Exceptions of Precast Prestressed Concrete Members to Minimum Reinforcement Requirements

by

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SYNOPSIS

This summary paper presents an overview of PCISFRAD Project No. 2, "Exceptions of Precast, Prestressed Members to Minimum Reinforcement Requirements (of American Concrete Institute Standard ACI 318-83)."

The objectives of this project were to (1) determine provisions in the ACI Building Code which require excessive minimum reinforcement and (2) compile an experience record and recommend appropriate testing to justify modification of these provisions.

To make the study more meaningful, an extensive industry survey of practice among American and Canadian precast producers was undertaken and subsequently analyzed.

The major focus of the investigation was on mass produced precast prestressed concrete members. In particular, the following topics were studied:

1. Shear reinforcement requirements for precast prestressed double tee members.
2. Development length of prestressing strands, including debonded strands, and allowable concrete stresses in pretensioned members.
3. Minimum reinforcement requirements for prestressed concrete flexural members.
4. Minimum reinforcement requirements for precast wall panels.
5. Prestressed walls and columns — minimum prestress level.
6. Horizontal shear transfer in composite concrete flexural members.

The key conclusions and recommendations from the research are presented for each topic.

Note: This summary paper is a condensation 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 (non-supporting firms) and $30.00 to non-PCI Members.

The summary 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.
SCOPE OF RESEARCH

The scope of this investigation was to study the provisions of the ACI Code (ACI 318-83) as related to reinforcement requirements of precast prestressed concrete members and to recommend appropriate changes and/or additions in these provisions.

The investigation was concerned primarily with mass produced elements such as double tees, rather than on usually custom made elements such as spandrel beams. All post-tensioned construction was excluded from the scope of this research. However, precast nonprestressed or nominally prestressed concrete components such as wall panels were included. Composite precast, cast-in-place construction was also a part of this investigation.

Table 1 summarizes the survey results of the performance of double tees. Tables 2 through 5 list the minimum reinforcement requirements and related provisions of the ACI Code for prestressed concrete flexural members, prestressed concrete slabs, precast concrete walls, and prestressed concrete columns, respectively. Close examination of the listed provisions formed a major part of the research.

To effectively carry out this project, substantial information and input from the precast prestressed concrete industry were needed. During the initial phase of this project, a detailed questionnaire was thus prepared. The questionnaire was mailed to about 350 PCI producer members, and to 40 other prestressed concrete producers not currently members of the PCI—all located within the United States and Canada. The questionnaire consisted of separate parts relating to:

1. Double tees
2. Nonprestressed walls
3. Prestressed walls and columns
4. Hollow-core slabs
5. Composite structural elements
6. Torsional reinforcement

Forty-one responses to the survey from American precasters and five additional responses from Canadian manufacturers were received. The responses were thoughtful and provided the research agency with a proper understanding of the industry’s perception of the problems involved. The survey results were thoroughly analyzed by the research agency. The findings from the survey set the direction of this project.

ORGANIZATION OF RESEARCH

Research carried out under this project was focused on the following topics:

1. Shear reinforcement requirements for precast prestressed double tee members.
2. Development length of prestressing strands, including debonded strands, and allowable concrete stresses in pretensioned members.
3. Minimum reinforcement requirements for prestressed concrete flexural members.
4. Minimum reinforcement requirements for precast wall panels.
5. Prestressed walls and columns—minimum prestress level.
6. Horizontal shear transfer in composite concrete flexural members.

The title of each topic is descriptive of the research carried out. Specific code change proposals are made with regard to Topics 1, 3, 4, 5 and 6. Research on Topic 2 provided justification of the current code provisions concerning the items mentioned in the title.

Results of the industry survey on hollow-core slabs were referred to the PCI Hollow-Core Slab Producers Commit-
tee for their consideration. No further material on hollow-core slabs was developed within this project.

The portion of the industry survey dealing with torsional reinforcement covered the same concerns as PCISFRAD Project No. 5 on spandrels. It was thus decided by the Steering Committee for Project No. 2 that the responsibility for further action lay with Project No. 5. As a result, no further material on torsional reinforcement was developed within Project No. 2.

Each of the research topics listed above is separately described in this paper. Conclusions and recommendations emerging from the research on each topic are presented.

