

FLEXURAL CRACK CONTROL IN CONCRETE BRIDGE STRUCTURES**Ralph J. DeStefano, PE**, Pennsylvania Department of Transportation, Hollidaysburg, PA**Jack Evans, PE**, Florida Department of Transportation, Tallahassee, FL**Maher K. Tadros, PhD, PE**, Dept of Civil Engineering, Univ. of Nebraska, Omaha, NE**Chuanbing Sun**, Dept of Civil Engineering, University of Nebraska, Omaha, NE**ABSTRACT**

Mitigation and control of cracking is important for obtaining acceptable appearance and for long-term durability of concrete bridge structures, especially those subjected to saltwater spray and deicing chemicals. In the current AASHTO LRFD Specifications and in the ACI 318 Code editions prior to 1999, flexural crack control requirements were based on the so-called Z-factor method developed by Gergely and Lutz in 1968. Their work was based on extensive statistical evaluation of laboratory experiments. ACI 318 decided in the 1999 edition to greatly simplify crack control requirements due increased evidence suggesting a reduced correlation between crack width and reinforcement corrosion. The ACI formula relates maximum bar spacing to steel stress under service loads and clear concrete cover. This paper presents the background of previous crack control methods and comparison of their impact on design of bridges and culverts. A new equation is proposed for adoption in the AASHTO LRFD Specifications that is intended to be simple, yet account for differences between bridge and building structures.

Keywords: Concrete, crack control, serviceability, reinforcement, corrosion

INTRODUCTION

With the introduction of the AASHTO LRFD specifications in 1994, several states have made the observation that crack control requirements tend to govern the design of flexural steel in reinforced concrete members more frequently than under the LFD Standard Specifications. While it is believed the primary reason for crack control governing more often under the LRFD specification is due changes made on the load side of the equation (load factors and reduced number of service limit states), designers and researchers have also taken the opportunity to re-evaluate the resistance side of the equation, which is essentially a direct carry over from the Standard Specifications.

Additional crack control research conducted in the 1980's and 1990's, which has failed to show any kind of definitive correlation between crack width and long-term durability of reinforced concrete structures has led specification organizations to re-evaluate the underlying design provisions. They began to question the logical behind requiring detailed computations for a phenomenon with wide inherent scatter and questionable end results. In 1999, ACI 318 adopted a much simpler provision to design for crack control. Now, AASHTO is investigating adoption of a similar simplified provision.

The intent of this paper is to provide background on the development of the existing AASHTO LRFD crack control specifications and to provide a discussion on the proposed revisions.

BACKGROUND

With the inception of ultimate strength design methodology and higher strength Grade 60 steel reinforcement, researchers and designers recognized the need for providing a mechanism by which crack widths would be minimized through reinforcement detailing. It was observed that the use of fewer, larger reinforcement bars at wider spacings resulted in larger cracks than the use of more smaller reinforcement bars at tighter spacings for the same amount of total reinforcement area. Controlling crack widths was believed important from a long-term durability standpoint (corrosion of the reinforcement) as well as from an aesthetics standpoint.

Research was undertaken in the 1960's to quantify the above concept and to develop design tools. Out of this research came the well-known Gergely-Lutz equation¹. The empirical equation was developed using statistical analysis techniques on experimental data from several different researchers. The original equation for predicting the width of a flexural crack on the tension face of a reinforced concrete member is:

$$w_c = 0.000076 \beta f_s \sqrt[3]{d_c A} \quad (1)$$

where,

w_c = crack width, in.;

- β = factor relating the strain at the tension face to the strain at the centroid of the reinforcement;
- f_s = stress in steel reinforcement, ksi;
- d_c = distance from tension face to centroid of nearest reinforcement layer, in.;
- A = average effective concrete area per bar of the flexural tension reinforcement, in². For a single layer of reinforcement of constant spacing, the term A simplifies to $2 d_c s$, where s = bar spacing.

Another well-known crack width predictive model developed in the same era was the Kaar-Mattock equation². Similar to the Gergely-Lutz equation, it was developed based on a statistical analysis of experimental data. The Kaar-Mattock equation for predicting the width of flexural cracks at the tension face of a reinforced concrete flexural member is:

$$w_c = 0.000115 \beta f_s \sqrt[4]{A}, \quad (2)$$

where the variables are the same as defined for the Gergely-Lutz equation.

