

CHAPTER 21—STRENGTH REDUCTION FACTORS

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21.1—Scope

21.1.1 This chapter shall apply to the selection of strength reduction factors used in design, except as permitted by [Chapter 27](#).

21.2—Strength reduction factors for structural concrete members and connections

21.2.1 Strength reduction factors ϕ shall be in accordance with Table 21.2.1, except as modified by 21.2.2, 21.2.3, 21.2.4, and 21.2.5.

R21.1—Scope

R21.1.1 The purposes of strength reduction factors ϕ are: (1) to account for the probability of under-strength members due to variations in material strengths and dimensions; (2) to account for inaccuracies in the design equations; (3) to reflect the available ductility and required reliability of the member under the load effects being considered; and (4) to reflect the importance of the member in the structure ([MacGregor 1976](#); [Winter 1979](#)).

R21.2—Strength reduction factors for structural concrete members and connections

R21.2.1 The strength reduction factors in the Code are compatible with the [ASCE/SEI 7](#) load combinations, which are the basis for the required factored load combinations in [Chapter 5](#).

The following notes pertain to the table entries by letter identifier:

(e) Bracket and corbel behavior is predominantly controlled by shear; therefore, a single value of $\phi = 0.75$ is used for all potential modes of failure.

(f) The strength reduction factor ϕ for plain concrete members is the same for all potential modes of failure. Because both the flexural tension strength and shear strength for plain concrete depend on the tensile strength of the concrete, without the reserve strength or ductility that might otherwise be provided by reinforcement, equal strength reduction factors for moment and shear are considered to be appropriate.

(h) Laboratory tests of post-tensioned anchorage zones ([Breen et al. 1994](#)) indicate a wide range of scatter in the results. This observation is addressed with a ϕ -factor of 0.85 and by limiting the nominal compressive strength of unconfined concrete in the general zone to $0.7\lambda f_{ci}'$ in [25.9.4.5.2](#), where λ is defined in [19.2.4](#). Thus, the effective design strength of unconfined concrete is $0.85 \times 0.7\lambda f_{ci}' = 0.6\lambda f_{ci}'$ in the general zone.

(l) Non-redundant connections are those without alternate load paths. Such connections can result in sudden failure. Refer to [R17.5.3](#).

(m) Redundant connections are those with alternate load paths. Refer to [R17.5.3](#).

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Table 21.2.1—Strength reduction factors ϕ

Action		ϕ	Exceptions
(a)	Moment, axial force, or combined moment and axial force	0.65 to 0.90 in accordance with 21.2.2	Near ends of pretensioned members where strands are not fully developed, ϕ shall be in accordance with 21.2.3.
(b)	Shear	0.75	Additional requirements are given in 21.2.4 for structures designed to resist earthquake effects.
(c)	Torsion	0.75	—
(d)	Bearing	0.65	—
Structural Element			
(e)	Brackets and corbels	0.75	—
(f)	Plain concrete elements	0.60	—
(g)	Struts, ties, nodal zones, and bearing areas designed in accordance with strut-and-tie method in Chapter 23	0.75	—
Anchorage Condition			
(h)	Post-tensioned anchorage zones	0.85	—
(i)	Components of connections of precast members controlled by yielding of steel elements in tension	0.90	—
(j)	Anchorage of reinforcing bars for breakout strength of reinforcing bar groups in accordance with 25.4.1.5 and 25.4.11.	0.75	Development length of reinforcement calculated in accordance with 25.4.2 through 25.4.9 does not require ϕ . Additional requirements are given in 21.2.5 for structures designed to resist earthquake effects.
(k)	Anchor reinforcement in accordance with 17.5.2.1	0.90	—
(l)	Concrete failure of anchors in tension, nonredundant	0.65	—
(m)	Concrete failure of anchors in tension, redundant	0.75	—
(n)	Concrete failure of anchors in shear	0.75	—
(o)	Anchor steel, tension, ductile ^{[1],[2]}	0.75	ϕ is based on using f_{uta} to determine the nominal strength of anchors.
(p)	Anchor steel, tension, nonductile ^{[1],[2]}	0.65	—
(q)	Anchor steel, shear, ductile ^{[1],[2]}	0.65	ϕ accounts for non-uniform distribution of shear in connections with multiple anchors.
(r)	Anchor steel, shear, nonductile ^{[1],[2]}	0.60	—

^[1]Anchor components such as bolt or sleeve.

