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PCI Standard Design Practice

Prepared by

PCI Committee on Building Code

PRECAST/PRESTRESSED CONCRETE INSTITUTE

INTRODUCTION

The PCI Building Code Committee has been working for the last several years to identify provisions in the ACI Building Code Requirements for Structural Concrete (ACI 318) that need clarification or expansion to support design practices commonly used with precast, prestressed concrete. This final document is a summary of that work and represents the second edition of this committee effort, as published in the Sixth Edition of the PCI Design Handbook. The original document, published in the January-February 2003 issue of the PCI JOURNAL, has been updated and revised to include discussions and comments from the membership and design profession in order to ensure that it represents the current design practices of the industry. Those reader comments, together with the Committee closure, appear in the November-December 2003 issue of the PCI JOURNAL.

PCI STANDARD DESIGN PRACTICE

Prepared by
PCI Committee on Building Code

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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 Building Code Committee, 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 318-02, and the numbers refer to sections in that document. References to the PCI Design Handbook are to the Sixth Edition, unless otherwise noted. Excerpts from ACI 318-02 are reprinted here with permission of the American Concrete Institute, Farmington Hills, Michigan.

ACI CODE	PCI PRACTICE
CHAPTER 1 – GENERAL REQUIREMENTS	
1.2.1(e) – Size and location of all structural elements, reinforcement, and anchors.	***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 piece 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)*
1.3.1 – Concrete construction shall be inspected as required by the legally adopted general building code. In the absence of such inspection requirements, concrete construction shall be inspected throughout the various work stages by or under the supervision of a registered design professional or by a qualified inspector.	***1.3.1*** *– Precast concrete products are inspected by internal quality control inspectors under the guidance of the "Manual for Quality Control for Plants and Production of Structural Precast Concrete Products" (PCI MNL-116) or "Manual for Quality Control for Plants and Production of Architectural Precast Concrete Products" (PCI MNL-117). PCI member producers are required to follow these procedures and are periodically monitored by independent Quality Certification inspectors. PCI Certified Plants are "Approved Fabricators," as defined in the model codes, and thus work done in the plant and approved by the building official is exempt from "special inspection" requirements.*

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CHAPTER 2 – DEFINITIONS

Moment frame – Frame in which members and joints resist forces through flexure, shear, and axial force. Moment frames shall be categorized as follows:

> ***Intermediate moment frame*** – A cast-in-place frame complying with the requirements of 21.2.2.3 and 21.12 in addition to the requirements for ordinary moment frames.
>
> ***Ordinary moment frame*** – A cast-in-place or precast concrete frame complying with the requirements of Chapters 1 through 18.
>
> ***Special moment frame*** – A cast-in-place frame complying with the requirements of 21.2 through 21.5, or a precast frame complying with the requirements of 21.2 through 21.6. In addition, the requirements for ordinary moment frames shall be satisfied.

Design of precast concrete moment frames is discussed in the PCI Design Handbook, Chapter 3.

Registered design professional – An individual who is registered or licensed to practice the respective design profession as defined by the statutory requirements of the professional registration laws of the state or jurisdiction in which the project is to be constructed.

Precast concrete shop drawings and product design calculations are normally done under the supervision of a licensed professional engineer, depending on project requirements.

Structural walls – Walls proportioned to resist combinations of shears, moments, and axial forces induced by earthquake motions. A shear wall is a structural wall. Structural walls shall be categorized as follows:

> ***Intermediate precast structural wall*** – A wall complying with all applicable requirements of Chapters 1 through 18 in addition to 21.13.
>
> ***Special precast structural wall*** – A precast wall complying with the requirements of 21.8. In addition, the requirements of ordinary reinforced concrete structural walls and the requirements of 21.2 shall be satisfied.
>
> ***Special reinforced concrete structural wall*** – A cast-in-place wall complying with the requirements of 21.2 and 21.7 in addition to the requirements for ordinary reinforced concrete structural walls.

Design of precast concrete shear wall buildings is discussed in the PCI Design Handbook, Chapter 3.

Tendon – In pretensioned applications, the tendon is the prestressing steel. In post-tensioned applications, the tendon is a complete assembly consisting of anchorages, prestressing steel, and sheathing with coating for unbonded applications or ducts with grout for bonded applications.

