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ACI 550R

Design Recommendations for Precast Concrete Structures

reported by ACI-ASCE Committee 550

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Recommendations for the design of precast concrete structures are presented. Design of individual members and of connections for the integration of members into structures are covered. Aspects of detailing, production, handling, erection, and strength evaluation that are related to design are also presented.

Keywords: anchorage (structural); bearing elements; composite construction (concrete to concrete); **concrete construction**; connections (structural); erection; fabrication; joints (junctions); load tests (structural); **precast concrete;** reinforced concrete; reinforcement; slabs; structural analysis; structural integrity; **structural design;** tolerances; volume changes; walls.

VARIANCES BETWEEN DESIGN RECOMMENDTIONS FOR PRECAST CONCRETE STRUCTURES AND ACI 318-89

1. Section 6.1 waives the requirements of ACI 318-89 Section 7.12 for precast one-way slabs not wider than 12 ft. Explanation is provided in ACI 550R Section 6.1

2. Section 6.2 modifies the requirements of ACI 318-89 Sections 14.3.1 through 14.3.3 and 14.3.5 for precast walls. Explanation is provided in ACI 550R Section 6.2

3. Section 8.2 waives the requirement of ACI 318-89 Section 12.11.1, which states that positive beam reinforcement is required to extend along the same face of the member into the support 6 in., if this would cause the reinforcement to extend beyond the end of the member. Explanation is provided in ACI 550R Section 8.2

4. Section 91. allows, under certain circumstances, waiving of the requirement of ACI 318-89 Section 7.5.1, which states that reinforcement shall be placed before concrete is placed. Explanation is provided in ACI 550R Section 9.1

5. Section 12.3 waives ACI 318-89 Section 20.4.11. Explanation is provided in ACI 550R Section 12.3.

CONTENTS

ACI Structural Journal, V. 90, No. 1, January-February 1993.
Pertinent discussion of the full report will be published in the September-October 1993 *ACI Structural Journal* if received by Apr. 1, 1993.

Chapter 12 — Strength evaluation of precast construction

Chapter 13 — References

CHAPTER 1—SCOPE

1.1

Recommendations of this report apply to design of precast concrete structures where all members or selected members are cast somewhere other than their final position in the structure.

1.2

This report should be used together with ACI 318, "Building Code Requirements for Reinforced Concrete," the minimum requirements of which may be legally binding. Because of the nature of precast concrete, certain recommendations contained in this report differ from the requirements of ACI 318.

1.3

Some of these recommendations may not be applicable to special conditions. Engineering judgment should be used in implementing this report.

1.3.1 Tilt-up concrete construction is a specialized type of precast concrete construction. Because panel dimensions in tilt-up are generally much larger than those in plant-cast precast, and roof and floor diaphragms are generally not constructed with precast sections, certain recommendations in this report differ from common practice found in tilt-up concrete construction.

CHAPTER 2—GENERAL

2.1

In design of precast members and connections, all loading and restraint conditions from casting to end use of the structure should be considered. The stresses developed in precast elements during the period from casting to final connection may be more critical than the service load stresses. Special attention should be given to the methods of stripping, storing, transporting, and erecting precast elements.

2.2

When precast members are incorporated into a structural system, the forces and deformations occurring in and adjacent to connections (in adjoining members and in the entire structure) should be considered.

The structural behavior of precast elements may differ substantially from that of similar members that are monolithically cast in place. Design of connections to transmit forces due to shrinkage, creep, temperature change, elastic deformation, wind forces, and earthquake forces require special attention. Details of such connections are especially important to insure adequate performance of precast structures.

2.3

Precast members and connections should be designed to meet tolerance requirements. The behavior of precast members and connections is sensitive to tolerances. Design should provide for the effects of adverse combinations of fabrication and erection tolerances.

Tolerance requirements should be listed on contract documents and may be specified by reference to accepted standards.[1-3] Tolerances that deviate from accepted standards should be so indicated.

2.4

All details of reinforcement, connections, bearing elements, inserts, anchors, concrete cover, openings and lifting devices, and specified strength of concrete at critical stages of fabrication and construction, should be shown on either the contract documents prepared by the architect/engineer of record or on the shop drawings furnished by the contractor. Whether this information is to be shown on the contract documents or shop drawings depends on the provisions of the contract documents. The shop drawings should show, as a minimum, all details of the precast concrete members and embedded items. The contract documents may specify that portions of connections exterior to the member are also to be shown on the shop drawings. The contract documents may also require the contractor to provide designs for the members and/or connections.

