Approved Changes to the Ninth Edition *AASHTO LRFD Bridge Design Specifications*: Reinforcing Bar Anchorage

by Dr. Oguzhan Bayrak, University of Texas at Austin

Current structural design codes in the United States treat the development of straight reinforcing bars, standard hooks, and deformed headed bars with separate equations and procedures. In contrast, the *fib Model Code for Concrete Structures 2010* (MC2010)[1] uses a unified approach. In their June 2022 meeting, the American Association of State Highway and Transportation Officials (AASHTO) Committee on Bridges and Structures approved an approach similar to the one used in MC2010 for inclusion in the forthcoming 10th edition of the *AASHTO LRFD Bridge Design Specifications*.

Unified Approach

According to the newly adopted reinforcement anchorage provisions, the modified tension development length ℓ_d (Eq. [1]) is to be taken as follows:

$$\ell_d = \ell_{db} \times (\lambda_{rl} \times \lambda_{cf} \times \lambda_{rc}) \qquad (1)$$

where

- ℓ_{db} = basic development length, in.
- $\lambda_{r\ell}$ = reinforcement location factor
- λ_{cf} = coating factor
- λ_{rc} = reinforcement confinement factor

These factors are described in Articles 5.10.8.2.1b and 5.10.8.2.1c of the AASHTO LRFD specifications[2] and in the following section.

$$\ell_{db} = 0.17 d_b \left(\frac{\lambda_{er} f_y - \frac{F_h}{A_b}}{1.97 \lambda f_c'^{0.25}} \right)^2 \qquad (2)$$

where

- d_b = nominal diameter of reinforcing bar or wire, in.
- λ_{er} = excess reinforcement factor
- f_y = specified minimum yield strength of reinforcement, ksi
- F_h = force developed by hooks or heads, kip
- A_b = nominal area of reinforcing bar or wire, in.^{2}
- λ = concrete density modification factor as specified in Article 5.4.2.8 of the AASHTO LRFD specifications[2]
- f_c' = compressive strength of concrete for use in design, ksi

The modified tension development length ℓ_d (Eq. [1]) is to be not less than the basic tension development length ℓ_{db} (Eq. [2]). The tension development length for nonheaded straight reinforcing bars and wires is to be not less than 12.0 in., except for development of shear reinforcement.

For nonheaded straight reinforcing bars and lap splices in tension, F_h is to be taken as zero. The basic development length in Eq. (2) is to be used for the straight portion of reinforcing bars developed in tension. It is primarily based on the partly cracked elastic model for bond behavior proposed by Tepfers.[3] The inclusion of the term F_h, which accounts for forces developed by hooks or heads, enables a unified approach to tension development length for straight bars, lap splices, hooked bars, and headed deformed bars. In the formula for ℓ_{db} (Eq. [2]), the denominator represents the effective splitting strength of concrete in bond, based on a fourth-root dependency on concrete strength.[4] The fourth-root dependency on the compressive strength of concrete for use in design has been shown to be more accurate for bond than the typical square-root dependency used in many tension strength approximations.[5,6]

Modification Factors

Modification factors are to be applied to the basic development length to account for the various effects specified in the AASHTO LRFD specifications. These factors are to be taken to be equal to 1.0, unless they are specified to either increase ℓ_d in Article 5.10.8.2.1b or decrease ℓ_d in Article 5.10.8.2.1c of the AASHTO LRFD specifications. The following parameters are to be used in calculating the development length of straight reinforcing bars.

- For horizontal reinforcement, placed such that more than 12.0 in. of fresh concrete is cast below the reinforcement, $\lambda_{r\ell} = 1.3$.
- For epoxy-coated or zinc and epoxy dual-coated bars with either cover less than $3d_b$ or clear spacing between bars less than $6d_b$, $\lambda_{cf} = 1.5$.
- For epoxy-coated or zinc and epoxy dual-coated bars not covered by the previous parameter, epoxy-coated or zinc and epoxy dual-coated hooked bars designed according to Article 5.10.8.2.4 of the AASHTO LRFD specifications, and epoxy-coated or zinc and epoxy dual-coated headed bars designed according to Article 5.10.8.2.7, $\lambda_{cf} = 1.2$.
- For uncoated or zinc-coated (galvanized) bars, $\lambda_{cf} = 1.0$.

