PCI Standard Design Practice

Prepared by

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This document represents the final version of the report "PCI Standard Design Practice," prepared jointly by the PCI Technical Activities Council and the PCI Committee on Building Code, which was published in the July-August 1996 issue of the PCI JOURNAL. The purpose in publishing the initial report was to solicit discussion from the membership and design profession. This discussion has been received and reviewed. The reader comments, together with the Committee closure, appear in this issue of the PCI JOURNAL immediately following this report.

STANDARD DESIGN PRACTICE

Precast, prestressed concrete design is based on the provisions of the ACI Building Code. In most cases, these provisions are followed literally. Occasionally, though, there is disagreement as to the interpretation of some sections of the ACI Code. Also, in some situations, research may support other design and construction practices. In such cases, strict compliance with the ACI provisions can cause design, production and performance problems that may unnecessarily increase the cost of a structure or may actually result in an inferior product.

In most cases, the practices reported herein are supported by many years of good performance and/or research. Members of the PCI Technical Activities Council and the PCI Committee on Building Code, along with other experienced precast concrete design engineers, have identified these code provisions as detailed herein. The list of provisions represents a starting point for discussion, and complete agreement with the positions taken is not expected. Nevertheless, a listing of the design practices followed by a majority of precast concrete design engineers is anticipated to be helpful in producing safe, economical precast, prestressed concrete structures by minimizing conflict among the members of the design and construction team.

This list of provisions is based on ACI 318-95, and the numbers refer to sections in that document. References to the PCI Design Handbook are to the Fourth Edition. Excerpts from ACI 318-95 are reprinted here with permission of the American Concrete Institute, Farmington Hills, Michigan.

ACI CODE	PCI PRACTICE
1.1.5 — This code does not govern design and installation of portions of concrete piles and drilled piers embedded in ground.	**1.1.5** — Prestressed concrete piles are normally designed using the PCI publication "Recommended Practice for Design, Manufacture and Installation of Prestressed Concrete Piling," PCI JOURNAL, V. 38, No. 2, March-April 1993, pp. 15-41. (Ref. Handbook Section 4.7.6)
1.2.1 (e) — Size and location of all structural elements and reinforcement.	**1.2.1 (e)** — "reinforcement" in this case does not refer to prestressing steel. In precast concrete members, reinforcement may be shown only on the shop drawings. (Ref. Handbook Section 10.3.3.2)
1.2.1 (g) — Magnitude and location of prestressing forces.	**1.2.1 (g)** — For pretensioned concrete products, the prestressing design and detailing may be left to an engineer employed or retained by the manufacturer. (Ref. Handbook Sections 10.3 and 10.4)
1.2.2 — Calculations pertinent to design shall be filed with the drawings when required by the building official. Analyses and designs using computer programs shall be permitted provided design assumptions, user input, and computer-generated output are submitted. Model analysis shall be permitted to supplement calculations.	**1.2.2** — Product calculations and frequently other items such as connections are usually done by the manufacturer's engineer. They are then submitted to the Engineer or Architect of Record, who is responsible for filing these documents with the building official. (Ref. Handbook Sections 10.3 and 10.4)
3.5.2 — Welding of reinforcing bars shall conform to "Structural Welding Code - Reinforcing Steel," ANSI/AWS D1.4 of the American Welding Society. Type and location of welded splices and other required welding of reinforcing bars shall be indicated on the design drawings or in the project specifications. ASTM reinforcing bar specifications, except for ASTM A 706, shall be supplemented to require a report of material properties necessary to conform to the requirements in ANSI/AWS D1.4.	**3.5.2** — A significant amount of connection field welding is common in precast concrete construction. The American Welding Society (AWS) and American Institute of Steel Construction (AISC) recommendations are generally followed, with some modifications as shown in the PCI Design Handbook and the PCI manual "Design and Typical Details of Connections for Precast and Prestressed Concrete." Other connection devices such as welded headed studs and deformed bar anchors are also shown in these publications. Special precaution is necessary when welding of stainless steel reinforcing bars or plates is used. (Ref. Handbook Section 6.5.1)
4.4.1 — For corrosion protection of reinforcement in concrete, maximum water soluble chloride ion concentrations in hardened concrete at ages from 28 to 42 days contributed from the ingredients including water, aggregates, cementitious materials, and admixtures shall not exceed the limits of Table 4.4.1. When testing is performed to determine water soluble chloride ion content, test procedures shall conform to ASTM C 1218.	**4.4.1** — Calcium chloride or other admixtures containing chlorides are rarely used in precast concrete, and never in prestressed concrete, as required in Section 3.6.3. The requirements of this section regarding prestressed concrete are assumed to be met when all materials used in the concrete meet the appropriate ASTM specifications. See report by Donald W. Pfeifer, J. R. Landgren, and William Perenchio, "Concrete, Chlorides, Cover and Corrosion," PCI JOURNAL, V. 31, No. 4, July-August 1986, pp. 42-53. (Ref. Handbook Section 1.3.4)
5.11.3.2 — Accelerated curing shall provide a compressive strength of the concrete at the load stage considered at least equal to required design strength at that load stage.	**5.11.3.2** — The Commentary states "... the elastic modulus, E_c, of steam-cured specimen may vary from that of specimens moist-cured at normal temperatures." It is, however, most common for the ACI equation to be used to calculate E_c even when accelerated curing is used. Some producers may recommend other values based on testing. (Ref. Handbook Section 1.3.1.4) Also note that curing by direct exposure to steam is seldom used in precasting plants.
7.5.2 — Unless otherwise specified by the engineer, reinforcement, prestressing tendons, and prestressing ducts shall	**7.5.2** — Precast concrete products will normally conform to PCI tolerance standards specified in PCI MNL 116, and

be placed within the following tolerances.

Chapter 8 of the PCI Design Handbook. Closer tolerances should not be specified except for special situations. (Ref. Handbook Section 8.2.4)

7.6.7.1 — Clear distance between pretensioning tendons at each end of a member shall be not less that $4d_b$ for wire, nor $3d_b$ for strands. See also 3.3.2. Closer vertical spacing and bundling of tendons shall be permitted in the middle portion of a span.

7.6.7.1 — 2 in. (51 mm) spacing of strands is typically used for strands up to 0.6 in. (15 mm) in diameter. Tests on bridge beams at the University of Texas at Austin have shown no negative effects. See report by Bruce W. Russell and Ned H. Burns, "Measured Transfer Lengths of 0.5 and 0.6 in. Strands in Pretensioned Concrete," PCI JOURNAL, V. 41, No. 5, September-October 1996. With the $^3/_4$ in. (19 mm) maximum aggregate size used on most products, consolidation of concrete has not been a problem.

