CHAPTER 25—REINFORCEMENT DETAILS

25.1—Scope

25.1.1 This chapter shall apply to reinforcement details, including:

(a) Minimum spacing
(b) Standard hooks, seismic hooks, and crossties
(c) Development and anchorage of reinforcement
(d) Splices
(e) Bundled reinforcement
(f) Transverse reinforcement
(g) Post-tensioning anchorages and couplers

25.1.2 Provisions of 25.9 shall apply to anchorage zones for post-tensioned tendons.

R25.1—Scope

Recommended methods and standards for preparing design drawings, typical details, and drawings for the fabrication and placing of steel reinforcement in reinforced concrete structures are given in the *ACI Detailing Manual* (MNL-66).

All provisions in the Code relating to bar, wire, or strand diameter (and area) are based on the nominal dimensions of the reinforcement as given in the appropriate ASTM specification. Nominal dimensions are equivalent to those of a circular area having the same weight per foot as the ASTM designated bar, wire, or strand sizes. Cross-sectional area of reinforcement is based on nominal dimensions.

R25.1.1 In addition to the requirements in this chapter that affect detailing of reinforcement, detailing specific to particular members is given in the corresponding member chapters. Additional detailing associated with structural integrity requirements is covered in 4.10.

25.2—Minimum spacing of reinforcement

25.2.1 For parallel nonprestressed reinforcement in a horizontal layer, clear spacing shall be at least the greatest of 1 in., d_b, and $(4/3)d_{agg}$.

25.2.2 For parallel nonprestressed reinforcement placed in two or more horizontal layers, reinforcement in the upper layers shall be placed directly above reinforcement in the bottom layer with a clear spacing between layers of at least 1 in.

R25.2—Minimum spacing of reinforcement

The minimum limits are set to permit concrete to flow readily into spaces between bars and between bars and forms without honeycombs, and to ensure against concentration of bars on a line that may cause shear or shrinkage cracking. Use of nominal bar diameter to define minimum spacing permits a uniform criterion for all bar sizes. In 2014, the size limitations on aggregates were translated to minimum spacing requirements, and are provided to promote proper encasement of reinforcement and to minimize honeycombing. The limitations associated with aggregate size need not be satisfied if, in the judgment of the licensed design professional, the workability and methods of consolidation of the concrete are such that the concrete can be placed without creating honeycombs or voids.

The development lengths given in 25.4 are a function of the bar spacing and cover. As a result, it may be desirable to use larger than minimum bar spacing or cover in some cases.

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25.2.3 For longitudinal reinforcement in columns, pedestals, struts, and boundary elements in walls, clear spacing between bars shall be at least the greatest of 1.5 in., $1.5d_b$, and $(4/3)d_{agg}$.

25.2.4 For pretensioned strands at ends of a member, minimum center-to-center spacing s shall be the greater of the value in Table 25.2.4, and $[(4/3)d_{agg} + d_b]$.

Table 25.2.4—Minimum center-to-center spacing of pretensioned strands at ends of members

f_{ci}', psi	Nominal strand diameter, in.	Minimum s
< 4000	All	$4d_b$
≥ 4000	< 0.5 in.	$4d_b$
	0.5 in.	1-3/4 in.
	0.6 in.	2 in.

25.2.5 For pretensioned wire at ends of a member, minimum center-to-center spacing, s, shall be the greater of $5d_b$ and $[(4/3)d_{agg} + d_b]$.

25.2.6 Reduced vertical spacing including bundling of prestressed reinforcement shall be permitted in the middle portion of a span.

25.2.7 For parallel nonprestressed reinforcement in shotcrete members, the clear spacing shall be in accordance with (a) or (b):

(a) The clear spacing between bars shall be at least the greater of $6d_b$ and 2-1/2 in.
(b) If two curtains of reinforcement are provided, the clear spacing between bars in the curtain nearer the nozzle shall be at least $12d_b$. The clear spacing between bars in the remaining curtain shall conform to (a).

25.2.7.1 It shall be permitted to use a clear spacing that does not meet 25.2.7(a) or 25.2.7(b) provided shotcrete mockup panels are used to demonstrate proper reinforcement encasement in accordance with (a) and (b):

(a) The shotcrete mockup panels shall be representative of the most complex reinforcement configurations to be encountered.
(b) The licensed design professional shall specify the shotcrete mock-up panel quantity, frequency of shooting per nozzleman and member type, and panel thickness to verify reinforcement encasement.

25.2.8 For prestressed strands in shotcrete members, minimum center-to-center spacing, s, shall satisfy 25.2.4, except as permitted in 25.2.6.

25.2.9 For prestressed wire in shotcrete members, minimum center-to-center spacing, s, shall satisfy the

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R25.2.4 The decreased spacing for transfer strengths of 4000 psi or greater is based on Deatherage et al. (1994) and Russell and Burns (1996).

R25.2.7.1 Information on shotcrete mockup panels is provided in ACI PRC-506, and information on evaluating shotcrete is provided in ACI PRC-506.4.

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requirements for wire in 25.2.5, except as permitted in and 25.2.6.

25.2.10 For ties, hoops, and spiral reinforcement in columns to be placed with shotcrete, minimum clear spacing shall be 3 in.

25.2.10.1 It shall be permitted to use a clear spacing other than 3 in. provided shotcrete mockup panels are used to demonstrate proper encasement of the reinforcement in accordance with 25.2.7.1

25.3—Standard hooks, seismic hooks, crossties, and minimum inside bend diameters

25.3.1 Standard hooks for the development of deformed bars in tension shall conform to Table 25.3.1.

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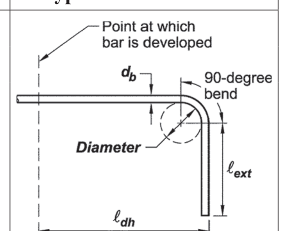
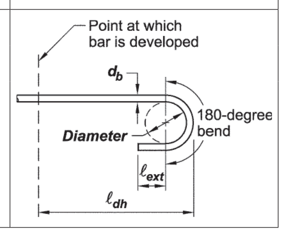
R25.2.10 Shotcrete is usually not used in new construction for columns because the close spacing between ties, hoops, or spiral reinforcement makes it difficult to achieve adequate encasement of the column longitudinal reinforcement. Spacing closer than required in 25.2.10 requires approval by the licensed design professional based on shotcrete mockup panels demonstrating that the reinforcement can be encased without voids.

R25.3—Standard hooks, seismic hooks, crossties, and minimum inside bend diameters

R25.3.1 Standard bends in reinforcing bars are described in terms of the inside diameter of bend because the inside bend diameter is easier to measure than the radius of bend. The primary factors affecting the minimum bend diameter are feasibility of bending without breakage and avoidance of crushing the concrete inside the bend.

Table 25.3.1—Standard hook geometry for development of deformed bars in tension

Type of standard hook	Bar size	Minimum inside bend diameter, in.	Straight extension[1] ℓ_{ext}, in.	Type of standard hook
90-degree hook	No. 3 through No. 8	$6d_b$	$12d_b$	
	No. 9 through No. 11	$8d_b$		
	No. 14 and No. 18	$10d_b$		
180-degree hook	No. 3 through No. 8	$6d_b$	Greater of $4d_b$ and 2.5 in.	
	No. 9 through No. 11	$8d_b$		
	No. 14 and No. 18	$10d_b$		

[1]A standard hook for deformed bars in tension includes the specific inside bend diameter and straight extension length. It shall be permitted to use a longer straight extension at the end of a hook. A longer extension shall not be considered to increase the anchorage capacity of the hook.

25.3.2 Minimum inside bend diameters for bars used as transverse reinforcement and standard hooks for bars used to anchor stirrups, ties, hoops, and spirals shall conform to Table 25.3.2. Standard hooks shall enclose longitudinal reinforcement.

R25.3.2 Standard stirrup, tie, and hoop hooks are limited to No. 8 bars and smaller, and the 90-degree hook with $6d_b$ extension is further limited to No. 5 bars and smaller, as the result of research showing that larger bar sizes with 90-degree hooks and $6d_b$ extensions tend to spall off the cover concrete when the reinforcement is stressed and the hook straightens.

The minimum $4d_b$ bend for the bar sizes commonly used for stirrups, ties, and hoops is based on accepted industry practice in the United States. Use of a stirrup bar size No. 5

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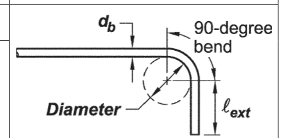
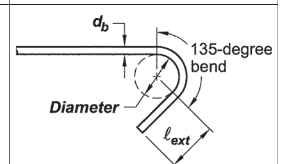
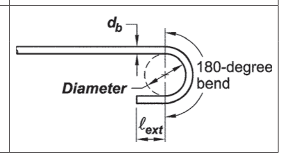
or smaller for the 90-, 135-, or 180-degree standard stirrup hook will permit multiple bending on standard stirrup bending equipment.

Constructibility issues should be considered in selecting anchorage details. In particular, the use of 180-degree hooks should be avoided in closed stirrups, ties, and hoops made of continuous reinforcement.

Table 25.3.2—Minimum inside bend diameters and standard hook geometry for stirrups, ties, and hoops

Type of standard hook	Bar size	Minimum inside bend diameter, in.	Straight extension[1] ℓ_{ext}, in.	Type of standard hook
90-degree hook	No. 3 through No. 5	$4d_b$	Greater of $6d_b$ and 3 in.	
	No. 6 through No. 8	$6d_b$	$12d_b$	
135-degree hook	No. 3 through No. 5	$4d_b$	Greater of $6d_b$ and 3 in.	
	No. 6 through No. 8	$6d_b$		
180-degree hook	No. 3 through No. 5	$4d_b$	Greater of $4d_b$ and 2.5 in.	
	No. 6 through No. 8	$6d_b$		

[1]A standard hook for stirrups, ties, and hoops includes the specific inside bend diameter and straight extension length. It shall be permitted to use a longer straight extension at the end of a hook. A longer extension shall not be considered to increase the anchorage capacity of the hook.

25.3.3 Minimum inside bend diameters for welded wire reinforcement used as stirrups or ties shall not be less than $4d_b$ for deformed wire larger than D6 and $2d_b$ for all other wires. Bends with inside diameter of less than $8d_b$ shall not be less than $4d_b$ from nearest welded intersection.

R25.3.3 Welded wire reinforcement can be used for stirrups and ties. The wire at welded intersections does not have the same uniform ductility and bendability as in areas that were not heated by welding in the manufacture of the welded wire reinforcement. These effects of the welding temperature are usually dissipated in a distance of approximately four wire diameters. Minimum bend diameters permitted are in most cases the same as those required in the ASTM bend tests for wire (ASTM A1064 and A1022).

25.3.4 Seismic hooks used to anchor stirrups, ties, hoops, and crossties shall be in accordance with (a) and (b):

(a) Minimum bend of 90 degrees for circular hoops and 135 degrees for all other hoops
(b) Hook shall engage longitudinal reinforcement and the extension shall project into the interior of the stirrup or hoop

25.3.5 Crossties shall be in accordance with (a) through (e):

(a) Crosstie shall be continuous between ends
(b) There shall be a seismic hook at one end
(c) There shall be a standard hook at other end with minimum bend of 90 degrees
(d) Hooks shall engage peripheral longitudinal bars

R25.3.5 Crossties are illustrated in Fig. R25.3.5.

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(e) 90-degree hooks of two successive crossties engaging the same longitudinal bars shall be alternated end for end, unless crossties satisfy 18.6.4.3 or 25.7.1.6.1

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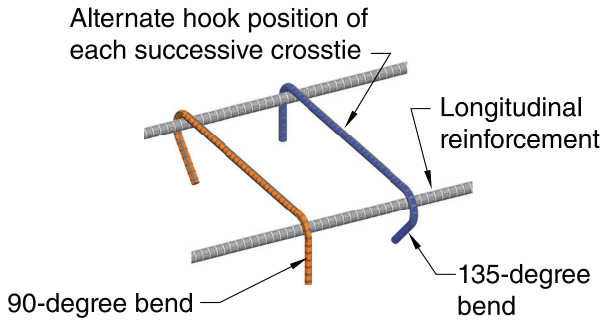

Fig. R25.3.5—Crosstie.

25.4—Development and anchorage of reinforcement

25.4.1 *General*

25.4.1.1 Calculated tension or compression in reinforcement at each section of a member shall be developed on each side of that section by embedment length, hook, headed deformed bar, mechanical device, or a combination thereof.

25.4.1.2 Hooks and heads shall not be used to develop bars in compression.

25.4.1.3 Development lengths do not require a strength reduction factor ϕ. The design strength of reinforcing bar groups associated with breakout failure requires a ϕ-factor in accordance with Table 21.2.1 (j).

R25.4—Development and anchorage of reinforcement

R25.4.1 *General*

R25.4.1.1 The development length concept is based on the attainable average bond stress over the length of embedment of the reinforcement (ACI Committee 408 1966). Development lengths are required because of the tendency of highly stressed bars to split relatively thin sections of restraining concrete. A single bar embedded in a mass of concrete should not require as great a development length, although a row of bars, even in mass concrete, can create a weakened plane with longitudinal splitting along the plane of the bars.

In application, the development length concept requires minimum lengths or extensions of reinforcement beyond all points of peak stress in the reinforcement. Such peak stresses generally occur at the points of maximum stress and points where reinforcement is bent or terminated. From a point of peak stress in reinforcement, some length of reinforcement or anchorage is necessary to develop the stress. This development length or anchorage is necessary on both sides of such peak stress points. Often, the reinforcement continues for a considerable distance on one side of a critical stress point so that calculations need involve only the other side, for example, the negative moment reinforcement continuing through a support to the middle of the next span. The requirement for a minimum value of K_{tr} along development and splice lengths in 9.7.1.4, 10.7.1.3, 25.4.2.2, and 25.5.1.5 improves ductility.

R25.4.1.2 Hooks and heads are ineffective in compression. No data are available to demonstrate that hooks and heads can reduce development length in compression.

R25.4.1.3 The strength reduction factor ϕ is not used in the development length and lap splice length equations. An allowance for strength reduction is already included in the expressions for determining development and splice lengths. The strength reduction factor ϕ for anchorage of reinforcing bar groups where the strength is controlled by concrete breakout failure modes is aligned with the reliability required for anchors in concrete.

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25.4.1.4 The values of $\sqrt{f_c'}$ used to calculate development length shall not exceed 100 psi.

25.4.1.5 For reinforcing bar groups in tension terminating in an anchorage region, the anchorage requirements of 25.4.11 shall be satisfied in addition to development length requirements.

25.4.1.6 Post-installed reinforcing bars designed in accordance with this section shall be qualified in accordance with ACI CODE-355.5.

25.4.2 *Development of deformed bars and deformed wires in tension*

25.4.2.1 Development length ℓ_d for deformed bars and deformed wires in tension shall be the greater of (a) and (b):

(a) Length calculated in accordance with 25.4.2.3 or 25.4.2.4 using the applicable modification factors of 25.4.2.5
(b) 12 in.

25.4.2.2 For bars with $f_y \geq 80{,}000$ psi spaced closer than 6 in. on center, transverse reinforcement shall be provided such that K_{tr} shall not be smaller than $0.5d_b$.

25.4.2.3 For deformed bars or deformed wires, ℓ_d shall be calculated in accordance with Table 25.4.2.3.

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R25.4.1.4 Darwin et al. (1996) shows that the force developed in a bar in development and lap splice tests increases at a lesser rate than $\sqrt{f_c'}$ with increasing compressive strength. Using $\sqrt{f_c'}$, however, is sufficiently accurate for values of $\sqrt{f_c'}$ up to 100 psi, and because of the long-standing use of the $\sqrt{f_c'}$ in design, ACI Committee 318 has chosen not to change the exponent applied to the compressive strength used to calculate development and lap splice lengths, but rather to set an upper limit of 100 psi on $\sqrt{f_c'}$.

R25.4.1.5 The strength of reinforcing bar groups may be limited by concrete breakout failure (Chicchi 2020; Kim and Chun 2022).

R25.4.1.6 Post-installed deformed reinforcing bars qualified in accordance with ACI CODE-355.5 are expected to exhibit a performance equivalent to cast-in bars of equal diameter and embedment for bond development and response to tension loading. ACI CODE-355.5 addresses critical aspects associated with the use of pourable or injectable bonding materials intended for use in post-installed reinforcing bar applications, including bond strength, stiffness, long-term stability, and suitability for use with the full range of bar diameters and associated embedments permitted by the Code.

R25.4.2 *Development of deformed bars and deformed wires in tension*

R25.4.2.1 This provision gives a two-tier approach for the calculation of tension development length. The user can either use the simplified provisions of 25.4.2.3 or the general development length equation (Eq. (25.4.2.4a)). In Table 25.4.2.3, ℓ_d is based on two preselected values of $(c_b + K_{tr})/d_b$, whereas ℓ_d from Eq. (25.4.2.4a) is based on the actual $(c_b + K_{tr})/d_b$.

Although there is no requirement for transverse reinforcement along the tension development or lap splice length, research (Azizinamini et al. 1999a,b) indicates that in concrete with very high compressive strength, brittle anchorage failure may occur for bars with inadequate transverse reinforcement. In lap splice tests of No. 8 and No. 11 bars in concrete with an f_c' of approximately 15,000 psi, transverse reinforcement improved ductile anchorage behavior.

R25.4.2.3 This provision recognizes that many current practical construction cases use spacing and cover values along with confining reinforcement, such as stirrups or ties, that result in a value of $(c_b + K_{tr})/d_b$ of at least 1.5. Examples include a minimum clear cover of d_b along with either minimum clear spacing of 2 d_b, or a combination of minimum clear spacing of d_b and minimum ties or stirrups.

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Table 25.4.2.3—Development length for deformed bars and deformed wires in tension

Spacing and cover	No. 6 and smaller bars and deformed wires	No. 7 and larger bars
Clear spacing of bars or wires being developed or lap spliced not less than d_b, clear cover at least d_b, and stirrups or ties throughout ℓ_d not less than the Code minimum or Clear spacing of bars or wires being developed or lap spliced at least $2d_b$ and clear cover at least d_b	$\left(\frac{f_y\psi_t\psi_e\psi_g}{25\lambda\sqrt{f_c'}}\right)d_b$	$\left(\frac{f_y\psi_t\psi_e\psi_g}{20\lambda\sqrt{f_c'}}\right)d_b$
Other cases	$\left(\frac{3f_y\psi_t\psi_e\psi_g}{50\lambda\sqrt{f_c'}}\right)d_b$	$\left(\frac{3f_y\psi_t\psi_e\psi_g}{40\lambda\sqrt{f_c'}}\right)d_b$

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For these frequently occurring cases, the development length for larger bars can be taken as $\ell_d = [f_y\psi_t\psi_e\psi_g/(20\lambda\sqrt{f_c'})]d_b$. In the formulation of the provisions in ACI 318-95, a comparison with past provisions and a check of a database of experimental results indicated that for No. 6 deformed bars and smaller, as well as for deformed wire, the development lengths could be reduced 20 percent using $\psi_s = 0.8$. This is the basis for the *No. 6 and smaller bars and deformed wires* column of Table 25.4.2.3. With less cover and in the absence of minimum ties or stirrups, the minimum clear spacing limits of 25.2.1 and the minimum concrete cover requirements of 20.5.1.3 result in minimum values of c_b equal to d_b. Thus, for "other cases," the values are based on using $(c_b + K_{tr})/d_b = 1.0$ in Eq. (25.4.2.4a).

The user may easily construct simple, useful expressions. For example, in all members with normalweight concrete ($\lambda = 1.0$), uncoated reinforcement ($\psi_e = 1.0$), No. 7 and larger bottom bars ($\psi_t = 1.0$) with $f_c' = 4000$ psi, and Grade 60 reinforcement ($\psi_g = 1.0$), the expressions reduce to

$$\ell_d = \frac{(60{,}000)(1.0)(1.0)(1.0)}{20(1.0)\sqrt{4000}}d_b = 47d_b$$

or

$$\ell_d = \frac{3(60{,}000)(1.0)(1.0)(1.0)}{40(1.0)\sqrt{4000}}d_b = 71d_b$$

Thus, as long as minimum cover of d_b is provided along with a minimum clear spacing of $2d_b$, or a minimum clear cover of d_b and a minimum clear spacing of d_b are provided along with minimum ties or stirrups, then $\ell_d = 47d_b$. The penalty for spacing bars closer or providing less cover is the requirement that $\ell_d = 71d_b$.

25.4.2.4 For deformed bars or deformed wires, ℓ_d shall be calculated by:

$$\ell_d = \left(\frac{3}{40}\frac{f_y}{\lambda\sqrt{f_c'}}\frac{\psi_t\psi_e\psi_s\psi_g}{\left(\frac{c_b+K_{tr}}{d_b}\right)}\right)d_b \qquad (25.4.2.4a)$$

in which the confinement term $(c_b + K_{tr})/d_b$ shall not exceed 2.5, and

$$K_{tr} = \frac{40A_{tr}}{sn} \qquad (25.4.2.4b)$$

where n is the number of bars or wires being developed or lap spliced along the plane of splitting. It shall be permitted to use $K_{tr} = 0$ as a design simplification even if transverse reinforcement is present or required.

R25.4.2.4 Equation (25.4.2.4a) includes the effects of all variables controlling the development length. In Eq. (25.4.2.4a), c_b is a factor that represents the least of the side cover, the concrete cover to the bar or wire (in both cases measured to the center of the bar or wire), or one-half the center-to-center spacing of the bars or wires. K_{tr} is a factor that represents the contribution of confining reinforcement across potential splitting planes. ψ_t is the reinforcement location factor to reflect the effect of the casting position (that is, formerly denoted as "top bar effect"). ψ_e is a coating factor reflecting the effects of epoxy coating. There is a limit on the product $\psi_t\psi_e$. The reinforcement size factor ψ_s reflects the more favorable performance of smaller-diameter reinforcement. ψ_g is the reinforcement grade factor accounting for the yield strength of the reinforcement. A limit of 2.5 is placed on the term $(c_b + K_{tr})/d_b$. When $(c_b + K_{tr})/d_b$ is less than 2.5, splitting failures are likely to occur. For values above 2.5, a pullout failure is expected, and an increase in cover or transverse reinforcement is unlikely to increase the anchorage capacity.

Many practical combinations of side cover, clear cover, and confining reinforcement can be used with 25.4.2.4 to produce significantly shorter development lengths than allowed by

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25.4.2.5 For the calculation of ℓ_d, modification factors shall be in accordance with Table 25.4.2.5.