AASHTO adopted the Gergely-Lutz equation for controlling flexural cracking, but in a slightly rearranged form³. The crack width variable and the β factor were consolidated into a single Z-factor, and the equation was written in terms of allowable stress. Using an approximate limiting crack width of 0.016 in. and an average β factor of 1.2 results in the current equation found in the AASHTO LRFD specifications:

$$f_{sa} = \frac{Z}{\sqrt[3]{d_c A}} \leq 0.6 f_y \quad (3)$$

where,

- f_{sa} = allowable reinforcement stress, ksi'
- Z = 170 for moderate exposure conditions,
 = 130 for severe exposure conditions,
 = 100 for precast box culverts,
 = $\frac{155}{\beta}$ for cast-in-place box culverts;

with the remaining terms as defined previously.

The equation has been a source of contention among designers for the some time for the following reasons:

- the equation has a built-in paradox in that increasing the concrete cover is detrimental to crack control even though it is well established that increasing cover is beneficial in enhancing long-term durability. In fact, concrete covers over 3 inches are essentially impractical to construct using the results from the equation.

- computation of the effective concrete area term, A , can be cumbersome for members with mixed bar sizes and bundled bars;
- precast box culverts are treated differently from cast-in-place box culverts.

AASHTO addressed the first issue by allowing designers to use 2 inches maximum for the concrete cover in the crack control computations regardless of the actual cover with the rationale that any additional cover is sacrificial. However, such arbitrary manipulation of the equation has generated suspicion as to the overall accuracy of the methodology.

A NEW CRACK WIDTH EQUATION

Frosch⁴ observed that a significant shortcoming of the Gergely-Lutz and Kaar-Mattock equations is that they were both developed empirically using statistical analysis techniques on experimental data that was limited in the range of concrete covers investigated. In fact, he noted that only three test specimens had concrete covers greater than 2.5 inches.

Frosch developed the following simple, theoretically-derived equation to predict crack widths that could be used regardless of the actual concrete cover:

$$w_c = 2 \frac{f_s}{E_s} \beta \sqrt{(d_c)^2 + \left(\frac{s}{2}\right)^2} \quad (4)$$

where,

w_c = crack width, in.;

f_s = stress in steel reinforcement, ksi;

E_s = modulus of elasticity of steel reinforcement, ksi;

β = factor relating the strain at the tension face to the strain at the reinforcement layer;

d_c = distance from tension face to centroid of nearest reinforcement layer, in.;

s = reinforcement bar spacing, in.

The equation can be re-written to solve for the maximum permitted reinforcement bar spacing for a limiting crack width, as follows:

$$s = 2 \sqrt{\left(\frac{w_c E_s}{2 f_s \beta}\right)^2 - d_c^2} \quad (5)$$

The equation is based on the following premises:

- the crack width is a direct function of the strain in the reinforcement times the crack spacing;
- the maximum crack spacing is estimated to be two times the controlling cover distance given by the expression, $d^* = [d_c^2 + (0.5s)^2]^{1/2}$;

- the concrete between cracks does not resist any tension (which will result in a more conservative estimate of crack width.)

Frosch compared the crack widths predicted with his theoretical cracking model to the widths predicted from the Gergely-Lutz and Kaar-Mattock empirical equations. The results showed very good correlation.

NEW ACI 318 CRACK CONTROL EQUATION

Based on the work by Frosch⁵ and additional crack control research^{6,7} performed in the 1980's and 1990's that failed to demonstrate a credible link between crack width and problems with long-term durability of reinforced concrete members, the ACI 318 Code switched from the long-held Gergely-Lutz equation and adopted a simplified version of the Frosch's cracking model⁸.

The final form of the cracking model equation adopted by ACI 318 in 1999 is:

$$s = \frac{540}{f_s} - 2.5c_c \leq \frac{432}{f_s} \quad (6)$$

where,

c_c = clear concrete cover on reinforcement nearest the tension face, in.;
with the other variables as defined previously.

This simplified equation of the cracking model is based on the following assumptions:

$$\begin{aligned} \beta &= 1 + 0.08 d_c; \\ w_c &= 0.016 \text{ in. limiting crack width, in.}; \\ d_c &= c_c + 0.5 \text{ in., (i.e., \#8 average bar size.)} \end{aligned}$$

The following should be noted about the approach taken by ACI:

- in lieu of explicit computation of f_s , ACI permits use of an assumed value of $0.6f_y$, or 36 ksi for Grade 60 reinforcement.
- emphasis is placed on limiting the reinforcement spacing and not limiting the allowable stress;
- no distinction is made between interior and exterior exposure conditions;
- no upper limit is defined for the stress in the reinforcement, f_s .