7.7.2 — Precast concrete (manufactured under plant control conditions)

The following minimum concrete cover shall be provided for reinforcement:

(a) Concrete exposed to earth or weather:

Wall panels:

No. 14 and No. 18 bars . . . . $1^1/_2$

No. 11 bar and smaller . . . . $^3/_4$

Other members:

No. 14 and No. 18 bars 2

No. 6 through No. 11 bars . . . . $1^1/_2$

No. 5 bar, W31 or D31 wire, and smaller . . . . $1^1/_4$

(b) Concrete not exposed to weather or in contact with ground:

Slabs, walls, joists:

No. 14 and No. 18 bars . . . . $1^1/_4$

No. 11 bar and smaller . . . . $^5/_8$

Beams, columns:

Primary reinforcement . . . . d_b but not less than $^5/_8$ and need not exceed $1^1/_2$

Ties, stirrups, spirals . . . . $^3/_8$

Shells, folded plate members:

No. 6 bar and larger . . . . $^5/_8$

No 5 bar, W31 or D31 wire, and smaller . . . . $^3/_8$

7.7.2 — When bars are welded to plates, the cover may be somewhat less in the vicinity of the plate. (Ref. Handbook Table 1.3.5)

7.7.3 — Prestressed concrete

7.7.3.1 — The following minimum concrete cover shall be provided for prestressed and nonprestressed reinforcement, ducts, and end fittings, except as provided in 7.7.3.2 and 7.7.3.3:

(a) Concrete cast against and permanently exposed to earth 3

7.7.3 — For precast, prestressed concrete, the provisions of Section 7.7.2 take precedence over Section 7.7.3, and prestressing steel cover requirements are the same as bars of the same diameter. Wythes, 2 in. (50 mm) thick, are frequently used in sandwich wall panels exposed to weather. Strand, $^3/_8$ in. (9.5 mm) in diameter, is most commonly used. Architects should use caution in specifying reveals in thin wythes. A minimum cover of $^3/_4$ in. (19 mm) behind the reveal is recommended. Architectural precast concrete, where appearance is very critical, may

(b) Concrete exposed to earth or weather:
- Wall panels slabs, joists 1
- Other members 1½

(c) Concrete not exposed to weather or in contact with ground:
- Slabs, walls, joists .... ¾
- Beams, columns:
  - Primary reinforcement 1½
  - Ties, stirrups, spirals 1
- Shells, folded plate members:
  - No. 5 bar, W31 or D31 wire, and smaller ... ⅜
  - Other reinforcement .... d_b but not less than ¾

require special consideration. (Ref. Handbook Table 1.3.5) For further information on sandwich wall panels, see "State-of-the-Art of Precast/Prestressed Sandwich Wall Panels" by the PCI Committee on Precast Sandwich Wall Panels in the March-April and May-June 1997 PCI JOURNALs.

7.7.3.2 — For prestressed concrete members exposed to earth, weather, or corrosive environments, and in which permissible tensile stress of 18.4.2(c) is exceeded, minimum cover shall be increased 50 percent.

7.7.3.2 — "Exposed to weather" is interpreted to *not* include double tee stems in parking garages. Side cover of the prestressing and non-prestressing steel may marginally not meet this requirement. Studies by Robert Mast and Donald Pfeifer have indicated that corrosion in prestressing strands is no greater problem than in non-prestressed reinforcement. [Ref. Handbook Table 1.3.5 Footnote (2)]

7.10.3 — It shall be permitted to waive the lateral reinforcement requirements of 7.10, 10.16, and 18.11 where tests and structural analysis show adequate strength and feasibility of construction.

7.10.3 — Section 7.10.3 waives minimum lateral ties with "tests and calculations. . ." Section 18.11.2.3 specifically excludes prestressed walls from lateral reinforcement requirements. (Ref. Handbook Example 4.7.1)

7.10.4 — Spirals

Spiral reinforcement for compression members shall conform to 10.9.3 and to the following:

7.10.4.1 — Spirals shall consist of evenly spaced continuous bar or wire of such size and so assembled to permit handling and placing without distortion from designed dimensions.

7.10.4 — Precast, prestressed concrete columns frequently use continuously wound rectangular wire for lateral reinforcement. Section 7.10.4.2 specifically applies to only cast-in-place construction, so the minimum size requirements do not apply. The usual practice is to design such columns as tied columns under Section 18.11.2.2, with the wire sized and spaced to provide an area equal to the minimum requirement for ties. There are several research reports to support reduced tie requirements for prestressed columns. For further information, see report by PCI Prestressed Concrete Columns Committee, "Recommended Practice for the Design of Prestressed Concrete Columns and Walls," PCI JOURNAL, V. 33, No. 4, July-August 1988, pp. 56-95.

7.13.3 — For precast concrete construction, tension ties shall be provided in the transverse, longitudinal, and vertical directions and around the perimeter of the structure to effectively tie elements together. The provisions of 16.5 shall apply.

7.13.3 — Methods of achieving structural integrity previously published in the PCI Design Handbook are now codified in Chapter 16. (Ref. Handbook Section 3.10)

8.3.2 — Except for prestressed concrete, approximate methods of frame analysis shall be permitted for buildings of usual types of construction, spans, and story heights.

8.3.2 — The intent of this section is to not allow Section 8.3.3 to be used for prestressed concrete framing. Approximate (e.g., "portal") methods are sometimes used to design precast "light walls" in parking structures. (Ref. Handbook Section 3.8.6)

8.10.2 — Width of slab effective as a T-beam flange shall not exceed one-quarter of the span length of the beam, and the effective overhanging flange width on each side of the web shall not exceed:

8.10.2 — Although Section 18.1.3 excludes this section, eight times the slab thickness is often used as a guide for determining the topping width to be used in designing composite beams. Thin flange members are commonly designed in-

(a) eight times the slab thickness, and

(b) one-half the clear distance to the next web.

cluding the entire flange width in the compression block. (Ref. Handbook Examples 4.2.6 and 4.3.6)

9.2.3 — If resistance to specified earthquake loads or forces E are included in design, load combinations of 9.2.2 shall apply, except that $1.1E$ shall be substituted for W.

9.2.3 — When the project is controlled by the Standard Building Code (SBC) or the BOCA National Building Code:

$$U = (1.1 + 0.5A_v)D + \text{Floor Live} + (0.7)\text{Snow} + E$$

or

$$U = (0.9 - 0.5A_v)D + E$$

(A_v is a coefficient that varies geographically)

Note that this means that seismic forces calculated by the methods in the above model codes are factored (ultimate) forces. (Ref. Handbook Example 3.11.9)

9.2.7 — Where structural effects T of differential settlement, creep, shrinkage, expansion of shrinkage-compensating concrete, or temperature change are significant in design, required strength U shall be at least equal to

$$U = 0.75(1.4D + 1.4T + 1.7L) \qquad (9\text{-}5)$$

but required strength U shall not be less than

$$U = 1.4(D + T) \qquad (9\text{-}6)$$

Estimations of differential settlement, creep, shrinkage, expansion of shrinkage-compensating concrete, or temperature change shall be based on a realistic assessment of such effects occurring in service.