Table 25.4.2.5—Modification factors for development of deformed bars and deformed wires in tension

Modification factor	Condition	Value of factor
Lightweight λ	Lightweight concrete	0.75
	Normalweight concrete	1.0
Reinforcement grade ψ_g	Grade 40 or Grade 60	1.0
	Grade 80	1.15
	Grade 100	1.3
Epoxy[1] ψ_e	Epoxy-coated or zinc and epoxy dual-coated reinforcement with clear cover less than $3d_b$ or clear spacing less than $6d_b$	1.5
	Epoxy-coated or zinc and epoxy dual-coated reinforcement for all other conditions	1.2
	Uncoated or zinc-coated (galvanized) reinforcement	1.0
Size ψ_s	No. 7 and larger bars	1.0
	No. 6 and smaller bars and deformed wires	0.8
Casting position[1] ψ_t	More than 12 in. of fresh concrete placed below horizontal reinforcement	1.3
	Other	1.0

[1]The product $\psi_t\psi_e$ need not exceed 1.7.

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25.4.2.3. For example, bars or wires with minimum clear cover not less than $2d_b$ and minimum clear spacing not less than $4d_b$ and without any confining reinforcement would have a $(c_b + K_{tr})/d_b$ value of 2.5 and would require a development length of only $28d_b$ for the example in R25.4.2.3.

Before ACI 318-08, Eq. (25.4.2.4b) for K_{tr} included the yield strength of transverse reinforcement. The current expression includes only the area and spacing of the transverse reinforcement and the number of wires or bars being developed or lap spliced because tests demonstrate that transverse reinforcement rarely yields during a bond failure (Azizinamini et al. 1995).

Terms in Eq. (25.4.2.4a) may be disregarded if such omission results in longer and, hence, more conservative, development lengths.

R25.4.2.5 The lightweight factor λ for calculating development length of deformed bars and deformed wire in tension is the same for all types of lightweight concrete. Research does not support the variations of this factor in Codes prior to 1989 for all-lightweight and sand-lightweight concrete (ACI PRC-408).

The reinforcement grade factor ψ_g accounts for the effect of reinforcement yield strength on required development length. Research has shown that required development length increases disproportionately with increases in yield strength (Orangun et al. 1977; Canbay and Frosch 2005).

The epoxy factor ψ_e is based on studies (Treece and Jirsa 1989; Johnston and Zia 1982; Mathey and Clifton 1976) of the anchorage of epoxy-coated bars that show bond strength is reduced because the coating prevents adhesion and lowers the coefficient of friction between the bar and the concrete. The factors reflect the type of anchorage failure likely to occur. If the cover or spacing is small, a splitting failure can occur and the anchorage or bond strength is substantially reduced. If the cover and spacing between bars is large, a splitting failure is precluded and the effect of the epoxy coating on anchorage strength is not as large. Studies (Orangun et al. 1977) have shown that although the cover or spacing may be small, the anchorage strength may be increased by adding transverse reinforcement crossing the plane of splitting, and restraining the splitting crack.

Because the bond of epoxy-coated bars or zinc and epoxy dual-coated bars is already reduced due to the loss of adhesion and lower coefficient of friction between the bar and the concrete, an upper limit of 1.7 is established for the product of the factors for top reinforcement casting position and epoxy-coated reinforcement or zinc and epoxy dual-coated reinforcement.

The reinforcement size factor ψ_s reflects the more favorable performance of smaller-diameter reinforcement.

The reinforcement location or casting position factor ψ_t accounts for the position of the reinforcement in freshly placed concrete. The factor 1.3 is based on research (Jirsa and Breen 1981; Jeanty et al. 1988). The application of the

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casting position factor should be considered in determination of development lengths for inclined reinforcement.

25.4.2.6 For post-installed reinforcing bars, (a) and (b) shall be permitted in the calculation of ℓ_d.

(a) casting position factor, $\psi_t = 1.0$
(b) include transverse reinforcement, crossing the potential plane of splitting, in the calculation of K_{tr}

25.4.2.7 Transverse reinforcement used to confine a straight bar being developed or lap spliced shall be developed on each side of potential splitting planes or shall consist of stirrups satisfying 25.7.1, ties satisfying 25.7.2, spirals satisfying 25.7.3, or hoops satisfying 25.7.4.

25.4.3 *Development of standard hooks in tension*

25.4.3.1 Development length ℓ_{dh} for deformed bars in tension terminating in a standard hook shall be the greater of (a) through (c):

(a) $\dfrac{f_y\psi_e\psi_s\psi_{cc}\psi_r}{50\lambda\sqrt{f_c'}}d_b$ with ψ_e, ψ_s, ψ_{cc}, ψ_r, and λ given in 25.4.3.2
(b) $8d_b$
(c) 6 in.

25.4.3.2 For the calculation of ℓ_{dh}, modification factors ψ_e, ψ_r, ψ_s, ψ_{cc}, and λ shall be in accordance with Table 25.4.3.2. Factors ψ_{cc} and ψ_r shall be permitted to be taken as 1.0. At discontinuous ends of members, 25.4.3.3 shall apply.

R25.4.3 *Development of standard hooks in tension*

R25.4.3.1 The provisions for hooked bars are only applicable to standard hooks (refer to 25.3.1). The development length ℓ_{dh} is measured from the critical section to the outside end (or edge) of the hook. Failure of hooked bar anchorages is typically due to splitting of the concrete cover in the plane of the hook or breakout failure. The required length to avoid these failures is influenced by reinforcement diameter, epoxy coating, and yield strength, as well as other factors that influence splitting and breakout strengths such as concrete strength, lightweight aggregate, cover dimensions, and confinement provided by transverse reinforcement enclosing the hooked reinforcement. The expression in 25.4.3.1(a) restores the equation for hooked bar development used in ACI 318-14 and several previous editions of the Code with minor modifications. Revisions to 25.4.3.1 introduced in ACI 318-19 were found to result in appreciably increased values of ℓ_{dh} for common cases, a result which the committee later found to be without adequate justification.

Minimum values of ℓ_{dh} are specified to prevent failure by direct pullout in cases where a hook may be located near the critical section. Hooks in beam-column joints and corbels should be placed as close as practical to the back face of the joint.

R25.4.3.2 Unlike straight bar development, no distinction is made for casting position.

The epoxy factor ψ_e is based on tests (Hamad et al. 1993) that indicate the development length for hooked bars should be increased by 20% to account for reduced bond when reinforcement is epoxy coated.

The reinforcement size factor ψ_s is based on test results (Banaeipour et al. 2023) that indicate that large bars may require increased development length compared with that calculated using the ACI 318-14 provisions. The confining reinforcement factor ψ_r is based on tests (Jirsa and Marques 1975) that indicate closely spaced ties at or near the bend portion of a hooked bar are most effective in confining the

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Table 25.4.3.2—Modification factors for development of hooked bars in tension

Modification factor	Condition	Value of factor
Lightweight λ	Lightweight concrete	0.75
	Normalweight concrete	1.0
Epoxy ψ_e	Epoxy-coated or zinc and epoxy dual-coated reinforcement	1.2
	Uncoated or zinc-coated (galvanized) reinforcement	1.0
Size ψ_s	No. 9 bars and smaller	1.0
	No. 10 and No. 11 bars	1.15
	No. 14 bars	1.3
	No. 18 bars	1.5
Cover ψ_{cc}	For No. 11 bar and smaller hooks with side cover (normal to plane of hook) $\geq$ 2-1/2 in. and for 90-degree hook with cover on bar extension beyond hook $\geq$ 2 in.	0.7
	Other	1.0
Confining reinforcement ψ_r[2]	For 90-degree hooks of No. 11 and smaller bars (1) enclosed along ℓ_{dh} within ties or stirrups[1] perpendicular to ℓ_{dh} at $s \leq 3d_b$, or (2) enclosed along the bar extension beyond hook including the bend within ties or stirrups[1] perpendicular to ℓ_{ext} at $s \leq 3d_b$	0.8
	For 180-degree hooks of No. 11 and smaller bars enclosed along ℓ_{dh} within ties or stirrups[1] perpendicular to ℓ_{dh} at $s \leq 3d_b$	
	Other	1.0

[1]The first tie or stirrup shall enclose the bent portion of the hook within $2d_b$ of the outside of the bend.

[2]d_b is nominal diameter of hooked bar.

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hooked bar. For construction purposes, this is not always practicable. The cases where the modification factor ψ_r may be used are illustrated in Fig. R25.4.3.2a and R25.4.3.2b. Figure R25.4.3.2a shows placement of ties or stirrups parallel to the bar being developed along the length of the tail extension of the hook plus bend. This configuration would be typical in a beam-column joint. R25.4.3.2b shows placement of ties or stirrups perpendicular to the bar being developed, spaced along the development length ℓ_{dh} of the hook.

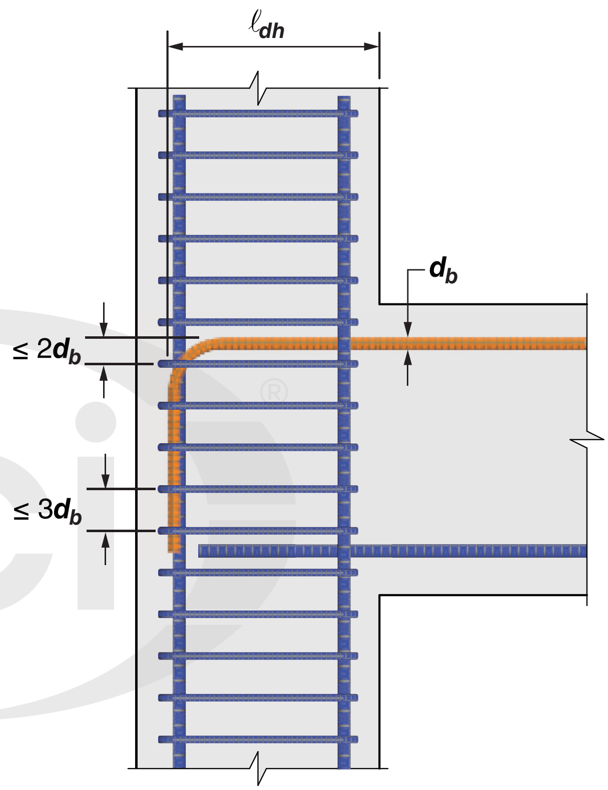

Fig. R25.4.3.2a—Ties or stirrups placed parallel to the bar being developed, spaced along the length of the tail extension of the hook plus bend.

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25.4.3.3 For bars being developed by a standard hook at discontinuous ends of members with both side cover and top (or bottom) cover to hook less than 2-1/2 in., (a) through (c) shall be satisfied:

(a) The hook shall be enclosed along ℓ_{dh} within ties or stirrups perpendicular to ℓ_{dh} at $s \leq 3d_b$.
(b) The first tie or stirrup shall enclose the bent portion of the hook within $2d_b$ of the outside of the bend.
(c) ψ_r shall be taken as 1.0 in calculating ℓ_{dh} in accordance with 25.4.3.1(a), where d_b is the nominal diameter of the hooked bar.

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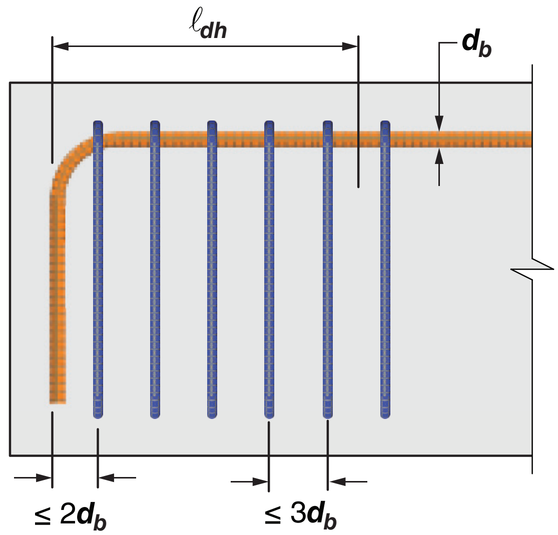

Fig. R25.4.3.2b—Ties or stirrups placed perpendicular to the bar being developed, spaced along the development length ℓ_{dh}.

R25.4.3.3 Bar hooks are especially susceptible to a concrete splitting failure if both side cover (perpendicular to plane of hook) and top or bottom cover (in plane of hook) are small (refer to Fig. R25.4.3.3). With minimum confinement provided by concrete, additional confinement provided by ties or stirrups is essential, especially if full bar strength is to be developed by a hooked bar with such small cover. Cases where hooks may require ties or stirrups for confinement are at ends of simply-supported beams, at the free end of cantilevers, and at ends of members framing into a joint where members do not extend beyond the joint. In contrast, if the calculated bar stress is so low that the hook is not needed for bar anchorage, ties or stirrups are not necessary. This provision does not apply for hooked bars at discontinuous ends of slabs where confinement is provided by the slab on both sides and perpendicular to the plane of the hook.

Section 25.7.1.8.4 prohibits reinforcement anchored with a head from being considered as contributing to confinement of hooked bars due to a lack of test data.

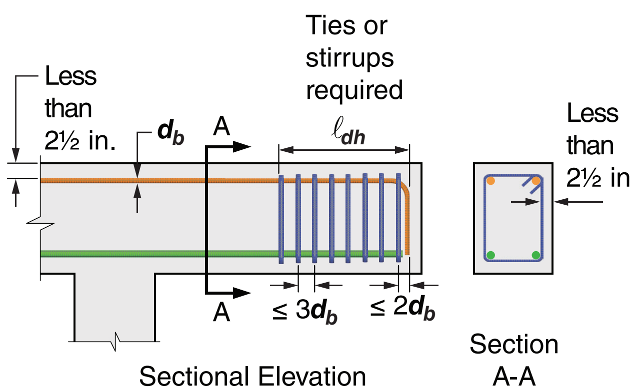

Fig. R25.4.3.3—Concrete cover according to 25.4.3.3.

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25.4.4 *Development of headed deformed bars in tension*

25.4.4.1 Use of a head to develop a deformed bar in tension shall be permitted if conditions (a) through (f) are satisfied:

(a) Bar shall conform to 20.2.1.6
(b) Bar size shall not exceed No. 11
(c) Net bearing area of head A_{brg} shall be at least $4A_b$ for Grade 60 bars, and at least $6A_b$ for Grade 80 and Grade 100 bars
(d) Concrete shall be normalweight
(e) Clear cover for bar shall be at least $2d_b$
(f) Center-to-center spacing between bars shall be at least $3d_b$

25.4.4.2 Development length ℓ_{dt} for headed deformed bars in tension shall be the longest of (a) through (c):

(a) $\left(\dfrac{f_y\psi_e\psi_p\psi_o\psi_c}{90\sqrt{f_c'}}\right)d_b^{1.5}$ with ψ_e, ψ_p, ψ_o, and ψ_c, given in 25.4.4.3
(b) $8d_b$
(c) 6 in.

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R25.4.4 *Development of headed deformed bars in tension*

R25.4.4.1 As used in this section, development describes cases in which the force in the bar is transferred to the concrete through a combination of a bearing force at the head and bond forces along the bar. In contrast, Chapter 17 anchorage provisions describe cases in which the force in the bar is transferred through bearing to the concrete at the head alone. Headed bars are limited to those types that meet the criteria in 20.2.1.6 for Class HA heads.

The provisions for headed deformed bars were formulated with due consideration of the provisions for anchorage in Chapter 17 (Shao et al. 2016). Chapter 17 contains provisions for headed anchors related to the individual failure modes of concrete breakout, side-face blowout, and pullout. These failure modes were considered in the formulation of 25.4.4.2. The restrictions to maximum bar size of No. 11 and normalweight concrete are based on a lack of data for larger bars or lightweight concrete (Thompson et al. 2005, 2006a,b; Shao et al. 2016). The upper limit of 60,000 psi on f_y that appeared prior to the 2019 Code has been removed.

For bars in tension, heads allow the bars to be developed in a shorter length than required for standard hooks, but otherwise perform in a similar manner (Thompson et al. 2005, 2006a,b; Shao et al. 2016). The head is considered to be part of the bar for the purposes of satisfying the specified cover requirements in 20.5.1.3 and aggregate size requirements of 26.4.2.1(a)(5).

Headed bars with $A_{brg} < 4A_b$ have been used in practice, but their performance is not accurately represented by the provisions in 25.4.4.2, and they should be used only with designs that are supported by test results under 25.4.5. These provisions do not address the design of studs or headed stud assemblies used for shear reinforcement.

R25.4.4.2 The provisions for developing headed deformed bars give the length of bar, ℓ_{dt}, measured from the critical section to the bearing face of the head, as shown in Fig. R25.4.4.2a. The provisions are primarily based on tests of simulated beam-column joints (Shao et al. 2016).

If longitudinal headed deformed bars from a beam, slab, or corbel terminate in a supporting member, such as the column shown in Fig. R25.4.4.2b, the bars should extend through the joint to the far face of the confined core of the supporting member, allowing for cover and avoidance of interference with column reinforcement, even though the resulting anchorage length may exceed ℓ_{dt}. Extending the bar to the far side of the column core helps engage the entire joint in resisting the anchorage forces and thereby improves the performance of the joint.

If closely spaced headed bars are used, the potential for concrete breakout failure exists. For joints as shown in Fig. R25.4.4.2c and R25.4.4.2d, anchorage strengths will be generally higher if the anchorage length is equal to or greater than $d/1.5$ (Eligehausen 2006b), as shown in Fig. R25.4.4.2c,

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or by providing reinforcement in the form of hoops and ties to establish a load path in accordance with strut-and-tie modeling principles, as shown in Fig. R25.4.4.2d. Strut-and-tie models should be verified in accordance with Chapter 23. Note that the strut-and-tie models illustrated in Fig. R25.4.4.2c and R25.4.4.2d rely on a vertical strut from a column extending above the joint. Beam-column joints at roof-level and portal frames are vulnerable to joint failure and should be properly detailed to restrain diagonal cracking through the joint and breakout of the bars through the top surface.

For cases where development length cannot be designed in accordance with 25.4.4.2, use of the provisions of Chapter 17 should be considered.

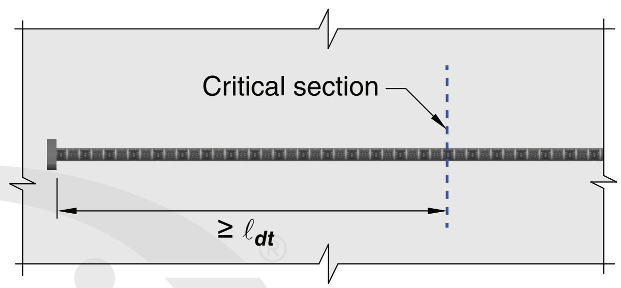

Fig. R25.4.4.2a—Development of headed deformed bars.

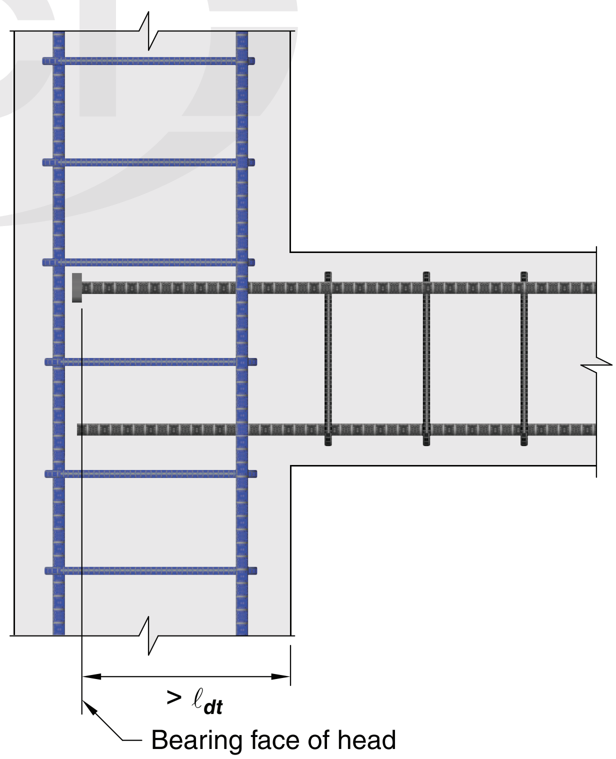

Fig. R25.4.4.2b—Headed deformed bar extended to far side of column core with anchorage length that exceeds ℓ_{dt}.

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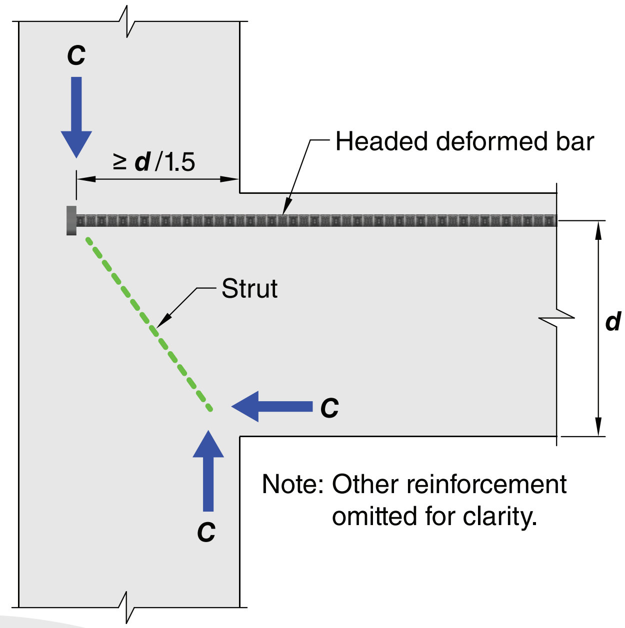

Fig. R25.4.4.2c—Breakout failure precluded in joint by keeping anchorage length greater than or equal to d*/1.5.*

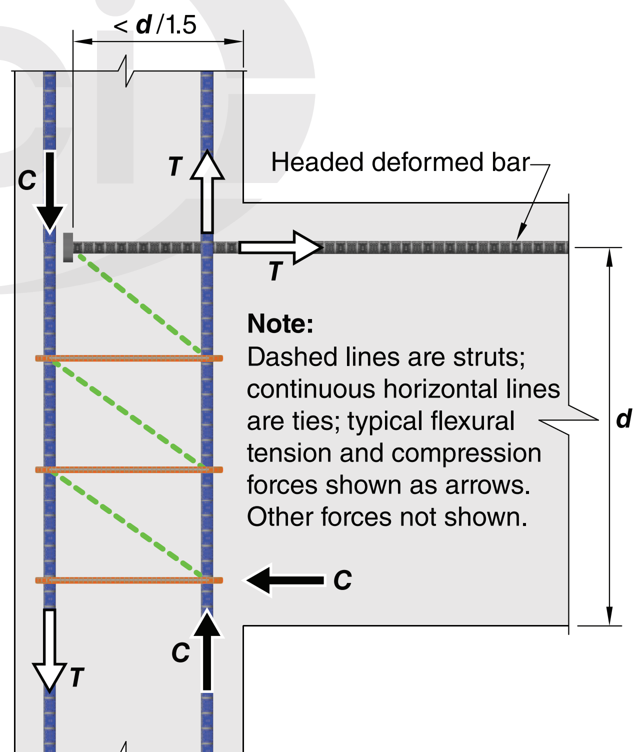

Fig. R25.4.4.2d—Breakout failure precluded in joint by providing transverse reinforcement to enable a strut-and-tie mechanism.