^[2]Ductility established in accordance with ACI CODE-355.2 or ACI CODE-355.4.

21.2.2 Strength reduction factor for moment, axial force, or combined moment and axial force shall be in accordance with Table 21.2.2, except as modified by 21.2.2.3.

R21.2.2 The nominal strength of a member that is subjected to moment or combined moment and axial force is determined for the condition where the strain in the extreme compression fiber is equal to the assumed strain limit of 0.003. The net tensile strain ϵ_t is the tensile strain calculated in the extreme tension reinforcement at nominal strength, exclusive of strains due to prestress, creep, shrinkage, and temperature. The net tensile strain in the extreme tension reinforcement is determined from a linear strain distribution at nominal strength, shown in Fig. R21.2.2a for a nonprestressed member.

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Members subjected to only axial compression are considered to be compression-controlled and members subjected to only axial tension are considered to be tension-controlled.

If the net tensile strain in the extreme tension reinforcement is sufficiently large ($\geq \epsilon_{ty} + 0.003$), the section is defined as tension-controlled, for which warning of failure by excessive deflection and cracking may be expected. The limit $\geq \epsilon_{ty} + 0.003$ provides sufficient ductility for most applications. Before the 2019 Code, the tension-controlled limit on ϵ_t was defined as 0.005 established primarily on the basis of Grade 60 nonprestressed reinforcement and prestressed reinforcement, with some consideration given to higher grades of nonprestressed reinforcement (Mast 1992). Beginning with the 2019 Code, to accommodate nonprestressed reinforcement of higher grades, the tension-controlled limit on ϵ_t in Table 21.2.2 is defined as $\epsilon_{ty} + 0.003$. This expression is consistent with the recommendations of Mast (1992) for the general case of reinforcement other than Grade 60, and test data show that the expression leads to elements with adequate ductility.

One condition where greater ductile behavior is required is in design for redistribution of moments in continuous members and frames, which is addressed in 6.6.5. Because redistribution of moment depends on the ductility available in the hinge regions, redistribution of moment is limited to sections that have a net tensile strain of at least 0.0075.

If the net tensile strain in the extreme tension reinforcement is small ($\leq \epsilon_{ty}$), a brittle compression failure condition is expected, with little warning of impending failure. Before ACI CODE-318-14, the compression-controlled strain limit was defined as 0.002 for Grade 60 reinforcement and all prestressed reinforcement, but it was not explicitly defined for other types of reinforcement. The compression-controlled strain limit ϵ_{ty} is defined in 21.2.2.1 and 21.2.2.2 for deformed and prestressed reinforcement, respectively.

Beams and slabs are usually tension-controlled, whereas columns may be compression-controlled. Some members, such as those with small axial forces and large bending moments, experience net tensile strain in the extreme tension reinforcement between the limits of ϵ_{ty} and $(\epsilon_{ty} + 0.003)$. These sections are in a transition region between compression-controlled and tension-controlled.

This section specifies the appropriate strength reduction factors for tension-controlled and compression-controlled sections, and for intermediate cases in the transition region. Beginning with the 2019 Code, the expression $(\epsilon_{ty} + 0.003)$ defines the limit on ϵ_{ty} for tension-controlled behavior in Table 21.2.2. For sections subjected to combined axial force and moment, design strengths are determined by multiplying both P_n and M_n by the appropriate single value of ϕ .

A lower ϕ -factor is used for compression-controlled sections than for tension-controlled sections because compression-controlled sections have less ductility, are more sensitive to variations in concrete strength, and generally occur in members that support larger loaded areas than members

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with tension-controlled sections. Columns with spiral reinforcement are assigned a higher ϕ -factor than columns with other types of transverse reinforcement because spiral columns have greater ductility and toughness. For sections within the transition region, the value of ϕ may be determined by linear interpolation, as shown in Fig. R21.2.2b.

