PCISFRAD Project No. 2

# Development Length of Prestressing Strands, Including Debonded Strands, and Allowable Concrete Stresses in Pretensioned Members

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**Note:** This paper is an edited version of Chapter 4 which is part of PCISFRAD Project No. 2, "Exceptions of Precast, Prestressed Members to Minimum Reinforcement Requirements." The full report is available from PCI Headquarters at \$10.00 to firms supporting the sponsored research, \$15.00 to PCI Members (nonsupporting firms) and \$30.00 to non-PCI Members.

The edited paper, and the full report, are based on a research project supported by the PCI Specially Funded Research and Development (PCISFRAD) Program. The conduct of the research and the preparation of the final reports for each of the PCISFRAD projects were performed under the general guidance and direction of selected industry Steering Committees. However, it should be recognized that the research conclusions and recommendations are those of the researchers. The results of the research are made available to producers, engineers and others to use with appropriate engineering judgment similar to that applied to any new technical information.

# 1. SUMMARY AND CONCLUSIONS

This paper represents a part of the Prestressed Concrete Institute Specially Funded Research and Development (PCISFRAD) Program, Project Report No. 2, "Exceptions of Precast, Prestressed Members to Minimum Reinforcement Requirements (of American Concrete Institute Standard ACI 318-83)."[1] The results of the other parts of the investigation will be published in the PCI JOURNAL and elsewhere.

In this paper the adequacy, realism and/or conservatism of Section 12.9 of ACI 318-83,[2] entitled "Development of Prestressing Strand," are examined. Particular attention is paid to the double development length requirement for debonded strands (Section 12.9.3 of ACI 318-83). The code provisions are evaluated through close scrutiny of test results available in the literature.

Also examined are the allowable concrete stresses of Section 18.4 of ACI 318-83. The history of this section is traced back to a 1958 report by ACI-ASCE Committee 423 (323), on which the very first chapter on prestressed concrete in an ACI Code (1963 edition) was based.

The following conclusions can be drawn on the basis of discussion in this paper:

1. The ACI 318-83 equation giving development length requirement for prestressing strand is based on good experimental authority. Certain investigators have proposed making the provisions more conservative, while others have found the requirements adequate. There does not appear to be any compelling basis for any significant change to the current provisions. In the case of short span members where the full development length required by the Code cannot be provided, the approach suggested in Ref. 3 [Eqs.(3) and (4)] may prove useful.

2. The double development length requirement for debonded strand (Section 12.9.3) is also based on reliable experimental evidence. Beams with debonded strands using single development lengths have shown a lack of performance, while those using double development lengths have performed satisfactorily. However, tests on beams with debonded strands using development lengths between one and two times those required by the Code have not been carried out. Such tests are needed to justify any possible relaxation of the provisions of Section 12.9.3.

Most of the allowable concrete stresses in Section 18.4 of the Code have been in use for a long time, and are linked with an extended record of satisfactory performance. The most recent modification (1977 Code) allowing a tensile stress of up to $6\sqrt{f'_{ci}}$ immediately upon transfer of prestress at the ends of simply supported members has not generated any adverse reports of lack of performance. Further modifications do not appear to be warranted at the present time. However, relaxation in two possible areas may be worthwhile pursuing in the future:

(a) Increasing the allowable compressive stress immediately after prestress transfer from $0.60 f'_{ci}$ to $0.70 f'_{ci}$ at the ends of simply supported members may not have an adverse effect on performance. However, this change needs to be verified in carefully conducted tests.

(b) The allowable tensile stresses, immediately after prestress transfer, of $3\sqrt{f'_{ci}}$ and $6\sqrt{f'_{ci}}$ are indirectly linked to the modulus of rupture. It may be possible to increase these stresses somewhat, at least for concrete produced under plant controlled conditions, if modulus of rupture tests on such concrete shows consistently high values (significantly in excess of $7.5\sqrt{f'_{ci}}$). A great many such modulus of rupture tests need to be carried out. Satisfactory performance of members designed on the basis of higher allowable initial tension stresses will also have to be established through careful testing, before a relaxation of the current stress limits can be sought.

It may be of interest to note that Herman Himes of Thomas Concrete Products, Oklahoma City, in reviewing this manuscript, took issue with Conclusion 2 above in the following words:

"We do agree that inadequate information is available to justify any changes to the development length if one is considering dynamic loading . . . However, there appears to be adequate evidence that one development length for debonded strand is sufficient for static loading conditions. We would suggest that, until additional testing is done, the Code be left as is for dynamic conditions such as bridges and parking garages and a modifier added which allows one development length for static load conditions such as found in most commercial buildings."

The authors, on re-reviewing the Kaar-Magura test results,[4] find themselves unable to agree with this recommendation.

The following observation quoted from Ref. 4 is relevant:

"Load-deflection observations were made in the static tests to detect, if possible, any bond slip in the strands in the working load range during the 5 million load cycles. The results of these observations showed no evidence of bond failure at these load levels. The load-deflection relation was linear and in none of the girders tested was there a significant difference between the relation at the first loading and that after 5 million load cycles."

Yet in subsequent static tests to failure, the performance of the beam with debonded strands using single development length was poorer than that of similar beams using double development lengths, and also poorer than the performance of beams with nonblanketed strands.

## 2. TRANSFER LENGTH AND DEVELOPMENT LENGTH

The following two paragraphs, adopted from Ref. 5, clarify some of the basic concepts that are relevant to the discussion in this paper:

In pretensioned members, the total force of prestressing is transferred to the concrete entirely by the bonding of the prestressing strand to the concrete surrounding it. This differs from post-tensioned construction, where the full compressive force is transferred to the concrete cross section by means of special end anchorages and bearing plates.

The bond mechanism in pretensioned members is accomplished in two ways, that is by transfer bond and flexural bond. Transfer bond is mobilized by the initial tensioning and release of the strand, and the length over which the initial prestress force is delivered to the concrete is termed the "transfer" bond length. Flexural bond becomes mobilized as the member is subjected to bending as a result of externally applied loads. As external loads increase, the resulting stress in the strand also increases. The additional length over which this increase in force is transferred is known as the "flexural" bond length. As the ultimate capacity of the member is approached, the length of strand required to transfer the full force in the strand, transfer length plus flexural bond length, is termed the "development" length.

If inadequate development length is provided, ultimate strength is governed by bond rather than by flexure.[6] Bond slippage of the strands occurs in three stages: (1) progressive bond slip begins at flexural cracks, (2) general bond slip is initiated along the entire development length, and (3) the mechanical interlock between the helical strand surface and the concrete is destroyed.