25.4.4.3 For the calculation of ℓ_{dt}, modification factors ψ_e, ψ_p, ψ_o, and ψ_c shall be in accordance with Table 25.4.4.3.

R25.4.4.3 The epoxy factor 1.2 is based conservatively on the value used for epoxy-coated standard hooks. The location factor ψ_o accounts for the confinement provided by the reinforcement within columns and large side cover for other members.

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Table 25.4.4.3—Modification factors for development of headed bars in tension

Modification factor	Condition	Value of factor
Epoxy ψ_e	Epoxy-coated or zinc and epoxy dual-coated reinforcement	1.2
	Uncoated or zinc-coated (galvanized) reinforcement	1.0
Parallel tie reinforcement ψ_p[2,3]	s[1] $\geq 8d_b$ or $A_{tt} \geq 0.30A_{hs}$	1.0
	Other	$2 - \frac{s}{8d_b} \leq 1.6$
Location ψ_o	For headed bars: (1) Terminating inside column core with side cover to bar $\geq$ 2.5 in.; or (2) With side cover to bar $\geq 6d_b$	1.0
	Other	1.25
Concrete strength ψ_c	For $f_c' < 6000$ psi	$f_c'/15{,}000 + 0.6$
	For $f_c' \geq 6000$ psi	1.0

[1]s is minimum center-to-center spacing of headed bars.
[2]d_b is nominal diameter of headed bar.
[3]Refer to 25.4.4.5.

25.4.4.4 Parallel tie reinforcement confining headed bars in beam-column joints used in the calculation of A_{tt} shall be in accordance with 25.4.4.4.1 through 25.4.4.4.3.

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The factor ψ_p for headed reinforcement is similar to the confining reinforcement factor for hooked bars (Shao et al. 2016). Parallel ties are more effective for closely-spaced headed bars, and the effects of increasing the area of parallel ties and increasing the spacing of headed bars are not directly additive. Thus, spacing and confinement effects are treated separately for simplicity in design.Test results indicate that only tie or hoop reinforcement parallel to headed bars contributes to anchorage strength and reduces development length (Thompson et al. 2005, 2006a,b).

R25.4.4.4 Reinforcement oriented parallel to the development length of the headed bars, located within the region defined in 25.4.4.4.2 (Fig. R25.4.4.4) contributes to anchorage strength in proportion to its area (Ghimire 2019a,b). This reinforcement serves to tie concrete near the head to concrete on the other side of the failure surface, thus mobilizing additional anchorage strength. With the exception of vertical joint reinforcement in the form of stirrups that are well anchored to the far side of the joint, reinforcement oriented perpendicular to the development length has been shown in a number of cases to be ineffective in improving the anchorage of headed deformed bars (Thompson et al. 2005, 2006a,b). Both legs of individual stirrups and ties parallel to the headed bars contribute to A_{tt}. Section 25.7.1.8.4 prohibits reinforcement anchored with a head from contributing to A_{tt} due to a lack of test data.

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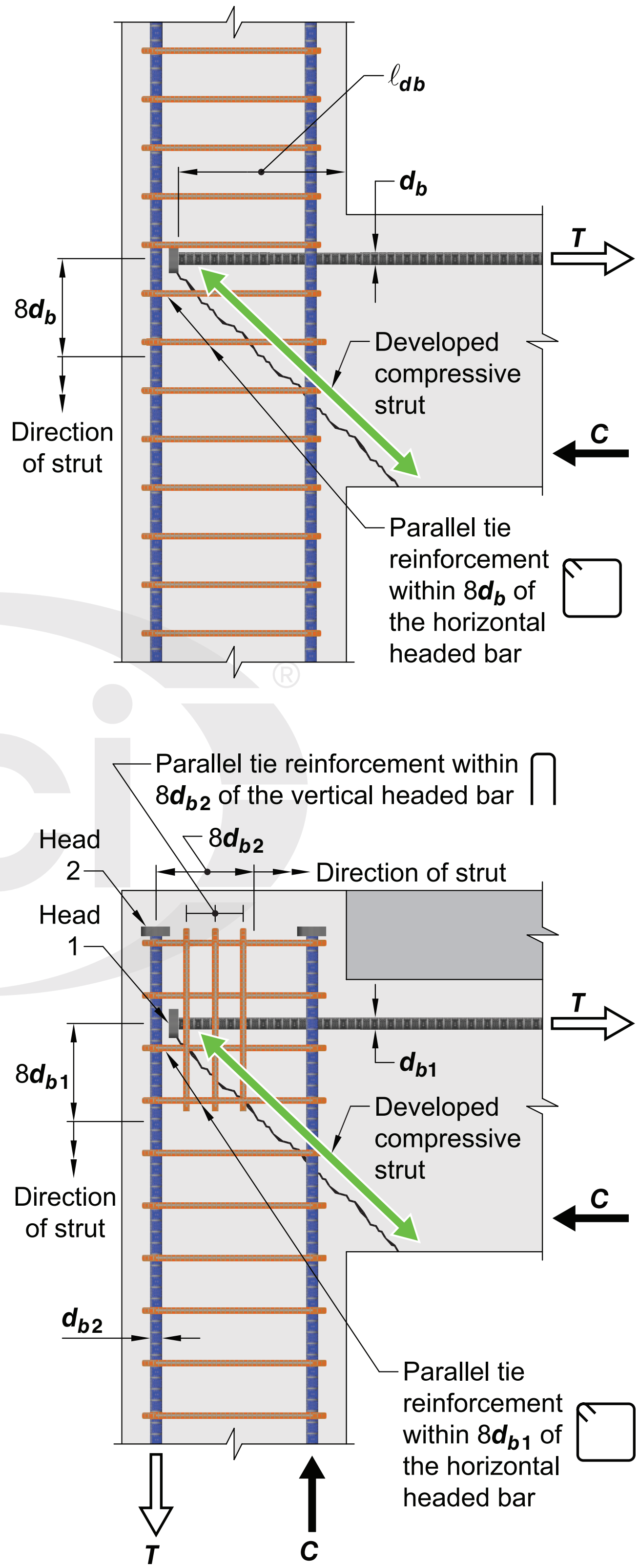

Fig. R25.4.4.4—Ties or stirrups placed parallel to the headed beam bars being developed in a beam-column joint that contribute to anchorage strength.

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25.4.4.4.1 Parallel tie reinforcement shall be oriented parallel to ℓ_{dt} and consist of closed stirrups, ties, or hoops.

25.4.4.4.2 Parallel tie reinforcement shall be located within $8d_b$ of the centerline of the headed bar toward the middle of the joint, where d_b is the nominal diameter of the headed bar.

25.4.4.4.3 The area of parallel tie reinforcement, A_{tt}, shall be calculated as the cross-sectional area of all legs of closed stirrups, ties, or hoops comprising the parallel tie reinforcement.

25.4.4.5 For anchorages other than in beam-column joints, tie reinforcement, A_{tt}, shall not be considered.

25.4.4.6 If beam negative moment reinforcement is provided by headed deformed bars that terminate in a joint, the column shall extend above the top of the joint a distance at least the depth h of the joint, where h is the horizontal dimension of the joint in the direction of the forces being considered. Alternatively, the beam reinforcement shall be enclosed by additional vertical joint reinforcement providing equivalent confinement to the top face of the joint.

25.4.5 *Development of mechanically anchored deformed bars in tension*

25.4.5.1 Any mechanical attachment or device capable of developing f_y of deformed bars shall be permitted, provided it is approved by the building official in accordance with 1.10. Development of deformed bars shall be permitted to consist of a combination of mechanical anchorage plus additional embedment length of the deformed bars between the critical section and the mechanical attachment or device.

25.4.6 *Development of welded deformed wire reinforcement in tension*

25.4.6.1 Development length ℓ_d for welded deformed wire reinforcement in tension measured from the critical section to the end of wire shall be the greater of (a) and (b), where wires in the direction of the development length shall all be deformed D31 or smaller.

(a) Length calculated in accordance with 25.4.6.2
(b) 8 in.

25.4.6.2 For welded deformed wire reinforcement, ℓ_d shall be calculated from 25.4.2.3 or 25.4.2.4, times welded deformed wire reinforcement factor ψ_w from 25.4.6.3 or 25.4.6.4. For epoxy-coated welded deformed wire reinforce-

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R25.4.4.5 No evidence is available regarding the effect of parallel reinforcement on the development length of headed bars except in beam-column joints.

R25.4.4.6 This provision refers to a corner joint in which beam reinforcement terminates with headed deformed bars. Such joints require confinement of the headed beam bars along the top face of the joint. This confinement can be provided by either: a) a column that extends above the top of the joint; or b) vertical reinforcement hooked around the beam top reinforcing bars and extending downward into the joint in addition to the column longitudinal reinforcement. Detailing guidance and design recommendations for vertical joint reinforcement may be found in ACI PRC-352.

R25.4.5 *Development of mechanically anchored deformed bars in tension*

R25.4.5.1 Anchorage of deformed bars through the use of mechanical devices within concrete that do not meet the requirements in 20.2.1.6, or are not developed in accordance with 25.4.4, may be used if tests demonstrate the ability of the head and bar system to develop or anchor the desired force in the bar, as described in this provision.

R25.4.6 *Development of welded deformed wire reinforcement in tension*

R25.4.6.1 ASTM A1064 for welded deformed wire reinforcement requires the same strength of the weld as required for welded plain wire reinforcement. Some of the development is assigned to welds and some assigned to the length of deformed wire.

R25.4.6.2 The welded deformed wire reinforcement factor ψ_w is applied to the deformed wire development length calculated from 25.4.2.3 or 25.4.2.4.

Tests (Bartoletti and Jirsa 1995) have indicated that epoxy-coated welded deformed wire reinforcement has essentially

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ment meeting 25.4.6.3, it shall be permitted to use $\psi_e = 1.0$ in 25.4.2.3 or 25.4.2.4.

25.4.6.3 For welded deformed wire reinforcement with at least one cross wire within ℓ_d that is at least 2 in. from the critical section, ψ_w shall be the greater of (a) and (b), and need not exceed 1.0:

(a) $\left(\dfrac{f_y - 35{,}000}{f_y}\right)$

(b) $\left(\dfrac{5d_b}{s}\right)$

where s is the spacing between the wires to be developed.

25.4.6.4 For welded deformed wire reinforcement with no cross wires within ℓ_d or with a single cross wire less than 2 in. from the critical section, ψ_w shall be taken as 1.0.

25.4.6.5 Where any plain wires, or deformed wires larger than D31, are present in the welded deformed wire reinforcement in the direction of the development length, the reinforcement shall be developed in accordance with 25.4.7.

25.4.6.6 Zinc-coated (galvanized) welded deformed wire reinforcement shall be developed in accordance with 25.4.7.

25.4.7 *Development of welded plain wire reinforcement in tension*

25.4.7.1 Development length ℓ_d for welded plain wire reinforcement in tension measured from the critical section to the outermost cross wire shall be the greater of (a) and (b) and shall require a minimum of two cross wires within ℓ_d.

(a) Length calculated in accordance with 25.4.7.2
(b) 6 in.

25.4.7.2 ℓ_d shall be the greater of (a) and (b):

(a) spacing of cross wires + 2 in.

(b) $0.27\left(\dfrac{f_y}{\lambda\sqrt{f_c'}}\right)\left(\dfrac{A_b}{s}\right)$, where s is the spacing between the wires to be developed, and λ is given in Table 25.4.2.5.

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the same development and splice strengths as uncoated welded deformed wire reinforcement because the cross wires provide the primary anchorage for the wire. Therefore, ψ_e of 1.0 is used for development and splice lengths of epoxy-coated welded deformed wire reinforcement with cross wires within the splice or development length.

R25.4.6.3 Figure R25.4.6.3 shows the development requirements for welded deformed wire reinforcement with one cross wire within the development length.

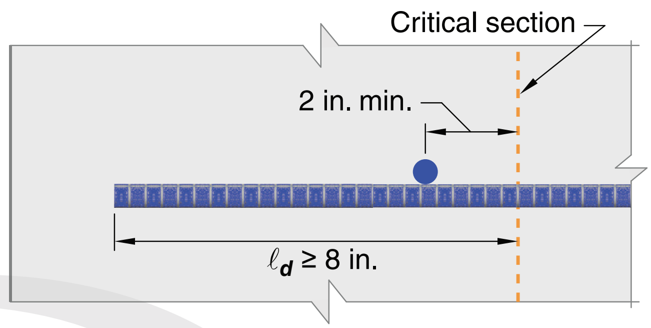

Fig. R25.4.6.3—Development of welded deformed wire reinforcement.

R25.4.6.5 Deformed wire larger than D31 is treated as plain wire because tests show that D45 wire will achieve only approximately 60% of the bond strength in tension given by Eq. (25.4.2.4a) (Rutledge and DeVries 2002).

R25.4.7 *Development of welded plain wire reinforcement in tension*

R25.4.7.1 ASTM A1064 for welded plain wire reinforcement requires the same strength of the weld as required for welded deformed wire reinforcement. All of the development is assigned to the welded cross wires; consequently, welded plain wire reinforcement requires at least two cross wires.

R25.4.7.2 Figure R25.4.7.2 shows the development requirements for welded plain wire reinforcement with development primarily dependent on the location of cross wires.

For welded plain wire reinforcement made with small wires, an embedment of at least two cross wires 2 in. or more beyond the point of critical section is adequate to develop the yield strength of the anchored wires. However, for welded

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plain wire reinforcement made with larger closely spaced wires, a longer embedment is required with the development length controlled by 25.4.7.2(b).

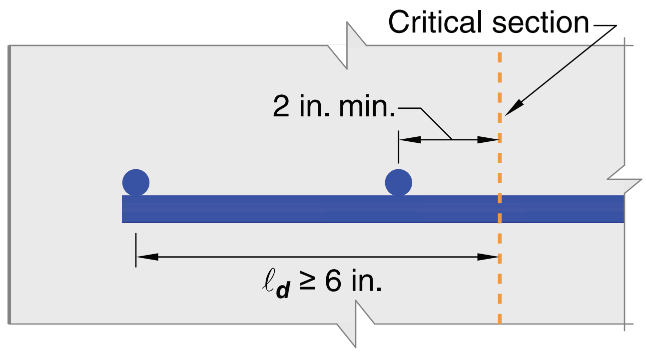

Fig. R25.4.7.2—Development of welded plain wire reinforcement.

25.4.8 *Development of pretensioned seven-wire strands in tension*

R25.4.8 *Development of pretensioned seven-wire strands in tension*

Development requirements for pretensioned strand are intended to provide bond integrity for the strength of the member. Provisions are based on tests performed on normalweight concrete members with a minimum cover of 2 in. These tests may not represent the behavior of strand in no-slump concrete. Concrete placement operations should ensure consolidation of concrete around the strand with complete contact between the steel and concrete.

The bond of strand is a function of a number of factors, including the configuration and surface condition of the steel, the stress in the steel, the depth of concrete beneath the strand, and the method used to transfer the force in the strand to the concrete. For bonded applications, quality assurance procedures should be used to confirm that the strand is capable of adequate bond (Rose and Russell 1997; Logan 1997). The precast concrete manufacturer may rely on certification from the strand manufacturer that the strand has bond characteristics that comply with this section.

This section does not apply to plain wires, to end-anchored tendons, or to unstressed strand. The development length for plain wire could be considerably greater due to the absence of mechanical interlock. Flexural bond failure would occur with plain wire when first slip occurred. Nontensioned prestressing steel is sometimes used as integrity reinforcement in precast concrete structures; however, there are limited data available regarding the bond length required to ensure development of the yield strength of the reinforcement (Salmons and McCrate 1977; PCA 1980).

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25.4.8.1 Development length ℓ_d of pretensioned seven-wire strands in tension shall be in accordance with (a) and (b):

(a) $$\ell_d = \left(\frac{f_{se}}{3000}\right)d_b + \left(\frac{f_{ps} - f_{se}}{1000}\right)d_b \qquad (25.4.8.1)$$

(b) If bonding of a strand does not extend to end of member, and design includes tension at service loads in the precompressed tension zone, ℓ_d calculated by Eq. (25.4.8.1) shall be doubled.

25.4.8.2 Seven-wire strand shall be bonded at least ℓ_d beyond the critical section except as provided in 25.4.8.3.

25.4.8.3 Embedment less than ℓ_d shall be permitted at a section of a member, provided the design strand stress at that section does not exceed values obtained from the bilinear relationship defined by Eq. (25.4.8.1).

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R25.4.8.1 The first term in Eq. (25.4.8.1) represents the transfer length of the strand, that is, the distance over which the strand should be bonded to the concrete to develop the effective prestress in the prestressed reinforcement, f_{se}. The second term represents the additional length over which the strand should be bonded so that the stress in the prestressed reinforcement at nominal strength of the member, f_{ps}, may develop.

Exploratory tests (Kaar and Magura 1965) that studied the effect of debonded strand (bond not permitted to extend to the ends of members) on performance of pretensioned girders indicated that the performance of these girders with embedment lengths twice those required by Eq. (25.4.8.1) closely matched the flexural performance of similar pretensioned girders with strand fully bonded to ends of girders. Accordingly, twice the development length is required for strand not bonded through to the end of a member. Subsequent tests (Rabbat et al. 1979) indicated that in pretensioned members designed for zero tension in the concrete under service load conditions (refer to 24.5.2), the development length for debonded strands need not be increased by a factor of 2. For analysis of sections with debonded strands at locations where strand is not fully developed, the procedure outlined in 21.2.3 is provided.

R25.4.8.3 Figure R25.4.8.3 shows the relationship between steel stress and the distance over which the strand is bonded to the concrete represented by Eq. (25.4.8.1). This idealized variation of strand stress may be used for analyzing sections within the development region (Martin and Korkosz 1995; PCI MNL 120). The expressions for transfer length and for the additional bonded length necessary to develop an increase in stress of ($f_{ps} - f_{se}$) are based on tests of members prestressed with clean, 1/4, 3/8, and 1/2 in. diameter strands for which the maximum value of f_{ps} was 275,000 psi (Kaar and Magura 1965; Hanson and Kaar 1959; Kaar et al. 1963).

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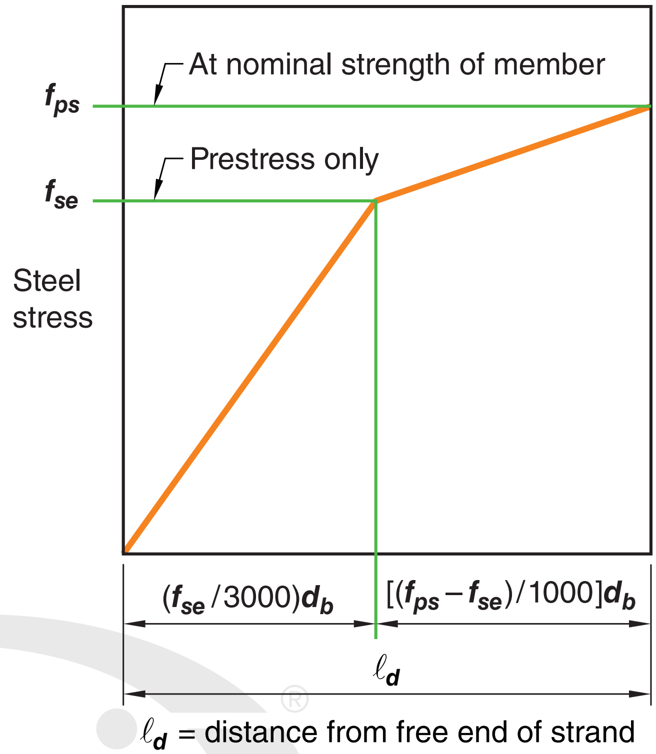

Fig. R25.4.8.3—Idealized bilinear relationship between steel stress and distance from the free end of strand.

25.4.9 *Development of deformed bars and deformed wires in compression*

25.4.9.1 Development length ℓ_{dc} for deformed bars and deformed wires in compression shall be the greater of (a) and (b)

(a) Length calculated in accordance with 25.4.9.2
(b) 8 in.

25.4.9.2 ℓ_{dc} shall be the greater of (a) and (b), using the modification factors of 25.4.9.3:

(a) $\left(\dfrac{f_y\psi_r}{50\lambda\sqrt{f_c'}}\right)d_b$

(b) $0.0003f_y\psi_r d_b$

25.4.9.3 For the calculation of ℓ_{dc}, modification factors shall be in accordance with Table 25.4.9.3, except ψ_r shall be permitted to be taken as 1.0.

R25.4.9 *Development of deformed bars and deformed wires in compression*

R25.4.9.1 The weakening effect of flexural tension cracks is not present for bars and wires in compression, and usually end bearing of the bars on the concrete is beneficial. Therefore, shorter development lengths are specified for compression than for tension.

R25.4.9.2 The constant 0.0003 has units of in.^{2}/lb.

The term λ is provided in the expression for development in 25.4.9.2 recognizing that there are no known test data on compression development in lightweight concrete but that splitting is more likely in lightweight concrete.

R25.4.9.3 The development length may be reduced 25 percent when the reinforcement is enclosed within closely spaced spirals, ties, or hoops.

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Table 25.4.9.3—Modification factors for deformed bars and wires in compression

Modification factor	Condition	Value of factor
Lightweight λ	Lightweight concrete	0.75
	Normalweight concrete	1.0
Confining reinforcement ψ_r	Reinforcement enclosed within (1), (2), (3), or (4): (1) a spiral (2) a circular continuously wound tie with $d_b \geq 1/4$ in. and pitch $\leq$ 4 in. (3) No. 4 bar or D20 wire ties in accordance with 25.7.2 spaced $\leq$ 4 in. on center (4) hoops in accordance with 25.7.4 spaced $\leq$ 4 in. on center	0.75
	Other	1.0

25.4.10 *Reduced embedment of straight reinforcement*

25.4.10.1 Except as provided in 25.4.10.2, embedment of straight reinforcement less than that prescribed for development shall be permitted if area of reinforcement provided, $A_{s,provided}$, exceeds area of reinforcement required, $A_{s,required}$. Embedment length beyond the critical section shall equal or exceed ℓ_d multiplied by $(A_{s,required})/(A_{s,provided})$, and shall not be less than the applicable minimum development lengths specified in 25.4.2.1(b), 25.4.6.1(b), 25.4.7.1(b), or 25.4.9.1(b).