1. SHEAR REINFORCEMENT REQUIREMENTS FOR PRECAST PRESTRESSED DOUBLE TEE MEMBERS

Overview of Investigation

The double tee floor or roof slab is the most common among the standard precast prestressed units used for buildings. The minimum reinforcement requirements of the ACI Code, as they apply to precast prestressed double tee units, are thus of concern to the prestressed concrete industry. Of particular concern are the minimum shear reinforcement requirements for double tees.

The shear design requirements of the ACI Code were reviewed as a part of this investigation. The results of the industry survey that were relevant to the shear design of precast prestressed double tee units were thoroughly analyzed. The available literature on the shear strength of double tees, and the results of load tests on double tees, mostly conducted by precast producer members of the industry, were reviewed.

The performance record of precast prestressed double tee members, used in building construction around the country over the last three decades, was examined. The investigations led to certain conclusions and recommendations concerning the minimum shear reinforcement requirements for precast prestressed double tee units.

ACI Code Requirements

Shear design requirements for prestressed concrete flexural members are given in Sections 11.4 and 11.5 of ACI 318-83. At least a certain minimum area of shear reinforcement is to be provided in all prestressed concrete members where the total factored shear force is greater than one-half the shear strength provided by the concrete. However, based on successful performance, the following types of members are exempted from this requirement:

1. Slabs and footings.
2. Concrete joist construction.
3. Beams with a total depth not greater than the largest of 10 in. (254 mm), 2½ times the thickness of the flange, and one-half the web width.

The minimum area of shear reinforcement to be provided in all other cases is to be taken equal to the smaller of the following values:

\[ A_v = 50 \frac{b_w s}{f_u} \]  

\[ A_v = \frac{A_{ps}}{80} \frac{f_{pu}}{f_u} \frac{s}{d} \sqrt{\frac{d}{b_w}} \]

in which

- \( A_{ps} \) = cross-sectional area of prestressing steel
- \( b_w \) = web width
- \( d \) = effective depth (need not be less than 80 percent of total depth)
- \( f_{pu} \) = ultimate tensile strength of prestressing steel
\( f_y \) = yield strength of stirrup steel
\( s \) = spacing of shear reinforcement

Eq. (1) generally requires a greater minimum web steel than Eq. (2); thus, Eq. (2) generally controls. However, it may be applied only if the effective pre-stress force is not less than 40 percent of the tensile strength of the tensioned reinforcement.

The ACI Code contains, in addition, certain restrictions on the maximum spacing of web reinforcement to ensure that any potential diagonal crack will be crossed by at least a minimum amount of web steel. For prestressed members this maximum spacing is not to exceed the smaller of 0.75\( h \) (where \( h \) = total depth) or 24 in. (610 mm). If the value of \( V_n \) (nominal shear strength provided by shear reinforcement) exceeds \( 4\sqrt{f_y b_w d} \), these limits are reduced by one-half.

**Load Tests**

Results of load tests on double tees that did not contain the minimum web reinforcement required by the ACI Code were obtained from the following sources, and thoroughly examined:

2. Inland Concrete Company — 1978, Lincoln, Nebraska (8DT24x54.9 ft with 3 in. composite topping).
4. Stanley Structures — 1975, Denver, Colorado (8DT24x53 ft, 8DT18x48 ft, 8DT12x30 ft, untopped).\(^5\)
5. Stresscon Corporation — 1982, Colorado Springs, Colorado (8DT24x61 ft w/3 in. composite topping).
6. Southern Prestressed Concrete — 1978, Pensacola, Florida (8DT24x63.4 ft, untopped).
7. The Tanner Companies — 1971, Phoenix, Arizona (8DT20x55.6 ft, 8DT16x39 ft, untopped).

(Note: 1 ft = 0.3048 m, 1 in. = 25.4 mm.)

The tests generally showed that flexural failure preceded shear failure even in double tees that did not conform with the minimum shear reinforcement requirements of the Code.

**Performance Record**

The survey questionnaire from the authors to precasters included one question concerning the performance of double tees not containing the Code-required minimum shear reinforcement. The responses, summarized in Table 1, show that the second, third and fourth largest producers responding to the survey produce 100, 95, and 80 percent of their double tees without shear reinforcement. None of them report any significant shear cracking or other distress in their products.

