbestos 7-TF-1, while the ingredients of system (2) were Epi-Cure 855, Epi-Cure 87, Alumina T-60, and Asbestos 7-TF-1. For further information concerning the properties and proportions of the mix, see reference No. 6.

### 3.3 FABRICATION

The beams were fabricated in a prestressing bed at the Fritz Engineering Laboratory. The features of this bed have been described previously in other Fritz Engineering Laboratory reports (12). The two anchorage plates were set 35 feet apart to provide space for fabrication
of two beams simultaneously, end-to-end.

The beams were marked from A to F according to time of fabrication, the first cast A, the second one B, and so on. Since there were two beams cast at each time, the longest beam was designated as number one and the other as number two. Thus, beam A-1 would be the longest of the two beams in the A casting.

Steel forms were used for the fabrication of the beams. Two 1-in. thick plates with 5/8-in. diameter holes were used, one at each end of the prestressing bed, for the vertical and horizontal positioning of the strands. The strands were anchored with standard gripping chucks, and were tensioned to the desired value of 28.90 kips per strand with two 50-ton jacks. Individual strand adjustments were made by means of a separate hydraulic jacking system. The prestress force was measured by means of calibrated load cells placed between the end plate and the gripping chucks on each strand. After completion of the pre-tensioning process, the strain gages were mounted on the strands, the web reinforcement was wired in place, and finally, the forms were erected. The concrete was placed in two layers of approximately equal depth, and vibrated with an internal vibrator. The first six beams were finished with a smooth top surface, but the surfaces of the last six beams were roughened with a fork type plate. All of the specimens were covered with wet burlap for a period of approximately three days. After three days, compressive cylinder tests were conducted. When the cylinder tests indicated a compressive strength of at least 4500 psi, the forms were removed and the Whittemore gage targets were placed on the beams. The prestress force was released slowly with the 50-ton jacks. An oxy-acetylene torch
was used to cut the strands. The beams were then removed from the casting bed, and stored in the laboratory. Wooden forms were used for the fabrication of the slabs. Fifteen standard 6 x 12-in. concrete cylinders were cast in waxed cardboard molds with each beam. The concrete was placed in two layers of approximately equal depth in the mold, and vibrated with an internal vibrator. The cylinders were stored near the test specimens in the laboratory.

3.4 INSTRUMENTATION

To evaluate the test results, it was necessary to measure the displacements in the form of strains, strand slip, and deflection of the specimens under load.

Strains

The load-deformation data obtained from the strain measurements was divided into two separate categories: (1) load-deformation data of the prestressing strands, and (2) load-deformation data of the concrete.

1. Load-deformation of the strands

Electrical strain gages were mounted at intervals along each of the three bottom strands in each of the test specimens, after the prestressing strands had been initially tensioned. These gages were attached along an individual wire of the strand at each location. The gages were waterproofed with Neoprene Primer and Rubber Coating, then covered with Armstrong adhesive A-6 for additional protection. Two types of gages were used. Eleven of the specimens contained A-12 type SR-4 gages, and one contained C6-141-B Budd Metalfilm gages. The location of the gages
in each specimen is given in Figs. 5 and 6.

The basic use of these internal gages was to measure the changes of force in the strand during the ultimate load test. However, they were also used to measure the immediate loss of stress at the time of release. The use of the data from these gages was based on calibration curves developed from separate load tests of individual samples of the strand. In these tests, four or more gages were attached along individual wires of each specimen, in the same manner in which the gages were installed on strands in the test beams. The strain gage readings were plotted versus load in the strand. The average load-strain curve obtained from these tests is given in Fig. 7. Also shown, is the actual relationship between load and axial strain measured with an extensometer.

2. Load-deformation data of the concrete surface

a) A Whittemore strain gage with a 5-in. gage length was used to obtain strains on the surface of the beam at the level of the bottom three strands. Brass plugs 7/32-in. in diameter and 3/32-in. in thickness were used as gage targets. These targets were cemented along both sides of the test beams with Armstrong adhesive A-6, immediately after the forms had been removed and prior to release.

The main use of the targets was in the determination of shortening, both at the time of release and in the interval prior to the test. Readings were also taken during the ultimate load test.

b) A number of A-9 type, SR-4 strain gages were used for the measurement of strains at different elevations in the constant moment
region between the load points. The gages were placed at the mid-span of the beams, and were cemented to the surface with Duco cement. These gages were used in the ultimate load tests.

**Strand Slip**

Mechanical dial gages were used to detect slip in the strand at the ends of the beams during the ultimate load tests. The dial gages indicated the movement of a small plate attached to each strand by means of a collar and set-screw. Two gages, one at each end, were used on each plate. All of the gages were mounted on brackets which were secured to the two ends of the test specimens. A view of the dial gage assembly is given in Fig. 8.

**Deflection Measurement**

The mid-span deflections of each beam were measured by means of level readings on strip scales graduated to the nearest 0.01 inch. These scales were attached to the beams at each support and at mid-span.
4. **TESTS**