25.4.10.2 Reduced embedment in accordance with 25.4.10.1 shall not be permitted for (a) through (f)

(a) At noncontinuous supports
(b) At locations where anchorage or development for f_y is required
(c) Where bars are required to be continuous
(d) For hooked, headed, and mechanically anchored deformed reinforcement
(e) In seismic-force-resisting systems in structures assigned to Seismic Design Categories C, D, E, or F
(f) Anchorage of concrete piles and concrete filled pipe piles to pile caps in structures assigned to Seismic Design Categories C, D, E, or F

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R25.4.10 *Reduced embedment of straight reinforcement*

R25.4.10.1 Reduced reinforcement embedment is permitted in limited circumstances.

R25.4.10.2 Embedment of straight bars less than that required for development should not be used if reinforcement is expected to permit redistribution of stress or if ductile behavior is required. Embedment of tension-loaded straight bars in closely spaced groups can result in concrete breakout failure (Chicchi 2020). Reducing bar embedment in such cases greatly increases the likelihood of such failures. Refer to 25.4.1.5. The excess reinforcement factor ($A_{s,required}$/$A_{s,provided}$), is not applicable for hooked or headed bars, where a reduction in development length with the application of the excess reinforcement factor could result in a potential concrete breakout failure.

Where a flexural member is part of the seismic-force-resisting-system, loads greater than those anticipated in design may cause reversal of moment at supports; some positive reinforcement should be developed into the support. This anchorage is required to ensure ductile response in the event of serious overstress, such as from earthquake or blast. It is not sufficient to use more reinforcement at lower stresses.

The reduction factor based on area is not to be used in those cases where anchorage development for f_y is required. For example, the excess reinforcement factor does not apply for development of shrinkage and temperature reinforcement according to 24.4.3.4 or for development of reinforcement provided according to 7.7.7, 8.7.4.2, 8.8.1.6, 9.7.7, and 9.8.1.6.

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25.4.11 *Anchorage of bar groups in tension*

25.4.11.1 The design breakout strength of a reinforcing bar group, ϕN_{rg}, shall be greater than or equal to the required strength of the reinforcing bar group. The nominal breakout strength is defined in 25.4.11.2 through 25.4.11.6.

25.4.11.1.1 Beam-column joints designed in accordance with (a), (b) or (c) are deemed to satisfy 25.4.11.

(a) Joints with hooked bar groups designed in accordance with Chapter 15
(b) Joints with headed bar groups designed in accordance with Chapter 15 and in which the provided embedment length is greater than or equal to the beam effective depth divided by 1.5
(c) Joints designed in accordance with 18.4.4 or 18.8

25.4.11.2 The nominal breakout strength of the bar group, N_{rg}, shall be calculated as

$$N_{rg} = N_{cbg} + N_{srg} \qquad (25.4.11.2)$$

25.4.11.2.1 Alternatively, it shall be permitted to calculate the nominal breakout strength of the bar group, N_{rg}, based on a design model that results in prediction of strength in substantial agreement with results of comprehensive tests.

25.4.11.3 ϕ shall be determined in accordance with Table 21.2.1(j) for anchorage of reinforcing bars.

25.4.11.4 N_{cbg} shall be calculated according to 17.6.2 with the following modifications:

(a) Effective embedment depth, h_{ef}, of straight, hooked, or headed reinforcement shall be as defined in Chapter 2.
(b) For the calculation of concrete breakout strength in the anchorage region, it shall be permitted to take k_c in Eq. 17.6.2.2.1 as:
  (i) 35 for straight reinforcing bars
  (ii) 40 for hooked and headed reinforcement.
(c) Alternatively, for the calculation of concrete breakout strength of headed reinforcement in the anchorage region, it shall be permitted to use 5/3 times the value calculated with Eq. 17.6.2.2.3.
(d) The breakout cracking factor, $\psi_{c,N}$, shall be taken as 1.0.

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R25.4.11 *Anchorage of bar groups in tension*

R25.4.11.1 The required strength of a reinforcing bar group depends on the applicable design provisions. Some reinforcing bars are required to develop a force from the applicable load combination, others are required to develop a force associated with f_y, and yet others are required to develop a force associated with $1.25f_y$.

R25.4.11.4 The concrete breakout strength, N_{cbg}, is based on the provisions of Chapter 17 modified to provide a prediction of median strength in uncracked concrete that is compatible with the development length provisions.

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25.4.11.5 If reinforcement satisfying 25.4.11.6 is distributed within the region extending throughout the projected concrete failure area of the reinforcing bar group, N_{srg} shall be calculated according to Eq. 25.4.11.5. Otherwise, N_{srg} shall be taken as zero.

$$N_{srg} = \rho_t A_{c,eff} f_y \tag{25.4.11.5}$$

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R25.4.11.5 Section 25.4.11 permits the inclusion of reinforcement in the breakout calculation based on laboratory tests and numerical analysis (Worsfold et al. 2022; Worsfold and Moehle 2023a,b). The limits on the contribution of transverse reinforcement as a function of maximum reinforcement spacing reflect tests of column-foundation joints and special moment frame joints, whereby the closer spacing of confinement reinforcement in special moment frames is correlated with a low likelihood of concrete breakout as a controlling failure mode (Worsfold and Moehle 2023b; Lee et al. 2024). Figure R25.4.11.5a illustrates uplift tension on a bar group terminating in a foundation. The theoretical concrete failure area projecting to the free surface is shown. The strain field associated with this failure area is complex and will produce multiple nested fracture planes prior to formation of the final breakout surface. Therefore, bars located within and adjacent to the bar group contribute to N_{rg} regardless of whether they are developed beyond the theoretical failure surface (Worsfold and Moehle 2023b). In accordance with the definition of $A_{c,eff}$, reinforcement parallel to the developed bars and within a distance $0.75h_{ef}$ from the outside boundary of the bar group is additive to the calculated breakout strength provided the requirements of 25.4.11.6 are met. Where possible, hooked reinforcement should engage perpendicular reinforcement.

Figure R25.4.11.5b illustrates a similar case with haunch reinforcement whereby closed hoops within $0.75h_{ef}$ above and below the primary corbel reinforcement are additive to the calculated breakout strength.

The inclusion of nearby reinforcement in the breakout calculation as provided in 25.4.11.2 is distinct from reinforcement designed to carry the entire anchorage force (anchor reinforcement). Alternatively, anchor reinforcement as provided in 17.5.2.1 could be applied to this condition whereby the breakout strength calculation is avoided. However, the requirements that anchor reinforcement be used exclusively to carry the anchorage tension force and satisfy development length requirements on both sides of the theoretical breakout surface should be considered.

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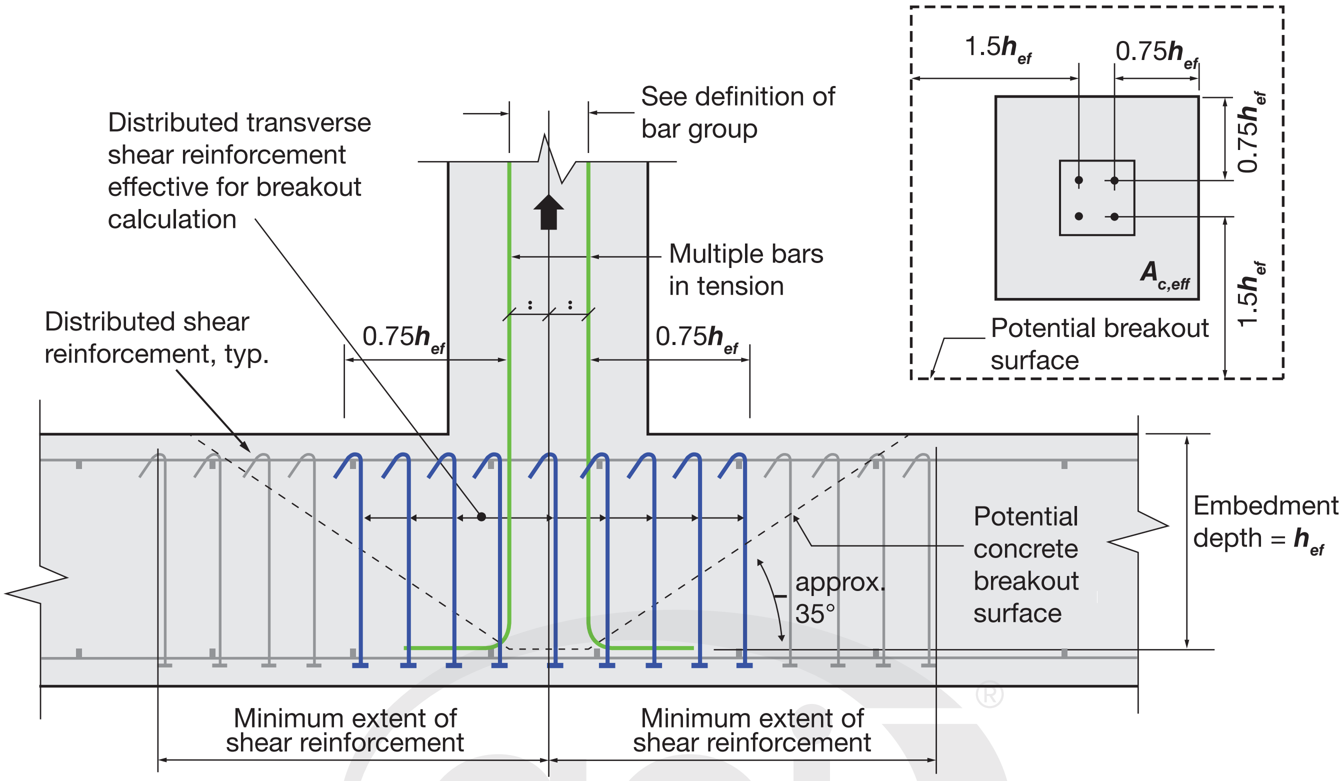

Fig. R25.4.11.5a—Use of shear reinforcement to increase the breakout strength of a group of column bars in tension

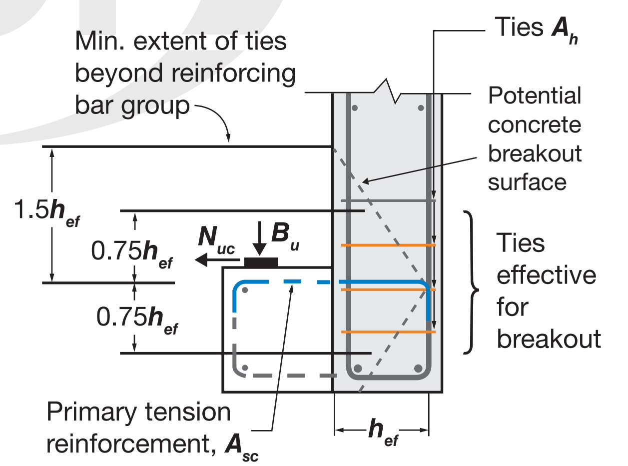

Fig. R25.4.11.5b—Use of closed tie reinforcement to increase the breakout strength of corbel primary reinforcement in tension

25.4.11.6 Distributed reinforcement effective for the calculation of N_{srg} shall satisfy (a) through (f):

(a) Reinforcement shall be parallel to the bar group and located within $A_{c,eff}$.
(b) If $N_{rg} \leq 2.5N_{cbg}$, reinforcement shall be spaced no greater than $0.5h_{ef}$ in each orthogonal direction.
(c) If $N_{rg} > 2.5N_{cbg}$, reinforcement shall be spaced no greater than $0.25h_{ef}$ in each orthogonal direction.

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(d) Reinforcement shall extend over at least 90 percent of the embedment depth of the bar group and terminate beyond the end of the bar group.
(e) Reinforcement shall be provided with a head or hook at each end, and hooked bars shall satisfy the requirements for stirrups in 25.7.1.3.
(f) Reinforcing bar diameter shall not exceed the diameter of the smallest bar in the bar group.

25.4.11.7 If the strength in the anchorage region is determined using anchor reinforcement conforming to 17.5.2.1, it shall not be additive to the breakout strength.

25.5—Splices

25.5.1 *General*

25.5.1.1 Lap splices shall not be permitted for bars larger than No. 11, except as provided in 25.5.5.3.

25.5.1.2 For contact lap splices, minimum clear spacing between the contact lap splice and adjacent splices or bars shall be in accordance with the requirements for individual bars in 25.2.1.

25.5.1.3 For noncontact splices in flexural members, the transverse center-to-center spacing of spliced bars shall not exceed the lesser of one-fifth the required lap splice length and 6 in.

25.5.1.4 Reduced embedment in accordance with 25.4.10 shall not apply to lap splice lengths.

25.5.1.5 For bars with $f_y \geq 80{,}000$ psi spaced closer than 6 in. on center, transverse reinforcement shall be provided such that K_{tr} shall not be smaller than $0.5d_b$.

25.5.1.6 Non-contact lap splices for reinforcement in shotcrete shall have clear spacing in accordance with (a) or (b):

(a) For. No. 6 and smaller bars, the clear spacing between bars shall be at least greater of $6d_b$ and 2-1/2 in.
(b) For. No. 7 and larger bars, the clear spacing shall be established using a shotcrete mockup panel to demonstrate that the reinforcement is properly encased.

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R25.5—Splices

R25.5.1 *General*

Lap splice lengths of longitudinal reinforcement in columns should be calculated in accordance with 10.7.5, 18.7.4.4, and this section.

R25.5.1.1 Because of lack of adequate experimental data on lap splices of No. 14 and No. 18 bars in compression and in tension, lap splicing of these bar sizes is prohibited except as permitted in 25.5.5.3 for compression lap splices of No. 14 and No. 18 bars with smaller bars.

R25.5.1.3 If individual bars in noncontact lap splices are too widely spaced, an unreinforced section is created. Forcing a potential crack to follow a zigzag line (5-to-1 slope) is considered a minimum precaution. The 6 in. maximum spacing is added because most research available on the lap splicing of deformed bars was conducted with reinforcement within this spacing.

R25.5.1.6 and **R25.5.1.7** Information on shotcrete mockup panels is provided in ACI PRC-506, and information on evaluating shotcrete is provided in ACI PRC-506.4.

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25.5.1.7 Contact lap splices for reinforcement in shotcrete shall be oriented with the plane of the spliced bars perpendicular to the surface of the shotcrete and approved by the licensed design professional based on a shotcrete mockup panel to demonstrate that the reinforcement is properly encased.

25.5.1.8 Lap splices of bundled bars shall be in accordance with 25.6.1.7.

25.5.2 *Lap splice lengths of deformed bars and deformed wires in tension*

25.5.2.1 Tension lap splice length ℓ_{st} for deformed bars and deformed wires in tension shall be in accordance with Table 25.5.2.1, where ℓ_d shall be in accordance with 25.4.2.1(a).

Table 25.5.2.1—Lap splice lengths of deformed bars and deformed wires in tension

$A_{s,provided}/A_{s,required}$[1] over length of splice	Maximum percent of A_s spliced within required lap length	Splice type	ℓ_{st}	
≥ 2.0	50	Class A	Greater of:	$1.0\ell_d$ and 12 in.
	100	Class B	Greater of:	$1.3\ell_d$ and 12 in.
< 2.0	All cases	Class B		

[1]Ratio of area of reinforcement provided to area of reinforcement required by analysis at splice location.

25.5.2.2 If bars of different size are lap spliced in tension, ℓ_{st} shall be the greater of ℓ_d of the larger bar and ℓ_{st} of the smaller bar.

25.5.3 *Lap splice lengths of welded deformed wire reinforcement in tension*

25.5.3.1 Tension lap splice length ℓ_{st} of welded deformed wire reinforcement in tension with cross wires within the lap splice length shall be the greater of $1.3\ell_d$ and 8 in., where ℓ_d is calculated in accordance with 25.4.6.1(a), provided (a) and (b) are satisfied:

(a) Overlap between outermost cross wires of each reinforcement sheet shall be at least 2 in.

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R25.5.2 *Lap splice lengths of deformed bars and deformed wires in tension*

R25.5.2.1 Lap splices in tension are classified as Class A or B, with length of lap a multiple of the tensile development length ℓ_d calculated in accordance with 25.4.2.3 or 25.4.2.4. The two-level lap splice requirements encourage splicing bars at points of minimum stress and staggering splices to improve behavior of critical details. For the purpose of calculating ℓ_d for staggered splices, the clear spacing is taken as the minimum distance between adjacent splices, as illustrated in Fig. R25.5.2.1.

The tension lap splice requirements encourage the location of splices away from regions of high tensile stress to locations where the area of steel provided is at least twice that required by analysis.

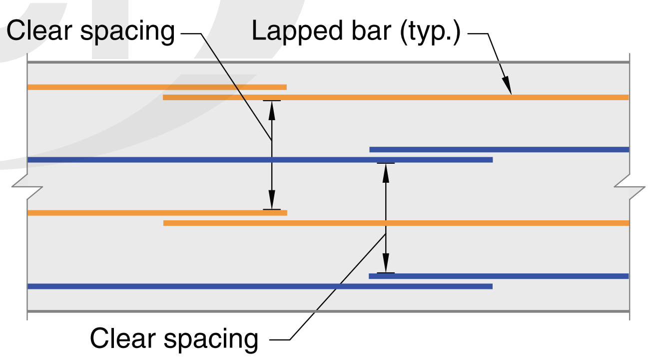

Fig. R25.5.2.1—Clear spacing of lap-spliced bars for determination of ℓ_d for staggered splices.

R25.5.3 *Lap splice lengths of welded deformed wire reinforcement in tension*

R25.5.3.1 Splice provisions for welded deformed wire reinforcement are based on available tests (Lloyd and Kesler 1969). Lap splices for welded deformed wire reinforcement meeting the requirements of this provision and 25.5.3.1.1 are illustrated in Fig. R25.5.3.1. If no cross wires are within the lap length, the provisions for deformed wire apply.

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(b) Wires in the direction of the development length shall all be deformed D31 or smaller

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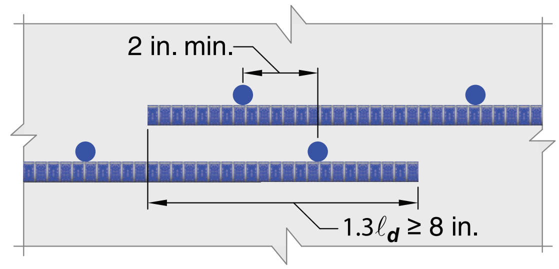
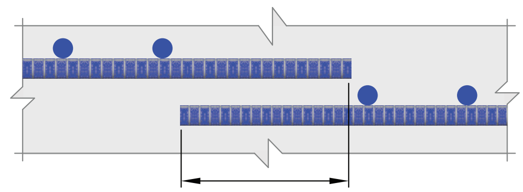


Lap splice satisfies R25.5.3.1a



Lap splice satisfies R25.5.3.1.1

Fig. R25.5.3.1—Lap splices of welded deformed wire reinforcement.

25.5.3.1.1 If 25.5.3.1(a) is not satisfied, ℓ_{st} shall be calculated in accordance with 25.5.2.

25.5.3.1.2 If 25.5.3.1(b) is not satisfied, ℓ_{st} shall be calculated in accordance with 25.5.4.

R25.5.3.1.2 Where any plain wires, or deformed wires larger than D31, are present in the welded deformed wire reinforcement in the direction of the lap splice or where welded deformed wire reinforcement is lap spliced to welded plain wire reinforcement, the reinforcement should be lap spliced in accordance with the plain wire reinforcement lap splice requirements. Deformed wire larger than D31 is treated as plain wire because tests show that D45 wire will achieve only approximately 60 percent of the bond strength in tension given by Eq. (25.4.2.4a) (Rutledge and DeVries 2002).

25.5.3.1.3 If the welded deformed wire reinforcement is zinc-coated (galvanized), ℓ_{st} shall be calculated in accordance with 25.5.4.

25.5.4 *Lap splice lengths of welded plain wire reinforcement in tension*

R25.5.4 *Lap splice lengths of welded plain wire reinforcement in tension*

25.5.4.1 Tension lap splice length ℓ_{st} of welded plain wire reinforcement in tension between outermost cross wires of each reinforcement sheet shall be at least the greatest of (a) through (c):

(a) $s + 2$ in.
(b) $1.5\ell_d$
(c) 6 in.

where s is the spacing of cross wires and ℓ_d is calculated in accordance with 25.4.7.2(b).

R25.5.4.1 The strength of lap splices of welded plain wire reinforcement is dependent primarily on the anchorage obtained from the cross wires rather than on the length of wire in the splice. For this reason, the lap is specified in terms of overlap of cross wires (in inches) rather than in wire diameters or length. The 2 in. additional lap required is to provide adequate overlap of the cross wires and to provide space for satisfactory consolidation of the concrete between cross wires. Research (Lloyd 1971) has shown an increased splice length is required when welded wire reinforcement of large, closely spaced wires is lapped and, as a consequence, additional splice length requirements are provided for this

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reinforcement in addition to an absolute minimum of 6 in. Splice requirements are illustrated in Fig. R25.5.4.1. If $A_{s,provided}/A_{s,required} \geq 2$ over the length of the splice, ℓ_{st} can be determined from 25.5.4.2.

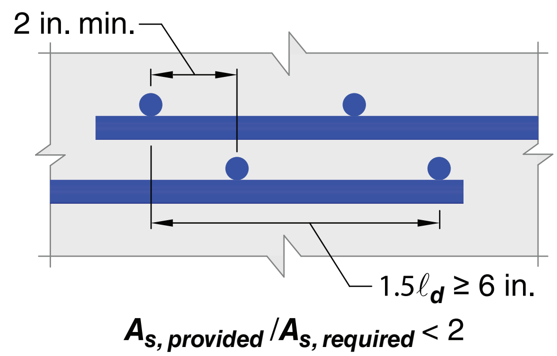

Fig. R25.5.4.1—Lap splices of plain welded wire reinforcement where $A_{s,provided}/A_{s,required} < 2$*.*

25.5.4.2 If $A_{s,provided}/A_{s,required} \geq 2.0$ over the length of the splice, ℓ_{st} measured between outermost cross wires of each reinforcement sheet shall be permitted to be the greater of (a) and (b).

(a) $1.5\ell_d$
(b) 2 in.

where ℓ_d is calculated by 25.4.7.2(b).