Precast, prestressed concrete products are nearly always pretensioned with seven-wire strand. Thus, the terms "tendon," "prestressing steel" and "strand" are used interchangeably.

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CHAPTER 3 – MATERIALS

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 the 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 welded stainless steel reinforcing bars or plates are used. (Ref. Handbook Section 6.5.1)*

3.5.5 – Prestressing steel

3.5.5 *– Nearly all strand used in precast, prestressed concrete products is seven-wire strand conforming with ASTM A 416, manufactured with low-relaxation wire conforming with the supplement to ASTM A 421.*

3.5.5.1 – Steel for prestressing shall conform to one of the following specifications:

(a) Wire conforming to "Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete" (ASTM A 421);

(b) Low-relaxation wire conforming to "Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete" including Supplement "Low-Relaxation Wire" (ASTM A 421);

(c) Strand conforming to "Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete" (ASTM A 416);

(d) Bar conforming to "Specification for Uncoated High Strength Steel Bars for Prestressing Concrete" (ASTM A 722).

3.5.5.2 – Wire, strands, and bars not specifically listed in ASTM A 421, A 416, or A 722 are allowed provided they conform to minimum requirements of these specifications and do not have properties that make them less satisfactory than those listed in ASTM A 421, A 416, or A 722.

3.8.8 – "Acceptance Criteria for Moment Frames Based on Structural Testing (ACI T1.1-01)," is declared to be part of this code as if fully set forth herein.

3.8.8 *– ACI T1.1-01 is primarily intended to address precast concrete moment frames.*

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CHAPTER 4 – DURABILITY REQUIREMENTS

4.2 – Freezing and thawing exposures

4.2.1 – Normal weight and lightweight concrete exposed to freezing and thawing or deicing chemicals shall be air-entrained with air content indicated in Table 4.2.1. Tolerance on air content as delivered shall be ±1.5 percent. For specified compressive strength $\mathbf{f'_c}$ greater than 5000 psi, reduction of air content indicated in Table 4.2.1 by 1.0 percent shall be permitted.

4.2.1 *– Some studies have shown that the very low water-cementitious materials ratios used in most precast concrete products require less air entrainment than cast-in-place concrete. (Ref. "Some Physical Properties of High Strength Concrete," Portland Cement Association, 1978; "Frost and Scaling Resistance of High Strength Concrete," PCA, 2001)*

4.2.2 – Concrete that will be subject to the exposures given in Table 4.2.2 shall conform to the corresponding maximum water-cementitious materials ratios and minimum specified concrete compressive strength requirements of that table. In addition, concrete that will be exposed to deicing chemicals shall conform to the limitations of 4.2.3.

4.2.2 *– (Note: See ACI 318-02 for the table referenced in this section.) The exposures discussed in this section affect the cover requirements given in Chapter 7 of the Code. While the high quality concrete produced in precasting plants is generally resistant to severe exposure, the use of deicing chemicals directly on all concrete surfaces is strongly discouraged.*

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 C1218.

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)*

CHAPTER 5 – CONCRETE QUALITY, MIXING, AND PLACING

5.2.3 – Concrete proportions shall be established in accordance with 5.3 or, alternatively, 5.4, and shall meet applicable requirements of Chapter 4.

5.2.3 *– Most producers of precast concrete products use standard mixes which have been designed and substantiated in accordance with this section.*

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 specimens 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.*

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CHAPTER 7 – DETAILS OF REINFORCEMENT

7.5.2 – Unless otherwise specified by the registered design professional, reinforcement, including tendons, and post-tensioning ducts shall be placed within the tolerances in 7.5.2.1 and 7.5.2.2.

7.5.2 *– Precast concrete products will normally conform to PCI tolerance standards specified in PCI MNL-135-00, and Chapter 8 of the PCI Design Handbook. Closer tolerances should not be specified except for special situations. (Ref. Handbook Section 8.2.4)*

7.7.5 – Corrosive environments

In corrosive environments or other severe exposure conditions, amount of concrete protection shall be suitably increased, and denseness and nonporosity of protecting concrete shall be considered, or other protection shall be provided.

7.7.5.1 – For prestressed concrete members exposed to corrosive environments or other severe exposure conditions, and which are classified as Class T or C in 18.3.3, minimum cover to the prestressed reinforcement shall be increased 50 percent. This requirement shall be permitted to be waived if the pre-compressed tensile zone is not in tension under sustained loads.