The variation of ϕ with P_n in accordance with 21.2.2.3 is shown in Fig. R21.2.2.c. If $P_{n,bal}$ is less than $0.1f'_cA_g$, 21.2.2.3 does not apply and ϕ is determined from Table 21.2.2. For nonprestressed members with flanged sections or with asymmetric reinforcement, the design axial compressive strength ϕP_n computed with ϕ from Table 21.2.2 can exceed $\phi_{cc}P_{n,bal}$ in the transition and tension-controlled regions and become unconservative in some cases (Lequesne and Pincheira 2014). The requirements of 21.2.2.3 are intended to prevent this occurrence. For some sections, the transition in Fig. R21.2.2c can result in lower axial and flexural design strengths than those calculated with the 2019 edition of the Code.

Table 21.2.2—Strength reduction factor ϕ for moment, axial force, or combined moment and axial force based on net tensile strain ϵ_t

Net tensile strain ϵ_t	Classification	ϕ			
		Type of transverse reinforcement			
		Spirals conforming to 25.7.3		Other	
$\epsilon_t \leq \epsilon_{ty}$	Compression-controlled	0.75	(a)	0.65	(b)
$\epsilon_{ty} < \epsilon_t < \epsilon_{ty} + 0.003$	Transition ^[1]	$0.75 + 0.15 \frac{(\epsilon_t - \epsilon_{ty})}{(0.003)}$	(c)	$0.65 + 0.25 \frac{(\epsilon_t - \epsilon_{ty})}{(0.003)}$	(d)
$\epsilon_t \geq \epsilon_{ty} + 0.003$	Tension-controlled	0.90	(e)	0.90	(f)

^[1]For sections classified as transition, it shall be permitted to use ϕ corresponding to compression-controlled sections.

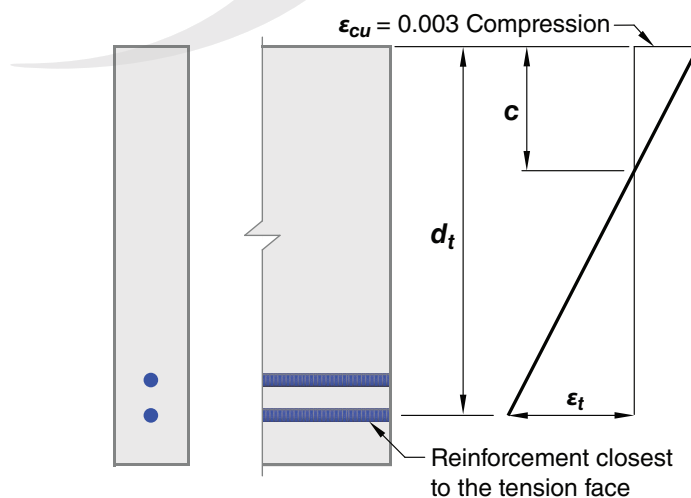


Fig. R21.2.2a—Strain distribution and net tensile strain in a nonprestressed member.

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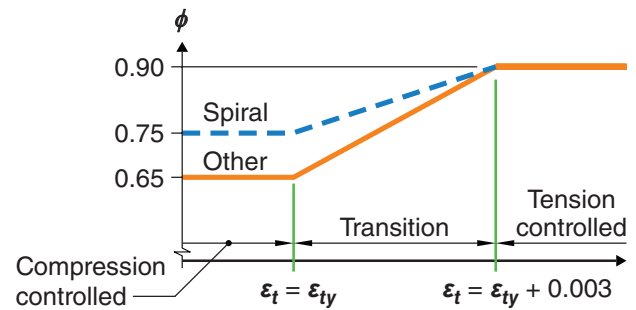


Fig. R21.2.2b—Variation of ϕ with net tensile strain in extreme tension reinforcement, ϵ_t .

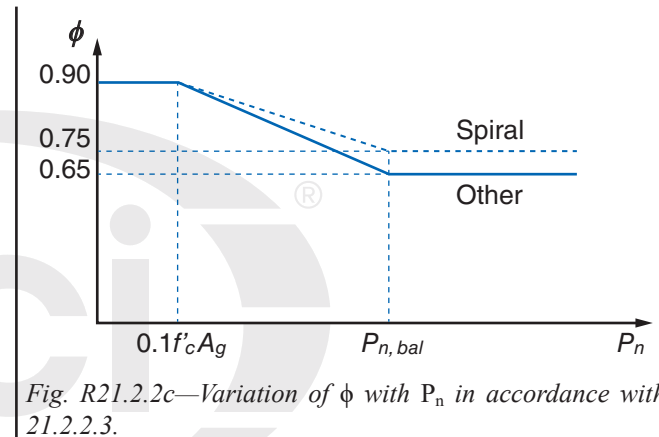


Fig. R21.2.2c—Variation of ϕ with P_n in accordance with 21.2.2.3.

