CHAPTER 24—SERVICEABILITY

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24.1—Scope

24.1.1 This chapter shall apply to member design for minimum serviceability, including (a) through (d):

(a) Deflections due to service-level gravity loads (24.2)
(b) Distribution of flexural reinforcement in one-way slabs and beams to control cracking (24.3)
(c) Shrinkage and temperature reinforcement (24.4)
(d) Permissible stresses in prestressed flexural members (24.5)

24.2—Deflections due to service-level gravity loads

24.2.1 Members subjected to flexure shall be designed with adequate stiffness to limit deflections or deformations that adversely affect strength or serviceability of a structure.

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R24.1—Scope

R24.1.1 This chapter prescribes serviceability requirements that are referenced by other chapters of the Code to provide adequate performance of structural members. This chapter does not stand on its own as a complete and cohesive compilation of serviceability requirements for the design of structural members. This chapter has no specific requirements for vibrations.

Cast-in-place floor systems designed in accordance with the minimum thickness and deflection requirements of 7.3, 8.3, 9.3, and 24.2 have generally been found, through experience, to provide vibration performance suitable for human comfort under typical service conditions. However, there may be situations where serviceability conditions are not satisfied, for example:

(a) Long spans and open floor plans
(b) Floors with strict vibration performance requirements such as precision manufacturing and laboratory spaces
(c) Facilities subject to rhythmic loadings or vibrating mechanical equipment

Prestressed floor systems are not subject to the minimum thickness requirements of 7.3, 8.3, and 9.3, and if precast they are frequently simple span systems. Consequently, these floor systems can be more susceptible to vibration.

Guidance on the consideration of vibrations in the design of floor systems and the evaluation of vibration frequency and amplitude for concrete floor systems is contained in the *PCI Design Handbook* (PCI MNL 120), *ATC Design Guide 1* (Applied Technology Council 1999), Mast (2001), Fanella and Mota (2014), and Wilford and Young (2006). An example application is described by West et al. (2008).

R24.2—Deflections due to service-level gravity loads

R24.2.1 This section is concerned only with deflections or deformations that may occur at service load levels. When time-dependent deflections are calculated, only the dead load and those portions of other loads that are sustained need be considered.

For nonprestressed one-way slabs and beams, including composite concrete members, the minimum overall thickness required by 7.3.1 and 9.3.1 is considered to satisfy the requirements of the Code for members not supporting or attached to nonstructural elements likely to be damaged by large deflections. For nonprestressed two-way construction, the minimum thickness required by 8.3.1 is considered to satisfy the requirements of the Code.

For nonprestressed members that do not meet these minimum thickness requirements, for nonprestressed one-way members that support or are attached to nonstructural elements likely to be damaged by large deflections, and for prestressed flexural members, deflections are required to be calculated by 24.2.3 through 24.2.5. Calculated deflections are limited to the values in Table 24.2.2.

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24.2.2 Deflections calculated in accordance with 24.2.3 through 24.2.5 shall not exceed the limits in Table 24.2.2.

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R24.2.2 The limitations given in Table 24.2.2 relate only to supported or attached nonstructural elements. For those structures in which structural members are likely to be affected by deflection or deformation of members to which they are attached in such a manner as to affect adversely the strength of the structure, these deflections and the resulting forces should be considered explicitly in the analysis and design of the structures as required by 24.2.1 (ACI PRC-209).

When time-dependent deflections are calculated, the portion of the deflection before attachment of the nonstructural elements may be deducted. In making this correction, use may be made of the curve in Fig. R24.2.4.1 for members of usual sizes and shapes.

Table 24.2.2—Maximum permissible calculated deflections

Member	Condition		Deflection to be considered	Deflection limitation[1]
Flat roofs	Not supporting or attached to nonstructural elements likely to be damaged by large deflections		Immediate deflection due to maximum of L_r, S, and R	$\ell/180$[2]
Floors			Immediate deflection due to L	$\ell/360$
Roof or floors	Supporting or attached to nonstructural elements	Likely to be damaged by large deflections	That part of the total deflection occurring after attachment of nonstructural elements, which is the sum of the time-dependent deflection due to all sustained loads and the immediate deflection due to any additional live load[3]	$\ell/480$[4]
		Not likely to be damaged by large deflections		$\ell/240$[5]

[1]For cantilevered members, ℓ shall be taken as twice the clear projection of the cantilever.
[2]Limit not intended to safeguard against ponding. Ponding shall be checked by calculations of deflection, including added deflections due to ponded water, and considering time-dependent effects of sustained loads, camber, construction tolerances, and reliability of provisions for drainage.
[3]Time-dependent deflection shall be calculated in accordance with 24.2.4, but shall be permitted to be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be calculated on basis of accepted engineering data relating to time-deflection characteristics of members similar to those being considered.
[4]Limit shall be permitted to be exceeded if measures are taken to prevent damage to supported or attached elements.
[5]Limit shall not exceed tolerance provided for nonstructural elements.