7.7.5.1 *– Nearly all precast, prestressed concrete members will be in compression in the precompressed tensile zone under sustained loads. Because of the compression and the high concrete quality achieved in precasting plants, members which meet this section have met the requirement of Section 7.7.5.*

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 with a minimum average prestress of 225 psi (1.55 MPa) 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.

7.10.4.2 – For cast-in-place construction, size of spirals shall not be less than 3/8 in. diameter.

7.10.4 and 7.10.5 *– 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 and Section 7.10.5.1 refers to "non-prestressed bars" so the minimum size requirements for ties clearly 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 concrete columns. For further information on this topic, 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.*

7.10.5 – Ties

Tie reinforcement for compression members shall conform to the following:

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7.10.5.1 – All non-prestressed bars shall be enclosed by lateral ties, at least No. 3 in size for longitudinal bars No. 10 or smaller, and at least No. 4 in size for No. 11, No. 14, No. 18, and bundled longitudinal bars. Deformed wire or welded wire reinforcement of equivalent area shall be permitted.

7.12 – Shrinkage and temperature reinforcement

7.12 – This section does not apply to flanges of precast, prestressed stemmed members. The flexural reinforcement in the flange of a stemmed member is transverse to the stems, and is usually welded wire reinforcement. Practice varies, but the Wire Reinforcement Institute requires that the longitudinal wires have an area at least 0.4 times that of the transverse wires. Section 7.12.3 is intended for post-tensioned slabs.

CHAPTER 8 – ANALYSIS AND DESIGN – GENERAL CONSIDERATIONS

8.1.3 – Anchors within the scope of Appendix D, Anchoring to Concrete, installed in concrete to transfer loads between connected elements shall be designed using Appendix D.

8.1.3 – Appendix D has specific provisions allowing modifications based on research. PCI has sponsored research on connections using welded headed studs that meet those provisions.

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 prohibit Section 8.3.3 to be used for post-tensioned concrete framing. Approximate (e.g., "portal") methods are sometimes used to design precast "litewalls" in parking structures.

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:

(a) eight times the slab thickness;

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

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 including the entire flange width in the compression block. (Ref. Handbook Examples 4.2.1.6 and 4.3.5.1)

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CHAPTER 9 – STRENGTH AND SERVICEABILITY REQUIREMENTS

9.2 – Required strength

9.2.1 – Required strength **U** shall be at least equal to the effects of factored loads in Eq. (9-1) through (9-7). The effect of one or more loads not acting simultaneously shall be investigated.

$$\mathbf{U = 1.4(D + F)} \quad (9\text{-}1)$$

$$\mathbf{U = 1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)} \quad (9\text{-}2)$$

$$\mathbf{U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.8W)} \quad (9\text{-}3)$$

$$\mathbf{U = 1.2D + 1.6W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R)} \quad (9\text{-}4)$$

$$\mathbf{U = 1.2D + 1.0E + 1.0L + 0.2S} \quad (9\text{-}5)$$

$$\mathbf{U = 0.9D + 1.6W + 1.6H} \quad (9\text{-}6)$$

$$\mathbf{U = 0.9D + 1.0E + 1.6H} \quad (9\text{-}7)$$

except as follows:

(a) The load factor on **L** in Eqs. (9-3) to (9-5) shall be permitted to be reduced to 0.5 except for garages, areas occupied as places of public assembly, and all areas where the live load **L** is greater than 100 lb/ft2.

9.2.1 *– It should be emphasized that volume changes, settlement, and other movements, T, are not to be considered simultaneously with wind or earthquake forces. Structural effects of T need only be considered when the structural element is restrained and can produce internal forces as a result of T.*

The load factor modification of 9.2.1(a) must be distinguished from live load reductions allowed in local codes. Where allowed, the reduced live loads establish a value for L to be used in the load combinations of 9.2.1 and 9.2.1(a). While the code provision does not allow a reduced load factor for garages, live load reductions are allowed, for example, under the International Building Code (IBC) and the Uniform Building Code (UBC) for parking garages.

9.2.3 – 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.

