Tentative Recommendations for Prestressed Concrete Flat Plates

Reported by ACI-ASCE Committee 423

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This report is presented as a guide to the design of prestressed concrete flat plates in buildings, post-tensioned with either bonded or unbonded tendons. The report presents: analysis and design procedures, permissible stresses for flexure and shear, recommendations on tendon distribution and spacing, nonprestressed reinforcement requirements, span-depth ratios for deflection and camber control, seismic design provisions, fire resistance rating criteria, and construction recommendations. The report is for the guidance and information of professional engineers who must add proper engineering judgment to applications of the recommendations.

Keywords: bending; bond (concrete to reinforcement); camber; compressive stress; **concrete slabs**; deflection; earthquake resistant structures; fire resistance; **flat concrete plates**; flexural strength; joints (junctions); loads (forces); **prestressed concrete**; prestressing steels; reinforcing steels; shear properties; stresses; **structural analysis**; **structural design**; tensile stress.

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CHAPTER 1 — INTRODUCTION

1.1—Scope

This report is presented as a guide to the design and construction of prestressed concrete flat plates in buildings, post-tensioned with either bonded or unbonded tendons. These recommendations are intended to supplement the ACI 318-71 Building Code.[1]

This report is a recommended practice, not a building code or specification. It is for the guidance and information of professional engineers who must add proper engineering judgment to applications of the recommendations.

In general, this report is applicable to flat plates constructed by the lift-slab method. However, the details of the lifting collars are a basic factor in determining the capacity of lift-slab structures, and these details are beyond the scope of this report. Capacity of lifting collars should be evaluated on the basis of research data or calculations pertinent to the specific details to be used.

This report does not include the design of post-tensioned flat slabs for bridges. This report is not intended for flat slabs (flat plates with drop panels) or waffle slabs.

1.2—Bonded and unbonded construction

For unbonded construction, reference is made to "Tentative Recommendations for Concrete Members Prestressed with Unbonded Tendons," ACI-ASCE Joint Committee on Prestressed Concrete, ACI JOURNAL, *Proceedings* V. 66, Feb. 1969, pp. 81-86.[2] These tentative recommendations are intended to enlarge upon and supersede Section 3.2 Flat Slabs of the above referenced ACI-ASCE joint committee report.

While these recommendations are primarily based on tests and experience with unbonded tendons, they are applicable to structures utilizing bonded tendons as well.

In the United States and Canada, economic and construction considerations have generally resulted in the selection of unbonded tendons for post-tensioned flat plates. These considerations include: higher friction during stressing of bonded tendons; protection of bonded tendons against corrosion during construction; and the problems associated with grouting procedures for large numbers of tendons in small diameter ducts. Load tests and laboratory research on slabs with unbonded tendons have indicated that cracking and deflection response under overloads near ultimate capacity are controlled adequately by use of nonprestressed bonded reinforcement in the negative moment areas over columns[3,4] (see Section 2.10).

1.3—Notation

Notation is the same as that used in ACI 318-71.

CHAPTER 2 — DESIGN

2.1—General

Bonded and unbonded prestressed concrete flat plates can be designed following ACI 318-71 with particular reference to Chapter 13 (excluding Sections 13.4.1.8 and 13.4.3), Slab Systems with Multiple Square or Rectangular Panels, in conjunction with Chapter 18, Prestressed Concrete. The purpose of this report is to point out certain applicable clauses of ACI 318-71 and to supplement them with additional recommendations. In design details, prestressed and conventionally reinforced flat plates differ to a large extent, especially with respect to the following points:

(a) Layout and details of reinforcement
(b) Secondary moments due to prestressing
(c) Permissible stress values
(d) Shrinkage and creep effects
(e) Camber and deflection.

Beams, openings and other interruptions of the slab may require special considerations beyond the scope of this report.

2.2—Secondary moments

In continuous post-tensioned structures, "secondary prestressing moments" are developed due to restraints to the deformations resulting from the primary prestressing moments. This is illustrated in Fig. 1. Here a two-span continuous beam has a post-tensioning force, P acting at a constant eccentricity, e. The moment created, Pe, is considered the "primary prestressing moment." If gravity loads are temporarily ignored, the moment due to post-tensioning will cause a theoretical upward deflection as shown. Should the beam be vertically restrained at the center support, the change in reactions resulting from the "primary prestressing moments" will be as shown in Fig. 1c. Because the upward deflection at the interior support is prevented, the secondary prestressing moments of Fig. 1d are induced. For continuous beams the secondary prestressing moments may be expressed as functions of the reactions and they always vary linearly between supports. The moment at any section may then be expressed as the superposition of the primary and secondary prestressing moments, Fig. 1e. The term "secondary prestressing moment" is used because the moment is induced by the primary prestressing moment, and not because the secondary prestressing moment is negligible or necessarily smaller than the primary prestressing moment.