9.2.7 — It should be emphasized that structural effects, T, are *not* to be considered simultaneously with wind or earthquake forces. (Ref. Handbook Section 3.3) Structural effects of T need only be considered when the structural element is restrained and can produce internal forces as a result of T.

9.5 — Control of deflections

9.5.4 — Prestressed concrete construction

9.5.4.1 — For flexural members designed in accordance with provisions of Chapter 18, immediate deflection shall be computed by usual methods or formulas for elastic deflections, and the moment of inertia of the gross concrete section shall be permitted to be used for uncracked sections.

9.5.4.2 — Additional long-term deflection of prestressed concrete members shall be computed taking into account stresses in concrete and steel under sustained load and including effects of creep and shrinkage of concrete and relaxation of steel.

9.5.4.3 — Deflection computed in accordance with 9.5.4.1 and 9.5.4.2 shall not exceed limits stipulated in Table 9.5(b).

9.5.4 — Deflections are always calculated for prestressed concrete members. Calculations will usually include both instantaneous and long-term camber and dead and live load deflection. The Engineer or Architect of Record will determine if this meets requirements, e.g., Table 9.5(b), as satisfactory performance may depend on many non-structural considerations. (Ref. Handbook Section 4.6 and Table 4.6.1)

10.4.1 — Spacing of lateral supports for a beam shall not exceed 50 times the least width b of compression flange or face.

10.4.1 — The spans of non-bearing spandrels on parking structures have frequently exceeded 50 times the width of the top of the member, and no problems have been observed. This is undoubtedly because they typically carry only their own weight, which, of course, is concentric (see ACI 318 Commentary to this section). Where lateral (bumper) loads are applied to the spandrel, lateral supports at mid height of the spandrel into the deck are typical.

10.6.4 — When design yield strength f_y for tension reinforcement exceeds 40,000 psi, cross sections of maximum positive and negative moment shall be so proportioned that the quantity z given by

$$z = f_s \sqrt[3]{d_c A} \quad (10\text{-}5)$$

does not exceed 175 kips/in. for interior exposure and 145 kips/in. for exterior exposure. Calculated stress in reinforcement at service load f_s (kips/in.^{2}) shall be computed as the moment divided by the product of steel area and internal moment arm. Alternatively, it shall be permitted to take f_s as 60 percent of specified yield strength f_y.

10.6.4 — Note that Section 10.6 is specifically excluded for prestressed concrete (Section 18.1.3). (Ref. Handbook Section 4.2.2.1 and Table 4.2.1)

10.9.3 — Ratio of spiral reinforcement ρ_s shall be not less than the value given by

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_y} \quad (10\text{-}6)$$

where f_y is the specified yield strength of spiral reinforcement but not more than 60,000 psi.

10.9.3 — See discussion of Sections 7.10.4 and 18.11.2.2.

10.10 — Slenderness effects in compression members

10.10.1 — Except as allowed in 10.10.2, the design of compression members, restraining beams, and other supporting members shall be based on the factored forces and moments from a second-order analysis considering material nonlinearity and cracking, as well as the effects of member curvature and lateral drift, duration of the loads, shrinkage and creep, and interaction with the supporting foundation. The dimensions of each member cross section used in the analysis shall be within 10 percent of the dimensions of the members shown on the design drawings or the analysis shall be repeated. The analysis procedure shall have been shown to result in prediction of strength in substantial agreement with the results of comprehensive tests of columns in statically indeterminate reinforced concrete structures.

10.10.2 — As an alternate to the procedure prescribed in 10.10.1, it shall be permitted to base the design of compression members, restraining beams, and other supporting members on axial forces and moments from the analyses described in 10.11.

10.10 — The PCI Design Handbook, Chapter 3, addresses the application of these sections to precast and prestressed columns. (Ref. Handbook Sections 3.5.1, 3.5.2 and 4.7.2)

11.1.3.2 — For prestressed members, sections located less than a distance $h/2$ from face of support shall be permitted to be designed for the same shear V_u as that computed at a distance $h/2$.

11.1.3.2 — In beams with loads applied near the bottom, such as L-beams or inverted tees, "h" is taken as the depth of the ledge. (Ref. Handbook Section 4.3)

ACI CODE	PCI PRACTICE
11.5.5 — Minimum shear reinforcement **11.5.5.1** — A minimum area of shear reinforcement shall be provided in all reinforced concrete flexural members (prestressed and nonprestressed) where factored shear force V_u exceeds one-half the shear strength provided by concrete ϕV_c, except: (a) Slabs and footings (b) Concrete joist construction defined by 8.11 (c) Beams with total depth not greater than 10 in., $2^1/_2$ times times of flange, or $^1/_2$ the width of web, whichever is greatest.	**11.5.5** — If V_u is less than ϕV_c, shear reinforcement is omitted in prestressed double tees. A nominal minimum is provided for 5 to 10 ft (1.5 to 3 m) from the ends. This is based on research by Alex Aswad and George Burnley, "Omission of Web Reinforcement in Prestressed Double Tees," PCI JOURNAL, V. 34, No. 2, March-April 1989, pp. 48-65. The approach is permitted by Section 11.5.5.2. (Ref. Handbook Section 4.3 and 4.3.4)
11.6 — Design of torsion	**11.6** — Torsion design has typically been done using the Zia-McGee (PCI Design Handbook, 2nd Edition) or Zia-Hsu (4th Edition) methods. Most computer programs may not yet be updated to ACI 318-95. The reinforcement requirements are similar for any of the methods. Note that concrete torsion strength T_c is no longer included in the new torsion design method. (Ref. Handbook Section 4.4)
11.7.3 — A crack shall be assumed to occur along the shear plane considered. The required area of shear-friction reinforcement A_{vf} across the shear plane shall be designed using either 11.7.4 or any other shear transfer design methods that result in prediction of strength in substantial agreement with results of comprehensive tests.	**11.7.3** — The "effective shear-friction" method described in the PCI Design Handbook is most often used. Use is permitted under Section 11.7.3. (Ref. Handbook Section 6.7)
11.9.3.2.1 — For normal weight concrete, shear strength V_n shall not be taken greater than $0.2 f_c' b_w d$ nor $800 b_w d$ in pounds.	**11.9.3.2.1** — The PCI Design Handbook allows V_n up to $1000 b_w d$. This is consistent with the "effective shear-friction" approach when concrete strengths of 5000 psi (34 MPa) and greater are used. (Ref. Handbook Table 6.7.1)
11.9.3.4 — Reinforcement A_n to resist tensile force N_{uc} shall be determined from $N_{uc} < \phi A_n f_y$. Tensile force N_{uc} shall not be taken less than $0.2 V_u$ unless special provisions are made to avoid tensile forces. Tensile force N_{uc} shall be regarded as a live load even when tension results from creep, shrinkage, or temperature change.	**11.9.3.4** — Bearing pads are used to "avoid tensile forces." The PCI Design Handbook suggests that a value of N_{uc} which will cause the pad to slip is the maximum that can occur, or, alternatively, a value of $0.2 V_{udead}$ is used as a guide. (Ref. Handbook Sections 6.3, 6.8, 6.9 and 6.11)
11.9.6 — At front face of bracket or corbel, primary tension reinforcement A_s shall be anchored by one of the following: (a) by a structural weld to a transverse bar of at least equal size; weld to be designed to develop specified yield strength f_y of A_s bars; (b) by bending primary tension bars A_s back to form a horizontal loop; or (c) by some other means of positive anchorage.	**11.9.6** — Frequently, front face anchorage is by welding to an angle or a plate with vertical anchors. This is permitted by Section 11.9.6(c). (Ref. Handbook Section 6.11)
11.9.7 — Bearing area of load on bracket or corbel shall not project beyond straight portion of primary tension bars A_s, nor project beyond interior face of transverse anchor bar (if one is provided).	**11.9.7** — If primary tension bars are anchored by welding (Section 11.9.6), the bearing area can be considered to extend to the exterior face of the anchoring bar or plate. This section is not typically applied to beam ledges, where ledge reinforcement is typically anchored by bending bars near the front face. Research sponsored by PCI Specially Funded Research and Development Project No. 5, "Design of Spandrel