For horizontal reinforcement, placed such that not more than 12.0 in. of concrete is cast below the reinforcement, no modification factor is necessary, in other words, $\lambda_{r\ell}$ is equal to 1.0. In the ninth edition of the AASHTO LRFD specifications, $\lambda_{r\ell}$ is not required for the development of hooked bars. The accumulation of bleed water underneath top-cast bars has been documented to reduce bond strength in nonheaded straight bars.[7] Equations (1) and (2), which are to be used to calculate the length of the straight portion of a hooked bar, are based on the behavior of a straight length of reinforcement subjected to uniform, average bond stresses. As a result, the influence of bar casting position is now required for hooked and headed bars. The proportion of anchorage resisted by the hook F_b is not affected by the reinforcement location factor.

The coating factor λ_{cf} is limited to no more than 1.2 for hooked and headed bars. The original coating factor for hooked bars was 1.2, based on tests by Hamad et al.[8] that did not satisfy the $3d_b$ clear cover and $6d_b$ clear spacing requirements. Therefore, it is unnecessary to use the larger value of 1.5. This limit also applies to headed bars based on recommendations in *Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (ACI 318R-19)*.[9]

The basic development length ℓ_{db} is to be modified by the previously discussed factors, as appropriate, and may be multiplied by the factors λ_{rc} and λ_{cr} as described in the following paragraphs.

For reinforcement being developed in the length under consideration, λ_{rc} shall satisfy the following:

$$0.3 \le \lambda_{rc} \le 1.0$$

and

$$\lambda_{rc} = \frac{d_b c_b}{\left[c_b\left(1-\beta_t\right)+1.67 n_s k_{tr}\right]^2} \quad (3)$$

where

- c_b = the smaller of the distance from the center of bar or wire being developed to the nearest concrete surface and one-half of the center-to-center spacing of the bars or wires being developed, in.
- β_t = ratio of unfactored compressive stress due to permanent loads (taken as a negative value) transverse to the plane of splitting to the modulus of rupture as determined by Article 5.4.2.6 of the AASHTO LRFD specifications ($-1 \le \beta_t \le 0$)
- n_s = modular ratio = E_s/E_c
- k_{tr} = transverse reinforcement index = $A_{tr}/(sn)$
- E_s = modulus of elasticity of reinforcing steel
- E_c = modulus of elasticity of concrete
- A_{tr} = total cross-sectional area of all transverse reinforcement that is within the spacing s and crosses the potential plane of splitting through the reinforcement being developed, in.[2]
- s = maximum center-to-center spacing of transverse reinforcement within ℓ_d, in.
- n = number of bars or wires developed along plane of splitting

In tests to determine development lengths, splitting cracks have been observed to occur along the bars being developed. When the center-to-center spacing of the bars is greater than about twice the distance from the center of the bar to the concrete surface, splitting cracks occur between the bars and the concrete surface (case 1 in **Fig. 1**). When the center-to-center spacing of the bars is less than about twice the distance from the center of the bar to the concrete surface, splitting cracks occur between the bars along the plane of the bars being developed (case 2 in Fig. 1). The presence of bars crossing the splitting plane, as denoted by A_{tr}, controls these splitting cracks and results in shorter development lengths. In any component, A_{tr} may be taken conservatively as zero.

The ratio of unfactored compressive stress due to permanent loads transverse to the plane of splitting to the effective concrete tension strength β_t accounts for the beneficial effects of clamping stresses that may exist perpendicular to the plane of splitting. Transverse compressive stresses act as confinement that delays the onset of splitting failures.[1] For example, concentrated stresses above or below a load/support can be assumed to enhance the bond strength over the length of reinforcement that passes through the load/support plate length.[1] For a new design, β_t can be conservatively taken as zero. While the concrete modulus of rupture determined according to Article 5.4.2.6. of the AASHTO LRFD specifications overestimates the concrete splitting strength, it is conservative for use in the calculation of β_t because overestimating the splitting strength reduces the calculated β_t.

Where anchorage or development for the full yield strength of reinforcement is not required, or where reinforcement in flexural components is in excess of that required by analysis:

$$\lambda_{cr} = \frac{\text{Required } A_s}{\text{Provided } A_s} \quad (4)$$

The excess reinforcement factor modifies the yield stress instead of the development length, based on the following relationship:

