ACI STRUCTURAL JOURNAL COMMITTEE REPORT

Title no. 86-S31 ACI 423.3R

Recommendations for Concrete Members Prestressed with Unbonded Tendons

reported by ACI-ASCE Committee 423

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This report is presented as a guide for the design of flexural structural members in buildings with unbonded tendons. Suggestions are presented for needed revisions and additions to the ACI 318 Building Code regarding this subject. Consideration is given to determination of fire endurance, design for seismic forces, and catastrophic loadings, in addition to design for gravity and lateral loads. Recommendations are presented concerning details and properties of tendons, protection against corrosion, and construction procedures.

Keywords: anchorage (structural); beams (supports); bond (concrete to reinforcement); **concrete construction**; concrete slabs; corrosion; cover; cracking (fracturing); earthquake-resistant structures; fire resistance; flat concrete plates; flat concrete slabs; joints (junctions); loads (forces); post-tensioning; **prestressed concrete**; prestressing; prestressing steels; shear properties; stresses; structural analysis; **structural design; unbonded prestressing.**

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ACI Structural Journal, V. 86, No. 3, May-June 1989.
Pertinent discussion will be published in the January-February 1990 *ACI Structural Journal* if received by Aug. 1, 1989.

CHAPTER 1—INTRODUCTION

1.1—General

This report is intended to update the previous ACI-ASCE Committee 423 report entitled "Recommendations for Concrete Members Prestressed with Unbonded Tendons," (ACI 423.3R-83) published in 1983. In the interval since the publication of that report and the two previous reports that it replaced, "Tentative Recommendations for Concrete Members Prestressed with Unbonded Tendons (ACI 423.1)" and "Tentative Recommendations for Prestressed Concrete Flat Plates" (ACI 423.2), many of its recommendations have been incorporated into the ACI Building Code (ACI-318). As a result, design with unbonded tendons is covered in ACI 318-83 in nearly the same degree of completeness as is design with bonded tendons.

Nonetheless, these recommendations have been prepared to provide an up-to-date and comprehensive guide for design, materials, and construction for concrete members prestressed with unbonded tendons. Suggested revisions and additions to the ACI Building Code are also included in this report.

1.2—Objective

1.2.1 The objective of this report is to present recommendations for materials, design, and construction for concrete structures prestressed with unbonded tendons that are commensurate with the safety and serviceability requirements of the ACI Building Code (ACI-318).

1.2.2 This report is a guide, not a building code or specification. The recommendations are presented for the guidance and information of professional engineers who must add their engineering judgment to applications of the recommendations.

1.3—Scope

1.3.1 The recommendations are intended to cover special considerations pertinent to design with unbonded tendons. Considered in this report are the design of beams, girders, and slabs, continuous members, and details and properties of tendons and anchors and their protection from corrosion during construction and throughout the life of the structure.

1.3.2 The recommendations are not intended for applications using tendons external to the concrete section, unbonded construction stages of elements utilizing bonded tendons, members subject to direct tension such as tiebacks, cable stays, arch ties, or circumferential tendons for pressure vessels.

1.4—Notations and definitions

Symbols have the meaning given in ACI 116R or ACI 318 or are defined in the text. Definitions of terms as used in this report follow.

Anchorage—In post-tensioning, a device used to anchor the prestressing steel to the concrete member; in pretensioning, a device used to anchor the prestressing steel during hardening of the concrete; in precast concrete construction, the devices for attaching precast units to the building frame; in slab or wall construction, the devices used to anchor the slab or wall to the foundations, rock, or adjacent structure.

Bonded tendons—Tendons that are bonded to the concrete through grouting or other approved means, and therefore are not free to move relative to the concrete.

Coating—Material used to protect against corrosion and lubricate the prestressing steel.

Coupler—Device for connecting reinforcing bars or prestressing steel end to end.

Duct—Hole formed in the concrete for the insertion of prestressing steel that is to be post-tensioned, usually formed by a ferrous-metal sheath of spirally wound or longitudinally seamed strip with flexible or semirigid seams.

Prestressing steel—High-strength steel used to prestress concrete, commonly seven-wire strands, single wires, bars, rods, or groups of wires or strands.

Sheath—An enclosure in which the prestressing steel is placed to prevent bonding during concrete placement and, in the case of tendons that are to remain unbonded, to protect the corrosion-inhibiting coating on the prestressing steel.

Tendon—The complete assembly used to impart prestressing forces to the concrete, consisting of anchorages and prestressing steel with sheathing when required.

Unbonded tendons—Tendons in which the prestressing steel is permanently free to move (between anchors) relative to the concrete to which they are applying prestressing forces.

CHAPTER 2—DESIGN CONSIDERATIONS

2.1—General

Strength and serviceability limitations (including stresses) should conform to the provisions of ACI 318, but some recommendations are offered that differ from the contents of the ACI Building Code or relate to areas not covered by the building code.

2.2—Continuous members

2.2.1 For slabs or beams continuous over two or more spans with one-way prestressing only, a loading condition or fire exposure that causes failure of all the tendons in one span will lead to a loss of prestress and much of the load-carrying capacity in the other spans. Consideration should be given to the consequence of such a catastrophic failure in any specific span to the overall stability of the structural system. ACI 318 has responded to this concern, as well as to other considerations such as crack width limitation, in Section 18.9.2. Section 18.9.2 specifies minimum bonded reinforcement equal to 0.40 percent of the area of that part of the cross section between the flexural tension face and the center of gravity of the gross section. It is recommended that Grade 60 reinforcement be used for this purpose. This amount of bonded reinforcement is approximately equal to the minimum reinforcement requirement for conventionally reinforced slabs (Section 10.5.3 of ACI 318).

One-way slabs may also incorporate unbonded partial length tendons, lapped tendons, or tendons with intermediate anchorages that would serve to limit the extent of the loss of load-carrying capacity. The Uniform Building Code requires an alternate load-carrying capacity provided by bonded reinforcement of $D + 0.25L$, with a ϕ factor of 1.0, for one-way elements post-tensioned with unbonded tendons. Depending on the span configuration and the loads, the $D + 0.25L$ criterion is sometimes satisfied in slabs by the bonded reinforcement requirements of Section 18.9.2 of ACI 318.

In negative moment regions of T-beams or other members where compression width is limited, the amount of reinforcement provided is limited (Section 18.8 of ACI 318) to avoid the possibility of a compression failure at design loads.

In accordance with Section 18.9.4.3 of ACI 318, bonded reinforcement for both beams and slabs should be detailed in accordance with the provisions of Chapter 12 of ACI 318 with sufficient lap between positive and negative moment bars to insure that the bonded reinforcement will function as an independent load-carrying system.

2.2.2 In the case of two-way slabs of the usual proportions, catastrophic loading beyond design capacity in one bay is generally not as critical to other spans as in one-way systems. For two-way slabs, the load-carrying capacity of the tendons in each direction should be considered. Tests[1-5] have demonstrated two-way flexural behavior under various partial loading patterns and the capacity of two-way post-tensioned systems to endure some types of catastrophic loadings; this behavior is intrinsically recognized in ACI 318, as well as in the Uniform Building Code and some local building codes by reduction in the amount of bonded reinforcement required in comparison with one-way systems.

2.3—Corrosion protection

Unbonded prestressing tendons should be protected against corrosion during storage, transit, construction, and after installation. Corrosion protection should conform to the requirements of the Post-Tensioning Institute, "Specification for Unbonded Single Strand Tendons."[6] This specification provides for two levels or degrees of corrosion protection, with additional corrosion protective measures required for tendons used in corrosive environments. Concrete cover for unbonded tendons should be detailed considering the factors discussed in Section 4.4.

2.4—Fire resistance

Fire resistive ratings may be determined in accordance with the heat transmission and dimensional provisions of Section 2.4.1 or by the rational design procedures for determining fire endurance discussed in Section 2.4.2[7,8] (also refer to ACI 216R and ASTM E 119). ASTM E 119 includes a guide for classifying construction as "restrained" or "unrestrained." The guide indicates that either restraint to thermal expansion or continuity restraint results in greatly improved fire endurance and that nearly all cast-in-place concrete construction may be considered to be restrained.

Table 2.1 — Suggested concrete thickness requirements for various fire endurances[8]

Aggregate type	Slab thickness, in., for fire endurance indicated				
	1 hr	1½ hr	2 hr	3 hr	4 hr
Carbonate	3¼	4⅛	4⅝	5¾	6⅝
Siliceous	3½	4¼	5	6¼	7
Lightweight	2⅝	3¼	3¾	4⅝	5¼

Table 2.2 — Suggested concrete cover thickness for slabs prestressed with post-tensioned reinforcement[8]

Restrained or unrestrained	Aggregate type	Cover thickness, in., for fire endurance of				
		1 hr	1½ hr	2 hr	3 hr	4 hr
Unrestrained	Carbonate	¾	1 1/16	1⅜	1⅞	—
Unrestrained	Siliceous	¾	1¼	1½	2⅛	—
Unrestrained	Lightweight	¾	1	1¼	1⅝	—
Restrained	Carbonate	¾	¾	¾	1	1¼
Restrained	Siliceous	¾	¾	¾	1	1¼
Restrained	Lightweight	¾	¾	¾	¾	1

2.4.1 *Minimum dimensions for various fire resistive classifications*[8]

2.4.1.1 *Slabs*—To meet minimum heat-transmission requirements, i.e., temperature rise of 250 F (139 C) of the unexposed surface, the thicknesses requirements for concrete slabs should be the same whether the concrete is plain, reinforced, or prestressed. Table 2.1 gives slab thickness recommended for this purpose. Cover thicknesses for post-tensioning tendons in unrestrained slabs are determined by the elapsed time during a fire test until the tendons reach a critical temperature. For cold-drawn prestressing steel, that temperature is 800 F (427 C). For restrained slabs, there are no steel temperature limitations, but the heat transmission end-point temperature limitation [250 F (139 C)] is the same as for unrestrained slabs. Fire tests of restrained slabs indicate that slabs with post-tensioned reinforcement behave about the same as reinforced concrete slabs of the same dimensions. Accordingly, the cover for post-tensioning tendons in slabs could be essentially the same as the cover for reinforcing steel in slabs. Applying these criteria to post-tensioned slabs, cover thicknesses are as recommended in Table 2.2.