4.1 **TEST PROCEDURE**

Ultimate load tests were conducted on each of the specimens. In general, two-point loading was used. The details of the loading arrangement are shown in Fig. 9. Ten of the specimens were tested in a hydraulic testing machine with a capacity of 300,000 lbs. A view of one of the specimens following the test is given in Fig. 10. Two of the specimens, F-1 and F-2, were tested in the 5,000,000 lb. hydraulic machine. Specimen F-2 was a short beam with a short embedment length, and required the higher load capacity. Also, an additional change was introduced in the loading. The end support was placed 12 inches from the end, rather than 6 inches as was used in all other tests. The intent was to obtain a preliminary indication of the effect of the length of overhang. Specimen F-1 was a long specimen (23 ft.), and the loading arrangement was changed in order to develop more useful information. The tests completed prior to the testing of F-1 indicated that failure was very unlikely under the standard loading arrangement. Therefore, another arrangement was used, utilizing separate tests of each end. This arrangement is given in Fig. 11. Initially the beam was loaded as shown in (a), producing a failure at the east end. The beam was then shifted to the position shown in (b), and loaded until failure occurred at the west end.
The specimens were loaded to ultimate failure in increments of about 8 percent of the expected ultimate load. The internal A-12 type, SR-4 gages and the C6-141-B Budd Metalfilm gages, mounted on individual wires of the strands, were used with two different types of strain recording equipment. Six of the gages were connected to channels of a Brush direct-writing recorder to provide a continuous record of the variation in load in the strands. The other internal gages, as well as the A-9 type, SR-4 gages mounted on the surface of the concrete in the maximum moment region, were connected to Baldwin strain indicators. These gages were read following the application of each load increment.

The Whittemore targets on the surface of the concrete were read at selected load intervals, while the mid-span deflection readings were taken at each load interval. The strand-slip dials were checked continuously to detect the initial slip. The development of the crack pattern was marked on the surface of the specimen after each load increment had been added.

4.2 TEST RESULTS AND DISCUSSION

4.2.1 Modes of Failure

A total of thirteen tests were conducted on twelve beams. Seven of the specimens failed by general bond slip at a load smaller than the computed ultimate load. During the loading period of these specimens, flexural cracks developed at different load levels. In general, the first crack occurred in the region of maximum moment. Upon further increase in the load, additional cracks were developed in the shear span. When the development of additional cracks propagated
toward the end of the beam and approached the prestress transfer zone of the beam, general bond slip occurred. The crack patterns of some of the beams that failed by general bond slip are shown in Figs. 12, 13, 14, and 15. Beam F-1 which was 23 ft. long, was first tested with an embedment length of 42 inches from the east end. The second test on F-1 was conducted with an embedment length of 54 inches from the west end. In both cases, the failure was by general bond slip.

Five of the test specimens failed in flexure, generally at a load slightly higher than the computed ultimate load. The typical crack pattern of beams which failed in flexure is shown in Figs. 16 and 17. An uncommon case of failure occurred in beam B-1. In this specimen the slab was bonded to the smooth surface of the beam with an epoxy resin compound, and failure occurred in the joint between the slab and beam. A view of this failure is given in Fig. 18.

The values of prestress losses, effective prestress force, and the prestress transfer length for each individual specimen is given in Table 3. The average prestress transfer length for the ½-in. 270 K strand was 26 inches. The average prestress force immediately after release was 25.88 kips, and average effective prestress force at the time of test was 21.52 kips. These values are compared in Table 4 with the theoretical computed values. The individual values of the effective prestress force for each was used, in conjunction with the ultimate strength of the concrete obtained from the cylinder tests given in Table 2, to compute the ultimate flexural capacity of the specimens. The computed value of the ultimate moment, $M_u$, for each beam is given in Table 5.
The principal test results and mode of failure of each specimen are summarized in Table 6.

4.2.2 Force in the Strand at Various Stages

The variation in the tensile stress along the embedment length at different load levels is given for specimens A-1, A-2, B-2, C-2, D-1, D-2, F-1E, F-1W, and F-2 in Figs. 19 through 29, with the exception of specimens B-1, C-1, E-1, and E-2.*

The following is a discussion of the stress variation along the embedment length in a typical specimen which failed by general bond slip (D-2), and in a typical specimen which failed by flexure (A-1).

Beam A-1 is shown in Fig. 16 after failure. This beam was tested with a strand embedment length of 96 inches and shear span of 90 inches. It failed at a load of 20 kips, by crushing of the concrete in the region of maximum moment. The variation in the force along the west end embedment length is given in Fig. 19. The initial tensile force in the strands prior to the placement of concrete was 28.90 kips per strand. The prestress force curve immediately after release is shown by the heavy dashed line. The value of the prestress force after release was 26.08 kips, and the effective prestress force at the time of test was 22.15 kips. The prestress transfer length was approximately 27 inches.

*Note: Data from the latter four specimens are not included for the following reasons:

Beam B-1 - Joint failure occurred prior to completion of the test
Beam C-1 - Inconsistency of the data from the internal gages during the test
Beam E-1 and E-2 - Loss of data taken at time of release
The effective prestress force variation prior to test is shown with a heavy solid line and indicated by $P = 0$, where $P$ represents the load on the beam.

The variation of force in the strand along the embedment length for load levels of 10, 12, 14, 16, 18, and 20 kips are plotted in Fig. 19. It can be seen that the region of high force gradually moves toward the end of the beam as the load is increased. When the load reached $P = 20$ kips, the progress of the region of high forces was stopped at approximately 55 inches from the west end of the beam due to failure of the beam in flexure.

Beam D-2 was tested with an embedment length of 36 inches and shear span of 30 inches. It is shown in Fig. 13 after the completion of test. The beam failed by general bond slip of the strands at the west end at a load of 96 kips. Immediately after the slip, diagonal shear cracks appeared and the load dropped to 82 kips. The load was again increased to 96 kips, but at this point, a shear failure occurred at the west end. The variation of force in the strand is given in Fig. 25 for the west end, and Fig. 26 for the east end. The initial tensile force in the strands was 28.90 kips per strand prior to the placement of concrete. The value of the average prestress force immediately after release was 25.91 kips, and the effective prestress force at the time of test was 22.87 kips. The prestress force distributions along the embedment length immediately after release and at the time of the test are shown in Figs. 25 and 26. The prestress transfer length was approximately equal to 26 inches. The forces in the strands at different locations along the east embedment length are plotted in Fig. 26 for
loads of 40, 80, 88, and 96 kips on the beam. The interesting point to notice is the gradual movement of the region of high forces in the strands toward the exterior end of the embedment length as the load on the beam was increased. The region of high forces in the strands in the east end embedment length reached the end of the prestress transfer length when the load on the beam was increased to 88 kips, but general bond slip was not noticed. Subsequently, the load on the beam was increased to 96 kips, and at this point, a general bond slip was noticed at the west end of the beam. This can be seen from the force distribution curve for $P = 96$ kips in the west embedment length (Fig. 25), where the region of high forces in the strand overlapped the end of the prestress transfer length and the general bond slip took place. This pattern was noticed in all of the beams that failed by general bond slip.

