PDHonline Course S225 (3 PDH)

The Design of Unbonded, Mono-Strand, Post-Tensioned Concrete Members

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The Design of Unbonded, Mono-Strand, Post-Tensioned Concrete Members

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COURSE CONTENT

Post-Tensioned Concrete Design

The basic steps involved in the design of an unbonded, post-tensioned, mono-strand system include:

1. Establish the preliminary member size using the span-to-depth ratios provided by the PTI.

2. Establish the preliminary strand drape by locating the strand as close to the extreme fiber in tension (i.e. at the bottom of the member at midspan and at the top of the member at the supports) while satisfying all cover requirements for durability, the environmental exposure or the fire rating.

3. Complete an analysis of the short-term (friction loss, wedge set loss, elastic shortening) and long-term losses (creep, shrinkage, strand relaxation) based on the preliminary strand drape to determine the effective post-tensioning force, balancing load and strand location. Methods for determining short and long-tem losses include:

    a. Short Term Losses:

        - Friction and Wedge: Assume 0.25 to .375 inch live end wedge set loss in conjunction with frictional losses based on either the PTL software or hand calculation methods provided in Appendix A of the PTI Manual (see page 19 and 20 of Lecture 9).

        - Elastic Shortening (ES): Assume 3 to 4% of the initial post-tensioning force is loss due to elastic shortening or calculate the ES value per the equation provided on page 17 of Lecture 9.

    b. Long-term Losses: Assume lump sum long-term losses based on Table 6 (Lecture 9) or calculate the long-term losses using the equations provided on pages 18 and 19 of Lecture 9.

4. Establish separate service shear and moment diagrams for the member based on the initial and final dead loads, live loads and balancing loads.

5. Perform an initial and final service stress analysis (i.e. $f_b = (P/A \pm M_{Net}/S) \leq f_{Allowable}$; where $M_{Net} = ((M_{DL} + M_{LL}) - M_{Balancing})$ in order to establish the number of strands required to satisfy the particular Class (U, T or C) of construction. Confirm that the minimum P/A stress is satisfied. Revise the strand drape or jacking sequence as required to minimize the number of strands required.

6. Perform an ultimate flexural analysis based on the number of strands established by Step #5. Include all secondary moment effects in the analysis. Provide all minimum bonded reinforcement required by the Code and or provide additional conventional reinforcing bars to satisfy the ultimate flexural strength requirements of the member.

7. Perform an ultimate shear analysis. Do not include the balancing loads in the design shear values. Provide all minimum shear/stirrup reinforcement required by the Code and or provide additional conventional stirrups to satisfy the ultimate shear strength requirements of the member.

8. Complete an analysis of the deflection of the member based on service loads including the balancing loads. As long as the net, long-term tensile stress, $f_t \leq 7.5(f'_c)^{1/2}$, use I_g and ignore long-term growth of the immediate dead load deflection.

9. Develop end anchorage reinforcing details (see Sections 18.13 and 18.14) and assure support details do not restrain the member during pre-compression of the concrete as a result of the tensioning operation.

10. Document the final design via plans, beam schedules and elevations.

It should be noted that the provisions of Appendix B and C of ACI 318-05 referenced in Lecture 6 relative to the use of the flexural design criteria and load and strength reduction factors of the ACI Code prior to the changes that were reflected in the 2002 Code are applicable to post-tensioned design. Also, this approach assumes that there is no redistribution of negative moments to the positive moment region

General Considerations:

In post-tensioned members, compressive stresses are introduced into the concrete to reduce tensile stresses resulting from applied loads including the self weight of the member. Post-tensioning strand is used to impart compressive stresses to the concrete. Post-tensioning is a method of prestressing in which the tendons are tensioned after the concrete has hardened and the prestressing force is transferred to the concrete through the end anchorages.

The act of post-tensioning a member introduces “pretensioning loads” to the member. The induced pretensioning loads, acting in conjunction with externally applied loads, must satisfy serviceability and strength requirements of the member beginning immediately after the post-tensioning force is transferred to the concrete and continuing throughout the life of the member. Post-tensioned structures must be analyzed taking into account pretensioning loads, service loads, temperature, creep, shrinkage and the structural properties of all of the materials involved.