2.4.1.2 *Beams*—Minimum dimensions for beams with post-tensioned reinforcement for various fire endurances are functions of the types of steel and concrete, beam width, and cover. For very wide beams, the cover requirements should be about the same as those for slabs. For restrained beams spaced more than 4 ft (1.22 m) on centers, the temperature of 800 F (427 C) for cold-drawn prestressing steel must not be exceeded to achieve a fire-endurance classification of 1 hr or less; for classifications longer than 1 hr, this temperature must not be exceeded for the first half of the classifi-

Table 2.3 — Suggested cover thickness for beams prestressed with post-tensioned reinforcement[8]

				Cover thickness, in., for fire endurance of				
Restrained or unrestrained	Steel type	Concrete type*	Beam width, in.[†]	1 hr	1½ hr	2 hr	3 hr	4 hr
Unrestrained	Cold-drawn	NW	8	1¾	2	2½	4½[‡]	—
Unrestrained	Cold-drawn	LW	8	1½	1¾	2	3¼	—
Unrestrained	H.S.A. bars	NW	8	1½	1½	1½	2½	—
Unrestrained	H.S.A. bars	LW	8	1½	1½	1½	2¼	—
Restrained	Cold-drawn	NW	8	1½	1½	1¾	2	2½
Restrained	Cold-drawn	LW	8	1½	1½	1½	1¾	2
Restrained	H.S.A. bars	NW	8	1½	1½	1½	1½	1½
Restrained	H.S.A. bars	LW	8	1½	1½	1½	1½	1½
Unrestrained	Cold-drawn	NW	>12	1½	1¾	2	2½	3
Unrestrained	Cold-drawn	LW	>12	1½	1½	1¾	2	2½
Unrestrained	H.S.A. bars	NW	>12	1½	1½	1½	1½	2
Unrestrained	H.S.A. bars	LW	>12	1½	1½	1½	1½	2
Restrained	Cold-drawn	NW	>12	1½	1½	1½	1¾	2
Restrained	Cold-drawn	LW	>12	1½	1½	1½	1½	1¾
Restrained	H.S.A. bars	NW	>12	1½	1½	1½	1½	1½
Restrained	H.S.A. bars	LW	>12	1½	1½	1½	1½	1½

*NW = normal weight; LW = lightweight.
[†]For beams with widths between 8 and 12 in., cover thickness can be determined by interpolation.
[‡]Not practical for 8-in. wide beam but shown for purposes of interpolation.
1 in. = 25.4 mm.

cation period or 1 hr, whichever is longer. The recommended cover thicknesses in Table 2.3 are based on these criteria. For post-tensioned beams or joists less than 8 in. (203 mm) wide utilizing strand tendons, ACI 216R can be used. Beams or joists that are narrower than 8 in. (203 mm) with post-tensioned high-strength alloy steel bars should have the same cover as reinforced concrete joists of the same size and fire endurance.

2.4.1.3 *Anchor protection*—The cover to the prestressing steel at the anchor should be at least ¼ in. (6.4 mm) greater than that required away from the anchor. Minimum cover to the steel bearing plate or anchor casting should be at least 1 in. (25 mm) in beams and ¾ in. (19 mm) in slabs.

2.4.2 *Rational design for fire endurance*—Rational analytical procedures for the determination of the fire endurance of post-tensioned prestressed concrete structures have been developed from analyses of results of fire tests conducted in accordance with the criteria for standard fire tests, ASTM E 119. Basic data on the strength-temperature relationships for steel and concrete are utilized together with information on temperatures within concrete beams and slabs during standard fire tests. Rational design procedures for concrete beams and slabs which are post-tensioned with unbonded tendons are essentially the same as those for pretensioned prestressed concrete elements.[9] Curved tendons, rather than straight or deflected tendons, introduce only minor differences that do not change the design procedures. Tests of post-tensioned elements indicate that the temperatures of the tendons in positive moment regions at the end of a fire test can be considered essentially the same regardless of whether the tendons are bonded or unbonded. Further, these tests indicate that the prestressing steel stress $f_{ps\theta}$ at failure during fire tests can be estimated as a function of the ultimate steel strength at temperature θ by the relationship

$$\frac{f_{ps\theta}}{f_{pu\theta}} = \frac{f_{ps}}{f_{pu}}$$

where f_{ps} = stress in post-tensioning tendons at nominal strength, psi (Pa). This stress may be calculated for unbonded tendons by Eq. (18-4) or Eq. (18-5) in ACI 318 (see also Section 3.4).

f_{pu} = specified tensile strength of tendons, psi (Pa)
$f_{ps\theta}$ = stress in post-tensioned tendons at nominal strength at high temperatures, psi (Pa)
$f_{pu\theta}$ = tensile strength of tendons at high temperatures, psi (Pa)

For continuous beams or slabs utilizing continuous draped unbonded tendons exposed to fire from below, the value of $f_{ps\theta}$ in the negative moment regions should be taken the same as those in the positive moment region. The capacity at any point along the length of an unbonded tendon is limited by the capacity at the point where the steel temperature is highest.

On this basis, it is possible to determine the retained theoretical moment strength at a specified period of fire endurance (say 2 hr) in the positive moment region and in both negative moment regions of a given panel in a building. The maximum moment capacity at exterior columns should not exceed that which can be transmitted to the column. To evaluate the retained theoretical moment strength, it may be assumed that if a fire occurs beneath the floor, a redistribution of moments will occur, yielding the negative moment bonded reinforcement. If the applied midspan moment is less than the retained moment capacity after redistribution, the fire endurance will be adequate. This is

$$M = M_{t\theta}^{+} + \tfrac{1}{2}\,(M_{t1\theta}^{-} + M_{t2\theta}^{-})$$

$$M = \text{total static moment (unfactored)} = \frac{wL^2}{8}$$

where

$M_{t\theta}^{+}$ = retained midspan moment
$M_{t1\theta}^{-}$ = retained negative moment at Column 1
$M_{t2\theta}^{-}$ = retained negative moment at Column 2

If, however, the applied midspan moment is greater than the retained moment capacity, changes should be made in the design. Several options for improving the fire endurance are available, including:

1. Increase the concrete cover in the positive moment region.
2. Increase the number of prestressing tendons.
3. Add positive moment reinforcing steel.
4. Add negative moment reinforcing steel.
5. Of course, there are other solutions, such as the use of a thicker slab, lightweight concrete, or the addition of a fire-resistant ceiling. Also, combinations of the options just listed can be used. The most appropriate solution depends on in-place cost, architectural acceptability, and perhaps other considerations. For example, to upgrade the fire endurance of an existing floor, Options 1 through 4 are not applicable, so either an undercoat or a ceiling might be most appropriate. Very often the best solution at the design stage is the addition of some reinforcing steel that improves not only the fire endurance but also the overall strength and ductility of the floor.

2.5—Earthquake loading

Most concrete structures located in areas subject to seismic disturbances that include post-tensioned elements in the gravity load-carrying structural system are provided with shearwalls, braced frames, or reinforced concrete ductile moment-resisting space frames for resisting lateral forces due to wind and earthquakes. Most, if not all, model building codes in the U.S. currently contain minimum seismic design criteria based upon the requirements and commentary published by the Seismology Committee of the Structural Engineers Association of California.[9] These codes (for Zones 3 and 4) do not permit concrete ductile moment-resisting space frames in which prestressed reinforcement is used as the principal reinforcement; bonded nonprestressed

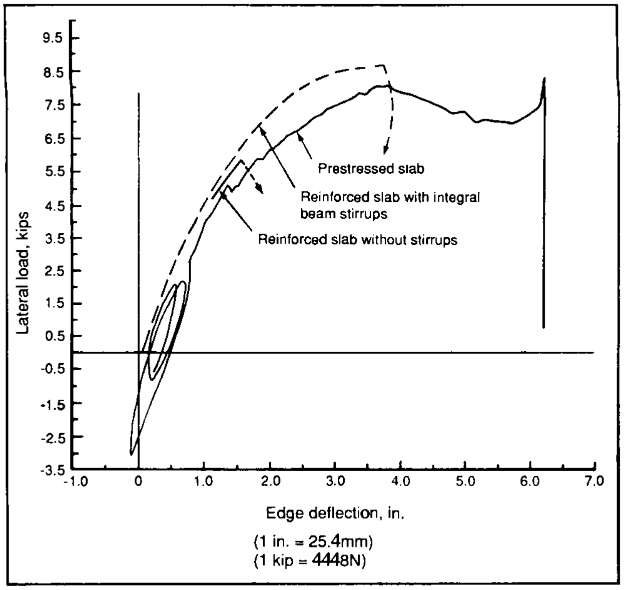


Fig. 2.1—Comparison of lateral load-edge deflection relationships for reinforced and prestressed concrete slab-interior column specimen[11]

reinforcement must be used, which conforms to special limitations on the maximum yield strength and the minimum tensile strength.