Beams," addressed this issue, and found that placement of bars is critical. (Ref. Handbook Section 6.11)

11.10.9 — Design of shear reinforcement for walls

11.10.9.1 — Where factored shear force V_u exceeds shear strength ϕV_c, horizontal shear reinforcement shall be provided to satisfy Eq. (11-1) and (11-2), where shear strength V_s shall be computed by

$$V_s = \frac{A_v f_y d}{s_2} \qquad (11\text{-}33)$$

where A_v is area of horizontal shear reinforcement within a distance s_2 and distance d is in accordance with 11.10.4. Vertical shear reinforcement shall be provided in accordance with 11.10.9.4.

11.10.9 — Sections 11.10.9.2 through 11.10.9.4 apply only when the in-plane shear, $V_u > \phi V_c$, as described in 11.10.9.1. Otherwise, minimum reinforcement required by Section 16.4.2 applies (0.001 times the gross cross-sectional area).

12.5.1 — Development length l_{dh}, in inches, for deformed bars in tension terminating in a standard hook (see 7.1) shall be computed as the product of the basic development length l_{hb} of 12.5.2 and the applicable modification factor or factors of 12.5.3, but l_{dh} shall not be less than $8d_b$ nor less than 6 in.

12.5.1 — Bars in beam ledges are assumed to be developed with a hook, even when the straight portion is less than 6 in. (152 mm), measured to the stem face. See the research project referenced in the Commentary of ACI 318 Section 11.9.7. (Ref. Handbook Section 6.14.2)

12.9 — Development of prestressing strand

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

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

where d_b is strand diameter in inches, and f_{ps} and f_{se} are expressed in kips/in.^{2}

12.9 — The provisions of this section are normally followed in the design of prestressed concrete members. Quality control measures are essential. See Buckner, C. Dale, "A Review of Strand Development Length for Pretensioned Concrete Members," PCI JOURNAL, V. 40, No. 2, March-April 1995, pp. 84-105; plus discussion in March-April 1996 PCI JOURNAL, pp. 112-127. See also special report by Donald R. Logan, "Acceptance Criteria for Bond Quality of Strand for Pretensioned Prestressed Concrete Applications" (March-April 1997 PCI JOURNAL).

12.9.2 — Limiting the investigation to cross sections nearest each end of the member that are required to develop full design strength under specified factored loads shall be permitted.

12.9.2 — When debonded strands are used, other sections may be more critical. See Martin, L., and Korkosz, W., "Strength of Prestressed Concrete Members at Sections Where Strands Are Not Fully Developed," PCI JOURNAL, V. 40, No. 5, September-October 1995, pp. 58-66. (Ref. Handbook Section 4.2.3)

12.11.1 — At least one-third the positive moment reinforcement in simple members and one-fourth the positive moment reinforcement in continuous members shall extend along the same face of member into the support. In beams, such reinforcement shall extend into the support at least 6 in.

12.11.1 — Does not apply to precast construction. Excluded by Section 16.6.2.3.

12.13.2.4 — For each end of a single leg stirrup of welded plain or deformed wire fabric, two longitudinal wires at a minimum spacing of 2 in. and with the inner wire at least the greater of $d/4$ or 2 in. from mid depth of member $d/2$. Outer longitudinal wire at tension face shall not be farther from the face than the portion of primary flexural reinforcement closest to the face.

12.13.2.4 — Fig. R12.13.2.4 shows how WWF is used as shear reinforcement in double tee stems. For further information, see TAC's Joint PCI/WRI Ad Hoc Committee on Welded Wire Fabric for Shear Reinforcement, "Welded Wire Fabric for Shear Reinforcement," PCI JOURNAL, V. 25, No. 4, July-August 1980, pp. 32-36.

ACI CODE	PCI PRACTICE
14.3 — Minimum reinforcement	**14.3** — Minimum reinforcement for precast walls is specified in Section 16.4.2.
14.6.1 — Thickness of nonbearing walls shall not be less than 4 in., nor less than $^1/_{30}$ the least distance between members that provide lateral support.	**14.6.1** — Minimum thickness is not applicable to prestressed walls. See Section 18.1.3.
15.8.3.1 — Connection between precast columns or pedestals and supporting members shall meet the requirements of 16.5.1.3(a).	**15.8.3.1** — Note reference to Chapter 16.
16.2.4 — In addition to the requirements for drawings and specifications in 1.2, the following shall be included in either the contract documents or shop drawings: (a) Details of reinforcement, inserts and lifting devices required to resist temporary loads from handling, storage, transportation, and erection. (b) Required concrete strength at stated ages or stages of construction.	**16.2.4** — Connection design is typically a part of the precast contract and connection forces are typically developed by the precast engineer, or sometimes listed on the Contract Drawings. (Ref. Handbook Sections 10.3 and 10.4)
16.6.2.2 — Unless shown by test or analysis that performance will not be impaired, the following minimum requirements shall be met: (a) Each member and its supporting system shall have design dimensions selected so that, after consideration of tolerances, the distance from the edge of the support to the end of the precast member in the direction of the span is at least $^1/_{180}$ of the clear span l, but not less than: For solid or hollow-core slabs. . . . . 2 in. For beams or stemmed members . . . . 3 in. (b) Bearing pads at unarmored edges shall be set back a minimum of $^1/_2$ in. from the face of the support, or at least the chamfer dimension at chamfered edges.	**16.6.2.2** — When shorter bearing lengths occur in the field, analysis is usually the basis for acceptability. When designing bearing lengths, the effects of member shortening at expansion joints should be considered.
16.8.1 — Each precast member shall be marked to indicate its location and orientation in the structure and date of manufacture.	**16.8.1** — Not all products are marked with the date of manufacture, but adequate records should be kept to verify casting conditions.
17.5.2.1 — When contact surfaces are clean, free of laitance, and intentionally roughened, shear strength V_{nh} shall not be taken greater than $80b_vd$ in pounds.	**17.5.2.1** — Standard precast concrete manufacturing procedures for standard deck members are assumed to meet the requirement for "intentionally roughened." (Ref. Handbook Section 4.3.5) Industry tests confirm this practice to be safe.
18.4.1 — Stresses in concrete immediately after prestress transfer (before time-dependent prestress losses) shall not exceed the following: (a) Extreme fiber stress in compression . . . . $0.60f'_{ci}$ (b) Extreme fiber stress in tension except as permitted in (c) . . . . $3\sqrt{f'_{ci}}$	**18.4.1 (a)** — Initial compression is frequently permitted to go higher in order to avoid debonding or depressing strands. No problems have been reported by allowing compression as high as $0.70f'_{ci}$. **18.4.1 (b) (c)** — Initial tension is typically allowed to go as high as $6\sqrt{f'_{ci}}$ throughout most of the member. Because of member self weight, the transfer stresses decrease from the end to midspan of a simply supported member. It is not clear