R25.5.4.2 Where $A_{s,provided}/A_{s,required} \geq 2$, the lap splice for plain welded wire reinforcement is illustrated in Fig. R25.5.4.2.

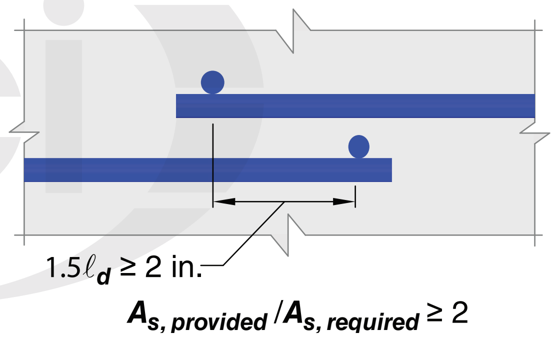

Fig. R25.5.4.2—Lap splices of plain welded wire reinforcement where $A_{s,provided}/A_{s,required} \geq 2$*.*

25.5.5 *Lap splice lengths of deformed bars in compression*

R25.5.5 *Lap splice lengths of deformed bars in compression*

Bond research has been primarily related to bars in tension. Bond behavior of compression bars is not complicated by the problem of transverse tension cracking and thus compression splices do not require provisions as strict as those specified for tension splices.

Lap splice requirements particular to columns are provided in Chapter 10.

25.5.5.1 Compression lap splice length ℓ_{sc} of No. 11 or smaller deformed bars in compression shall be calculated in accordance with (a), (b), or (c):

(a) For $f_y \leq 60{,}000$ psi: ℓ_{sc} is the longer of $0.0005f_yd_b$ and 12 in.
(b) For 60,000 psi $< f_y \leq 80{,}000$ psi: ℓ_{sc} is the longer of $(0.0009f_y - 24)d_b$ and 12 in.

R25.5.5.1 Tests (ACI Committee 408 1966; Pfister and Mattock 1963) have shown that splice strengths in compression depend considerably on end bearing and do not increase proportionally in strength when the splice length is doubled. Accordingly, for specified yield strengths above 60,000 psi, compression lap lengths are significantly increased.

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(c) For $f_y > 80{,}000$ psi, ℓ_{sc} is the longer of $(0.0009f_y - 24)d_b$ and ℓ_{st} calculated in accordance with 25.5.2.1.

For $f_c' < 3000$ psi, the length of lap shall be increased by one-third.

25.5.5.2 Compression lap splices shall not be used for bars larger than No. 11, except as permitted in 25.5.5.3.

25.5.5.3 Compression lap splices of No. 14 or No. 18 bars to No. 11 or smaller bars shall be permitted and shall be in accordance with 25.5.5.4.

25.5.5.4 Where bars of different size are lap spliced in compression, ℓ_{sc} shall be the longer of ℓ_{dc} of larger bar calculated in accordance with 25.4.9.1 and ℓ_{sc} of smaller bar calculated in accordance with 25.5.5.1 as appropriate.

25.5.6 *End-bearing splices of deformed bars in compression*

25.5.6.1 For bars required for compression only, transmission of compressive stress by end bearing of square-cut ends held in concentric contact by a suitable device shall be permitted.

25.5.6.2 End-bearing splices shall be permitted only in members containing closed stirrups, ties, spirals, or hoops.

25.5.6.3 Bar ends shall terminate in flat surfaces within 1.5 degrees of a right angle to the axis of the bars and shall be fitted within 3 degrees of full bearing after assembly.

25.5.7 *Mechanical and welded splices of deformed bars in tension or compression*

25.5.7.1 Mechanical splices shall satisfy the requirements of ASTM A1034 except as required in 25.5.7.2 and 25.5.7.6. Requirements in this Code shall take precedence over those in ASTM A1034.

25.5.7.2 Mechanical splices shall be classified as Class L, Class G, or Class S in accordance with the requirements of Table 25.5.7.2, with associated prequalification testing requirements specified in 25.5.7.6. Use of mechanical splices shall be in accordance with (a) and (b):

(a) Class L mechanical splices shall not be permitted for use in locations where yielding of reinforcement is expected under applicable load combinations of 5.3

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R25.5.5.3 Lap splices are generally prohibited for No. 14 or No. 18 bars. For compression only, however, lap splices are permitted between No. 14 or No. 18 bars and No. 11 or smaller bars.

R25.5.6 *End-bearing splices of deformed bars in compression*

R25.5.6.1 Experience with end-bearing splices has been almost exclusively with vertical bars in columns. If bars are significantly inclined from the vertical, attention is required to ensure that adequate end-bearing contact can be achieved and maintained.

R25.5.6.2 This limitation ensures a minimum shear resistance in sections containing end-bearing splices.

R25.5.6.3 These tolerances represent practice based on tests of full-size members containing No. 18 bars.

R25.5.7 *Mechanical and welded splices of deformed bars in tension or compression*

R25.5.7.1 The mechanical splice requirements of 25.5.7.2 replace the mechanical splice requirements given in ASTM A1034, including classification of mechanical splices and names of classes. The prequalification testing requirements of 25.5.7.6 replace similar testing requirements given in ASTM A1034.

R25.5.7.2 Requirements for three classes of mechanical splices are provided based on Sharma et al. (2025): Class L, Class G, and Class S. These classifications replace the former classifications of Type 1 and Type 2. Additional requirements for mechanical splices used in seismic applications are given in Chapter 18. Class L (Limited) mechanical splices are restricted to applications where development of actual yield in the reinforcing bars at the location of the mechanical splice is not anticipated. Applications where Class L mechanical splices are not permitted include, but

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(b) Class L mechanical splices shall not be permitted for splicing the following reinforcement:

(i) integrity reinforcement
(ii) reinforcement in regions where moment redistribution has been applied in design
(iii) reinforcement in regions where moments are determined using the simplified method of analysis of 6.5
(iv) reinforcement in regions where moments are determined using the direct design method

Table 25.5.7.2 — Mechanical splice requirements

Property	Prequalification tests	Mechanical splice classification: Class L	Class G	Class S
		Requirement		
Tensile strength, minimum, and compressive strength, minimum	25.5.7.6(a) and 25.5.7.6(b)	f_u		
Tensile strain, minimum	25.5.7.6(c)	No requirement	2%	6%
Elastic cyclic endurance, minimum	25.5.7.6(d)	No requirement	20 cycles	
Residual slip, maximum	25.5.7.6(e)	No requirement	0.02 in.	
Inelastic cyclic endurance, minimum	25.5.7.6(f)	No requirement		30 cycles

25.5.7.2.1 Where Class S mechanical splices are required, reinforcement shall conform to 20.2.2.5.

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may not be limited to, locations where bar yielding may occur under non-cyclic loading, where yielding may occur under wind loading as anticipated by the performance-based design provisions of Appendix B, where integrity reinforcement is mechanically spliced, and where design is based on analysis methods that implicitly consider yielding of reinforcement such as moment redistribution, the simplified method of analysis of 6.5, and the direct design method for two-way slabs. Restrictions on use of Class L mechanical splices in seismic applications are given in Chapter 18.

Class G (General) mechanical splices are intended for applications where development of actual yield in the reinforcing bars at the location of the mechanical splice may occur, but where significant inelastic cyclic loading is not anticipated and strain-hardening demands on the reinforcement will be limited. The minimum tensile strain capacity of 2 percent specified for a Class G mechanical splice is intended to develop, under non-cyclic loading, the actual yield strength and a limited degree of strain hardening in the bars being spliced. Restrictions on the use of Class G mechanical splices in seismic applications are given in Chapter 18.

Class S (Special) mechanical splices are intended for applications where development of considerable inelastic strain in the reinforcing bars at the location of the mechanical splice may occur, either monotonically or cyclically, such as under earthquake loads. The minimum tensile strain capacity specified for a Class S mechanical splice is intended to provide strain capacities approaching the specified minimum uniform elongation of the reinforcement. For Class G and Class S mechanical splices, the elastic cyclic testing requirement is intended to provide for limited elastic cyclic endurance. For Class G and Class S mechanical splices, the requirement to assess slip, potentially occurring at the interface between the mechanical splicing device and the reinforcing bars being joined, is intended to control concrete crack widths at service loads and aggregate interlock at factored loads. Control of slip also promotes capacity for cyclic energy dissipation in bars spliced with a Class S mechanical splice, because slip can cause pinching of hysteretic loops under inelastic cyclic loads.

The inelastic cyclic endurance requirement for the Class S mechanical splice is intended to provide an inelastic, yield-reversal cyclic endurance for the mechanical splice comparable to the 5th percentile of the number of cycles to failure of an unspliced bar under similar cyclic inelastic strain loading (Slavin and Ghannoum 2015; Ghannoum and Slavin 2016; Sokoli et al. 2019).

R25.5.7.2.1 ASTM A706 reinforcement is required to be used in Class S mechanical splices because it is the only reinforcing bar specification that includes requirements for minimum uniform elongation, which equal or exceed the minimum tensile strain requirement in Table 25.5.7.2 for Class S mechanical splices. Additionally, the radius at the deformation base is controlled with ASTM A706 reinforcement, which allows the ASTM A706 reinforcing bar in the

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Class S mechanical splice system to develop the inelastic cyclic endurance specified in Table 25.5.7.2. Reinforcing bars that are manufactured according to any standard lacking these particular requirements, such as ASTM A615, ASTM A955, ASTM A996, and ASTM A1035, are not permitted for use in Class S splices because the reinforcing bar itself may not satisfy the Class S tensile strain and cyclic endurance requirements of Table 25.5.7.2. Consequently, where a Class S mechanical splicing device is used with reinforcement other than ASTM A706, the resulting mechanical splice may not satisfy Class S requirements.

25.5.7.2.2 Reinforcing bars in a Class S mechanical splice shall be longitudinally aligned.

R25.5.7.2.2 Use of an aligned configuration of the reinforcing bars being mechanically spliced is intended to minimize effects of eccentricity. The two bars joined with a Class S mechanical splice have longitudinal centerlines aligned. Mechanical splice devices that connect overlapping bars cannot be Class S.

25.5.7.2.3 Test lots for a mechanical splice of a given combination of bar grade and bar size satisfying all Class S prequalification testing requirements shall be deemed to satisfy Class G and Class L prequalification testing requirements for the given combination of bar grade and bar size. Test lots for a mechanical splice of a given combination of bar grade and bar size satisfying all Class G prequalification testing requirements shall be deemed to satisfy Class L prequalification testing requirements for the given combination of bar grade and bar size.

25.5.7.2.4 Test lots for a mechanical splice of a given combination of bar grade and bar size satisfying prequalification testing requirements for a particular Class shall be deemed to satisfy prequalification testing requirements for that Class for lower grade reinforcing bars of the same size.

25.5.7.2.5 Mechanical splices that are prequalified to Type 2 mechanical splice requirements of ACI CODE-318-19 shall be deemed to satisfy the requirements of Class L mechanical splices, provided that the methods used for prequalification conform to the test methods specified in 20.5.7.6 for Class L mechanical splices.

R25.5.7.2.5 Class L mechanical splices have requirements equivalent to Type 2 mechanical splices in the 2019 edition of the Code.

25.5.7.3 Welded splices shall satisfy (a) through (d):

(a) The weld in a welded splice shall be proportioned to develop in tension or compression, as required, at least the specified minimum tensile strength of the bars being spliced.
(b) Welding of reinforcing bars shall conform to 26.6.4.
(c) Welded splices shall not be permitted for use in locations where yielding of reinforcement is expected under applicable load combinations of 5.3.
(d) Welded splices used in seismic applications shall be in accordance with 18.2.8.

R25.5.7.3 Welded splices are restricted to locations where yielding of reinforcing bars at the location of the welded splice is not anticipated.

A welded splice is primarily intended for No. 6 bars and larger. The requirement to develop at least the specified tensile strength of the bars being spliced is intended to provide sound welding that is also adequate for compression.

While direct butt welds are not required, AWS D1.4 states that wherever practical, direct butt welds are preferable for No. 7 bars and larger.

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25.5.7.4 Mechanical or welded splices need not be staggered except as required by 25.5.7.5.

25.5.7.5 Splices in tension tie members shall be made with a mechanical or welded splice in accordance with 25.5.7. Splices in adjacent bars shall be staggered at least 30 in.

25.5.7.6 Prequalification testing of mechanical splices to demonstrate conformance with the requirements of Table 25.5.7.2 shall be in accordance with (a) through (g):

(a) Tensile strength and tensile strain shall be determined by monotonic tension tests to fracture. For Class G and Class S mechanical splices, the monotonic tensile test to fracture shall be performed on the test specimen from the elastic endurance test after application of the specified minimum number of elastic endurance cycles and after measurement of residual slip.
(b) Compressive strength shall be determined by monotonic compression tests, performed on test specimens not subject to any prior testing.
(c) For Class G and Class S mechanical splices, strain shall be measured and recorded throughout the cyclic endurance test and monotonic tension test. Strain shall be measured in the reinforcing bar adjacent to but not including the mechanical splicing device and related features, and outside of any portion of the reinforcing bar that has been altered by the mechanical splice installation and fabrication. Minimum gauge length for measurement of strain shall be $2d_b$.
(d) Cycles of loading for the elastic endurance test shall be tension-compression cycles, between $0.95f_y$ in tension and $0.5f_y$ in compression, with forces determined using the nominal area of the reinforcing bar being spliced. The minimum number of elastic load cycles applied shall be in accordance with Table 25.5.7.2. All specimens tested for elastic endurance shall survive the specified minimum number of cycles without fracture. Slip across the mechanical splicing device shall be measured and recorded throughout the cyclic endurance test. The measurement

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R25.5.7.4 Although mechanical and welded splices need not be staggered, staggering is encouraged and may be necessary to provide for installation of the splice or to meet clear spacing requirements. Staggering splices will also help to avoid creating a discontinuity in the section that could promote the development of a concentrated crack at the discontinuity.

R25.5.7.5 A tension tie member has the following characteristics: member having an axial tensile force sufficient to create tension over the cross section; and limited concrete cover on all sides. Examples of members that may be classified as tension ties are arch ties, hangers carrying load to an overhead supporting structure, and main tension elements in a truss.

In determining if a member should be classified as a tension tie, consideration should be given to the importance, function, proportions, and stress conditions of the member related to the above characteristics. For example, a usual large circular tank, with many bars and with splices well staggered and widely spaced, should not be classified as a tension tie member.

R25.5.7.6 It is anticipated that conformance of a mechanical splicing device with the requirements of Table 25.5.7.2 will be demonstrated by prequalification testing. Applicable test methods may, in part, be found in ASTM A370 Test Methods and Definitions for Mechanical Testing of Steel Products, ASTM E8/E8M Test Methods for Tension Testing of Metallic Materials, and ASTM E9 Standard Test Methods of Compression Testing of Metallic Materials at Room Temperature. There is no single, consensus-based standard test method that comprehensively addresses laboratory testing related to the requirements given Table 25.5.7.2. These tests are not intended for field or project-specific quality control.

(d) The specified elastic cyclic loading is identical to the Stage 1 elastic cyclic loading provided in ICC-ES Acceptance Criteria AC133 (2020). A common loading rate for the elastic cyclic testing is approximately 4 cycles per minute.
(d and e) The cyclic loading for slip is the same as the Stage 1 elastic cyclic loading provided in AC133 (2020). The maximum limit on slip of 0.016 in. is based on common design practices for crack width control and differs from the 0.012 in. limit in AC133.
(f) During inelastic cycling, the yielding state in the reinforcing bar will alternate between yielding in tension and yielding in compression. Consequently, the inelastic cycles should be applied at a relatively slow rate. Commonly used rates are approximately 0.5 to 1.0 cycles per minute (Slavin and Ghannoum 2015).

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gauge length for slip shall include the mechanical splicing device plus d_b to $2d_b$ beyond each end of the mechanical splicing device.

(e) Residual slip shall be determined as the distance between the slip measurement gauge points specified in 25.5.7.6(d) at zero force in the reinforcing bar, taken at the end of unloading from tension upon completion of the last load cycle of the elastic cyclic endurance test specified in 25.5.7.6(d), minus the distance between the same gauge points prior to the application of any load at the start of the first load cycle of the elastic cyclic endurance test.

(f) Cycles of loading for the inelastic cyclic endurance test shall be inelastic strain cycles between 2% strain in tension and 0.5% strain in compression, where the strain shall be measured in the reinforcing bar adjacent to but not including the mechanical splicing device. Clear distance between the face of a test machine grip and each end of the mechanical splicing device shall be no less than $1.5d_b$. Minimum gauge length for measurement of strain shall be $1.5d_b$ in the bar beyond each end of the mechanical splicing device, and the strains measured beyond each end of the mechanical splicing device shall be averaged. It shall be permitted to laterally brace the test specimen, provided that the lateral braces do not attract axial forces from the test loads applied to the test specimen. The minimum number of inelastic load cycles applied shall be in accordance with Table 25.5.7.2. Each inelastic cyclic endurance test shall be performed on a test specimen not subject to any prior testing.

(g) One lot of tests shall be performed for each applicable test type for a given combination of mechanical splice class, bar grade, and bar size. Each lot of tests shall consist of five tests of the same test type on replicate mechanical splice specimens of the same combination of mechanical splice device, bar grade, and bar size. The results of the tests for a given test lot are acceptable if all five tests satisfy the applicable requirements of Table 25.5.7.2. If only four of the five tests in the test lot satisfy the applicable requirements of Table 25.5.7.2, then two additional tests of that test type shall be performed on additional replicate mechanical splice specimens. If both additional tests satisfy the applicable requirements of Table 25.5.7.2, the test lot is acceptable. For a mechanical splicing device to be acceptable for use as a given class of mechanical splice on a given combination of bar grade and bar size, all lots of tests for test types applicable to the given class of mechanical splice shall be acceptable.

25.6—Bundled reinforcement

25.6.1 *Nonprestressed reinforcement*

25.6.1.1 Groups of parallel reinforcing bars bundled in contact to act as a unit shall be limited to four in any one bundle.

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R25.6—Bundled reinforcement

R25.6.1 *Nonprestressed reinforcement*

R25.6.1.1 The Code phrase "bundled in contact to act as a unit" is intended to preclude bundling more than two bars in the same plane. Typical bundle shapes in cross section are triangular, L-shaped, or square-shaped patterns for three- or

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25.6.1.2 Bundled bars shall be enclosed within transverse reinforcement. Bundled bars in compression members shall be enclosed by transverse reinforcement at least No. 4 in size.

25.6.1.3 Bars larger than a No. 11 shall not be bundled in beams.

25.6.1.4 Individual bars within a bundle terminated within the span of flexural members shall terminate at different points with at least $40d_b$ stagger.

25.6.1.5 Development length for individual bars within a bundle, in tension or compression, shall be that of the individual bar, increased 20% for a three-bar bundle, and 33% for a four-bar bundle.

25.6.1.6 A unit of bundled bars shall be treated as a single bar with an area equivalent to that of the bundle and a centroid coinciding with that of the bundle. The diameter of the equivalent bar shall be used for d_b in (a) through (e):

(a) Spacing limitations based on d_b
(b) Cover requirements based on d_b
(c) Spacing and cover values in 25.4.2.3
(d) Confinement term in 25.4.2.4
(e) ψ_e factor in 25.4.2.5

25.6.1.7 Lap splices of bars in a bundle shall be based on the lap splice length required for individual bars within the bundle, increased in accordance with 25.6.1.5. Individual bar splices within a bundle shall not overlap. Entire bundles shall not be lap spliced.

25.6.2 *Post-tensioning ducts*

25.6.2.1 Bundling of post-tensioning ducts shall be permitted if shown that concrete can be satisfactorily placed and if provision is made to prevent the prestressed reinforcement from breaking through the duct.

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four-bar bundles. As a practical caution, bundles more than one bar deep in the plane of bending should not be hooked or bent as a unit. Where end hooks are required, it is preferable to stagger the individual bar hooks within a bundle.

R25.6.1.3 A limitation that bars larger than No. 11 not be bundled in beams is a practical limit for application to building size members. (AASHTO LRFDUS Article 5.9.4 permits two-bar bundles for No. 14 and No. 18 bars in bridge girders.) Conformance to the crack control requirements of 24.3 will effectively preclude bundling of bars larger than No. 11 as tension reinforcement.

R25.6.1.4 Bond research (ACI Committee 408 1966) has shown that cutoff points within bundles should be staggered.

R25.6.1.5 An increased development length for individual bars is required when three or four bars are bundled together. The extra extension is needed because the grouping makes it more difficult to mobilize bond resistance from the core between the bars.

The development of bundled bars by a standard hook of the bundle is not covered by the provisions of 25.4.3.

R25.6.1.6 Although splice and development lengths of bundled bars are a multiple of the diameter of the individual bars being spliced increased by 20 or 33 percent, as appropriate, it is necessary to use an equivalent diameter of the entire bundle derived from the equivalent total area of bars for determining the spacing and cover values in 25.4.2.3, the confinement term, $[(c_b + K_{tr})/d_b]$, in 25.4.2.4, and the ψ_e factor in 25.4.2.5. For bundled bars, bar diameter d_b outside the brackets in the expressions of 25.4.2.3 and of Eq. (25.4.2.4a) is that of a single bar.

R25.6.1.7 The increased length of lap required for bars in bundles is based on the reduction in the exposed perimeter of the bars. Only individual bars are lap spliced along the bundle.

R25.6.2 *Post-tensioning ducts*

R25.6.2.1 Where ducts for prestressing reinforcement in a beam are arranged closely together vertically, provisions should be made to prevent the prestressed reinforcement from breaking through the duct when tensioned. Horizontal arrangement of ducts should allow proper placement of concrete. A clear spacing of one and one-third times the

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nominal maximum size of the coarse aggregate, but not less than 1 in., has proven satisfactory.

Where concentration of tendons or ducts tends to create a weakened plane in the concrete cover, reinforcement should be provided to control cracking.

25.7—Transverse reinforcement

25.7.1 *Stirrups*

25.7.1.1 Stirrups shall extend as close to the extreme compression and tension surfaces of the member as cover requirements and proximity of other reinforcement permits and shall be anchored at both ends. Where used as shear reinforcement, stirrups shall extend a distance d from extreme compression fiber.

25.7.1.2 Between anchored ends, each bend in the continuous portion of a single or multiple U-stirrup and each bend in a closed stirrup shall enclose a longitudinal bar or strand.

