Strength Design of Pretensioned Flexural Concrete Members at Prestress Transfer

This paper presents a rational method for the design of pretensioned flexural concrete members due to the effects of prestress transfer. Conditions at prestress transfer often control the level of prestress that can be placed in pretensioned flexural members. The level of prestress significantly influences the maximum span capability. It is proposed that the flexural design of pretensioned, prestressed concrete members for the effects of prestress transfer be based on strength design criteria. In practice, the proposed method will generally lead to higher prestress levels than the empirical limit of 0.6 $f'_{ct}$ given in ACI 318-99 Building Code and AASHTO Standard Specifications for Highway Bridges. Satisfying the current 0.6 $f'_{ct}$ limit often results in excessive strand debonding or necessitates draping strand at member ends. Debonding results in weakening of shear strength and poses possible durability concerns. Another significant advantage of the proposed strength design approach is that it automatically and rationally allows for calculation of any top bonded reinforcement required to maintain strength at transfer with controlled tension cracking. The current method directs the design engineer to use an uncracked section analysis of an already cracked section to calculate the bonded reinforcement area.
The main reason for setting permissible stress limits due to unfactored loads in prestressed concrete flexural members is to ensure that serviceability criteria are met. These criteria include deflection, camber control, crack control, and fatigue, if applicable. In addition, the allowable extreme fiber compressive stress at transfer appears to be an indirect way of checking that concrete will not "crush" due to prestress transfer.

Extreme fiber tension stress limits at transfer and at service are a means to check whether cracking takes place and if any measures are needed to control crack width. However, to be consistent with current design practice, the compression stress limit should be based on a strength design approach, while the tension limit is related to serviceability at unfactored load levels.

This paper presents the results of a theoretical and experimental research program for conditions due to transfer of prestress in precast, pretensioned flexural members. The main objectives of this study are to:

1. Establish a simplified strength design method.
2. Recommend appropriate load factors.
3. Determine the correlation, if any, between the new strength design and working stress design limits given in current codes.
4. Investigate various serviceability effects.
5. Develop relevant design criteria.

A further objective of this research project is to develop a semi-empirical formula for compressive stress limits as a more justifiable substitute for the current 0.6 $f'_{ci}$ limit given in ACI 318-99 Building Code and AASHTO Standard Specifications for Highway Bridges. It is anticipated that a more appropriate allowable compressive stress formula will be used for a transitional period until design engineers and computer software developers become familiar with the strength design procedure.

To overcome excessive compressive stresses at member ends, design engineers sometimes resort to debonding the strands near member ends. However, such debonding is believed to contribute to bond slippage, reduction of shear resistance, and corrosion of the strands because of moisture penetration along the “sleeved” or debonded length of strand at the member end. This practice could significantly reduce the level of anchor-age of the strands at the beam bearing when the beam's end zone acts as a tied arch as the applied load approaches the ultimate capacity of the beam.

Strands are depressed in double tees and other products to control member end stresses. However, for double tees, the stems are narrow and the strands are often pushed down from the top after stressing, rather than being held down during stressing as is common practice in bridge beams.

This operation is not as safe as keeping all the strands straight. The device used to push strands down from the top of the member is often pulled out after prestress transfer. The vertical hole created in this operation is then filled with grout. This location becomes vulnerable to moisture and salt penetration which can cause premature corrosion of the strands.

Today, many design engineers tend to believe that allowable compressive stress limits in the code are intended to guard against concrete crushing. However, it is more rational to use an ultimate strength calculation to protect against concrete crushing.3,4

The description of the strength design procedure is given using either commercially available column design software, such as PCACOL. Alternatively, a designer-developed spreadsheet may be used for the strength design calculations described in this paper.

The prestress force just before transfer, the self-weight moment and the section geometry are input. The output is the required concrete compression strength and area of bonded reinforcement. It is shown that in this design procedure, a design engineer can select a suitable $f'_{ci}$ and solve for the corresponding area of top tension reinforcement.

To test this approach, an experimental program was undertaken. Inverted tee members were pretensioned to a level as high as 0.84 $f'_{ci}$. Their performance was observed for several months before additional dead loads were applied. Prestress loss and camber growth with time were found to be predictable with available calculation methods.

A parametric study was conducted to determine an equivalent allowable compressive stress limit for sections designed using the proposed strength design method. An accurate relationship could not be found. However, it has been observed that the equivalent allowable compression varies from 0.6 $f'_{ci}$, the current code limit, to 0.75 $f'_{ci}$ depending on section shape and the effect of prestress relative to self-weight moments.

**HISTORICAL BACKGROUND**

The Prestressed Concrete Institute (PCI) issued the first specification5 for bonded pretensioned, prestressed concrete on October 7, 1954. In this specification, the maximum allowable compression at the time of transfer of prestressing was 0.5 $f'_{ci}$ for bridge members and 0.55 $f'_{ci}$ for building members. No explanation was given on the justification of these limits or why they were related to $f'_{ci}$, the specified compressive strength of concrete, rather than $f'_{ci}$, the compressive strength of concrete at the time of initial prestress.

Following publication of the first PCI specification in the 1957 Proceedings of the World Conference on Prestressed Concrete, there was a discussion on the "Criteria for Prestressed Concrete Bridges," by E. L. Erickson,7 who at the time was chief of the Bridge Division, Bureau of Public Roads (BPR). Erickson’s paper gave the reasoning that led to the adoption of the allowable compression 0.60 $f'_{ci}$ limit as the following:

"It should be noted that the allowable concrete stresses are based on the strength at the time of prestressing and not the 6 x 12 in. cylinder strength at 28 days.

There seems to be considerable difference of opinion among authorities as to the maximum allowable stresses which may be imposed temporarily, i.e., before creep and shrinkage takes place. For the maximum temporary..."
The issue of allowable compressive stress for prestress transfer has become a restrictive factor in the modern design of pretensioned flexural members. Much of the cost-effectiveness of precast concrete members results from a daily production cycle of a long-line pretensioning bed.

The requirement for artificially high concrete strengths at transfer may hinder this cycle. When overnight strengths are not achieved, precast concrete producers must delay, or cancel, production in that pretensioning bed for that day.

As a result, some precast producers have exceeded the allowable compressive stress under dead load alone is always at the ends of the beam where it is comparatively harmless, whereas in the post-tensioned beam with draped tensioning elements, it is more nearly uniform.

The value of 0.55 $f'_{ct}$ adopted in the criteria for post-tensioned construction is slightly more conservative than Holley’s and Simpson’s figures and would give a little more allowance for sustained load, variation in concrete strength, and eccentricity in the posttensioning procedure as suggested by Siess.

For pretensioned construction, a slightly higher value (0.60 $f'_{ct}$) was adopted. Mr. Dean, Bridge Engineer of the Florida Highway Department, points out that the high compression in pretensioned construction only occurs at the ends. Furthermore, one would expect better control in the casting yard where pretensioning is done than in the case of post-tensioning which is usually done as a field operation.

The earliest versions of the PCI Building Code (1961) upgraded the release stress limit to 0.6 $f'_{ct}$. This limit was kept unchanged by the ACI-ASCE Committee 323 (now ACI-ASCE 423, Prestressed Concrete), and the 1963 ACI 318 Building Code.

The requirement for artificially high concrete strengths at transfer may hinder this cycle. When overnight strengths are not achieved, precast concrete producers must delay, or cancel, production in that pretensioning bed for that day.

As a result, some precast producers have exceeded the allowable compressive stress limit of 0.6 $f'_{ct}$ without any apparent detrimental effects. Also, a PCI Standard Design Practice Report, prepared jointly by the PCI Technical Activities Council and the PCI Committee on Building Code, indicates that no problems have been reported by allowing compression to go as high as 0.75 $f'_{ct}$.