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24.2.3 *Calculation of immediate deflections*

24.2.3.1 Immediate deflections shall be calculated using methods or formulas for elastic deflections, considering effects of cracking and reinforcement on member stiffness.

24.2.3.2 Effect of variation of cross-sectional properties, such as haunches, shall be considered when calculating deflections.

24.2.3.3 Deflections in two-way slab systems shall be calculated taking into account size and shape of the panel, conditions of support, and nature of restraints at the panel edges.

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R24.2.3 *Calculation of immediate deflections*

R24.2.3.1 For calculation of immediate deflections of uncracked prismatic members, the usual methods or formulas for elastic deflections may be used with a constant value of E_cI_g along the length of the member. However, if the member is expected to crack at one or more sections, or if its depth varies along the span, a more rigorous calculation becomes necessary.

R24.2.3.3 For immediate deflections, the values of E_c and I_e specified in 24.2.3.4 and 24.2.3.5, respectively, may be used (ACI PRC-209). However, other procedures and other values of the stiffness E_cI_e may be used if they result in predictions of deflection in reasonable agreement with the results of comprehensive tests. Additional information on

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deflection of nonprestressed concrete structures is provided in ACI PRC-435.

24.2.3.4 Modulus of elasticity, E_c, shall be permitted to be calculated in accordance with 19.2.2.

24.2.3.5 For nonprestressed members, unless obtained by a more comprehensive analysis, effective moment of inertia, I_e, shall be calculated in accordance with Table 24.2.3.5 using:

$$M_{cr} = \frac{f_r I_g}{y_t} \tag{24.2.3.5}$$

Table 24.2.3.5—Effective moment of inertia, I_e

Service moment	Effective moment of inertia, I_e, in.^{4}	
$M_a \le (2/3)M_{cr}$	I_g	(a)
$M_a > (2/3)M_{cr}$	$\dfrac{I_{cr}}{1-\left(\dfrac{(2/3)M_{cr}}{M_a}\right)^2\left(1-\dfrac{I_{cr}}{I_g}\right)}$	(b)

R24.2.3.5 The effective moment of inertia approximation, developed by Bischoff (2005), has been shown to result in calculated deflections that have sufficient accuracy for a wide range of reinforcement ratios (Bischoff and Scanlon 2007). M_{cr} is multiplied by two-thirds to consider restraint that can reduce the effective cracking moment as well as to account for reduced tensile strength of concrete during construction that can lead to cracking that later affects service deflections (Scanlon and Bischoff 2008).

Before 2019, ACI 318 used a different equation (Branson 1965) to calculate I_e. The previous equation has subsequently been shown to underestimate deflections for members with low reinforcement ratios, which often occur in slabs, and does not consider the effects of restraint. For members with greater than 1 percent reinforcement and a service moment at least twice the cracking moment, there is little difference between deflections calculated using Branson (1965) or Bischoff and Scanlon (2007). Refer to ACI PRC-435.

24.2.3.6 For continuous one-way slabs and beams, I_e shall be permitted to be taken as the average of values obtained from Table 24.2.3.5 for the critical positive and negative moment sections.

24.2.3.7 For prismatic one-way slabs and beams, I_e shall be permitted to be taken as the value obtained from Table 24.2.3.5 at midspan for simple and continuous spans, and at the support for cantilevers.

R24.2.3.7 The use of the midspan section properties for continuous prismatic members is considered satisfactory in approximate calculations primarily because the midspan stiffness (including the effect of cracking) has the dominant effect on deflections, as shown by ACI PRC-435.5, ACI Committee 435 (1978), and Sabnis et al. (1974).

24.2.3.8 For prestressed Class U slabs and beams as defined in 24.5.2, it shall be permitted to calculate deflections based on I_g.

R24.2.3.8 Immediate deflections of Class U prestressed concrete members may be calculated by the usual methods or formulas for elastic deflections using the moment of inertia of the gross (uncracked) concrete section and the modulus of elasticity for concrete specified in 19.2.2.1.

24.2.3.9 For prestressed Class T and Class C slabs and beams as defined in 24.5.2, deflection calculations shall be based on a cracked transformed section analysis. It shall be permitted to base deflection calculations on a bilinear moment-deflection relationship or I_e in accordance with Eq. (24.2.3.9a)

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1-\left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \tag{24.2.3.9a}$$

where M_{cr} is calculated as

$$M_{cr} = \frac{(f_r + f_{pe}) I_g}{y_t} \tag{24.2.3.9b}$$

R24.2.3.9 The effective moment of inertia equation in 24.2.3.5 was revised in the 2019 Code. The revision is not applicable to prestressed members. Equation (24.2.3.9a) maintains the provisions of previous editions of the Code for these types of members. The *PCI Design Handbook* (PCI MNL 120) gives information on deflection calculations using a bilinear moment-deflection relationship and using an effective moment of inertia. Mast (1998) gives additional information on deflection of cracked prestressed concrete members.