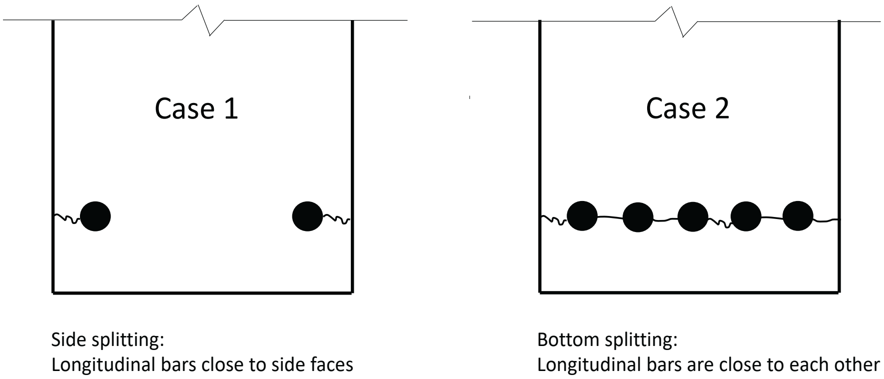
$f_s \times (\text{Provided } A_s) = f_y \times (\text{Required } A_s)$

Hooked Bars

New provisions for hooked bars were also included in this update. For hooks in reinforcing bars having a specified minimum yield strength greater than 75.0 ksi, ties satisfying the requirements of Article 5.10.8.2.4c shall be provided.

In the ninth edition of the AASHTO LRFD specifications, Article 5.10.8.2.4 was verified for concrete compressive strengths up to 15.0 ksi used in design based on the National Cooperative Highway Research Program (NCHRP) Report 603,[10] except for lightweight concrete. The previous limit of 10.0 ksi was retained for lightweight concrete. The updated procedure described herein has been validated over a similar range of concrete strengths and recommends

Figure 1. Cross section of beam subjected to tension showing the reinforcing bars being developed in tension with potential splitting-crack locations illustrated. Figure: Dr. Oguzhan Bayrak.



the same limits as Zaborac and Bayrak.[4] To improve the bond strength of no. 11 and larger reinforcing bars in tension terminating in a standard hook, NCHRP Report 603 recommends a minimum amount of transverse reinforcement consisting of no. 3 U-bars at $3d_b$ spacing in the anchorage length. Similar to the provisions of ACI 318-19, NCHRP Report 603 does not consider hooks to be effective in developing bars in compression.

The modified development length ℓ_{dh} in inches for deformed bars in tension terminating in a standard hook specified in Article 5.10.2.1 shall be taken according to Eq. (5), but shall not be less than the greater of 8.0 bar diameters and 6.0 in.

$$\ell_{dh} = \ell_d + R + \frac{d_b}{2} \qquad (5)$$

where

R = radius of the hook measured from the center of the bend to the centroid of the bar, in.

To calculate the modified development length of a standard hook in tension, the force developed by a standard hook F_h is to be calculated according to Eq. (6).

$$F_h = d_b R \nu f_c' \qquad (6)$$

where

ν = concrete efficiency factor for a compression-tension-tension (CTT) node specified in Table 5.8.2.5.3a-1 of the AASHTO LRFD specifications

Confinement of hooked bars by stirrups perpendicular and parallel to the bar being developed is illustrated in Fig. C5.10.8.2.4b-1 of the AASHTO LRFD specifications. The influence of parallel tie reinforcement is conservatively neglected in deriving the development length expressions for hooked bars; however, research has shown that parallel tie reinforcement is an effective method for improving hooked bar performance.[11] This provision does not apply for hooked bars at discontinuous ends of slabs where confinement is provided by the slab on both sides, perpendicular to the plane of the hook.

The concrete efficiency factor for a CTT node specified in Table 5.8.2.5.3a-1 is appropriate, regardless of the percentage of reinforcement confining the bar, due to low strain values associated with tensile failure of concrete around the bar being anchored and based on the experimental validation reported by Zaborac and Bayrak.[4]

There are no publicly available test results for no. 14 and no. 18 bars. As a result, the hooked bar development length calculations are limited to no. 11 bars and smaller based on experimental validation.[4] However, ACI 318-19 doubles the calculated hooked-bar development length for no. 14 and no. 18 bars, regardless of the provided confinement reinforcement and clear cover. Based on this ACI 318-19 guidance, the new commentary for the AASHTO LRFD specifications will recognize that it may be possible to develop a conservative estimate of the hooked-bar development length for no. 14 and no. 18 bars by doubling the value of ℓ_{dh}. Based on guidance from the Concrete Reinforcing Steel Institute, to address worker safety concerns, bars larger than no. 14 with Grade 75 or higher should not be bent.[12]

It is permissible to conservatively take k_{tr} as zero and reduce the calculated ℓ_{dh} by 20%, provided any of the detailing requirements in **Fig. 2** are met. However, for bars being developed by a standard hook at discontinuous ends of components with both side cover and top or bottom cover less than 2.5 in., the hooked bar shall be enclosed within ties or stirrups spaced not greater than $3d_b$ along the full development length ℓ_{dh} (Fig. 2). The transverse reinforcement index k_{tr} is to be taken as zero in this case, and ℓ_{dh} is not to be reduced.