Post-Tensioning Materials:

The most commonly used post-tensioning tendons in the United Sates in Grade 270 ksi low-relaxation, seven-wire strand, defined by ASTM A 416. The most common size is ½ in., although there is increasing use of 0.6-in strand.. The properties of these strands are as follows:

Nominal Diameter, inches	1/2	0.6
Area, in^2	0.153	0.217
Breaking stress, f_{pu}, ksi	270	270
Breaking strength, kips	41.3	58.6
Jacking Stress, 0.75 f_{pu},	202.5	202.5

Flexural Strength:

Post-Tensioning Steel Stress at Nominal Flexural Strength

The nominal strength of unbonded strands is determined using Equations 18-4 (span-to-depth ratio < 35) and 18-5 (span-to-depth > 35):

$$f_{ps} = f_{se} + 10{,}000 + (f'_c/100\rho_p) \qquad [18\text{-}4]$$

$$f_{ps} < \text{the lesser of } f_{py} \text{ or } (f_{se} + 60{,}000)$$

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

$$f_{ps} < \text{the lesser of } f_{py} \text{ or } (f_{se} + 30{,}000)$$

Where; f_{se} = Effective stress in post-tensioning strand after including all losses.

f_{py} = 0.90fpu (for low-relaxation strand); = 0.85fpu (stress relieved strand)

$\rho_p = A_{ps}/bd_p$

Nominal Flexural Moment Strength

With the value of f_{ps} known, the nominal moment strength of a rectangular section, or a flanged section (i.e. T-beam) where the stress block is within the compression flange, can be calculated as follows:

$$M_n = A_{ps}f_{ps}(d_p - (a/2))$$

Where; $a = (A_{ps}f_{ps})/(0.85bf'_c)$

If bonded conventional reinforcement is included into the nominal flexural strength:

$$M_n = A_{ps}f_{ps}(d_p - (a/2)) + A_sf_y(d - a/2)$$

Where; $a = ((A_{ps}f_{ps}) + (A_sf_y))/(0.85bf'_c)$

Reinforcing Upper Limit

ACI places an upper limit on the total amount of prestressed and non-prestressed reinforcement with the use of the following equation, which is applicable only to pretensioned members:

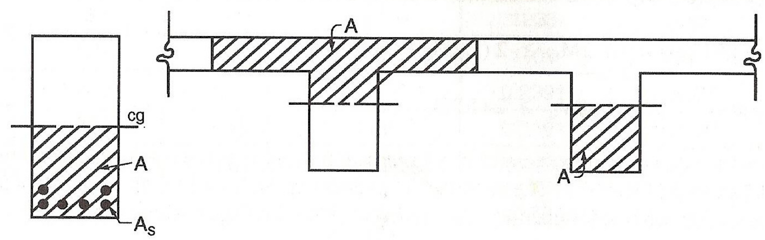
$$\omega = ((A_{ps}f_{ps})/(bd_pf'_c)) + ((A_sf_y)/(bdf'_c) < .36\beta_1$$

This requirement ensures ductility at nominal strength and guarantees that the stress values assumed for pretensioned and non-pretensioned reinforcement will be achieved prior to crushing of the concrete.

For tension-controlled flexural members ε_t (net tensile strain in extreme layer of tension steel at nominal strength excluding strains due to effective post-tensioning, creep, shrinkage and temperature effects) must be equal to or greater than 0.005. This is equivalent to a reinforcing index $\omega = 0.32\ \beta_1$ or about a 12% reduction in the maximum amount of permissible reinforcing. The minimum strain value of 0.005 is equivalent to $c/d_p = 0.375$ maximum. If $c/d_p > 0.375$ then compression reinforcement must be added until $c/d_p \leq 0.375$.