The model codes also contain a provision that all framing elements not required by design to be part of the lateral force resisting system, must be capable of resisting moments induced by the distortions of the structure resulting from lateral forces in addition to the moments caused by vertical loads; this applies to prestressed concrete elements as well as to those composed of other materials. It has been shown that under-reinforced prestressed concrete elements (i.e., those with combined steel indexes not greater than 0.30 as provided in Section 18.8.1 of ACI 318) can meet ductility requirements of this code provision.[10] Fig. 2.1[11] shows that after low-intensity reversed cyclic loading of interior column-slab specimens, conventionally reinforced slabs required the addition of closely spaced stirrup reinforcement to attain ductility comparable to that of a post-tensioned slab. As noted, the use of bonded or unbonded tendons in members of ductile moment-resisting frames that must resist seismic forces is restricted by U.S. building codes in high-seismicity areas. Since strains in an unbonded tendon are distributed over the length of the tendon, the tendons would not be expected to be stressed beyond the elastic range, even in a severe earthquake. As a result, the tendons do not dissipate much energy. Both laboratory tests and field experience indicate that this objection may be overcome by the use of elements containing a combination of unbonded tendons and nonprestressed bonded reinforcement.

Laboratory tests of post-tensioned structural elements have indicated that energy dissipation characteristics under seismic loadings conforming with accepted standards can be achieved by appropriate combinations of prestressed and nonprestressed (bonded) reinforcement.[12-16] In addition to these laboratory tests, which deal with members having both bonded and unbonded tendons, several midrise and high-rise structures incorporating unbonded tendons in ductile frame members resisted high lateral forces during the 1971 San Fernando earthquake with no structural distress. In the design of these structures, the contribution of the tendons as tensile reinforcement under seismic loading was neglected, but the moments induced in the frame by tendon action were considered. Grade 60 reinforcing bars were provided for moment capacity and to supply energy dissipation. Since the tendons were not stressed beyond the elastic range, they reduced the deterioration of shear capacity by providing a nearly constant "shear friction" force at beam-column joints. Recent observations (1987) of unbonded tendon anchorages following the construction failure of a flat plate lift-slab structure demonstrated the integrity of the anchorages even after collapse of the structure, tensile failure of the strand, and shattering of the end blocks.[17]

Post-tensioned beams may be proportioned to be more slender than conventionally reinforced members. This reduction in beam section stiffness can offset the increase in stiffness resulting from prestressing (reduced inelastic hinge lengths), and the overall performance of the frame compares favorably with conventional ductile frames.

Results of recent high-intensity reversed cyclic loading tests[18] of specimens representing concrete ductile moment-resistant frames with unbonded post-tensioned beams indicated that post-tensioning did not adversely affect the seismic characteristics of the specimens. This test report recommends that the nominal average prestress, based on the rectangular cross-sectional area of the beam, should be limited to approximately 350 psi. The stiffness after seismic loading of the post-tensioned frame specimens was larger than the stiffness of the non-post-tensioned specimen. Post-tensioning improved the behavior of nonprestressed reinforcement in the beam-column connection.

Standard specifications for anchorage systems for unbonded tendons[8] contain static and dynamic test requirements that are more severe than would be anticipated in an earthquake of high intensity. These specifications also require anchorages for unbonded tendons to meet fatigue test requirements.

CHAPTER 3—DESIGN

3.1—General

The design provisions of Chapter 18 of ACI 318 apply to the contents of this chapter, but some recommendations are offered that differ from those of the Building Code.

3.2—One-way systems

3.2.1 *Minimum bonded reinforcement*—The minimum bonded reinforcement specified in Section 18.9.2 of ACI 318 is considered adequate to limit crack widths due to dead load and live load by crack distribution.[19-21] As discussed in Section 2.2.1 of this report, this amount of reinforcement also provides an alternate load-carrying system in the event of a catastrophic failure or abnormal loading in one span of a continuous one-way post-tensioned element with unbonded tendons. For this reason, it is recommended that bonded reinforcement used as part of the design moment strength or intended to provide an alternate load path in one-way systems be detailed in accordance with the provisions of Chapter 12 of ACI 318. Slab reinforcement spacing requirements specified in Section 7.6.5 of ACI 318 are not applicable to bonded reinforcement in unbonded post-tensioned slabs.

In one-way slabs, economical use of the minimum bonded reinforcement specified in Section 18.9.2 of ACI 318 leads to the use of design tensile stresses in the range of 9 $\sqrt{f_c'}$ psi (0.75 $\sqrt{f_c'}$ MPa) to 12 $\sqrt{f_c'}$ psi (1.00 $\sqrt{f_c'}$ MPa). Tests have shown satisfactory performance of slabs with this level of design tensile stress in conjunction with the bonded reinforcement requirements of Section 18.9.2.[19] However, the use of lower design tensile stresses may be preferable from the durability standpoint for applications such as parking structure decks in severe climates.[22]

Section 18.8.3 of ACI 318 requires a total amount of bonded and unbonded tendons adequate to develop a factored load at least 1.2 times the cracking load based on the modulus of rupture f_r of 7.5 $\sqrt{f_c'}$specified in Section 9.5.2.3 of ACI 318. This provision is included to guard against an abrupt flexural failure at cracking due to rupture of the reinforcement. In contrast to this brittle failure mode, tests of one-way slabs and beams have demonstrated that unbonded tendons do not rupture and generally do not even yield at the time of flexural cracking.[19-21] Further, the minimum amount of bonded reinforcement required by Section 18.9.2 of ACI 318 for one-way post-tensioned members equals or exceeds the minimum reinforcement requirements for conventionally reinforced members. Since all one-way post-tensioned members will have some unbonded post-tensioned reinforcement in addition to the minimum bonded reinforcement, the total minimum reinforcement will in all cases exceed the minimum for conventionally reinforced one-way members by a substantial margin.

For this reason, and considering the fact that unbonded tendons do not yield or rupture at cracking, it is recommended that Committee 318 waive the minimum reinforcement requirement of Section 18.8.3 (1.2 times the cracking load) for one-way beams and slabs with unbonded tendons, and that Section 18.8.3 be revised to exclude application to one-way beams and slabs with unbonded tendons. Section 18.8.3 usually does not control reinforcement requirements in post-tensioned T-beams and one-way joists.

For applications of Eq. (18-6) of ACI 318 (see following) to negative moment areas in T-beam and joist construction, the flange width should be the minimum width that will provide section properties that will satisfy the 0.45 f_c' service load compressive stress limitation at the bottom of the beam or stem. The top fiber tensile stress limitation should also be checked. The total bonded and unbonded reinforcement supplied should also satisfy flexural design strength requirements without exceeding the limiting ratio of prestressed and nonprestressed reinforcement of ACI 318, Section 18.8.1

$$A_s = 0.004A \qquad (18\text{-}6)$$

where

A_s = area of nonprestressed tension reinforcement, in.^{2}
A = area of that part of cross section between flexural tension face and center of gravity of cross section, in.^{2}

3.2.2 *Tendon spacing*—The minimum bonded reinforcement requirements for one-way slabs under current code provisions, as discussed previously, typically result in the use of #4 bars (12.7 mm diameter) at 21 in. (533 mm) centers for both positive and negative moments for a 4½ in. (114 mm) thick slab. For an 8 in. (203 mm) deep one-way slab, #4 bars (12.7 mm diameter) are required at about 12 in. (305 mm) centers; larger bars are required at somewhat wider spacings. In consideration of this amount and spacing of bonded reinforcement, maximum tendon spacings in the range of six to eight times the slab thickness are recommended for one-way slabs, without the additional restriction of a minimum prestress level of 125 psi (0.86 MPa) specified for two-way slabs in Section 3.3.5.

3.2.3 *Minimum stirrups*—A minimum amount of stirrup reinforcement is necessary in all post-tensioned joists, waffle slabs, and T-beams to provide a means of supporting tendons in the tendon design profile. When tendons are not adequately supported by stirrups, local deviations of the tendons from the smooth parabolic curvature assumed in design may result during placement of the concrete. When the tendons in such cases are stressed, the deviations from the intended curvature tend to straighten out, and this process may impose large tensile stresses in webs of post-tensioned beams, joists, or waffle slabs.

Severe cracking has been observed in several instances where no stirrups were provided. Unintended curvature of the tendons may be avoided by securely tying tendons to stirrups that are rigidly held in place by other elements of the reinforcing cage. For bundles of two to four monostrand tendons, ties to a minimum of No. 3 (9.5 mm diameter) stirrups at 2 ft 6 in. (0.76 m) centers are suggested, and for bundles of five or more monostrand tendons, ties to a minimum of No. 4 stirrups (12.7 mm diameter) at 3 ft 6 in. (1.07 m) centers are recommended. This amount and spacing of stirrups is recommended even when the magnitude of the shear stress is such that no stirrups are required under the provisions of Section 11.5.5 of ACI 318. In most cases, closer stirrup spacings will be required to satisfy the shear reinforcement requirements of ACI 318.