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

how far into the span "at ends" can be used. (Ref. Handbook Section 4.2.2.2)

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

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

(a) Extreme fiber stress in compression due to prestress plus sustained loads . . . . $0.45f'_c$

(b) Extreme fiber stress in compression due to prestress plus total load . . . . $0.60f'_c$

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

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

18.4.2 (c) — The limitation of $6\sqrt{f'_c}$ is seldom used. Bilinear deflection behavior, as shown in the PCI Design Handbook, uses $7.5\sqrt{f'_c}$ as the cracking stress, so anything at that value or below would comply with 18.4.2(c).

18.4.2 (d) — See discussion of Section 7.7.3.2.

18.6 — Loss of prestress

18.6.1 — To determine effective prestress f_{se}, allowance for the following sources of loss of prestress shall be considered:

(a) Anchorage seating loss

(b) Elastic shortening of concrete

(c) Creep of concrete

(d) Shrinkage of concrete

(e) Relaxation of tendon stress

(f) Friction loss due to intended or unintended curvature in post-tensioning tendons

18.6 — Most structural engineers who specialize in the design of prestressed concrete follow the recommendations of an ACI-ASCE Committee 423 task force given in Ref. 18.6. (Ref. Handbook Section 4.5)

18.7.2 — As an alternative to a more accurate determination of f_{ps} based on strain compatibility, the following approximate values of f_{ps} shall be used if f_{se} is not less than $0.5f_{pu}$.

(a) For members with bonded prestressing tendons:

$$f_{ps} = f_{pu}\left\{1 - \frac{\gamma_p}{\beta_1}\left[\rho_p \frac{f_{pu}}{f'_c} + \frac{d}{d_p}(\omega - \omega')\right]\right\} \qquad (18\text{-}3)$$

18.7.2 — Many engineers use strain compatibility analysis for determining f_{ps}. Others use Eq. (18-3). With low-relaxation strand, the results are not substantially different. (Ref. Handbook Section 4.2.1)

If any compression reinforcement is taken into account when calculating f_{ps} by Eq. (18-3), the term

$$\left[\rho_p \frac{f_{pu}}{f'_c} + \frac{d}{d_p}(\omega - \omega')\right]$$

shall be taken not less than 0.17 and d' shall be no greater than $0.15d_p$.

18.8.3 — Total amount of prestressed and nonprestressed reinforcement shall be adequate to develop a factored load at least 1.2 times the cracking load computed on the basis of the modulus of rupture f_r specified in 9.5.2.3, except for flexural members with shear and flexural strength at least twice that required by 9.2.

18.8.3 — For simple span members, this provision is generally assumed to apply only at critical flexural sections. (Ref. Handbook Section 4.2.1)

18.11.2.1 — Members with average prestress f_{pc} less than 225 psi shall have minimum reinforcement in accordance with 7.10, 10.9.1 and 10.9.2 for columns, or 14.3 for walls.

18.11.2.1 — Columns which are larger than required for architectural purposes will use the level of prestress for the size of column needed. For example, if a 16 × 16 in. (406 × 406 mm) column will carry the load, but a 24 × 24 in. (610 × 610 mm) column is used, the total prestress force necessary is 225 (16 × 16) = 57,600 lb (26127 kg). This practice is supported by Sections 10.8.4 and 16.5.1.3 (a).

18.11.2.2 — Except for walls, members with average prestress f_{pc} equal to or greater than 225 psi shall have all prestressing tendons enclosed by spirals or lateral ties in accordance with the following:

(a) Spirals shall conform to 7.10.4.

(b) Lateral ties shall be at least No. 3 in size or welded wire fabric of equivalent area, and spaced vertically not to exceed 48 tie bar or wire diameters, or least dimension of compression member.

(c) Ties shall be located vertically not more than half a tie spacing above top of footing or slab in any story, and shall be spaced as provided herein to not more than half a tie spacing below lowest horizontal reinforcement in members supported above.

(d) Where beams or brackets frame into all sides of a column, it shall be permitted to terminate ties not more than 3 in. below lowest reinforcement in such beams or brackets.

18.11.2.2 — The PCI Prestressed Concrete Columns Committee report, "Recommended Practice for the Design of Prestressed Concrete Columns and Walls," recommends that column capacity be reduced to 85 percent of calculated if ties do not meet all of the requirements. Most producers use some ties, but may modify the size and spacing based on research. Note that walls are excluded from the lateral tie requirements. Column ties are required in seismic regions. (Ref. Handbook Example 4.7.2)

21.6.4.2 — Cast-in-place composite topping slab diaphragms

A composite topping slab cast-in-place on a precast floor or roof system shall be permitted to be used as a diaphragm provided the topping slab is reinforced and its connections are proportioned and detailed to provide for a complete transfer of forces to chords, collector elements, and resisting

21.6.4.2 — When the composite requirements are met, the diaphragm thickness includes both the topping and the precast flange or top wythe. (Ref. Handbook Section 3.6) elements. The surface of the previously hardened concrete on which the topping slab is placed shall be clean, free of laitance, and shall be intentionally roughened.

21.7.1 — Frame members assumed not to contribute to lateral resistance shall be detailed according to 21.7.2 or 21.7.3 depending on the magnitude of moments induced in those members when subjected to twice the lateral displacements under the factored lateral forces. When effects of lateral displacements are not explicitly checked, it shall be permitted to apply the requirements of 21.7.3.

21.7.1 — The Northridge Earthquake showed the importance of this section. See James K. Iverson and Neil M. Hawkins, "Performance of Precast/Prestressed Concrete Building Structures During Northridge Earthquake," PCI JOURNAL, V. 39, No. 2, March-April 1994, pp. 38-55. It should also be emphasized that some nominal ties, at least equivalent to the structural integrity requirements of Chapter 16, should be used in these non-lateral-load-resisting frames.