Minimum Reinforcing Requirements

The ACI Code (Section 18.9) requires a minimum amount of bonded reinforcement in all flexural members with unbonded post-tensioning tendons. These requirements ensure ductility and crack distribution equivalent to members with bonded reinforcement and prevent concentrated cracking and tied-arch behavior. Except two-way solid slabs (which will be discussed below), a minimum area of bonded reinforcement (A_s) computed by Equation 18-6 ($A_s = 0.004A_{ct}$) must be uniformly distributed over the precompressed tensile zone as close as practicable to the extreme tension fiber. Figure 1 illustrates the application of Equation 18-6; where A_{ct} = A in the diagram.

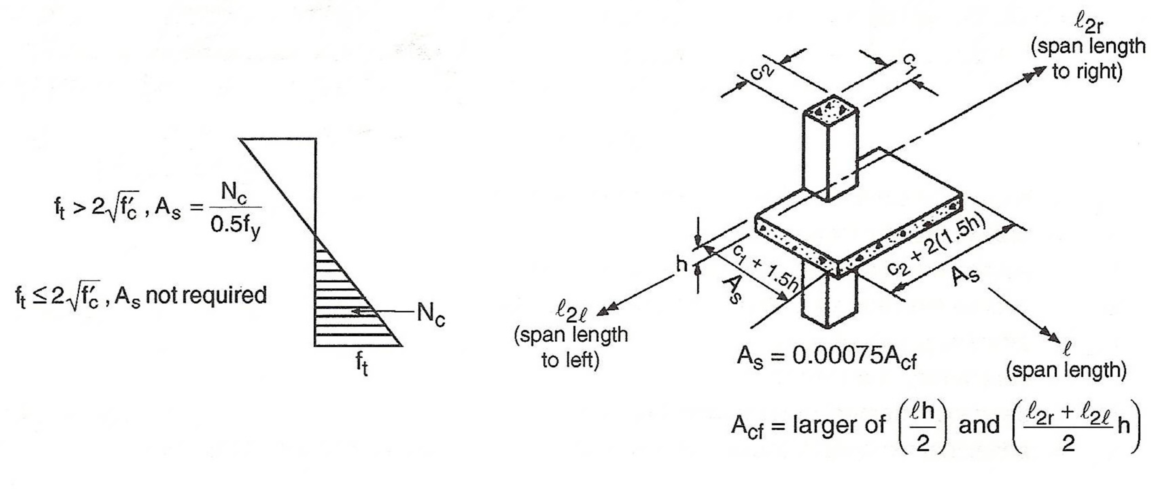


$A_s = 0.004A$

FIGURE 1

The provisions of Section 18.9.3 dictate the minimum reinforcing required for two-way flat slabs. Depending on the magnitude of the tensile stresses in the concrete at service loads, the requirements for positive moment areas of two-way flat slabs are illustrated in Figure 2a. Formerly Section 18.9.3 applied only to flat plates. Starting with ACI 318-02, this Section was also applied to two-way flat slab systems with drop panels. The provision (i.e. $f_t \leq 2\,(f'_c)^{1/2}$ of this section of ACI is the only condition in the Code that does not require minimum bonded reinforcement in a member with unbonded tendons.

The requirement for minimum area of bonded reinforcement in two-way flat plates at column supports was revised in the 1999 ACI Code to reflect the intent of the original research recommendations of ACI 423.3 R-96. This revision increases the minimum reinforcement requirement over interior columns for rectangular panels in one direction, and for square panels, doubles the minimum reinforcement requirement over exterior columns normal to the slab edge. Figure 2b illustrates the minimum bonded reinforcement requirements for the negative moment areas at column supports. The bonded reinforcement must be located within the width ($c_2 + 2(1.5h)$) as shown in Figure 2b, with a minimum of four bars spaced at not more than 12 inches on center. Similarly, minimum bonded reinforcement should be provided parallel to the slab edge.