9.2.3 *– Chapter 3 of the PCI Design Handbook provides guidelines for estimating creep, shrinkage and temperature changes in precast concrete structures.*

R9.2.3 – The designer should consider the effects of differential settlement, creep, shrinkage, temperature, and shrinkage-compensating concrete. The term realistic assessment is used to indicate that the most probable values rather than the upper bound values of the variables should be used.

9.3.2.7 – Flexure sections without axial load in pretensioned members where strand embedment is less than the development length as provided in 12.9.1.1 .... 0.75

9.3.2.7 *– Section 4.2.3 of the PCI Design Handbook shows examples of designing for partially developed strands.*

9.5 – Control of deflections

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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 Class U flexural members, as defined in 18.3.3.

9.5.4.2 – For Class C and Class T flexural members, as defined in 18.3.3, deflection calculations shall be based on a cracked transformed section analysis. It shall be permitted to base computations on a bilinear moment-deflection relationship, or an effective moment of inertia as defined by Eq. (9-8).

9.5.4.3 – 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.

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

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9.5.4 *– Deflections are always calculated for precast, 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). Satisfactory performance may depend on many non-structural considerations. (Ref. Handbook Section 4.8)*

CHAPTER 10 – FLEXURE AND AXIAL LOADS

10.4.1 – Spacings 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-loadbearing 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 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 – The spacing s of reinforcement closest to a surface in tension shall not exceed that given by

$$\mathbf{s = \frac{540}{f_s} - 2.5c_c} \qquad (10\text{-}4)$$

but not greater than $\mathbf{12(36/f_s)}$.

10.6.4 *– Note that Section 10.6 is specifically excluded for prestressed concrete (Section 18.1.3), except as specified in 18.4.4.4. (Ref. Handbook Section 4.2.2.1)*

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

$$\rho_s = \mathbf{0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_y}} \qquad (10\text{-}5)$$

where $\mathbf{f_y}$ 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.*

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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** – The PCI Design Handbook, Chapter 4, addresses the application of this section to precast and prestressed concrete columns. (Ref. Handbook Section 4.9)*

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.

***10.10.2** – The "Moment Magnifier" method is not recommended for prestressed concrete compression members.*

CHAPTER 11 – SHEAR AND TORSION

11.0 – Notation

$\mathbf{b_w}$ = web width, or diameter of circular section, in.

***11.0** – The quantity b_w is the sum of the average stem width of all stems in tapered stem members such as double tees. This is critical in Eq. 11-14. In hollow-core slabs, b_w is the minimum web width.*

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 $\mathbf{V_u}$ 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 for shear calculations, but not necessarily for torsion. (Ref. Handbook Section 4.3)*

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 non-prestressed) where factored shear force $\mathbf{V_u}$ exceeds one-half the shear strength provided by concrete $\boldsymbol{\phi}\mathbf{V_c}$, except:

(a) Slabs and footings;

(b) Concrete joist construction defined by 8.11;

(c) Beams with total depth not greater than 10 in., 2.5 times thickness of flange, or 0.5 the width of web, whichever is greatest.

***11.5.5** – If V_u is less than ϕV_c, shear reinforcement may be omitted in prestressed double tees, with a nominal minimum 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)*

Prestressed hollow-core and flat slab units fall under (a) slabs and footings, and require no shear reinforcement, provided $V_u \le \phi V_c$.