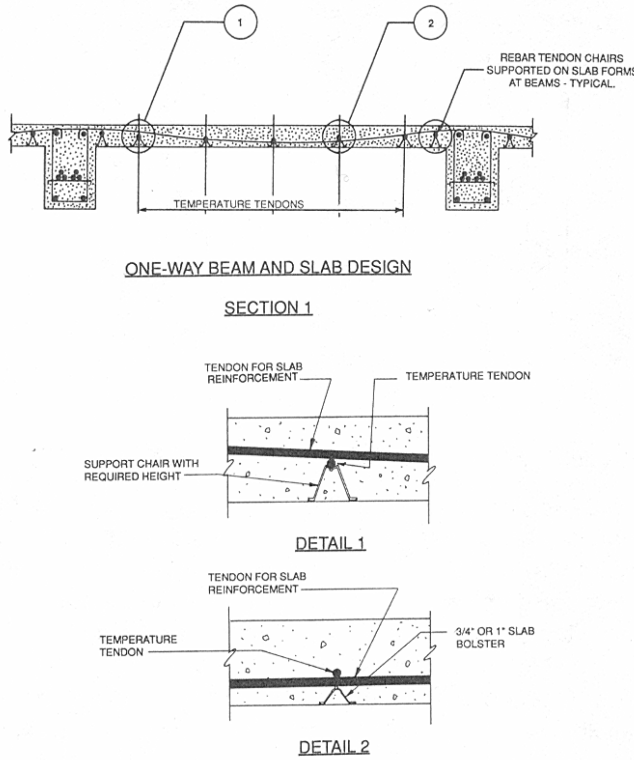


Fig. 3.1—Details for use of unbonded tendons as shrinkage and temperature reinforcement in one-way beam and slab construction

3.2.4 *Prestressed shrinkage and temperature reinforcement*—In Section 7.12 of ACI 318, prestressed shrinkage and temperature reinforcement may be used that has a minimum average compressive stress of at least 100 psi (0.7 MPa) on the gross concrete area using the effective stress in the prestressing steel, after losses, in conformance with Section 18.6 of ACI 318.

In monolithic cast-in-place post-tensioned beam and slab construction, the portion of a slab that is used as a beam "flange" should satisfy the minimum reinforcement requirements of Chapter 18 of ACI 318 applicable to the beam. In addition, in positive moment areas, the slab should meet the requirements of Section 7.12 of ACI 318 unless a compressive stress of 100 psi (0.7 MPa) is maintained under prestress plus dead load. In the central region of the bay between beam flanges, additional tendons should be used to provide 100 psi compression (0.7 MPa) in the portion of the slab that is not used as a part of the beam. Tendons used for shrinkage and temperature reinforcement should be positioned vertically as close as practicable to the center of the slab. In cases where shrinkage and temperature tendons are used for supporting the principal tendons, variation from the slab centroid is permissible. However, the resultant eccentricity of the shrinkage and temperature tendons should not extend outside the kern limits of the slab. Fig. 3.1 illustrates details for the use of unbonded tendons as shrinkage and temperature reinforcement in one-way beam and slab construction.

3.2.5 *T-beam flange width*—The effective flange width of post-tensioned T-beams in bending may be

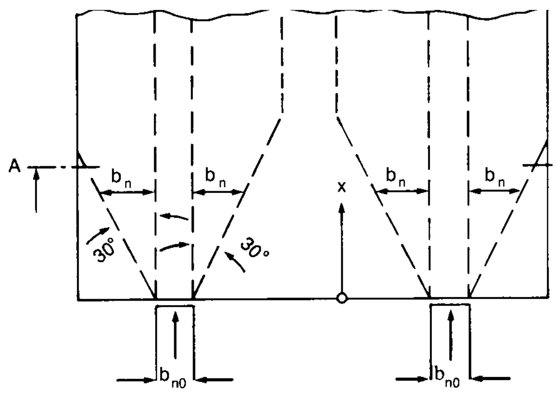


Fig. 3.2—Effective flange widths for normal forces

taken in accordance with Section 8.10 of ACI 318, or may be based on elastic analysis procedures. Flange widths in excess of those specified for conventionally reinforced concrete T-beams in ACI 318, Section 8.10 have been used (see ACI 318 Commentary, Fig. 7.12.3). The effective flange width for normal forces due to post-tensioning anchorages may be assumed in accordance with Fig. 3.2 as $2b_n + b_{no}$.

3.3—Two-way systems

3.3.1 *Analysis*—Prestressed slab systems reinforced in more than one direction for flexure should be analyzed in accordance with the provisions of Section 13.7 of ACI 318 (excluding Sections 13.7.7.4 and 13.7.7.5) or by more precise methods. The equivalent frame method of analysis has been shown by tests of large structural models to satisfactorily predict factored moments and shears in prestressed slab systems.[1,3,4,5,23,24] The referenced research also shows that analysis using prismatic sections or other approximations of stiffness may provide erroneous results on the unsafe side. Section 13.7.7.4 is excluded from application to prestressed slab systems because it relates to reinforced slabs designed by the direct design method and because moment redistribution for prestressed slabs is covered in Section 18.10.4 of ACI 318. Section 13.7.7.5 is excluded from application to prestressed slab systems because the distribution of moments between column strips and middle strips required by Section 13.7.7.5 is based on analysis of elastic slabs plus tests of reinforced concrete slabs. Simplified methods of analysis using average coefficients do not apply to prestressed concrete slab systems. All other provisions of Section 13.7, specifically including the arrangement of live loads specified in Section 13.7.6, are applicable for the analysis of post-tensioned flat plates.

If the probability of cracking of the slab is small, the lateral load stiffness should be assessed using ACI 318, Section 13.7. If, however, there is a high probability of cracking, the cracked section bending stiffness should be used and the torsional stiffness taken as one-tenth that calculated from Eq. (13-7) of ACI 318.[11] The cracked section bending stiffness should always be used for the computation of drift under seismic loads. Strength under lateral loads may be evaluated using the load factor combinations of Section 9.2 of ACI 318 in conjunction with the provisions of Section 18.10.3 of ACI 318. Evaluation of strength requirements under lateral loads may disclose the need for reinforcement for moment reversals. Such reinforcement should be located within a distance of 1.5h outside opposite faces of the column.

3.3.2 *Limits for reinforcement*—It is recommended that Committee 318 waive the requirement of Section 18.8.3 of ACI 318 for a total amount of prestressed and nonprestressed reinforcement sufficient to develop 1.2 times the cracking load for two-way post-tensioned systems with unbonded tendons. Due to the very limited amount and extent of the initial cracking in the negative moment region near columns of two-way flat plates, load-deflection patterns do not reflect any abrupt change in stiffness at this point in the loading history.

Only at load levels beyond the design (factored) loads is the additional cracking extensive enough to cause an abrupt change in the load-deflection pattern. Tests have also shown that it is not possible to rupture (or even yield) unbonded post-tensioning tendons in two-way slabs prior to a punching shear failure.[1,3,11,4,5,23,25-27] The use of unbonded tendons in combination with the minimum bonded reinforcement requirements of Sections 18.9.3 and 18.9.4 of ACI 318 has been shown to assure post-cracking ductility and that a brittle failure mode will not develop at first cracking.

3.3.3 *Minimum bonded reinforcement*—Minimum bonded reinforcement in negative moment areas of two-way systems is governed by Eq. (18-8) of ACI 318

$$A_s = 0.00075h\ell$$

This amount of bonded reinforcement is required within a slab width between lines that are $1.5h$ outside opposite faces of the column support. Tests on square panel specimens have shown a steel area of 0.00075 A_c' to be adequate to assure sufficient punching shear strength, where A_c' is the tributary cross-sectional area of the slab between panel centerlines perpendicular to the bonded reinforcement.[3-5,26-28] This value was expressed in the code as $0.00075h\ell$, where ℓ is the span in the direction of the reinforcement, to generalize the expression for rectangular panels, placing more bars in the direction of the longer span. The use of $h\ell$ as opposed to A_c' is appropriate to determine bonded reinforcement requirements at the interior columns and reinforcement perpendicular to the slab edge at exterior columns.

Tests[3-5,26-28] show that it is appropriate to provide bonded reinforcement parallel to the slab edge at exterior columns on the basis of 0.00075 A_c' where A_c' is the tributary cross-sectional area of the slab perpendicular to the direction of the bonded reinforcement between the center of the exterior span and the slab edge. At exterior columns of flat plates with square panels and no projection of the slab beyond the exterior column face, the bonded reinforcement parallel to the slab edge should be 50 percent of the bonded reinforcement perpendicular to the slab edge. Bonded reinforcement in positive moment areas of two-way flat plates is required where the computed tensile stress in the concrete at service load exceeds 2 $\sqrt{f'_c}$ psi, (0.17 $\sqrt{f'_c}$ MPa). The amount of positive moment bonded reinforcement, when required, is specified by Eq. (18-7) of ACI 318

$$A_s = \frac{N_c}{0.5 f_y} \qquad (18\text{-}7)$$

where the specified yield strength of nonprestressed reinforcement f_y shall not exceed 60,000 psi (414 MPa), and N_c is the tensile force in concrete due to unfactored dead load plus live load $D + L$. Details of placement for the reinforcement provided in this section are included in Section 3.3.5. Slab reinforcement spacing requirements specified in Section 7.6.5 of ACI 318 are not applicable to bonded reinforcement in unbonded post-tensioned slabs.

3.3.4 *Shear and moment transfer*—Fig. 3.3 shows the results of single column-slab specimen punching shear tests and results of multipanel slabs tested in shear.[27] Eq. (11-37) expressed in terms of the perimeter of critical section for slabs b_o is

$$V_{c\,(lb)} = b_o d\,(3.5\,\sqrt{f'_c} + 0.3\,f_{pc}) + V_p \qquad (11\text{-}37)$$

$$V_{c\,(N)} = (0.29\,\sqrt{f'_c} + 0.3\,f_{pc})\,b_o d + V_p \qquad (11\text{-}37)\ \text{SI}$$

f_{pc}= average compressive stress in concrete due to effective prestress force only (after allowance for all prestress losses); V_c = permissible shear force at design (factored) load; and V_p = vertical component of effective prestress force at section.

An upper limit of 500 psi (3.45 MPa) and a lower limit of 125 psi (0.86 MPa) are specified for f_{pc}. For values of precompression less than 125 psi (0.86 MPa), shear is limited to the value obtained using Eq. (11-36) of ACI 318 as for nonprestressed construction. For thin slabs, V_p must be carefully evaluated, as field placing practices can have a great effect on the profile of the tendons through the critical section. V_p may be conservatively taken as zero.