(a) Positive Moment Areas (b) Negative Moment Areas

Figure 2a and 2b

The minimum lengths of all of the bars associated with the minimum steel requirements are one-third of the clear span, centered in positive moment areas, and one-sixth the clear span on each side of the column in negative moment areas. When this minimum bonded reinforcement is required for moment strength in either positive or negative moment regions, or to satisfy Equation 18-7 (i.e. Figure 2a) in positive moment regions, the bars must be fully developed in accordance with Chapter 12 of ACI 318-05. This means that they must extend a full development length beyond the point at which they are no longer required for flexural strength.

Cracking Moment

Section 18.8.2 requires the total amount of post-tensioned and conventional flexural reinforcement to be adequate enough to develop a design moment strength equal to at least 1.2 times the cracking moment strength ($\Phi M_n = 1.2M_{cr}$); where M_{cr} is computed by elastic theory using a modulus of rupture equal to $7.5(f'_c)^{1/2}$. This requirement is waived for two-way post-tensioned slabs with unbonded tendons and flexural members with shear and flexural strength at least twice that applied by the factored design loads. Section 18.8.2 is intended to prevent abrupt flexural failure developing immediately after cracking. This requirement is satisfied, when required, at each cross-section when:

$$\Phi M_n \geq 1.2((7.5(f'_c)^{1/2}S) + (S(P/A)) + M_{bal})$$

Where; S = the appropriate top of bottom section modulus

Shear Strength:

Unless a more detailed calculation is made per Section 11.4.3, for members with an effective post-tensioned force of not less than 40% of the tensile strength of the flexural reinforcement:

$$V_c = ((0.6(f'_c)^{1/2}) + (700((V_u d_p)/M_u)))b_w d\text{; but not less than } 2(f'_c)^{1/2}b_w d \text{ or greater than } 5(f'_c)^{1/2}b_w d$$

In addition $((V_u d_p)/M_u) \leq 1.0$

It should also be noted that d_p in the above equation must equal the actual d to the centroid of the post-tensioning strands. Whereas dp for all other shear equations need not be less than 0.80h.

In order to understand Equations 11-10 and 11-12 of Section 11.4.3, it is best to review the principles on which ACI shear design is based. These principles are empirical and are based on a large number of tests which illustrated that:

- The shear resisted by concrete and the shear resisted by stirrups are additive.
- The shear resisted by the concrete after shear cracks form is at least equal to the shear existing in the concrete at the location of the shear crack at the time the shear crack forms.

In order to understand how to calculate the shear resisted by the concrete at the time the shear crack forms it should be noted that one of the two following conditions may control:

1. Web shear. A diagonal shear crack originates in the web, near the neutral axis, and is caused by principal tension in the web.
2. Flexure-shear. A crack starts as a flexural crack on the tension face of a flexural member. It then extends up into the web, and develops into a diagonal shear crack. This can happen at a much lower principal tensile stress than that causing a web shear crack, because of the tensile stress concentration at the tip of the crack.

Figure 3 illustrates the two types of inclined cracking described above.

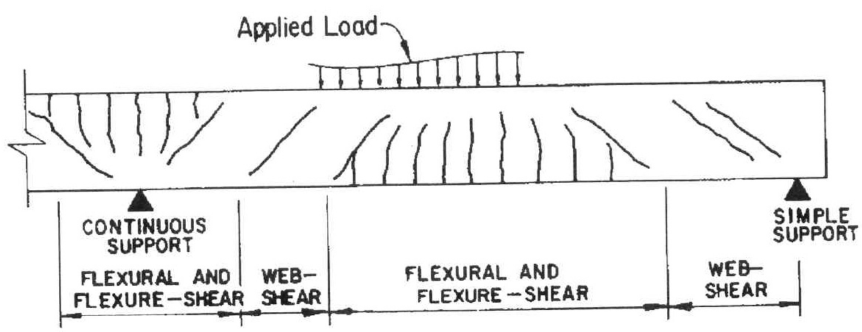


FIGURE 3

Web Shear

The apparent tensile strength of concrete in direction tension is about $4(f'_c)^{1/2}$. When the principal tension at the center of gravity of the cross section reaches $4(f'_c)^{1/2}$, a web shear crack will occur. The compression from the post-tensioning force helps to reduce the principal tension. The computation of principal tension due to combined shear and compression can be somewhat tedious, however, the code provides a simplified procedure.

$$V_{cw} = (((3.5(f'_c)^{1/2}) + 0.3f_{pc}))b_w d_p) + V_p$$

The term V_p in the above equation (Equation 11-12) is the vertical component of the tension in the post-tensioning tendons. This value is additive for web shear strength (but not for flexure-shear strength). The compression from post-tensioning increases the shear strength by about 30% of the P/A value, f_{pc}.