**PROPOSED STRENGTH DESIGN FOR PRESTRESS TRANSFER**

The strength design for prestress transfer can be approached in a manner similar to that for non-prestressed reinforced concrete. The member can be treated as a “reinforced concrete column” subjected to moment combined with axial compression equal to the force in the prestressing steel just before prestress transfer.

The incremental “compression” in the “recompression” at the compression face of the section is equivalent to the strain and stress reduction in the prestressing steel. This reduction is generally known in current design, using unfactored loads, as elastic prestress loss. Thus, the prestressing steel and the concrete share in resisting the effects of the applied moment and axial compression.

A load factor of 1.2 is applied to the prestress force in prestressing steel just before prestress transfer, $P_t$. This is the same load factor used in the design of post-tensioned member anchorage zones (see ACI 318-99, Section 9.2.8).

A load factor of 0.8 or 1.2 is applied to the self-weight moment, $M_g$, due to the uncertainty of lifting locations. When the members are handled at pick-up points located somewhat inward from their theoretical support points, the moment due to self-weight should be accounted for. The load factor of 1.2 is used when the member self-weight effect is opposite to the prestress force moment, which is the common practice. However, if the member weight moment is in the same direction as the prestress force moment, the factor should be 0.8.

This situation is encountered when the lifting points are beyond the member ends, using for example structural steel ledges or strong backs. A strength reduction, $\phi$, of 0.7, is applied to the nominal axial capacity, $P_a$ and bending moment capacity, $M$. This is the same value used for tied columns.

At this time, it may be too conservative for this application as concrete strength is known because cylinders are tested just before prestress is transferred. Thus, the “applied” loads are temporary and the compressive prestressing force is internally induced, having a self-relieving ability. Until further results are available, $\phi = 0.7$ should provide a conservative solution.

It is interesting to note that a very similar approach has already been introduced in Australian practice, according to Reference 10 which refers to the Australian Code, AS 3600. In the Australian Code, the corresponding load factors are 1.15 and 0.8, and the corresponding strength reduction factor is 0.6. The prestress force used there is the jacking force rather than the force just before transfer as recommended here.

This appears to be a minor difference, and it is fortunate that the Australian practice was brought to the authors’ attention just before publication of this paper. It is unclear, however, whether the Australian practice advocates, as the proposed method given herein, total elimination of cracked section service load analysis, even if the member top fiber stresses exceed the modulus of rupture.

In calculating the strength of the cross section subjected to axial compression and bending, equilibrium and strain compatibility conditions are satisfied. The following commonly used
Assumptions are made:
1. Plane sections normal to the axis of bending remain plane after bending.
2. Concrete has no tensile strength.
3. The ACI 318 equivalent rectangular compressive stress block is used.
4. The ultimate concrete compressive strain is equal to 0.003.

Fig. 1 shows the proposed strength design diagram for prestress transfer at the member ends. The standard assumptions are used. A rectangular compressive stress block with constant intensity of 0.85 \( f' \) and depth \( a = \beta_1 c \) is assumed, where \( c \) is the neutral axis depth, and \( \beta_1 \) varies from 0.85 for \( f' = 4000 \) psi (27.6 MPa) to 0.65 for \( f' \geq 8000 \) psi (55.2 MPa).

The strain compatibility conditions provide for the following relationships:

\[
\varepsilon' = \left( \frac{c - d'}{c} \right) 0.003 \tag{1}
\]
\[
\varepsilon_s = \left( \frac{d - c}{c} \right) 0.003 \tag{2}
\]

where
- \( d \) = effective depth of section
- \( d' \) = distance from extreme compression fiber to centroid of “compression reinforcement,” i.e., prestressing steel
- \( \varepsilon_s \) = tension reinforcement strain
- \( \varepsilon'_s \) = “compression reinforcement” strain

Once the values of \( c, \varepsilon' \) and \( \varepsilon_s \) are known, the stresses in each layer of the reinforcing steel can be computed, \( f'_s = \varepsilon'_s E_s \) and \( f_s = \varepsilon_s E_s \). The equilibrium conditions require that:

\[
0.85 f'_{ci} ba + A'_s f'_s - A_s f_s = \frac{1.2 P_1}{\phi} \tag{3}
\]
\[
0.85 f'_{ci} ba \left( \frac{a}{2} - d' \right) - A_s f_s (d - d') = \frac{0.8 M_g}{\phi} \tag{4}
\]

where
- \( a \) = depth of equivalent rectangular stress block
- \( A_s = \) area of tension reinforcement
- \( A'_s = \) area of “compression reinforcement”
- \( b \) = width of compression face of member
- \( f'_{ci} = \) compressive strength of concrete at time of initial prestress
- \( f_s = \) calculated stress in tension reinforcement
- \( f'_s = \) calculated stress in “compression reinforcement”
- \( M_g = \) moment due to self weight
- \( P_1 = \) prestress force immediately after prestress release
- \( \phi = \) strength reduction factor

The design engineer can theoretically find an unlimited number of acceptable solutions. Therefore, one of the five unknowns must be assumed. For example, if no top tension reinforcement is desirable, one can set \( A'_s = 0 \), and obtain the corresponding \( f'_s \) which will be relatively high. If one wants to have a relatively low release strength, the design engineer can set the stress at that desired strength and obtain a relatively large required bonded tension reinforcement.

**Comparison with Working Stress Design Results**

An example of a rectangular section, 16RB40, from the PCI Design Handbook is used for the comparison. The beam span is 30 ft (9.14 m). Twenty-two Grade 270 ksi (1860 MPa) low-relaxation, 0.5 in. (12.7 mm) diameter strands with an eccen-

<table>
<thead>
<tr>
<th>Load</th>
<th>Transfer section at release</th>
<th>Midspan at release</th>
</tr>
</thead>
<tbody>
<tr>
<td>( P/A )</td>
<td>959 (6.6)</td>
<td>959 (6.6)</td>
</tr>
<tr>
<td>( Pe/S )</td>
<td>-1869 (-12.9)</td>
<td>1869 (12.9)</td>
</tr>
<tr>
<td>( M_s/S )</td>
<td>55 (0.4)</td>
<td>-55 (-0.4)</td>
</tr>
<tr>
<td>Total stresses</td>
<td>-855 (-5.9)</td>
<td>2773 (19.1)</td>
</tr>
</tbody>
</table>

**Notes:**
1. Positive sign is compression and negative sign is tension.
2. \( P/A \) and \( Pe/S \) are the stresses due to prestressing.
3. \( M_s/S \) is the stress due to self-weight of precast rectangular beam.
4. 1 ksi = 6.9 MPa.
tricity of 13 in. (330 mm) are used in the member.

It is assumed that the jacking stress just before prestress transfer is equal to 0.75(270) = 202.5 ksi (1395 MPa) and the initial prestress loss for working stress design is equal to 20.25 ksi (139.6 MPa) at transfer. The required concrete release strength based on working stress design, using both ACT 318-99 and the PCI Design Handbook, will be compared to that determined by the proposed strength design method.

The allowable compression at transfer is limited to 0.6f in ACT 318-99 and to 0.7f in the PCI Design Handbook. The allowable tension without added reinforcement is limited to 6'J (0.5') in both ACT 318-99 and the PCI Design Handbook. The working stresses at a section located a distance equal to the transfer length, 25 in. (635 mm) from the member end, and at midspan at release are summarized in Table 1.

The total compressive stress of 2773 psi (19.1 MPa) at the extreme bottom fibers, f_b, at the transfer section controls the working stress design. A concrete strength of 4622 psi (31.9 MPa) would be required according to ACI 318-99, and 3961 psi (27.3 MPa) would be required according to the PCI Design Handbook. Because the tensile stress of 855 psi (5.9 MPa) at the top fiber, f_t, at transfer exceeds the limit of 6√f' (0.5√f' ), it is assumed that the member is cracked, and requires top bonded reinforcement.