PCI Standard Design Practice†

by the PCI Technical Activities Council and the PCI Committee on Building Code

Comments by Alex Aswad, George Laszlo, Alan H. Mattock, Antoine E. Naaman, H. Kent Preston and Committees Closure

ALEX ASWAD‡

Section 10.4.1— Lateral support of beams: I find the restriction to non-loadbearing spandrels is too limiting. I recommend adding a sentence to the effect that:

> "For loadbearing spandrels, use rational analysis methods to verify the lateral-torsional stability of thin spandrels. Also consider the location of load application because stability is enhanced when loads are applied near the spandrel bottom."

Section 10.10 — Slenderness effects: One important case not covered by the PCI Design Handbook in Sections 3.51, 3.52, and 4.7.2 is the very common case of a pinned-pinned column with corbels carrying the usual inverted beam eccentric reactions. The primary moment diagram (see Fig. A) does not fit the common cases in textbooks or ACI. The junior engineer is at a loss in figuring out the C_m value.

I recommend adding a sentence to the effect that C_m can be conservatively taken as 0.65 in that case in conjunction with a max $M = M^*$.

Section 18.4.1(a) — Compression stresses at release: I am uncomfortable with $0.75f'_{ci}$ for a compression stress right now. This is due to unresolved concerns of excessive creep and micro-cracking that are now being investigated by Professor Bruce Russell.

I strongly recommend changing $0.75f'_{ci}$ to read instead:

"...$0.67f'_{ci}$ near midspan and $0.70f'_{ci}$ at the beam's ends."

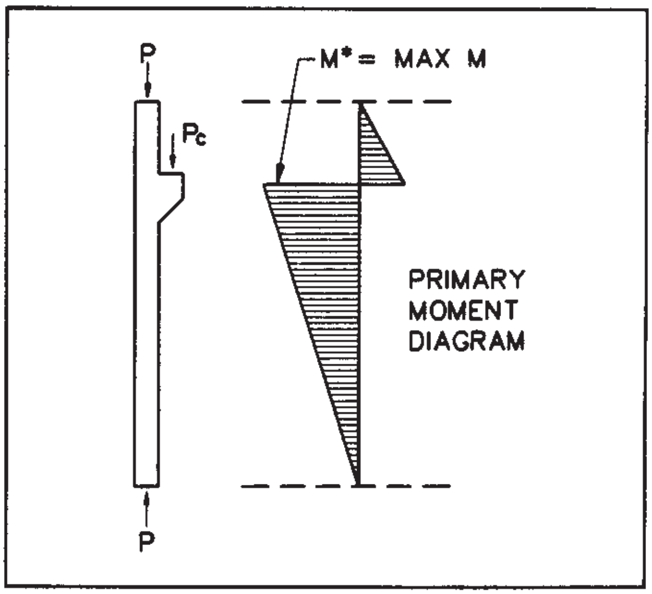

Fig. A. Primary moment diagram.

† PCI JOURNAL, V. 41, No. 4, July-August 1996, pp. 31-43.

‡ Professor of Engineering, Department of Civil Engineering, Pennsylvania State University at Harrisburg, Middletown, Pennsylvania.

GEORGE LASZLO*

I congratulate the Technical Activities Council and the Building Code Committee on their fine job of selecting sections of the ACI 318 Building Code that need expansion or clarification in order to account for common precast, prestressed concrete design practices.

Despite the excellence of the document, there are a number of sections in the Standard Design Practice that I believe merit further consideration:

Section 1.2.2 — The proper design of connections requires calculation of all the forces acting on the joint. Either the precast concrete manufacturer or the Engineer of Record supplying the information to the manufacturer must be held responsible for the complete structural design. I wonder which option is preferred by PCI and what the possible legal or ethical consequences are.

Section 3.5.2 — This section states "Special precautions are necessary when welding of stainless steel reinforcing bars or plates is used." The same precautions apply to galvanized bars or plates. In fact, special precautions should be applied to all welding processes.

Section 7.7.3 — According to this section, "...the provisions of Section 7.7.2 take precedence over 7.7.3, and prestressing steel cover requirements are the same as bars of the same diameter." I do not think this is true. According to ACI 318-95, Section 7.7.3.3, "For prestressed concrete members manufactured under plant conditions, minimum concrete cover for non-prestressed reinforcement shall be as required in 7.7.2." This permits only non-prestressed reinforcement with lesser cover.

Section 7.7.3.2 — Exclusion of double tee stems should be included in the ACI 318 Code; however, the Fire Code requirements are often more restrictive than ACI 318-95 for protective cover. This is not addressed at all in PCI's proposed Standard Practice.

Section 10.9.3 — Eq. (10-6) is very old; in fact, I believe it was included in ACI 318-36. With the advent of the current high strength spirals [120 to 150 ksi (827 to 1034 MPa)], this equation is obsolete. New research should be undertaken on this subject.

Section 11.5.5 — This section of the PCI proposed Standard Practice should be part of the ACI 318 Commentary. "If V_u is less than ϕV_c, shear reinforcement is omitted in prestressed double tees." This is not stated in ACI 318-95. If there are any problems with respect to a double tee in this case, the party responsible has no means to defend itself from a legal standpoint.

Section 16.2.4 — The same questions I raised regarding Section 1.2.2 apply to this section on connection design. In this case, who is the Engineer of Record and how are the responsibilities shared?

Section 16.8.1 — This section states "Not all products are marked with the date of manufacture." I do not believe this statement is correct. If products are not dated, there is no way to establish the strength, stressing and other PCI Plant Certification Program requirements of the product. This leaves the designer in a difficult position from a legal standpoint.

Section 17.5.2.1 — This section needs clarification, particularly regarding extruded products such as hollow-core slabs.

Section 18.4.2 — The last sentence of this section about a limitation on cover requirements is not true. Refer to the ACI 318-95 Commentary, Section R18.4.2(d).

Section 21.6.4.2 — According to this section, "When the composite requirements are met, the diaphragm thickness includes both the topping and the precast flange or top wythe. (Ref. Handbook Section 3.6)." It should also be noted that the precast flange must also be connected (by weld clips or extended flange reinforced and cast-in-place joint or by other mechanical connection) in order to carry a portion of the diaphragm forces acted on the composite diaphragm.

In addition to the preceding comments, Chapter 21 of ACI 318-95 on seismic design has many sections that need clarification or expansion when they are applied to precast, prestressed concrete.

My congratulations to both the committees that worked on the proposed PCI Standard Design Practice. It is an important first step in ensuring the proper treatment of precast, prestressed concrete in the building codes.

ALAN H. MATTOCK†

Section 11.9.7 — This section appears to endorse a risky practice, i.e., totally ignoring the requirements of ACI Code Section 11.9.7. At ultimate, a corbel acts like a truss rather than as a short cantilever beam. The primary reinforcement acts as a tension tie, connecting to an inclined concrete compression strut below the center of action of the vertical load. The strength of the corbel depends on the integrity of this connection between the tie and the strut. ACI Code Section 11.9.7 is intended to ensure that this integrity is maintained.