Kaar and Magura[4] pointed out that the mechanical interlock is adequate to maintain considerable strand stress even after extensive bond slip. In many cases the strand stress after general bond slip drops only toward the prestress level and not to zero as one might fear. Thus, the final effect of inadequate development length may be a premature flexural failure at a reduced strand stress, corresponding to a final bending moment less than the computed ultimate strength in flexure.

# 3. CURRENT CODE PROVISIONS

The present provisions for development length of prestressing strand are contained in Section 12.9 of ACI 318-83.[2] The provisions read as follows:

**Section 12.9.1** — Three- or seven-wire pretensioning strand shall be bonded beyond the critical section for a development length, in inches, not less than:

$$\left(f_{ps} - \frac{2}{3} f_{se}\right) d_b \qquad (1)$$

where

$f_{ps}$ = stress in prestressed reinforcement at nominal strength, ksi

$f_{se}$ = effective stress in prestressed reinforcement (after allowance for all losses), ksi

$d_b$ = nominal strand diameter, in.

and the expression in parentheses is used as a constant without units.

**Section 12.9.2** — Investigation may be limited to cross sections nearest each end of the member that are required to develop full design strength under specified factored loads.

**Section 12.9.3** — Where bonding of a strand does not extend to end of member, and design includes tension at service load in precompressed tensile zone as permitted by Section 18.4.2, development length specified in Section 12.9.1 shall be doubled.

In the Commentary to the ACI Code,[7] the equation in Section 12.9.1 is rewritten as:

$$l_d = \frac{f_{se}}{3} d_b + (f_{ps} - f_{se})\, d_b \qquad (2)$$

where the first and second terms represent transfer length and flexural bond length, respectively.

The effective steel stress $f_{se}$ obviously depends on the initial prestress, $f_{si}$, and the amount of prestress loss. Zia and Mostafa[8] have pointed out that the denominator "3" in the expression for transfer length represents a conservative average concrete strength in ksi.

Similarly, in the expression for flexural bond length, a denominator of 1 ksi (6.9 MPa) is implied, which represents an average bond stress of 250 psi (1.7 MPa) within the development length [see Eq. (8)].

According to the ACI Code requirement, the transfer length would be 47 nominal strand diameters and the flexural bond length would be 110 strand diameters for 250 ksi (1725 MPa) grade strand, assuming an initial prestress of $0.7f_{pu}$ (where $f_{pu}$ is the specified tensile strength of prestressing strand, ksi) and a 20 percent loss of prestress.[8]

Similarly, for 270 ksi grade strand, the transfer length would be 51 strand diameters and the flexural bond length would be 119 strand diameters. Note that the value of 50 strand diameters is mentioned as the assumed transfer length in Section 11.4.3 of ACI 318-83.

## 4. PARTIAL RESULTS FROM INDUSTRY SURVEY

As a part of the broader investigation[1] of which this study formed a part, American and Canadian prestressed concrete producers were surveyed[9] about their concerns with the ACI Code requirements governing the design and manufacture of precast prestressed elements. One of the questions included in the survey was: "Do the provisions governing the development of prestressing strand (Section 12.9) pose any hardship?" The answers were 10 yes and 29 no. Of the 10 yes answers, 8 related to doubling the development length for sheathed strands:

1. Section 12.9.3 is too severe (from two respondents).

2. Section 12.9.3 does not make any sense. Why should $l_d$ be doubled? Does it make any difference if the strand is debonded in 6 in. (150 mm) length or say 10 ft (3 m) length? Per this section debonding will cause problems in most prestressed members of moderate 20 to 30 ft (6 to 9 m) length.

3. Doubling the development length for wrapped strands.

4. Seems excessive; otherwise not a problem for our members.

5. For sheathed strand the extended bond development is too great based on our observations. Otherwise, I do not consider the strand development provisions a "hardship."

6. Masking is a real problem if complying with Section 12.9.3.

Other comments claiming hardship were:

1. On very short span members the development length creates a theoretical problem in flexural strength.

2. Difficulties are experienced on heavily loaded short spans.

3. Development length is long and poses some difficulties when holes are cut in hollow-core floor slabs. Research to prove that the ultimate tensile strength of strand can be developed in a shorter length would be welcome. Not a problem insofar as double tees are concerned.

4. Section 12.9.1 of ACI 318-83 needs 170 $d_b$ development length.

5. The term $(f_{ps} - f_{se})\ d_b$ in the Code equation for development length is excessive. However, this requirement is generally a problem in short simple span members in which case the strand diameter must be reduced. Experience with railroad ties seems to indicate the conservative nature of this requirement.

6. Our experience shows that the prevention of splitting during detensioning merits the use of short length (5 ft) (1.5 m) shear reinforcing in the ends of double tees. There is reinforcing then, in the ends in the development length region regardless of Code provisions.

7. Generally, double tees have long spans and development is not a problem.

8. Dapped end reinforcing is too heavy. Development length of 2.0 $l_d$ causes congestion in dapped regions. The factor 1.0 $l_d$ worked before provisions were changed.

## 5. EVALUATION OF THE ACI CODE EQUATION FOR DEVELOPMENT LENGTH OF STRAND

More than 30 separate investigations have been reported in the literature concerning bond development length for prestressing steel.[8] However, many of these tests were performed with small wires and not the multi-wire strands currently used in the United States and Canada. Discussion here is limited to