The contract documents should show the loads to be considered in design of the precast concrete elements of the structure, and they should indicate any special requirements or functions (for example: seismic loads, allowance for movements, etc.) that should be considered in design assigned to the contractor. In this case, the shop drawings should include complete details of the connections involved.

CHAPTER 3—DISTRIBUTION OF FORCES DUE TO GRAVITY LOADS

3.1

Design of precast floors with or without bonded concrete toppings that are subjected to concentrated or line loads may take into account distribution of forces. This distribution between hollow core or solid slabs is well documented.[4-7] It occurs even if no transverse moment strength exists across the joint, but shear continuity is maintained. Lateral distribution is made possible largely by the torsional stiffness of the members and the shear strength of the joint. Stemmed members with thin flanges have relatively low torsional stiffness and provide limited distribution.

3.2

The distribution of forces should be established by rational analysis or test. Distributions of deflections, moments, and shears are independent of each other, so one should not be inferred from any of the others.[8] Extensive tests[7] have shown that modes of failure in hollow core slab systems can include longitudinal splitting due to transverse bending, punching shear, or joint shear in addition to member flexure and shear. Strengths in these modes depend on parameters such as material properties, cross-sectional geometry, and location of the load relative to voids and joints. Openings in the floor system can influence lateral distribution.

3.2.1 Many methods of analysis, such as the finite element method, orthotropic plate theory,[9-10] the finite strip method,[8] and others[11] are available. It is important to model the appropriate transverse moment continuity across the joints and, in some types of members, the vertical displacements due to transverse shear.

3.2.2 The PCI Hollow Core Design Manual[12] contains a method based largely on test results.

3.2.3 All of the preceding methods may be used to predict force distributions between members. Research is continuing in this area.

CHAPTER 4—DIAPHRAGM AND SHEARWALL DESIGN

4.1

Precast concrete members can be assembled and connected to produce a structural system capable of resisting in-plane forces that result from wind, earthquake, or other lateral loads. Hollow-core slabs, solid slabs, or stemmed members used as either deck members or wall panels may be used in such structural systems.

4.2

Complete integrity of the structural system, which may include diaphragms, shearwalls, and their connections, should be assured. This includes, but is not limited to, the following: connections to transfer in-plane forces into the system; flexural integrity including proper tension and compression elements and any necessary internal connections; shear integrity including any necessary internal shear transfer connections; and proper connections to transfer in-plane forces out of the system.

4.3

Analysis of a diaphragm and shearwall system should include consideration of the diaphragm flexibility and the shearwall stiffnesses relative one to another. Diaphragm flexibility can affect the distribution of lateral forces to vertical elements and may also affect the general performance of the structure.

4.4

The PCI Design Handbook[13] and the PCI Hollow Core Design Manual[12] contain methods of analysis and design, and provide additional references.

CHAPTER 5—STRUCTURAL INTEGRITY

5.1

The structural integrity provisions of the ACI Building Code (ACI 318) are intended to provide toughness that will increase a structure's likelihood of surviving abnormal loads or displacements. The overall integrity of a structure can often be substantially enhanced by minor changes in the amount, location, and detailing of member reinforcement and in the detailing of connections.

5.2

Integrity connections should not rely solely on friction caused by gravity loads. An exception could be heavy modular unit structures where resistance to overturning or sliding has a factor of safety of 5 or more, or where sliding or rocking will not affect adversely the performance of the structure.

5.3

Integrity connections should be located to minimize the potential for cracking due to restraint of volume changes.

5.4

Integrity connections should be proportioned to develop a failure mode by yielding of steel.

5.5

Since the ACI 318 provisions for integrity of precast concrete structures are quite general, the following recommendations are provided to aid the designer in meeting the intent of those provisions. Since the design forces specified in these recommendations are chosen somewhat arbitrarily, it is not necessary to include a strength reduction factor in the calculations. These recommendations are minimums and all applicable loads, including dead, live, lateral, and volume-change restraint, should be considered in the design.

5.5.1 For precast concrete structures other than bearing wall structures above two stories high, the following integrity recommendations are made.[13]

5.5.1.1 All members should be connected longitudinally and transversely into the lateral load-resisting system, and the load path in the lateral load-resisting system should be continuous to the foundation.