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11.6 – Design for torsion	*11.6 – Torsion design has typically been done using the Zia-McGee method (PCI Design Handbook, Second Edition) or Zia-Hsu method (Fourth Edition), with excellent results. See also Zia-Hsu article, "Design for Torsion and Shear in Prestressed Concrete Flexural Members," in May-June 2004 PCI JOURNAL. The "thin-walled tube" model which has been in ACI 318 since 1995 typically requires significantly greater reinforcement than the previous methods in, for example, spandrel beams in parking structures. Based on performance of beams designed by the previous methods, this additional reinforcement is unnecessary and uneconomical, and most precast engineers are using the methods indicated above. (Ref. Handbook Section 4.4)*
11.7 – Shear-friction	
11.7.3 – A crack shall be assumed to occur along the shear plane considered. The required area of shear-friction reinforcement $\mathbf{A_{vf}}$ 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 4.3.6)*
11.7.7 – Net tension across shear plane shall be resisted by additional reinforcement. Permanent net compression across shear plane shall be permitted to be taken as additive to the force in the shear-friction reinforcement, $\mathbf{A_{vf}f_y}$, when calculating required $\mathbf{A_{vf}}$.	*11.7.7 – At shear wall bases, for example, the sustained dead load on the wall (including the weight of the wall) is added to the force developed in the bars across the shear plane. The minimum positive anchorage requirements of Chapter 16 still apply.*
11.9.1 – Brackets and corbels with a shear span-to-depth ratio **a/d** less than 2 shall be permitted to be designed using Appendix A. Design shall be permitted using 11.9.3 and 11.9.4 for brackets and corbels with: (a) **a/d** not greater than 1, and (b) subject to horizontal tensile force for $\mathbf{N_{uc}}$ not larger than $\mathbf{V_u}$. The requirements of 11.9.2, 11.9.3.2.1, 11.9.3.2.2, 11.9.5, 11.9.6, and 11.9.7 shall apply to design of brackets and corbels. Distance **d** shall be measured at the face of the support.	*11.9.1 – Section 6.8 of the PCI Design Handbook, Sixth Edition, describes a method of corbel design that has been used successfully. It is consistent with the strut-and-tie method of Appendix A permitted by this section.*
11.9.3.2 – Design of shear-friction reinforcement $\mathbf{A_{vf}}$ to resist shear $\mathbf{V_u}$ shall be in accordance with 11.7.	
11.9.3.2.1 – For normal weight concrete, shear strength $\mathbf{V_n}$ shall not be taken greater than $\mathbf{0.2f'_cb_wd}$ nor $\mathbf{800b_wd}$, in lb.	*11.9.3.2.1 – The PCI Design Handbook allows V_n up to $1000b_wd$. This is consistent with the "effective shear-friction" approach when concrete strengths of 5000 psi (34 MPa) and greater are used. (Ref. Handbook Table 4.3.6.1)*

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11.9.3.2.2 – For all-lightweight or sand-lightweight concrete, shear strength $\mathbf{V_n}$ shall not be taken greater than $\mathbf{(0.2 - 0.07a/d)\ f'_c b_w d}$ nor $\mathbf{(800 - 280a/d)\ b_w d}$, in lb.	*11.9.3.2.2 – The equations given here are more conservative in relation to normal weight concrete than the use of the λ factor in the effective shear-friction coefficient would produce.*
11.9.3.4 – Reinforcement $\mathbf{A_n}$ to resist tensile force $\mathbf{N_{uc}}$ shall be determined from $\mathbf{N_{uc} \le \phi A_n f_y}$. Tensile force $\mathbf{N_{uc}}$ shall not be taken less than $\mathbf{0.2V_u}$ unless special provisions are made to avoid tensile forces. Tensile force $\mathbf{N_{uc}}$ 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 $0.2V_{dead}$ is used as a guide. (Ref. Handbook Chapter 6)*
11.9.6 – At front face of bracket or corbel, primary tension reinforcement, $\mathbf{A_s}$, 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 $\mathbf{f_y}$ of $\mathbf{A_s}$ bars; (b) By bending primary tension bars $\mathbf{A_s}$ 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 6.13)*
11.9.7 – Bearing area of load on bracket or corbel shall not project beyond straight portion of primary tension bars $\mathbf{A_s}$, 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 6.13)*
11.10.8 – When factored shear force $\mathbf{V_u}$ is less than $\mathbf{\phi V_c/2}$, reinforcement shall be provided in accordance with 11.10.9 or in accordance with Chapter 14. When $\mathbf{V_u}$ exceeds $\mathbf{\phi V_c/2}$, wall reinforcement for resisting shear shall be provided in accordance with 11.10.9.	*11.10.8 – For precast walls, the reference should be to Section 16.4.2 rather than to Chapter 14.*

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11.10.9 – Design of shear reinforcement for walls

11.10.9.1 – Where factored shear force $\mathbf{V_u}$ exceeds shear strength $\boldsymbol{\phi}\mathbf{V_c}$, horizontal shear reinforcement shall be provided to satisfy Eqs. (11-1) and (11-2), where shear strength $\mathbf{V_s}$ shall be computed by

$$\mathbf{V_s = \frac{A_v f_y d}{s_2}} \qquad (11\text{-}31)$$

where $\mathbf{A_v}$ is area of horizontal shear reinforcement within a distance $\mathbf{s_2}$ 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 Section 11.10.9.1. Otherwise, minimum reinforcement required by Section 16.4.2 applies (0.001 times the gross cross-sectional area).*