For non-post-tensioned beams, the principal tension at the center of gravity of the section is equal to the shear. So why does Equation 11-3 for shear in non-post-tensioned members permit only $2(f'_c)^{1/2}$ shear to be resisted by the concrete? Because shear strength is reduced by flexural cracking. In non-post-tensioned beams, shear is almost always influenced by flexural tension, whereas with post-tensioning the flexural cracking is reduced.

Flexural-Shear

In post-tensioned beams, flexural cracking is delayed by the prestress force, usually until the member is loaded beyond its service load capacity. Therefore it is worthwhile to account for the beneficial effects of post-tensioning in the analysis of the shear capacity of the section.

Equation 11-10 is used for shear resistance provided by the concrete, as governed by flexural cracks that develop into shear cracks. The shear strength of the concrete at a given cross section is taken equal to the shear at the section at the time a flexural crack occurs, plus a small increment of shear which transforms the flexural crack into an inclined crack. Equation 11-10 can be described as follows. In addition, for a better understanding the terms used in Equation 11-10, see Figure 4.

- V_{ci} = shear existing at the time of flexural cracking plus an added increment to convert it into a shear crack. The added increment is $0.6b_wd_p(f'_c)^{½}$.

- The shear existing at the time of flexural cracking is the dead load shear V_d plus the added shear $(V_iM_{cre})/M_{max}$; where V_i is the factored ultimate shear at the section, less the dead load shear, and M_{cre} is the added moment (over and above stresses due to prestress and dead load) causing $6(f'_c)^{½}$ tension in the extreme fiber.
- M_{cre} is calculated by finding the bottom fiber stress f_{pe} due to prestress, subtracting the bottom fiber stress f_d due to dead loads, adding $6(f'_c)^{½}$ tension and multiplying the results by the section modulus for the section resisting live loads (see Equation 11-11). Please note that in the discussion above, "bottom" means the tension face for continuous members.

- The term M_{max} is the factored ultimate moment of the section, less the deal load moment.

- The quantity $(V_iM_{cre})/M_{max}$ is the shear due to an added load (over and above the dead load) which causes the tensile stress in the extreme fiber to reach $6(f'_c)^{½}$.

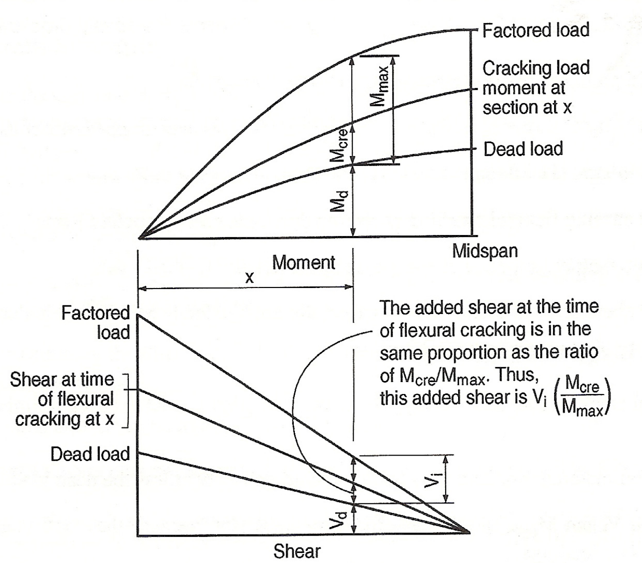


FIGURE 4

- After a flexural crack forms, a small amount of additional shear is needed to transform the crack into a shear crack. This is determined empirically, as shown in Figure 5. The intercept at 0.6 produces the first term in Equation 11-10. Also, it should be noted that the quantity -dp/2 shown in the expressions Figure 5 was later dropped, as a conservative simplification.