25.7.1.3 Anchorage of deformed bar and wire with one or more bends shall be in accordance with (a), (b), or (c):

(a) For No. 5 bar and D31 wire, and smaller, and for No. 6 through No. 8 bars with $f_{yt} \leq 40{,}000$ psi, a standard hook around longitudinal reinforcement

(b) For No. 6 through No. 8 bars with $f_{yt} > 40{,}000$ psi, a standard hook around a longitudinal bar plus an embedment between midheight of the member and the outside end of the hook equal to or greater than $0.014d_b f_{yt}/(\lambda\sqrt{f_c'})$, with λ as given in Table 25.4.3.2

(c) In joist construction, for No. 4 bar and D20 wire and smaller, a standard hook

25.7.1.4 Anchorage of each leg of welded wire reinforcement forming a single U-stirrup shall be in accordance with (a) or (b):

R25.7—Transverse reinforcement

R25.7.1 *Stirrups*

R25.7.1.1 Stirrup legs should be extended as close as practicable to the compression face of the member because, near ultimate load, the flexural tension cracks penetrate deeply toward the compression zone.

It is essential that transverse reinforcement be developed on both sides of the shear plane. This generally requires a hook, head, or bend at the end of the reinforcement as provided by this section.

R25.7.1.3 Straight deformed bar and wire anchorage is not permitted because it is difficult to hold such a stirrup in position during concrete placement. Moreover, the lack of a standard stirrup hook may make the stirrup ineffective as it crosses shear cracks near the end of the stirrup.

For a No. 5 or D31 or smaller stirrup, anchorage is provided by a standard hook, as defined in 25.3.2, hooked around a longitudinal bar.

For a No. 6, No. 7, or No. 8 stirrup with f_{yt} of only 40,000 psi, a standard stirrup hook around a longitudinal bar provides sufficient anchorage. For a No. 6, No. 7, or No. 8 stirrup with higher strength, the embedment should be checked. A 135-degree or 180-degree hook is preferred, but a 90-degree hook may be used provided the free end of the 90-degree hook is extended the full 12 bar diameters as required in 25.3.2. Because it is not possible to bend a No. 6, No. 7, or No. 8 stirrup tightly around a longitudinal bar and due to the force in a bar with a design stress greater than 40,000 psi, stirrup anchorage depends on both the type of hook and whatever development length is provided. A longitudinal bar within a stirrup hook limits the width of any flexural cracks, even in a tension zone. Because such a stirrup hook cannot fail by splitting parallel to the plane of the hooked bar, the hook strength as used in 25.4.3.1(a) has been adjusted to reflect cover and confinement around the stirrup hook.

In joists, a small bar or wire can be anchored by a standard hook not engaging longitudinal reinforcement, allowing a continuously bent bar to form a series of single-leg stirrups along the length of the joist.

R25.7.1.4 The requirements for anchorage of welded wire reinforcement stirrups are illustrated in Fig. R25.7.1.4.

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(a) Two longitudinal wires spaced at a 2 in. spacing along the member at the top of the U

(b) One longitudinal wire located not more than $d/4$ from the compression face and a second wire closer to the compression face and spaced not less than 2 in. from the first wire. The second wire shall be permitted to be located on the stirrup leg beyond a bend, or on a bend with an inside diameter of bend of at least $8d_b$.

25.7.1.5 Anchorage of each end of a single leg stirrup of welded wire reinforcement shall be with two longitudinal wires at a minimum spacing of 2 in. in accordance with (a) and (b):

(a) Inner longitudinal wire at least the greater of $d/4$ or 2 in. from $d/2$

(b) Outer longitudinal wire at tension face shall not be farther from the face than the portion of primary flexural reinforcement closest to the face

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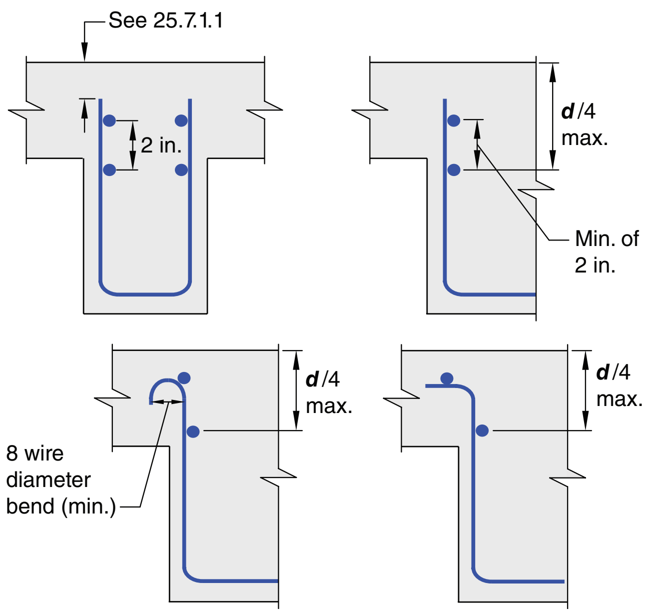

Fig. R25.7.1.4—Anchorage in compression zone of welded wire reinforcement U-stirrups.

R25.7.1.5 Welded wire reinforcement for shear reinforcement is commonly used in the precast, prestressed concrete industry. The rationale for acceptance of straight sheets of welded wire reinforcement as shear reinforcement is presented in a report by the Joint PCI/WRI Ad Hoc Committee on Welded Wire Fabric for Shear Reinforcement (1980).

The provisions for anchorage of single-leg welded wire reinforcement in the tension face emphasize the location of the longitudinal wire at the same depth as the primary flexural reinforcement to avoid a splitting problem at the level of the tension reinforcement. Figure R25.7.1.5 illustrates the anchorage requirements for single-leg welded wire reinforcement. For anchorage of single-leg welded wire reinforcement, the Code permits hooks and embedment length in the compression and tension faces of members (refer to 25.7.1.3(a) and 25.7.1.4), and embedment only in the compression face (refer to 25.7.1.3(b)). This section provides for anchorage of straight, single-leg, welded wire reinforcement using longitudinal wire anchorage with adequate embedment length in compression and tension faces of members.

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25.7.1.6 Stirrups used for torsion or integrity reinforcement shall be closed stirrups perpendicular to the axis of the member. Where welded wire reinforcement is used, transverse wires shall be perpendicular to the axis of the member. Such stirrups shall be anchored by (a) or (b):

(a) Ends shall terminate with 135-degree standard hooks around a longitudinal bar

(b) In accordance with 25.7.1.3(a) or (b) or 25.7.1.4, where the concrete surrounding the anchorage is restrained against spalling by a flange or slab or similar member

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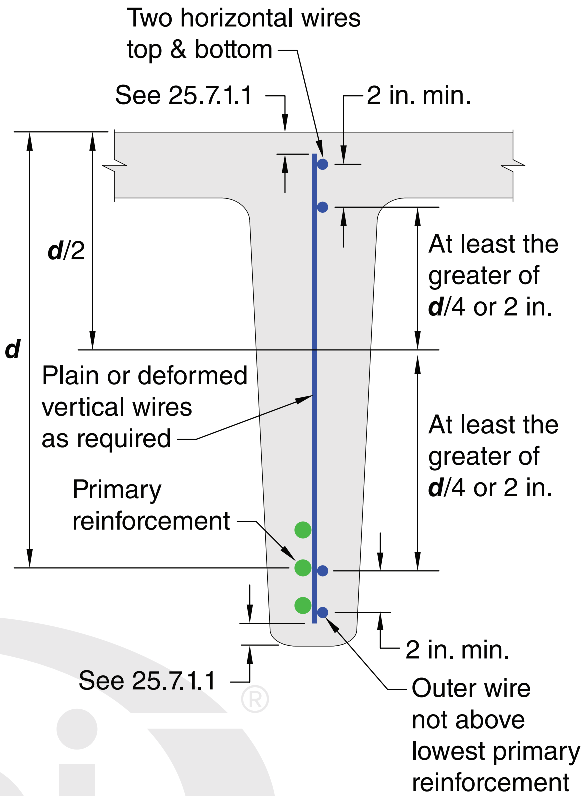

Fig. R25.7.1.5—Anchorage of single-leg welded wire reinforcement for shear.

R25.7.1.6 Both longitudinal and closed transverse reinforcement are required to resist the diagonal tension stresses due to torsion. The stirrups should be closed because inclined cracking due to torsion may occur on all faces of a member.

In the case of sections subjected primarily to torsion, the concrete side cover to the stirrups spalls off at high torsional moments (Mitchell and Collins 1976). This renders lap-spliced stirrups ineffective, leading to a premature torsional failure (Behera and Rajagopalan 1969). In such cases, closed stirrups should not be made up of pairs of U-stirrups lapping one another.

When a rectangular beam fails in torsion, the corners of the beam tend to spall off due to the inclined compressive stresses in the concrete diagonals of the space truss changing direction at the corner as shown in Fig. R25.7.1.6(b). In tests (Mitchell and Collins 1976), closed stirrups anchored by 90-degree hooks failed when this occurred. For this reason, 135-degree standard hooks or seismic hooks are preferable for torsional stirrups in all cases. In regions where this spalling is prevented by an adjacent slab or flange, 25.7.1.6(a) relaxes this requirement and allows 90-degree hooks because of the added confinement from the slab (refer to Fig. R25.7.1.6(a)).

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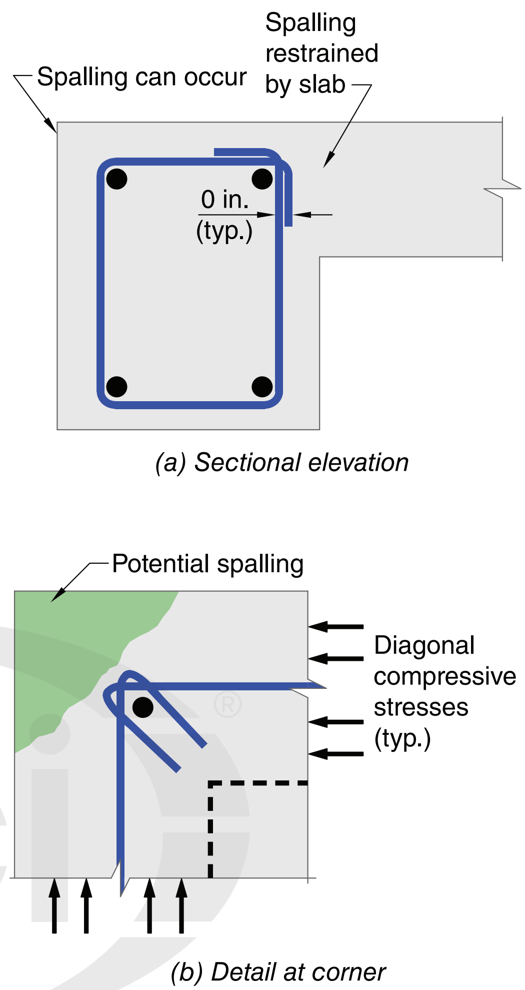

Fig. R25.7.1.6—Spalling of corners of beams subjected to torsion.

25.7.1.6.1 Stirrups used for torsion or integrity reinforcement shall be permitted to be made up of two pieces of reinforcement: a single U-stirrup anchored according to 25.7.1.6(a) closed by a crosstie where the 90-degree hook of the crosstie shall be restrained against spalling by a flange or slab or similar member.

R25.7.1.6.1 Figure R25.7.1.6.1 shows an example of a two-piece stirrup that satisfies the requirement of 25.7.1.6.1. The 90-degree hook of the cap tie is located on the slab side so that it is better confined. Pairs of U-stirrups lapping one another as defined in 25.7.1.7 are not permitted in perimeter or spandrel beams. In the event of damage to the side concrete cover, the top longitudinal reinforcement may tend to tear out of the concrete and will not be adequately restrained by the exposed lap splice of the stirrup. Thus, the top longitudinal reinforcement will not provide the catenary action needed to bridge over a damaged region. Further, lapped U-stirrups will not be effective at high torsional moments as discussed in R25.7.1.6.

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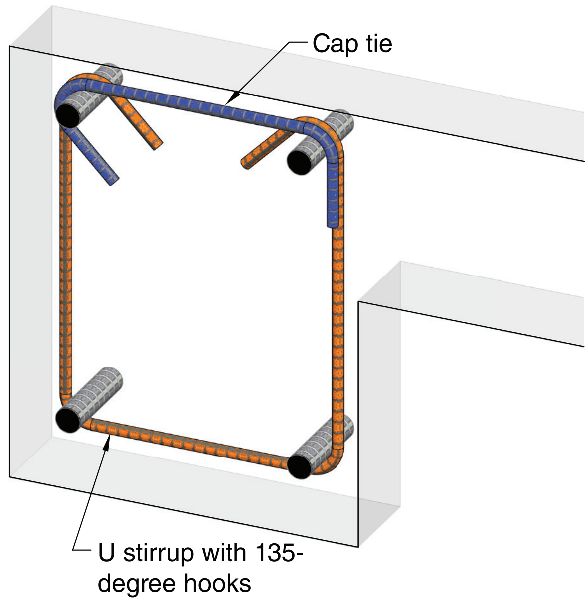

Fig. R25.7.1.6.1—Example of a two-piece stirrup that complies with the requirements of 25.7.1.6.1.

25.7.1.7 Except where used for torsion, integrity reinforcement, or at beam ends in intermediate or special moment frames, closed stirrups are permitted to be made using pairs of U-stirrups spliced to form a closed unit where lap lengths are at least $1.3\ell_d$. In members with a total depth of at least 18 in., such splices with $A_bf_{yt} \leq 9000$ lb per leg shall be considered adequate if stirrup legs extend the full available depth of member.

R25.7.1.7 Requirements for lapping of double U-stirrups to form closed stirrups control over the lap splice provisions of 25.5.2. Figure R25.7.1.7 illustrates closed stirrup configurations created with lap splices.

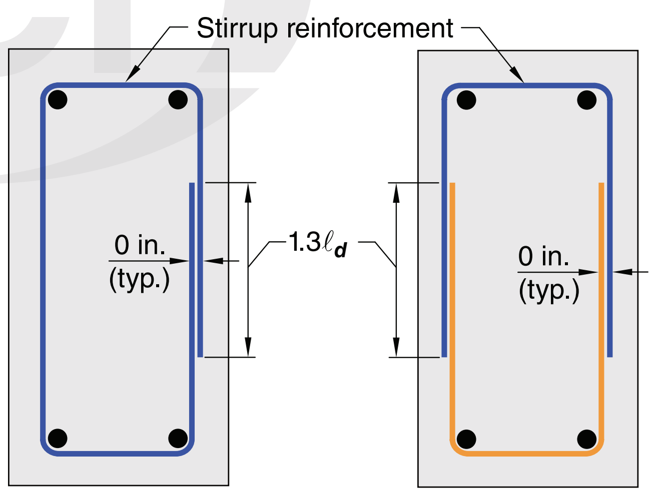

Fig. R25.7.1.7—Closed stirrup configurations.

25.7.1.8 *Stirrups anchored with a head*

25.7.1.8.1 Stirrups anchored with a head conforming to 20.2.1.6 shall be permitted if 25.7.1.8.2 and 25.7.1.8.3 are satisfied, except as prohibited by 25.7.1.8.4.

25.7.1.8.2 If stirrups are anchored with a head, (a) through (c) shall be satisfied:

R25.7.1.8 *Stirrups anchored with a head*

R25.7.1.8.1 Tests of beams have demonstrated that heads can be an effective alternative to standard hooks for anchoring shear reinforcement when detailed in accordance with these provisions (Al-Sabawy et al. 2020; Yang et al. 2021).

These provisions do not address headed shear stud reinforcement conforming to 20.4.

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(a) Stirrup shall be fabricated from a deformed bar conforming to 20.2.1.3
(b) The deformed bar shall be a No. 9 or smaller, except in foundations with overall depth h greater than 48 in., where the deformed bar shall be a No. 11 or smaller
(c) Concrete shall be normalweight

25.7.1.8.3 Stirrups anchored with a head shall be secured to prevent movement during concrete placement and arranged to satisfy (a) through (e):

(a) Clear concrete side cover to a stirrup leg terminating with a head shall be at least $8d_b$, where d_b is the nominal diameter of the stirrup
(b) At least one longitudinal bar shall be located between the side of a stirrup leg terminating with a head and any surface of the member parallel to the stirrup leg
(c) Except in foundations with overall depth h greater than 48 in., the end of each stirrup leg terminating with a head shall be in contact with a longitudinal bar
(d) Center-to-center spacing of stirrup legs terminating with a head shall be at least $6d_b$, where d_b is the nominal diameter of the stirrup
(e) The embedment length between midheight of the member and the bearing face of the head shall be at least $0.014d_bf_{yt}/\sqrt{f_c'}$.

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R25.7.1.8.3 To avoid side-face blowout failures, a minimum side cover of $8d_b$ is required for stirrup legs terminating with a head, and at least one longitudinal bar is required within that side cover. These requirements can be satisfied when headed deformed bars are used as through-thickness reinforcement in slabs or foundations or in combination with hooked stirrups in beams (Fig. R25.7.1.8.3). If inclined crack widths are a serviceability concern, stirrup legs anchored with hooks should be placed along the side faces of the member (Fig. R25.7.1.8.3(c)).

While contact with longitudinal reinforcement improves anchorage, physical limitations may prevent the bottom of stirrup legs terminating with a head from being secured to a longitudinal bar in foundations with overall depths h greater than 48 in.

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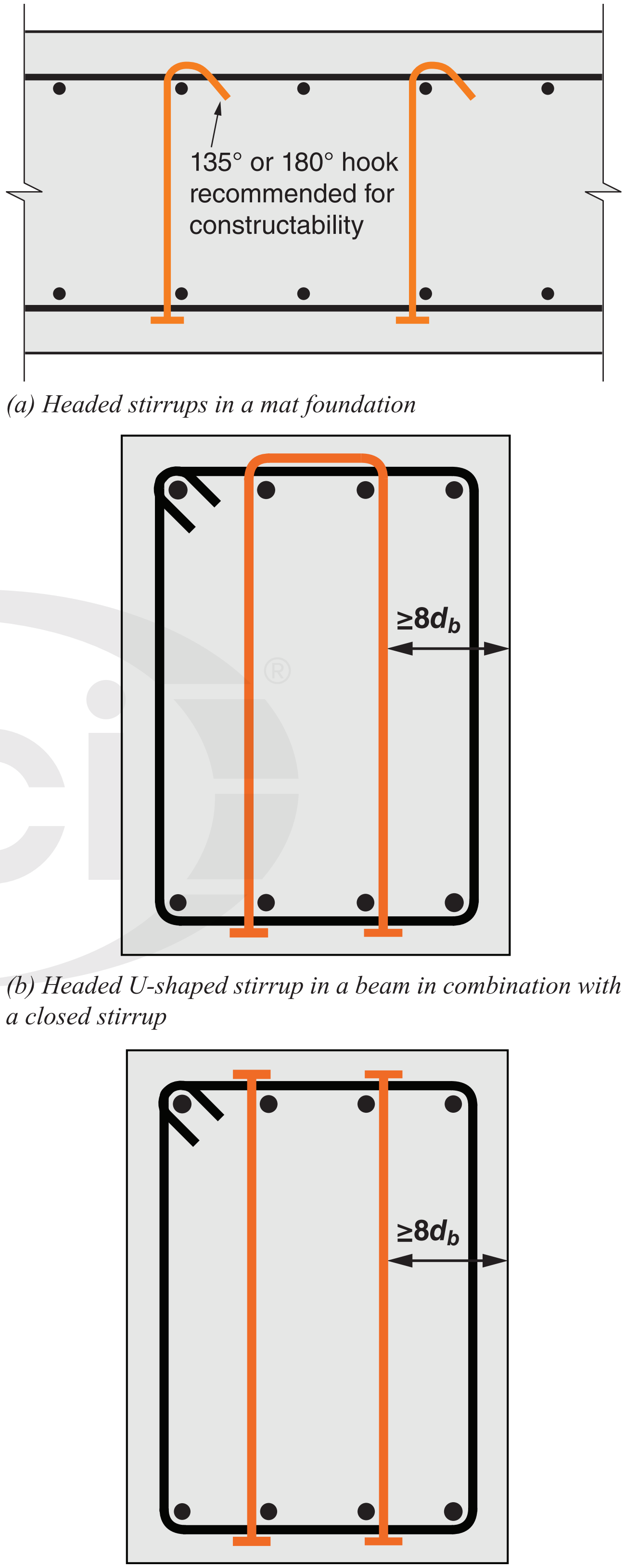

(a) Headed stirrups in a mat foundation

(b) Headed U-shaped stirrup in a beam in combination with a closed stirrup

(c) Headed single-leg stirrups in a beam in combination with a closed stirrup

Fig. R25.7.1.8.3—Permissible headed shear reinforcement details.

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25.7.1.8.4 Stirrups anchored with a head are prohibited in applications (a) through (e):

(a) An alternative to, or component of, closed stirrups, ties, or hoops
(b) Torsion reinforcement
(c) Integrity reinforcement
(d) Confining reinforcement for hooked bars or parallel ties for headed bars
(e) Confining reinforcement for straight bar development or lap splices

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R25.7.1.8.4 Headed stirrups are prohibited in applications that require the bars to be continuous around or near the perimeter of a cross section, including closed stirrups, ties, and hoops; stirrups used to confine bars being developed; or stirrups used for torsion or integrity reinforcement. If a closed stirrup has interior legs that are used to resist shear, the interior legs are permitted to terminate with a head (Fig. R25.7.1.8.3(c)).

25.7.2 *Ties*

R25.7.2 *Ties*

25.7.2.1 Ties shall consist of a closed loop of deformed bar with spacing in accordance with (a) and (b):

(a) Clear spacing of at least $(4/3)d_{agg}$
(b) Center-to-center spacing shall not exceed the least of $16d_b$ of longitudinal bar, $48d_b$ of tie bar, and smallest dimension of member

25.7.2.2 Diameter of tie bar shall be at least (a) or (b):

(a) No. 3 enclosing No. 10 or smaller longitudinal bars
(b) No. 4 enclosing No. 11 or larger longitudinal bars or bundled longitudinal bars

R25.7.2.2 These provisions apply to crossties as well as ties.

25.7.2.2.1 As an alternative to deformed bars, deformed wire or welded wire reinforcement of equivalent area to that required in 25.7.2.1 shall be permitted subject to the requirements of Table 20.2.2.4(a).

25.7.2.3 Rectilinear ties shall be arranged to satisfy (a) and (b):

(a) Every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degrees
(b) No unsupported bar shall be farther than 6 in. clear on each side along the tie from a laterally supported bar

R25.7.2.3 The maximum permissible included angle of 135 degrees and the exemption of bars located within 6 in. clear on each side along the tie from adequately tied bars are illustrated in Fig. R25.7.2.3a. Limited tests (Pfister 1964) on full-size, axially-loaded, tied columns containing full-length bars (without splices) showed that ties on alternate longitudinal bars within 6 in. clear of a laterally supported longitudinal bar are adequate in columns subjected to axial force.