If a value of 36 ksi is assumed for f_s , the ACI equation yields maximum permitted bar spacings of 11.25", 10", 7.5" and 5" for concrete covers of 1.5", 2", 3" and 4", respectively.

PROPOSED AASHTO LRFD CRACK CONTROL EQUATION

Based on a review of past research, parametric studies comparing various crack width predictive methods, and results from actual design example problems, an equation very similar to the Frosch cracking model adopted by ACI 318 is also proposed for consideration by AASHTO. The proposed AASHTO equation is as follows:

$$f_{sa} = \frac{700\gamma_e\gamma_r}{\beta(s + 2d_c)} \leq 0.8f_y \quad (7)$$

where,

f_{sa} = allowable service level stress in the reinforcement, ksi;

γ_e = exposure factor,

= 1.0 for Case 1,

= 0.75 for Case 2;

γ_r = reinforcement factor,

= 0.75 for smooth weld-wire fabric,

= 1.00 for all other types of reinforcement;

β = $1 + \frac{d_c}{0.7(h - d_c)}$;

with the remaining variables as defined previously.

The proposed AASHTO equation is essentially the same as the ACI 318 equation with the following modifications:

- the limiting crack width is approximately 0.017 in., slightly larger than the value of 0.016 in. used previously;
- addition of an exposure factor, γ_e , is included to maintain a distinction between different environmental conditions;
- addition of a reinforcement factor, γ_r , to recognize the fact that research suggests smooth welded-wire fabric has reduced bond properties, most notably for wider wire spacings, compared to other reinforcement types;
- the equation is written in terms of allowable stress to be consistent with past practice, although rearranging the equation to be written in terms of maximum permitted bar spacing, s , can be done easily;
- the β -factor is made an integral part of the equation rather than an assumed "average" value;
- the d_c term is retained in the formal development of the cracking model instead of using the clear cover with an assumed "average" rebar size of #8;

A larger limiting crack width of 0.017 inch is used because of the proposal to retain the distinction between exposure conditions. By providing two different levels of cracking tolerance, a slightly larger crack width is considered acceptable for the less severe exposure condition (Case 1.)

An upper limit of $0.8f_y$ on the allowable stress is proposed to provide a factor of safety against permanent yielding of the reinforcement under service loads. This provision is similar to that stipulated for steel flexural members in Article 6.10.5.2 of the AASHTO LRFD specifications³.

Additional discussion is presented in later sections of the paper on the β -factor and the exposure factor being proposed for the AASHTO equation.

COMPARISON OF RESULTS

Figures 1, 2 and 3 show a comparison of results between the current AASHTO LRFD equation (Gergely-Lutz), the Kaar-Mattock equation, the ACI 318-99 equation, the cracking model by Frosch and the proposed AASHTO LRFD equation for concrete covers of 2", 3" and 4", respectively. In these figures, a constant value of 1.2 is assumed for β . Furthermore, Case 1 exposure conditions ('moderate' exposure) are evaluated as well as a constant reinforcement bar size of #8. It is important to note that for the current AASHTO equation, the 2" maximum concrete cover stipulation permitted by the specification is used.

The following observations are made concerning the results:

- The simplified ACI 318-99 equation fits the theoretical cracking model well for all concrete cover cases. It permits slightly higher allowable stresses for the 2-inch cover case and slightly lower allowable stresses for the 3-inch and 4-inch cover cases.
- The proposed AASHTO equation permits slightly higher allowable stresses compared to the ACI 318-99 and the crack model equation, which is to be expected since the proposed AASHTO equation is based on a slightly larger limiting crack width.
- The Kaar-Mattock equation permits the highest levels of allowable stresses compared to all of the other equations, and for stress levels less than approximately 40 ksi, it permits significantly higher allowable stresses.
- The current AASHTO equation, in general, permits higher allowable stresses at wider bar spacings and lower allowable stresses at tighter bar spacings. It is interesting to note, though, that if the comparisons are limited to a stress level less than 36 ksi, which is the upper limit stipulated in the AASHTO specifications, then the current AASHTO equation is less conservative (permits higher allowable stresses) than the cracking model, the ACI 318-99 equation and the proposed AASHTO equation. However, what is important to note is that these comparisons are limited to a single layer of reinforcement consisting of #8 bars (i.e., $d_c = 2.5$ inches.) Whenever situations are encountered in which additional depth must be accounted for in the computation of the d_c term, then the current AASHTO equation can be significantly more conservative. This fact is demonstrated in the design example for the footing reinforcement at the end of this paper.