For normalweight concrete with design concrete compressive strengths between 10.0 and 15.0 ksi, the development length of the hooked bars shall be enclosed with no. 3 bars or larger ties or stirrups along the full development ℓ_{dh} at a spacing not greater than $3d_b$. A minimum of three ties or stirrups shall be provided.

Headed Bars

The new provisions for headed bars are to be used for headed no. 11 bars or smaller with a net bearing area of head equal to or larger than four times the bar area, and in normalweight concrete with a design concrete compressive strength of up to 15.0 ksi or lightweight concrete up to 10.0 ksi. Headed bars designed with these provisions shall conform to ASTM A970/A970M[13] and satisfy requirements for Class HA head dimensions (Annex 1 of ASTM A970/A970M).

Headed deformed bars designed according to the new AASHTO LRFD specifications must have a clear cover to the heads not less than two times the bar diameter and a center-to-center spacing not less than three times the bar diameter. The head must be considered when satisfying minimum cover requirements for main reinforcing steel, as listed in Table 5.10.1-1 of the AASHTO LRFD specifications, and the minimum clear spacing requirements of Article 5.10.3.1.

The modified development length in inches for headed deformed bars in tension ℓ_{dt} shall be taken as $\ell_{dt} = \ell_d$ but shall not be less than the greater of 8.0 bar diameters and 6.0 in.

To calculate ℓ_{dt} of a headed deformed bar in tension, the force developed by a head F_h is to be calculated as:

$$F_h = 0.5 A_{ht} \nu f_c' \qquad (7)$$

where

A_{ht} = net area of the head that bears on concrete

ν = concrete efficiency factor for a CTT node specified in Table 5.8.2.5.3a-1

For headed no. 11 bars or smaller within beam-column joints, it is

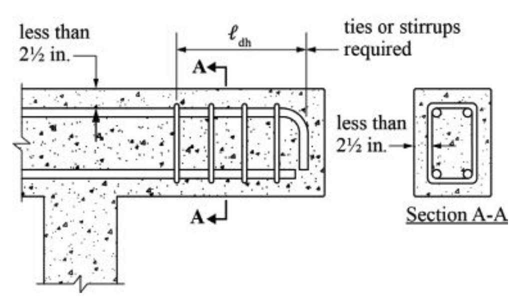


Figure 2. For bars being developed by a standard hook at discontinuous end of components with both side cover and top or bottom cover less than 2.5 in., the hooked bar shall be enclosed within ties of stirrups spaced along the development length ℓ_{dh}. The ties shall have a spacing not greater than $3d_b$. Source: Adapted from Fig. 5.10.8.2.4c-1 of the *AASHTO LRFD Bridge Design Specifications*, 9th ed.

permissible to reduce the development length by 20% if a total area of parallel tie reinforcement equal to 30% of the total area of headed bars is provided between the headed bar and the center of the beam-column joint within $8d_b$ of the centerline of the headed bar.

The development length equations presented in this article have been validated by Zaborac and Bayrak[4] for headed deformed bar sizes up to no. 11 with net bearing area of head equal to or larger than four times the bar area in normalweight concrete with a design concrete compressive strength of up to 15.0 ksi. The MC2010[1] contains a similar general formulation, and it does not restrict the use of lightweight concrete in combination with headed bar anchorage. Equation 5.10.8.2.1a-2 of the AASHTO LRFD specifications accounts for the influence of lightweight concrete by reducing the effective concrete tension strength in bond with the concrete density modification factor λ. Therefore, the use of lightweight concrete is allowed by these provisions, and it is subject to the same concrete strength limits as nonheaded straight bars, lap splices in tension, and hooked bars.

The minimum spacing, cover, and development length requirements are based on the requirements of ACI 318-19. The influence of parallel tie reinforcement is conservatively neglected in the derivation of the headed-bar equations; however, research has shown that parallel tie reinforcement is an effective method for improving the performance of headed deformed bars.[14] The reduction of ℓ_{dt} permitted for the presence of parallel tie reinforcement within beam-column joints is a conservative interpretation of the ACI 318-19 provisions, and it is analogous to the previously described reduction for hooked bars. The influence of confinement reinforcement can be investigated with the strut-and-tie modeling procedure in Article 5.8 of the AASHTO LRFD specifications for most typical cases. If a headed bar does not comply with Article 5.10.8.2.7, then Article 5.10.8.3 or Article 5.13 should be considered.