Moment transfer from prestressed concrete slabs to interior column connections can be evaluated using the procedures of Section 11.12.2 and 13.3.4 of ACI 318.[11] In this case, for normal weight concretes, the design shear stress v_u should not exceed the value of v_c calculated from Eq. (11-37) of the code expressed in terms of shear stress rather than force. The value of f_{pc} used in Eq. (11-37) should be the average precompression in the direction of moment transfer. All reinforcement, bonded and unbonded, within lines one and one-half times the slab thickness on either side of the column, is effective for transferring the portion of the moment not transferred by shear. No increase in forces for unbonded tendons should be assumed in calculations of the moment transfer capacity. Tendons bundled

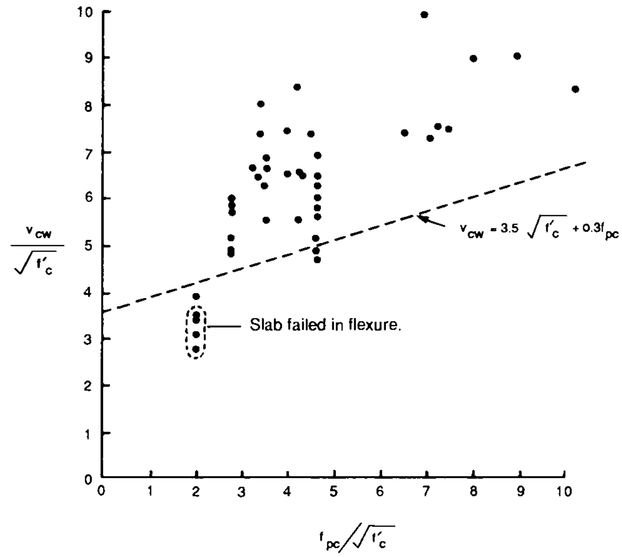


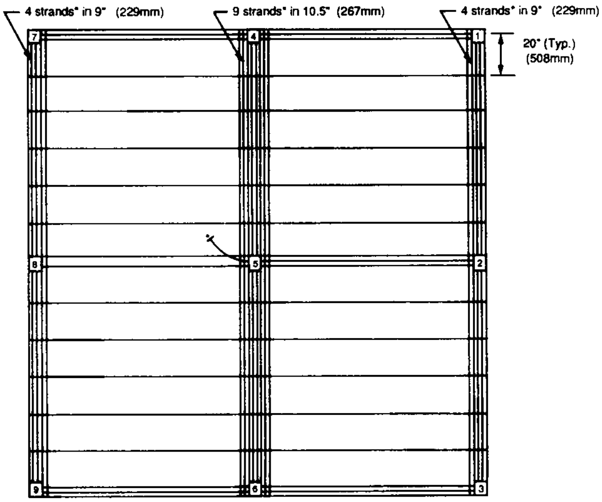
Fig. 3.3—Two-way flat plate shear test data versus Eq. (11-37) of ACI 318[27]

through the column or over the lifting collar in lift slabs are an effective means of increasing the moment transfer strength of lift-slab connections. The moment transfer strength of lift-slab connections is also controlled by details of the lift-slab collar-to-column connection.

The procedures of Sections 11.12.2 and 13.3.4 of ACI 318 are also applicable to calculations of the moment transfer from prestressed concrete slabs to exterior column connections for moments normal to a discontinuous edge. However, bonded reinforcement, detailed as closed ties or hooks so that it can act as torsional reinforcement, should be provided when the calculated upward design shear stress v_u at the discontinuous edge exceeds 2 $\sqrt{f'_c}$ psi (0.17 $\sqrt{f'_c}$ MPa), and, until further research data become available, the maximum calculated shear stress at such exterior columns should be limited to 4 $\sqrt{f'_c}$ psi (0.33 $\sqrt{f'_c}$ MPa). However, tests completed in 1982 of four edge column specimens of a post-tensioned flat plate with banded tendon details, support the use of Eq. (11-37) of ACI 318 for shear design.[28]

The limited test data available[27,29] do not show beneficial effects on shear strength due to use of shear reinforcement with conventional anchorage details in post-tensioned flat plates. The use of stud shear reinforcement with special anchorage details and stirrups with special anchorage details has been shown to increase shear strength substantially.[30-33]

3.3.5 *Tendon and bonded reinforcement distribution and spacing*—Within the limits of tendon distributions that have been tested, research indicates that the moment and shear strength of two-way prestressed slabs is controlled by total tendon strength and by the amount and location of nonprestressed reinforcement, rather than by tendon distribution.[2-5,11,24] While it is important that some tendons pass within the shear perimeter over columns, distribution elsewhere is not critical, and any rational method which satisfies statics may be used. For



*1/4 in. ϕ 240K (6.4mm ϕ 1656MPa) seven wirestrand was used to simulate strand tendons in model. Partial length tendon over interior column used for experimental purposes.

Fig. 3.4—Banded tendon distribution[5]

uniform loading, the maximum spacing of single tendons or groups of tendons in one direction should not exceed 8 times the slab thickness, with a maximum spacing of 5 ft (1.52 m). In addition, tendons should be spaced to provide a minimum average prestress of 125 psi (0.86 MPa) on the local slab section tributary to the tendon or tendon group (the section one-half of the spacing on either side of the center of the tendon or tendon group). The spacing of single strand tendons is usually governed by the minimum average prestress requirements. For groups of two or more tendons, the $8h$ criterion usually controls maximum tendon spacing. Special consideration of tendon spacing may be required to accommodate concentrated loads.

When more than two strands are bundled in a group, additional cover may be necessary to assure proper concrete placement under the tendon group. Horizontal curvature of bundled monostrand tendons should be avoided. If this is not possible, additional transverse reinforcement and accessories may be required at points of horizontal curvature to maintain the horizontal plane of tendon bundles during stressing.

The predominant and recommended method of placing tendons in two-way slab systems is the banded distribution illustrated in Fig. 3.4. The use of a banded tendon distribution greatly simplifies the process of placing tendons, and therefore provides a significant reduction in field labor cost. Recommended details of reinforcement for banded tendon distribution are given in the following paragraphs.

The number of tendons required in the design strip (center-to-center of adjacent panels) may be banded close to the column in one direction and distributed in the other direction. At least two tendons should be placed inside the design shear section at columns in each direction.

For lift-slab construction, the same general details of tendon distribution apply, and provision should be made for tendons to pass through or over the lifting heads.

The maximum spacing of tendons or bundles of tendons that are distributed should be $8h$ but not to exceed the spacing that provides a minimum average prestress of 125 psi (0.86 MPa). Even though no tendons are provided in one direction between bands, this maximum spacing assures one-way reinforcement for this part of the slab. Except for small triangular sections adjacent to the slab edges, the area between bands is also prestressed in both directions.

Recommended details for nonprestressed reinforcement are as follows:

a. Minimum A_s at columns is $A_s = 0.00075h\ell$ [Eq. (18-8) in ACI 318] where ℓ is the length of span in the direction parallel to that of the reinforcement being determined. At least four bars should be provided in each direction in negative moment areas at columns. As indicated in Section 3.3.3, the amount of bonded reinforcement parallel to the slab edge at exterior columns should be based on 0.00075 A_c' where A_c' is the tributary cross-sectional area of the slab perpendicular to the bonded reinforcement between the center of the exterior span and the slab edge.

b. Bonded reinforcement should be placed within a slab width between lines that are $1.5h$ outside opposite faces of columns (ACI 318, Section 18.9.3.3). Maximum spacing of these bars is 12 in. (305 mm).

c. The minimum length for negative moment bars is one-sixth the clear span on each side of support.

d. Where service load positive moment gives stress in excess of 2 $\sqrt{f_c'}$ psi (0.17 $\sqrt{f_c'}$ MPa) minimum bonded reinforcement is specified by Eq. (18-7) of ACI 318

$$A_s = N_c/0.5f_y$$

where N_c is the tensile force in concrete due to unfactored dead load plus live load $D + L$.

e. The minimum reinforcement for positive moment (when required) should have a length at least one-third the clear span with the bars centered in the positive moment area.

f. Where bonded reinforcement is used along with unbonded tendons based on strength requirements *(rather than minimum A_s), attention should be given to* the cutoff points, and they should be specified as per Chapter 12 of ACI 318.

3.4—Tendon stress at design load

Eq. (18-4) of ACI 318 was developed primarily from results of tests of beams and is limited to members with span-depth ratios of 35 or less[21]

$$f_{ps}\,(\text{psi}) = f_{se} + 10{,}000 + \frac{f_c'}{100\rho_p} \qquad (18\text{-}4)$$

$$f_{ps}\,(\text{MPa}) = f_{se} + 70 + \frac{f_c'}{100\,\rho_p} \qquad (18\text{-}4)\ \text{SI}$$

More recent tests have shown that Eq. (18-4) apparently overestimates the amount of stress increase in unbonded tendons in one-way slabs, two-way flat plates, and flat slabs with higher span-depth ratios.[34] Until a generally acceptable formula is developed, the capacity of one-way slabs, flat plates, and flat slabs should be calculated using ACI 318 formula [Eq. (18-5)] for design stress in unbonded tendons

$$f_{ps}\text{ (psi)} = f_{se} + 10{,}000 + \frac{f'_c}{300\,\rho_p} \qquad (18\text{-}5)$$

$$f_{ps}\text{ (MPa)} = f_{se} + 70 + \frac{f'_c}{300\,\rho_p} \qquad (18\text{-}5)\text{ SI}$$

In applying Eq. (18-4) and (18-5) just shown, an average of values calculated at positive and negative moment sections along the element should be used as the design value. Eq. (18-4) in ACI 318 satisfactorily represents the stress in unbonded tendons at design loads for beams and other members with span-depth ratios up to 35.[21]

3.5—Prestress losses

Prestress losses, considering the factors noted in Section 18.6 of ACI 318, should be calculated by the design engineer and stated on the design drawings. Articles have been published that make it possible to calculate reasonably accurate values for the various code-defined sources of loss without excessive effort.[35,36] For typical applications, the values of prestress loss given in Table 3.1 may be used in lieu of more detailed loss calculations. The loss values in Table 3.1 are based on use of normal weight concrete and on average values of concrete strength, prestress level, and exposure conditions.