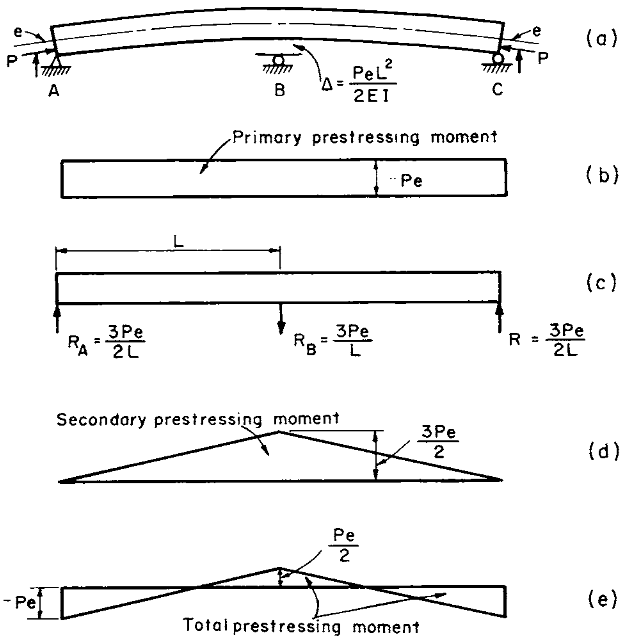

Fig. 1—Moments in a continuous post-tensioned beam

In practical applications, the total post-tensioning moments are obtained directly. If necessary, the secondary prestressing moment can be found by subtracting the primary prestressing moment, Pe, from the total moment. The total moment effect of the post-tensioning tendons on a member may be derived from the loads exerted on the member by the tendons.[5] Simplified design procedures based on load balancing[6] or equivalent load[7,8] concepts have been developed which eliminate the need to directly consider either primary or secondary prestressing moments due to post-tensioning. The load balancing procedure in conjunction with an equivalent frame model of the structure as described in Section 2.4 below, is a widely used approach to analysis of post-tensioned flat plates.

2.3—Loading

The loading arrangement used in design should conform to ACI 318-71, excluding Section 13.4.1.8, and the applicable building code.

2.4—Equivalent frame method of analysis

2.4.1 *General*—This section describes the equivalent frame method of analysis, also known as the beam method. This method of analysis utilizes the conventional elastic analysis assumptions and models the slab or slab and columns, as a beam or as a frame, respectively. This is the most widely used and applied method of analysis for post-tensioned flat plates.

The effect of vertical or lateral service and design loadings on post-tensioned flat plates, bonded or unbonded, may be analyzed as for rigid frames in accordance with the provisions of Sections 13.2.1, 13.4 (excluding 13.4.1.8 and 13.4.3), 18.12, and 18.13 of ACI 318-71. When columns are relatively slender or not rigidly connected to the slab (as in the case of lift slabs), their stiffness may be neglected and continuous beam analysis applied. The moments induced by prestressing may also be determined by a similar analysis of a rigid frame or continuous beam, using equivalent load or load balancing concepts. However, it should be kept in mind that the distribution of moments due to loads may differ considerably from the distribution of moments due to prestressing.[3,9] Service loads produce very pronounced moment peaks at columns, whereas the moment curve produced by post-tensioning has a more gentle undulating variation of the same form as the tendon profile.

The effects of reversed tendon curvature at supports are generally neglected in applying the load balancing method to design of flat plates since the reversed curvature has only a minor influence on the elastic moments (in the order of 5 to 10 percent), and does not affect the ultimate moment capacity. As noted in Section 2.9.3, it is necessary to consider reverse tendon curvature to adequately evaluate the shear carried by the tendons inside the critical section.

2.4.2 *Allowable stresses*—When using the equivalent frame (or beam) method of analysis, the following allowable stresses at service loads are recommended for the design of solid post-tensioned flat plates with bonded or unbonded tendons. Deviations from these values are permissible when the more rigorous methods of analysis specified in Section 2.5 are used, but it must be conclusively shown that the slab will perform satisfactorily under all design conditions.

2.4.2.1 *Compression in concrete.* This should be limited to 0.30 f_c' in negative moment areas around columns. The actual compressive stress in the bottom fibers of the slab around the columns is much higher than indicated by the equivalent frame analysis.[3] Therefore, the usual allowable compressive stress of 0.45 f_c' should be reduced to 0.30 f_c' in these areas. The allowable compressive stresses of Section 18.4, ACI 318-71 apply in the positive moment areas, because calculated positive moments and the resulting stresses are closer to the actual values.

2.4.2.2 *Tension in concrete.* In slabs with an average prestress* of 125 psi or higher, the following concrete tensile stresses can be permitted at service loads, after allowance for all prestress losses:

For positive moments without addition of nonprestressed reinforcement .... $2\sqrt{f_c'}$

For positive moments with the addition of nonprestressed reinforcement in accordance with Eq. (18-5) of ACI 318-71 .... $6\sqrt{f_c'}$

For negative moments without addition of nonprestressed reinforcement 0

For negative moments with addition of nonprestressed reinforcement (as defined in Section 2.7.4) in accordance with Section 2.10.1 .... $6\sqrt{f_c'}$

As listed below, these permissible tensile stresses and the related requirements for nonprestressed reinforcement vary significantly from the provisions of ACI 318-71.

1. The maximum permissible tensile stress is $6\sqrt{f_c'}$ which is a substantial reduction from the value of $12\sqrt{f_c'}$ permitted under Section 18.4 of ACI 318-71. Committee 423 does not recommend tensile stresses greater than $6\sqrt{f_c'}$ in post-tensioned flat plates designed using the equivalent frame method of analysis because the negative moments calculated by this method are usually unconservative approximations of the actual maximum negative moments in the immediate vicinity of the columns.

2. The requirements for nonprestressed reinforcement in positive moment areas as related to tensile stresses are less conservative than ACI 318-71. The requirements have been modified on the basis of research in which complete positive and negative moment yield lines developed in specimens without any nonprestressed positive moment reinforcement.[3†] These yield lines developed at moments significantly above the design (ultimate) moments. The use of Eq. (18-6) in ACI 318-71 to determine the amount of reinforcement in positive moment areas is not specified in these tentative recommendations because this amount of reinforcement has been found to be unnecessary in the research referenced above.

