AASHTO LRFD Bridge Design Specifications: Substructure Design Using Strut-and-Tie-Modeling

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The 2024 meeting of the American Association of State Highway and Transportation Officials (AASHTO) Committee on Bridges and Structures (COBS) took place in June 2024 in Indianapolis, Ind. The AASHTO Concrete Committee (formerly AASHTO T-10) did not present any new agenda items for approval by COBS. While the committee is actively engaged on several detailed working agenda items, the discussions on those items are ongoing and, in view of the publication date of the next edition of the *AASHTO LRFD Bridge Design Specifications*,[1] the committee opted to complete that effort in the coming months. That decision creates an opportunity for our *ASPIRE*® team to tackle some recently received questions and provide clarifications that are needed in the bridge design community. This article focuses on two important aspects of structural design considering both strut-and-tie modeling (STM) and conventional sectional design procedures.

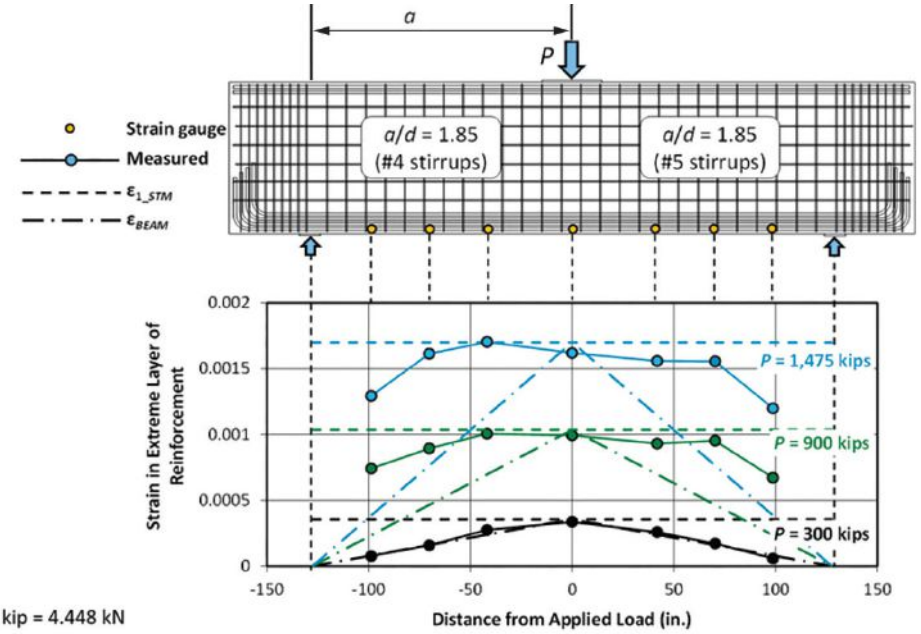
Reinforcement Anchorage

To begin, let us consider reinforcing bar anchorage. Anchorage of reinforcement is essential to ensure that the structural response assumed in design calculations remains consistent with the actual response of a structure when (or if) it is loaded to its ultimate state. The ultimate state is rarely seen in the field except in extreme events such as natural disasters or human-caused extreme loading conditions. With that caveat, in load- and resistance-factor design (LRFD), we routinely consider ultimate states (worst-case scenarios) to build sufficient safety margins against structural failure.

With that introduction, let us consider the simply supported beam that we tested in one of our experiments at the Phil M. Ferguson Structural Engineering Laboratory at the University of Texas at Austin. **Figure 1** shows the strain distribution in the flexural tension reinforcement in a beam tested with a shear span–to–depth ratio a/d of 1.85, when two different reinforcement quantities are employed in each of the two test regions. It is important to note that this shear span–to–depth ratio is near the boundary (that is, a/d = 2.0) at which the behavior of the beam would transition from being nonlinear (D-region) to linear (B-region).

The first question is whether the behavior of the beam is better represented by STM or by the classical beam-bending theory postulated by Bernoulli. Figure 1 shows that at early stages of loading, the measured strains from gauges installed on the flexural tension reinforcement agree with strains calculated using classical beam-bending theory. That is, when a 300-kip midspan load is applied, the nearly triangular strain distribution is consistent with the triangular bending-moment diagram that would have a maximum moment in the middle of the span ($PL/4$). As the loading increases from 300 kips (linear-elastic range) to 900 kips (cracked state) to 1475 kips (near ultimate state), the strain distribution on the longitudinal reinforcement becomes nearly uniform. It no longer follows the triangular distribution that is consistent with the linear-elastic behavior and consistent with the bending-moment diagram of a simply supported beam. Let us keep this behavior in mind as we discuss the anchorage of flexural reinforcement in this beam.