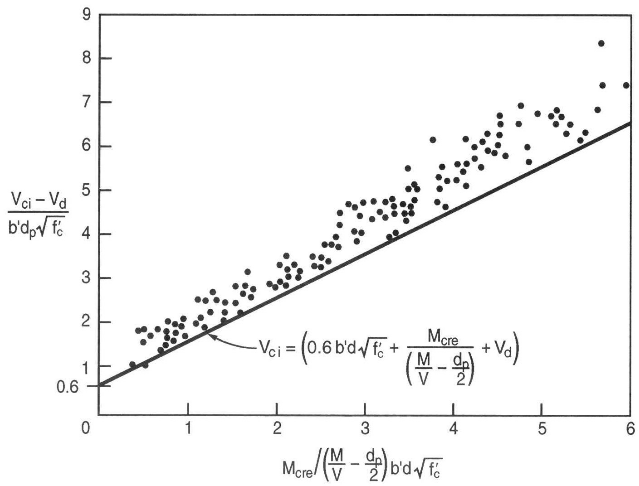


FIGURE 5

- M_{cre} is not the total cracking moment. It is not the same as M_{cr} that is used in the cracking moment flexural design check.

- It would seem that V_i and M_{max} should have the same subscript, because they both relate to the differences between the same two loadings, however, this is not the case. To make matters worse, the term "externally applied load' is ambiguous. Apparently, dead load is not regarded as "externally applied" by ACI, perhaps because the weight comes from the "internal" mass of the member. At the same time Section R11.4.3 states that "superimposed dead load" on a section should be considered an externally applied load. The commentary provides a good reason for this situation, but the confusion still exists.

- The flexural-shear strength must be checked at various locations along the span, a process that is tedious. Therefore, when using manual shear calculations, the simplified process described in Section 11.4.2 is adequate for most cases.

Shear Strength

The basic requirement for shear design of post-tensioned concrete member is the same as for a conventional reinforced concrete members in that the design shear strength, ΦV_n, must be greater than the factored shear force V_u at all sections; i.e. $\Phi V_n \geq V_u$

For both conventionally reinforced and post-tensioned concrete members, the nominal shear strength V_n is the sum of two components V_c and V_s; where V_c is the nominal shear strength provided by the concrete, and V_s is the nominal shear strength provided by the shear reinforcement. Therefore:

$$V_n = V_c + V_s,\ \text{and}\ \Phi V_c + \Phi V_s \geq V_u$$

The nominal shear strength, V_c, is assumed to be equal to the shear existing at the time an inclined crack forms in the concrete.

The nominal strength of vertical shear reinforcement (i.e. stirrups) in a post-tensioned beam is;

$$V_s = (A_v f_y d_p)/s \leq 8(f'_c)^{1/2} b_w d$$

A_v, as discussed in Lecture 6, equals 2 x A_b, where A_b is the area of one leg of a double leg stirrup.

Combining all of the equations above results in;

$$A_v/s = (V_u - \Phi V_c)/\Phi f_y d_p$$

Section 11.5.6 requires a minimum amount of shear reinforcement in beams when $V_u > \Phi V_c/2$, except where the total depth h is the smaller than the largest of the following:

a. 10 inches

b. 2.5 x the flange thickness

c. ½ the web thickness

When shear reinforcement is required the minimum amount of shear reinforcement per unit length of beam is the smaller of the following two quantities;

$$A_v/s = 0.75(f'_c)^{1/2}(b_w/f_y)\text{, but not less than } (50b_w)/f_y$$

$$\text{or } A_v/s = ((A_{ps} f_{pu})/(80 f_y d_p))(d_p/b_w)^{1/2}$$

Spacing of shear reinforcement in all cases shall not exceed 0.75h or 24 inches, however, where $V_s > (4(f'_c)^{1/2} b_w d_p)$, the maximum spacing shall be reduced by one-half.