Top bonded reinforcement is designed to carry the tensile force determined using an uncracked section analysis at a stress of 0.6 f_t, but not more than 30 ksi (207 MPa). An area of 2.15 sq in. (1389 mm²) of Grade 60 (414 MPa), reinforcing steel would be required using both ACI 318-99 and the PCI Design Handbook.

The same example beam is worked out using the proposed strength design method. A spreadsheet program was developed. In this example, the program was used to calculate the required concrete strength for an assumed area of the top bonded reinforcing steel, A_s. The area, A_n, of the top reinforcement was varied from 1.29 to 5.78 sq in. (830 to 3726 mm²).

The results show that a concrete strength of 3811 psi (26.3 MPa) is required by the strength design method, when the same amount of reinforcement as that required in the working stress design method, 2.15 sq in. (1389 mm²), is used. The required concrete strength of 3811 psi (26.3 MPa) would correspond to an equivalent compressive stress limit of 0.73 f' by the working stress design method, as shown in Fig. 2.

One of the major advantages of the strength design approach is that it leads to a rational method for sizing top bonded reinforcement, as opposed to the arbitrary working stress design method now used. Fig. 2 demonstrates that the demand for concrete strength at transfer can be reduced by the design through an increase in the amount of top bonded reinforcement.

Using strain compatibility, the stress in the top reinforcement at nominal strength level can be determined. In this example, the calculated steel stress ranges from 30 ksi (207 MPa), when A_s is 5.78 sq in. (3726 mm²), to 60 ksi (414 MPa) when A_s is 1.29 sq in. (830 mm²), as shown in Fig. 2.

Obviously, the grade of top bonded reinforcement must meet the calculated stress requirement shown in Fig. 2. For example, if Grade 60 (414 MPa) steel is used, the minimum amount of top bonded reinforcement should be 1.29 sq in. (830 mm²).

Another simple method to verify the strength design is by using commercially available computer programs for the design of the compression member, of which PCACOL is an example. An interaction diagram for this example “column” is shown in Fig. 3. The mark, +1, in the figure represents the load effects due to axial force combined with bending moment.

In this example problem, the mark is matched with the interaction diagram. This indicates that the results from the spreadsheet program and PCACOL are consistent. Additional design examples of the proposed method are included in Appendix B.

Control of Top Fiber Cracking

The stress at the top fiber of the sec-
tion in the preceding example exceeds
the modulus of rupture of concrete,
7.5 \( \sqrt{f_c} \) (0.62 \( \sqrt{f_c} \)). However, the
strength design approach proposed
herein does not directly establish the
adequacy of top tension bonded rein-
forcement for crack control.

The current code provisions also
fail to directly relate the empirically
computed amount of bonded rein-
forcement to crack width or spacing.
The reinforcement area is calculated
as if the section were uncracked, with
the steel stress limited to an arbitrary
30 ksi (207 MPa) to control cracking.

The most rigorous approach is to
analyze a cracked section subjected to
the combined effects of unfactored
prestress and self-weight. An analysis
of cracked prestressed concrete sec-
tions is described in detail in Refer-
ence 12.

Using that method for the above ex-
ample, the depth of the neutral axis
can be found to vary from 24.66 to
28.52 in. (626 to 724 mm) and the
corresponding steel stress to vary
from 12.03 to 7.21 ksi (83 to 50 MPa)
if the top reinforcement is varied from
1.29 to 5.78 sq in. (830 to 3726 mm²)
(see Fig. 4). The same range of rein-
forcement area is shown in Fig. 2 to
correspond to a steel stress of 60 to 30
ksi (414 to 207 MPa) at nominal
strength levels.

The PCI Design Handbook rectan-
gular and inverted tee sections, and
the standard NU (Nebraska University)
inverted tee sections were in-
vestigated using the cracked section
analysis of Reference 11. Table 2 pre-
ents a summary of the level of steel
stress after cracking. Note that in all
cases, the steel area used in the analy-
sis was the area corresponding to the
one producing a stress of 60 ksi (414
MPa) at nominal strength levels.

Values of steel stresses at unfac-
tored bond levels in all cases were
below the current ACI Code limit of
30 ksi (207 MPa). Therefore, it is rec-
ommended that if a maximum steel
strain of 0.0021 and a steel stress of
(0.0021)(29000) = 60 ksi (414 MPa)
are used in the strength design
method, there is no need to perform a
service load cracked section analysis
or to check for crack control for steel
grades not greater than Grade 60.

Fig. 3. PCACOL interaction diagram for prestress transfer. Note: 1 kip = 4.44 kN,
1 kip-ft = 1.356 kN-m, 1 ft = 0.3048 m.

Fig. 4. Cracked section analysis of 16RB40.
Approximate Equivalent Allowable Stress for Pretensioned Members

An effort was made to establish the compressive stress limit at unfactored load levels that would produce comparable results to the proposed strength design method. The objective is to give design engineers an alternative limit to the 0.6 $f'$ currently used, while still permitting them to use the familiar allowable (working) stress design procedures.

Standard double tee, rectangular and inverted tee cross sections were chosen to represent geometry variation. For comparison purposes, the PCI Design Handbook rectangular and double tee sections, and NU inverted tee sections were used in the study.

The analysis indicated that the geometry of a member cross section is one of the most important parameters. The stress limits for the NU inverted tee sections varied from 0.66 $f'_{ci}$ to 0.67 $f'_{ci}$, while the values for the PCI rectangular sections varied from 0.69 $f'_{ci}$ to 0.70 $f'_{ci}$. The values for the PCI double tee sections were the highest, namely, 0.73 $f'_{ci}$ to 0.76 $f'_{ci}$.

For simplification purposes, it is recommended that the limit 0.6 $f'_{ci}$ be replaced with the following formula:

$$\left(0.6 + \frac{\gamma_b}{5h}\right)f'_{ci} \leq 0.75f'_{ci} \quad (5)$$

where

- $\gamma_b$ = distance between section centroid and extreme bottom fiber
- $h$ = overall depth of member

In the rectangular example previously given, Eq. (5) produces an allowable stress limit equal to (0.6 + 0.5/5) $f'_{ci}$ = 0.7 $f'_{ci}$, compared to 0.73 $f'_{ci}$ by the strength design approach.

This shows that the proposed formula is more conservative than the theoretically more rigorous strength design method. Results of the parametric study for the proposed formula are summarized in Appendix C.

EXPERIMENTAL PROGRAM

One of the tasks of this project was to conduct experiments to determine the impact of creep of concrete under very high compressive stress levels induced by prestress. To achieve this goal, two simply supported inverted tee specimens with a cast-in-place composite topping were designed, produced and tested at the University of Nebraska Structures Laboratory (see Fig. 5). They are referred to here as Specimens 1 and 2.