21.2.2.1 For deformed reinforcement, ϵ_{ty} shall be f_y/E_s . For Grade 60 deformed reinforcement, it shall be permitted to take ϵ_{ty} equal to 0.002.

21.2.2.2 For all prestressed reinforcement, ϵ_{ty} shall be taken as 0.002.

21.2.2.3 For nonprestressed members subject to combined moment and axial compression with $0.1f'_cA_g \leq P_n \leq P_{n,bal}$, ϕ computed from Table 21.2.2 shall not exceed that determined from linear interpolation between 0.9 at $0.1f'_cA_g$ and ϕ_{cc} at $P_{n,bal}$.

21.2.3 For sections in pretensioned flexural members where all strands are not fully developed, ϕ for moment shall be calculated in accordance with Table 21.2.3, where ℓ_{tr} is calculated using Eq. (21.2.3), ϕ_p is the value of ϕ determined in accordance with Table 21.2.2 at the cross section closest to the end of member where all strands are developed, and ℓ_d is given in 25.4.8.1.

$$\ell_{tr} = \left(\frac{f_{se}}{3000} \right) d_b \quad (21.2.3)$$

R21.2.3 If a critical section along a pretensioned member occurs in a region where not all the strands are fully developed, bond slip failure may occur. This mode of failure resembles a brittle shear failure; hence, ϕ values for flexure are reduced relative to the value of ϕ at the cross section where all strands are fully developed. For sections between the end of the transfer length and the end of the development length, the value of ϕ may be determined by linear interpolation, as shown in Fig. R21.2.3a, where ϕ_p corresponds to the value of ϕ at the cross section closest to the end of the member where all strands are fully developed.

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Table 21.2.3—Strength reduction factor ϕ for sections near the end of pretensioned members

Condition near end of member	Stress in concrete under service load ^[1]	Distance from end of member to section under consideration	ϕ	
All strands bonded	Not applicable	$\leq \ell_{tr}$	0.75	(a)
		ℓ_{tr} to ℓ_d	Linear interpolation from 0.75 to ϕ_p ^[2]	(b)
One or more strands debonded	No tension calculated	$\leq (\ell_{db} + \ell_{tr})$	0.75	(c)
		$(\ell_{db} + \ell_{tr})$ to $(\ell_{db} + \ell_d)$	Linear interpolation from 0.75 to ϕ_p ^[2]	(d)
	Tension calculated	$\leq (\ell_{db} + \ell_{tr})$	0.75	(e)
		$(\ell_{db} + \ell_{tr})$ to $(\ell_{db} + 2\ell_d)$	Linear interpolation from 0.75 to ϕ_p ^[2]	(f)

^[1]Stress calculated using gross cross-sectional properties in extreme concrete fiber of precompressed tension zone under service loads after allowance for all prestress losses at section under consideration.

^[2]It shall be permitted to use a strength reduction factor of 0.75.

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Where bonding of one or more strands does not extend to the end of the member, instead of more rigorous analysis, ϕ should be taken as 0.75 from the end of the member to the end of the transfer length of the strand with the longest debonded length. Beyond this point, ϕ may be varied linearly to ϕ_p at the cross section where all strands are developed, as shown in Fig. R21.2.3b. Alternatively, the value of ϕ may be taken as 0.75 until all strands are fully developed. Embedment of debonded strand is considered to begin at the termination of the debonding sleeves. Beyond this point, the provisions of 25.4.8.1 are used to determine whether the strands develop over a length of ℓ_d or $2\ell_d$, depending on the calculated stress in the precompressed tension zone under service loads (Fig. R21.2.3b).