Table 1 lists only 34 precast manufacturers who responded, at least partially, to the investigators’ question about production volume. The table indicates that over 8 million sq ft (740,000 m\(^2\)) of double tees without Code-required minimum web reinforcement are produced annually by the 34 manufacturers, and that the same producers have manufactured (until 1984) nearly 100 million sq ft (9,000,000 m\(^2\)) of such double tees that are in service today. The performance of these double tees has been satisfactory, as can be seen quite clearly from Table 1. Since there are nearly 400 precast manufacturers in the United States and Canada, the volume of double tees without minimum reinforcement in satisfactory service today is probably several hundred million square feet.

**Conclusion and Recommendations**

In view of the evidence accumulated as a result of the investigation described, the following Code change proposal appears to be warranted:

1. Add Item (d) to the existing Section 11.5.5.1 to read as follows: “(d) Simply supported precast, prestressed double
Table 1. Summary results of industry survey—Performance record of double tees without code required minimum reinforcement.

<table>
<thead>
<tr>
<th>Respondent number</th>
<th>Annual production, sq ft</th>
<th>Percent without shear reinforcement</th>
<th>Annual production without web reinforcement, sq ft</th>
<th>Year started producing</th>
<th>Total production up to 1984, sq ft</th>
<th>Total production without web reinforcement, sq ft</th>
<th>Experienced shear cracking</th>
<th>Failures</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Mild%</td>
<td>Intermediate%</td>
<td>Severe%</td>
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<td></td>
<td></td>
<td></td>
<td>x</td>
<td>0</td>
<td>0</td>
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<td>?</td>
<td></td>
<td>1959</td>
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<td>600,000</td>
<td>x</td>
<td>0</td>
<td>0</td>
</tr>
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<td>2,980,000</td>
<td>5</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
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<td></td>
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<td>1</td>
<td>0</td>
<td>0</td>
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<td>80</td>
<td>800,000</td>
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<td>5*</td>
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<td>Never</td>
<td>Never</td>
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<td>2,000,000</td>
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<td>100*</td>
<td>200,000</td>
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<td>2,000,000</td>
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<td>765,000</td>
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<tr>
<td>A130</td>
<td>850,000</td>
<td>90 +*</td>
<td>1984</td>
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<td>10</td>
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<td>100</td>
<td>400,000</td>
<td>1983</td>
<td>600,000</td>
<td>600,000</td>
<td>—</td>
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<tr>
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<tr>
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<td>100*</td>
<td>3,000,000</td>
<td>1975</td>
<td>18,000,000</td>
<td>18,000,000</td>
<td>—</td>
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<tr>
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<td>95</td>
<td>2,375,000</td>
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<td>38,000,000</td>
<td>36,100,000</td>
<td>1*</td>
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<tr>
<td>A180</td>
<td>5,000,000</td>
<td>Very little</td>
<td>—</td>
<td>—</td>
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<td>75</td>
<td>210,000</td>
<td>1969</td>
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<tr>
<td>A190</td>
<td>700,000</td>
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<td>1958</td>
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<tr>
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<td>1964</td>
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</tr>
<tr>
<td>C15</td>
<td>10,000</td>
<td>0</td>
<td>—</td>
<td>—</td>
<td>—</td>
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<td>0</td>
<td>—</td>
<td>—</td>
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<td></td>
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<tr>
<td>C25</td>
<td>400,000</td>
<td>99</td>
<td>396,000</td>
<td>1956</td>
<td>2,850,000</td>
<td>—</td>
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</table>

Total 20,497,000 8,384,700 181,430,000 99,425,500

1 Early experience led us to always provide stirrups. Allowable waiver in ACI Section 11.5.5.2 seldom includes sufficiently “realistic assessment” of settlement, creep, shrinkage, temperature and other unstipulated events.
2 Hairline cracks often present when lightweight mix is used.
3 Infrequent occurrences, not expressible as a percentage.
4 Note: 14 producers manufacture part of their double tees without web reinforcement. Annually, they produce 8,384,700 sq ft (~ 41% of present production by 34 responding producers) without web reinforcing. These same 14 producers have manufactured about 100,000,000 sq ft of double tees without web reinforcement (to 1984).