Design of Specimens

The specimens were designed to have a total length of 33 ft (10.06 m), a span length of 32 ft (9.75 m) and a span-to-depth ratio of 32. Section di-

**Table 2. Summary of steel stress after cracking.**

<table>
<thead>
<tr>
<th>Section</th>
<th>No. of strands</th>
<th>$A_s$ [sq in. $(\text{mm}^2)$]</th>
<th>$e$ [in. (mm)]</th>
<th>Neutral axis [in. (mm)]</th>
<th>Steel stress $f_s$ [ksi (MPa)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>12RB16</td>
<td>5</td>
<td>0.52 (335)</td>
<td>6.00 (152)</td>
<td>8.20 (208)</td>
<td>15.99 (110)</td>
</tr>
<tr>
<td>12RB20</td>
<td>8</td>
<td>0.72 (465)</td>
<td>7.25 (184)</td>
<td>10.76 (273)</td>
<td>16.40 (113)</td>
</tr>
<tr>
<td>12RB24</td>
<td>10</td>
<td>1.03 (665)</td>
<td>9.00 (229)</td>
<td>12.85 (325)</td>
<td>17.95 (124)</td>
</tr>
<tr>
<td>12RB28</td>
<td>12</td>
<td>1.23 (794)</td>
<td>10.50 (267)</td>
<td>15.14 (385)</td>
<td>18.25 (126)</td>
</tr>
<tr>
<td>12RB32</td>
<td>13</td>
<td>1.53 (987)</td>
<td>12.31 (313)</td>
<td>17.31 (440)</td>
<td>18.45 (127)</td>
</tr>
<tr>
<td>12RB36</td>
<td>15</td>
<td>1.81 (1168)</td>
<td>14.00 (356)</td>
<td>19.49 (495)</td>
<td>19.00 (131)</td>
</tr>
<tr>
<td>16RB24</td>
<td>13</td>
<td>1.40 (903)</td>
<td>9.08 (231)</td>
<td>12.82 (325)</td>
<td>17.97 (124)</td>
</tr>
<tr>
<td>16RB28</td>
<td>13</td>
<td>1.71 (1103)</td>
<td>11.08 (281)</td>
<td>14.77 (375)</td>
<td>18.29 (126)</td>
</tr>
<tr>
<td>16RB32</td>
<td>18</td>
<td>2.07 (1335)</td>
<td>12.33 (313)</td>
<td>15.18 (385)</td>
<td>18.89 (130)</td>
</tr>
<tr>
<td>16RB36</td>
<td>20</td>
<td>2.46 (1587)</td>
<td>14.10 (358)</td>
<td>19.44 (494)</td>
<td>19.20 (132)</td>
</tr>
<tr>
<td>16RB40</td>
<td>22</td>
<td>2.82 (1819)</td>
<td>15.82 (402)</td>
<td>21.64 (550)</td>
<td>19.35 (133)</td>
</tr>
<tr>
<td>28IT28</td>
<td>13</td>
<td>1.94 (1252)</td>
<td>9.09 (231)</td>
<td>15.18 (385)</td>
<td>15.45 (107)</td>
</tr>
<tr>
<td>28IT44</td>
<td>20</td>
<td>2.78 (1794)</td>
<td>14.72 (374)</td>
<td>24.25 (565)</td>
<td>17.45 (120)</td>
</tr>
<tr>
<td>28IT60</td>
<td>28</td>
<td>3.58 (2310)</td>
<td>21.02 (534)</td>
<td>32.36 (822)</td>
<td>20.05 (138)</td>
</tr>
<tr>
<td>34IT24</td>
<td>17</td>
<td>2.68 (1729)</td>
<td>8.03 (204)</td>
<td>13.33 (339)</td>
<td>14.66 (101)</td>
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<tr>
<td>34IT40</td>
<td>30</td>
<td>4.27 (2755)</td>
<td>13.92 (354)</td>
<td>24.57 (570)</td>
<td>17.58 (121)</td>
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<tr>
<td>34IT60</td>
<td>42</td>
<td>5.55 (3581)</td>
<td>22.07 (561)</td>
<td>32.34 (827)</td>
<td>20.37 (140)</td>
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<tr>
<td>40IT20</td>
<td>18</td>
<td>2.18 (1406)</td>
<td>6.74 (171)</td>
<td>10.37 (263)</td>
<td>16.36 (113)</td>
</tr>
<tr>
<td>40IT32</td>
<td>30</td>
<td>3.91 (2523)</td>
<td>11.27 (286)</td>
<td>17.45 (443)</td>
<td>17.39 (120)</td>
</tr>
<tr>
<td>40IT40</td>
<td>38</td>
<td>5.47 (3529)</td>
<td>14.47 (368)</td>
<td>22.98 (584)</td>
<td>16.71 (111)</td>
</tr>
<tr>
<td>40IT48</td>
<td>40</td>
<td>6.05 (3903)</td>
<td>17.68 (449)</td>
<td>26.27 (667)</td>
<td>19.30 (133)</td>
</tr>
<tr>
<td>40IT52</td>
<td>48</td>
<td>6.52 (4206)</td>
<td>19.32 (491)</td>
<td>28.39 (721)</td>
<td>19.77 (136)</td>
</tr>
<tr>
<td>NU-IT300</td>
<td>22</td>
<td>0</td>
<td>1.35 (35)</td>
<td>8.27 (210)</td>
<td>–</td>
</tr>
<tr>
<td>NU-IT400</td>
<td>22</td>
<td>0</td>
<td>2.66 (66)</td>
<td>8.27 (210)</td>
<td>–</td>
</tr>
<tr>
<td>NU-IT500</td>
<td>22</td>
<td>0.22 (142)</td>
<td>3.99 (101)</td>
<td>10.43 (265)</td>
<td>19.31 (133)</td>
</tr>
<tr>
<td>NU-IT600</td>
<td>22</td>
<td>0.47 (303)</td>
<td>5.50 (140)</td>
<td>11.95 (304)</td>
<td>21.77 (150)</td>
</tr>
<tr>
<td>NU-IT700</td>
<td>22</td>
<td>0.73 (471)</td>
<td>7.08 (180)</td>
<td>13.89 (353)</td>
<td>21.82 (150)</td>
</tr>
<tr>
<td>NU-IT800</td>
<td>22</td>
<td>0.97 (626)</td>
<td>8.74 (222)</td>
<td>15.95 (405)</td>
<td>21.37 (147)</td>
</tr>
<tr>
<td>NU-IT900</td>
<td>22</td>
<td>1.20 (774)</td>
<td>10.44 (265)</td>
<td>18.08 (459)</td>
<td>20.83 (144)</td>
</tr>
</tbody>
</table>

Notes: $M_0$ is assumed equal to zero.
1 in. = 25.4 mm, 1 ksi = 6.9 MPa.

Fig. 5. The inverted tee specimens with cast-in-place composite topping.
Dimensions and reinforcement details are shown in Fig. 6. Stirrups (#3 U-bars) at 2 in. (51 mm) spacing were used in the bottom flange only for 3 ft (914 mm) from each member end. This confinement at member ends was used to control member end splitting cracks at the time of prestress transfer.

In designing the specimens, it was assumed that the initial prestress loss was 10 percent and the final loss was 20 percent of the prestress just before transfer. The strand tension before transfer was set at 0.75 $f_{puv}$ or 202.5 ksi (1396 MPa).

These losses were later calculated in more detail, using the method in Reference 14, and compared with the measured quantities. Note, however, that the specimens were designed to have significantly higher compressive stresses of 0.85 $f'_{ci}$ at member ends and 0.71 $f'_{ci}$ at midspan. This stress is less conservative than a design using the recommended equivalent stress limit formula:

$$(0.6 + y_b / 5h) f'_{ci} = (0.6 + 2.34 / 40) f'_{ci} = 0.66 f'_{ci}$$

A summary of the member stresses at the transfer point and midspan as compared to the allowable code limit is shown in Table 3.

### Specimen Construction and Materials

The specimens were prestressed using Grade 270 (1860 MPa), 0.5 in. (12.7 mm) diameter, low-relaxation seven-wire strands. A total of nine strands were used for each specimen. The average elongation of strands during prestressing was measured to check the jacking force. The jacking forces were 74 and 77 percent of 270 ksi (1862 MPa) for Specimens 1 and 2, respectively. After jacking of the strands was completed, the height of the form was adjusted to provide the correct eccentricity of the strands.