4.2.3 Effect of Embedment Length on $P_{\text{ult.}}$

Beams with embedment lengths from 36 inches to 60 inches failed by general bond slip, while beams with embedment lengths from 72 inches to 96 inches failed in flexure. However, beam E-2, with an embedment length of 48 inches failed in flexure rather than bond, in spite of high stresses developed in the strands.

The ratio of $\frac{P_{\text{test}}}{P_{\text{ult.}}}$, which is a comparison of the loads sustained at failure with theoretical ultimate loads, are given in Table 6. The ratios $\frac{P_{\text{test}}}{P_{\text{ult.}}}$ are plotted versus embedment length in Fig. 30, where $P_{\text{ult.}}$ is the computed ultimate load at flexural failure.
The effect of strand embedment length on the load carrying capacity of the beams is illustrated by the curve in Fig. 30. It can be seen that the beams with embedment lengths less than 60 inches have smaller ultimate load carrying capacities than their computed flexural capacities.

One interesting point to notice is the ratio of beams F-1E and F-2, where this ratio is equal to 0.919 for beam F-1E (shown by ), and to 0.998 for beam F-2 (shown by ). These two beams have the same cross-section, concrete strength, and embedment length (42-in.). The only difference was in the shear span and overhang. Beam F-1E was tested with a shear span of 36-inches and an overhang of 6-inches, while beam F-2 had a shear span of 30-inches and an overhang of 12-inches. In the tests, beam F-2 was able to sustain a higher load before general bond slip than beam F-1E. As a result the maximum stress in the strands of beam F-2 at the region of maximum moment \( f_s = 233,000 \) psi was greater than the maximum in beam F-1E \( f_s = 196,000 \) psi.

The maximum stresses developed at the interior ends of the embedment lengths at general bond slip are given for all specimens in Table 6.

4.2.4 Average Bond Stress

The average bond stress along the embedment length was calculated for each beam, using the equation

\[
\frac{f_A}{u_a} = \frac{f_s}{L_e} \frac{A_s}{a}
\]
where $f_s$ is the steel stress at the end of the embedment length at the time of failure. The ratio $\frac{a}{A_s}$ is given in Fig. 4, where $a$ is the circumference of the strand. For $\frac{1}{2}$-in. 270 K strand, this ratio is equal to 13.95. Thus, by substituting 13.95 in the above equation we obtain $u_a = 0.0717 \frac{S}{L_e}$. These average bond stresses are plotted versus the corresponding embedment lengths in Fig. 31, in conjunction with a curve representing the average bond stress required to develop the full tensile strength of the strand. In Fig. 31 the average bond stress in beam F-2 at general bond slip is shown by $\Box$, and beam F-1E by $\bigcirc$. It can be seen that the average bond stress at which strand slip occurred is higher for beam F-2.

In Fig. 32, the average bond stresses in the beams prestressed with $\frac{1}{2}$-in. 270 K strand are compared with the average bond stresses obtained by Hanson and Kaar,\(^{(3)}\) from the tests conducted on beams prestressed with conventional $\frac{1}{2}$-in. strand. The average bond stress required to develop the full tensile strength obtained by Hanson and Kaar was based on the actual ultimate tensile strength (263,000 psi) rather than the nominal tensile strength (250,000 psi). However, for comparison of test results, the nominal tensile strengths of the two types of strand are used.

This comparison indicates that the $\frac{1}{2}$-in. 270 K strand had developed larger average bond stresses than the conventional $\frac{1}{2}$-in. strand, and that the critical embedment length is definitely shorter than the one obtained by Hanson and Kaar. The critical embedment length for the 270 K strand is approximately 80 inches.
as shown in Fig. 31. Using the curve in Fig. 31 for average bond stress at which slip will occur, in conjunction with the relationship \( f_s = 13.95 \frac{L}{u_a} \), the stresses at general bond slip were computed for given embedment lengths. The values of these stresses are plotted in Fig. 33, and compared with a similar curve obtained by Hanson and Kaar in Fig. 34. From this comparison it can be seen that for a given embedment length, the \( \frac{1}{4} \)-in. 270 K strand is capable of sustaining higher stresses before general bond slip than the conventional \( \frac{1}{4} \)-in. strand. The curve in Fig. 33 resulted from tests of \( \frac{1}{4} \)-in. 270 K strand, initially pre-tensioned to 189,000 psi and embedded in a concrete with a compressive strength of 6000 psi, while the curve represented by Hanson and Kaar is applicable to \( \frac{1}{4} \)-in. conventional strand, initially tensioned to 150,000 psi and embedded in a concrete with a compression strength of 5500 psi.
In this section, a theoretical method is developed for the determination of bond stress in the strand. The result of the analysis is an expression which yields the force in a strand at any time and at any location.