While I do not agree with the present encouragement to ignore ACI Code Section 11.9.7, I believe that it would be reasonable to relax the requirement of this code section for the case where the primary reinforcement is anchored by a structural weld to a transverse bar of at least equal diameter. In this case, I would propose that the bearing area be allowed to project to the exterior face of the transverse anchor bar. From what I have seen of the effectiveness of this type of anchorage in tests, I believe that with this relaxation, the integrity of the connection between the strut and the tie would be maintained and the strength of the corbel would not be impaired.

I do not believe that the comments under this section of the PCI Proposed Standard Practice relating to beam ledges are justified. The report on PCI SFR&D Project No. 5, "De-

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sign of Spandrel Beams," did not include any specific conclusions regarding the anchorage of the primary ledge reinforcement by bending this reinforcement downward at the front face of the ledge. In fact, the use of this detail may have contributed to the unexpectedly low punching shear failure loads experienced in the ledge of Specimen 2. It can be seen in Fig. 4.9(a) of this report that the punching shear failure surface in the concrete of the ledge does not intercept the primary reinforcement, but goes around the outside of the bend in the primary reinforcing bar at this location. Further tests should be made before the ACI Code Section 11.9.7 requirement that the bearing area not project over the bend in the primary reinforcement be written off.

Section 17.5.2.1 — The phrase "Standard precast concrete manufacturing procedures for standard products" should be defined, and substantiating references should also be provided. The provision of an adequate "intentionally roughened" surface is essential if the calculated shear strength of the interface, assuming an "intentionally roughened" surface, is to be realized.

Tests[1] have shown that if the surface of the precast concrete at a composite interface is smooth, the shear strength is only a small fraction of the shear strength of a composite interface in which the precast concrete surface meets the requirements of ACI Code Section 11.7.9. It is essential if a less rigorous requirement for an intentionally roughened surface than that of ACI Code Section 11.7.9 is to be permitted, then the degree of roughness must be defined explicitly and supporting evidence be referenced to show that the relaxed requirement results in shear friction strengths not less than those obtained with the degree of roughness specified in ACI Code Section 11.7.9.

Section 18.4.1(d) — The statement in this section of the PCI proposed Standard Practice that "the limitation on cover requirements in (d) is largely ignored on the assumption that 7.7.3.3 supersedes 7.7.3.2" is not in agreement with the intention of the ACI Code. ACI Code Section 7.7.3.3 relates only to non-prestressed reinforcement. ACI Code Section 7.7.3.2 relates to both prestressed and non-prestressed reinforcement. It was never intended that Section 7.7.3.3 should supersede Section 7.7.3.2. If it is recommended that ACI Code Section 18.4.1(d) be changed to eliminate the requirement for extra cover concrete, then a convincing reason for doing so should be set out.

Reference

1. Mattock, A. H., "Shear Transfer Under Monotonic Loading, Across an Interface Between Concrete Cast at Different Times," University of Washington Department of Civil Engineering Report SM 76-3, Seattle, WA, 1976.

ANTOINE E. NAAMAN*

The committees responsible for preparing the PCI Standard Design Practice should be commended for providing a summary as well as a simplified assessment and interpretation of code specifications as they apply to precast/prestressed concrete structures.

The writer noted two areas in which some clarification is necessary:

Section 18.4.2(c)(d) — "In practice, the limitation of $6\sqrt{f'_c}$ is meaningless."

Indeed, if a cracked section is allowed in prestressed or partially prestressed concrete, then limiting the tensile stress in service is meaningless. This may give the impression that nothing needs to be done, and that seems inconsistent because the design is based on allowable stresses, whether the stress is realistic or fictitious. Thus, an important clarification should be made in relation to the above provision. The closer the allowable tensile stress is to the tensile strength (or modulus of rupture) of concrete, the higher the probability that cracking will occur. Indeed, cracking is very likely to occur due to overload even when the design tensile stress in service is smaller than $6\sqrt{f'_c}$.

Once first cracking occurs, crack width will open after decompression and the stresses and stress changes in the component materials, steel and concrete, increase at a faster rate with the applied load. The effect of cyclic load on fatigue life may become significant. Prior extensive computerized investigation on partially prestressed concrete beams has confirmed that the fictitious tensile stress limitation cannot be used as a rational design criterion. Indeed, it was shown that beams designed for $46\sqrt{f'_c}$ were adequate in satisfying all strength and serviceability limit states while others (assumed pre-cracked) designed for $6\sqrt{f'_c}$ were not.

In short, if by design we allow cracking to occur, then cracked section analysis or cracked member analysis should be carried out and various serviceability limit states such as fatigue, crack width, corrosion and long-term deflection must be checked out. This is the trade-off we must pay if we move in the cracking range. Because the limit states of crack width, fatigue, corrosion and even deflection are generally not binding when an uncracked section is considered, it has become almost customary to assume that they are satisfactory in fully prestressed beams. However, this is not the case at all once cracking is allowed.

The writer would certainly support the total elimination of the tensile stress limitation in prestressed and partially prestressed concrete and to simply require that strength and serviceability limit states be satisfied; it is a reminder of code requirements anyway. The allowable tensile stress provided

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by the Code should be used mostly as a limit or a gauge to check if cracked or uncracked section analysis should be carried out.

Section 18.7.2 — Eq. (18.3)

Numerous equations have been proposed to predict the stress in the prestressing steel at nominal bending resistance. It should be observed that the new equation adopted by the AASHTO LRFD Specification for Highway Bridge Design offers a number of advantages over Eq. (18.3). The AASHTO equation is as follows:

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \qquad (1)$$

where f_{pu} = ultimate strength of the prestressing steel, d_p = depth to the prestressing steel and k is given by:

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) \qquad (2)$$

If any compression reinforcement is taken into account when calculating c or f_{ps}, the value of c should be larger than or equal to $3d'$ to ensure yielding of the compression reinforcement, where d' is the depth from the extreme compression fiber to the centroid of the compression reinforcement. If c is less than $3d'$, the contribution of the compressive reinforcement may be neglected.

Not only has the above equation been calibrated with a computerized nonlinear analysis and led to more accurate predictions than Eq. (18.3), but also it is significantly simpler when T-section analysis is considered. Indeed, the handling of reinforcing ratios and indices (ρ_p, ω, ω') when T-section behavior is present leaves significant room for misinterpretation and error. The AASHTO equation also has the advantage of allowing continuous yield strength ranges from bar to low relaxation strands.

Writing the above equation in combination with the first equation of force equilibrium at ultimate leads to a solution for f_{ps} and c. A flow chart has been developed to illustrate the designs steps. Also, examples and flow charts illustrating beams prestressed or partially prestressed with combined internal and external prestressing and with bonded or unbonded tendons have been developed according to the AASHTO Specifications.[2]

The above information, which is particularly useful in bridge design, should be brought to the attention of the users.