Shaikh and Branson (1970) shows that the I_e method can be used to calculate deflections of Class C and Class T prestressed members loaded above the cracking load. For

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24.2.4 *Calculation of time-dependent deflections*

24.2.4.1 *Nonprestressed members*

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this case, the cracking moment should take into account the effect of prestress as provided in Eq. (24.2.3.9).

A method for predicting the effect of nonprestressed tension reinforcement in reducing creep camber is also given in Shaikh and Branson (1970), with approximate forms given in ACI PRC-209 and Branson (1970).

R24.2.4 *Calculation of time-dependent deflections*

R24.2.4.1 *Nonprestressed members*

Shrinkage and creep cause time-dependent deflections in addition to the elastic deflections that occur when loads are first placed on the structure. Such deflections are influenced by temperature, humidity, curing conditions, age at time of loading, amount of compression reinforcement, and magnitude of the sustained load. The expression given in this section is considered satisfactory for use with the Code procedures for the calculation of immediate deflections, and with the limits given in Table 24.2.2. The deflection calculated in accordance with this section is the additional time-dependent deflection due to the dead load and those portions of other loads that will be sustained for a sufficient period to cause significant time-dependent deflections.

Equation (24.2.4.1.1) was developed in Branson (1971). In Eq. (24.2.4.1.1), the term $(1 + 50\rho')$ accounts for the effect of compression reinforcement in reducing time-dependent deflections. $\xi = 2.0$ represents a nominal time-dependent factor for a 5-year duration of loading. The curve in Fig. R24.2.4.1 may be used to estimate values of ξ for loading periods less than 5 years.

If it is desired to consider creep and shrinkage separately, approximate equations provided in Branson (1965, 1971, 1977) and ACI Committee 435 (1966) may be used.

Because available data on time-dependent deflections of two-way slabs are too limited to justify more elaborate procedures, calculation of the additional time-dependent deflection for two-way construction in accordance with Eq. (24.2.4.1.1) is required to use the multipliers given in 24.2.4.1.3.

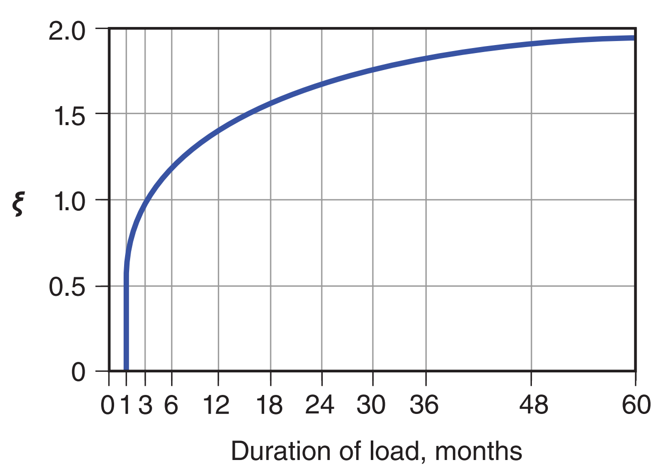

Fig. R24.2.4.1—Multipliers for time-dependent deflections.

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24.2.4.1.1 Unless obtained from a more comprehensive analysis, additional time-dependent deflection resulting from creep and shrinkage of flexural members shall be calculated as the product of the immediate deflection caused by sustained load and the factor λ_Δ

$$\lambda_\Delta = \frac{\xi}{1 + 50\rho'} \qquad (24.2.4.1.1)$$

24.2.4.1.2 In Eq. (24.2.4.1.1), ρ' shall be calculated at midspan for simple and continuous spans, and at the support for cantilevers.

24.2.4.1.3 In Eq. (24.2.4.1.1), values of the time-dependent factor for sustained loads, ξ, shall be in accordance with Table 24.2.4.1.3.

Table 24.2.4.1.3—Time-dependent factor for sustained loads

Sustained load duration, months	Time-dependent factor ξ
3	1.0
6	1.2
12	1.4
60 or more	2.0

24.2.4.2 *Prestressed members*

24.2.4.2.1 Additional time-dependent deflection of prestressed concrete members shall be calculated considering stresses in concrete and reinforcement under sustained load, and the effects of creep and shrinkage of concrete and relaxation of prestressed reinforcement.

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R24.2.4.2 *Prestressed members*

R24.2.4.2.1 Calculations should consider not only the increased deflections due to flexural stresses, but also the time-dependent deflections resulting from time-dependent shortening of the flexural member.