Metric (SI) conversion factors: 1 ft = 0.3048 m = 304.8 mm; 1 in. = 25.4 mm; 1 sq ft = 0.0929 m².
tee roof or floor members supporting ordinary interior occupancy, such as office, residential, passenger car parking (excluding roofs on which significant snow accumulation can be expected), or retail uses, and loaded in an essentially uniform manner, excluding one-tenth of the span length or 5 ft (1.5 m), whichever is smaller, at either end.

2. Replace the last two sentences of the first paragraph of the Commentary on Section 11.5.5 to read as follows:

"Four types of members are excluded from the minimum shear reinforcement requirement: slabs and footings; floor joists; wide shallow beams; and simply supported precast prestressed double tee roof or floor members (middle 80 percent of span only). Slabs, footings and joists are excluded because there is a possibility of load sharing between weak and strong areas. Precast prestressed double tees are excluded because numerous load tests have shown conclusively that the required ultimate flexural and shear strengths can be developed in such members when minimum shear reinforcement as required by Section 11.5.5.1 is omitted. There is also a long record of satisfactory performance in service of double tees not conforming with the minimum shear reinforcement requirement of Section 11.5.5.1. End shear reinforcement is necessary to guard against accidental damage that can occur during fabrication and handling, and to account for splitting forces. The use of continuity or substantial cantilevers at one or both ends of a beam or the presence of moderate to heavy concentrated loads can increase shear requirements; such situations are excluded from the waiver. Certain parking garage roof decks are also excluded from the waiver because they are subject to special loading from snow removal equipment, snow drifts, etc. Double tee members supporting ordinary interior occupancy would normally be those designed for live load intensities not exceeding 125 psf (6.0 kPa)." Also, when a member is subjected to concentrated loads not contributing more than 10 percent of the required shear strength at the critical section(s), the member may still be considered as essentially uniformly loaded."

The author would like to further recommend that the precast concrete industry consider sponsoring tests of double tees subjected to variable loads representative of those encountered in heavy storage decks, as well as repetitive loads such as those experienced in manufacturing facilities. Such tests are needed before a waiver of minimum reinforcement requirements for double tees used in applications, such as those mentioned, can be sought.

*The reference number obviously will have to change when the suggested part of the paragraph is inserted into the Commentary to the ACI Code.
2. DEVELOPMENT LENGTH OF PRESTRESSING STRANDS, INCLUDING DEBONDED STRANDS, AND ALLOWABLE CONCRETE STRESSES IN PRETENSIONED MEMBERS

Overview of Investigation

The adequacy, realism and/or conservatism of Section 12.9 of ACI 318-83,
entitled “Development of Prestressing Strand,” were examined in this part of
the investigation. In view of responses to the industry survey, particular attention
was paid to the double development length requirement for debonded
prestressing strands (Section 12.9.3 of ACI 318-83).

The Code provisions were evaluated through close examination of all available
test results. Also examined were the allowable concrete stresses of Section
18.4 of ACI 318-83, by tracing the history of this section back to Ref. 21 on
which the very first chapter on prestressed concrete in an ACI Code (1963
dition) was based.

Conclusions

The following conclusions emerged from these studies.

1. The ACI 318-83 equation giving development length requirement for
prestressing strand is based on good experimental authority. Certain investiga-
tors have proposed making the provisions more conservative, while others
have found the requirements adequate. There does not appear to be any com-
pelling basis for any significant change to the current provisions. In the case of
short span members where the full development length required by the ACI
Code cannot be provided, the approach suggested in Ref. 17 may prove to be
useful.

2. The double development length requirement for debonded strand (Section
12.9.3 or ACI 318-83) 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 per-
formed satisfactorily. However, tests on beams with debonded strands using de-
velopment 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.

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 satis-
factory 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 re-
ports of lack of performance. Further modifications do not appear to be war-
ranted 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 $0.60f_{ci}$ to $0.70f_{ci}$ at the ends of simply supported members may
not have an adverse effect on performance. However, this needs to be ver-
ified in carefully conducted tests.