When the prestressing steel is tensioned to the desired level of tensile force, the diameter of the strand contracts due to the effect of Poisson's ratio. After the concrete has been cast and cured, the strand is released and the prestressing force is transferred to the concrete through bond. After release the stress in the strand increases from zero at the end of the beam to an essentially constant stress at the end of the anchorage length. This variation in the tensile stress of the strand causes an expansion in the diameter of the strand in proportion to the reduction in the initial tensile stress. This expansion is resisted by the surrounding concrete, inducing radial pressure at the interface. This interface pressure between the strand and concrete is responsible for the development of frictional resistance to the slippage of the strand. When external load is applied to the beam, the prestressing strand will be subjected to additional tensile stress, resulting in radial contraction. However, the concrete at the level of the strand, which is initially under compression, will be subjected to tensile stresses as external load is applied. The reduction in the longitudinal compressive stress will cause the concrete surrounding the
strand to undergo radial contraction due to the effect of Poisson's ratio. When the diameter of the strand decreases due to the external applied load, the interface pressure between the strand and concrete decreases also. Theoretically, when this diameter becomes the same size that it was at $f_{sl}$, the radial pressure becomes zero, and by further increase of the external load, the bond between steel and concrete will be completely broken. As the load is increased, the breakdown of bond will continue to progress toward the end of the embedment length, and when the interface pressure becomes zero at a critical length from the end of the beam, general bond slip will occur. However, there will be some creep in the concrete, longitudinally due to the prestress force, and radially due to the radial compression at the interface between the strand and concrete. Since the displacements due to creep will not be recovered after the removal of the force, this effect should be taken into account when considering the contraction of the diameter of the concrete surrounding the strand.

In this development the concrete surrounding the strand was analyzed as a thick-walled elastic cylinder, with the following assumptions:

1. The thickness of each concrete cylinder surrounding the strand was assumed to be one-half of the clear spacing between the two strands.

2. The interaction between the two adjacent concrete cylinders is neglected. Therefore, the radial pressure on the outer surface was assumed to be zero.

3. Recognizing that the longitudinal stress, $\sigma_L$, will vary linearly across the end of the cylinder, for the purpose of simplifying the analysis, it was
assumed that \( \sigma_w \) will be constant and that the magnitude will be taken as the value at the center of the cylinder.

Other assumptions and limitations are:

4. Both concrete and steel are considered to be elastic, homogeneous, and isotropic.

5. Friction and mechanical action are considered to be responsible for resisting the slippage of the strand.

6. Bond stress is assumed to be a function of friction and the combination of adhesion and mechanical action.

7. The analysis may be used at any section where cracking has not occurred.

The general expression for axial stress in the strand at any time, \( \sigma_Z \), is given by

\[
\sigma_Z = f_{si} - \Delta f_{s} - \Delta f_{cs} + n \sigma_L
\]

where

- \( f_{si} \) = initial strand stress
- \( \Delta f_{s} \) = loss due to elastic deformation at release
- \( \Delta f_{cs} \) = loss due to creep and shrinkage
- \( \sigma_L \) = stress in the concrete produced by loading
- \( n = \frac{E_s}{E_c} \) = modular ratio

For \( \sigma_Z \), tensile stress is considered to be positive.

At this point, it will be desirable to express \( \Delta f_{cs} \) as a function of other quantities:

\[
\Delta f_{cs} = (C_c - 1) n f_c
\]
where \( C_c \) is defined as the creep coefficient and \( f_c \) = prestress stress in the concrete at the level of the strand.

Then

\[
f_c = (f_{s1} - \Delta f_s) \left( \frac{A_s}{A_c} + \frac{A_e}{S_b} \right)
\]

where \( A_s \) = total area of prestressing strand,

\( A_c \) = cross-sectional area of the concrete beam,

\( e \) = eccentricity of the prestress force,

\( S_b \) = section modulus of the beam with respect to the level of the strand.

If

\[
K_1 = (C_c - 1) n \left( \frac{A_s}{A_c} + \frac{A_e}{S_b} \right)
\]

Then

\[
\Delta f_{cs} = K_1 (f_{s1} - \Delta f_s)
\]

and

\[
\sigma_z = (1 - K_1) (f_{s1} - \Delta f_s) + n\sigma_L
\]

Considering the radial deformation of the strand

\[
\Delta r_s = \Delta r_1 - \Delta r_2
\]

where \( r_0 \) = original radius of the strand at zero stress

\( \Delta r_1 \) = change in the radius of the strand due to the initial stress \( f_{s1} \)
\( \Delta r_2 = \) change in the radius of the strand to \( \sigma_Z \) and radial stress \( \sigma_r \)

By applying the expression for radial strain in cylindrical coordinates, \( \Delta r_1 \) and \( \Delta r_2 \) can be expressed as:

\[
\Delta r_1 = - \frac{f_s}{E_s} \sigma_r r_0 \tag{5}
\]

\[
\Delta r_2 = \frac{r_0}{E_s} \left( - \sigma_r - \mu_s \sigma_Z \right) \tag{6}
\]

Then

\[
\Delta r_s = - \frac{f_s}{E_s} \sigma_r r_0 + \frac{r_0}{E_s} \left( \sigma_r + \mu_s \sigma_Z \right) \tag{7}
\]

Substituting Eq. (4) into Eq. (7), the expression for \( \Delta r_s \) can be written as:

\[
\Delta r_s = \frac{r_0}{E_s} \sigma_r + \frac{r_0}{E_s} \mu_s \left[ n \sigma_L - K_l f_s \sigma_r - \left( 1 - K_l \right) \Delta f_s \right] \tag{8}
\]

An expression will now be developed for \( \Delta r_c \), the radial displacement of the inner face of the concrete surrounding the strand. As mentioned previously, this concrete is assumed to be in the form of a concrete cylinder.

\[
\varepsilon_r = \frac{1}{E_c} \left[ \sigma_r - \mu_c \left( \sigma_\theta - \sigma_w \right) \right]
\]