Ready mixed concrete was supplied. Concrete with Type III cement for the

### Table 3. Summary of stresses for inverted tees.

<table>
<thead>
<tr>
<th>Load</th>
<th>$f'_i$ [ksi (MPa)]</th>
<th>$f'_b$ [ksi (MPa)]</th>
<th>$f'_f$ [ksi (MPa)]</th>
<th>$f'_d$ [ksi (MPa)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>P/A</td>
<td>3403 (23.5)</td>
<td>3403 (23.5)</td>
<td>3403 (23.5)</td>
<td>3403 (23.5)</td>
</tr>
<tr>
<td>$P_e/S$</td>
<td>1849 (12.7)</td>
<td>-4253 (-29.3)</td>
<td>1849 (12.7)</td>
<td>-4253 (-29.3)</td>
</tr>
<tr>
<td>$M_{d,one} / S$</td>
<td>-259 (-1.8)</td>
<td>594 (4.1)</td>
<td>-1088 (-7.5)</td>
<td>2501 (17.2)</td>
</tr>
<tr>
<td>Total stresses</td>
<td>4993 (34.4)</td>
<td>-255 (-1.8)</td>
<td>4164 (28.7)</td>
<td>1651 (11.4)</td>
</tr>
<tr>
<td>Current AASHTO limit and ACI 318 limit</td>
<td>$0.6 f'_d = 3500 (24.1)$</td>
<td>200 w/o/ft.</td>
<td>$200 w/o/ft.$</td>
<td>3500 (24.1)</td>
</tr>
</tbody>
</table>

Notes: 1. Positive sign is compression and negative sign is tension.
2. $P/A$ and $P_e/S$ are the stresses due to prestressing.
3. $M_{d,one} / S$ is the stress due to precast IT beam self-weight.
4. $M_{d,one} / S$ is the stress due to deck self-weight.
5. 1 ksi = 6.9 MPa.

---

January-February 2001
precast portion of Specimen 1 was placed on March 12, 1999, and concrete with Type I cement for Specimen 2 was placed on August 2, 1999. Burlap soaked with water was used to cure the freshly cast concrete.

One day after concrete placement, the side forms were removed in order to install mechanical demec gauges on the specimen. The demec gauges were attached to the concrete surface to monitor the change in the concrete strain with time (see Fig. 7). The relative displacement of the gauge points was measured with an accuracy of 4.0 x 10⁻⁶ of the concrete strain. A total of 148 demec gauges were used for each specimen, as shown in Fig. 8.

Concrete placement for the topping concrete of Specimens 1 and 2 was done on June 4, 1999 and December 17, 1999, respectively. The ages of the precast members of Specimens 1 and 2 were 84 and 137 days, respectively. These relatively long periods were used to determine if any detrimental creep effects or unpredictable cambers take place. Cylinders were prepared in accordance with ASTM C31-87. They were kept in the same environmental condition as the specimens, and their compressive strengths are listed in Table 4.

**TEST RESULTS**

Discussed below is concrete stress at transfer, camber, and the transverse strain effect.

**Concrete Stress at Transfer**

Prestress losses due to steel relaxation and elastic shortening at midspan of the specimens were calculated. They were found to be 11.78 and 12.64 percent of the jacking force for Specimens 1 and 2, respectively. The actual concrete stresses at transfer were 0.79$f_c^t$ and 0.84$f_c^t$ for Specimens 1 and 2, respectively.

Variation of strain with time is attributed to creep and shrinkage of the concrete, and relaxation in the steel. A comparison between the calculated concrete strain and the measured concrete strain at midspan (Level A) is shown in Figs. 9 and 10. There appears to be good agreement between the theoretical and measured concrete strain in both specimens.

**Camber**

The stresses in the concrete and pre-stressing steel vary continually with time due to the effects of creep and
shrinkage of concrete and stress relaxation of steel. The measured unit weight of concrete was 150 and 147 lb per cu ft (2400 and 2355 kg/m³) for Specimens 1 and 2, respectively.

A comparison between the theoretical and the measured camber after releasing the strands is shown in Figs. 11 and 12. The measured camber at midspan immediately after prestress transfer was 1.18 and 1.38 in. (35 and 30 mm) for Specimens 1 and 2, respectively.

These values are in agreement with the calculated values of 1.17 and 1.24 in. (29.7 and 31.5 mm) for Specimens 1 and 2, respectively. The time-dependent camber was calculated based on linear elastic analysis, and was plotted at 81 days for Specimen 1 and at 128 days for Specimen 2 before casting the deck.

The relatively high initial compression in relationship to the concrete strength could still be reasonably estimated using the straight-line elastic analysis. The calculated camber immediately after prestress transfer was still predictable by linear elastic analysis, within the reasonable margins of error. It was possible to fairly accurately predict time-dependent camber using available creep prediction methods and the usual assumption of linear proportionality of creep strain to elastic strain.

**Transverse Strain Effect**

The transverse strain, due to high compressive stress at the ends of the beams, was also monitored after release. Near the ends of the specimens, a possible increase in transverse tensile strain with time had been mentioned as a possible concern due to the high compressive stress at prestress transfer. After the strands were released, a total of 30 demec gauges were installed at the bottom surface at both ends of Specimen 2, as shown in Fig. 13.

Note that it was not possible to install the demec gauges before releasing the prestress. The transverse strain measurement was monitored immediately after prestress release. It was observed that no sign of splitting or any microcracking occurred at the bottom surface of the specimen.

The highest transverse strain occurred at the time of prestress transfer. Beyond that time, compression (rather than tension) strain increments, due to creep and shrinkage, developed with time. The average measured strain at both ends is plotted in Fig. 14. Note that the transverse strain near the member ends actually diminished, rather than increased with time. This may be attributed to the effects of shrinkage and reduction of prestress with time.

Although not specifically mentioned in ACI 318, some design engineers be-

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Specified</th>
<th>Actual</th>
<th>Specified</th>
<th>Actual</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5870 (40 MPa)</td>
<td>5900 (41 MPa)</td>
<td>7000 (48 MPa)</td>
<td>10780 (74 MPa)</td>
</tr>
<tr>
<td>2</td>
<td>5870 (40 MPa)</td>
<td>5500 (39 MPa)</td>
<td>7000 (48 MPa)</td>
<td>6200 (43 MPa)</td>
</tr>
</tbody>
</table>

Table 4. Measured concrete strength.

![Fig. 9. Concrete strain vs. time for Specimen 1.](image)

![Fig. 10. Concrete strain vs time for Specimen 2.](image)
lieve that limiting the compressive stress at transfer to $0.6 f'_{c}$ would prevent members from having "too much creep deformation, too much prestress loss, and excessive transverse strain (dilation)."

Based on the test results of the inverted tee specimens, no negative impact was detected due to the prestress levels used in the test when significantly exceeding the allowable concrete compressive stress limit at transfer, and even the limits corresponding to the proposed strength design approach. There are two reasons why design engineers should not be unnecessarily alarmed when they calculate high compressive stresses in the concrete due to prestress transfer:

1. Compared to externally loaded columns, bonded pretensioned prestress is an internal system of forces that has some degree of self-adjusting capability. High concrete compressive stress and strain result in high prestress losses, which in turn cause a reduction in concrete compression.

2. Another factor that tends to exaggerate the effects on concrete from prestress transfer is the conventional linear concrete stress-strain relationship. It is recognized that nonlinear analysis may be unattractive to many design engineers. A study by Huo and Tadros, titled "Allowable Compression Strength of Concrete at Prestress Release," compares accurate nonlinear behavior with approximate linear stress analyses.

In that paper, it is shown that a linear analysis significantly overestimates the concrete and steel stresses. Thus, a linear analysis may give the false impression that a member is overstressed when in fact a more accurate nonlinear analysis would show this is not the case.

CONCLUSIONS

Based on this analytical and experimental study, the following conclusions can be drawn:

1. The authors propose that the allowable compression stress limit requirement at prestress transfer be eliminated for pretensioned concrete members. The factor of safety using working stress design can vary significantly with such parameters as reinforcement ratio, concrete strength, and

---

Fig. 11. Midspan camber of Specimen 1.

Fig. 12. Midspan camber of Specimen 2.

Fig. 13. Demec gauge locations for transverse strain of Specimen 2 at bottom of beam. Note: 1 ft = 0.3048 m, 1 in. = 25.4 mm.
section geometry. Using a constant allowable concrete compressive stress at transfer cannot be justified as the basis for controlling compression failure.