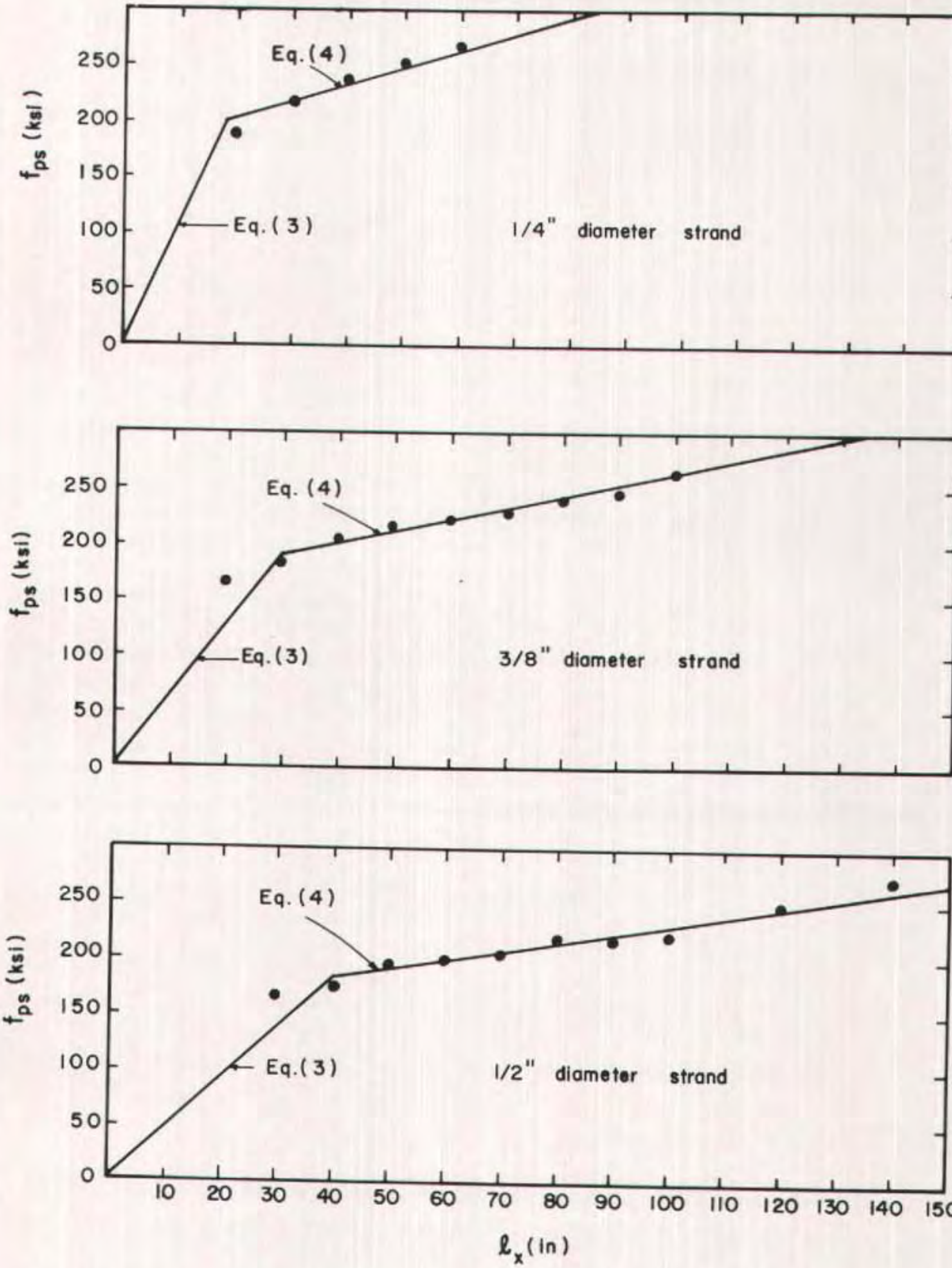


Fig. 1. Comparison of Martin-Scott equations[3] with test results by Hanson and Kaar.[6]

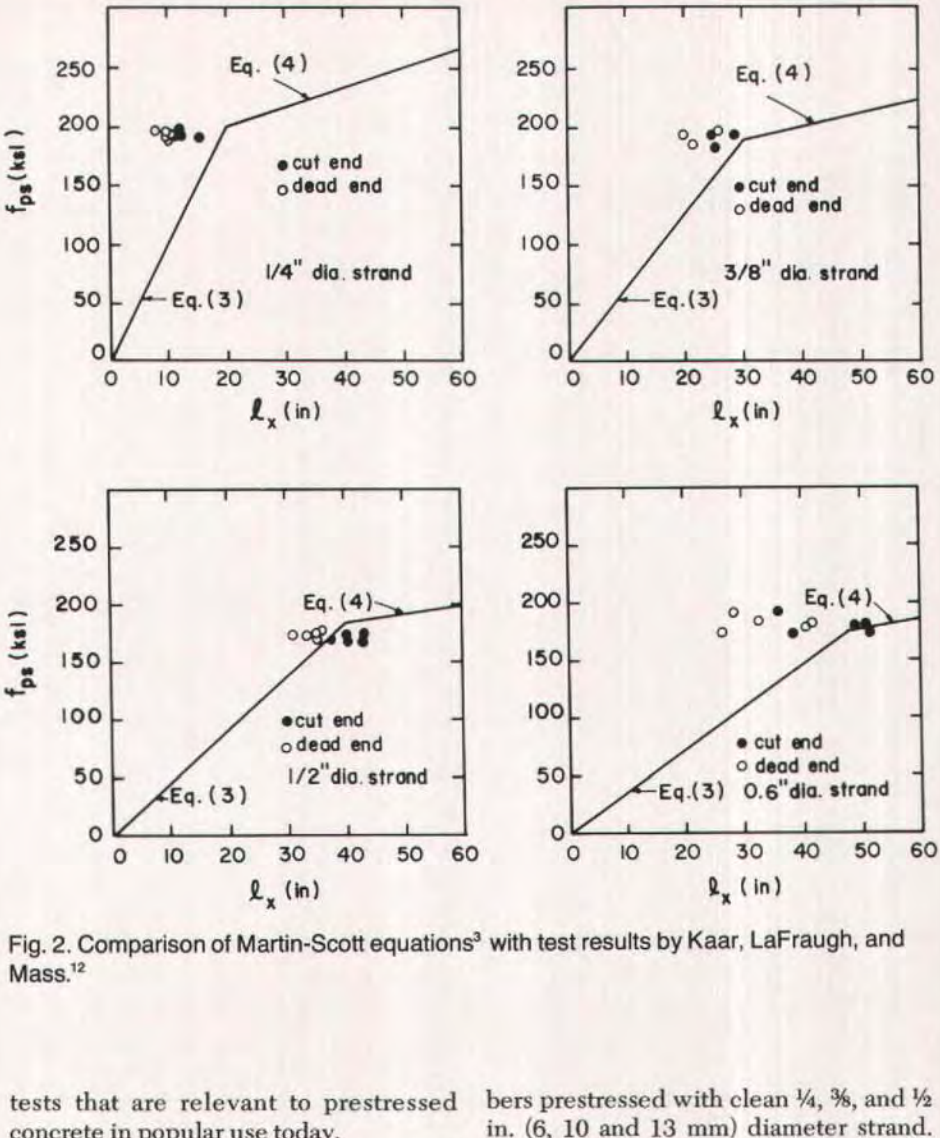


Fig. 2. Comparison of Martin-Scott equations[3] with test results by Kaar, LaFraugh, and Mass.[12]

tests that are relevant to prestressed concrete in popular use today.