Figure 1. Deep-beam testing results demonstrate the progression of strain patterns in the extreme layer of longitudinal reinforcement at different load levels. Note: a = shear span; d = effective depth of member; P = applied load; ε_{BEAM} = strain predicted by classical beam theory; ε_{1_STM} = strain predicted by the strut-and-tie model. Figure: Derived from Birrcher et al.[2]



1 kip = 4.448 kN
1 in. = 25.4 mm

The classic sectional design method indicates that the critical section in this beam is at midspan for flexure and that the development length of reinforcement should be measured from this critical section. The strain patterns of the deep beam that we investigated indicate that at the ultimate or near-ultimate state, the strains experienced by the flexural tension reinforcement are more uniform over the length of the beam and only decrease in the immediate vicinity of the supports. This behavior is more consistent with the assumed response in the truss model shown in **Fig. 2**, where the tie force remains constant along the length of the beam. This condition indicates the need to anchor the tie (primary flexural tension reinforcement) from the critical section near the support (node), which is a different critical section location than that for more slender beams (Fig. 2). For the STM model, the flexural reinforcement near the support on the left needs to be adequately anchored beyond this critical section. Because of structural and loading symmetry, the same requirement also applies near the right-hand support.

Figure 3 shows a close-up of the left-hand support and illustrates additional geometric details to define the extended nodal zone that we can use in locating the critical section from which we can measure the development length of the reinforcement. More specifically, the critical section passes through the point where the flexural tension reinforcement exits the compression field that forms near the support.

The previously discussed location of the critical section results in a need for hooked bars where straight bars were often used in the past. Figure 2 shows that the available length to develop bars is substantially greater for sectional design than it is for STM-based design. With that stated, the behavior illustrated in Fig. 1 clearly shows that deep beams that have a shear span–to–depth ratio less than 2 ($a/d < 2$) exhibit a structural response that is more consistent with the STM model shown in Fig. 2 than with the traditional flexural response we see in Bernoulli beams. In short, observed structural responses underlie the specification changes made with the introduction of STM in the AASHTO LRFD specifications in 1994 and the further refinement of the STM method during the reorganization of the AASHTO LRFD specifications. Bent caps and most substructure components can be classified as D-regions and fall into this category of structural behavior.

Figure 2. Comparison of the critical sections of a deep beam (D-regions, strut-and-tie model [STM]) and a Bernoulli beam (B region, beam theory). The critical section determines the point from which the development length of reinforcement should be measured. Figure: Dr. Oguzhan Bayrak.

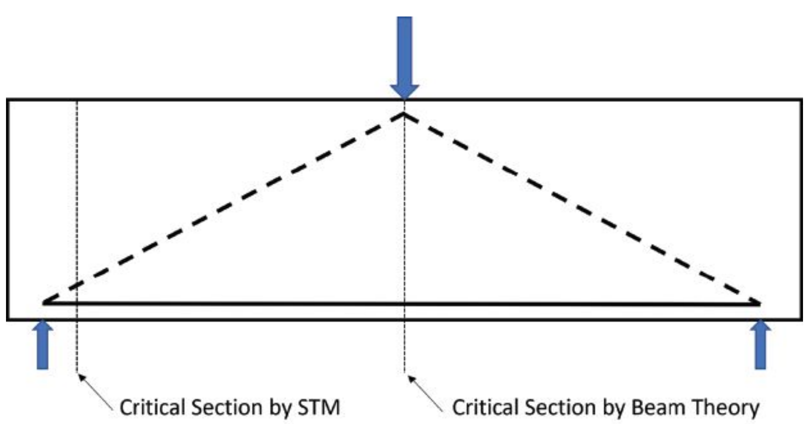
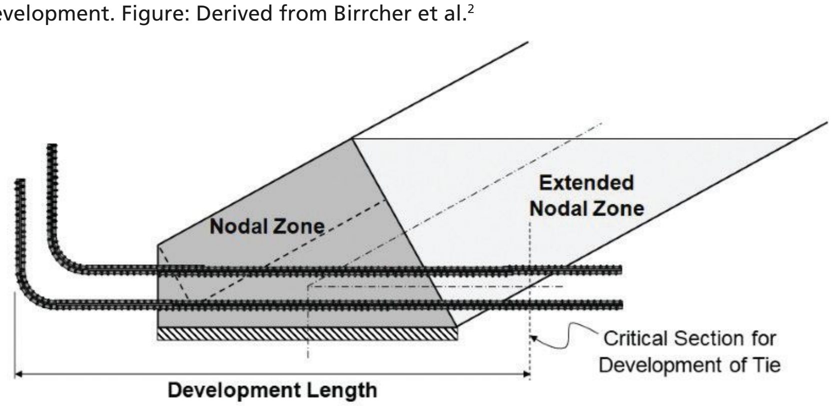