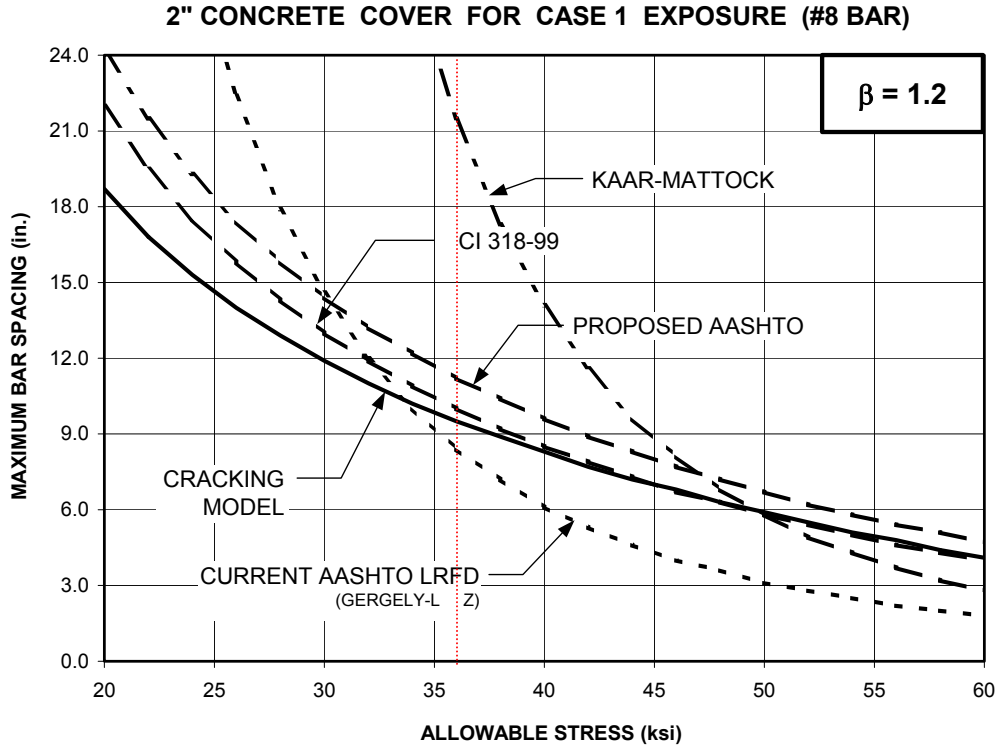


Fig. 1: Allowable Stress vs. Bar Spacing for Case 1 Exposure — 2" Concrete Cover

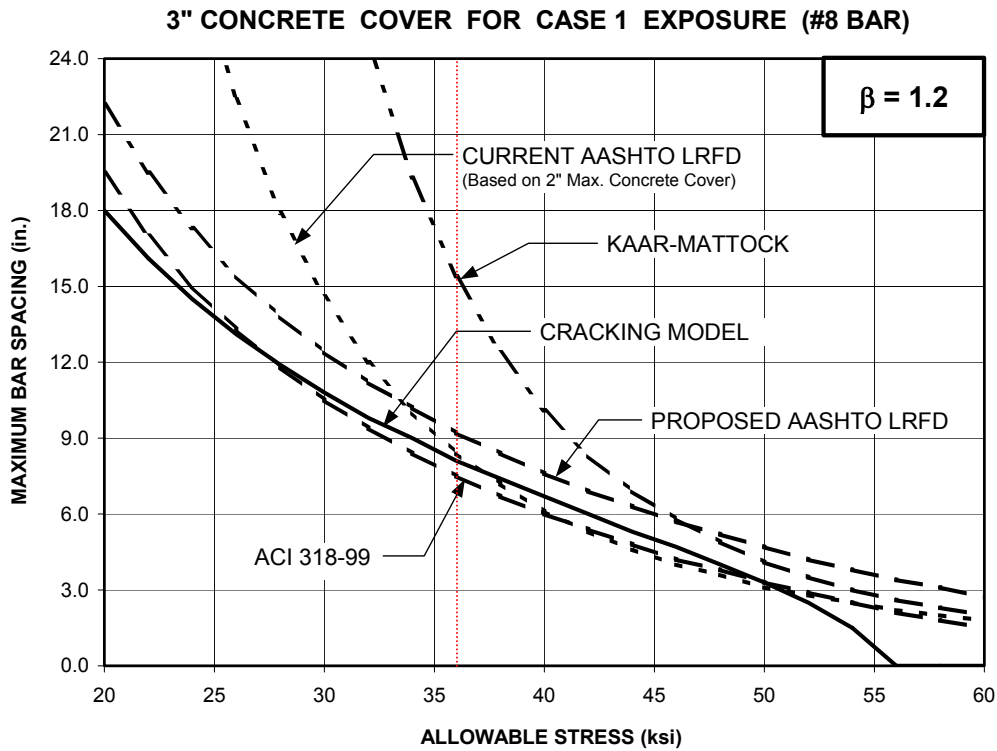


Fig. 2: Allowable Stress vs. Bar Spacing for Case 1 Exposure — 3" Concrete Cover

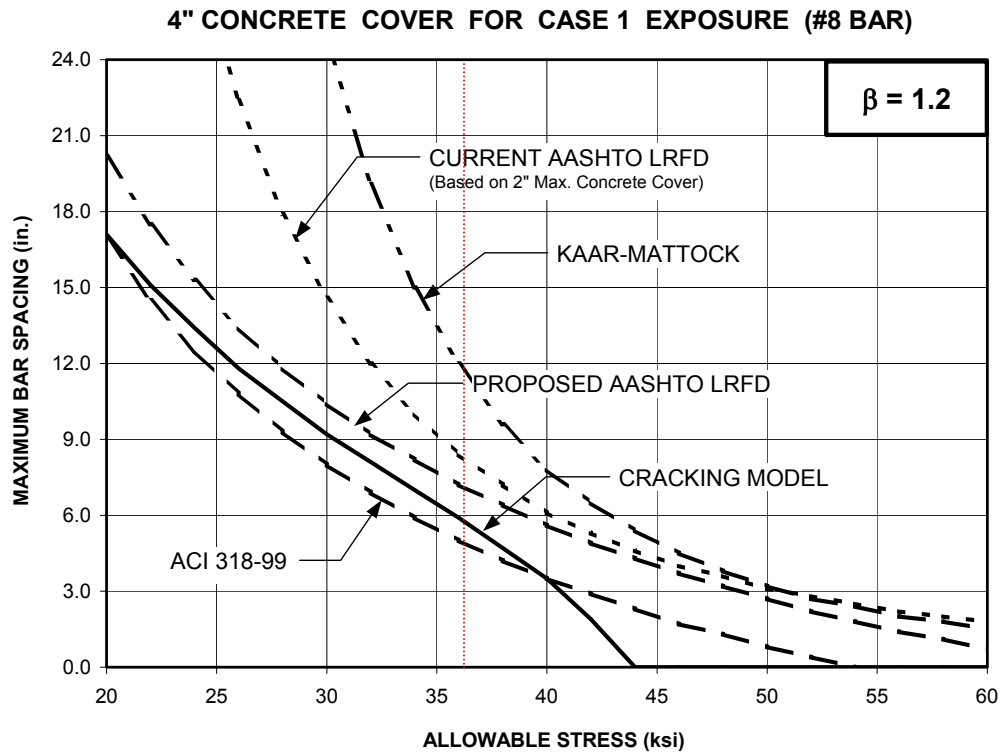


Fig. 3: Allowable Stress vs. Bar Spacing for Case 1 Exposure — 4" Concrete Cover

DISCUSSION OF β -FACTOR

One of the primary differences between the ACI 318-99 equation and the equation proposed for AASHTO involves explicitly including the β -factor in the crack control equation. The ACI equation does consider the β -factor, but in an implicit manner. The β -factor built into the development of the equation is:

$$\beta = 1 + 0.08 d_c \quad (8)$$

which was developed by Frosch based on a review of the sections from the crack control test data.

The β -factor simply represents the increase in crack width at the tension face of the member compared to the crack width at the centroid of the outer most layer of steel reinforcement assuming linear strain distribution. The theoretical definition of the β factor is:

$$\beta = \frac{\varepsilon_2}{\varepsilon_1} = \frac{h - c}{d - c} \quad (9)$$

A sketch showing the various parameters defining β is shown in Figure 4.