Early investigations on the nature of bond were conducted in the 1950's.[6,10,11] These tests concluded that strand diameter, method of releasing the strand, and the physical condition of the strand were all factors influencing the transfer bond length and flexural bond length. As a result of these tests, primarily the ones reported in Ref. 6, ACI 318-63 adopted what is still the current expression for development strength. The Hanson-Kaar tests[6] were run on members prestressed with clean ¼, ⅜, and ½ in. (6, 10 and 13 mm) diameter strand. The test specimens had a wide range of steel percentages and the strands were released slowly, rather than cut by flame or saw.

In most of the specimens there was a significant increase in load carrying capacity between the point at which first bond slip was detected by strain gauges and final bond failure. The difference in load carrying capacity was due to mechanical interlock of the strand. The ACI Code equation approximates the

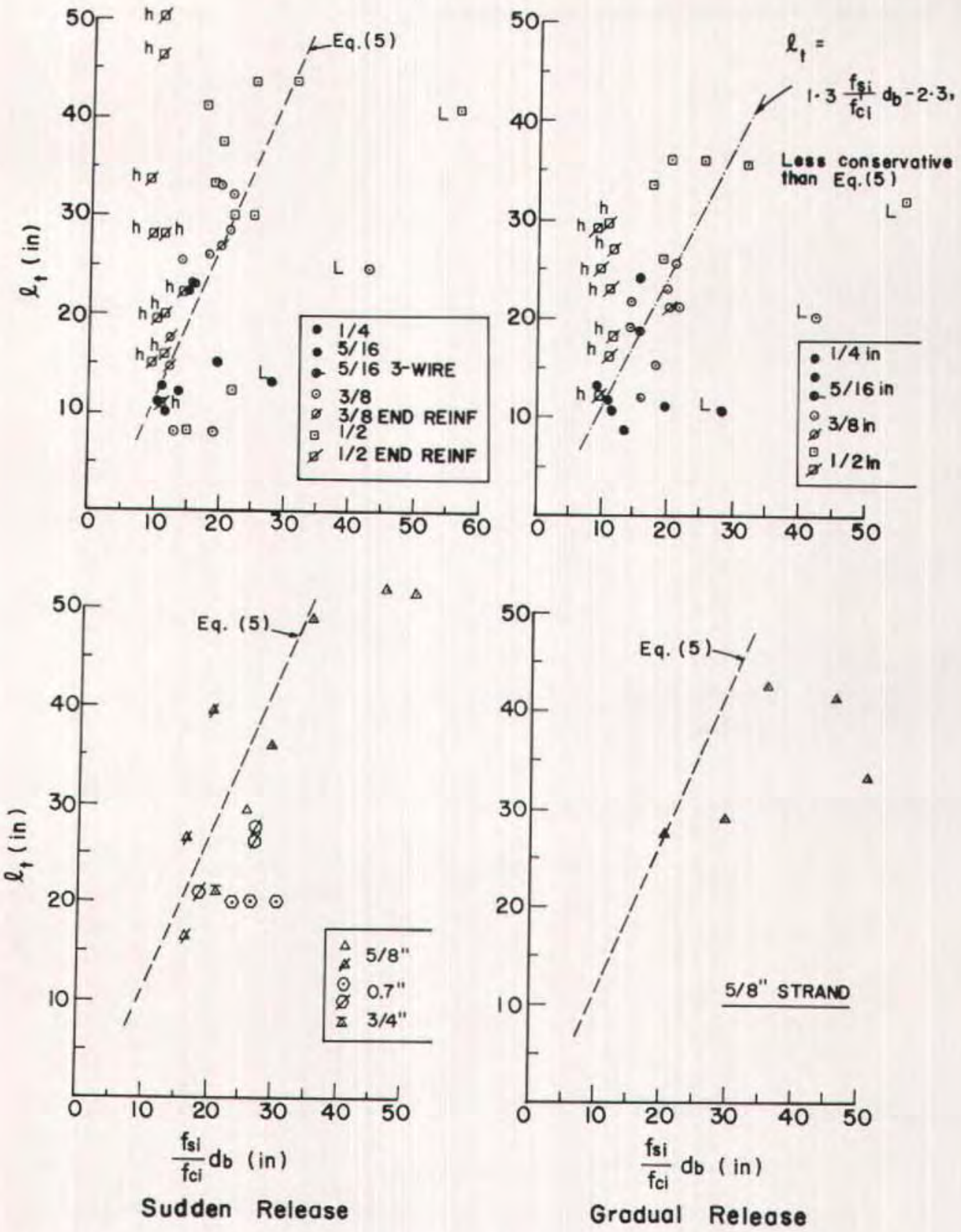


Fig. 3. Comparison of Zia-Mostafa equations[8] with experimental results from various sources (L indicates low strength concrete with $f_c' < 2000$ psi; *h* indicates high strength concrete with $f_c' > 8000$ psi).

average value of all the points representing first bond slip and final bond failure.[3]

Results of tests performed by Kaar, LaFraugh, and Mass[12] greatly added to the knowledge concerning transfer length. Tests were performed on members with varying strand diameters and concrete strengths. The results indicated that, although higher strength concrete could develop 75 to 80 percent of the transfer bond in a shorter distance than lower strength concrete, the total distance required to develop 100 percent of the transfer bond was approximately the same irrespective of concrete strength.

In recent years, several researchers have proposed new equations for transfer and development lengths. Martin and Scott,[3] in a statistical evaluation of the early tests performed by Hanson and Kaar,[6] proposed the following expressions (see Figs. 1 and 2):

For $l_x$ less than 80 $d_b$:

$$f_{ps} \leq \frac{l_x}{80\, d_b}\left(\frac{135}{d_b^{1/6}} + 31\right) \qquad (3)$$

where $l_x$ is the distance from the end of the member to the section under consideration, in inches.

For $l_x$ greater than 80 $d_b$:

$$f_{ps} \leq \frac{135}{d_b^{1/6}} + \frac{0.39\, l_x}{d_b} \qquad (4)$$

In no case shall $f_{ps}$ be greater than that given by Eq. (18-3) of ACI 318-83 or that obtained from a determination based on strain compatibility.

The above expressions provide an approach to designing precast, pretensioned units for spans too short to provide an embedment length that will develop the full strength of the strand, and thus allay some of the concerns raised in response to the survey mentioned earlier. However, here is Zia's and Mostafa's[8] evaluation of these expressions:

"Martin and Scott proposed a transfer length of 80 diameters for strands of all sizes, and a flexural bond length of 160, 187, and 200 diameters for the ¼, ⅜, and ½ in. (6, 10 and 13 mm) diameter strands, respectively. These values are considerably higher than those specified by the current ACI Code."

On the other hand, based on the results of a test program of 36 pretensioned hollow-core units, Anderson and Anderson[13] concluded that the current ACI Code requirement on the development length is adequate provided that the free end slip of the strand, upon transfer of prestress, does not exceed an empirical value which is roughly 0.2 times the strand diameter.