Any individual member may be connected into this load path by alternative methods. For example, a load-bearing spandrel could be connected directly to the diaphragm (part of the lateral load-resisting system) by connection to deck members that are part of the diaphragm. Alternatively, the spandrel could be connected indirectly to the diaphragm by connection to its supporting columns, which in turn are connected to the diaphragm.

Connections to diaphragms should be designed for all applied loads but not less than 200 lb per lineal foot. This requirement is a traditional minimum requirement for both concrete and masonry walls.

5.5.1.2 Column base and splice connections should be designed for all applied loads, but not less than a tensile force of 200 times the gross area, in in.^{2}, of the column section. For a column with a cross section larger than required by loading considerations, a reduced effective area sufficient to resist the loads, but not less than one-half the total area, may be used in this calculation.

For cast-in-place columns, ACI 318 requires a minimum area of reinforcement equal to 0.005 times the gross area across the column footing interface to provide some degree of structural integrity. For precast columns, ACI 318 expresses this requirement in terms of an equivalent tensile force, 200 psi times the gross area, which is to be transferred.

5.5.1.3 Wall panels, including shearwalls, should be designed for all applied loads and should have a minimum of two vertical ties, with a nominal tensile strength of 10 kips per tie, extending through the panel and the joints above and below. It is standard industry practice for these ties to be located symmetrically about the vertical centerline of the wall panel and within the outer quarters of the panel width, where it is possible to do so.

The value of 10 kips, like other requirements in this section, is an arbitrary minimum presently used in standard industry practice.

5.5.1.4 Diaphragms should have tension ties around their perimeter and around openings large enough to interrupt diaphragm action.

5.5.2 For precast concrete bearing wall structures above two stories high, general structural integrity should be provided by incorporating continuous tension ties into the structure to resist the minimum forces specified in the following sections. Fig. 1 shows a typical layout for tension ties in wall systems and floor and roof systems.[14]

Nominal tie capacity for deformed reinforcing should be based on yield strength of the bar. When using unstressed prestressing strand, maximum allowable stress should correspond to a maximum strain of 0.35 percent (98 ksi for seven-wire 270-ksi strand). In all cases, the embedment should be

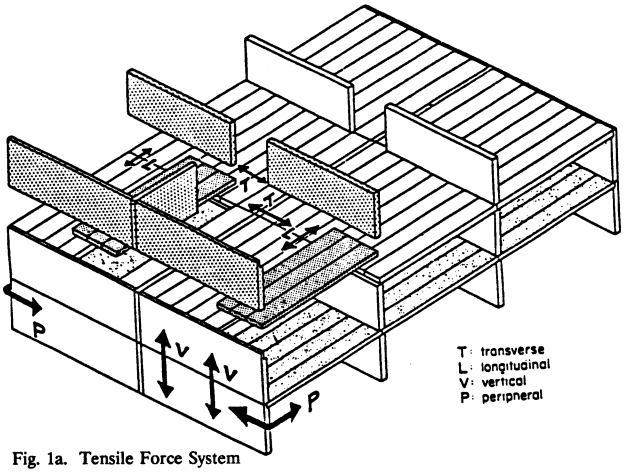

Fig. 1a. Tensile Force System

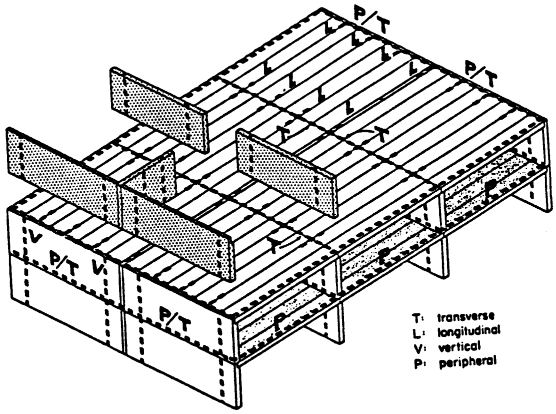

Fig. 1b. Suggested Layout of Tensile Ties

Fig. 1—Structural integrity of large-panel structures

sufficient to develop the tie capacity.[15] Note that while unstressed prestressing strand may be used to meet these integrity recommendations, ACI 318 does not allow its use for resisting seismic loads in regions of high seismic risk.

5.5.2.1 Transverse tension ties, perpendicular to the span of the floor elements and placed in the horizontal joints between floor and wall panels, should be provided to permit cantilever and beam action in the wall system in transverse direction (to span over a portion of wall lost due to abnormal load) and to contribute to floor diaphragm action. Reinforcement should provide a minimum nominal resisting strength of 1500 lb per lineal foot of vertical height of wall.

