Control of Flexural Cracking

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Control of flexural cracking by distribution of reinforcement is addressed in Article 5.7.3.4 of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*.[1] This article requires that the spacing, s, of nonprestressed steel reinforcement in the layer closest to the tension face shall satisfy the following equation:

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \qquad \text{(AASHTO Eq. 5.7.3.4-1)}$$

in which:

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

where:

γ_e = exposure factor
= 1.00 for Class 1 exposure condition
= 0.75 for Class 2 exposure condition

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.)

f_{ss} = calculated tensile stress in nonprestressed steel reinforcement at the service limit state not to exceed 0.60 f_y (ksi)

h = overall thickness or depth of the component (in.)

In most situations, Eq. 5.7.3.4-1 results in reasonable bar spacings. However, the following two situations exist where the equation results in small bar spacings and even negative values for s:

1. The use of Grade 100 reinforcement in bridge decks, which was permitted in the 2013 Interim Revisions.[2] This affects both ASTM A1035 and the new ASTM A615 (AASHTO M31) Grade 100 bars.
2. The clear cover is relatively large compared to the section depth. An extreme example is where piles extend into pile caps and the bottom layer of pile cap reinforcement is placed above the top of the piles. This results in a value of d_c at least equal to the embedment depth of the piles into the pile cap.

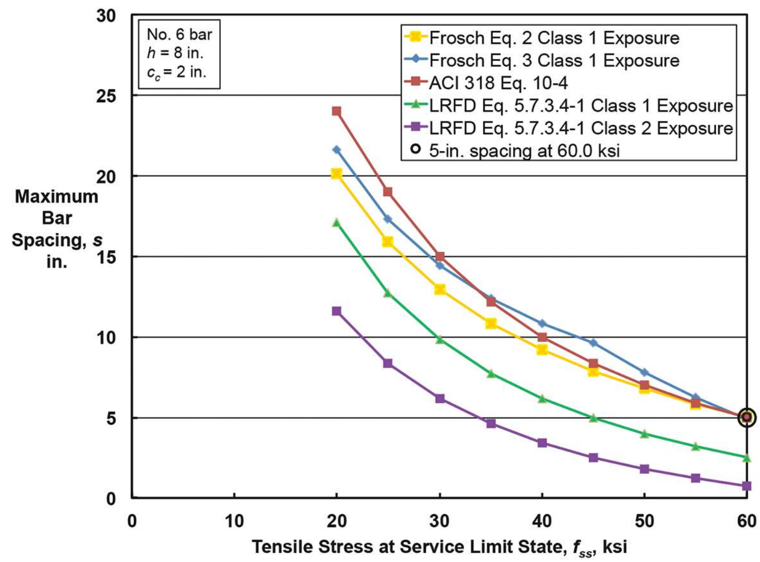

Figure 1. Comparison of crack control equations from several sources. All Figures: Henry Russell.

Equation 5.7.3.4-1 was developed by DeStefano et al.[3] based on a review of past research, parametric studies, and various crack width predictive methods. The equation is based on an assumed crack width of 0.017 in. for Class 1 exposure. It replaced a previous equation developed by Gergely and Lutz.[4] In the previous equation, the value of clear cover, c_c, used to calculate d_c was limited to a maximum value of 2 in. In the current equation, d_c is to be calculated using the actual concrete cover.

To ascertain the validity of the small values of s, values calculated using Eq. 5.7.3.4-1 were compared with values calculated using equations developed by Frosch[5] and ACI Committee 318[6] as described in the next sections.

Frosch Equations

Frosch[5] developed the following equation to predict crack widths with uncoated reinforcement:

$$w_{cu} = 2\frac{f_s}{E_s}\beta_s\sqrt{(d_c)^2 + \left(\frac{s}{2}\right)^2} \qquad \text{(Eq. 1)}$$

in which:

β_s = $1.0 + 0.08d_c$

where:

w_{cu} = maximum crack width for uncoated reinforcement (in.)

f_s = stress in steel reinforcement (ksi)

E_s = modulus of elasticity of reinforcing bars (ksi)

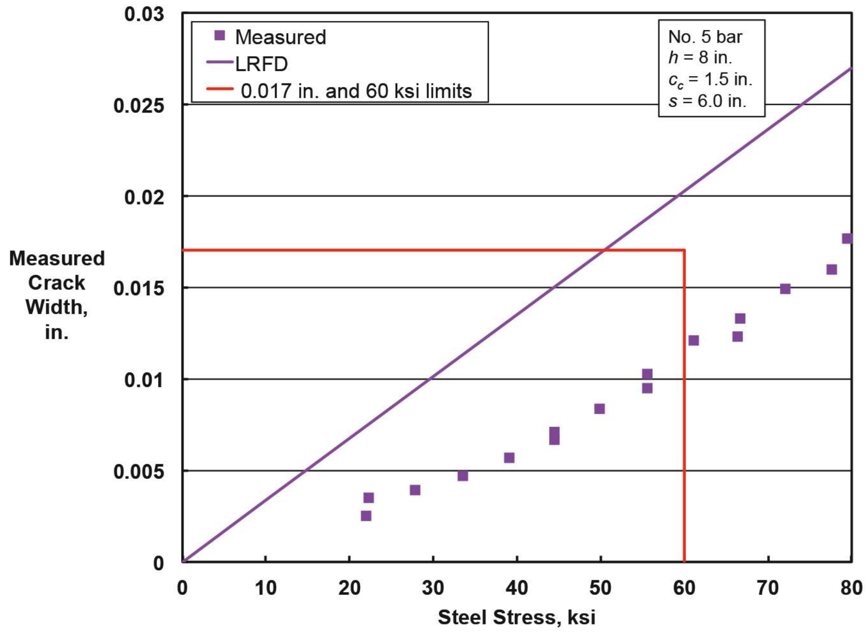

Figure 2. Comparison of crack widths versus steel stress for data from Sim[7] and relationship assumed for AASHTO LRFD Eq. 5.7.3.4-1.