Reference

2. Naaman, A. E., "Unified Bending Strength Design of Concrete Members: AASHTO LRFD Code," *Journal of Structural Engineering*, American Society of Civil Engineers, V. 121, No. 6, June 1995, pp. 964-970.

H. KENT PRESTON*

The committees that prepared the PCI proposed Standard Practice have done a good job. I have the following comments:

Section 12.9.1 — The report states "...some changes in this section may be forthcoming." This is an important criterion. I will be interested in seeing any revision and the basis for it.

Section 18.5.1 — This section needs revision. For low relaxation strand, the ACI Code permits a jacking force tension of 0.94×0.90 or $0.85f_{pu}$.

Section 4.2.2.2 of the PCI Design Handbook permits a prestressing steel tension due to jacking force of $0.85f_{pu}$ or $0.94f_{py}$, but then goes on to say "It is common practice in the precast, prestressed concrete industry to follow the above recommendations with the following clarifications — Initial stress in strand due to jacking forces: Stress-relieved strand $0.7f_{pu}$ and low relaxation strand $0.75f_{pu}$. These values should not be exceeded without consulting the product manufacturer."

Some years ago when I was on the Technical Activities Council, we became so concerned about excessive tensions and worker safety that we wrote a "Safety Alert" that was published in the PCI JOURNAL and sent to all producers. I am not aware of any significant changes in materials or procedures that make excessive tensions any safer now than they were then.

ACI Code Section 18.5.1 and PCI Handbook Section 4.2.2.2 should be rewritten so that every designer and plant operator is made fully aware of the last part of 4.2.2.2, which requires consultation with the product's manufacturer before exceeding the tensions listed at the end of 4.2.2.2. It might be better to follow the method used in Section 9.15 of the AASHTO Code which limits stress in pretensioned strands at anchorage after seating to $0.7f_{pu}$ for stress relieved strand and $0.75f_{pu}$ for low relaxation strand and permits overstressing for a short time of $0.85f_{py}$.

There are probably other ways to revise 18.5.1. The important thing is to change it so that it does not continue to give the impression that $0.85f_{pu}$ is a standard allowable jacking stress.

I realize that the last part of 4.2.2.2 under the heading of PCI Practice recommends lower stresses, but it is not very emphatic and some readers who are in a hurry will not get that far after reading the ACI expression that permits $0.85f_{pu}$.

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COMMITTEE CLOSURE

The Technical Activities Council and the Building Code Committee very much appreciate the many fine comments received relating to the PCI Standard Design Practice. In addition to those printed here, comments, both editorial and substantive, were made on specific items by Larry G. Fischer, Daniel P. Jenny, Andrew Osborn, Courtney Phillips, Richard A. Ramsey, Stephen J. Seguirant and A. Fattah Shaikh. Most of those suggestions were adopted into the final document printed in this issue of the PCI JOURNAL.

Dr. Aswad raised three points:

1. He found that restricting the exception to the $50t$ limit on compression flanges is too restrictive. The experience of the committee members is that the $50t$ limit is rarely exceeded on loadbearing members. Please note that exceptions to this rule based on a rational analysis are allowed in Section 1.4 of ACI 318.

2. The suggestion for incorporating the "pinned-pinned column with corbels" has been forwarded to the Industry Handbook Committee for consideration.

3. Several other reviewers were not comfortable with allowing $0.75f'_{ci}$ for release compression. The document has been changed as suggested, pending results of research.

Mr. Laszlo's primary point was that changes should be made to ACI 318 to reflect many of the standard practices. The committees agree. One purpose of the development of this document was to identify items that the committees believe should be changed. Code changes, however, take time and are sometimes difficult to achieve. The earliest that any Code changes could take effect would be the year 2001.

Dr. Mattock has discussed three sections of the ACI Code and PCI Standard Practice:

1. He raises several key issues regarding Section 11.9.7 of ACI 318. The document has been changed so that potentially unsafe practices are not encouraged.

2. His statements regarding the roughness of the interface in a composite member are apparently based on some tests at the University of Washington. Unfortunately, the reference was not available to the committees, so it is difficult to evaluate. His statement that the shear strength of smooth surfaces is "only a small fraction" of that of surfaces roughened to approximately $^1/_4$ in. (6.35 mm) amplitude is undoubtedly true. However, in deck members with a composite topping, the shear stresses are very low, and without reference to absolute numbers, the committees could not determine whether or not the "small fraction" would meet design requirements or not.

Providing a measurable roughness on a deck member is difficult on many standard products. For example, machine-made hollow-core slabs will have varying degrees of roughness depending on the type of manufacturing equipment used, and most wet-cast deck members receive a magnesium float finish, which is certainly not smooth. It is doubtful if the amplitude of roughness could be accurately measured on any of these products. It seems that "amplitude" would imply some uniformity of peaks and valleys, and this is not what is typically achieved on deck products.

Further, it should be noted that the reference to $^1/_4$ in. (6.35 mm) amplitude does not appear until Section 17.5.2.3, whereas both 17.4.2.1 and 17.5.2.2 refer to "intentionally roughened." This has often led to disputes in practice. As a result, several producers have done push-off tests to verify interface shear strength. These tests typically show that standard surface treatments can develop the 80 psi (0.55 MPa) shear strength required in the Code. Several investigations of field problems have shown that delaminations are nearly always the result of loose materials at the interface. In other words, the "clean and free of laitance" requirements are not met.

3. Several reviewers objected to the wording in Section 18.4.2(d) of the Practice, so it was changed. Recent research has shown that cracking, per se, has little or no effect on corrosion. Rather, it is the cover on the steel that has the most influence on corrosion, so it would not seem necessary to require additional cover because the concrete is more likely to crack. Research by Robert Mast and Donald Pfeifer has shown that prestressed reinforcement is no more susceptible to corrosion than non-prestressed reinforcement.

Dr. Naaman has supported the practice of ignoring the limitation of $6\sqrt{f'_c}$, Section 18.4.1(c), but correctly points out that serviceability checks are extremely important. He also indicates he would support the total elimination of service stress limitations if proper serviceability checks would be required. This possibility is currently being studied by ACI Committee 318(G). However, it appears that revisions in crack control criteria, which would apply to prestressed as well as non-prestressed concrete, need to be developed. This is currently being studied by a task group of Committee 318 headed by Robert Mast.

Dr. Naaman has also suggested that Eq. (18.3) be revised. While this is something to be considered by ACI Committee 318, the PCI committees that prepared this Standard Design Practice did not feel it was a high priority issue. The current equation appears to agree closely with strain compatibility analysis, and the load carrying capacity of a member is not very sensitive to minor variations in allowable stress f_{ps}.

Mr. Preston has emphasized the safety considerations involved during the stressing operations in a pretensioning facility. He has suggested that ACI Section 18.5.1 and PCI Design Handbook Section 4.2.2.2 should be rewritten. We are sure that the respective committees involved in writing these provisions would be interested in any specific suggestions Mr. Preston can offer.