(b) The allowable tensile stresses, immediately after prestress transfer, of
$3 \sqrt{f_{ei}}$ and $6 \sqrt{f_{ei}}$ (0.25 $f_{ci}$ and 0.5 $f_{ci}$) 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 show consistently high values (significantly in ex-
cess of $7.5 \sqrt{f_{ei}}$ (0.6 $f_{ei}$)). A great many
Table 2. Minimum reinforcement requirements and related provisions for prestressed concrete flexural members, extracted from ACI 318-83. *

<table>
<thead>
<tr>
<th>Section</th>
<th>Provision</th>
</tr>
</thead>
<tbody>
<tr>
<td>11.4</td>
<td>Shear strength provided by concrete for prestressed members</td>
</tr>
<tr>
<td>11.5</td>
<td>Shear strength provided by shear reinforcement</td>
</tr>
<tr>
<td>11.5.4</td>
<td>Spacing limits for shear reinforcement</td>
</tr>
<tr>
<td>11.5.5</td>
<td>Minimum shear reinforcement</td>
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<tr>
<td>11.5.5.1</td>
<td>Exemptions for slabs and footings, concrete joist construction, etc.</td>
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<tr>
<td>11.5.5.2</td>
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<td>Minimum reinforcement requirement for nonprestressed or prestressed members</td>
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<td>11.5.5.4</td>
<td>Minimum reinforcement requirement for prestressed members</td>
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<tr>
<td>11.5.5.5</td>
<td>Combined minimum shear and torsional reinforcement requirement for members subject to significant torsion</td>
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<tr>
<td>11.6</td>
<td>Combined shear and torsion strength for nonprestressed members†</td>
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<td>11.6.8</td>
<td>Spacing limits for torsion reinforcement</td>
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<td>12.9</td>
<td>Development of prestressing strand</td>
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<td>17.5</td>
<td>Horizontal shear strength (composite concrete flexural members)</td>
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<td>17.6</td>
<td>Ties for horizontal shear (in composite concrete flexural members)</td>
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<td>18.8</td>
<td>Limits for reinforcement of (prestressed concrete) flexural members</td>
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<td>18.8.3</td>
<td>Minimum amount of prestressed and nonprestressed reinforcement</td>
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<td>18.9</td>
<td>Minimum bonded reinforcement</td>
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<td>18.9.2</td>
<td>Minimum bonded reinforcement for flexural members excluding two-way flat plates</td>
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<td>18.9.4</td>
<td>Minimum length of bonded reinforcement</td>
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</table>

*Provisions for deep beams are not included. †Many of these provisions apply also to prestressed concrete members, some in an amended form, according to current practice.

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 a careful testing program before a relaxation of the current stress limits can be sought.

## 3. MINIMUM REINFORCEMENT REQUIREMENTS FOR PRESTRESSED CONCRETE FLEXURAL MEMBERS

### Overview of Investigation

Section 10.5.1 of ACI 318-83 prescribes a minimum amount of reinforcement for reinforced concrete flexural members. Section 10.5.2 states that: “Alternatively, area of reinforcement provided at every section, positive or negative, shall be at least one-third greater than that required by analysis.

Section 18.8.3 requires that the total amount of prestressed and nonprestressed reinforcement in prestressed concrete flexural members shall be ade-
Table 3. Minimum reinforcement requirements and related provisions for prestressed concrete slabs, extracted from ACI 318-83.

<table>
<thead>
<tr>
<th>Section</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.6</td>
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</tr>
<tr>
<td>7.6.5</td>
<td>Spacing limits for reinforcement in walls and slabs</td>
</tr>
<tr>
<td>7.12</td>
<td>Shrinkage and temperature reinforcement</td>
</tr>
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<td>8.10</td>
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<tr>
<td>8.10.1</td>
<td>Transverse reinforcement in flanges of T-beams</td>
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<td>8.10.2</td>
<td>Shear carried by concrete in two-way prestressed slabs and footings</td>
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<tr>
<td>11.11</td>
<td>Special (shear) provisions for slabs and footings</td>
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<tr>
<td>11.11.2</td>
<td>Design of slab or footing for two-way action</td>
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<td>11.11.2.2</td>
<td>Shear carried by concrete in two-way prestressed slabs and footings</td>
</tr>
<tr>
<td>11.12</td>
<td>Transfer of moments to columns</td>
</tr>
<tr>
<td>11.12.2</td>
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</tr>
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<td>11.12.2.4</td>
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<td>11.12.2.4.2</td>
<td>Shear stress capacity of concrete in two-way prestressed slabs and footings</td>
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<td>18.9</td>
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<tr>
<td>18.9.3</td>
<td>Minimum bonded reinforcement for two-way flat plates</td>
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<tr>
<td>18.9.4</td>
<td>Minimum length of bonded reinforcement</td>
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<tr>
<td>18.12</td>
<td>(Prestressed) slab systems</td>
</tr>
<tr>
<td>18.12.4</td>
<td>Detailing of tendons in two-way banded post-tensioned flat plates</td>
</tr>
</tbody>
</table>