3. The amount of nonprestressed reinforcement required in Section 2.10.1 in negative moment areas for various tensile stress levels represents a complete revision of the provisions of Section 18.9 of ACI 318-71 as related to prestressed flat plates. The amount and placement of nonprestressed reinforcement for negative moment areas in these requirements are based on published research data[3,4,10,11] and on the results of physical research completed but not yet published.*†

Committee 423 recommends revision of the provisions for tensile stress limitations and nonprestressed reinforcement requirements of ACI 318-71 for prestressed flat plates in accordance with these "Tentative Recommendations."

*See Section 2.11 for definition and discussion of average prestress.

†"Structural Model Tests of Post-Tensioned Flat Plates" conducted by the University of Texas at Austin. First test completed July 17, 1973.

2.4.2.3 *Bearing stresses under anchorage plates.* The average bearing stresses on the concrete created by the anchorage plates shall not exceed the values allowed by the following equations:[12]

At service load:

$$f_{cp} = 0.6\, f_c' \sqrt{A_b'/A_b}$$

but not greater than f_c'

At maximum jacking load:

$$f_{cp} = 0.8\, f'_{ci} \sqrt{(A_b'/A_b) - 0.2}$$

but not greater than $1.25\, f'_{ci}$

where

f_{cp} = permissible compressive concrete stress

f_c' = specified compressive strength of concrete

f'_{ci} = specified compressive strength of concrete at time of initial prestress

A_b' = maximum area of the portion of the concrete anchorage surface that is geometrically similar to and concentric with the area of the anchorage

A_b = bearing area of the anchorage

As used in the preceding equations f_{cp} is the average bearing stress, P/A_b in the concrete computed by dividing the force P of the prestressing steel by the net projected area, A_b, between the concrete and the bearing plate or other structural element of the anchorage which has the function of transferring the force to the concrete.

Post-tensioning anchorages which do not utilize bearing plates may be used and need not comply with the above provisions for bearing stresses if the anchorage is one of recognized proven performance, and test data satisfactory to the engineer is provided.

2.4.2.4 *Effective prestress in steel.* The loss of prestress in post-tensioned flat plates is a function of the average prestress. Since application and design usually result in lower average prestress in slabs than beams, the loss of prestress is lower. The actual losses due to concrete elastic shortening, creep and shrinkage vary significantly with the maturity of the concrete at the time of stressing. Therefore, the strength of the concrete at the time of stressing must be specified by the engineer. In general, it is advisable to calculate prestress losses based on the characteristics of the materials involved and the specific construction procedures contemplated. Such calculations should be in accordance with Section 18.6 of ACI 318-71.

2.5—Other methods of analysis

2.5.1 *Methods*—More rigorous analysis methods of determination of moments in post-tensioned flat plates may include the following:

(a) Elastic theory of the bending of thin plates[13]
(b) Finite element analysis[14]
(c) Finite difference analysis[13,15]

2.5.2 *Allowable stresses*—When more rigorous methods of analysis are used to determine maximum moments in post-tensioned flat plates, and where the reinforcement specified in Section 2.10 is provided, higher allowable stresses than those specified in Sections 2.4.2.1 and 2.4.2.2 are suggested for peak service load moments at columns.[3]

2.6—Tendon distribution and spacing

2.6.1 *Definition of column and middle strips*—A column strip is a design strip with a width of $0.25\, L_2$, but not greater than $0.25\, L_1$, on each side of the column center line where:

L_1 = length of span in the direction moments are being determined, measured center-to-center of supports

L_2 = length of span transverse to L_1, measured center-to-center of supports

A middle strip is a design strip bounded by two column strips. See Fig. 13-2 in the Commentary to ACI 318-71 for illustration of column strip and middle strip.

2.6.2 *Tendon distribution*—The ultimate strength of a flat plate is controlled primarily by the total amount of tendons in each direction. However, tests,[4,16] indicate that tendons passing through columns or directly around column edges contribute more to load carrying capacity than tendons remote from the columns. For this reason, it is recommended that some tendons should be placed through the columns or at least around their edges. In lift slab construction, some tendons should be placed over the lifting collars.[4,16]

For panels with length-width ratios not exceeding 1.33, the following approximate distribution may be used:

Simple spans; 55 to 60 percent of the tendons in the column strip with the remainder in the middle strip.

Continuous spans; 65 to 75 percent of the tendons in the column strip with the remainder in the middle strip.[3]

2.6.3 *Tendon spacing*—The tendon spacing recommendations in this section have been arbitrarily determined based on satisfactory field performance.

As a general guide, the recommended maximum spacing of tendons in the column strips is about four times the slab thickness, and the recommended maximum spacing in the middle strips is about six times the slab thickness.[2] However, for very short spans, tendon spacings up to eight times the slab thickness may be appropriate.

When bundled individual tendons are used, the above spacings apply to the bundle.