Zia and Mostafa,[8] in a comprehensive study of all past research, proposed the following expressions (see Fig. 3):

$$l_t = \frac{1.5 f_{si} d_b}{f'_{ci}} - 4.6 \qquad (5)$$

$$l_b = 1.25(f_{ps} - f_{se})\, d_b \qquad (6)$$

$$l_d = l_t + l_b \qquad (7)$$

where

$f_{si}$ = stress in prestressing steel at transfer, ksi

$f'_{ci}$ = compressive strength of concrete at time of initial prestress, ksi

$l_t$ = transfer length of prestressing strand, in.

$l_b$ = flexural bond length of prestressing strand, in.

Eq. (6) is based on the theoretically derived expression:

$$l_b = \frac{f_{ps} - f_{se}}{4\, u_{ave}}\, d_b \qquad (8)$$

where $u_{ave}$ is average bond stress within $l_b$. Note that in the current ACI Code, it is implied that $u_{ave}$ = 250 psi (1.7 MPa). Eq. (6) assumes an $u_{ave}$ = 200 psi (1.4 MPa).

The Zia-Mostafa equation for transfer length is applicable for concrete strength ranging from 2000 to 8000 psi (14 to 55 MPa). It accounts for effects of

Table 1. Comparison of Eq. (5) with ACI Code requirement for transfer length $l_t$ (in.).

| | 250-K Grade $f_{si} = 175$ ksi, $f_{se} = 140$ ksi | | | 270-K Grade $f_{si} = 189$ ksi, $f_{se} = 151$ ksi | | |
|---|---|---|---|---|---|---|
| Strand size, in. | $f'_{ci} = 3500$ psi | $f'_{ci} = 4000$ psi | ACI | $f'_{ci} = 3500$ psi | $f'_{ci} = 4000$ psi | ACI |
| 1/4 | 14 | 12 | 12 | 16 | 13 | 13 |
| 5/16 | 19 | 16 | 15 | 21 | 18 | 16 |
| 3/8 | 24 | 20 | 18 | 26 | 22 | 19 |
| 7/16 | 28 | 24 | 21 | 31 | 26 | 22 |
| 1/2 | 33 | 28 | 24 | 36 | 31 | 25 |

Note: 1 in. = 25.4 mm; 1 ksi = 6.9 MPa; 1 psi = 0.0069 MPa.

strand size, the initial prestress and the concrete strength at transfer. The equation for transfer length gives comparable results as the current ACI Code requirement for small sized strands, but is more conservative than the ACI Code, particularly for cases where the concrete strength at transfer is low (see Table 1). The flexural bond length specified by the current ACI Code (ACI 318-83) is increased by 25 percent by the Zia-Mostafa proposal.

## 6. EVALUATION OF THE ACI CODE DEVELOPMENT LENGTH PROVISION FOR DEBONDED STRAND

Kaar and Magura[4] have pointed out that the development length required by Eq. (1) was based on tests of beams with all strands bonded from the section of maximum moment to the beam ends. The end of the development length can then overlap the stress transfer length near the beam supports, where a state of flexural precompression exists even at high loads, and where a lateral compression is provided by the vertical beam support reaction. When strands are blanketed for a considerable distance into a member, however, both stress transfer and flexural bond development may take place in a concrete region subjected to tension, and even cracking, before the ultimate load is reached. Under these conditions, the embedment length given by Eq. (1) may be inadequate.

Tests reported in Ref. 14 were conducted to accurately determine prestress losses and creep camber in prestressed girders. The tests included a study of beams containing debonded strands, one with normal weight concrete and one with lightweight concrete. The study, which compared the beams containing debonded strands with beams containing draped bonded strands, concluded that the midspan prestress losses for a given concrete were about the same for both designs studied and that beams with debonded strands can be designed to have less initial and time-dependent camber than beams containing draped strands.

Later tests reported in Ref. 15 were performed on both half sized and full sized girders containing draped bonded strands, debonded "wrapped" strands, and debonded strands with end anchors. The beams were designed using one development length and the debonded strands were "wrapped" with a cage of mild steel reinforcement in the transfer region to confine the concrete immediately surrounding the strands, thereby eliminating the possible need for longer development lengths. All the test beams performed satisfactorily. However, fatigue was not a consideration in these tests. Also, later tests conducted at the Portland Cement Association[16] showed that wrapping has little, if any, benefit.

Tests by Kaar and Magura[4] explored the possible effects of blanketing or debonding on the flexural behavior at service load and on the ultimate flexural, bond, and shear strength of pretensioned prestressed girders.

Three girders were designed and tested for the study of flexural behavior. Girder 1 had no strands debonded. Girder 2, designated as "partially blanketed," had strands so debonded that the development lengths were twice those computed by Eq. (1). The "fully blanketed" Girder 3 was designed with development lengths of the blanketed strands equal to the lengths required by Eq. (1). All three girders were overreinforced with stirrups to prevent interference of shear distress with flexural and bond behavior.

The girders were subjected to 5 million cycles of the design live load prior to static testing to failure. Each girder was first loaded statically through five live load cycles. Thereafter, the girder was loaded dynamically, with static tests carried out after approximately 1, 2½, and 5 million cycles. At the completion of the 5 million cycles, the girder was removed and tested statically to destruction.

The shear investigation involved static testing to destruction of a nonblanketed girder, Girder 4, and a girder with partially blanketed strands, Girder 5. The girders were similar to those in the flexural study except that the number of stirrups was reduced in order that any effects on shear capacity of blanketing strands would be demonstrated.

The three tests utilizing dynamic loads showed no detrimental effects of strand blanketing on pretensioned members subjected to 5 million repetitions in the working load range.

Beyond the cracking load and under static loading, some bond slip occurred for all blanketed strand.

The results from the two tests in which the girders had less than the required shear reinforcement indicated no detrimental effects of blanketing upon shear strength.

There was evidence that the ACI Code requirement for bond development length of the strand cannot be directly applied to blanketed strand. However, the performance of blanketed strand girders with development lengths twice those required by Eq. (1) closely matched the flexural performance of a similar pretensioned girder entirely without blankets.

The provisions of Section 12.9.3 of ACI 318-83 are based on the Kaar-Magura tests,[4] modified as indicated below.

An experimental investigation, subsequent to the Kaar-Magura tests, has been carried out at the Portland Cement Association to determine the effect of repetitive loading on the behavior and strength of girders with blanketed strands.[16] Controlled variables in the test program were load level, development length, and use of ties to confine the concrete in the stress transfer region of the blanketed strands.