Continuously wound bars or wires can be considered as ties, provided their pitch and area are at least equivalent to the area and spacing of separate ties. Anchorage at the end of a continuously wound bar or wire should be by a standard hook as for separate bars or by one additional turn of the tie pattern (refer to Fig. R25.7.2.3b). A circular, continuously wound bar or wire is considered a spiral if it conforms to 25.7.3; otherwise, it is considered a tie.

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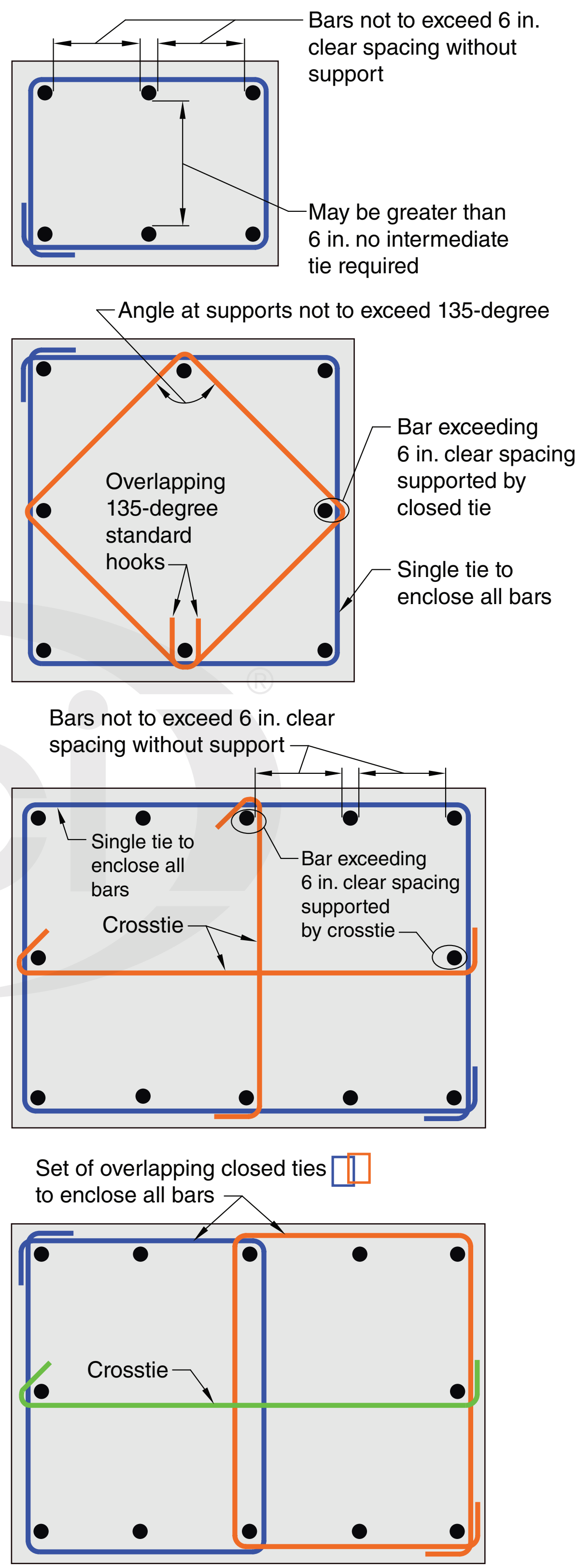

Fig. R25.7.2.3a—Illustrations to clarify measurements between laterally supported column bars and rectilinear tie anchorage.

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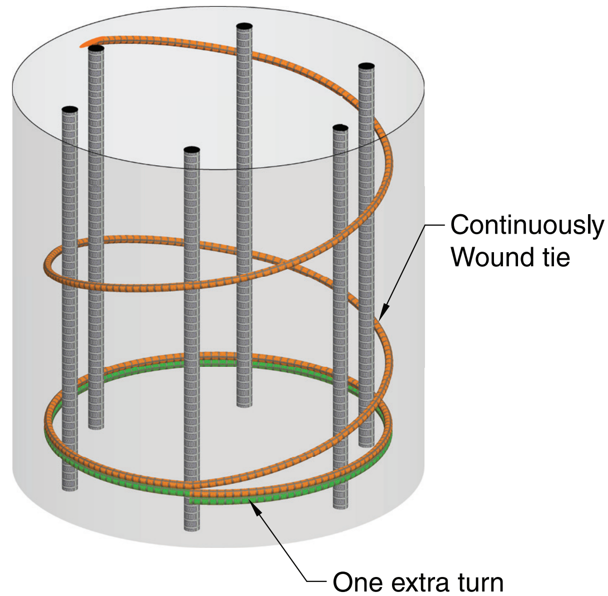

Fig. R25.7.2.3b—Continuous tie anchorage.

25.7.2.3.1 Anchorage of rectilinear ties shall be provided by standard hooks that conform to 25.3.2 and engage a longitudinal bar. A tie shall not be made up of interlocking headed deformed bars.

R25.7.2.3.1 Standard tie hooks are intended for use with deformed bars only and should be staggered where possible.

25.7.2.4 Circular ties shall be permitted where longitudinal bars are located around the perimeter of a circle.

R25.7.2.4 While the transverse reinforcement in members with longitudinal bars located around the periphery of a circle can be either spirals or circular ties, spirals are usually more effective.

25.7.2.4.1 Anchorage of individual circular ties shall be in accordance with (a) through (c):

(a) Ends shall overlap by at least 6 in.
(b) Ends shall terminate with standard hooks in accordance with 25.3.2 that engage a longitudinal bar
(c) Overlaps at ends of adjacent circular ties shall be staggered around the perimeter enclosing the longitudinal bars

R25.7.2.4.1 Vertical splitting and loss of tie restraint are possible where the overlapped ends of adjacent circular ties are anchored at a single longitudinal bar. Adjacent circular ties should not engage the same longitudinal bar with end hook anchorages (refer to Fig. R25.7.2.4.1).

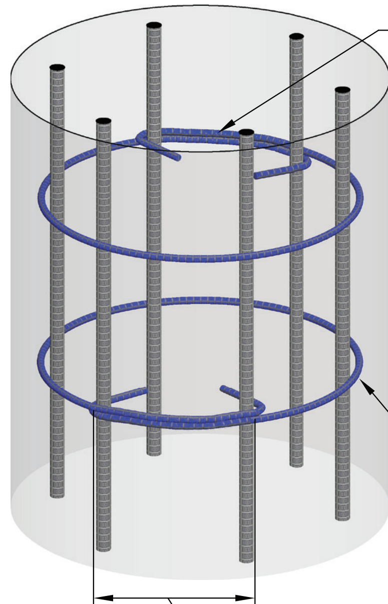

Fig. R25.7.2.4.1—Circular tie anchorage.

25.7.2.5 Ties to resist torsion shall be perpendicular to the axis of the member anchored by either (a) or (b):

(a) Ends shall terminate with 135-degree standard hooks or seismic hooks around a longitudinal bar
(b) In accordance with 25.7.1.3(a) or (b) or 25.7.1.4, where the concrete surrounding the anchorage is restrained against spalling

R25.7.2.5 Refer to R25.7.1.6.

25.7.3 *Spirals*

R25.7.3 *Spirals*

25.7.3.1 Spirals shall consist of evenly spaced continuous bar or wire with clear spacing conforming to (a) and (b):

(a) At least the greater of 1 in. and $(4/3)d_{agg}$
(b) Not greater than 3 in.

R25.7.3.1 Spirals should be held firmly in place, at proper pitch and alignment, to prevent displacement during concrete placement.

25.7.3.2 For cast-in-place construction, spiral bar or wire diameter shall be at least 3/8 in.

R25.7.3.2 For practical considerations in cast-in-place construction, the minimum diameter of spiral reinforcement is 3/8 in. (No. 3 deformed or plain bar, or D11 deformed or W11 plain wire).

Standard spiral sizes are 3/8, 1/2, and 5/8 in. diameter for hot-rolled or cold-drawn material, plain or deformed.

25.7.3.3 Except for transverse reinforcement in deep foundations, the volumetric spiral reinforcement ratio ρ_s shall satisfy Eq. (25.7.3.3)

$$\rho_s \geq 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yt}} \qquad (25.7.3.3)$$

where the value of f_{yt} shall not be taken greater than 100,000 psi.

R25.7.3.3 The effect of spiral reinforcement in increasing the strength of the concrete within the core is not fully realized until the column has been subjected to a load and deformation sufficient to cause the concrete shell outside the core to spall off. The amount of spiral reinforcement required by Eq. (25.7.3.3) is intended to provide additional strength for concentrically loaded columns equal to or slightly greater than the strength lost when the shell spalls off. The deriva-

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tion of Eq. (25.7.3.3) is given by Richart (1933). Tests and experience show that columns containing the amount of spiral reinforcement required by this section exhibit considerable toughness and ductility. Research (Richart et al. 1929; Richart 1933; Pessiki et al. 2001; Saatcioglu and Razvi 2002) has also indicated that up to 100,000 psi yield strength reinforcement can be effectively used for confinement.

25.7.3.4 Spirals shall be anchored by 1-1/2 extra turns of spiral bar or wire at each end.

R25.7.3.4 Spiral anchorage is illustrated in Fig. R25.7.3.4.

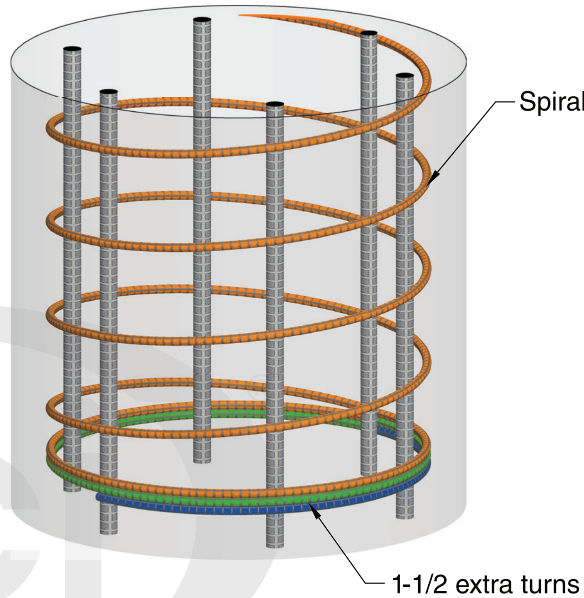

Fig. R25.7.3.4—Spiral anchorage.

25.7.3.5 Spirals are permitted to be spliced by (a) or (b):

(a) Mechanical or welded splices in accordance with 25.5.7

(b) Lap splices in accordance with 25.7.3.6 for f_{yt} not exceeding 60,000 psi

25.7.3.6 Spiral lap splices shall be at least the greater of 12 in. and the lap length in Table 25.7.3.6.

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Table 25.7.3.6—Lap length for spiral reinforcement

Reinforcement	Coating	Ends of lapped spiral bar or wire	Lap length, in.
Deformed bar	Uncoated or zinc-coated (galvanized)	Hook not required	$48d_b$
	Epoxy-coated or zinc and epoxy dual-coated	Hook not required	$72d_b$
		Standard hook of 25.3.2[1]	$48d_b$
Deformed wire	Uncoated	Hook not required	$48d_b$
	Epoxy-coated	Hook not required	$72d_b$
		Standard hook of 25.3.2[1]	$48d_b$
Plain bar	Uncoated or zinc-coated (galvanized)	Hook not required	$72d_b$
		Standard hook of 25.3.2[1]	$48d_b$
Plain wire	Uncoated	Hook not required	$72d_b$
		Standard hook of 25.3.2[1]	$48d_b$

[1]Hooks shall be embedded within the core confined by the spiral.

25.7.4 *Hoops*

25.7.4.1 Hoops shall consist of a continuous closed tie or continuously wound tie having seismic hooks at both ends that conform to 25.3.4. A hoop shall not be made up of interlocking headed deformed bars.

25.7.4.2 It shall be permitted to overlap hoops.

25.8—Post-tensioning anchorages and couplers

25.8.1 Anchorages and couplers for tendons shall develop at least 95% of f_{pu} when tested in an unbonded condition, without exceeding anticipated set.

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R25.7.4 *Hoops*

R25.7.4.1 Refer to R25.7.2.4.

R25.7.4.2 Overlapping hoops may be required in boundary elements of special structural walls depending on the longitudinal reinforcement layout and the boundary element dimensions (refer to 18.10.6.4f).

R25.8—Post-tensioning anchorages and couplers

R25.8.1 The required strength of the tendon-anchorage or tendon-coupler assemblies for both unbonded and bonded tendons, when tested in an unbonded state, is based on 95% of the specified tensile strength of the prestressing reinforcement in the test. The prestressing reinforcement is required to comply with the minimum provisions of the applicable ASTM standards as prescribed in 20.3.1. The specified strength of anchorages and couplers exceeds the maximum design strength of the prestressing reinforcement by a substantial margin and, at the same time, recognizes the stress-riser effects associated with most available post-tensioning anchorages and couplers. Anchorage and coupler strength should be attained with a minimum amount of permanent deformation and successive set, recognizing that some deformation and set will occur when testing to failure.

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25.8.2 Anchorages and couplers for bonded tendons shall be located so that 100% of f_{pu} shall be developed at critical sections after the post-tensioned reinforcement is bonded in the member.

25.8.3 In unbonded construction subject to repetitive loads, the possibility of fatigue of prestressed reinforcement in anchorages and couplers shall be considered.

25.8.4 Couplers shall be placed at locations approved by the licensed design professional and enclosed in housings long enough to permit necessary movements.

25.9—Anchorage zones for post-tensioned tendons

25.9.1 *General*

25.9.1.1 Anchorage regions of post-tensioned tendons shall consist of two zones, (a) and (b):

(a) The local zone shall be assumed to be a rectangular prism (or equivalent rectangular prism for circular or oval anchorages) of concrete immediately surrounding the anchorage device and any confining reinforcement

(b) The general zone includes the local zone and shall be assumed to be the portion of the member through which the concentrated prestressing force is transferred to the concrete and distributed more uniformly across the section

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Tendon assemblies should conform to the 2 percent elongation requirements in ACI SPEC-423.7.

Static and fatigue test methods for anchorage and couplers are provided in ICC-ES Acceptance Criteria AC303 (2011).

R25.8.2 Anchorages and couplers for bonded tendons that develop less than 100% of the specified tensile strength of the prestressing reinforcement should be used only where the bond transfer length between the anchorage or coupler and critical sections equals or exceeds that required to develop the prestressed reinforcement strength. This bond length may be calculated based on the results of tests of bond characteristics of non-tensioned prestressing strand (Salmons and McCrate 1977; PCA 1980), or bond tests on other prestressing reinforcement, as appropriate.

R25.8.3 A discussion on fatigue loading is provided in ACI PRC-215.

Detailed recommendations on tests for static and cyclic loading conditions for tendons and anchorage fittings of unbonded tendons are provided in ACI PRC-423.3-17 (Section 4.1.3) and ACI SPEC-301-20 (Section 15.2.2).

R25.9—Anchorage zones for post-tensioned tendons

R25.9.1 *General*

The detailed provisions in the AASHTO LRFD Bridge Design Specifications (AASHTO LRFDUS) for analysis and reinforcement detailing of post-tensioned anchorage zones are considered to satisfy the more general requirements of this Code. In the specific areas of anchorage device evaluation and acceptance testing, this Code references the detailed AASHTO provisions.

R25.9.1.1 Based on St. Venant's principle, the extent of the anchorage zone may be estimated as approximately equal to the largest dimension of the cross section. Local zones and general zones are shown in Fig. R25.9.1.1a.

When anchorage devices located away from the end of the member are tensioned, large local tensile stresses are generated ahead of and behind the device. These tensile stresses are induced by incompatibility of deformations. The entire shaded region shown in Fig. R25.9.1.1b should be considered in the design of the general zone.

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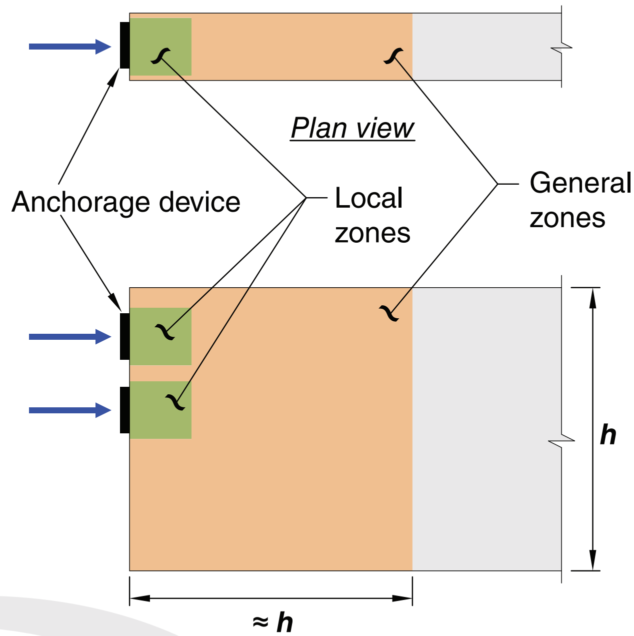

Fig. R25.9.1.1a—Local and general zones.

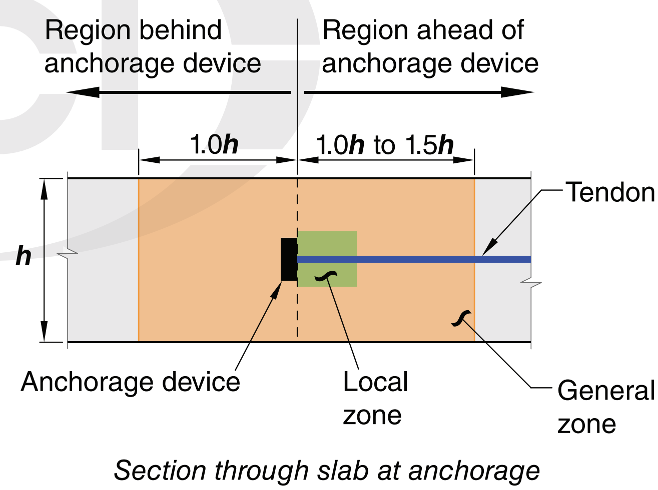

Fig. R25.9.1.1b—Local and general zones for anchorage device located away from the end of a member.

25.9.1.2 The local zone shall be designed in accordance with 25.9.3.

25.9.1.3 The general zone shall be designed in accordance with 25.9.4.

25.9.1.4 Compressive strength of concrete required at time of post-tensioning shall be specified as required by 26.10.

25.9.1.5 Stressing sequence shall be considered in the design process and specified as required by 26.10.

R25.9.1.5 The sequence of anchorage device stressing can have a significant effect on the general zone stresses. Therefore, it is important to consider not only the final stage

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25.9.2 *Required strength*

25.9.2.1 Factored prestressing force at the anchorage device, P_{pu}, shall exceed the least of (a) through (c), where 1.2 is the load factor from 5.3.15:

(a) $1.2(0.94f_{py})A_{ps}$
(b) $1.2(0.80f_{pu})A_{ps}$
(c) Maximum jacking force designated by the supplier of anchorage devices multiplied by 1.2

25.9.3 *Local zone*

25.9.3.1 The design of local zone in post-tensioned anchorages shall meet the requirements of (a), (b), or (c):

(a) Monostrand or single 5/8 in. or smaller diameter bar anchorage devices shall meet the bearing resistance and local zone requirements of ACI SPEC-423.7
(b) Basic multistrand anchorage devices shall meet the bearing resistance requirements of AASHTO LRFD Bridge Design Specifications, Article 5.8.4.4.2, except that the load factors shall be in accordance with 5.3.15 and ϕ shall be in accordance with 21.2.1
(c) Special anchorage devices shall satisfy the tests required in AASHTO LRFD Bridge Design Specifications, Article 5.8.4.4.3, and described in AASHTO LRFD Bridge Construction Specifications, Article 10.3.2.3

25.9.3.2 Where special anchorage devices are used, supplementary skin reinforcement shall be provided in

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of a stressing sequence with all tendons stressed, but also intermediate stages during construction. The most critical bursting forces caused by each of the sequentially post-tensioned tendon combinations, as well as that of the entire group of tendons, should be taken into account.

R25.9.2 *Required strength*

R25.9.2.1 The factored prestressing force is the product of the load factor and the maximum prestressing force permitted. The maximum permissible tensile stresses during jacking are defined in 20.3.2.5.1.

R25.9.3 *Local zone*

The local zone resists very high local stresses introduced by the anchorage device and transfers them to the remainder of the anchorage zone. The behavior of the local zone is strongly influenced by the specific characteristics of the anchorage device and its confining reinforcement, and is less influenced by the geometry and loading of the overall structure. Local-zone design sometimes cannot be completed until specific anchorage devices are selected. If special anchorage devices are used, the anchorage device supplier should furnish test information to demonstrate that the device is satisfactory under Article 10.3.2.3 of the AASHTO LRFD Bridge Construction Specifications (LRFDCONS) and provide information regarding necessary conditions for use of the device. The main considerations in local-zone design are the effects of high bearing pressure and the adequacy of any confining reinforcement provided to increase concrete bearing resistance.

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addition to the confining reinforcement specified for the anchorage device.

25.9.3.2.1 Supplementary skin reinforcement shall be similar in configuration and at least equivalent in volumetric ratio to any supplementary skin reinforcement used in the qualifying acceptance tests of the anchorage device.

25.9.4 *General zone*

25.9.4.1 The extent of the general zone is equal to the largest dimension of the cross section. In the case of slabs with anchorages or groups of anchorages spaced along the slab edge, the depth of the general zone shall be taken as the spacing of the tendons.

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R25.9.3.2.1 Skin reinforcement is placed near the outer faces in the anchorage zone to limit local crack width and spacing. Reinforcement in the general zone for other actions (such as shrinkage and temperature) may be used in satisfying the supplementary skin reinforcement requirement. Determination of the supplementary skin reinforcement depends on the anchorage device hardware used and frequently cannot be determined until the specific anchorage devices are selected.

R25.9.4 *General zone*

Within the general zone, the assumption that plane sections remain plane is not valid. Tensile stresses that can be caused by the tendon anchorage device, including bursting, spalling, and edge tension, as shown in Fig. R25.9.4, should be considered in design. In addition, the compressive stresses immediately ahead of the local zone should be checked (Fig. R25.9.1.1b).