5.5.2.2 Continuous vertical tension ties should extend from foundation to roof to provide a minimum nominal resisting strength of 3000 lb per horizontal lineal foot of wall. Not less than two ties should be provided for each panel, and ties should not be spaced more than 12 ft on center.

5.5.2.3 Longitudinal tension ties in the direction of the floor or roof span should be provided to insure continuity for partial membrane action over interior walls and to connect external bearing walls with floor and roof diaphragms. The ties should provide a minimum nominal resisting strength of 1500 lb per foot of wall support transverse to the floor or roof span and should be spaced not more than 10 ft on center.

5.5.2.4 Perimeter tension ties should be provided in floor or roof diaphragms by means of a continuous tension tie positioned within 4 ft of the floor or roof edge. Ties should provide a minimum nominal resisting strength of 16,000 lb. These perimeter ties may be placed in the walls if they are developed with the diaphragm. Their requirements need not be additive with the transverse tie requirements.

CHAPTER 6—MEMBER DESIGN

6.1

In units of one-way precast floor and roof slabs and one-way precast, prestressed wall slabs not wider than 12 ft, requirements for shrinkage and temperature reinforcement may be waived. For reinforced concrete floor and roof elements such as hollow-core slabs, solid slabs, or slabs with close-spaced ribs, whether prestressed or not, there is generally no need to provide transverse reinforcement to withstand shrinkage and temperature stresses in the short direction. The short dimension of the element is limited to that which is practical to handle and ship and, thus, is less than a dimension, wherein shrinkage and temperature stresses can build up to a magnitude sufficient to cause cracking. Much of the initial shrinkage occurs before the elements are tied into the structure, and once in the final structure, the elements usually are not as rigidly connected transversely as in monolithically cast concrete floor systems.

In elements such as single and double tees with thin wide flanges, reinforcement is required in the flanges to resist the flexural moments transverse to the member axis. The amount of reinforcement should not be less than the minimum shrinkage and temperature reinforcement requirements of ACI 318.

6.2

For precast, nonprestressed walls, the reinforcement should be designed in accordance with the wall provisions of ACI 318, except that the area of horizontal and vertical reinforcement should each be not less than 0.001 times the gross cross-sectional area of the wall panel perpendicular to the direction of the reinforcement, and that spacing of reinforcement should not exceed five times the wall thickness, or 30 in., for interior walls or 18 in. for exterior walls. The lower minimum reinforcement requirements and greater permissible spacing of reinforcement in precast wall panels recognize that precast panels have very little restraint at their edges during early stages of curing. The wall panels build up lower shrinkage stresses than those found in comparable cast-in-place panels. This minimum area of wall reinforcement has been used generally for many years and is recommended by the Precast/Prestressed Concrete Institute[13] and the Canadian Code.[16]

6.3

Precast concrete flexural members are often made composite with cast-in-place concrete after the members are erected. The provisions of ACI 318 Chapter 17 should be followed for the design of such members.

CHAPTER 7—CONNECTION DESIGN

7.1

Application of ACI 318 permits a variety of methods for connecting members. Forces may be transferred between members by grouted joints, shear keys, mechanical connectors, reinforcing steel connections, bonded concrete toppings, or a combination of these means. These methods may be used for transfer of forces both in-plane and perpendicular to the plane of the members.

Mechanical connectors are defined as assemblies of steel plates or shapes, bolts, welds, metal castings, and/or other specialized items that are used to connect precast concrete members to each other or to other materials.

7.1.1 When grouted joints and shear keyways are used to transfer shear forces, the joint shear strength depends on the permanent net compression across the joint, the amount of steel crossing the shear plane, the in-place strength of the grout, and/or the configuration of the keyway. The shear strength may be computed by shear friction procedures or be based on the results of tests.

7.1.2 When mechanical connectors are used, the forces should be transferred properly between each element of the connection. The adequacy of each link in the connector, including anchorage of the connector into each member, should be considered. Shear transfer into a concrete member may be analyzed using shear friction.

7.1.3 Reinforcing steel connections include but are not limited to grouted dowels, steel extensions, and post-tensioning. The reinforcement should provide the design tension strength required in the connection.

Reinforcement details should be such that tension forces passing through principal connections are transferred to primary reinforcement in the members being connected. Principal connections refer to those that form part of the primary load-resisting system of the structure.