Prestressed concrete members are susceptible to increased time-dependent deflections. Prestressed concrete members shorten more with time than similar nonprestressed members due to the precompression in the slab or beam, which causes creep. Creep combined with concrete shrinkage can result in significant shortening of flexural members that continues for several years after construction. This shortening can reduce the tension in the prestressed reinforcement and increase time-dependent deflections.

Another factor that can influence time-dependent deflections of prestressed flexural members is adjacent concrete or masonry that is nonprestressed in the direction of the prestressed member. This can be a slab nonprestressed in the beam direction adjacent to a prestressed beam or a nonprestressed slab system. As the prestressed member tends to shrink and creep more than the adjacent nonprestressed concrete, the structure will tend to reach a compatibility of the shortening effects. This results in a reduction of the precompression in the prestressed member as the adjacent concrete absorbs the compression. This reduction in precompression of the prestressed member can occur over a period of years and will result in additional time-dependent deflections and an increase in tensile stresses in the prestressed member.

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Any suitable method for calculating time-dependent deflections of prestressed members may be used, provided all effects are considered. Guidance may be found in ACI PRC-209, ACI Committee 435 (1963), Branson et al. (1970), and Ghali and Favre (1986).

24.2.5 *Calculation of deflections of composite concrete construction*

24.2.5.1 If composite concrete flexural members are shored during construction so that, after removal of temporary supports, the dead load is resisted by the full composite section, it shall be permitted to consider the composite member equivalent to a monolithically cast member for calculation of deflections.

24.2.5.2 If composite concrete flexural members are not shored during construction, the magnitude and duration of load before and after composite action becomes effective shall be considered in calculating time-dependent deflections.

24.2.5.3 Deflections resulting from differential shrinkage of precast and cast-in-place components, and of axial creep effects in prestressed members, shall be considered.

R24.2.5 *Calculation of deflections of composite concrete construction*

R24.2.5.1 Composite concrete members are designed to meet the horizontal shear strength requirements of 16.4. Because few tests have been made to study the immediate and time-dependent deflections of composite members, the requirements given in this section are based on the judgment of ACI Committee 318 and on experience.

In 22.3.3.3, it is stated that distinction need not be made between shored and unshored members. This refers to strength calculations, not to deflections. Construction documents should indicate whether composite concrete design is based on shored or unshored construction, as required by 26.11.1.1.

24.3—Distribution of flexural reinforcement in one-way slabs and beams

24.3.1 Bonded reinforcement shall be distributed to control flexural cracking in tension zones of nonprestressed and Class C prestressed slabs and beams reinforced for flexure in one direction only.

R24.3—Distribution of flexural reinforcement in one-way slabs and beams

R24.3.1 Where service loads result in high stresses in the reinforcement, visible cracks should be expected, and steps should be taken in detailing of the reinforcement to control cracking. For reasons of durability and appearance, many fine cracks are preferable to a few wide cracks. Detailing practices limiting bar spacing will usually lead to adequate crack control where Grade 60 reinforcement is used.

Extensive laboratory experiments (Gergely and Lutz 1968; Kaar 1966; Base et al. 1966) involving deformed bars demonstrated that crack width at service loads is proportional to reinforcement stress. The significant variables reflecting reinforcement detailing were found to be thickness of concrete cover and the spacing of reinforcement.

Crack width is inherently subject to wide scatter even in careful laboratory experiments and is influenced by shrinkage and other time-dependent effects. Improved crack control is obtained where the reinforcement is well distributed over the zone of maximum concrete tension. Several bars at moderate spacing are much more effective in controlling cracking than one or two larger bars of equivalent area.

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24.3.2 Spacing of bonded reinforcement closest to the tension face shall not exceed the limits in Table 24.3.2, where c_c is the least distance from surface of deformed or prestressed reinforcement to the tension face. Calculated stress in deformed reinforcement, f_s, and calculated change in stress in bonded prestressed reinforcement, Δf_{ps}, shall be in accordance with 24.3.2.1 and 24.3.2.2, respectively.

Table 24.3.2—Maximum spacing of bonded reinforcement in nonprestressed and Class C prestressed one-way slabs and beams

Reinforcement type	Maximum spacing s	
Deformed bars or wires	Lesser of:	$15\left(\frac{40{,}000}{f_s}\right) - 2.5c_c$
		$12\left(\frac{40{,}000}{f_s}\right)$
Bonded prestressed reinforcement	Lesser of:	$\left(\frac{2}{3}\right)\left[15\left(\frac{40{,}000}{\Delta f_{ps}}\right) - 2.5c_c\right]$
		$\left(\frac{2}{3}\right)\left[12\left(\frac{40{,}000}{\Delta f_{ps}}\right)\right]$
Combined deformed bars or wires and bonded prestressed reinforcement	Lesser of:	$\left(\frac{5}{6}\right)\left[15\left(\frac{40{,}000}{\Delta f_{ps}}\right) - 2.5c_c\right]$
		$\left(\frac{5}{6}\right)\left[12\left(\frac{40{,}000}{\Delta f_{ps}}\right)\right]$

24.3.2.1 Stress f_s in deformed reinforcement closest to the tension face at service loads shall be calculated based on the unfactored moment, or it shall be permitted to take f_s as $(2/3)f_y$.