Since \( \sigma_\theta = - \sigma_r \) at the inner surface

\[
\varepsilon_r = \frac{1}{E_c} \left[ \sigma_r \left( 1 + \mu_c \right) - \mu_c \sigma_w \right] \tag{9}
\]

where \( \sigma_w = \) longitudinal stress in the concrete cylinder due to prestress force and external load. Compression is considered to be positive.
The expression for $\sigma_w$ is

$$\sigma_w = \frac{F}{A_c} + \frac{F_e}{A_b} - \sigma_L \quad (10)$$

where

$$F = (f_{si} - \Delta f_s + \Delta f_{cs}) A_s \quad (11)$$

Substituting Eq. (11) into Eq. (10),

$$\sigma_w = (1 - K_1) (f_{si} - \Delta f_s) \left( \frac{A_s}{A_c} + \frac{A_e}{A_b} \right) - \sigma_L$$

If

$$K_2 = (1 - K_1) \left( \frac{A_s}{A_c} + \frac{A_e}{A_b} \right) \quad (12)$$

Then

$$\sigma_w = K_2 (f_{si} - \Delta f_s) - \sigma_L \quad (13)$$

Combining Eq. (13) and Eq. (9),

$$\Delta r_r = r_0 \left[ \sigma_r (1 + \mu_r) - \mu_r \left[ K_2 (f_{si} - \Delta f_s) - \sigma_L \right] \right] + \Delta r_c \quad (14)$$

Because of radial creep of concrete, the total radial displacement of the inner face of the concrete surrounding the strand will be given by:

$$\Delta r_c = (\Delta r_r + \Delta r_c)_{\text{elastic}} + (\Delta r_c)_{\text{creep}}$$

$$\Delta r_c = r_0 \left( \varepsilon_r \right)_{\text{elastic}} \left[ 1 + \frac{\left( \varepsilon_r \right)_{\text{creep}}}{\left( \varepsilon_r \right)_{\text{elastic}}} \right]$$

If

$$C_{cr} = \frac{\left( \varepsilon_r \right)_{\text{creep}}}{\left( \varepsilon_r \right)_{\text{elastic}}}$$

where $C_{cr} =$ radial creep factor
then
\[ \Delta r_c = r_0 \left( \varepsilon_r \right)_{\text{elastic}} \] (15)

Substituting Eq. (14) into Eq. (15)

\[ \Delta r_c = \frac{r_0}{E_c} \left( 1 + C_{cr} \right) \left[ \sigma_r \left( 1 + \mu_c \right) - \mu_c \left[ K_2 (f_{si} - \Delta f_s) - \sigma_L \right] \right] \] (16)

Since \( \Delta r_s \) must be equal to \( \Delta r_c \), an expression for \( \sigma_r \) may be derived by equating Eq. (8) to Eq. (16)

\[ \sigma_r = f_{si} \left[ 1 - n \left( 1 + \mu_c \right) \left( 1 + C_{cr} \right) \right] \] (17)

where tensile stress is considered to be positive for \( \sigma_r \).

If \( K_3, K_4, \) and \( K_5 \) are defined as

\[ K_3 = \frac{[\mu_s K_1 - n \mu_c K_2 (1 + C_{cr})]}{[1 - n \left( 1 + \mu_c \right) \left( 1 + C_{cr} \right)]} \] (18)

\[ K_4 = \frac{-[\mu_s (1 - K_1) + n \mu_c K_2 (1 + C_{cr})]}{[1 - n \left( 1 + \mu_c \right) \left( 1 + C_{cr} \right)]} \] (19)

\[ K_5 = \frac{[n \mu_s + n \mu_c (1 + C_{cr})]}{[1 - n \left( 1 + \mu_c \right) \left( 1 + C_{cr} \right)]} \] (20)

Then

\[ \sigma_r = K_3 f_{si} - K_4 \Delta f_s + K_5 \sigma_L \] (21)

Consider a free body diagram of a differential length of strand:
From equilibrium

\[
\frac{d\sigma_Z}{dZ} = \frac{a}{A_s} \tau
\]  

(22)

where \( \tau \) = unit bond stress
\( a \) = perimeter of the strand
\( A_s \) = area of the strand

Assuming that

\[
\tau = \varphi (-\sigma_r) + \beta
\]  

(23)

where \( \varphi \) = coefficient of friction between steel and concrete
\( \beta \) = adhesion and mechanical action
\( -\sigma_r \) = compressive radial stress

Substituting Eq. (23) into Eq. (22)

\[
\frac{d\sigma_Z}{dZ} = \frac{a}{A_s} [\varphi (-\sigma_r) + \beta]
\]  

(24)
In order to evaluate $\Delta f_s$, it is realized that $\sigma_L$ is equal to zero at the time of release of the prestress force. Substituting the values of $\sigma_r$ and $\sigma_Z$ into Eq. (24), and after simplifying, the resulting equation is

$$\frac{d\Delta f_s}{dz} + \frac{\Phi}{A_s} \frac{K_4}{(1 - K_1)} \Delta f_s = \frac{\Phi}{A_s} \frac{K_3}{(1 - K_1)} f_{si} - \frac{\beta}{A_s} \frac{1}{(1 - K_1)}$$  \hspace{1cm} (24)

The solution of the homogeneous equation is

$$\begin{align*}
\Delta f_s^h \ &= \ & C_1 e^{-\frac{\Phi}{A_s} \frac{K_4}{(1 - K_1)} z} \\
\Delta f_s^p \ &= \ & C_2 
\end{align*}$$  \hspace{1cm} (25)

For a particular solution assume

$$\Delta f_s^p = C_2$$

After evaluating $C_2$, the solution of Eq. (24) is

$$\Delta f_s = C_1 e^{-\frac{\Phi}{A_s} \frac{K_4}{(1 - K_1)} z} + \frac{K_3}{K_4} f_{si} - \frac{\beta}{\Phi} \frac{1}{K_4}$$