2. The authors further propose that an alternative approach using the strength design method presented in this paper be adopted to replace the working stress design approach. The proposed strength design approach can be applied using commercially available column design software or a design spreadsheet. The input is the prestress just before transfer, factored by a 1.2 load factor, and the self-weight moment factored by either a 1.2 or 0.8 factor, whichever is more critical. Also, the section dimensions and prestressing steel are input. The program output is the concrete strength $f'_c$, area of top bonded reinforcement $A_t$, and stress in the top bonded reinforcement $f_p$. Since there are an unlimited number of acceptable solutions, the design engineer has the option to choose $f'_c$ and obtain the required corresponding $f_p$, then check that the corresponding stress is less than 60 ksi (414 MPa).

3. As a transitional measure, to convert from the working stress approach to the strength design method, this study has furnished a semi-empirical allowable compressive stress limit that is a more realistic substitute for the 0.6 $f'_c$ limit. In the proposed limit, the coefficient 0.6 should be replaced with $0.6 + y/5h \leq 0.75$. This formula was based on curve-fitting the strength design results for highly prestressed member ends of various cross-sectional geometries.

4. The strength design method provides a more uniform factor of safety against concrete crushing than does the working stress approach. It is expected that this new method will have significant benefits for designing precast/prestressed concrete structures. This approach allows the prestress to be released at a lower concrete strength than currently permitted and thus a more rapid production cycle would result. The demand for debonding or draping of strands at member ends would be reduced. The cost of accelerated curing would be minimized. Alternatively, more prestressing could be introduced into a member, thus increasing its external load carrying capacity.

5. The proposed strength design approach provides for adequate safety against concrete crushing at prestress transfer. As a by-product of the analysis, the area of bonded reinforcement at the top of the member can be found. Thus, the need for empirically calculating the steel area as is done in current codes is eliminated. If such reinforcement is found to be needed for a given member, it is recommended that its strain be limited to 60 ksi (414 MPa)/$E_o$ or 0.0021. That limit would curtail the steel stress at unfactored load levels to less than the 30 ksi (207 MPa) limit specified in the codes, without requiring any need to perform a complicated cracked section stress analysis.

**RECOMMENDATIONS**

1. The current ACI 318-99 Code provisions should be revised as given in Appendix D.

2. The current ASSHTO Standard Specifications for Highway Bridges requires that in precast, prestressed concrete members, the extreme fiber stress in tension not exceed the limit of 7.5 $\sqrt{f'_c}$ (0.62 $\sqrt{f'_c}$). For composite members, this limit should be waived at the extreme top fiber of precast, prestressed concrete members. Bonded reinforcement can be used when the extreme fiber stress in tension of precast composite bridge members exceeds the limit of 6 $\sqrt{f'_c}$ (0.50 $\sqrt{f'_c}$). A direct result of this is that a satisfactory design can be accomplished when using the strength design analysis approach with the serviceability criteria requirements.

3. The load factors and strength reduction factor proposed in this paper appear to be conservative. The 20 percent fluctuation of applied prestress force appears to be too high because the accuracy of the jacking force can be measured. In addition, the strength reduction factor $\phi = 0.7$ is low because the eccentricity of strand can be controlled. However, the load factor of 0.8 or 1.2 for self-weight moment may be reasonable. Therefore, further study, based on the theoretical and experimental program, of the load factors and strength reduction factor is still needed, in order to refine these recommendations.

**ACKNOWLEDGMENT**

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APPENDIX A – NOTATION

- \( a \) = depth of equivalent rectangular stress block
- \( A \) = cross-sectional area
- \( A_t \) = area of tension reinforcement
- \( A_p' \) = area of “compression reinforcement,” i.e., prestressing steel
- \( b \) = width of compression face of member
- \( c \) = distance from extreme compression fiber to neutral axis
- \( d \) = effective depth of section
- \( d' \) = distance from extreme compression fiber to centroid of compression reinforcement
- \( e \) = eccentricity of prestress force from centroid of section
- \( E_c \) = modulus of elasticity of concrete
- \( E_p' \) = modulus of elasticity of prestressing steel
- \( E_n' \) = modulus of elasticity of nonprestressed steel
- \( E_{p'}' \) = modulus of elasticity of compression reinforcement
- \( f_b \) = stress in bottom fiber of cross section
- \( f_t \) = stress in top fiber of cross section
- \( f_c' \) = specified compressive strength of concrete at service
- \( f_{ci} \) = specified compressive strength of concrete at time of initial prestress
- \( f_{pu} \) = specified tensile strength of prestressing tendons
- \( f_y \) = specified yield strength of non-prestressed reinforcement
- \( f_t' \) = calculated stress in tension reinforcement
- \( f_c' \) = calculated stress in compression reinforcement, i.e. reduction in tensile stress in prestressing steel
- \( h \) = overall depth of member
- \( K \) = coefficient of allowable concrete compressive stress at prestress release
- \( L \) = span length
- \( M_w \) = moment due to self weight of member
- \( M_n \) = nominal moment strength
- \( M_{d,w} \) = moment due to superimposed dead load
- \( M_{l,w} \) = moment due to superimposed live load
- \( M_u \) = factored moment
- \( P_e \) = effective prestress force after all prestress losses
- \( P_i \) = prestress force immediately after prestress release
- \( P_a \) = nominal axial load strength
- \( y_b \) = distance between section centroid and extreme bottom fiber
- \( S \) = section modulus
- \( \beta_1 \) = equivalent stress block factor
- \( \varepsilon_c \) = concrete strain
- \( \varepsilon_s \) = tension reinforcement strain
- \( \varepsilon_u \) = compression reinforcement strain
- \( \phi \) = strength reduction factor
EXAMPLE 1

Given:

8DT24: Span = 44 ft (13.4 m), \( f'_{ci} = 5000 \text{ psi (34.5 MPa)} \),

superimposed DL load = 10 psf (0.48 kPa), superimposed
LL load = 90 psf (4.31 kPa).

Determine strand pattern and strength at transfer.

(a) **Current Design**

According to the PCI Design Handbook, Fifth Edition:

Use a draped strand pattern, eight strands, \( e_c = 9.15 \text{ in. (232 mm)} \) and \( e_c = 14.40 \text{ in. (366 mm)} \), as shown in Fig. B1.

The required \( f'_{ci} \) is based on the compression at transfer at midspan.

\[
0.6 f'_{ci} = \frac{P_t}{A} + \frac{P e}{S_b} - \frac{M_g}{S_b} - \frac{M_{sd}}{S_b} - \frac{M_{sl}}{S_b}
\]

\[
= \frac{223.07 \times 1000}{401} + \frac{223.07 \times 4.4}{1224} \times 1000
\]

\[
- \frac{1213.9}{1224} \times 1000
\]

\[
f'_{ci} = 3648 \text{ psi (25.2 MPa)}
\]

(b) **Proposed Design**

A straight strand pattern will be attempted. The strand pattern is shown in Fig. B2.

The top strands are fully tensioned. They are intended to control the conditions at transfer at the member end.