24.3.2.2 Change in stress, Δf_{ps}, in bonded prestressed reinforcement at service loads shall be equal to the calculated stress based on a cracked section analysis minus the decompression stress f_{dc}. It shall be permitted to take f_{dc} equal to the effective stress in the prestressed reinforcement f_{se}. The value of Δf_{ps} shall not exceed 36,000 psi. If Δf_{ps} does not exceed 20,000 psi, the spacing limits in Table 24.3.2 need not be satisfied.

24.3.3 If there is only one bonded bar, pretensioned strand, or bonded tendon nearest to the extreme tension face, the width of the extreme tension face shall not exceed s determined in accordance with Table 24.3.2

24.3.4 If the flange of a T-beam is in tension, the portion of the bonded flexural tension reinforcement not located over the beam web shall be distributed within the lesser of the effective flange width, as defined in accordance with 6.3.2 and $\ell_n/10$. If $\ell_n/10$ controls, additional bonded longitudinal reinforcement

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R24.3.2 The spacing of reinforcement is limited to control cracking (Beeby 1979; Frosch 1999; ACI Committee 318 1999). For the case of beams with Grade 60 reinforcement and 2 in. clear cover to the primary reinforcement, with f_s = 40,000 psi, the maximum bar spacing is 10 in.

Crack widths in structures are highly variable. The Code provisions for spacing are intended to limit surface cracks to a width that is generally acceptable in practice but may vary widely in a given structure.

The role of cracks in the corrosion of reinforcement is controversial. Research (Darwin et al. 1985; Oesterle 1997) shows that corrosion is not clearly correlated with surface crack widths in the range normally found with reinforcement stresses at service load levels. For this reason, the Code does not differentiate between interior and exterior exposures.

Only tension reinforcement nearest the tension face need be considered in selecting the value of c_c used in calculating spacing requirements. To account for prestressed reinforcement, such as strand, having bond characteristics less effective than deformed reinforcement, a two-thirds effectiveness factor is used in Table 24.3.2.

For post-tensioned members designed as cracked members, it will usually be advantageous to provide crack control by the use of deformed reinforcement, for which the provisions in Table 24.3.2 for deformed bars or wires may be used. Bonded reinforcement required by other provisions of the Code may also be used as crack control reinforcement.

R24.3.2.1 For applications in which crack control is critical, the designer should consider reducing the value of f_s to help control cracking. Research by Frosch et al. (2014) and Puranam (2018) supports the use of these design provisions for Grade 100 reinforcement.

R24.3.2.2 It is conservative to take the decompression stress f_{dc} equal to f_{se}, the effective stress in the prestressed reinforcement. The maximum limitation of 36,000 psi for Δf_{ps} is intended to be similar to the maximum allowable stress in Grade 60 reinforcement (f_s = 40,000 psi). The exemption for members with Δf_{ps} less than 20,000 psi reflects that many structures designed by working stress methods and with low reinforcement stress served their intended functions with very limited flexural cracking.

R24.3.4 In a T-beam, distribution of the negative moment reinforcement for control of cracking should take into account two considerations: 1) wide spacing of the reinforcement across the full effective width of flange may cause some wide cracks to form in the slab near the web; and 2) close spacing near the web leaves the outer regions of the flange

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satisfying 24.4.3.1 shall be provided in the outer portions of the flange.

24.3.5 The spacing of bonded flexural reinforcement in nonprestressed and Class C prestressed one-way slabs and beams subject to fatigue, designed to be watertight, or exposed to corrosive environments, shall be selected based on investigations and precautions specific to those conditions and shall not exceed the limits of 24.3.2.

24.4—Shrinkage and temperature reinforcement

24.4.1 Reinforcement to resist shrinkage and temperature stresses shall be provided in accordance with 24.4.3 or 24.4.4.

24.4.2 If shrinkage and temperature movements are restrained, the effects of T shall be considered in accordance with 5.3.6.

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unprotected. The one-tenth limitation is to guard against a spacing that is too wide, with some additional reinforcement required to protect the outer portions of the flange.

For T-beams designed to resist negative moments due to gravity and wind loads, all tensile reinforcement required for strength is located within the lesser of the effective flange width and $\ell_n/10$. Common practice is to place more than half of the reinforcement over the beam web. For T-beams resisting load combinations including earthquake effects, all reinforcement placed within the effective flange width may contribute to the beam flexural strength for the anticipated drift (refer to 18.7.3).