TABLE 1—COMPARISON OF FLAT PLATE SHEAR TEST DATA VERSUS RECOMMENDED PROCEDURE FOR CALCULATING PERMISSIBLE SHEAR STRESS

Test*	f_c', psi	f_{pc}, psi	v_u (test)		$v_{cw} = 3.5\sqrt{f_c'} + 0.3\ f_{pc}$	
			psi	$Nx\sqrt{f_c'}$	psi	$Nx\sqrt{f_c'}$
S4	3956	450	620	9.84	355	5.64
S5	2935	250	387	7.15	264	4.88
S6	4700	250	388	5.66	315	4.80
S7	2890	500	463	8.63	388	6.30
S8	4348	500	492	7.48	381	5.78
S9	4376	250	482	7.29	327	4.95
S10	4668	500	542	7.94	389	5.69
S11	5125	300	392	5.48	341	4.77
S12	4919	300	460	6.56	336	4.80
S13	5228	300	539	7.44	344	4.76
S14	4800	375	428	6.19	364	5.26
S15	5121	375	593	8.28	362	5.06
C1	5300	250	483	6.68	329	4.52
C2	4600	250	524	7.79	312	4.60
C3	4915	250	552	7.96	320	4.57
C4	4930	250	497	7.13	322	4.58
L1	5300	250	451	6.20	329	4.52
L2	4500	250	459	6.85	309	4.61
L3	4610	250	482	7.10	313	4.61
L4	4820	250	441	6.35	317	4.57
G0	4000	006	364	5.76	223	3.53
G2	4260	115	417	6.39	263	4.03
G3	4140	168	400	6.22	275	4.28
G4	3610	276	417	6.94	293	4.87
G5	4580	280	440	6.61	320	4.73
G6	4250	335	405	6.21	329	5.05
G7	4430	390	440	6.61	350	5.26
G8	4630	444	498	7.30	373	5.48
G10	4400	536	587	8.85	393	5.92
G12	4096	656	543	8.50	421	6.59

*Tests S4 through S15 from Reference 16; Tests C1 through C4 and L1 through L4 from Reference 4; and Tests G0 through G12 from Reference 17.

2.7—Ultimate strength

2.7.1 *Required strength*—Post-tensioned flat plates must meet the ultimate strength requirements of ACI 318-71. The required strength should be in accordance with the load factors specified in ACI 318-71 Section 9.3. The strength of a member should be calculated in accordance with the requirements and assumptions of ACI 318-71, including the capacity reduction factors presented in Section 9.2.

2.7.2 *Moment redistribution*—The moment redistribution provisions of Section 18.12 of ACI 318-71 are applicable to design of prestressed flat plates.

2.7.3 *Calculation of f_{ps}*—The stress in the post-tensioning tendons at design load, f_{ps}, should be calculated in accordance with Section 18.7.1 of ACI 318-71.

2.7.4 *Nonprestressed reinforcement*—Nonprestressed conventional reinforcement conforming to ASTM Specifications A 615, A 616, A 617, A 185 or A 497 may be considered to contribute to the tension force in a member at design load an amount equal to its area times its yield strength, except that f_y should not be assumed to exceed 60,000 psi. For other types of nonprestressed reinforcement, a strain compatibility analysis should be made to determine its contribution to the tension force.

2.8—Deflection and camber

Calculations should verify that deflections comply with Section 9.5.4 of ACI 318-71.

For prestressed slabs continuous over two or more spans in each direction, a span-depth ratio (for light live loads, say about 50 psf) of 40-45 may be used for floors, and a ratio of 45-48 for roofs. These limits may be increased to 48 and 52, respectively, if calculations verify that deflection, camber and vibration frequency and amplitude are not objectionable.

2.9—Shear stress

2.9.1 *Design sections*—The shear strength of post-tensioned flat plates is governed by the more severe of two conditions:

1. The plate acting as a wide beam with a potential diagonal crack extending a plane across the entire width. Design for this case should be in accordance with Section 2.9.2 below.

2. Two-way action with potential diagonal cracking along the surface of a truncated cone or pyramid around the concentrated load or reaction. Design for this case should be in accordance with Section 2.9.3 below.

2.9.2 *Shear design as a wide beam*—When the plate is considered as a wide beam, the shear stress should conform to the provisions of ACI 318-71, Sections 11.1 through 11.6 (primarily Sections 11.2.2 and 11.5).

2.9.3 *Shear design for two-way action*—The critical section for two-way action should be as defined in ACI 318-71, Section 11.10.2 (A periphery outside the column at a distance $d/2$). This critical section usually controls over shear design considering the plate as a wide beam.

ACI 318-71 currently recommends a limit of $4\sqrt{f_c'}$ to the allowable shear stress for both conventionally reinforced and prestressed flat plates. Committee 423 feels that this is too restrictive for prestressed concrete flat plates and recommends that the ACI 318-71 limit be liberalized. Specifically, Committee 423 recommends consideration of the following design method for shear for two-way action for prestressed flat plates, which may result in higher allowable shear stresses than the current maximum of $4\sqrt{f_c'}$ in ACI 318-71.

An approximate method of shear design for two-way action is to calculate the nominal ultimate shear stress in accordance with Eq. (11-25) of ACI 318-71. The shear stress thus calculated should be limited to that given by Eq. (11-12) in ACI 318-71, but the value of f_{pc} used in the equation should not exceed 500 psi since only limited test data are available for higher values. In addition, the value of f_c' used in Eq. (11-12) should not exceed 5000 psi until the applicability of the equation for higher strength concrete has been demonstrated by research. The term $V_p/b_w d$ in Eq. (11-12) usually contributes only a small amount to the permissible shear stress in a post-tensioned flat plate, and for convenience this term may be conservatively neglected in design. If the term $V_p/b_w d$ is included, it is necessary to use the actual reverse curvature tendon geometry in calculations to assess the shear carried by the tendons inside the critical section.

When situations exist in which moment must be transferred from a post-tensioned flat plate to a column, the requirements of Section 11.13 of ACI 318-71 should be satisfied. However, in application of Section 11.13 to post-tensioned flat plates, Committee 423 recommends that the limiting shear stress be that obtained from Eq. (11-12) as discussed above, and recommends revision of ACI 318-71 accordingly. The use of Eq. (11-12) for exterior columns presumes tendon placement which will provide a compression level at the critical section comparable to that obtained for interior columns. To achieve this, tendons usually have to be placed through the column.