Steel plates and shapes with headed studs and similar anchors are used commonly for connections other than principal (as defined previously) connections. The PCI Design Handbook[13] provides guidance for their design.

7.2

When joining members by connections with differing structural properties, the relative stiffnesses, strengths, and ductilities of the connections should be accounted for in predicting their combined behavior under the anticipated joint loads and deformations.

7.3

The adequacy of connections to transfer forces between members may be determined by analysis or by test.

7.4

Several references are available to assist the design and detailing of connections.[13,17,18]

CHAPTER 8—BEARING DESIGN

8.1

Bearing for precast floor and roof members on simple supports should satisfy the following:

8.1.1 The bearing stress at the contact surface between supported and supporting members should not exceed the design bearing strength for either surface and the bearing element. Concrete bearing strength should be as given in ACI 318. The PCI Design Handbook[13] provides additional guidance where horizontal forces are present at bearings.

8.1.2 Each member and its supporting system should have design dimensions selected so that, under the least favorable addition of reasonable assumed 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 $\ell/180$ of the clear span but not less than

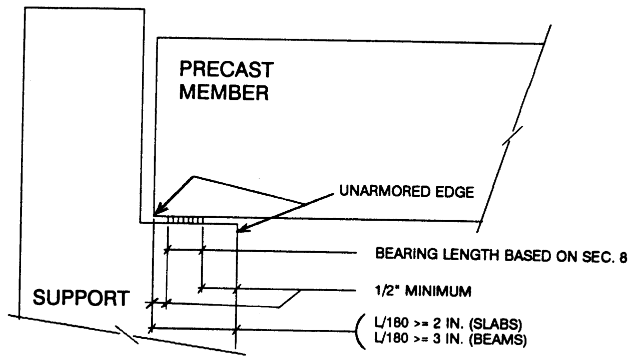

Fig. 2—Bearing length versus length of member over support

For solid or hollow core slabs 2 in.
For beams or stemmed members 3 in.

Differentiation is made between bearing length and length of the end of a precast member over the support (Fig. 2).

8.1.3 Bearing pads at unarmored edges should be set back a minimum of ½ in. (or at least the chamfer dimension at chamfered edges) from the edge to prevent spalling. Slab bearings are excepted from this recommendation due to the typically smaller bearing stresses involved.

8.2

Positive reinforcement need not extend into the support beyond the end of the precast member but should extend at least to the center of the bearing length. It is unnecessary to develop positive reinforcement beyond the ends of the precast element if the system is statically determinate because there is no shifting of the moments.

8.3

End-supported members other than hollow core and solid slabs should be provided with end reinforcement, unless only lightly loaded. The PCI Design Handbook[13] provides a method for computing required reinforcement using shear-friction theory. Anchorage of this reinforcement should be in accordance with ACI 318. Mechanical anchorage such as welding to a shoe angle or plate with transverse anchorage, is suggested. Welding of reinforcement should conform to AWS D1.4. Properly detailed hooks are also suitable.

CHAPTER 9—ITEMS EMBEDDED AFTER CONCRETE PLACEMENT

9.1

Many precast products are manufactured in such a way that it is difficult, if not impossible, to position reinforcement that protrudes from the concrete before the concrete is placed, as required by the provisions of ACI 318. Experience has shown that embedded items (such as dowels or inserts) that either protrude or remain exposed for inspection may be embedded while the concrete is in a plastic state, providing:

a. The embedded items are not required to be hooked or tied to reinforcement within the concrete;

b. The concrete is consolidated properly around the embedded item; and

c. The embedded items are maintained in the correct position while the concrete remains plastic.

This exception is not applicable to reinforcement that is completely embedded.

CHAPTER 10—MARKING AND IDENTIFICATION

10.1

Each precast member should be marked with an identification number to indicate its location and orientation in the structure according to the erection drawings, as well as with its date of manufacture.

CHAPTER 11—HANDLING

11.1

Member design should consider all appropriate forces and distortions to insure that during curing, stripping, storage, transportation, and erection, precast members are not overstressed or otherwise damaged. ACI 318 requires adequate performance at service loads and adequate strength under factored loads. Handling loads should not produce permanent stresses, strains, cracking, or deflection that are inconsistent with provisions of ACI 318.