Prestress losses may vary significantly above or below the values in Table 3.1 in cases where the concrete is stressed at low strengths, where concrete is highly prestressed, or in very dry or very wet exposure conditions. The loss values in Table 3.1 do not include losses due to friction or anchor seating losses. Design calculations should consider friction losses in accordance with Section 18.6.2 of ACI 318. Some portion of the friction loss can usually be offset by use of temporary initial tendon stresses in excess of 0.70 f_{pu}. Special consideration should be given to friction losses whenever tendons in excess of 100 ft (30.5 m) long are stressed from only one end.

For calculation of friction losses for greased unbonded strand tendons in plastic sheathing using the formulas in Section 18.6.2 of ACI 318, the friction factor μ, usually ranging from 0.05 to 0.25, and the wobble factor K, usually ranging from 5 to 15 $\times$ 10^{-4} /ft may be used for design calculations. It may be necessary to obtain more precise values for the friction-factor and wobble-factor coefficients to calculate tendon elongations during stressing to conform with the 7-percent tolerance specified in Section 18.18.1 of ACI 318 for comparing tendon force as measured by gage pressure and tendon elongation.

Table 3.1 — Approximate prestress loss values[8]

Post-tensioning tendon material	Prestress loss, psi	
	Slabs	Beams and joists
Stress-relieved 270k strand and stress-relieved 240k wire	30,000	35,000
Bar	20,000	25,000
Low-relaxation 270k strand	15,000	20,000

3.6—Average prestress

3.6.1 *Minimum average prestress*—The average prestress is defined as the total prestress force (after losses) divided by the total area of concrete. There has been much satisfactory experience in recent years in one-way slabs and flat plates with an average prestress of about 125 psi (0.86 MPa). Lower values have also been used successfully for short span applications. These short span applications can be characterized as having flexural stresses substantially below 6 $\sqrt{f'_c}$ (0.5 $\sqrt{f'_c}$ MPa), minimal volume-change effects, and vertical element stiffness such that restraint to shortening is minimized.

In view of the amount and distribution of bonded reinforcement required in one-way slabs as discussed in Sections 3.2.1 and 3.2.2, minimum average prestress is considered less significant for one-way slabs than for two-way slabs, which usually do not have bonded positive moment reinforcement in interior panels. For applications such as parking structures where control of cracking is very significant from the standpoint of improving durability against application of deicing chemicals, average prestress levels of the order 200 psi (1.39 MPa) are recommended.

3.6.2 *Maximum average prestress*—A high value of average prestress may induce excessive shortening due to elastic deformation and creep. A maximum average prestress of 500 psi (3.45 MPa) is considered appropriate for solid slabs if shortening will not cause problems. Detailing, as discussed in Section 3.7, to assure that restraint to immediate and long-term shortening does not interfere with the imposition of the calculated average prestress in the concrete, is of increasing importance as the average prestress is increased toward the maximum value.

3.7—Supporting walls and columns

When columns and walls have significant stiffness in the direction of prestress, consideration should be given to the effects of the mutual restraining actions of the slab, columns, and walls.[37] These restraining actions may result in cracks in either the slab or the supporting elements or both. This effect can be quite serious for long slabs with high shrinkage and creep.[38] Likewise, the effects of the prestressing forces on stiff supporting elements should be investigated. However, design and construction options are available to reduce the effects of the shortening on both the slab and the supporting elements, as discussed in the following paragraphs. The moments or stresses that occur over a period of time due to creep and shrinkage shortening are themselves reduced approximately 50 percent by creep.[39]

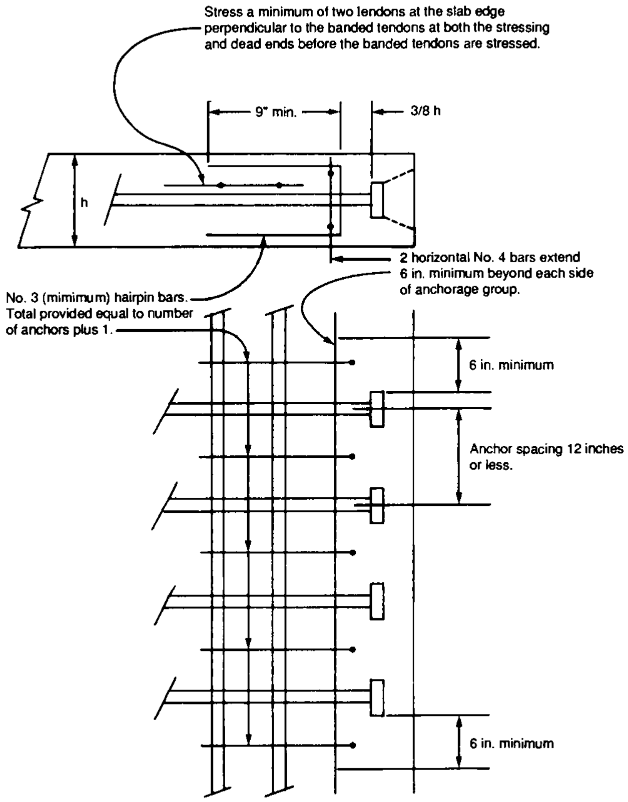


Fig. 3.5—Anchorage zone reinforcement for groups of ½ in. (13 mm) ϕ 270 k (1862 MPa) monostrand tendon anchorages in slabs[40]

Dimensional changes due to changes in temperature occur over a relatively short time period, and their effect would not be reduced by creep of the concrete. The restraining effects due to dimensional changes can be accommodated in the following ways:

a. Design or locate supporting elements to minimize restraint. Relatively long flexible columns may reduce restraint forces to the point where they can be accommodated easily by column reinforcement. Lateral load-resisting elements can often be located near the center of movement so that no restraint develops. Special consideration should be given to irregular layouts where a small slab area cannot deform with the overall deformations of the slab. In such cases, it is advisable to provide a complete structural separation between the two slab areas. Small areas may be designed as reinforced concrete when it is determined that post-tensioning cannot be used effectively.

b. Special consideration should be given to the effects of slab shortening on restraining walls and columns whenever slab lengths between construction joints exceed 150 ft (45.7 m). In such cases, the structure may be segmented with pour strips or temporary joints to minimize the movement and restraint developed during post-tensioning and due to early volume changes. Reinforcement, either prestressed or nonprestressed, should be provided to achieve continuity when the strip is closed with concrete. These strips should preferably be left open for a sufficient length of time to help minimize the effects of slab shortening. The design of reinforcement should be based on the amount of reinforcement required to achieve continuity, taking into consideration the deflection or camber that is expected to occur prior to casting the closure strip. Temporary shoring may be used to assure full continuity for both dead and live loads.

c. Detail the connection between the flexural elements and columns to permit movement.

d. Add or improve the layout of reinforcement. Bonded reinforcement placed parallel to restraining walls is highly effective in distributing potential restraint cracks. A reinforcement ratio of 0.15 percent with bars placed half at the top and half at the bottom over a width of about one-third of the span normal to the wall can be considered adequate for this purpose. The effect of potential diagonal cracks at slab corners, reentrant slab corners, and corners of walls can be similarly reduced by providing either diagonal or orthogonal bonded reinforcement. Diversion of prestress into supporting elements can be counteracted by overlapping tendons in these areas. Overlapping of tendons is recommended around openings to counteract potential diagonal cracks at the corners of the openings in accordance with Fig. 5.1.

In two-way flat plates, the average prestress is often on the order of 125 psi (0.86 MPa). Stresses of this magnitude do not usually produce large dimensional changes due to elastic shortening or concrete creep. However, even in these applications, care should be exercised when the building dimensions, or the dimensions between joints, become large, or when the flexural elements are supported by rigid elements that could produce substantial restraint forces if not properly detailed.

3.8—Serviceability requirements

Design for performance at service load should consider the factors included in Sections 9.5.4 and 18.10.2 of ACI 318. All serviceability limitations, including specified limits on deflections should be satisfied.

3.9—Design strength

The strength of prestressed systems should be at least equal to the required strength provisions contained in Section 9.2, 9.3, 18.10.3, and 18.10.4 of ACI 318.

3.10—Anchorage zone reinforcement

Anchorage zones in normal weight concrete slabs for groups of six or more ½ in. (13 mm) diameter single strand unbonded tendons with horizontal anchor spacing of 12 in. (305 mm) or less should be reinforced in accordance with Fig. 3.5[40] or with a similar detail using closed stirrups. The concrete strengths for the specimens tested in the research described in Reference 43 ranged from 2460 psi (16.9 MPa) to 2960 psi (20.4 MPa) to be representative of typical concrete strengths at the time of stressing tendons. Similar reinforcement should also be provided for anchorages located within 12 in. (305 mm) of slab corners. A minimum of two tendons at the slab edge perpendicular to the banded tendons at both the stressing end and dead end should be stressed preferably before the banded tendons are stressed.