Location for Computing the Maximum Factored Shear

Section 11.1.3 allows the maximum factored shear, V_u, to be computed at a predetermined distance (h/2) from the face of the support when all of the following conditions are satisfied:

a. The support reaction, in the direction of the applied shear, introduces compression into the end regions of the member.

b. Loads are applied at or near the top of the members.

c. No concentrated load occurs between the face of the support and the critical section.

For post-tensioned concrete sections, Section 11.1.3.2 states that the critical section for computing the maximum factored shear V_u is located at a distance of h/2 from the face of the support. This differs from the provisions for conventionally reinforced concrete members, in which the critical section is located at a distance "d" from the face of the support.

Shear-Torsion Strength

A detail discussion of the design of post-tensioned members for the combined effects of shear and torsion is beyond the scope of this lecture. Section 11.6 of ACI 318-05 includes provisions for the design of post-tensioned concrete members subjected to torsion that follow the same steps outlined in Lecture 6, with the exception that alternate equations and design criteria (similar to the special provisions associated with prestressed concrete members subjected to shear) are provided for prestressed members.

The method of shear-torsion analysis used in the POSTEN spreadsheet program discussed below is based on the design recommendations of the 3rd Edition of the PCI Design Handbook. This method of analysis was updated for the 6th Edition of the PCI Handbook, and serves as an alternate method of shear-torsion design of prestressed members to that provided by ACI 318-05. This updated PCI design methodology essentially follows that provided in the 3rd Edition except for the following modifications and refinements:

1. All of the design equations are expressed in terms of forces and moments instead of nominal stresses, which is in accordance with the latest ACI Code.

2. For the sake of simplicity, the torsional coefficient in the basic torsional stress equation is taken as one-third instead of a variable depending on the aspect ratio as was original used in the 3rd Edition method.

3. New equations for minimum torsional and shear web reinforcement are used based on more recent research data.

4. The shear-torsion interaction curve is based on the "concrete contribution curve" rather than on the "cracking curve" as was used in the original 3rd Edition method.

5. The expression for maximum torsional strength has been revised.

Two-Way Slab Systems:

Recent Code Changes

Recent changes reflected in the ACI 318-05 Code include:

- Section 6.4.4, which requires that construction joints in slabs be located at the middle third of the span is excluded from application to post-tensioned concrete. Most construction joints in continuous post-tensioned concrete slabs are located close to the quarter point of the span where the tendon profile is near mid-depth of the section. This approach allows for intermediate stressing of the tendons at construction joints at the centroid of the slab as required to avoid anchorage eccentricities.

- Section 18.3.3 defines two-way prestressed slab systems as Class U and reduces the maximum permissible flexural tensile stress, f_t, from $7.5(f'_c)^{½}$ to $6(f'_c)^{½}$.

General Requirements

Section 18.12 provides the analysis and design procedures for two-way post-tensioned slab systems and includes the following requirements:

1. The use of the Equivalent Frame Method of Section 13.7 (excluding Sections 13.7.7.4 and 13.7.7.5), or more detailed analysis procedures, is required for the determination of the factored moments and shears in post-tensioned slab systems. For two-way post-tensioned slabs, the equivalent frame slab-beam strips should not be divided into column and middle strips as is the case with a typical conventionally reinforced two-way slab. Instead, each panel or design strip should be analyzed as a single beam strip.

2. Spacing of tendons or groups of tendons in one direction shall not exceed 8 times the slab thickness or 5 feet. Spacing of tendons shall also provide a minimum average prestress, after allowance for all prestress losses, of 125 psi (unless a higher minimum P/A is required due to the environmental exposure of the structure) on the tributary slab section of a tendon or tendon group. Special consideration must be given to tendon spacing in slabs with concentrated loads.

3. A minimum of two tendons shall be provided through the critical shear section over each column in both directions. This provision, in conjunction with the limits on tendon spacing outlined in Item 2 above, provides specific guidance for distributing tendons in post-tensioned flat plates in accordance with the "banded" pattern discussed previously. This method of tendon placement is widely used and greatly simplifies detailing and installation procedures.