CHAPTER 12 – DEVELOPMENT AND SPLICES OF REINFORCEMENT

12.5 – Development of standard hooks in tension

12.5.1 – Development length ℓ_{dh}, in inches, for deformed bars in tension terminating in a standard hook (see 7.1) shall be determined from 12.5.2 and the applicable modification factors of 12.5.3, but ℓ_{dh} shall not be less than $\mathbf{8d_b}$, 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 listed as Ref. 11-46 in ACI 318-02.*

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 **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 *– Figure R12.13.2.4 shows how WWR is used as shear reinforcement in double tee stems. For further information, see the Joint PCI/WRI Ad Hoc Committee on Welded Wire Fabric for Shear Reinforcement report, "Welded Wire Fabric for Shear Reinforcement," PCI JOURNAL, V. 25, No. 4, July-August 1980, pp. 32-36.*

CHAPTER 14 – WALLS

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).*

CHAPTER 15 – FOOTINGS

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.*

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CHAPTER 16 – PRECAST CONCRETE

16.2.4 – In addition to the requirements for drawings and specifications in 1.2, (a) and (b) 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)*

16.5.1.3 – Vertical tension tie requirements of 7.13.3 shall apply to all vertical structural members, except cladding, and shall be achieved by providing connections at horizontal joints in accordance with (a) through (c):

(a) Precast columns shall have a nominal strength in tension not less than $\mathbf{200A_g}$, in lb. For columns with a larger cross section than required by consideration of loading, a reduced effective area $\mathbf{A_g}$, based on cross section required but not less than one-half the total area, shall be permitted;

(b) Precast wall panels shall have a minimum of two ties per panel, with a nominal tensile strength not less than 10,000 lb per tie;

(c) When design forces result in no tension at the base, the ties required by 16.5.1.3(b) shall be permitted to be anchored into an appropriately reinforced concrete floor slab on grade.

16.5.1.3 *– This section applies to structures composed of many elements which must be tied together. Structures which use modules, or "boxes" will require different details to ensure integrity.*

16.5.1.3(b) *– Some panels may be too narrow to accommodate two connections. In such cases, as judged by the engineer, one connection may be adequate.*

16.5.1.4 – Connection details that rely solely on friction caused by gravity loads shall not be used.

16.5.1.4 *–This section should not be interpreted to prohibit connections designed using shear-friction principles.*

16.6.2.2 – Unless shown by test or analysis that performance will not be impaired, (a) and (b) 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 and movement at expansion joints should be considered.*

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16.6.2.3 – The requirements of 12.11.1 shall not apply to the positive bending moment reinforcement for statically determinate precast members, but at least one-third of such reinforcement shall extend to the center of the bearing length, taking into account permitted tolerances in 7.5.2.2 and 16.2.3.

CHAPTER 17 – COMPOSITE CONCRETE FLEXURAL MEMBERS

17.5.2.1 – When contact surfaces are clean, free of laitance, and intentionally roughened, shear strength $\mathbf{V_{nh}}$ shall not be taken greater than $\mathbf{80b_vd}$, in lb.

17.5.2.1 *– The $80b_vd$ horizontal shear strength level can be obtained by many finishes that appear smooth when compared with the roughness required in 17.5.2.2. Examples include floated, light broomed, or machine extruded surfaces. (Ref. Handbook Section 4.3.5) Because the strength of the interface in this case is developed by cementitious bond, proper preparation of the surface is of utmost importance.*

17.5.2.3 – When ties are provided in accordance with 17.6, and contact surfaces are clean, free of laitance, and intentionally roughened to a full amplitude of approximately **1/4** in., shear strength $\mathbf{V_{nh}}$ shall be taken equal to $\mathbf{(260 + 0.6\rho_v f_y)\lambda b_v d}$, in lb, but not greater than $\mathbf{500b_vd}$, in lb. Values for $\boldsymbol{\lambda}$ in 11.7.4.3 shall apply.

17.5.2.3 *– The surface should not be so rough as to allow bridging of the cast-in-place coarse aggregate and the formation of voids at the interface. The most important element is the "clean, free of laitance."*

17.6.3 – All ties shall be fully anchored into interconnected elements in accordance with 12.13.