The boundary conditions are

at $Z = 0$, \hspace{0.5cm} $\Delta f_s = f_{si}$

Thus

$$C_1 = f_{si} \left(1 - \frac{K_3}{K_4}\right) + \frac{\beta}{\Phi} \frac{1}{K_4}$$

and

$$\Delta f_s = \left[f_{si} \left(1 - \frac{K_3}{K_4}\right) + \frac{\beta}{\Phi} \frac{1}{K_4} \right] e^{-\frac{\Phi}{A_s} \frac{K_4}{(1 - K_1)} z} + \frac{K_3}{K_4} f_{si} - \frac{\beta}{\Phi} \frac{1}{K_4}$$  \hspace{1cm} (26)
Substituting Eq. (26) into Eq. (4)

\[
\sigma_z = (1 - K_1) \left[ f_s i \left( 1 - \frac{K_3}{K_4} \right) + \frac{1}{\Phi} \frac{1}{K_4} \right] \left[ 1 - e^{-\frac{a}{A_s} \frac{K_4}{(1 - K_1)} Z} \right] + n \sigma_L \tag{27}
\]

In Eq. (27), \( \sigma_L \) is the stress produced by loading, and is defined as:

\[
\sigma_L = \frac{R}{S_c} (Z - L_o)
\]

where

- \( R \) = end reaction
- \( S_c \) = section modulus of the composite cross-section with respect to the level of the strand
- \( Z \) = distance from the end of the member
- \( L_o \) = overhang, which is the distance from the end of the member to the end reaction

In this report, the analytical concept has not been compared with the experimental results, mainly due to the lack of information on the values for the coefficients \( \Phi, \beta, C_{cr}, \) and \( C_c \). If the analysis is valid, and these coefficients can be properly evaluated, the plot of \( \sigma_z \) as given by Eq. (27) can be compared with the experimental results given in Figs. 19 through 29.

At this point it should be pointed out that a review of experimental results indicates that for all specimens in which general bond slip occurred, the stress in the strand at the end of the anchorage length had reached the magnitude \( (f_{si} - \Delta f_{cs}) \). In the specimens in which flexural failure occurred prior to bond slip, the stress in the strand at the end of the anchorage length had not reached \( (f_{si} - \Delta f_{cs}) \). Consequently, a simple check as to whether general bond slip will occur prior to flexural failure would involve a computation
of the stress in the strand at the end of the anchorage length. If
the computed value of $\sigma_Z$ is less than $(f_{si} - \Delta f_{cs})$, then general bond
slip will not have occurred at that load. If the computed value of
$\sigma_Z$ is greater than $(f_{si} - \Delta f_{cs})$, then general bond slip would have
occurred.
6. **SUMMARY AND CONCLUSIONS**

The objective of this investigation was the development of information on the flexural bond characteristics of prestressed concrete beams pre-tensioned with \( \frac{1}{2}\)-in. 270 K, stress-relieved seven-wire strand.

The test specimens were designed to represent the most critical case relative to bond failure. A total of thirteen tests were conducted on twelve beams to evaluate the influence of variation in the embedment length upon the ultimate load carrying capacity of the beams. Seven of the specimens with short embedment lengths failed by general bond slip at a load smaller than the computed ultimate load for flexural failure. Five of the specimens failed in flexure, generally at a load slightly higher than the computed ultimate load.

The test results were compared with the results obtained by Hanson and Kaar from the tests conducted on beams prestressed with \( \frac{1}{2}\)-in. conventional strand (nominal \( f' = 250,000 \text{ psi} \)). It was found that for a given embedment length, a higher axial stress could be developed prior to slip in the 270 K strand than in the conventional strand. Therefore, the maximum bond stress which was developed prior to slip was greater for the 270 K strand.

The critical embedment length, which is defined as the shortest embedment length required to develop the minimum specified ultimate strength of the strand, was found to be 80 inches for the \( \frac{1}{2}\)-in. 270 K strand. This length is considerably shorter than the 135 inches found by Hanson and Kaar for the \( \frac{1}{2}\)-in. conventional strand.
The results of the tests given in Figs. 19 through 29 clearly indicate the movement of the region of high flexural bond stress toward the end of the members, as the ultimate load is approached. The general bond slip occurred when the region of high flexural bond stress overlapped the prestress transfer zone. Analytically, the results indicate that when the stress in the strand at the end of the transfer zone reaches the value \( f_{si} - \Delta f_{cs} \), general bond slip will occur.

An analytical concept was developed describing the state of the stress in the strand at any time, and any location. This relationship is valid for any section, provided that cracking has not occurred between that section and the end of the beam. The value of the stress at the end of the prestress transfer zone can be computed for an ultimate load which theoretically would cause flexural failure. The calculated stress in the strand can then be compared to the value of the initial prestress, less the losses due to creep and shrinkage \( f_{si} - \Delta f_{cs} \). If the calculated stress is larger than \( f_{si} - \Delta f_{cs} \), general bond slip will occur prior to flexural failure.

One factor which may have a major effect on the occurrence of bond slip is the pinching effect of the end reaction. In this investigation, as in those previously reported, the strands studied were very near to the bottom of the member. In a beam containing layers of strand, it is possible that bond slip may be more critical in the strands at the upper layers, since the pinching action would probably be less effective at a higher level. Therefore, it is felt that this effect should be studied in further investigations.
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8. NOTATION