The tests described in Reference 40 were limited to anchorages of ½ in. (13 mm) diameter, 270 ksi (1862 MPa) strand unbonded tendons in normal weight concrete. For anchorage of 0.6-in. (15 mm) diameter, 270 ksi (1862 MPa) strand tendons, or for anchorages used in lightweight concrete slabs, the amount and spacing of reinforcement should be conservatively adjusted to provide for the larger anchorage force and for the smaller splitting tensile strength of lightweight concrete. Tests are not available for anchorage zone reinforcement details for 0.6 in. (15 mm) diameter strand tendon anchorages, or for either 0.5 or 0.6 in. (13 or 15 mm) diameter strand tendon anchorages used in lightweight concrete.

For anchorage zones of groups of unbonded tendons in beams, the splitting tensile force may be taken as[8]

$$F_{st} = 0.30\left(1 - \frac{d_a}{d_{sp}}\right) P_j$$

in which d_a = depth of anchor casting (for a single line of anchors) or depth of section covered by a group of anchors; d_{sp} = total depth of symmetric concrete prism above and below a single anchor or group of anchors; and P_j = tendon jacking force for all tendons anchored in a group.

Reinforcement required for splitting tensile forces calculated in accordance with the previous equation should be proportioned with $f_s = 0.6 f_y$, where f_y should not exceed 60 ksi (414 MPa). Splitting reinforcement may not be required for tendon groups anchored in columns where confinement is provided by column loads and column reinforcement, or for anchorages where lateral confinement is provided by a beam perpendicular to the trajectory of the tendons which is monolithic with the slab and increases the depth of the section by at least the slab thickness above or below the slab.

Reinforcement may be in the form of spirals, stirrups, orthogonal reinforcement, or combinations of these. Groups of anchorages should be restrained with reinforcement (direction perpendicular to tendons) extending through the entire group of anchorages. All orthogonal reinforcement should be mechanically anchored around reinforcement running parallel to the tendons. Spirals, stirrups, or orthogonal reinforcement should have sufficient extra length to develop full bond with the concrete, or should be mechanically anchored by 135 bends around reinforcement. The clear distance between bars or pitch of spirals used as anchorage zone reinforcement should be at least the maximum size of the aggregate plus ½ in. (13 mm) but not less than 1½ in. (38 mm).

CHAPTER 4—MATERIALS

4.1—Tendons

4.1.1 *Specifications*—Prestressing steel for unbonded post-tensioning tendons should meet the requirements of Section 3.5.5 of ACI 318. The total elongation under ultimate load of the tendon and anchorage assembly should not be less than 2 percent measured in a minimum gage length of 10 ft (3.05 m).

4.1.2 *Anchorages*—The anchorages of unbonded tendons shall develop at least 95 percent of the minimum specified ultimate strength of the prestressing steel without exceeding the anticipated set. This is substantially in excess of the maximum possible design stress of unbonded tendons f_{ps}, discussed in Section 3.4.

4.1.3 *Tests of tendons and anchor fittings*

4.1.3.1 *Static tests*[8]—The test assembly should consist of standard production-quality components and the tendons should be at least 10 ft (3.05 m) long. The test assembly should be tested in a manner to allow accurate determination of the yield strength, ultimate strength, and percent elongation of the complete tendon. The specimen used for the static test need not be one that has been subjected to fatigue loading.

4.1.3.2 *Fatigue tests*[8]—A fatigue test should be performed on a representative test assembly, and the tendon should withstand without failure 500,000 cycles from 60 to 66 percent of its minimum specified ultimate strength, and also 50 cycles from 40 to 80 percent of its minimum specified ultimate strength. Each cycle involves the change from the lower stress level to the upper stress level and back to the lower.

The specimen used for the second fatigue test (50 cycles) need not be the same used for the first fatigue test although this is allowed since the specimen doesn't fail (500,000 cycles). Systems incorporating multiple strands, wires, or bars may be tested using a test tendon of smaller capacity than the full-size tendon. The test tendon should duplicate the behavior of the full-size tendon and generally should not have less than 10 percent of the capacity of the full-size tendon.

4.1.4 *Couplers*[8]—Unbonded tendons should be coupled only at locations specifically indicated and/or approved by the engineer. Couplers should not be used at points of sharp tendon curvature. All couplers should develop at least 95 percent of the minimum specified ultimate strength of the prestressing steel without exceeding the anticipated set. The couplers should not reduce the elongation at rupture below the requirements of the tendon itself.

Couplers should meet the fatigue test requirement recommended in Section 4.1.3.2. Couplers and/or components should be enclosed in housings long enough to permit the necessary movements. All coupler components should be completely protected with a coating material prior to final encasement in concrete.

4.2—Protection materials

4.2.1 *Coating*[6]—Unbonded single strand tendons should utilize a corrosion preventive coating in conformance with the Post-Tensioning Institute "Specification for Unbonded Single Strand Tendons."[6]

Galvanizing may be used to protect prestressing bar tendons that are to be left exposed. These coatings should be applied by the hot-dip process and in accordance with ASTM A 123. Other related components such as anchorages, fittings, couplers, and coupler bars may be protected by electrodeposited coatings of zinc, as per ASTM B 633 (Type LS) or of cadmium, as per ASTM B 633 (Type NS). This type of tendon, particularly if exposed, requires periodic inspection for corrosion protection integrity and is not recommended for locations where it can be damaged easily.

4.2.2 *Sheathing*—Sheathing for unbonded single strand tendons should conform to the requirements of the Post-Tensioning Institute "Specification for Unbonded Single Strand Tendons."[6]

4.2.3 *Ducts*—Ducts for unbonded tendons are similar to those for post-tensioned grouted tendons. They should be mortar and grease-tight and nonreactive with concrete, prestressing steel, or the filler material. Ducts should be completely filled with an approved corrosion-preventive greaselike material[6] injected under pressure. It is also necessary that a permanent grease cap be attached to and cover the tendon anchorage, and that both the grease caps and the tendon duct be grease-tight so that the corrosion preventive material cannot escape into the surrounding concrete.

4.3—Protection of anchorage zones

The anchorages of unbonded single strand tendons should be protected adequately from corrosion and fire. Except in special cases, anchorages should preferably be encased in concrete with details complying with the Post-Tensioning Institute "Specification for Unbonded Single Strand Tendons."[6]

Where concrete or grout encasement cannot be used, the tendon anchorage should be completely coated with a corrosion-resistant paint or grease equivalent to that applied to the tendons. A suitable enclosure should be placed where necessary to prevent the entrance of moisture or the deterioration or removal of this coating. The anchorage encasement should provide fire resistance at least equal to that required for the structure.

4.4—Concrete cover

Specification of concrete cover for unbonded tendons should consider the placement tolerances specified in Section 5.3.2 and the exposure conditions. The use of good-quality concrete, adequate cover, good construction practices, and a limit on the amount of water-soluble chloride in the concrete (refer to ACI 201.2R) are all necessary to assure long-term durability, particularly in aggressive environments. Use of at least the additional cover specified in ACI 318, Section 7.7.3.2, and consideration of a somewhat higher average prestress level, is recommended for applications exposed to deicer chemicals or for locations in the immediate vicinity of seacoasts. Extra cover cannot be a substitute for good-quality concrete.

Although research[41-44] and experience[22,45] have demonstrated the durability of unbonded tendons exposed to seawater and other aggressive environments, it is not recommended that unbonded tendons be utilized in applications directly exposed to seawater or other severe corrosive environments unless special corrosion-protection measures are taken. There are proprietary protection systems available that provide enhanced corrosion protection to the total tendon assembly for highly corrosive environments.[46]

CHAPTER 5—CONSTRUCTION

5.1—Construction joints

Construction joints may be used to divide the floor system into segments of suitable size for placement of concrete. Intermediate stressing anchors may be used at construction joints, or the tendons may run through the joint without anchors.

5.2—Closure strips

Open strips may temporarily separate adjacent slabs during construction as discussed in Section 3.7.

5.3—Placement of tendons

5.3.1 *Tendon profile*—The placement of tendons should closely follow the specified profile within the tolerances recommended in Section 5.3.2. Tendon profiles are maintained by tying to reinforcing steel, chairs, or other supports with wire ties. Ties should be installed so that they do not visibly imprint or dent the polyethylene or polypropylene sheathing. Recommendations for spacing of ties for bundles of unbonded tendons are presented in Section 3.2.3.

5.3.2 *Tolerances*—Vertical deviations in tendon location should be kept to ± ¼ in. (6.4 mm) for slab thickness dimensions less than 8 in. (203 mm), ⅜ in. (9.5 mm) in concrete with dimensions between 8 in. (203 mm) and 2 ft (610 mm), and ½ in. (13 mm) in concrete with dimensions over 2 ft (610 mm). These tolerances should be considered in establishing minimum tendon cover dimensions, particularly in applications exposed to deicer chemicals or saltwater environments where use of additional cover is recommended to compensate for placing tolerances. Slab behavior is relatively insensitive to horizontal location of tendons.

5.3.3 *Openings*—Deviations of tendons in the horizontal plane that may be necessary to avoid interferences such as openings, ducts, chases, inserts, etc., should be considered in view of potential cracking due to lateral forces. Appropriate means to avoid or control cracking include an adequately large radius of curvature, sufficient clearance of the tendons from the edge of an opening, a straight tendon extension beyond the opening corners, and hairpin reinforcement to transfer the lateral forces to the surrounding concrete.[47]

For larger openings where it is necessary to terminate some tendons at the opening, the "crack inhibiting" layout of tendons shown in Fig. 5.1[37] is recommended rather than the "crack promoting" layout. In some cases, it may be preferable to isolate small slab sections

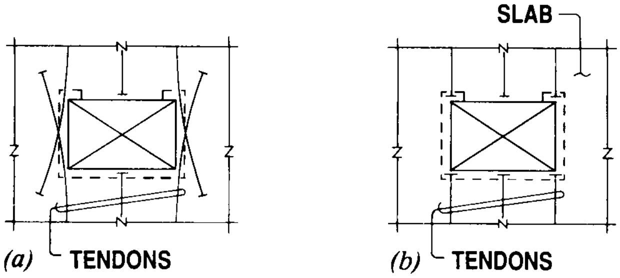


Fig. 5.1—Tendon layouts—(a) Crack inhibiting layout; and (b) crack promoting layout

adjacent to openings with slab joints, as shown in Fig. 5.2.[37] The isolated slab sections should be reinforced as required with conventional bonded reinforcement.