R24.3.5 Although a number of studies have been conducted, clear experimental evidence is not available regarding the crack width beyond which a corrosion danger exists (ACI PRC-222). Exposure tests indicate that concrete quality, adequate consolidation, and ample concrete cover may be of greater importance for corrosion protection than crack width at the concrete surface (Schießl and Raupach 1997).

Provisions related to increased concrete cover and durability of reinforcement is covered in 20.5, while durability of concrete is covered in 19.3.

R24.4—Shrinkage and temperature reinforcement

R24.4.1 Shrinkage and temperature reinforcement is provided at right angles to the principal reinforcement to control cracking and to tie the structure together. The provisions of this section are not intended for slabs-on-ground, unless the slab-on-ground is designed as a structural diaphragm.

R24.4.2 The area of shrinkage and temperature reinforcement required by 24.4.3.2 has been satisfactory where shrinkage and temperature movements are permitted to occur. Where structural walls or columns provide significant restraint to shrinkage and temperature movements, the restraint of volume changes causes tension in slabs, as well as displacements, shear forces, and flexural moments in columns or walls. In these cases, it may be necessary to increase the amount of slab reinforcement required by 24.4.3.2 due to the shrinkage and thermal effects in both principal directions (PCI MNL 120; Gilbert 1992). Top and bottom reinforcement are both effective in controlling cracks. Control strips during the construction period, which permit initial shrinkage to occur without causing an increase in stress, are also effective in reducing cracks caused by restraint.

Topping slabs also experience tension due to restraint of differential shrinkage between the topping and the precast elements or metal deck (which has zero shrinkage) that should be considered in reinforcing the slab. Consideration should be given to strain demands on reinforcement crossing joints of precast elements where most of the restraint is likely to be relieved.

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24.4.3 *Nonprestressed reinforcement*

24.4.3.1 Deformed reinforcement to resist shrinkage and temperature stresses shall conform to Table 20.2.2.4(a) and shall be in accordance with 24.4.3.2 through 24.4.3.5.

24.4.3.2 The ratio of deformed shrinkage and temperature reinforcement area to gross concrete area shall be greater than or equal to 0.0018.

24.4.3.3 The spacing of deformed shrinkage and temperature reinforcement shall not exceed the lesser of 5*h* and 18 in.

24.4.3.4 At all sections where required, deformed reinforcement used to resist shrinkage and temperature stresses shall develop f_y in tension.

24.4.3.5 For one-way precast slabs and one-way precast, prestressed wall panels, shrinkage and temperature reinforcement is not required in the direction perpendicular to the flexural reinforcement if (a) through (c) are satisfied.

(a) Precast members are not wider than 12 ft
(b) Precast members are not mechanically connected to cause restraint in the transverse direction
(c) Reinforcement is not required to resist transverse flexural stresses

24.4.4 *Prestressed reinforcement*

24.4.4.1 Prestressed reinforcement to resist shrinkage and temperature stresses shall conform to Table 20.3.2.2, and the effective prestress after losses shall provide an average compressive stress of at least 100 psi on gross concrete area.

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R24.4.3 *Nonprestressed reinforcement*

R24.4.3.2 The minimum ratio of deformed bar or welded wire reinforcement area to gross concrete area of 0.0018 is empirical but has been used satisfactorily for many years. The resulting area of reinforcement may be distributed within the member as deemed appropriate for specific conditions. Previous editions of the Code permitted a reduction in shrinkage and temperature reinforcement for reinforcement with yield strength greater than 60,000 psi. However, the mechanics of cracking suggest that increased yield strength provides no benefit for the control of cracking. If crack width or leakage prevention is a design limit state, refer to ACI PRC-224 or ACI CODE-350 for recommended reinforcement ratios.

R24.4.3.4 Splices and end anchorages of shrinkage and temperature reinforcement are to be designed to develop the specified yield strength of the reinforcement in accordance with Chapter 25.

R24.4.3.5 For precast, prestressed concrete members not wider than 12 ft, such as hollow-core slabs, solid slabs, or slabs with closely spaced ribs, there is usually no need to provide reinforcement to withstand shrinkage and temperature stresses in the short direction. This is generally also true for precast, nonprestressed floor and roof slabs. The 12 ft width is less than that in which shrinkage and temperature stresses can build up to a magnitude requiring reinforcement. In addition, much of the shrinkage occurs before the members are tied into the structure. Once in the final structure, the members are usually not as rigidly connected transversely as monolithic concrete, thus, the transverse restraint stresses due to both shrinkage and temperature change are significantly reduced.

The waiver does not apply where reinforcement is required to resist flexural stresses, such as in thin flanges of precast single and double tees.

R24.4.4 *Prestressed reinforcement*

R24.4.4.1 Prestressed reinforcement requirements have been selected to provide an effective force on the slab approximately equal to the force required to yield nonprestressed shrinkage and temperature reinforcement. This amount of prestressing—100 psi on the gross concrete area—has been used successfully on a large number of projects.