The recommendation for determining permissible shear stress is compared with test data, v_u, from References 4, 16, and 17 in Table 1 and in Fig. 2. The data has been recalculated to an effective depth d at the centroid of the tendons from the total thickness h used in the references, v_u (test) $= P/b_o d$, where $P =$ the load at failure, and $b_o =$ the shear perimeter at a periphery $d/2$. In the case of lift slabs, the effective depth was reduced by the distance from the extreme compression fiber to the inside of the angle iron on the lifting collar. Recommended values to test values, v_{cw}/v_u, vary from 0.57 to 0.85.

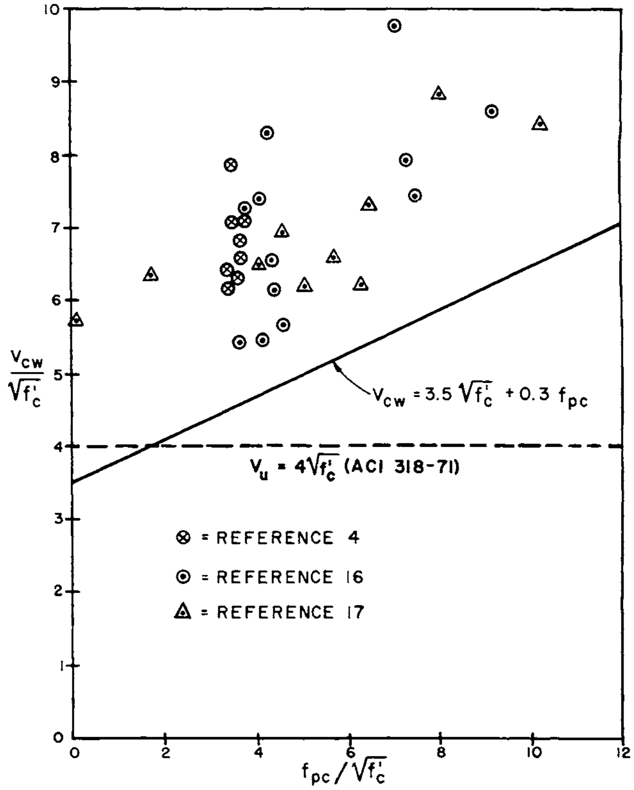

Fig. 2—Shear test data versus recommended design equation

Additional empirical equations and test results for shear design of post-tensioned flat plates for two-way action are presented in References 4, 16, and 17.

2.10—Nonprestressed reinforcement

2.10.1 *Nonprestressed reinforcement in column areas*—The minimum amount of nonprestressed bonded reinforcement placed in the top of post-tensioned flat plates in each direction in column areas should be 0.15 percent of the cross-sectional area of the column strip.[4,10,11]* Within the limitations of details and minimum bar spacing specifications, this reinforcement should be concentrated directly over and immediately adjacent to the column. The spacing of the bars should not exceed 12 in. on centers, and not less than four bars should be used in each direction. For normal span ratios, these bars should have a total length equal to ⅓ of the span (use average span length when adjacent spans are unequal in length).

As discussed in Section 2.4.2.2, the above nonprestressed reinforcement requirement represents a revision and liberalization of Section 18.9 of ACI 318-71 as related to prestressed flat plates. Committee 423 recommends revision of the non-

*This percentage of reinforcement has also been verified as adequate through testing of a nine-panel one-third scale model of a post-tensioned flat plate at the University of Texas at Austin in July 1973. The results of this test have not yet been published.

prestressed reinforcement requirements of Section 18.9 of ACI 318-71 for prestressed flat plates in accordance with these "Tentative Recommendations."

Where unbalanced gravity load, wind, earthquake or other lateral forces cause transfer of bending between slab and column, the fraction of the moment as defined by Section 11.13.2 of ACI 317-71 should be considered transferred by eccentricity of the shear about the centroid of the critical section defined above. Shear stresses should be taken as varying linearly about the centroid of the critical section, and the shear stress v_u limited to that of Eq. (11-12) of ACI 318-71.

Capacity to transfer that portion of the moment not transferred by shear should be provided within the slab between lines that are one and one-half slab thicknesses, 1.5*h* on each side of the column. Connections at exterior columns should also satisfy requirements of Section 13.3.4.10 of ACI 318-71. Nonprestressed reinforcement should be adequately anchored at the discontinuous edge in accordance with Section 13.5.2 of ACI 318-71. For connections to interior columns, bonded reinforcement should be placed in the bottom of the slab over the column when the difference between the maximum and minimum shear stresses on the critical section on either side of the column exceeds $3\sqrt{f_c'}$.[18,19] The amount of reinforcement should be not less than

$$\rho f y = 2.5\sqrt{f_c'}$$

and that reinforcement may be distributed uniformly over a slab width between the lines that are three-slab thickness, 3*h*, on each side of the column.[18,19] The amount of reinforcement within lines that are one and one-half slab thickness on each side of the column or collar can be considered effective in conjunction with top reinforcement for transfer of that portion of the moment not transferred by shear.

2.10.2 *Nonprestressed reinforcement in positive moment areas*—Where tensile stresses in positive moment areas exceed $2\sqrt{f_c'}$, nonprestressed reinforcement should be provided in accordance with Eq. (18-5) of ACI 318-71. As discussed in Section 2.4.2.2, this recommendation represents a deviation from the requirements of Section 18.9 of ACI 318-71, and Committee 423 recommends a revision of Section 18.9 of ACI 318-71 with respect to prestressed flat plates in accordance with these "Tentative Recommendations."