a  circumference of the strand
A_c  cross-sectional area of the beam
A_s  cross-sectional area of the steel
b  width of the beam cross-section
B  width of the slab
c  depth of compression zone
c.g.  center of gravity of beam cross-section
C_c  creep coefficient
C_cr  creep coefficient for radial direction
C  horizontal component of the resultant compressive force in the concrete
d  effective depth
e  eccentricity of the prestress force
E_c  modulus of elasticity of concrete
E_s  modulus of elasticity of steel
f_c  stress in the concrete
f'_c  ultimate compressive strength of concrete
f'_s  ultimate tensile strength of steel
f_{si}  initial prestress stress in the strand
f_{sp}  splitting tensile strength of concrete
f_{su}  stress in the strand at ultimate moment
F = \frac{\epsilon_{su} - \epsilon_{se}}{\epsilon_{cu}}
k_1, k_2, k_3  ultimate strength factors
\[ K_1 = (C_c - 1) \frac{n}{A_c} \left( \frac{A_s}{A_c} + \frac{A_e}{S_b} \right) \]

\[ K_2 = (1 - K_1) \frac{A_s}{A_c} + \frac{A_e}{S_b} \]

\[ K_3 = \frac{[\mu_s K_1 - n \mu_c K_2 (1 + C_{cr})]}{[1 - n (1 + \mu_c) (1 + C_{cr})]} \]

\[ K_4 = - \frac{[\mu_s (1 - K_1) + n \mu_c K_2 (1 + C_{cr})]}{[1 - n (1 + \mu_c) (1 + C_{cr})]} \]

\[ K_5 = - \frac{[n \mu_s + n \mu_c (1 + C_{cr})]}{[1 - n (1 + \mu_c)(1 + C_{cr})]} \]

- \( L \) length of the beam
- \( L_e \) length of the embedment
- \( L_o \) length of overhang
- \( L_s \) length of shear span
- \( L_t \) prestress transfer length
- \( M_{\text{flex}} \) computed ultimate flexural moment
- \( n \) \( E_s/E_c \) : modular ratio
- \( P_{\text{ult}} \) computed ultimate load
- \( P_{\text{test}} \) measured load at time of failure
- \( r_o \) radius of the strand at zero stress
- \( R \) end reaction
- \( S_b \) section modulus of the beam cross-section with respect to the level of the strand
- \( S_c \) section modulus of the composite cross-section with respect to the level of the strand
\( T \)  
resultant tensile force in the strand

\( \Delta f_s \)  
loss due to elastic deformation at release

\( \Delta f_{cs} \)  
loss due to creep and shrinkage

\( \Delta r_1 \)  
change in the radius of the strand due to the initial stress \( f_{si} \)

\( \Delta r_2 \)  
change in the radius of the strand due to radial and longitudinal stresses

\( \Delta r_s \)  
\( \Delta r_1 - \Delta r_2 \)

\( \Delta r_c \)  
change in the internal radius of the concrete cylinder surrounding the strand

\( \beta \)  
contribution of adhesion and mechanical resistance

\( \varphi \)  
coefficient of friction between strand and concrete

\( \varepsilon_{cu} \)  
tensile strain in concrete at level of the strand at ultimate moment

\( \varepsilon_r \)  
radial strain

\( \varepsilon_{se} \)  
tensile strain in the strand due to effective prestress

\( \varepsilon_{su} \)  
tensile strain in the strand at ultimate moment

\( \varepsilon_u \)  
ultimate compressive strain of concrete

\( \sigma_r \)  
radial stress, tension positive

\( \sigma_L \)  
flexural stress in the concrete due to loading

\( \sigma_Z \)  
longitudinal stress in the strand at any time, tension positive

\( \sigma_w \)  
longitudinal stress in the concrete cylinder surrounding the strand, compression positive

\( \tau \)  
unit bond stress

\( \mu_c \)  
Poisson's ratio of concrete

\( \mu_s \)  
Poisson's ratio of strand
9. TABLES
Table 1  Overall Dimensions of the Test Specimens

<table>
<thead>
<tr>
<th>Beam</th>
<th>Length, ft.</th>
<th>Embedment Length, ( L_e ), in.</th>
<th>Shear Span, ( L_s ), in.</th>
<th>Slab Thickness, in.</th>
<th>Slab Width, in.</th>
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<td>66</td>
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<td>19(\frac{1}{2})</td>
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<td>19(\frac{1}{2})</td>
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Table 2  Properties of Concrete

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Table 3  Measured Prestress Losses

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<th>Beam</th>
<th>*Initial stress, kips</th>
<th>Loss at release, kips</th>
<th>Stress after release, kips</th>
<th>% Loss after release</th>
<th>Add. Loss prior to test, kips</th>
<th>Total loss, kips</th>
<th>Total loss, %</th>
<th>Effective stress, kips</th>
<th>Transfer length, in.</th>
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<td>+21.52</td>
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* Prior to placing concrete
+ Average of other tests
### Table 4  Losses in Prestressing Strand

<table>
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<tr>
<th>Losses</th>
<th>Elast. Shortening, kips</th>
<th>Shrinkage, kips</th>
<th>Creep, kips</th>
<th>Total Loss, kips</th>
<th>Initial Stress, kips</th>
<th>Stress After Release, kips</th>
<th>Stress at Test, kips</th>
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<tr>
<td>Measured Average</td>
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<td>28.90</td>
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<td>21.52</td>
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Table 5  Computed Ultimate Moment Capacities of Test Specimens

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<th>$f'_c$</th>
<th>$f'_c$</th>
<th>$k_1$</th>
<th>$k_2$</th>
<th>$\varepsilon_u$</th>
<th>$f_{se}$</th>
<th>$f_{se} \times 10^{-3}$</th>
<th>$f_{su}$</th>
<th>$M_u$</th>
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<td></td>
<td>beam,</td>
<td>slab,</td>
<td></td>
<td></td>
<td></td>
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<td>0.43</td>
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<td>250</td>
<td>985</td>
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<tr>
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<td>5920</td>
<td>-</td>
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<td>0.43</td>
<td>3.089</td>
<td>135.7</td>
<td>4.88</td>
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<td>1670</td>
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Table 6  Summary of Test Results