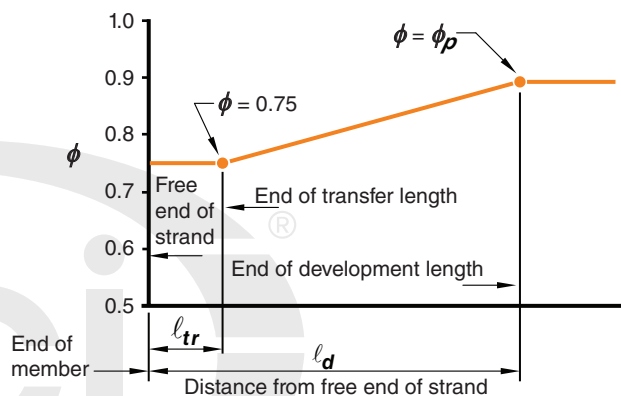
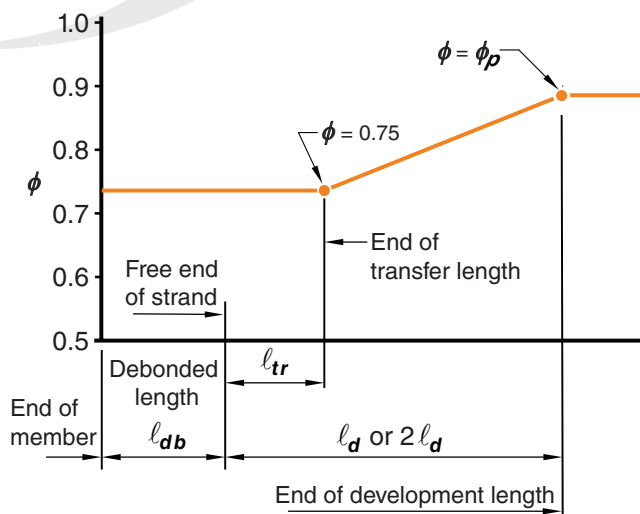


Fig. R21.2.3a—Variation of ϕ with distance from the free end of strand in pretensioned member with fully bonded strands.



Note: The location of the end of development length depends on the calculated stresses in the extreme concrete fiber of the precompressed tension zone under service loads.

Fig. R21.2.3b—Variation of ϕ with distance from the free end of strand in pretensioned member with debonded strands.

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21.2.4 For structures that rely on elements in (a), (b), or (c) to resist earthquake effects E , the value of ϕ for shear shall be modified in accordance with 21.2.4.1 through 21.2.4.4:

- (a) Special moment frames
- (b) Special structural walls
- (c) Intermediate precast structural walls in structures assigned to Seismic Design Category D, E, or F

21.2.4.1 For any member designed to resist E , except for walls where $\Omega_v \geq 1.5$ or if $\omega_v \Omega_v$ is taken equal to Ω_o , ϕ for shear shall be 0.60 if the nominal shear strength of the member is less than the shear corresponding to the development of the nominal moment strength of the member. The nominal moment strength shall be the maximum value calculated considering factored axial loads from load combinations that include E .

21.2.4.2 For diaphragms, ϕ for shear shall not exceed the least value of ϕ for shear used for the vertical components of the primary seismic-force-resisting system.

21.2.4.3 For foundation elements supporting the primary seismic-force-resisting system, ϕ for shear shall not exceed the least value of ϕ for shear used for the vertical components of the primary seismic-force-resisting system.

21.2.4.4 For beam-column joints of special moment frames and diagonally reinforced coupling beams, ϕ for shear shall be 0.85.

21.2.5 For reinforcing bar groups governed by concrete breakout in seismic-force-resisting systems, ϕ for tension shall be 0.65.

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R21.2.4.1 This provision addresses shear-controlled members, such as low-rise walls, portions of walls between openings, or diaphragms, for which nominal shear strength is less than the shear corresponding to development of nominal flexural strength for the pertinent loading conditions. The reduced value of ϕ does not apply to a wall designed with $\Omega_v \geq 1.5$ or if $\omega_v \Omega_v$ is taken equal to Ω_o because design shear forces have been amplified to account for flexural overstrength.

R21.2.4.2 Short structural walls were the primary vertical elements of the lateral-force-resisting system in many of the parking structures that sustained damage during the 1994 Northridge earthquake. In some cases, walls remained essentially linear elastic, while diaphragms responded inelastically. This provision is intended to increase strength of the diaphragm and its connections in buildings for which the shear strength reduction factor for walls is 0.60, as those structures tend to have relatively high overstrength.

R21.2.4.3 This provision is intended to provide consistent reliability for shear in foundation elements that support shear-controlled walls designed with a strength reduction factor of 0.6.

R21.2.5 The reduced ϕ factor is based on judgment and reflects a lack of data on the performance of bar groups under earthquake loading.

Notes