2.10.3 *Reinforcement around openings*—In principle, ACI 318-71, Section 13.6 applies to openings in post-tensioned flat plates. Tendons should be continuous and splayed horizontally to get around smaller openings. If tendons are terminated at edges of larger openings, such as at stairwells, an analysis should be made to insure sufficient strength and proper behavior. Edges around openings may be reinforced in a manner similar to conventionally reinforced slabs, or, in the case of larger openings, supplementary post-tensioning tendons may be used to strengthen the edges.

2.10.4 *Nonprestressed reinforcement at end anchorages* — Some nonprestressed reinforcement should be added at end anchorages to avoid possible splitting of the concrete. Two #4 bars are commonly used continuously around the perimeter of the slab behind the anchorages for this purpose. For highly prestressed slabs, or where anchorages are concentrated in a narrow slab width, the need for reinforcement to resist horizontal splitting of the slab should be investigated.[20] Additional special reinforcement, required for the performance of the anchorage, should be indicated by the tendon supplier.

2.11—Average prestress

2.11.1 *Minimum average prestress*—The average prestress is defined as the total prestress force (after losses) divided by the total area of concrete. To minimize cracks in post-tensioned flat plates, it is desirable to maintain a minimum average prestress of 200 to 250 psi. Lower values of average prestress have been used successfully for short spans or for slabs with rectangular panels for prestress parallel to the short slab dimension. However, only limited test data have been obtained for slabs with average prestress below 150 psi, hence, these "Tentative Recommendations" should be applied to such slabs with caution. Values of average prestress less than 125 psi are not recommended.

2.11.2 *Maximum average prestress*—A high value of average prestress may induce excessive elastic shortening and creep. An arbitrary value of 500 psi is generally considered the maximum for solid slabs. When axial shortening is not a problem this may be raised.

2.12—Supporting walls and columns

When walls and columns are rigidly connected to the slabs, their stiffness should be taken into consideration in the equivalent frame (or other) analysis.

When columns and walls have significant stiffness in the direction of prestress, the effects of possible diversion of prestress into bending and/or translating substructure elements on the performance of the flat plate should be considered. The columns or walls may also restrain the slab from shortening, thus resulting in cracks. This effect can be quite serious for long slabs with high shrinkage and creep. Likewise, the effects of the prestressing forces on stiff supporting elements may require investigation. However, design

TABLE 2—FIRE RESISTANCE REQUIREMENTS FOR PRESTRESSED CONCRETE SLABS COLD-DRAWN PRESTRESSING STEEL[27]

	Dimension	Aggregate type	Minimum dimension, in., for fire resistance ratings indicated				
			1 hr	1½ hr	2 hr	3 hr	4 hr
Slabs, continuous or restrained†	Cover*	Carbonate	¾	¾	¾	1	1¼
		Siliceous	¾	¾	¾	1	1¼
		Lightweight	¾	¾	¾	¾	1
Slabs, simply supported and unrestrained†	Cover*	Carbonate	¾	1-1/16	1⅜	1⅞	—
		Siliceous	¾	1¼	1½	2⅛	—
		Lightweight	¾	1	1¼	1⅝	—
Slabs, all	Thickness	Carbonate	3¼	4⅛	4⅝	5¾	6⅝
		Siliceous	3½	4¼	5	6¼	7
		Lightweight	2⅝	3¼	3¾	4⅝	5¼

*Minimum thickness of concrete between the surface of the tendon and the nearest fire exposed surface of the concrete.

†For definition of restrained and unrestrained construction, see "Standard Methods of Fire Tests of Building Construction and Materials" Designation E 119-71, American Society for Testing and Materials, Philadelphia.

and construction options are available to reduce the effects of shortening on both the plate and the supporting elements,[21] and the moments of stresses which occur over a period of time due to creep and shrinkage shortening are themselves reduced approximately 50 percent by creep.[22]

2.13—Closure strips

Open strips may temporarily separate adjacent slabs during construction. 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 amounts due to continuity taking into consideration the amount of deflection and camber which occur prior to casting the closure strip. Propping may be used to assure full continuity for both dead and live load.

2.14—Seismic design

2.14.1 *Diaphragms*—Concrete floor slabs may be assumed to serve as horizontal diaphragms to distribute lateral forces throughout a story.

2.14.2 *Resisting frames*—The resistance to lateral forces is usually provided by vertical resisting elements such as walls or columns with the floor slabs serving primarily as horizontal diaphragms. When the slabs form an integral part of the resisting rigid frames, the slab-column joints should be carefully analyzed and detailed to provide both the strength and the ductility required.

2.15—Fire resistance ratings for flat plate construction

Fire tests[23,24] and actual fires in structures[25] have shown post-tensioned flat plate floors with covers and slab thicknesses in accordance with Table 2 to provide excellent fire resistive properties.

Cover dimensions and slab thicknesses required for various fire ratings are provided in Table 2. These ratings may be improved by addition of a protective coating to the slab.[26]

Fire ratings for post-tensioned flat plates with various cover and thickness dimensions may be determined in accordance with Table 2.[27,28] Alternatively, fire ratings may be determined by fire tests or by detailed computations.[27]

CHAPTER 3—CONSTRUCTION

3.1—Construction joints

The maximum length of a slab between construction joints should be limited to about 150 ft to minimize the effect of slab shortening, and to avoid excessive loss of prestress due to friction. When special consideration is given to reducing the effects of axial shortening on both the plate and the substructure elements, and where tendons with low friction losses are utilized, somewhat longer distances between construction joints may be used. Tendons should be stressed from both ends when their length exceeds 100 ft.