Use ten strands, \( e_c = 8.75 \text{ in. (222 mm)} \).

\[
M = 6199 \text{ kip-in. (700 kN-m)}
\]

\[
M_s = 5579 \text{ kip-in. (630 kN-m)}
\]

(1) **Check strength:**

(2) **Check tensile stress at final condition:**

\[
f_b = \frac{P_s}{A} + \frac{P e}{S_b} - \frac{M_g}{S_b} - \frac{M_{sd}}{S_b} - \frac{M_{sl}}{S_b} < 12 \sqrt{f'_c}
\]

\[
= 247.9 \times 1000 + \frac{247.9 \times 8.75}{1224} \times 1000
\]

\[
1213.9 \times 1000 + \frac{232.3}{1224} \times 1000
\]

\[
2090.9 \times 1000
\]

\[
= -499 \text{ psi (3.44 MPa) (tension)}
\]

\[
499 < 12 \sqrt{5000} = 848 \text{ psi (5.85 MPa) ok.}
\]

(3) **Check conditions at transfer**

Because strands are straight, only the section at transfer point needs to be checked.

\[
P_t = 0.75 \times 270 \times 10 \times 0.153 = 309.8 \text{ kips (1378 kN)}
\]

\[
1.2 P_t = 371.8 \text{ kips (1654 kN)}
\]

Self-weight moment at transfer point from member end:

\[
M_g = 219 \text{ kip-in. (24.74 kN-m)}
\]

\[
0.8 M_g = 175.2 \text{ kip-in. (19.79 kN-m)}
\]

Compression member is designed for axial force = 371.8 kips (1654 kN) and bending moment = 1.2 \( P_t e - 0.8 M_g \)

\[
= (371.8 \times 8.75 - 175.2)/12 \text{ kips = 256.5 kip-ft (348 kN-m)}
\]

---

**Fig. B1. Draped strand. Note: 1 ft = 0.3048 m, 1 in. = 25.4 mm.**

**Fig. B2. Straight strand. Note: 1 ft = 0.3048 m, 1 in. = 25.4 mm.**
Using these applied loads, input any commercially available computer program for design of compression member; PCACOL is one such example.

Enter section geometry, compression "reinforcement" $A' = 8 \times 0.153 = 1.224$ sq in. (790 mm²), at bottom fibers and tension reinforcement $A_t = 2 \times 0.153 = 0.306$ sq in. (197 mm²) at 2 in. (51 mm) from top fibers. Also, input the factored loads $P_u = 371.8$ kips (1654 kN) and $M_u = 256.5$ kip-ft (348 kN-m).

Solve for $f'$ through trial and adjustment. Fig. B3 shows a typical interaction diagram, output with Trial No. 1 using $f_{ci}' = 3000$ psi (20.68 MPa) and Trial No. 2, $f_{ci}' = 3230$ psi (22.27 MPa). In Trial No. 2, the design strength matches the required strength.

It is interesting to note that if "Current Design" for release strength is used, the required $f_{ci}'$ will be as follows:

$$0.6f_{ci}' = \frac{P}{A} + \frac{P_e}{S_b} - \frac{M_g}{S_b}$$

$$= \frac{278.84}{401} \times 1000 + \frac{278.84 \times 8.75}{1224} \times 1000 - \frac{219.01}{1224} \times 1000$$

$$f_{ci}' = 4183$$ psi (28.84 MPa)

The above value is significantly higher than that required by the proposed strength design method.

EXAMPLE 2

Given:
40IT52: Span = 48 ft (14.63 m), $f_{ci}' = 3500$ psi (24.13 MPa).

Determine strand pattern and strength at transfer.

(a) Current Design

According to the PCI Design Handbook, Fifth Edition:
Use 48 strands, $e_c = 17.62$ in. (448 mm), as shown in Fig. B4.

The required $f_{ci}'$ is determined based on compression at release at midspan.

$$f_{b} = \frac{P}{A} + \frac{P_e}{S_b} - \frac{M_g}{S_b}$$

$$= \frac{1338.4}{1504} \times 1000 + \frac{1338.4 \times 17.62}{15,497} \times 1000$$

$$- \frac{5415.6}{15,497} \times 1000$$

$$= 2062$$ psi

$$0.6f_{ci}' = 2062$$ psi (14.22 MPa)

$$f_{ci}' = 3437$$ psi (23.7 MPa)

A total of 12 strands, 25 percent of total, were debonded. The new prestress is 1003.8 kips (6920 kN), the new eccentricity is 17.28 in. (439 mm) and the stresses at transfer are determined:
Using these applied loads, input PCACOL. The required concrete strength is 3500 psi (24.13 MPa).

EXAMPLE 3

Given:
NU1100 (43.3 in. (1100 mm) deep): Span = 130 ft (39.62 m)
Use 58 strands, $e_s = 9.04$ in. (230 mm)
Determine strength at release when a total of 12 strands were draped as high in the web as possible, as shown in Fig. B6.

(a) Current Design

The required $f_{ci}'$ is determined based on compression at prestress transfer at transfer point.

$$f_b = \frac{P_t}{A_s} + \frac{P_e}{A_s} - \frac{M_g}{S_b}$$

$$= \frac{1896 \times 14.99 - 719.5}{12}$$

$$= 2274.8 \text{ kip}-\text{ft (3084 kN-m)}$$

(b) Proposed Design

A total of three 1/2 in. (12.7 mm) diameter fully pretensioned strands were added at the top fiber. No debonding at the member ends would be required, as shown below (see Fig. B5).

Check conditions at transfer:

Because the strands are straight, only the section at the transfer point needs to be checked.

$A_{ps} = 51(0.153) = 7.803 \text{ sq in. (5034 mm}^2\text{)}$

$P_t = 0.75 \times 270 \times 7.803 = 1580 \text{ kips (7028 kN)}$

$1.2P_t = 1896 \text{ kips (8433 kN)}$

Eccentricity, $e = 14.99$ in. (381 mm)

$M_g = 899.4 \text{ kip}-\text{in. (102 kN-m)}$

$0.8 \times 719.5 = 575.6 \text{ kip-in. (81 kN-m)}$

Design a compression member subjected to axial force = 1896 kips (8433 kN) and bending moment:

$1.2P_t - 0.8M_g = (1896 \times 14.99 - 719.5)/12$

$= 2274.8 \text{ kip-ft (3084 kN-m)}$
APPENDIX C – PARAMETRIC ANALYSIS FOR APPROXIMATE EQUIVALENT COMpressive STRESS LIMIT

C.1 Assumptions for Developing a Semi-Empirical Working Stress Design (WSD)

The following assumptions were used to establish the coefficient \( K \) in \( Kf_{ct} \), where \( K \) is the allowable concrete compressive stress at prestress transfer. Effort was made to establish values of \( K \) that produce equivalent results to the strength design approach being proposed.

- Use either Grade 60 mild steel reinforcement, \( A_s \), with \( E_s = 29,000 \text{ ksi} \) (200 GPa) or low relaxation, 270 ksi (1860 MPa), strand; with \( E_{ps} = 28,500 \text{ ksi} \) (197 GPa) is used when service load tensile stress limit at top fiber is greater than \( 6\sigma_f \) (0.5 \( \sigma_f \)).

- Prestress immediately before transfer is equal to 75 percent of 270 ksi (1860 MPa). Only the end section is considered while the self-weight moment is ignored.

- For working stress design, elastic loss is assumed equal to 10 percent of initial prestress.

C.2 Selected Parameters

The standard double tee, rectangular and inverted tee sections were chosen to represent geometry variations. PCI Design Handbook rectangular and double tee sections were used. Service load stress at bottom fibers of 2450 psi (16.89 MPa) was assumed for rectangular and double tee sections. NU inverted tee sections were used in the study. The maximum number of strands that can be accommodated in the standard NU inverted tee, 22, was used in the bottom flange. Microsoft Excel Spreadsheet program was developed and used for computation. All runs were verified with the PCA-COL software.

C.3 Proposed Formula

An approximate formula was developed to determine the \( K \) value at the end section of the pretensioned member:

\[
K = 0.6 + (\gamma_b / 5h) \leq 0.75 \quad (5)
\]

Based on Eqs. (3) and (4), the \( K \) values range from 0.66 to 0.76. Double tee sections with the highest strand eccentricity allow for the greatest \( K \) value, while the inverted tee sections require the smallest \( K \) value.

C.4 Tables of Results

Tables C1, C2 and C3 show a comparison between the \( K \) value based on strength design and the proposed formula.

Table C1. \( K \) value for rectangular section.