Guidance on assessing cracks in precast members is given in Precast/Prestressed Concrete Institute reports on fabrication and shipment cracks.[19, 20]

11.2

Precast members should be supported adequately and braced during erection to insure proper alignment and structural integrity until permanent connections are completed. It is important that all required temporary erection connections, bracing, and shoring be shown on erection drawings, as well as the sequencing of removal of these items.

CHAPTER 12—STRENGTH EVALUATION OF PRECAST CONSTRUCTION

12.1

This section contains recommendations that amplify ACI 318 Chapter 20 to include testing and evaluation of precast flexural members that are to become composite in the completed structure. A precast member that is intended to respond to loads after being made composite with cast-in-place concrete may be tested as a precast member alone (prior to integration into the structure) in accordance with the following recommendations:

12.1.1 The test load should be that load which, when applied to the precast member alone, induces the same total force in the tension reinforcement as would be induced by loading the composite member with the test load required by ACI 318, Chapter 20. If the member has prestressed reinforcement, the nonlinear stress-strain relationship for the steel should be used in calculations.

12.1.2 Acceptance should be based on the criteria of ACI 318, Chapter 20. Attention is drawn to ACI 318R-89 (ACI Building Code Commentary), Chapter 20 for analysis of cracking that may occur during the test.

12.2

If analysis shows that the noncomposite member could fail by compression or buckling before the full test load is attained, the test should not be conducted. This may be the case, for example, in a member with a cast-in-place compression flange. Alternatively, the test may be made on the composite member.

12.3

Retest of precast members, prestressed as well as nonprestressed, should be allowed. There is no reason to ban retesting of prestressed members, as suggested in ACI 318, Chapter 20, as long as the retest criterion for deflection recovery (which is more stringent than for the initial test) governs acceptance.

CHAPTER 13—REFERENCES

13.1—Recommended references

The documents of the various standards-producing organizations referred to in this document are listed with their serial designation.

American Concrete Institute

117	Standard Specifications for Tolerances for Concrete Construction and Materials
318/318R	Building Code Requirements for Reinforced Concrete and Commentary

American Welding Society

D1.4	Structural Welding Code — Reinforcing Steel

These publications may be obtained from the following organizations:

American Concrete Institute
P.O. Box 19150
Detroit, MI 48219

American Welding Society, Inc.
2501 N.W. 7th Street
Miami, FL 33125

13.2—Cited references

1. *Manual for Quality Control for Plants and Production of Precast Prestressed Concrete Products,* MNL-116-85, Prestressed Concrete Institute, Chicago, 1985, 123 pp.

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14. Speyer, Irwin J. (for PCI Committee on Precast Concrete Bearing Wall Buildings), "Considerations for the Design of Precast Concrete Bearing Wall Buildings to Withstand Abnormal Loads," *PCI Journal,* V. 21, No. 2, Mar.-Apr. 1976, pp. 18-51.

15. Salmons, J. R., "Bond Characteristics in Untensioned Prestressing Strand," *PCI Journal,* V. 22, No. 1, Jan.-Feb. 1977, pp. 52-65.

16. *Design of Concrete Structures for Buildings,* Canadian Standard Association, Ontario, Canada, 1984, 281 pp.

17. *PCI Manual on Design and Typical Details of Connections for Precast and Prestressed Concrete,* 2nd Edition, MNL-123-88, Prestressed Concrete Institute, Chicago, 1988, 270 pp.

18. Martin, L. D., and Korkosz, W. J., *Connections for Precast Prestressed Concrete Buildings—Including Earthquake Resistance, Technical Report* No. 2, Prestressed Concrete Institute, Chicago, 1982, 297 pp.

19. PCI Committee on Quality Control Performance Criteria, "Fabrication and Shipment Cracks in Prestressed Hollow-Core Slabs and Double Tees," *PCI Journal,* V. 28, No. 1, Jan.-Feb. 1983, pp. 18-39.

20. PCI Committee on Quality Control Performance Criteria, "Fabrication and Shipment Cracks in Precast or Prestressed Beams and Columns," *PCI Journal,* V. 30, No. 3, May-June 1985, pp. 24-29.

CONVERSION FACTORS

1 kip	=	4.45 kn
1 psi	=	0.006895 MPa
1 ksi	=	6.895 MPa
1 lb/ft	=	157 N/m^3
1 lb	=	1.356 N - m
1 lb	=	0.4536 kg
1 in.	=	25.4 mm

This report was submitted to letter ballot of the committee and approved in accordance with ACI standardization procedures.