The concrete efficiency factor for a CTT node specified in Table 5.8.2.5.3a-1 is appropriate for headed bars, regardless of the percentage of reinforcement confining the bar. This is due to low strain values associated with tensile failure of concrete around the bar being anchored and based on the experimental validation reported by Zaborac and Bayrak.[4]

Conclusion

The design expressions presented in this article and approved by the AASHTO Committee on Bridges and Structures were validated by Zaborac and Bayrak by using 1006 tests from the relevant, published technical literature.[4] The statistical performance of the expressions was deemed to be consistent with the calibration intent of the AASHTO LRFD specifications. The revised specifications will reduce the number of unconservatively estimated hooked bar development lengths, particularly for larger bar sizes.

References

1. *fib* (International Federation for Structural Concrete). 2013. *fib Model Code for Concrete Structures 2010*. Berlin, Germany: Ernst & Sohn. https://doi.org/10.1002/9783433604090.
2. American Association of State Highway and Transportation Officials (AASHTO). 2020. *AASHTO LRFD Bridge Design Specifications*, 9th ed. Washington, DC: AASHTO.
3. Tepfers, R. 1979. “Cracking of Concrete Cover Along Anchored Deformed Reinforcing Bars.” *Magazine of Concrete Research* 31 (106): 3–12. https://doi.org/10.1680/macr.1979.31.106.3.
4. Zaborac, J., and O. Bayrak. 2022.“Unified Approach for Reinforcement Development Length Using the Partly Cracked Elastic Stage for Bond Strength.” *Structural Concrete*. https://doi.org/10.1002/suco.202200117.
5. Zuo, J., and D. Darwin. 2000. “Splice Strength of Conventional and High Relative Rib Area Bars in Normal and High-Strength Concrete.” *ACI Structural Journal* 97 (4): 630–641. https://doi.org/10.14359/7428.
6. Canbay, E., and R. J. Frosch. 2005. “Bond Strength of Lap-Spliced Bars.” *ACI Structural Journal* 102 (4): 605–614. https://doi.org/10.14359/14565.
7. American Concrete Institute (ACI) 2003. *Bond and Development of Straight Reinforcing Bars in Tension*. ACI PRC-408. Farmington Hills, MI: ACI.
8. Hamad, B. S., J. O. Jirsa, and N. I. D’Abreu de Paulo. 1993. “Anchorage Strength of Epoxy-Coated Hooked Bars.” *ACI Structural Journal* 90 (2): 210–217. https://doi.org/10.14359/4127.
9. ACI. 2019. *Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (ACI 318R-19).* Farmington Hills, MI: ACI.
10. Ramirez, J. A., and B. W. Russell. 2008. *Transfer, Development, and Splice Length for Strand/Reinforcement in High-Strength Concrete*. National Cooperative Highway Research Program Report 603. Washington, DC: National Academies Press. https://doi.org/10.17226/13916.
11. Sperry, J., S. Yasso, N. Searle, M. DeRubeis, D. Darwin, M. O’Reilly, A. Matamoros, L. R. Feldman, A. Lepage, R. D. Lequesne, and A. Ajaam. 2017. “Conventional and High-Strength Hooked Bars—Part 1: Anchorage Tests.” *ACI Structural Journal* 114 (1) 255–265. https://doi.org/10.14359/51689456.
12. Concrete Reinforcing Steel Institute (CRSI). 2020. *Guide to the Use of Grade 100 Reinforcing Bars in ACI 318-19*. ETN-D-6-20. Schaumburg, IL: CRSI.
13. ASTM International. 2018. *Standard Specification for Headed Steel Bars for Concrete Reinforcement*. ASTM A970/A970M-18. West Conshohocken, PA: ASTM International. https://doi.org/10.1520/A0970_A0970M-18.
14. Ghimire, K. P., Y. Shao, D. Darwin, and M. O’Reilly. 2019. “Conventional and High-Strength Headed Bars—Part 1: Anchorage Tests.” *ACI Structural Journal* 116 (3): 255–264. https://doi.org/10.14359/51714479.

Dr. Oguzhan Bayrak is a chaired professor at the University of Texas at Austin, where he serves as the director of the Concrete Bridge Engineering Institute.