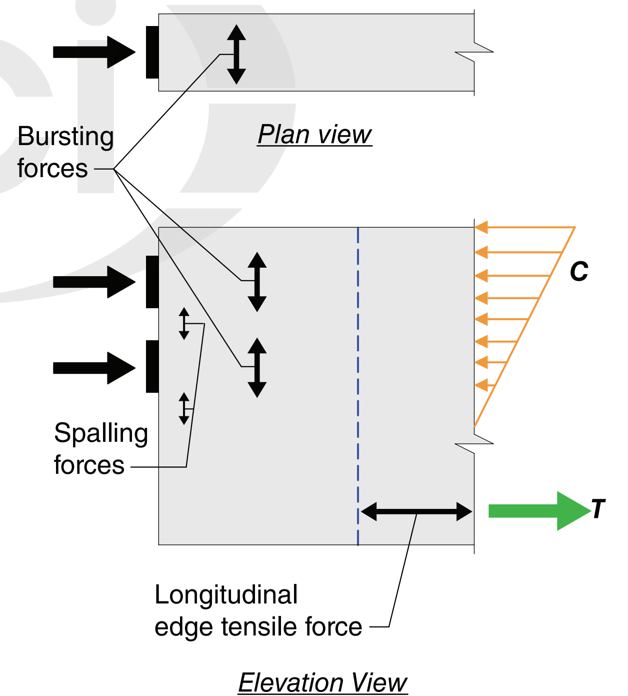

Fig. R25.9.4—Tensile stress zones within the general zone.

R25.9.4.1 The depth of the general zone in slabs is defined in AASHTO LRFD Bridge Design Specifications (LRFDUS), Article 5.9.5.6 as the spacing of the tendons (Fig. R25.9.4.1). Refer to 25.9.4.4.6 for monostrand anchorages.

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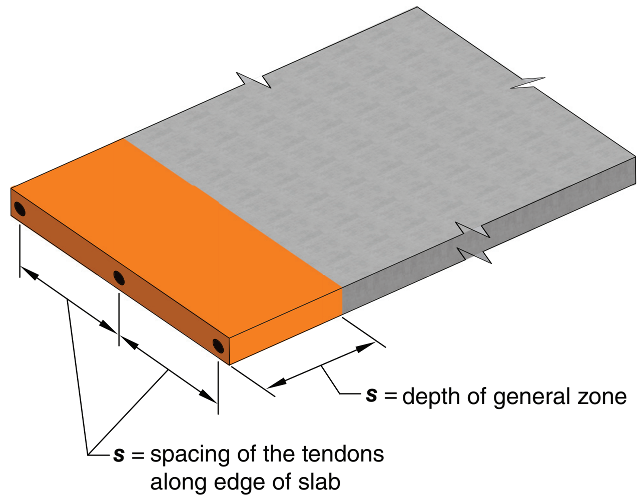

Fig. R25.9.4.1—Dimensions of general zone in post-tensioned slab.

25.9.4.2 For anchorage devices located away from the end of a member, the general zone shall include the disturbed regions ahead of and behind the anchorage devices.

R25.9.4.2 The dimensions of the general zone for anchorage devices located away from the end of the member are defined in Fig. R25.9.1.1b.

25.9.4.3 *Analysis of general zones*

25.9.4.3.1 Methods (a) through (c) shall be permitted for design of general zones:

(a) The strut-and-tie method in accordance with Chapter 23
(b) Linear stress analysis, including finite element analysis or equivalent
(c) Simplified equations in AASHTO LRFD Bridge Design Specifications, Article 5.8.4.5, except where restricted by 25.9.4.3.2

The design of general zones by other methods shall be permitted, provided that the specific procedures used for design result in prediction of strength in substantial agreement with results of comprehensive tests.

R25.9.4.3 *Analysis of general zones*

R25.9.4.3.1 The design methods include those procedures for which guidelines have been given in AASHTO LRFDUS and Breen et al. (1994). These procedures have been shown to be conservative predictors of strength compared to test results (Breen et al. 1994). The use of the strut-and-tie method is especially helpful for general zone design (Breen et al. 1994). In many anchorage applications, where substantial or massive concrete regions surround the anchorages, simplified equations based on AASHTO LRFDUS and Breen et al. (1994) can be used except in the cases noted in 25.9.4.3.2.

Values for the magnitude of the bursting force, T_{burst}, and for its centroidal distance from the major bearing surface of the anchorage, d_{burst}, may be estimated from Eq. (R25.9.4.3.1a) and (R25.9.4.3.1b), respectively. The terms used in these equations are shown in Fig. R25.9.4.3.1 for a prestressing force with a small eccentricity. In the application of these equations, the specified stressing sequence should be considered if more than one tendon is present.

$$T_{burst} = 0.25\sum P_{pu}\left(1 - \frac{h_{anc}}{h}\right) \quad \text{(R25.9.4.3.1a)}$$

$$d_{burst} = 0.5(h - 2e_{anc}) \quad \text{(R25.9.4.3.1b)}$$

where $\sum P_{pu}$ is the sum of the P_{pu} forces from the individual tendons; h_{anc} is the depth of the anchorage device or single group of closely spaced devices in the direction considered; and e_{anc} is the eccentricity (always taken as positive) of the

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25.9.4.3.2 Simplified equations as permitted by 25.9.4.3.1(c) shall not be used for the design of a general zone if any of the situations listed in (a) through (g) occur:

(a) Member cross sections are nonrectangular
(b) Discontinuities in or near the general zone cause deviations in the force flow path
(c) Minimum edge distance is less than 1.5 times the anchorage device lateral dimension in that direction
(d) Multiple anchorage devices are used in other than one closely spaced group
(e) Centroid of the tendons is located outside the kern
(f) Angle of inclination of the tendon in the general zone is less than –5 degrees from the centerline of axis of the member, where the angle is negative if the anchor force points away from the centroid of the section
(g) Angle of inclination of the tendon in the general zone is greater than +20 degrees from the centerline of axis of the member, where the angle is positive if the anchor force points towards the centroid of the section

25.9.4.3.3 Three-dimensional effects shall be considered in design and analyzed by (a) or (b):

(a) Three-dimensional analysis procedures
(b) Approximated by considering the summation of effects for two orthogonal planes

25.9.4.4 *Reinforcement limits*

25.9.4.4.1 Tensile strength of concrete shall be neglected in calculations of reinforcement requirements.

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anchorage device or group of closely spaced devices with respect to the centroid of the cross section (Fig. R25.9.4.3.1).

Anchorage devices should be treated as closely spaced if their center-to-center spacing does not exceed 1.5 times the width of the anchorage device in the direction considered.

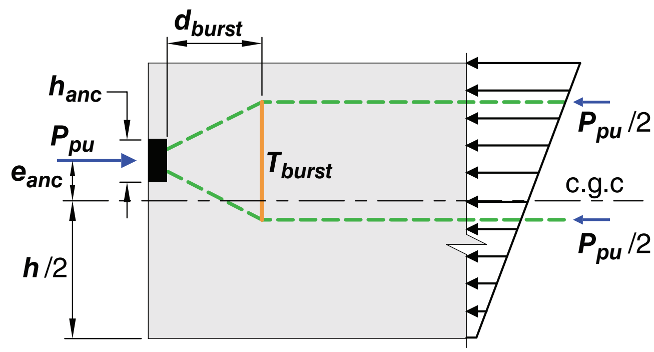

Fig. R25.9.4.3.1—Definition of terms used to define the general zone.

R25.9.4.3.2 The simplified equations in the AASHTO LRFDUS are not applicable in several common situations listed in 25.9.4.3.2. In these cases, a detailed analysis is required. In addition, in the post-tensioning of thin sections, flanged sections, or irregular sections, or where the tendons have appreciable curvature within the general zone, more general procedures such as those of AASHTO LRFDUS Articles 5.8.2.7 and 5.8.3 are required. Detailed recommendations for design principles that apply to all design methods are given in Article 5.9.5.6.5b of the AASHTO LRFDUS.

Groups of monostrand tendons with individual monostrand anchorage devices are often used in beams. If a beam has a single anchorage device or a single group of closely spaced anchorage devices, the use of simplified equations such as those given in R25.9.4.3.1 is permitted, unless 25.9.4.3.2 governs. More complex conditions can be designed using the strut-and-tie method. Detailed recommendations for use of such models are given in AASHTO LRFDUS and Breen et al. (1994).

R25.9.4.3.3 The provision for three-dimensional effects is to ensure that the effects perpendicular to the main plane of the member, such as bursting forces in the thin direction of webs or slabs are considered. In many cases, these effects can be determined independently for each direction, but some applications require a full three-dimensional analysis (for example, diaphragms for the anchorage of external tendons).

R25.9.4.4 *Reinforcement limits*

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25.9.4.4.2 Reinforcement shall be provided in the general zone to resist bursting, spalling, and longitudinal edge tension forces induced by anchorage devices, as applicable. Effects of abrupt changes in section and stressing sequence shall be considered.

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R25.9.4.4.2 In some cases, reinforcement requirements cannot be determined until specific tendon and anchorage device layouts are selected. Design and approval responsibilities should be clearly assigned in the construction documents.

Abrupt changes in section can cause substantial deviation in force paths. These deviations can greatly increase tensile forces, as shown in Fig. R25.9.4.4.2.

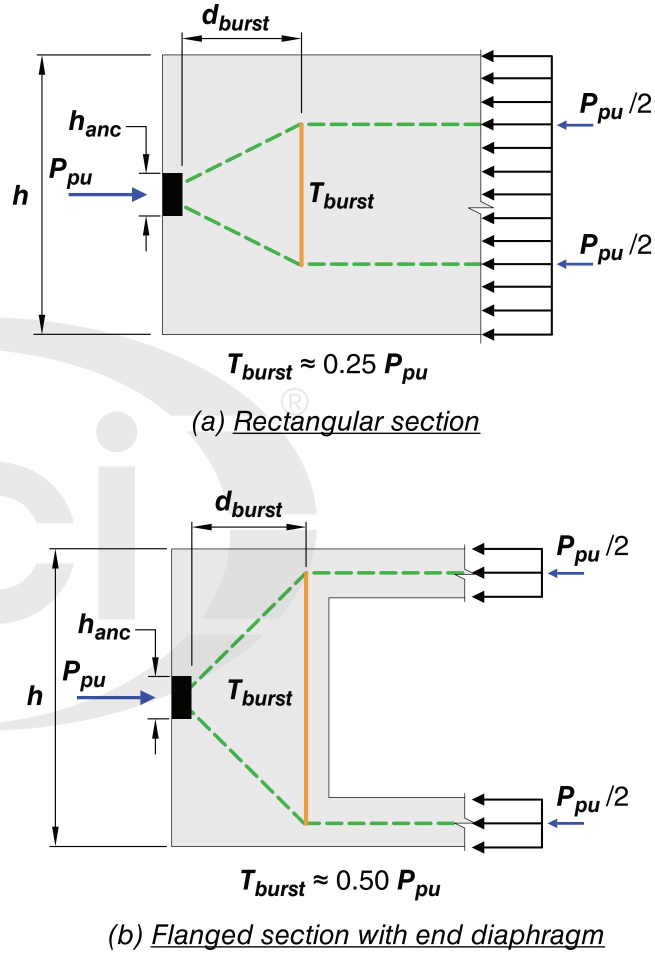

Fig. R25.9.4.4.2—Effect of cross section change.

25.9.4.4.3 For anchorage devices located away from the end of the member, bonded reinforcement shall be provided to transfer at least $0.35P_{pu}$ into the concrete section behind the anchor. Such reinforcement shall be placed symmetrically around the anchorage device and shall be developed both behind and ahead of the anchorage device.

R25.9.4.4.3 Where anchorages are located away from the end of a member, local tensile stresses are generated behind these anchorages (Fig. R25.9.1.1b) due to compatibility of deformations ahead of and behind the anchorages. Bonded tie-back reinforcement parallel to the tendon is required in the immediate vicinity of the anchorage to limit the extent of cracking behind the anchorage. The requirement of $0.35P_{pu}$ was derived using 25% of the unfactored prestressing force being resisted by reinforcement at $0.6f_y$ considering a load factor of 1.2. Therefore, the full yield strength of the reinforcement, f_y, should be used in calculating the provided capacity.

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25.9.4.4.4 If tendons are curved in the general zone, bonded reinforcement shall be provided to resist radial and splitting forces, except for monostrand tendons in slabs or where analysis shows reinforcement is not required.

25.9.4.4.5 Reinforcement with a nominal tensile strength equal to 2 percent of the factored prestressing force shall be provided in orthogonal directions parallel to the loaded face of the anchorage zone to limit spalling, except for monostrand tendons in slabs or where analysis shows reinforcement is not required.

25.9.4.4.6 For monostrand anchorage devices for 1/2 in. or smaller diameter strands in normalweight concrete slabs, reinforcement satisfying (a) and (b) shall be provided in the anchorage zone, unless a detailed analysis in accordance with 25.9.4.3 shows that this reinforcement is not required:

(a) Two horizontal bars at least No. 4 in size shall be provided within the local zone parallel to the slab edge ahead of the bearing face of the anchorage device. They shall be permitted to be in contact with the bearing face of the anchorage device, the center of the bars shall be no farther than 4 in. ahead of the bearing face of the device, and the bars shall extend at least 6 in. either side of the outer edges of the device.
(b) If the center-to-center spacing of anchorage devices is 12 in. or less, the anchorage devices shall be considered as a group. For each group of six or more anchorage devices, at least $n + 1$ hairpin bars or closed stirrups at least No. 3 in size shall be provided, where n is the number of anchorage devices. One hairpin bar or stirrup shall be placed between adjacent anchorage devices and one on each side of the group. The hairpin bars or stirrups shall be placed with the horizontal legs extending into the slab perpendicular to the edge. The center line of the vertical leg of the hairpin bars, or the vertical leg of stirrups closest to the anchorage device, shall be placed $3h/8$ to $h/2$ ahead of the bearing face of the anchorage device. Hairpin bars or stirrups shall be detailed in accordance with 25.7.1.1 and 25.7.1.2.

COMMENTARY

R25.9.4.4.5 The spalling force for tendons for which the centroid lies within the kern of the section may be estimated as 2 percent of the total factored prestressing force, except for multiple anchorage devices with center-to-center spacing greater than 0.4 times the depth of the section.

R25.9.4.4.6 For monostrand slab tendons, the anchorage-zone minimum reinforcement requirements are based on the recommendations of Breen et al. (1994) and confirmed based on analysis of other test results by Roberts-Wollmann and Wollmann (2008). Typical details are shown in Fig. R25.9.4.4.6. For slabs not thicker than 8 in., with groups of anchors requiring hairpins, the bars parallel to the loaded face can satisfy 25.9.4.4.6(a) and also provide anchorage for the hairpin bars. Thicker slabs require two bars for 25.9.4.4.6 (a) and two additional bars to provide anchorage for the hairpins in accordance with 25.7.1.2. The horizontal bars parallel to the edge required by 25.9.4.4.6(a) should be continuous where possible.

The tests on which the recommendations of Breen et al. (1994) were based were limited to anchorage devices for 1/2 in. diameter, Grade 270 strand, and unbonded tendons in normalweight concrete. For larger strand anchorage devices or for use in lightweight concrete slabs, ACI Committee 423 recommends that the amount and spacing of reinforcement should be conservatively adjusted to provide for the larger anchorage force and smaller splitting tensile strength of lightweight concrete (ACI PRC-423.3).

ACI PRC-423.3 and Breen et al. (1994) both recommend that hairpin bars also be furnished for anchorages located within 12 in. of slab corners to resist edge tension forces. The meaning of “ahead of” in 25.9.4.4.6 is illustrated in Fig. R25.9.1.1b.

In those cases where multistrand anchorage devices are used for slab tendons, all provisions of 25.9.4 are to be satisfied.

CODE | **COMMENTARY**

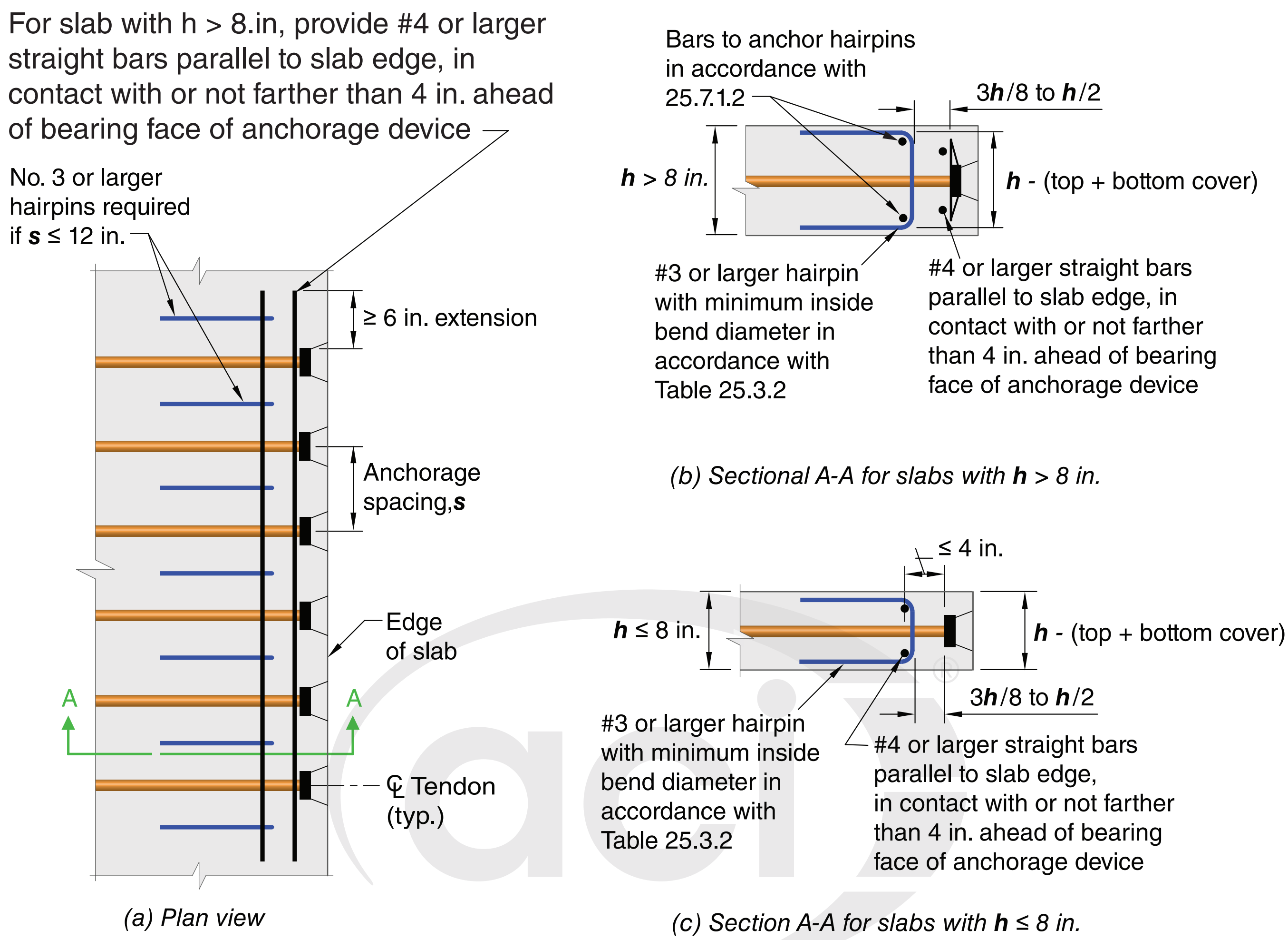

Fig. R25.9.4.4.6—Anchorage zone reinforcement for groups of 1/2 in. or smaller diameter tendons in slabs (other reinforcement not shown).

25.9.4.5 *Limiting stresses in general zones*

25.9.4.5.1 Maximum design tensile stress in reinforcement at nominal strength shall not exceed the limits in Table 25.9.4.5.1.

Table 25.9.4.5.1—Maximum design tensile stress in reinforcement

Type of reinforcement	Maximum design tensile stress
Nonprestressed reinforcement	f_y
Bonded, prestressed reinforcement	f_{py}
Unbonded, prestressed reinforcement	f_{se} + 10,000

25.9.4.5.2 Compressive stress in concrete at nominal strength shall not exceed $0.7\lambda f_{ci}'$, where λ is defined in 19.2.4.

R25.9.4.5 *Limiting stresses in general zones*

R25.9.4.5.1 The value for maximum design tensile stress of bonded prestressed reinforcement is limited to the yield strength of the prestressing reinforcement because Eq. (20.3.2.3.1) may not apply to these nonflexural applications. The value for unbonded prestressed reinforcement is based on 20.3.2.4.1, but limited for these short-length, nonflexural applications.

R25.9.4.5.2 Some inelastic deformation of concrete within general zones is expected because anchorage zone design is based on a strength approach. Unless shown by tests, the λ factor for lightweight concrete should be applied to reflect a lower tensile strength, which is an indirect factor in limiting compressive stresses, as well as the wide scatter and brittleness exhibited in some lightweight concrete anchorage zone tests.

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25.9.4.5.3 If concrete is confined by spirals or hoops and the effect of confining reinforcement is documented by tests and analysis, it shall be permitted to use an increased value of compressive stress in concrete when calculating the nominal strength of the general zone.

25.9.4.5.4 Prestressing reinforcement shall not be stressed until compressive strength of concrete, as indicated by tests of cylinders cured in a manner consistent with curing of the member, is at least 2500 psi for single-strand or bar tendons or at least 4000 psi for multistrand tendons unless 25.9.4.5.5 is satisfied.

25.9.4.5.5 Provisions of 25.9.4.5.4 need not be satisfied if (a) or (b) is satisfied:

(a) Oversized anchorage devices are used to compensate for a lower concrete compressive strength
(b) Prestressing reinforcement is stressed to no more than 50% of the final prestressing force

25.9.5 *Reinforcement detailing*

25.9.5.1 Selection of reinforcement size, spacing, cover, and other details for anchorage zones shall make allowances for tolerances on fabrication and placement of reinforcement; for the size of aggregate; and for adequate placement and consolidation of the concrete.

COMMENTARY

R25.9.4.5.3 For well-confined concrete, the effective compressive strength may be increased (Breen et al. 1994). Test results given in Breen et al. (1994) indicate that the compressive stress introduced by auxiliary prestressing applied perpendicular to the axis of the main tendons can be effective in increasing anchorage zone strength.

R25.9.4.5.4 To limit early shrinkage cracking, monostrand tendons are sometimes stressed at concrete strengths less than 2500 psi. In such cases, either oversized monostrand anchorages are used, or the strands are stressed in stages, often to levels one-third to one-half of the final prestressing force as permitted by 25.9.4.5.5.