Sufficient to develop a factored load at least 1.2 times the cracking load specified in Section 9.5.2.3, except for flexural members with shear and flexural strength at least twice that required by Section 9.2. The last part of Section 18.8.3, providing the exception, is new in the 1983 edition of the Code.

The safety factor of 2 for prestressed concrete flexural members versus the apparently smaller factor of 4/3 for flexural members of reinforced concrete appeared to require proper explanation or suitable modification, and was the subject of this part of the investigation. The ACI 318 Commentary on Section 18.8.3 provides an explanation for the safety factor of 2. A significant flaw in this explanation was found.

Conclusions and Recommendations

In view of the discussion in Refs. 1 and 22, it is recommended that Section 18.8.3 of ACI 318-83 be modified to read as follows:

"Total amount of prestressed and nonprestressed reinforcement shall be adequate to develop at every section a design flexural strength at least 1.2 times the cracking moment computed on the basis of the modulus of rupture \( f_c \), specified in Section 9.5.2.3, except where the design flexural strength is at least 1.6 times that required by Section 9.2."

The second paragraph of the Commentary on Section 18.8.3 of ACI 318-83 should also be modified to read:

"An exception is added to provide for those cases when the reinforcement required to develop 1.2 times the cracking moment would be excessive. The exception waives the 1.2 times cracking strength requirement for those cases where the design flexural strength provided is at least 1.6 times the flexural strength required by Section 9.2. The exception is similar to the 4/3 factor al-
lowed for nonprestressed members in Section 10.5.2. The required strength increase by a factor of 1.6 was derived by taking the 4/3 factor and modifying it by the ratio of \( f_{pu} \) to \( f_{pu} \) of stress-relieved prestressing tendons: \( 4/3 \times 1/0.85 = 1.6. \)

It should be noted that whereas Section 18.8.3 of ACI 318-83 refers to factored load and cracking load, the proposed modified provision is in terms of flexural strength and cracking moment. It is hoped that this change would add to the clarity of the provision. If all references to shear strength are removed, there would no longer be any need to phrase it in terms of loads anyway.

4. MINIMUM REINFORCEMENT REQUIREMENTS FOR PRECAST WALL PANELS

Overview of Investigation

The minimum reinforcement requirements of the ACI Code for reinforced concrete walls, including precast wall panels, were reviewed in this part of the investigation. Wall panels were categorized into three groups: unreinforced panels (Level 1 walls), walls that are designable by the empirical design procedure of Chapter 14 of the Code (Level 2 walls), and walls that must be designed as compression members by Chapter 10 (Level 3 walls).

Certain differences between precast and cast-in-place walls, that have a bearing on minimum reinforcement requirements for walls, were pointed out. It was also shown that as a result of successful experience with a longstanding practice by many precast manufacturers of using lesser amounts of reinforcement in precast walls than is required by the ACI Code provisions, the PCI Committee on Precast Concrete Bearing Wall Buildings has recommended the use of a minimum reinforcement ratio of 0.001 (0.1 percent) for both vertical and horizontal wall reinforcement.

Spacing of this reinforcement is not to exceed 30 in. (760 mm) for interior walls or 18 in. (460 mm) for exterior walls. Also, PCI’s Manual for Structural Design of Architectural Precast Concrete makes the same recommendations for minimum wall reinforcement based on years of successful use of precast panels with a reinforcement ratio of 0.001. Some relaxation of the current minimum reinforcement requirements are warranted for precast wall panels.

Recommendations

The following Code changes have been suggested in Refs. 1 and 23.

Section 14.3.2, add:

(d) 0.0010 for precast wall panels using bars not larger than #5 (16 mm) with a specified yield strength not less than 60,000 psi (414 MPa) or using welded wire fabric (smooth or deformed) not larger than W31 or D31 (16 mm).