Figure 3. Critical section of a strut-and-tie model for determining reinforcing bar development. Figure: Derived from Birrcher et al.[2]



Serviceability Considerations

The second item I will cover in this article relates to substructure design and, in particular, serviceability of members designed by STM. The commentary to Article 5.8.2.2 of the AASHTO LRFD specifications states the following:

> *The estimated resistance at which diagonal cracks begin to form,* V_{cr}*, is determined for the initial geometries of the D-Regions using the following expression (Birrcher, et al., 2009):*
>
> $$V_{cr} = \left[0.2 - 0.1\left(\frac{a}{b}\right)\right]\sqrt{f'_c}\,b_w d \qquad \text{(C5.8.2.2-1)}$$
>
> *but not greater than* $0.158\sqrt{f'_c}\,b_w d$
>
> *nor less than* $0.0632\sqrt{f'_c}\,b_w d$.
>
> *where:*
>
> a = *shear span (in.)*
>
> d = *effective depth of the member (in.)*
>
> f'_c = *compressive strength of concrete for use in design (ksi)*
>
> b_w = *width of member's web (in.)*
>
> *Where the shear in service is less than* V_{cr}*, reasonable assurance is provided that shear cracks are unlikely to form.*

The basis of this equation can be found in Birrcher et al.[2] An examination of **Fig. 4**, which is adopted from Birrcher et al., clearly shows that Eq. (C5.8.2.2-1) of the AASHTO LRFD specifications was developed as a reasonable lower bound to the cracking shear stresses observed in a

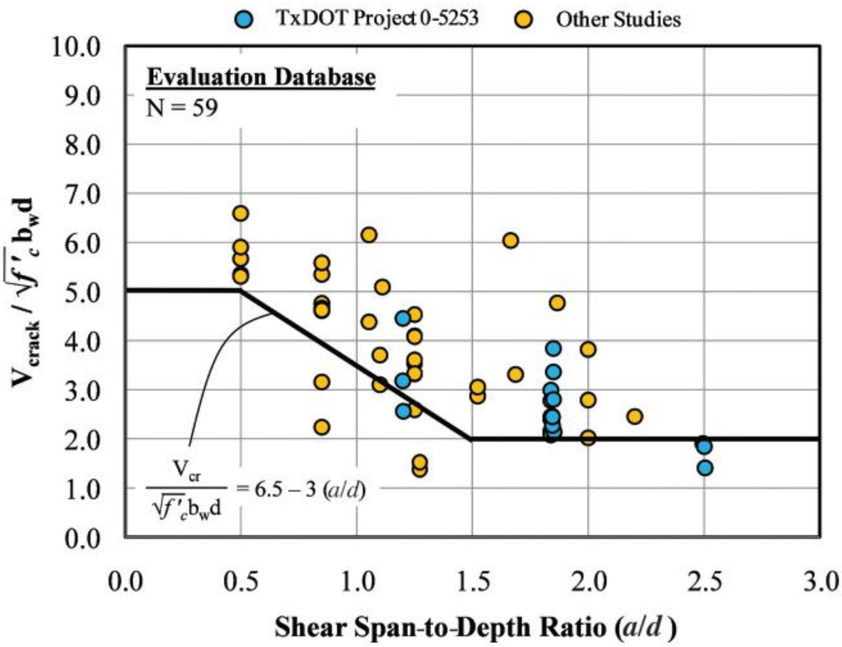


Figure 4. Test results used for the determination of the cracking shear stress. Figure: Derived from Birrcher et al.[2]

representative set of test results. It is important to note that the original equation presented in Fig. 4 was derived in psi units and converted to ksi units when it was adopted by the AASHTO LRFD specifications.

While the shear service check covered in this commentary is not in mandatory language, some states have adopted this recommendation in the structural design policies in their bridge design manuals. This deliberate decision results in some bent caps that have somewhat larger cross-sectional dimensions than would be needed by the strength state considerations. The larger bent cap dimensions resulting from the use of this expression are intended to reduce the frequency of occurrence of shear cracks seen in some substructure components. In this context, we must all remember that we are aspiring to design bridges that will last about 100 years.

References

1. American Association of State Highway and Transportation Officials (AASHTO). 2020. *AASHTO LRFD Bridge Design Specifications*. 9th ed. Washington, DC: AASHTO.
2. Birrcher, D., R. Tuchscherer, M. Huizinga, O. Bayrak, S. Wood, and J. Jirsa. 2009. *Strength and Serviceability Design of Reinforced Concrete Deep Beams*, Technical Report 0-5253-1. Austin: Center for Transportation Research, Bureau of Engineering Research, University of Texas at Austin. https://ctr.utexas.edu/wp-content/uploads/pubs/0_5253_1.pdf.