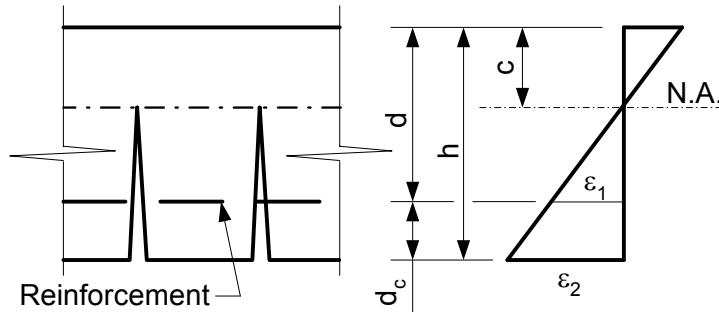


Fig. 4: β -Factor Definition

Figures 5, 6 & 7 show the significant effect of the β -factor because of the wide range of flexural member depths encountered in bridge construction. Box culvert slabs can be on the order of 8" thick while pier cap depths and footing thicknesses can be on the order of 7 feet and more. The β -factor effect is fairly dramatic for the 2-inch concrete cover case, but diminishes as the concrete cover increases primarily because thin, shallow flexural members are uncommon for the thicker cover cases. Take, for example, the case of 2" concrete cover for an 8"-thick flexural member in which $\beta = 1.65$. The allowable stress for reinforcement spaced at 12" is only 25 ksi (based on Case 1 exposure condition.) However, for a 72"-thick member in which $\beta = 1.05$, the allowable stress increases to approximately 39 ksi — a 56% increase just by considering the depth of the flexural member.

The effect of β has long been recognized by AASHTO as an important parameter in the crack control equation. However, since the inception of the provision there has been a general reluctance to explicitly include the term in the general equation even though the variables needed to compute the parameter are minimal and readily available. Instead, special cases were used. For example, the AASHTO equation has always had $\beta = 1.2$ implicitly included. But, in the 1982 Interim AASHTO Standard Specifications (Article 1.5.6), a Z-factor of 98 was introduced specifically and solely for box culverts. The commentary provided in the interim specification stated, "The expression for maximum service load stress for crack control more nearly reflects the Gergely and Lutz recommendations (SP-20 A.C.I. 1968, pp. 87-117) as confirmed by Lloyd, Rejali and Kesler (A.C.I. Journal No. 5, V. 66, May 1969, pp. 366-376) for one-way slabs than does the formula in Section 5." By applying the ratio of a $\beta = 1.2$ divided by $\beta = 1.6$ to the severe exposure condition value of $Z = 130$, the value of 98 is obtained.

In the AASHTO LRFD specifications, a value of $Z = 100$ was selected for box culverts. Unfortunately, this value for Z applied to all cases was recognized as being overly conservative for many cast-in-place box culvert applications in which slab and wall thicknesses typically exceed 12 inches. It is for this reason that subsequent to the 1st edition of the AASHTO LRFD specifications released in 1994, an expression for β was introduced. However, use of this equation was stipulated for cast-in-place box culverts only. $Z = 100$ continued to be specified for precast box culverts.

The expression for Z given in the current AASHTO LRFD specifications is:

$$Z = \frac{155}{\beta}, \text{ where } \beta = 1 + \frac{d_c}{0.7d} \tag{10}$$

Substituting $\beta = 1.2$ in the above equation results in the typical value of $Z = 130$ that has been used for severe exposure conditions since inception of the provision. The expression for β can be derived easily using an assumed value of 0.3 for 'k' in the familiar 'kd' term (the location of the neutral axis) from working stress design.