For larger openings, it is always desirable to reinforce the top and bottom of the slab at openings with diagonal bars to control cracking initiated at the corners of the opening. In some cases, additional structural reinforcement may be necessary around the slab perimeter to distribute any loads applied at the opening to the slab. Loads at openings can normally be accommodated by use of tendons and additional bonded reinforcement around the perimeter. However, additional beams may sometimes be required to carry the loads at perimeters of openings, and a structural analysis should be made to determine whether these loads can be carried by use of additional tendons and additional bonded reinforcement or whether beams are required. It is generally preferable to locate openings in the midspan areas of one-way slabs and two-way flat plates to minimize the effect of the opening on the shear capacity of the slab at walls or columns. When openings are located where they may reduce shear capacity, a more exact analysis of the capacity of the actual slab configuration is essential. In flat plates, Section 18.12 of ACI 318 requires that: "A minimum of two tendons shall be provided in each direction through the critical shear section over columns," as discussed in Section 3.3.5.

5.4—Concrete placement and curing

Concrete should be placed in such a manner that tendon alignment and reinforcing steel positions remain unchanged. Special attention must be given to vibration of concrete at tendon anchorages to insure uniform compaction at these points. Voids behind the bearing plate, or insufficient concrete strength, will cause concrete failure. Careful vibration and proper curing will eliminate most of these difficulties. Voids behind the bearing plate should be repaired prior to the stressing operation.

Curing in accordance with the recommendations in ACI 308 and ACI 517.2R should be followed to avoid various types of shrinkage-related cracking and to insure proper quality concrete. Calcium chloride or additives containing calcium chloride or other chlorides should not be used in prestressed concrete construction or in the material used to protect end anchorages. Set accelerators that do not contain calcium chloride are commercially available and may be used when required.

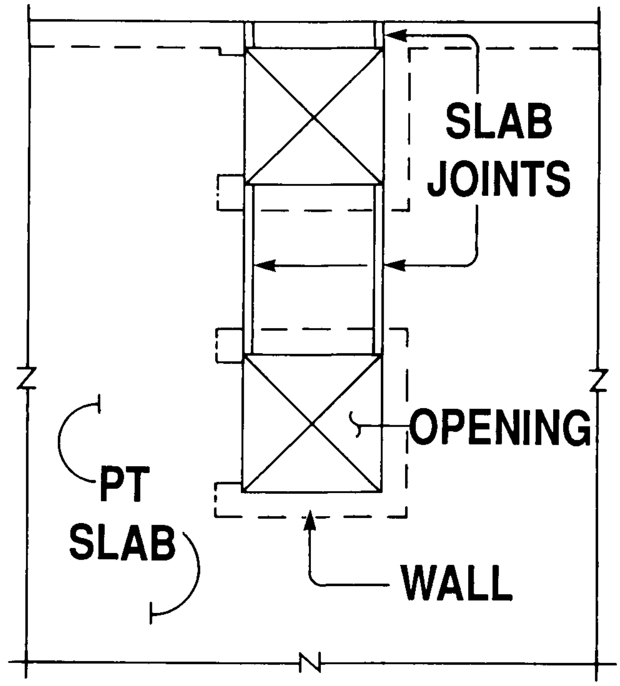


Fig. 5.2—Isolated slab sections

5.5—Stressing operations

The stressing operation may begin when test cylinders cured under job-site conditions and representative of the concrete strength in the immediate vicinity of the anchorages indicate that the concrete has attained the strength specified for stressing (usually 60 to 80 percent of the 28-day strength). Alternatively, nondestructive testing methods may be used to verify approximately the strength of the concrete in the structure. ACI 318 does not currently include nondestructive methods in its criteria for acceptance of concrete. However, such methods may be satisfactory for evaluating the concrete in the immediate vicinity of the anchorages.

Stressing of tendons should be monitored in two ways. First, the gage reading on the pump should be translated into force in the tendon at the anchorage. This information is generally provided in a tendon stressing data table supplied as part of the shop drawings. Second, the elongation of the tendon may be calculated using the formula

$$\Delta\ell = \frac{P\ell}{A_{ps} E_s}$$

where

$\Delta\ell$ = elongation in in. (mm)

P = average prestress force (considering friction effects along the length of the tendon) in lb (N)

ℓ = length of tendon in in. (mm)
A_{ps} = area of prestressing steel in in.^{2} (mm^2)
E_s = modulus of elasticity of prestressing steel, psi (MPa)

The moduli of elasticity of various post-tensioning tendon materials may be assumed as follows:[8]

Seven-wire strand:	E_s =	28,000,000 psi (193,000 MPa)
Wire:	E_s =	29,000,000 psi (200,000 MPa)
Bars:	E_s =	30,000,000 psi (207,000 Mpa)

A table of elongation values for various tendons on a project and/or a graphical presentation of expected elongations should be provided as part of the shop drawings for a project.

It is a requirement of ACI 318 that the tendon force measured by gage pressure agree within 7 percent of the tendon force calculated by elongation measurements (revision from 5 to 7 percent is proposed for ACI 318-89).[48] The cause of variations in force determination in excess of 7 percent must be ascertained and corrected. The modulus of elasticity of seven-wire strand varies somewhat from the 28,000,000 psi (193,000 MPa) average value suggested. Since a variation of 1,000,000 psi (6900 MPa) in the modulus of elasticity represents a difference of about 4 percent in elongation, it is always preferable to use the actual modulus of elasticity of the strand used on the project when comparing tendon elongation and gage pressure in the field.

The tendon elongation is affected by the variation in force due to friction losses throughout the tendon length. For this reason, friction losses should be considered in translating tendon elongation measurements into tendon forces. The elongation measurement provides a measure of the average force throughout the length of the tendon, whereas the gage pressure gives the force in the tendon at the anchorage. Methods for calculating the effects of friction along the length of the tendon are presented in ACI 318, Section 18.6.2

Stressing equipment for post-tensioning tendons incorporate reasonable factors of safety. Occasionally, flaws in material are undetected or the equipment may have been misused. For this reason, extreme caution should be exercised at all times, as stressing is carried out at extremely high pressure. The primary safety rule is to keep personnel from being directly in back, over, or under stressing equipment.

Failure during the stressing operation may cause serious injury to any personnel in back of or in the immediate vicinity of the stressing equipment. Should stressing reveal that voids exist behind the bearing plate, release all pressure on the equipment at once, remove the faulty concrete, and patch the void with suitable material. The patching material must attain the required strength before the tendon is restressed. Calcium chloride or admixtures that contain chloride ions should not be used in the patching operation.

5.6—Form removal and reshoring

Shoring must be left in place until the stressing operation is completed. Edge or pocket forms and bulkheads should be removed well ahead of the stressing operation. Beam or side forms may be removed prior to stressing with permission from the engineer.

Removal of shoring and forms may follow immediately after the stressing operation. After stressing, reshoring may be required to prevent overloading during additional construction. Usually, reshoring practices are a precaution against overloading. Do not wedge shoring beyond a snug fit against prestressed members.

5.7—Welding and burning

When welding or burning near tendons, care must be exercised to prevent the prestressing steel from overheating, to keep electric arc jumps from occurring, and to keep molten slag from coming in contact with the prestressing steel. Grounding of welding equipment to the prestressing steel should not be allowed.

CHAPTER 6—REFERENCES

6.1—Specified and recommended references

The documents of the various standards-producing organizations referred to in this document follow with their serial designation, including year of adoption or revision. The documents listed were the latest version at the time this document was written. Since some of these documents are revised frequently, generally in minor detail only, the user of this document should check directly with the sponsoring group if it is desired to refer to the latest revision.

American Concrete Institute

116R-85	Cement and Concrete Terminology
201.2R-77 (Reapproved 1982)	Guide to Durable Concrete
216R-81 (Revised 1987)	Guide to Determining the Fire Endurance of Concrete Elements
308-81	Standard Practice for Curing Concrete
318-83	Building Code Requirements for Reinforced Concrete
423.1R-69 (Reapproved 1980)	Tentative Recommendations for Concrete Members Prestressed with Unbonded Tendons
423.2R-74 (Reapproved 1980)	Tentative Recommendations for Prestressed Concrete Flat Plates
423.3R-83	Recommendations for Concrete Members Prestressed with Unbonded Tendons
517.2R-80	Accelerated Curing of Concrete at Atmospheric Pressure — State of the Art

ASTM

A 123-78	Standard Specifications for Zinc (Hot-Galvanized) Coatings on Products Fabricated from Rolled, Pressed, and Forced Steel Shapes, Plates, Bars, and Strip

B 633-78	Standard Specification for Electrodeposited Coatings of Zinc on Iron and Steel
E 119-81	Standard Methods of Fire Tests of Building Construction and Materials

International Conference of Building Officials
Uniform Building Code 1988

These publications may be obtained from the following organizations:

American Concrete Institute
P. O. Box 19150
Detroit, MI 48219-0150

ASTM
1916 Race Street
Philadelphia, PA 19103

International Conference of Building Officials
5360 South Workman Mill Road
Whittier, CA 90601

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These revisions were submitted to letter ballot of the Committee and approved in accordance with ACI balloting procedures.