3.2—Placement of tendons

3.2.1 *Tendon profile*—The placement of tendons should closely follow the specified profiles. Theoretical sharp breaks over supports should be replaced with smooth reverse curves approximating the theoretical profile within the practical limits of tendon curvature.

3.2.2 *Tolerances*—Tendon placement for slabs 8 in. or less in thickness may have a vertical tolerance of ± 1/8 in. at critical points, such as at midspan and at supports. For slabs thicker than 8 in., the vertical tolerance may be ± 1/4 in. Horizontal location for a tendon may be varied a few inches without affecting the behavior of the slabs.

METRIC CONVERSION FACTORS

Multiply	by	to obtain
in.	2.54	cm
ft	0.3048	m
psi	0.0703	kgf/cm^2
psf	4.882	kgf/m^2

REFERENCES

Cited references

1. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-71)," American Concrete Institute, Detroit, 1971, 78 pp.

2. ACI-ASCE Committee 423, "Tentative Recommendations for Concrete Members Prestressed with Unbonded Tendons," ACI Journal, *Proceedings* V. 66, No. 2, Feb. 1969, pp. 81-86.

3. Lin, T. Y.; Scordelis, A. C.; and Itaya, R., "Behavior of a Continuous Slab Prestressed in Two Directions," ACI Journal, *Proceedings* V. 56, No. 6, Dec. 1959, pp. 441-460.

4. Gerber, Louis L., and Burns, Ned. H., "Ultimate Strength Tests of Post-Tensioned Flat Plates," *Journal*, Prestressed Concrete Institute, V. 16, No. 6, Nov.-Dec. 1971, pp. 40-58.

5. Parme, Alfred L., and Paris, George H., "Designing for Continuity in Prestressed Concrete Structures," ACI Journal, *Proceedings* V. 48, No. 1, Sept. 1951, pp. 45-64.

6. Lin, T. Y., "Load-Balancing Method for Design and Analysis of Prestressed Concrete Structures," ACI Journal, *Proceedings* V. 60, No. 6, June 1963, pp. 719-742.

7. Bailey, Duryl M., and Ferguson, Phil M., "Fixed-End Moment Equations for Continuous Prestressed Concrete Beams," *Journal*, Prestressed Concrete Institute, V. 11, No. 1, Feb. 1966, pp. 76-94.

8. Turula, Peter, and Freyermuth, Clifford L., "Moment Influence Coefficients for Continuous Post-Tensioned Structures," *Journal*, Prestressed Concrete Institute, V. 17, No. 1, Jan.-Feb. 1972, pp. 35-57.

9. Parme, A. L., "Prestressing Flat Plates," *Journal*, Prestressed Concrete Institute, V. 13, No. 6, Dec. 1968, pp. 14-32.

10. Smith, Stephen Whitfield, "Behavior of Post-Tensioned Flat Plate Column Connections," MSc Thesis, University of Texas at Austin, Dec. 1972, 152 pp.

11. Changwatchai, Chaiwat, "Behavior of Unbonded Prestress Concrete Flat Plates With and Without Bonded Reinforcement," MSc Thesis, University of Texas at Austin, Aug. 1973, 96 pp.

12. PCI Committee on Post-Tensioning, "Tentative Specifications for Post-Tensioning Materials," *Journal*, Prestressed Concrete Institute, V. 16, No. 1, Jan.-Feb. 1971, pp. 14-20.

13. Timoshenko, S., and Woinowsky-Krieger, S., *Theory of Plates and Shells*, McGraw-Hill Book Co., New York, 2nd Edition, 1959, 580 pp.

14. Zienkiewicz, O. C., and Cheung, Y. K., *The Finite Element Method in Structural and Continuum Mechanics*, McGraw-Hill Book Co., New York, 1967, 274 pp.

15. Salvadori, Mario G., and Baron, M. L., *Numerical Methods in Engineering*, Prentice-Hall, Inc., Englewood Cliffs, N. J., 1952, 320 pp.

16. Lin, T. Y.; Scordelis, A. C.; and May, H. R., "Shearing Strength of Prestressed Concrete Lift Slabs," Document Section, State of California, Sacramento, Oct. 1957. Also, summarized in "Shearing Strength of Prestressed Lift Slabs," ACI Journal, *Proceedings* V. 55, No. 4, Oct. 1958, pp. 485-506.

17. Grow, J. B., and Vanderbilt, M. D., "Shear Strength of Prestressed Lightweight Aggregate Concrete Flat Plates," *Journal*, Prestressed Concrete Institute, V. 12, No. 4, Aug. 1967, pp. 18-28.

18. Moe, Johannes, "Shearing Strength of Reinforced Concrete Slabs and Footings Under Concentrated Loads," *Development Department Bulletin* D47, Portland Cement Association, Skokie, Apr. 1961, 130 pp.

19. Hawkins, N. M., "Shear and Moment Transfer Between Concrete Flat Plates and Columns," *Progress Report*, NSF Grant GK-16375, Department of Civil Engineering, University of Washington, Dec. 1971.

20. Leonhardt, Fritz, *Prestressed Concrete; Design and Construction*, Translated by C. van Amerongen, Wilhelm Ernst and Sohn, Berlin, 2nd Edition, 1964, 677 pp.

21. Freyermuth, Clifford L., and Shoolbred, Robert A., "Post-Tensioned Prestressed Concrete," *Publication* No. EB013D, Portland Cement Association, Skokie, 1967, 24 pp.