Shear Strength

Section 11.12.2 contains specific provisions for the calculation of shear strength in two-way post-tensioned concrete systems. At columns of two-way post-tensioned slabs (and footings) that use unbonded tendons and meet the bonded reinforcement requirements of Section 18.9.3, the shear strength V_n must not be taken greater than the shear strength V_c computed in accordance with Sections 11.12.2.1 or 11.12.2.2, unless shear reinforcement is provided in accordance with Sections 11.12.3 or 11.12.4. Section 11.12.2.2 gives the following value of the shear strength V_c at columns of two-way post-tensioned slabs:

$$V_c = (((\beta_p(f'_c)^{1/2}) + (0.3f_{pc}))b_o d_p) + V_p$$

Where: β_p = the smaller of 3.5 or $((\alpha_s d_p/b_o) + 1.5)$

$\alpha_s d_p/b_o$ (used to account for a decrease in shear strength affected by the perimeter area aspect ratio of the column) = 40 for interior columns, 30 for edge column and 20 for corner columns.

f_{pc} = the average value of f_{pc} for the two orthogonal directions

V_p = the vertical component of all (i.e. both directions) effective post-tensioning forces crossing the critical section

For the shear strength to be computed using the above equation the following must be satisfied, otherwise, Section 11.12.2.1 for conventionally reinforced slabs applies:

a. No portion of the column cross-section shall be closer to a discontinuous edge than 4 times the slab thickness.

b. f'_c shall not be taken greater than 5000 psi.

c. f_{pc} in each direction shall not be less than 125 psi, nor be taken greater than 500 psi.

In accordance with the above limitations, shear strength must be calculated using the smaller value of either Equations 11-33, 11-34, and 11-35 for post-tensioned slabs in which the columns are closer to the discontinuous edge than 4 times the slab thickness. For usual design conditions (i.e. typical slab thicknesses and column sizes), the controlling shear strength at edge columns will be $(4(f'_c)^{1/2}(b_o d_p))$.

Shear Strength with Moment Transfer

For moment transfer calculations, the controlling shear stress at columns of two-way post-tensioned slabs with bonded reinforcement provided in accordance with Section 18.9.3 is governed by Equation 11-36, which can be expressed as a shear stress for use in Equation 11-40 as:

$$v_c = (\beta_p (f'_c)^{1/2}) + (0.3 f_{pc}) + (V_p / b_o d_p)$$

If the permissible shear stress is computed using the above equation the provisions that were applicable to the equation for V_c must also be satisfied:

a. No portion of the column cross-section shall be closer to a discontinuous edge than 4 times the slab thickness.

b. f'_c shall not be taken greater than 5000 psi.

c. f_{pc} in each direction shall not be less than 125 psi, nor be taken greater than 500 psi.

For edge columns under moment transfer conditions, the controlling shear stress will be the same as that permitted for conventionally reinforced slabs. For usual design conditions, the governing shear stress at edge columns will be $4(f'_c)^{1/2}$.

Allowable Flexural Compressive Stress

In 1995, Section 18.4.2 increased the permissible concrete service load flexural compressive stress resulting from the total applied load from $0.45(f'_c)^{1/2}$ to $0.60(f'_c)^{1/2}$, but at the same time imposed a new limit of $0.45(f'_c)^{1/2}$ for sustained load. This involves some judgment on the part of the designer in determining the appropriate sustained load, as was discussed above for flexural-shear in beams.

fps for Unbonded Strands

As was indicated above, the nominal strength of unbonded strands is determined using Equations 18-5 for span-to-depth ratios > 35, which applies to almost all two-way post-tensioned slabs and plates.

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

$$f_{ps} < \text{the lesser of } f_{py} \text{ or } (f_{se} + 30{,}000)$$

Equation 18-5 provides values of f_{ps} which are generally 15,000 to 20,000 psi lower than the values of f_{ps} given by Equation 18-4. These lower values of f_{ps} are more compatible with values of f_{ps} obtained in recent tests of post-tensioned one-way slabs and two-way flat plates.