17.6.3 *– Ties for horizontal shear in precast concrete members are typically U-shaped reinforcing bars which are embedded (mucked in) after the member has been cast and the top surface has been intentionally roughened and finished. The anchorage of the tie in the precast member is achieved by embedding the bar for the required development length without hooks (see also R16.7.1).*

Anchorage of hooked or bent ties in CIP topping is considered adequate if a minimum distance of 2 1/4, 2 3/4 and 3 1/4 in. (55, 70 and 80 mm) is provided between the shear transfer interface and the outside ends of standard hooks or U-bends of No. 3, No. 4, and No. 5 ties, respectively, based on research cited in PCI Handbook Section 4.3.5.

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CHAPTER 18 – PRESTRESSED CONCRETE

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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 .... $\mathbf{0.60f'_{ci}}$

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

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

Where computed tensile stresses exceed these values, bonded additional reinforcement (non-prestressed 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.

PCI PRACTICE

18.4.1 *– Recent research (see "Strength Design of Pretensioned Flexural Concrete Members at Prestress Transfer" by Noppakunwijai, Tadros, Ma, and Mast, PCI JOURNAL, January-February 2001, pp. 34-52) has shown that the compression limitations at transfer are more conservative than necessary, and have an effect on economy and safety. It has been common practice to allow compression up to* $0.70f'_{ci}$*. Other sections of the code define cracking stress as* $7.5\sqrt{f'_{ci}}$*, so the* $6\sqrt{f'_{ci}}$ *is not consistent. There also does not seem to be a logical reason for limiting the transfer tension at midspan to less than at the ends, since service load compression in the top is higher at midspan. Thus, at all sections, tension limits of* $7.5\sqrt{f'_{ci}}$ *are more consistent with Code philosophy. It is recommended that nominal reinforcement (at least two No. 4 or nominally tensioned strands) be provided in tops of beams even when tension stress is less than* $7.5\sqrt{f'_{ci}}$*.*

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R18.4.1(b) and (c) – ...Where tensile stresses exceed the permissible values, the total force in the tensile stress zone may be calculated and reinforcement proportioned on the basis of this force at a stress of $\mathbf{0.6f_y}$ but not more than 30,000 psi...

PCI PRACTICE

R18.4.1(b) and (c) *– Where beam tops are in tension at transfer of prestress forces, but are in compression under service load, and are not exposed to weather, the large amounts of steel indicated by this commentary item is excessive. Experience has shown that nominal top reinforcing bars or prestressing strand will adequately control temporary top cracking. Use of* f_y *(up to 60,000 psi) for the steel stress has been shown to be adequate. Bars must be detailed (e.g., hooks, C-bars, U-bars) to ensure development in the top tensile region.*

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18.4.2 – For Class U and Class T prestressed flexural members, stresses in concrete at service loads (based on uncracked section properties, and after allowance for all prestress losses) shall not exceed the following:

(a) Extreme fiber stress in compression due to prestress plus sustained load .... $\mathbf{0.45f'_c}$

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

PCI PRACTICE

18.4.2 *– Table R18.3.3 clearly defines the differences between Class U (uncracked), Class T (transition) and Class C (cracked) prestressed concrete members. Most members are designed as Class U or T, and thus the design procedures are essentially unchanged from previous editions of the Code. For special cases, a member may be designed as Class C, but more attention must be paid to performance. Chapter 4 of the PCI Design Handbook, Sixth Edition, provides examples of detailed transformed cracked section analysis.*

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18.4.4 – For Class C prestressed flexural members not subject to fatigue or to aggressive exposure, the spacing of bonded reinforcement nearest the extreme tension face shall not exceed that given by 10.6.4.

For structures subject to fatigue or exposed to corrosive environments, special investigations and precautions are required.

18.4.4.1 – The spacing requirements shall be met by nonprestressed reinforcement and bonded tendons. The spacing of bonded tendons shall not exceed 2/3 of the maximum spacing permitted for nonprestressed reinforcement.

Where both reinforcement and bonded tendons are used to meet the spacing requirement, the spacing between a bar and a tendon shall not exceed 5/6 of that permitted by 10.6.4. See also 18.4.4.3.

18.4.4.3 – The magnitude of $\Delta \mathbf{f_{ps}}$ shall not exceed 36 ksi. When $\Delta \mathbf{f_{ps}}$ is less than or equal to 20 ksi, the spacing requirements of 18.4.4.1 and 18.4.4.2 shall not apply.