<table>
<thead>
<tr>
<th>Rectangular section</th>
<th>No. of strand</th>
<th>( A_p ) (sq in.)</th>
<th>( A_s ) (sq in.)</th>
<th>( \gamma ) (in.)</th>
<th>( \delta ) (in.)</th>
<th>( e ) (in.)</th>
<th>( A ) (sq in.)</th>
<th>( h ) (in.)</th>
<th>( S_o ) (in^2)</th>
<th>( \gamma_o ) (in.)</th>
<th>( f_o ) (psi)</th>
<th>( f_{st} ) (psi)</th>
<th>( K )</th>
<th>0.6 + (( \gamma_b/5h ))</th>
</tr>
</thead>
<tbody>
<tr>
<td>12RB20</td>
<td>8</td>
<td>1.224</td>
<td>0.459</td>
<td>4.547</td>
<td>18.5</td>
<td>5.453</td>
<td>240</td>
<td>20</td>
<td>800</td>
<td>10</td>
<td>2450</td>
<td>3517</td>
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<td>0.70</td>
</tr>
<tr>
<td>12RB24</td>
<td>10</td>
<td>1.53</td>
<td>0.427</td>
<td>5.878</td>
<td>22.5</td>
<td>6.122</td>
<td>288</td>
<td>24</td>
<td>1152</td>
<td>12</td>
<td>2450</td>
<td>3530</td>
<td>0.69</td>
<td>0.70</td>
</tr>
<tr>
<td>12RB28</td>
<td>12</td>
<td>1.836</td>
<td>0.404</td>
<td>7.186</td>
<td>26.5</td>
<td>6.814</td>
<td>336</td>
<td>28</td>
<td>1568</td>
<td>14</td>
<td>2450</td>
<td>3536</td>
<td>0.69</td>
<td>0.70</td>
</tr>
<tr>
<td>12RB32</td>
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<td>1.989</td>
<td>0.671</td>
<td>7.491</td>
<td>30.5</td>
<td>8.509</td>
<td>384</td>
<td>32</td>
<td>2048</td>
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</tr>
<tr>
<td>12RB36</td>
<td>15</td>
<td>2.295</td>
<td>0.641</td>
<td>8.817</td>
<td>34.5</td>
<td>9.183</td>
<td>432</td>
<td>36</td>
<td>2592</td>
<td>18</td>
<td>2450</td>
<td>3508</td>
<td>0.70</td>
<td>0.70</td>
</tr>
<tr>
<td>16RB24</td>
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<td>1.989</td>
<td>0.671</td>
<td>5.619</td>
<td>22.5</td>
<td>6.381</td>
<td>384</td>
<td>24</td>
<td>1536</td>
<td>12</td>
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<td>3513</td>
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<td>0.70</td>
</tr>
<tr>
<td>16RB32</td>
<td>18</td>
<td>2.754</td>
<td>0.695</td>
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<td>30.5</td>
<td>7.997</td>
<td>512</td>
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<td>2731</td>
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<tr>
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<td>0.854</td>
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<td>9.183</td>
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<td>3456</td>
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<td>9.626</td>
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<td>10.374</td>
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<td>3495</td>
<td>0.70</td>
<td>0.70</td>
</tr>
</tbody>
</table>

* \( K \) is based on strength design. Note: 1 in. = 25.4 mm. 12RB20: 12 in. wide; 20 in. deep.
APPENDIX D – PROPOSED CODE CHANGE

Change Submittal:
Subject: Modifications of ACI 318 Code wording and equations that currently permit stress limits in flexural pre-tensioned concrete members as a method of design for pre-stress transfer force.

Current Sections Affected: 18.4.1, 18.4.2, R18.4.1, R18.4.2.

Reason: Permit the strength design theory to be the primary method of design of pretensioned flexural members at prestress transfer. Allowable tensile stress of 6 will be retained if the member is desired to be uncracked in the top fibers at prestress transfer. Another design requirement includes the design of top bonded reinforcement. If the member is allowed to crack, this can be achieved through the strength design approach given here. An approximate alternative to the strength design method is the compressive stress limit formula given in Section 18.4.2.

Proposed Code Change:
Add a new Section 18.4.1 and renumber Section 18.4.1 to Section 18.4.2, Section 18.4.2 to Section 18.4.3 and Section 18.4.3 to Section 18.4.4.

18.4.1 – The requirement of Section 18.4.2, except Section 18.4.2 (a) and (b), may be waived if a strength analysis is performed based on the assumptions that Section 10.2 is satisfactorily completed. For the strength analysis, the required strength \( U \) according to Section 9.2, shall be based on a combination of 1.2\( P_L \) and either 1.2\( M_g \) or 0.8\( M_g \), whichever is more critical, and the strength reduction factor \( \phi \), according to Section 9.3, shall be 0.7. If required, the amount of bonded tension reinforcement placed near the member extreme tension fibers may be obtained using this strength design approach. If the bonded tension reinforcement stress at the strength level is not greater than 60 ksi, the equivalent unfactored load reinforcement stress should be expected to be below the stress that causes unacceptable top fiber cracks.

18.4.1 – Stresses in concrete immediately after prestress transfer (before time dependent prestress losses) shall not exceed the following: 18.4.2 – In lieu of the more accurate strength design approach, the following working stress analysis, using unfactored loads, should give satisfactory results:

(a) Extreme fiber stress in compression at ends of simply supported members
(b) Extreme fiber stress in compression except as permitted in (a)
(c) Extreme fiber stress in tension except as permitted in (c)
(d) Extreme fiber stress in tension at ends of simply supported members

Proposed Commentary Change:
Add a new Sections R 18.4.1 and renumber Section R 18.4.1 to Section R 18.4.2.

R.18.4.1: – Studies in Reference 18.X1, have demonstrated the validity of the strength design method of preten-
sioned concrete members at prestress transfer. The load factor 1.2 applied to the prestress force \( P_1 \), just before prestress transfer, is consistent with the factor used for strength design of post-tensioning anchorage zone. The strength reduction factor \( \phi = 0.7 \) is consistent with that used for tied columns. Both factors may be too conservative for this application as \( P_1 \) is known with high precision and concrete strength is normally tested for prestress transfer. Further, the prestressing reinforcement is in tension and there is no possibility of reinforcement buckling under factored loads, as is the case of reinforcement in tied columns. Additional studies may produce further refinement of these factors.

Reference 18.X1 studies have shown that if the stress in bonded tension reinforcement at factored load is below 0.0021(29000) = 60 ksi, which is the yield point of Grade 60 steel, the corresponding stress at unfactored load level is found to be in the range of 15 to 22 ksi, which is below that causing unacceptable cracking.

Interestingly, it was discovered near the completion of this report that a very similar concept is being promoted in Australia. See Reference 18.X2. The Australian Code, AS 3600 Clause 8.1.4.2, already accepts strength design for prestress transfer in lieu of allowable compressive stress. In Australia, the load factors used are 1.15 for prestress force, and either 1.2 or 0.8 for member self-weight. The strength reduction factor is 0.6. These factors confirm the reasonableness of the factors recommended herein. It is unclear, however, how the tension reinforcement stress due to factored loads is utilized for crack control.

Add a new Section R18.4.2 (a) and (b)

R.18.4.2: (a) and (b) – The formula for allowable compression is equivalent to using 0.7 \( f_{ct}' \) for rectangular and I-shape sections, approximately 0.65 \( f_{ct}' \) for inverted tee and 0.75 \( f_{ct}' \) for double tee members. The proposed coefficients were based on correlation with strength design results for a large number of common pretensioned concrete member shapes and levels of prestress. For post-tensioned concrete members, it is recommended that the compression stress limit of 0.60 \( f_{ct}' \) at ends of member should be retained.

Renumber Commentary R18.4.1(b) and (c) to read:
R.18.4.1(c) and R18.4.1(d)

Add new References 18.X1 and 18.X2:

References