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<tr>
<th>Beam No.</th>
<th>Embed. length, in.</th>
<th>Shear span, in.</th>
<th>Reactions at failure, kips</th>
<th>P&lt;sub&gt;test&lt;/sub&gt; at failure, kips</th>
<th>P&lt;sub&gt;ult.&lt;/sub&gt; (Theoretical) kips</th>
<th>P&lt;sub&gt;test&lt;/sub&gt; / P&lt;sub&gt;ult.&lt;/sub&gt;</th>
<th>M&lt;sub&gt;flex&lt;/sub&gt; Calc. Ult., k-in.</th>
<th>Maximum Stress in Strand, ksi</th>
<th>Mode of Failure&lt;sup&gt;+&lt;/sup&gt;</th>
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<td>42</td>
<td>36-204</td>
<td>*41.25-8.75</td>
<td>50.0</td>
<td>54.4</td>
<td>0.919</td>
<td>1670</td>
<td>196</td>
<td>B</td>
</tr>
<tr>
<td>F-1W</td>
<td>54</td>
<td>48-78</td>
<td>*34.40-19.60</td>
<td>54.0</td>
<td>56.0</td>
<td>0.964</td>
<td>1670</td>
<td>212</td>
<td>B</td>
</tr>
<tr>
<td>F-2</td>
<td>42</td>
<td>30</td>
<td>55.50</td>
<td>111.0</td>
<td>111.2</td>
<td>0.998</td>
<td>1670</td>
<td>233</td>
<td>B</td>
</tr>
</tbody>
</table>

* See Fig. 11

<sup>+</sup> F - Flexural
B - Bond
Slab - Failure of epoxy shear connection
10. FIGURES
Prestress Stress Distribution

Immediately after release
Elastic Shortening included

-120 -100
+1200 +970
+2135 +1730
+2520 psi +2035 psi

After all losses

Strand: Type 270k, 1/2-inch.
Shear Reinforcement: No. 3 deformed bars

Fig. 1 Dimensions of the Test Specimens
Fig. 2  Stress and Strain Distributions at Ultimate Load

Fig. 3  Load-Elongation Curve for the 1/8-in. 270 K Strand
D_1 = 0.174 in.

D_2 = 0.167 in.

D = D_1 + 2D_2 = 0.508 in.

a = 2.139 in.

A_s = 0.1531 in^2

\frac{a}{A_s} = 13.95 in./in^2

Fig. 4  Cross-Sectional Properties of the \( \frac{1}{2} \)-in. 270 K Strand
Fig. 5 Location of the Strain Gages on the Strands
Fig. 6 Location of the Strain Gages on the Strands
Fig. 7  Calibration Curve for Strain Gages Mounted on Individual Wires of the Strand
Fig. 8  Assembly of the End-Slip Dial Gages
Fig. 9 Testing Arrangement
Fig. 10  Test Specimen E-1 after Completion of Test
Fig. 11 Testing Arrangement for Beam F-1
Fig. 12  Crack Pattern of Beam A-2

Fig. 13  Crack Pattern of Beam D-2
Fig. 14  Crack Pattern of Beam F-1E

Fig. 15  Crack Pattern of Beam F-2
Fig. 16 Crack Pattern of Beam A-1

Fig. 17 Crack Pattern of Beam C-1
Fig. 18 Slab Failure in Beam B-1
Fig. 19 Force Distribution in the Strand

Beam A-1, West End
L₀ = 96"
Fig. 20 Force Distribution in the Strand
Fig. 21  Force Distribution in the Strand
Fig. 22 Force Distribution in the Strand
Fig. 23  Force Distribution in the Strand
Fig. 24  Force Distribution in the Strand

Beam D-I, West End
$L_e = 96''$
Fig. 25  Force Distribution in the Strand
Fig. 26  Force Distribution in the Strand
Fig. 27  Force Distribution in the Strand

Beam F-1, East End

$L_e = 42''$

$P = 50$ kips

$P = 35$ kips

$P = 25$ kips

$P = 0$ kips

$f_{si} = 28.90$ kips

$(f_{si} - \Delta f_{cs}) = 24.08$ kips

DISTANCE FROM END (inches)

force distribution in the strands (kips)

10

5

0
Fig. 28 Force Distribution in the Strand
Fig. 29    Force Distribution in the Strand
Fig. 30 Effect of Embedment Length on the Load Carrying Capacity of the Member
Average bond stress required to develop full tensile strength of strand (270,000 psi)

- General Bond Slip (a-12 in. overhang)
- Flexural Failure

Bond stress at which slip will probably occur

Fig. 31 Effect of Embedment Length on Average Bond Stress for \( \frac{1}{4} \)-in. 270 K Strand
Average bond stress required to develop full tensile strength of strand.

- General Bond Slip (0-12 in. overhang)
- Flexural Failure
- Hanson-Kaar

270 k, \( \frac{1}{2} \)-inch Strand
Nominal \( f_s = 270,000 \) psi

Conventional, \( \frac{1}{2} \)-inch Strand
Nominal \( f_s = 250,000 \) psi
( Hanson-Kaar)

Fig. 32 Comparison of Average Bond Stresses of \( \frac{1}{2} \)-in. 270 K Strand with \( \frac{1}{2} \)-in. Conventional Strand
Strand: 270 k 1/8-inch

Nominal Concrete Strength

\( f_{ci} = 4,500 \text{ psi at release} \)

\( f_{ct} = 6,000 \text{ psi at test} \)

Strand Embedment Length (in inches)

Fig. 33 Embedment Length vs. Stress in the 1/8-in. 270 K Strand at General Bond Slip
Fig. 34  Comparison of Strand Stress at General Bond Slip of the \( \frac{1}{2} \)-in. 270 K Strand with the \( \frac{1}{2} \)-in. Conventional Strand (Hanson-Kaar)
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