__18.4.4__ – This section refers to maximum spacing requirements by applying Eq. (10-4) to prestressed concrete:

$$s = 540/f_s - 2.5c_c$$

where s is the maximum spacing and c_c is the clear cover to the reinforcement nearest the tension face. Note that these requirements are for Class C members only (that is, those in which the concrete tension under service loads exceeds $12\sqrt{f'_c}$. In checking this requirement, first check Section 18.4.4.3. If the spacing requirements can be met by substituting 36 ksi (250 MPa) for f_s in Eq. (10-4), no further check is necessary. If not, check if Δf_{ps} is less than 20 ksi (140 MPa) (which will often be the case). If so, no further check is necessary.

18.6 – Loss of prestress

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

(a) Prestressing steel seating at transfer;

(b) Elastic shortening of concrete;

(c) Creep of concrete;

(d) Shrinkage of concrete;

(e) Relaxation of prestressing steel 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 ACI-ASCE Committee 423 task force given in Ref. 18.6. (Ref. Handbook Section 4.7)

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

(a) For members with bonded tendons:

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

If any compression reinforcement is taken into account when calculating **fps** by Eq. (18-3), the term

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

shall be taken not less than 0.17 and **d′** shall be no greater than $\mathbf{0.15d_p}$.

__18.7.2__ – Many engineers and most computer programs 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)

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18.8.2 – Total amount of prestressed and non-prestressed 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 $\mathbf{f_r}$ specified in 9.5.2.3. This provision shall be permitted to be waived for:

(a) Two-way, unbonded post-tensioned slabs; and

(b) Flexural members with shear and flexural strength at least twice that required by 9.2.

18.11.2 – Limits for reinforcement of prestressed compression members

18.11.2.1 – Members with average prestress $\mathbf{f_{pc}}$ 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.

18.11.2.2 – Except for walls, members with average prestress $\mathbf{f_{pc}}$ equal to or greater than 225 psi shall have all tendons enclosed by spirals or lateral ties in accordance with (a) through (d):

(a) Spirals shall conform to 7.10.4;

(b) Lateral ties shall be at least No. 3 in size or welded wire reinforcement of equivalent area, and shall be spaced vertically not to exceed 48 tie bar or wire diameters, or the least dimension of the 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 not more than half a tie spacing below the lowest horizontal reinforcement in members supported above;

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

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18.8.2 *– For simple span members, this provision is generally assumed to apply only at critical flexural sections. (Ref. Handbook Section 4.2.1). This provision is intended to ensure adequate ductility in flexural members.*

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 *– 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 4.9.1.2)*

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CHAPTER 21 – SPECIAL PROVISIONS FOR SEISMIC DESIGN

Chapter 3 of the PCI Design Handbook, the PCI Seismic Design Manual (in progress), and other publications and research reports are available to assist the designer in the design of precast concrete structures in seismic areas.

APPENDIX A – STRUT-AND-TIE MODELS

Strut-and-tie modeling may have many applications in precast concrete construction, including corbels and dapped ends of beams. The design procedures for these elements given in the PCI Design Handbook have certain limits of applicability, and strut-and-tie methods may be used for cases which fall outside these limits.

APPENDIX D – ANCHORING TO CONCRETE

The PCI Design Handbook and the PCI Connections Manual have for many years given design recommendations for connections which use welded headed studs and other anchorage devices. Connections designed by these recommendations have performed satisfactorily.

D4.2 – The nominal strength for any anchor or group of anchors shall be based on design models that result in predictions of strength in substantial agreement with results of comprehensive tests. The materials used in the tests shall be compatible with the materials used in the structure. The nominal strength shall be based on the 5 percent fractile of the basic individual anchor strength. For nominal strengths related to concrete strength, modifications for size effects, the number of anchors, the effects of close spacing of anchors, proximity to edges, depth of the concrete member, eccentric loadings of anchor groups, and presence or absence of cracking shall be taken into account. Limits on edge distances and anchor spacing in the design models shall be consistent with the tests that verified the model.

***D4.2** – PCI has sponsored tests of stud assemblies which meet these requirements and result in design criteria which can be used in lieu of the requirements of this Appendix.*

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