The test program called for 5 million cycles of loading between dead load and dead plus live load. Static tests to full dead load plus live load were performed before cyclic loading and after 1, 2½, and 5 million load cycles. At the completion of 5 million cycles, the girders were tested to destruction under static load.

The results of the fatigue tests indicated the following:

1. For similar loading conditions, the behavior and strength were the same for girders having either blanketed or draped strands.

2. The fatigue life of specimens de-

signed for a maximum tensile stress of $6\sqrt{f'_c}$ psi ($0.5\sqrt{f'_c}$ MPa) under full service load was significantly less than that of specimens designed for zero tension.

3. In specimens designed for zero tension in the concrete under service load condition, and having blanketed strands designed for one development length [as given by Eq. (1)], the behavior and strength of the specimens with blanketed strands were similar to those of girders with draped strands.

4. In the specimen designed for a maximum tensile stress of $6\sqrt{f'_c}$ psi in the uncracked concrete under full service load and having blanketed strands designed for twice the development length given by Eq. (1), only small slip of the strands occurred. This indicated adequate bond of the blanketed strands for about 3 million cycles of repetitive loading.

5. In the specimen designed for a maximum tensile stress of $6\sqrt{f'_c}$ psi* in the uncracked concrete under full service load and having blanketed strands designed for one development length, blanketed strands slipped, indicating occurrence of bond fatigue.

6. In the three specimens where cyclic loading produced tension of $6\sqrt{f'_c}$ psi in the concrete at midspan, fatigue fracture of the strands occurred at about 3 million cycles of repetitive loading. These specimens included a control girder with draped strands. Therefore, blanketing did not cause fatigue of strands.

7. Use of ties to confine the concrete in the stress transfer region of blanketed strands in one specimen did not provide any substantial improvement in the behavior of that specimen.

In view of the findings of the above test series, the original double development requirement for blanketed strands of the 1971 ACI Code was modified in the 1983 Code edition, allowing that in pretensioned members designed for zero tension in the concrete under full service load conditions, the development length for debonded strands need not be doubled.

For further details on the effect of blanketing strand, see Refs. 4, 5 and 16.

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*The following comment by Dr. Alex Aswad of Stanley Structures in private correspondence with the authors may be of interest:

"I reviewed the PCA report in 1978 before its publication . . . Rabbat et al did not use AASHTO losses; instead, they assumed a smaller 'flat value.' If AASHTO losses were used, the calculated bottom tension would have been $8.9\sqrt{f'_c}$ psi ($0.74\sqrt{f'_c}$ MPa)."

In the above mentioned review Dr. Aswad also expressed the opinion that the three girders designed for zero tension in the concrete under service load conditions actually had bottom tension approximately equal to $3\sqrt{f'_c}$ psi ($0.25\sqrt{f'_c}$ MPa). It should additionally be noted that in the Kaar-Magura tests,[4] a bottom tension of $2.4\sqrt{f'_c}$ psi ($0.20\sqrt{f'_c}$ MPa) under the full design service load had been allowed.

# 7. PERMISSIBLE CONCRETE STRESSES IN PRESTRESSED CONCRETE FLEXURAL MEMBERS

In this section the current ACI Code provisions and their significance are discussed together with some results from an industry survey. Lastly, the background and evaluation of the Code provisions on allowable concrete stresses are brought into focus.

## ACI Code Provisions

The current ACI Code provisions[2] concerning permissible concrete stresses in prestressed concrete flexural members are given below:

**Section 18.4.1** — Stresses in concrete immediately after prestress transfer (before time-dependent prestress losses) shall not exceed the following:*

(a) Extreme fiber stress in compression .......... $0.60 f'_{ci}$

(b) Extreme fiber stress in tension except as permitted in (c) ......... $3\sqrt{f'_{ci}}$

(c) Extreme fiber stress in tension at ends of simply supported members ..... $6\sqrt{f'_{ci}}$

Where computed tensile stresses exceed these values, bonded auxiliary reinforcement (nonprestressed or prestressed) shall be provided in the tensile zone to resist the total tensile force in concrete computed with the assumption of an uncracked section.

**Section 18.4.2** — Stresses in concrete at service loads (after allowance for all prestress losses) shall not exceed the following:

(a) Extreme fiber stress in compression .......... $0.45 f'_c$

(b) Extreme fiber stress in tension in precompressed tensile zone .................. $6\sqrt{f'_c}$

(c) Extreme fiber stress in tension in precompressed tensile zone of members (except two-way slab systems), where analysis based on transformed cracked sections and on bilinear moment-deflection relationships show that immediate and long-time deflections comply with requirements of Section 9.5.4, and where cover requirements comply with Section 7.7.3.2 ..... $12\sqrt{f'_c}$

**Section 18.4.3** — Permissible stresses in concrete of Sections 18.4.1 and 18.4.2 may be exceeded if shown by test or analysis that performance will not be impaired.

*Metric (SI) conversion factors:
1.0 psi = 0.006895 MPa; 1.0 $\sqrt{f'_c}$ = 0.083 $\sqrt{f'_c}$ MPa.

## Significance of the Code Provisions

The tension stress limits of $3\sqrt{f'_{ci}}$ and $6\sqrt{f'_{ci}}$ refer to tensile stress at locations other than the precompressed tensile zone (that portion of the member cross section in which flexural tension occurs under dead and live loads). If the tensile stress exceeds the applicable limiting value, the total force in the tension zone should be calculated and bonded auxiliary reinforcement provided to resist this force. For design purposes, such steel is assumed to act at a stress of 60 percent of its yield stress, but not at a stress greater than 30 ksi.

The service load stress limits apply after all losses have occurred and when the full service load acts. The allowable concrete tensile stress of $6\sqrt{f'_c}$ has been established mostly on the basis of experience with test members and actual structures. Use of this stress limit, rather than a lower value or zero, requires that there be a sufficient amount of bonded reinforcement in the precompressed tension zone to control cracking, that the amount of concrete cover over the reinforcement be sufficient to avoid corrosion, and that unusually corrosive conditions not be encountered.

Bonded reinforcement may consist of bonded prestressed or nonprestressed tendons, or of bonded reinforcing bars, well distributed over the tension zone. In all flexural members where the prestressing tendons are not bonded, the minimum bonded reinforcement requirements of Section 18.9 must be followed. Unbonded construction, which is invariably post-tensioned, is beyond the scope of the broader investigation of which this study forms a part. Whether, in bonded construction, the allowable stress limits of $6\sqrt{f'_c}$ (or $12\sqrt{f'_c}$), $0.45 f'_c$ and/or $0.60\ f'_{ci}$ can be exceeded, and auxiliary reinforcement used to carry the entire tension and/or the excess compression force, is not clear from the Code.