Section 14.3.3, add:

(d) 0.0010 for precast wall panels not exceeding 18 ft (5.5 m) in length using bars not larger than #5 (16 mm) with a specified yield strength not less than 60,000 psi (414 MPa) or using welded wire fabric (smooth or deformed) not larger than W31 or D31 (16 mm).

(e) 0.0015 for precast wall panels exceeding 18 ft (5.5 m) in length using bars not larger than #5 (16 mm) with a specified yield strength not less than 60,000 psi (414 MPa) or using welded wire fabric (smooth or deformed) not larger than W31 or D31 (16 mm).

Add a new section:

14.3.5.1 — In precast walls vertical and horizontal reinforcement shall not be spaced farther apart than 36 in. (915 mm).
Table 4. Minimum reinforcement requirements and related provisions for precast walls, extracted from ACI 318-83.

<table>
<thead>
<tr>
<th>Section</th>
<th>Description</th>
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<tbody>
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</tr>
<tr>
<td>11.10</td>
<td>Special (shear) provisions for walls</td>
</tr>
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<td>14.3</td>
<td>Minimum reinforcement (for walls)</td>
</tr>
<tr>
<td>15.8</td>
<td>Transfer of force at base of column, wall, or reinforced pedestal</td>
</tr>
<tr>
<td>15.8.3</td>
<td>Transfer of force at base of precast column or wall</td>
</tr>
<tr>
<td>15.8.3.2</td>
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</tr>
<tr>
<td>18.11</td>
<td>Compression members — Combined flexure and axial loads</td>
</tr>
<tr>
<td>18.11.2</td>
<td>Limits for reinforcement of prestressed compression members</td>
</tr>
<tr>
<td>18.11.2.3</td>
<td>Walls with average prestress equal to or greater than 225 psi (1.6 MPa)</td>
</tr>
</tbody>
</table>

Table 5. Minimum reinforcement requirements and related provisions for prestressed concrete columns, extracted from ACI 318-83.

<table>
<thead>
<tr>
<th>Section</th>
<th>Description</th>
</tr>
</thead>
<tbody>
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<td>7.10</td>
<td>Lateral reinforcement for compression members</td>
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<td>10.9</td>
<td>Limits for reinforcement of compression members</td>
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<tr>
<td>15.8</td>
<td>Transfer of force at base of column, wall, or reinforced pedestal</td>
</tr>
<tr>
<td>15.8.3</td>
<td>Transfer of force at base of precast column or wall</td>
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<tr>
<td>15.8.3.1</td>
<td>Connection between precast column and connecting member</td>
</tr>
<tr>
<td>18.11</td>
<td>Compression members — Combined flexure and axial loads</td>
</tr>
<tr>
<td>18.11.2</td>
<td>Limits for reinforcement of prestressed compression members</td>
</tr>
<tr>
<td>18.11.2.2</td>
<td>Columns with average prestress ≥ 225 psi (1.6 MPa)</td>
</tr>
</tbody>
</table>

5. PRESTRESSED WALLS AND COLUMNS — MINIMUM PRESTRESS LEVEL

Overview of Investigation

The 225 psi (1.6 MPa) effective prestress limit of the ACI Code* that divides prestressed concrete walls and columns from those that are considered nonprestressed was critically examined in this part of the investigation.1,26 It was shown that the strength capabilities of a wall containing the minimum vertical reinforcement required by the Code are more than matched by those of a wall with an effective prestress level of 100 psi (0.7 MPa). It was pointed out that the minimum wall reinforcement (horizontal as well as vertical), as required by Section 14.3, is provided primarily for control of cracking due to shrinkage and temperature stresses, although it can be only rarely that the shrinkage and temperature stresses in the vertical direction would exceed the stresses due to gravity loads, causing net tension.

In Section 7.12 of ACI 318-83, prestressing tendons proportioned to pro-
vide a minimum average compressive stress of 100 psi (0.7 MPa) on gross concrete area, using effective prestress after losses, with the spacing of tendons not exceeding 54 in. (1370 mm), are considered sufficient as shrinkage and temperature reinforcement. Conservatively, assuming that an additional 25 psi (0.2 MPa) of prestress would compensate for the loss of prestressing due to added creep resulting from wall dead weight and other superimposed gravity loads, it was felt that an effective prestress level of 125 psi (0.9 MPa) should be sufficient for control of cracking due to shrinkage and temperature stresses.