22. Ghali, Amin; Dilger, Walter; and Neville, Adam M., "Time-Dependent Forces Induced by Settlement of Supports in Continuous Reinforced Concrete Beams," ACI Journal, *Proceedings* V. 66, No. 11, Nov. 1969, pp. 907-915.

23. Troxell, George E., "Fire Tests of Six Inch Deep Prestressed Concrete Flat Slab," *Publication* SFT-2, Fire Prevention Research Institute, Feb. 1960.

24. "Report on Unbonded Post-Tensioned Prestressed, Reinforced Concrete Flat Plate Floor with Expanded Shale Aggregate," *File* R 5084-3, Underwriters' Laboratories, Inc., *Journal*, Prestressed Concrete Institute, V. 13, No. 2, Apr. 1968, pp. 45-56.

25. Troxell, G. E., "Prestressed Lift Slabs Withstand Fire," *Civil Engineering*—ASCE, V. 35, No. 9, Sept. 1965, pp. 64-66.

26. Abrams, M. S., and Gustaferro, A. H., "Fire Endurance of Prestressed Concrete Units Coated with Spray-Applied Insulation," *Journal*, Prestressed Concrete Institute, V. 17, No. 1, Jan.-Feb. 1972, pp. 82-103.

27. Gustaferro, Armand H., "Fire Resistance of Post-Tensioned Structures," *PCI Post-Tensioning Manual*, Prestressed Concrete Institute, Chicago, 1972, Chapter 6.

28. FIP Commission on the Fire Resistance of Prestressed Concrete Structures, "Draft of Code Provisions for the Fire Resistance of Prestressed Concrete Structures," FIP Administrative Office, Terminal House, Grosvenor Gardens, London, Revised Mar. 1969.

Additional references

29. Scordelis, A. C.; Pister, K. S.; and Lin, T. Y., "Strength of a Concrete Slab Prestressed in Two Directions," ACI Journal, *Proceedings* V. 53, No. 3, Sept. 1956, pp. 241-256.

30. Peterson, Charles, and Brownfield, A. H., "Our Experience with Prestressed Lift Slabs," *Proceedings*, World Conference on Prestressed Concrete (San Francisco, 1957), World Conference on Prestressed Concrete, Inc., San Francisco, 1957, pp. A18-1 through A18-4.

31. Brownfield, A. H., "Growing Pains in Prestressed Concrete Buildings," *Civil Engineering—ASCE*, V. 28, No. 2, Feb. 1958, p. 96.

32. Riley, W. E., "Lift-Slab Construction for College Dorm Features Precast Concrete Columns," *Civil Engineering—ASCE*, V. 28, No. 3, Mar. 1958, p. 193.

33. Rice, Edward K., "Economic Factors in Prestressed Lift-Slab Construction," ACI Journal, *Proceedings* V. 55, No. 3, Sept. 1958, pp. 347-358.

34. Lin, T. Y., "Prestressed Concrete Slabs and Shells, Design and Research in the United States," *Civil Engineering—ASCE*, V. 28, No. 10, Oct. 1958, p. 757.

35. Rice, Edward K., and Kulka, Felix, "Design of Prestressed Lift-Slabs for Deflection Control," ACI Journal, *Proceedings* V. 56, No. 8, Feb. 1960, pp. 681-694.

36. Saether, Kolbjorn, "The Structural Membrane Theory Applied to the Design of Prestressed Flat Slabs," *Journal*, Prestressed Concrete Institute, V. 8, No. 5, Oct. 1963, pp. 68-79.

37. Lu, Frank, "Strength and Behavior of a Nine-Bay Continuous Concrete Slab Prestressed in Two Directions," *Interim Report*, University of Canterbury, Christchurch, New Zealand, Mar. 1966.

38. Gamble, W. L., "An Experimental Investigation of the Strength and Behavior of a Prestressed Concrete Flat Plate," *Report* No. T8.0-9, Division of Building Research, Commonwealth Scientific and Industrial Research Organization, Melbourne, 1964, 53 pp.

39. Odello, R. J., and Mehta, B. M., "Behavior of a Continuous Prestressed Concrete Slab with Drop Panels," *Report*, Division of Structural Engineering and Structural Mechanics, University of California, Berkeley, 1967.

40. Thornton, Keith, "Post-Tensioned Slabs with Dropped Panels," *Civil Engineering—ASCE*, V. 38, No. 4, Apr. 1968, p. 75.

41. Nasser, George D., "A Look at Prestressed Flat Plate Design and Construction," *Journal*, Prestressed Concrete Institute, V. 14, No. 6, Dec. 1969, pp. 62-77.

42. Omsted, Harold, "Notes on Post-Tensioned Lift Slabs," *Journal*, Prestressed Concrete Institute, V. 10, No. 6, Dec. 1965, pp. 84-90.

43. Ghali, Amin, "Bending Moments in Prestressed Concrete Structures by Prestressing Moment Influence Coefficients," ACI Journal, *Proceedings* V. 66, No. 6, June 1969, pp. 494-496.

This report was submitted to letter ballot of the committee which consists of 15 members. Thirteen members returned ballots of whom 12 voted affirmatively and 1 voted negatively.

TECHNICAL FEATURES WANTED — The Technical Activities Committee is always on the lookout for short items that tell how much some practical problem was solved in everyday design and construction of concrete structures. If you recently solved one of those knotty problems that regularly occur, share it with us; the item need not be more than 500 or 600 words long. TAC welcomes longer papers on practical subjects too, however!