The Code neither specifically allows, nor expressly forbids, such practice. The authors are not sure as to whether such practice is amenable to mechanized plant precasting. The added cost of production may outweigh any potential advantages that might exist. Also, the anticipated service load behavior of (partially prestressed) members that may be designed following the above practice must be preascertained, based on such experimental information and performance record as may be available.

The allowable concrete stresses of the ACI Code are also significant in that they lead directly to the practice of debonding of tendons. Two methods are in popular use for limitation of compressive and tensile concrete stresses near the ends of pretensioned prestressed concrete members. Some of the pretensioned reinforcement may be "harped," that is, deflected upward near the ends of the members; or bond to the concrete may be prevented for some of the pretensioned reinforcement in the end regions.

Harping or draping of strands in pretensioned beams can present problems for designers, fabricators, and inspectors in some plants. The tensioning procedure is time consuming, expensive and leaves doubt as to the actual prestress level obtained throughout the length of the strand. Relevant aspects of the practice of debonding have already been discussed at length.

## Some Results From Industry Survey[9]

In response to the question, "Do the auxiliary reinforcement provisions of Section 18.4.1 cause any hardship?", 8 respondents answered yes, 39 answered no. Comments from those with affirmative answers were as follows:

1. How far from the end does Section 18.4.1(c) stop and Section 18.4.1(b) start applying? We use Section 18.4.1(c) full length with no problems.

2. It may be more appropriate to provide tension reinforcement based on a force-distance from centroid relationship instead of force only.

3. Taking the "total tensile force with reinforcement" per Section 18.4.1 may be too conservative.

4. Allow an increase in top fiber tension at ends of simply supported members.

Comments from those with negative answers included the following:

1. $3\sqrt{f'_{ci}}$ should be changed to $4\sqrt{f'_{ci}}$ at release.

2. Investigate whether a higher allowable stress can be achieved.

3. Section 18.4.1 should include the following item: allowable tension under dead load only — zero.

4. Section 18.4.1(b) — increase to $5\sqrt{f'_{ci}}$. Justification:

$$\frac{f_r}{1.5} = \frac{7.5}{1.5}\sqrt{f'_{ci}} = 5\sqrt{f'_{ci}}$$

5. Cracking can and will result if auxiliary reinforcement is not used when needed.

6. The auxiliary reinforcement requirement is not a problem for double tees.

In response to the question, "Are there any other ACI Code requirements [other than those specifically cited in the survey] relative to double tees causing difficulties?" the following answers of interest to the current discussion were received:

1. Section 18.4.1(a) — We frequently allow initial compression stresses greater than $0.60f'_{ci}$. Often I feel wrapping of strands is more detrimental than is the higher compression stress, especially in a thin stemmed member. I see no problem in allowing an initial stress equal to $0.70f'_{ci}$. Losses are not appreciably affected.

2. Section 18.4.1(a) and 18.4.2(a) — The ACI Code should stipulate addition of reinforcing steel if allowable com-

pression stress in the concrete is exceeded. It does so in the case of tension [Section 18.4.1(c)]. This eliminates the necessity to drape strands. Sometimes we use this practice because the Code does not forbid it. However, some designers do not agree.

3. Release stresses of Section 18.4.1 for compression are too conservative. A flat factor of safety of 1000 psi or 1200 psi (6.9 or 8.3 MPa) is preferable.

4. An allowable compressive stress of $0.60 f'_{ci}$ is too restrictive; suggest using $0.70 f'_{ci}$.

5. The effects of a compressive stress at release greater than $0.60 f'_{ci}$ should be investigated. What, for example, would happen if the stress were $0.70 f'_{ci}$?

6. We don't believe that a maximum tensile stress is needed. We often design for camber and let stress go over 1000 psi (6.9 MPa).

7. Lack of acceptance of $12\sqrt{f'_c}$ allowable stress by conservative consulting engineers.

## Background and Evaluation of Code Provisions

A chapter on prestressed concrete was included for the first time in the 1963 edition of the ACI Code. The chapter was based on recommendations by ACI-ASCE Committee 423 on prestressed concrete.[17] The chapter included the following allowable stresses in concrete.

(a) Temporary stresses immediately after transfer, before losses due to creep and shrinkage, shall not exceed the following:

1. Compression ......... $0.60 f'_{ci}$
2. Tension stresses in members without auxiliary reinforcement (unprestressed or prestressed) in the tension zone .......... $3\sqrt{f'_{ci}}$
   Where the calculated tension stress exceeds this value, reinforcement shall be provided to resist the total tension force in the concrete computed on the assumption of an uncracked section.

(b) Stresses at design loads, after allowance for all prestress losses, shall not exceed the following:

1. Compression ......... $0.45 f'_c$
2. Tension in the precompressed tension zone: Members not exposed to freezing temperatures nor to a corrosive environment, which contain bonded prestressed or unprestressed reinforcement located so as to control cracking ...................... $6\sqrt{f'_c}$
   All other members .... 0
   These values may be exceeded when not detrimental to proper structural behavior as provided in Section . . .

In his discussion of Ref. 17, T. Y. Lin wrote:[18]

"As an example of the dangerous errors contained in these allowable stresses, let us consider the temporary stresses allowed. . . . Here tension in the concrete is limited to $3\sqrt{f'_{ci}}$ for single elements. . . . A recently completed investigation at the University of California proved definitely that the strength and behavior of beams at transfer cannot be simply described by stresses but are dependent upon a number of factors, such as the shape of the section, the amount and location of prestress, etc."

In its closure to the discussion of Ref. 17, Committee 423 wrote:[19]

"The specific (allowable) stress values . . . were chosen after a thorough study of all pertinent data. During the last year before publication of the report, numerous comments were received and some modifications made in the allowable stresses. It is a fact that the values published reflect the very best on which agreement could be obtained."

"In reference to some of Lin's remarks about allowable stresses, it should be reaffirmed that the (proposed) provisions ... are intended to be advisory rather than intransigent. Special circumstances may dictate a downward revision of certain values. Liberalization may be indicated in other instances where sufficient supporting data can be submitted, including analytical studies, test results, or performance records. In the latter case, the burden of proof should fall upon those who wish to deviate from the generally accepted values."

Specifically on the limit $0.60 f'_{ci}$, as applicable to pretensioned members, the Committee wrote:

"Here, production had preceded design recommendations, and the stress of $0.60 f'_{ci}$ had already been widely established in the pretensioning industry. No ill effect had been reported in regard to strength and performance. Only camber proved difficult to control for certain building members."