The equation can be rewritten to solve for maximum permitted reinforcement bar spacing as follows:

$$s = 2\sqrt{\left(\frac{w_{cu}E_s}{2f_s\beta_s}\right)^2 - (d_c)^2} \qquad \text{(Eq.2)}$$

Frosch compared crack widths calculated by his equation with existing test data for reinforcement stress levels, f_s, ranging from 20 to 50 ksi, but not including any at 60 ksi. Based on maximum crack widths between 0.016 and 0.021 in., Frosch then proposed the following simplified design equation:

$$s = 12\alpha_s\left[2 - \frac{d_c}{3\alpha_s}\right] \le 12\alpha_s \qquad \text{(Eq.3)}$$

where:

α_s = reinforcement factor = $\frac{36}{f_{ss}}\gamma_c$

γ_c = reinforcement coating factor: 1.0 for uncoated reinforcement; 0.5 for epoxy-coated reinforcement, unless test data can justify a higher value.

ACI 318 Equation

In 2005, ACI Committee 318[6] adopted the following simplified version of Frosch's equation:

$$s = \frac{600}{f_{ss}} - 2.5c_c \le \frac{480}{f_{ss}} \qquad \text{(ACI Eq. 10-4)}$$

and f_{ss} may be taken as 2/3 f_y.

Comparison of Calculated Bar Spacings

The maximum bar spacing calculated from the Frosch equations, ACI 318 equation, and the current AASHTO LRFD equation for Class 1 or Class 2 exposure are shown in Figure 1 for an 8-in.-thick slab containing No. 6 bars with a clear cover, c_c, of 2 in. As expected, the maximum spacing per the Frosch equations and ACI 318 equation agree reasonably well and are all greater than the maximum spacing calculated using the AASHTO LRFD equation for both Class 1 and Class 2 exposure conditions. Figure 1 indicates that the AASHTO LRFD equation is more conservative than the Frosch or ACI 318 equations, which is generally the case for bridge decks.

Clearly, one variable that influences the calculated maximum bar spacing using the AASHTO LRFD equation is the exposure factor—either 1.0 or 0.75. The exposure factor is applied to the first term of Eq. 5.7.3.4-1. Mathematically, this results in bar spacings being calculated as the difference between two numbers that are close together and, as mentioned previously, can result in negative values.

The exposure factor for Class 1 exposure condition of 1.00 is based on an assumed crack width of 0.017 in. When this crack width is substituted into Frosch's Eq. 2 and 3 for a No. 6 bar with h = 8 in., c_c = 2 in., and f_{ss} = 60 ksi for Grade 100 reinforcement, the following bar spacings are obtained:

Frosch's equation: 5.01 in.

Frosch's design equation: 4.90 in.

This is illustrated in Figure 1 with convergence of the two lines for Frosch's equations and the ACI equation at a spacing of about 5 in. and tensile stress of 60 ksi.

Proposed Solution

With larger bar spacings, there is concern about larger crack widths. Sim[7] conducted flexural tests using 8.0-in.-deep beams containing No. 5 Grade 60 and 100 uncoated reinforcing bars at a center-to-center spacing of 6.0 in. and with a clear cover of 1.5 in. Figure 2 shows the reported values of average crack widths and corresponding steel stresses. All measured crack widths at steel stress levels below 60 ksi were less than the crack width of 0.017 in. assumed for Class 1 exposure condition as depicted by the red lines in Figure 2. The 60.0 ksi limit is the maximum stress permitted for Grade 100 reinforcement in Eq. 5.7.3.4-1.

Figure 2 also shows the relationship between crack width and steel stress that was assumed by DeStefano et al.[3] in their development of Eq. 5.7.3.4-1. The diagonal line labelled "LRFD" is calculated for the cross section tested by Sim. The measured crack widths are all less than the assumed values for the same stress levels.

To address the two situations described above, the AASHTO Subcommittee on Bridges and Structures has approved the following addition to the Commentary of Article 5.7.3.4:

"In certain situations involving higher-strength reinforcement or large concrete cover, the use of Eq. 5.7.3.4-1 can result in small or negative values for s. Where higher-strength reinforcement is used, an analysis of five crack control equations suggests a bar spacing not less than 5.0 in. for control of flexural cracking. Where large concrete cover is used, past successful practice suggests a value of d_c not greater than 2.0 in. plus the bar radius for calculation purposes. The limit of 2 in. plus bar radius is the same limit that was used in the Specifications prior to 2005 when the current version of Eq. 5.7.3.4-1 was introduced."

This revision will become effective with the 2016 Interim Revisions. Although the revision provides a simple solution, more research concerning Eq. 5.7.3.4-1 is needed as higher-strength reinforcing bars become available.

References

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