In the case of columns it was shown that at low axial load levels, moment capacities provided by 0.5 percent mild steel reinforcement are barely matched by those provided by a 225 psi (1.6 MPa) prestress level. Since the loss of prestress due to added creep resulting from column dead weight and superimposed gravity loads is not accounted for in the effective prestress level, there definitely did not appear to be any justification for reducing the 225 psi prestress level.

No increase in the 225 psi (1.6 MPa) effective prestress level appeared to be warranted either, in view of satisfactory experience over many years with the above limit. Nonrectangular walls, rectangular walls that are essentially concentrically loaded and carry axial load levels higher than those allowed by Eq. (14-1) of the Code, and rectangular walls carrying significant out-of-plane bending moments must all be designed as compression members by Chapter 10 of the Code. It was felt that the 225 psi effective (vertical) prestress limit should apply to all these walls in order for them to qualify for exemption from the minimum reinforcement requirements of Section 14.3.

**Recommendations**

In view of the discussion and evidence presented in Refs. 1 and 26, the following recommendations for changes in Section 18.11.2 of ACI 318-83 can be made:

Section 18.11.2.1 — Members with . . . Section 14.3 for walls, except as provided under Section 18.11.2.3. (The italicized words indicate a new addition.)

Section 18.11.2.3 — For walls that may be designed by the empirical design method of Section 14.5, minimum reinforcement required by Section 14.3 may be waived in favor of an average prestress $f_{pc}$ equal to or greater than 125 psi (0.9 MPa).

It should be noted that Section 18.11.2.3 suggested above is new. Walls that are not designable by Section 14.5, and that have an average prestress $f_{pc}$ greater than or equal to 225 psi (1.6 MPa), are exempt from the minimum reinforcement requirements of Section 14.3 anyway, under the current Section 18.11.2.1. The phrase requiring analysis showing adequate strength and stability appears to be unnecessary, and belongs, as it has been placed, in Section 14.2.7.
6. HORIZONTAL SHEAR TRANSFER IN COMPOSITE CONCRETE FLEXURAL MEMBERS

Overview of Investigation

This part of the investigation considered the various types of composite precast/cast-in-place concrete flexural members in use in the construction industry today, and certain aspects of their design.\textsuperscript{1,27} The ACI Code\textsuperscript{2} provisions governing the transfer of horizontal shear stresses across the interface between the precast and cast-in-place portions of such members were reviewed. Interpretations of these provisions were developed, and certain modifications suggested.

Recommendations

1. Add the following Section 17.5.2.1, and renumber Sections 17.5.2.2 through 17.5.2.4 of ACI 318-83\textsuperscript{2} accordingly:

   \textbf{17.5.2.1} — When contact surfaces are clean, free of laitance, but not intentionally roughened, shear strength $V_{nh}$ shall not be taken greater than 40 $b_r d$ in pounds.

2. Add a new paragraph to the Commentary\textsuperscript{6} on Section 17.5.2, pointing out that the use of an effective friction coefficient $\mu_r$, as proposed in Ref. 28, and as adopted in the PCI Design Handbook, is quite appropriate for the determination of the area of shear friction reinforcement, when required, across the interface between the precast and cast-in-place portions of a composite flexural member. Such use is in fact sanctioned by Section 11.7.3 of the ACI Code.\textsuperscript{2}

   It may be noted that Recommendation 1 is based on a provision included in the National Standard of Canada, CAN3-A23.3-M77.\textsuperscript{30}

ACKNOWLEDGMENT

The author wishes to acknowledge the most valuable contributions of Mark Fintel, consulting structural engineer, Chicago, Illinois.

* * *
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2. ACI Committee 318, “Building Code Requirements for Reinforced Concrete (ACI 318-83),” American Concrete Institute, Detroit, Michigan, 1983, 111 pp.


6. ACI Committee 318, “Commentary on Building Code Requirements for Reinforced Concrete (ACI 318R-83),” American Concrete Institute, Detroit, Michigan, 1983, 155 pp.


20. Horn, D. G., and Preston, H. K. (for PCI Committee on Bridges), “Use of De-


* * *

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