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The effects of slab shortening should be evaluated to ensure serviceable behavior of the structure. In most cases, the low level of prestressing recommended should not cause difficulties in a properly detailed structure. Additional attention may be required where thermal effects or restraint become significant.

24.5—Permissible stresses in prestressed concrete flexural members

24.5.1 *General*

24.5.1.1 Concrete stresses in prestressed flexural members shall be limited in accordance with 24.5.2 through 24.5.4 unless it is shown by test or analysis that performance will not be impaired.

24.5.1.2 For calculation of stresses at transfer of prestress, at service loads, and at cracking loads, elastic theory shall be used with assumptions (a) and (b):

(a) Strains vary linearly with distance from neutral axis in accordance with 22.2.1.
(b) At cracked sections, concrete resists no tension.

24.5.2 *Classification of prestressed flexural members*

24.5.2.1 Prestressed flexural members shall be classified as Class U, T, or C in accordance with Table 24.5.2.1, based on the extreme fiber stress in tension f_t in the precompressed tension zone calculated at service loads assuming an uncracked section.

Table 24.5.2.1—Classification of prestressed flexural members based on f_t

Assumed behavior	Class	Limits of f_t
Uncracked	U[1]	$f_t \leq 7.5\sqrt{f_c'}$
Transition between uncracked and cracked	T	$7.5\sqrt{f_c'} < f_t \leq 12\sqrt{f_c'}$
Cracked	C	$f_t > 12\sqrt{f_c'}$

[1]Prestressed two-way slabs shall be designed as Class U with $f_t \leq 6\sqrt{f_c'}$.

R24.5—Permissible stresses in prestressed concrete flexural members

R24.5.1 *General*

R24.5.1.1 Permissible stresses in concrete address serviceability but do not ensure adequate design strength, which should be checked in accordance with other Code requirements.

A mechanism is provided such that Code limits on stress need not inhibit the development of new products, materials, and techniques in prestressed concrete construction. Approvals for the design should be in accordance with 1.10 of the Code.

R24.5.2 *Classification of prestressed flexural members*

R24.5.2.1 Three classes of behavior of prestressed flexural members are defined. Class U members are assumed to behave as uncracked members. Class C members are assumed to behave as cracked members. The behavior of Class T members is assumed to be in transition between uncracked and cracked. These classes apply to both bonded and unbonded prestressed flexural members, but prestressed two-way slab systems are required to be designed as Class U with $f_t \leq 6\sqrt{f_c'}$.

The serviceability requirements for each class are summarized in Table R24.5.2.1. For comparison, Table R24.5.2.1 also shows corresponding requirements for nonprestressed members. Due to lack of strain compatibility, it is inappropriate to include the area of unbonded prestressed reinforcement in the calculation of gross or cracked section properties, although the effective prestress force should be considered when determining the location of the neutral axis. Conversely, the calculation of section properties should account for the area of the voids created by the sheathing or duct for unbonded prestressed reinforcement. A method for evaluating stresses, deflections, and crack control in cracked prestressed members is given in Mast (1998).

The precompressed tension zone is that portion of a prestressed member where flexural tension, calculated using gross section properties, would occur under unfactored dead and live loads if the prestress force was not present.

CODE

COMMENTARY

Prestressed concrete is usually designed so that the prestress force introduces compression into this zone, thus effectively reducing the magnitude of the tensile stress.

For corrosive environments, defined as an environment in which chemical attack (such as seawater, corrosive industrial atmosphere, or sewer gas) is encountered, cracking at service loads becomes more critical to long-term performance. For these conditions, cover should be increased in accordance with 20.5.1.4, and tensile stresses in the concrete reduced to minimize possible cracking at service loads.

Table R24.5.2.1—Serviceability design requirements

	Prestressed			Nonprestressed
	Class U	**Class T**	**Class C**	
Assumed behavior	Uncracked	Transition between uncracked and cracked	Cracked	Cracked
Section properties for stress calculation at service loads	Gross section 24.5.2.2	Gross section 24.5.2.2	Cracked section 24.5.2.3	No requirement
Allowable stress at transfer	24.5.3	24.5.3	24.5.3	No requirement
Allowable compressive stress based on uncracked section properties	24.5.4	24.5.4	No requirement	No requirement
Tensile stress at service loads 24.5.2.1	$\leq 7.5\sqrt{f_c'}$	$7.5\sqrt{f_c'} < f_t \leq 12\sqrt{f_c'}$	No requirement	No requirement
Deflection calculation basis	24.2.3.8, 24.2.4.2 Gross section	24.2.3.9, 24.2.4.2 Cracked section, bilinear	24.2.3.9, 24.2.4.2 Cracked section, bilinear	24.2.3, 24.2.4.1 Effective moment of inertia
Crack control	No requirement	No requirement	24.3	24.3
Computation of Δf_{ps} or f_s for crack control	—	—	Cracked section analysis	$M/(A_s \times$ lever arm), or $2/3f_y$
Side skin reinforcement	No requirement	No requirement	9.7.2.3	9.7.2.3

24.5.2.2 For Class U and T members, stresses at service loads shall be permitted to be calculated using the uncracked section.