The authors would also like to record here their belief that the choice of $0.60 f'_{ci}$ must have been dictated originally by a desire not to go too far into the inelastic range of stresses. The elastic limit is usually at a stress of about half the compressive strength. The stress 0.6 $f'_{ci}$ is beyond, but not too far beyond, that limit.

On the $6\sqrt{f'_c}$ limit on tension stresses at full service load, Committee 423 wrote: "This is another instance in which the pretensioning industry for many years had followed a standard of production that had given satisfactory results."

Section 18.4.2 of ACI 318-83, in its present form, first became part of the 1971 edition of the ACI Code. The use of a tensile stress limit of $12\sqrt{f'_c}$ was permitted to obtain improved service load deflection characteristics, particularly when a substantial part of the live load is of a transient nature. It should be emphasized that an allowable tensile stress of $12\sqrt{f'_c}$, calculated on the basis of an uncracked cross section, is a nominal stress only, since its value is well above any reasonable estimate of the modulus of rupture of the concrete. If this stress limit is used, the concrete protection for the reinforcement must be increased 50 percent above its usual value, according to the Code, and an explicit check made of service load deflections.

The $6\sqrt{f'_{ci}}$ limit on initial tension stresses, applicable to the ends of simply supported members only, was introduced into the 1977 edition of the ACI Code in an effort to mitigate hardships faced by the hollow-core industry. The industry had difficulty in satisfying the $3\sqrt{f'_{ci}}$ stress limit at the ends of hollow-core planks. Draping of strands is not a practical solution for such shallow members. Debonding of strands is not an economically viable solution either. The use of auxiliary reinforcement is not practical in view of the extrusion process of manufacture. The industry also produced evidence that a relaxation of the $3\sqrt{f'_{ci}}$ stress limit only for the ends of simply supported members would not adversely affect performance.

Whether the $3\sqrt{f'_{ci}}$ or the $6\sqrt{f'_{ci}}$ limit applies should be fairly obvious in most design situations. The sole exception to that is where debonded strands are used at the ends of simply supported members. In checking the stresses where the debonding ends, a strict reading of the Code would appear to indicate use of the $3\sqrt{f'_{ci}}$ limit. If the debonding is over a significant length, it should not be difficult to satisfy this stricter stress limit, because the dead load moments would relieve some of the extreme tension fiber stresses. If the debonding is over a very short length, so that benefit from the dead load stresses does not obtain, use of the $6\sqrt{f'_{ci}}$ stress limit would appear to be justified.

The allowable tensile stresses of $3\sqrt{f'_{ci}}$ and $6\sqrt{f'_{ci}}$ are obviously related to the modulus of rupture which, accord-

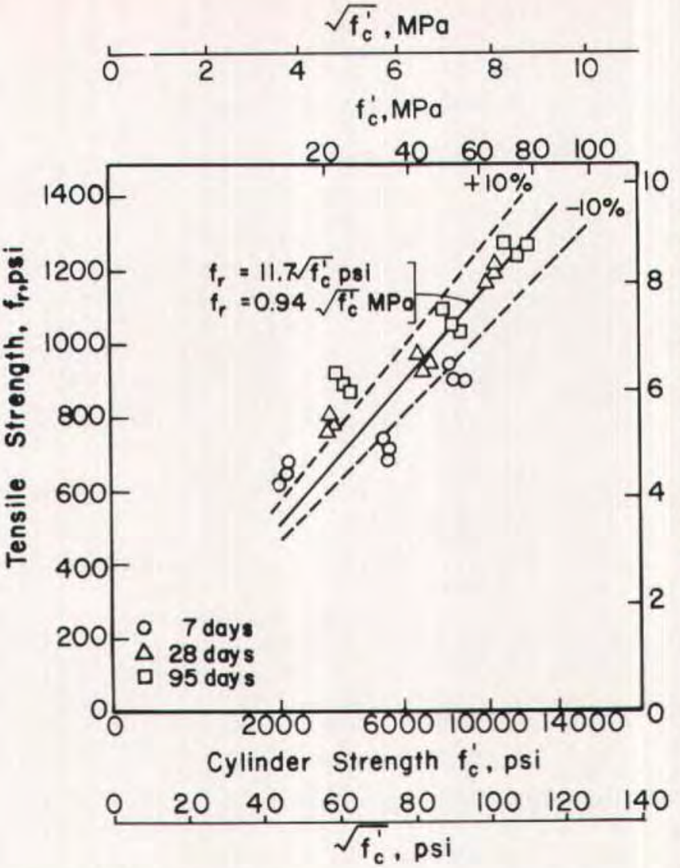


Fig. 4. Modulus of rupture as a function of concrete strength.[21]

ing to the Code, for normal weight concrete is [Eq. (9-9), Section 9.5.2.3]:

$$f_r = 7.5\sqrt{f_c'}$$

According to the recently published "State-of-the-Art Report on High-Strength Concrete" by ACI Committee 363,[20] the values reported by various investigators for the modulus of rupture of both lightweight and normal weight high strength concretes fall in the range of $7.5\sqrt{f_c'}$ to $12\sqrt{f_c'}$ where both the modulus of rupture and the compressive strength are expressed in psi. The following equation was recommended[21] for the prediction of the tensile strength of normal weight concrete, as measured by the modulus of rupture (Fig. 4):

$$f_r = 11.7\sqrt{f_c'}$$
$$\text{for } 3000 \text{ psi} < f_c' < 12{,}000 \text{ psi}$$

The reader should consult Ref. 22 for further valuable background information on the ACI Code provisions concerning allowable stresses in prestressed concrete flexural members.

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## APPENDIX — NOTATION

$d_b$ = nominal diameter of strand, in.
$f'_c$ = specified compressive strength of concrete, psi
$f'_{ci}$ = compressive strength of concrete at time of initial prestress, psi
$f_{ps}$ = stress in prestressed reinforcement at nominal strength, ksi
$f_{pu}$ = specified tensile strength of prestressing strand, ksi
$f_r$ = modulus of rupture of concrete, psi
$f_{se}$ = effective stress in prestressed reinforcement (after allowance for all losses), ksi
$f_{si}$ = initial stress in prestressed reinforcement, ksi
$l_d$ = development length of prestressing strand, in.
$l_x$ = distance from end of member to section under consideration, in.
$l_b$ = flexural bond length of prestressing strand, in.
$l_t$ = transfer length of prestressing strand, in.
$u_{ave}$ = average bond stress of prestressing strand within flexural bond length, in.

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**NOTE:** Discussion of this paper is invited. Please submit your comments to PCI Headquarters by May 1, 1987.