24.5.2.3 For Class C members, stresses at service loads shall be calculated using the cracked transformed section.

R24.5.2.3 Prestressed members are classified based on the magnitude of the stress in the precompressed tension zone, calculated assuming the section remains uncracked. Once it is determined that a member is Class C, with $f_t > 12\sqrt{f_c'}$, subsequent calculations of service load stresses are based on the cracked transformed section.

24.5.3 *Permissible concrete stresses at transfer of prestress*

R24.5.3 *Permissible concrete stresses at transfer of prestress*

The concrete stresses at this stage are caused by the weight of the member and the force in the prestressed reinforcement after jacking reduced by the losses due to seating of the prestressed reinforcement and elastic shortening of the concrete. Shrinkage, creep, and relaxation effects are generally not included at this stage. These stresses apply to both pretensioned and post-tensioned concrete with proper modifications of the losses at transfer. Minimum values of f_{ci}' are provided in 19.2.1.4 for pretensioned members and 25.9.4.5.4 for post-tensioning.

CODE

24.5.3.1 Calculated extreme concrete fiber stress in compression immediately after transfer of prestress, but before time-dependent prestress losses, shall not exceed the limits in Table 24.5.3.1.

Table 24.5.3.1—Concrete compressive stress limits immediately after transfer of prestress

Location	Concrete compressive stress limits
End of simply-supported members	$0.70f_{ci}'$
All other locations	$0.60f_{ci}'$

24.5.3.2 Calculated extreme concrete fiber stress in tension immediately after transfer of prestress, but before time-dependent prestress losses, shall not exceed the limits in Table 24.5.3.2, unless permitted by 24.5.3.2.1.

Table 24.5.3.2—Concrete tensile stress limits immediately after transfer of prestress, without additional bonded reinforcement in tension zone

Location	Concrete tensile stress limits
Ends of simply-supported members	$6\sqrt{f_{ci}'}$
All other locations	$3\sqrt{f_{ci}'}$

24.5.3.2.1 The limits in Table 24.5.3.2 shall be permitted to be exceeded where additional bonded reinforcement in the tension zone resists the total tensile force in the concrete calculated with the assumption of an uncracked section.

24.5.4 *Permissible concrete compressive stresses at service loads*

24.5.4.1 For Class U and T members, the calculated extreme concrete fiber stress in compression at service loads, after allowance for all prestress losses, shall not exceed the limits in Table 24.5.4.1.

Table 24.5.4.1—Concrete compressive stress limits at service loads

Load condition	Concrete compressive stress limits
Prestress plus sustained load	$0.45f_c'$
Prestress plus total load	$0.60f_c'$

COMMENTARY

R24.5.3.1 The permissible concrete compressive stresses at transfer of prestress are higher at ends of simply supported members than at other locations based on research in the precast, prestressed concrete industry (Castro et al. 2004; Dolan and Krohn 2007; Hale and Russell 2006).

R24.5.3.2 The tensile stress limits of $3\sqrt{f_{ci}'}$ and $6\sqrt{f_{ci}'}$ refer to tensile stresses at transfer of prestress at locations other than the precompressed tension zone. Where tensile stresses exceed the permissible values, the total force in the tensile stress zone may be calculated and reinforcement proportioned on the basis of this force at a stress of $0.6f_y$, but not more than 30,000 psi. The effects of creep and shrinkage begin to reduce the tensile stress almost immediately; however, some tension remains in these locations after allowance is made for all prestress losses.

R24.5.4 *Permissible concrete compressive stresses at service loads*

R24.5.4.1 The compressive stress limit of $0.45f_c'$ was originally established to decrease the probability of failure of prestressed concrete members due to repeated loads. This limit also seemed reasonable to preclude excessive creep deformation. At higher values of stress, creep strains tend to increase more rapidly as applied stress increases.

Fatigue tests of prestressed concrete beams have shown that concrete compressive failures are not the controlling criterion. Therefore, the stress limit of $0.60f_c'$ permits a one-third increase in allowable compressive stress for members subject to transient loads.

Sustained live load is any portion of the service live load that will be sustained for a sufficient period to cause significant time-dependent deflections. Thus, when the sustained live and dead loads are a large percentage of the total service load, the $0.45f_c'$ limit of Table 24.5.4.1 typically controls. On the other hand, when a large portion of the total service load consists of a transient or temporary service live load, the increased stress limit of $0.60f_c'$ typically controls.

The compression limit of $0.45f_c'$ for prestress plus sustained loads will continue to control the time-dependent behavior of prestressed members.