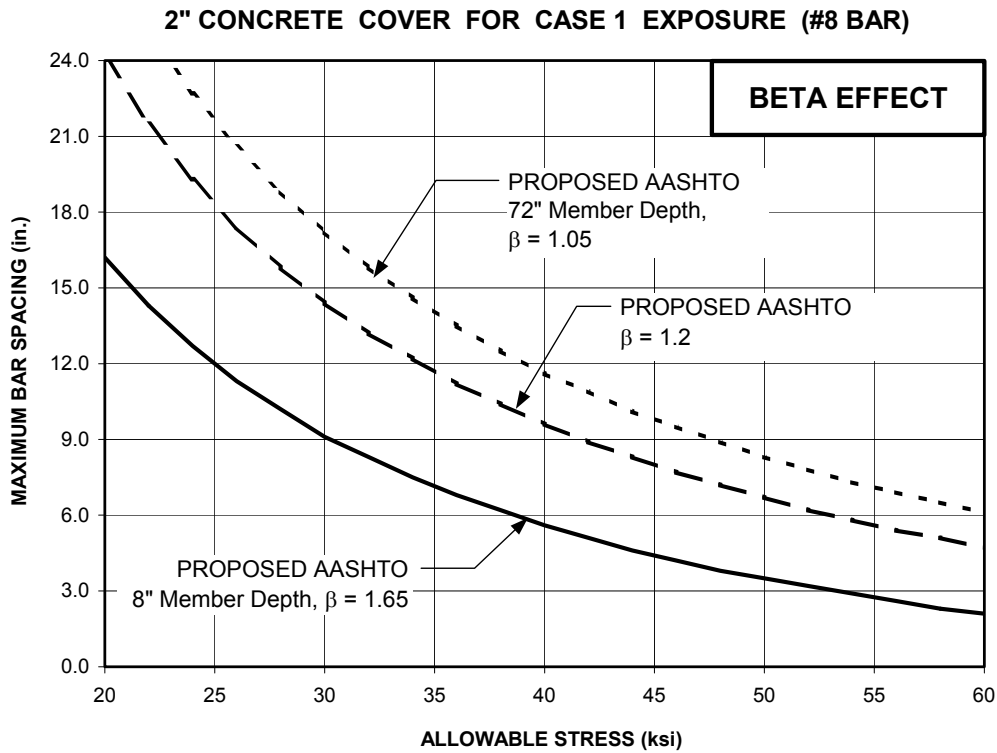


Fig. 5: Allowable Stress vs. Bar Spacing Demonstrating β Effect — 2" Concrete Cover

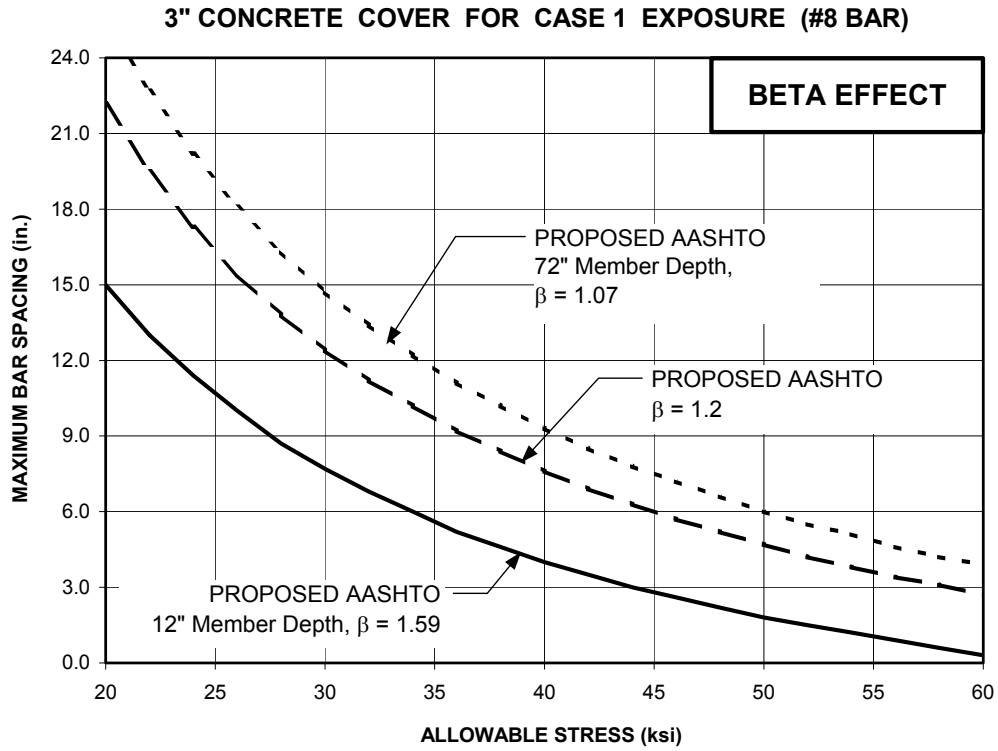


Fig. 6: Allowable Stress vs. Bar Spacing Demonstrating β Effect — 3" Concrete Cover

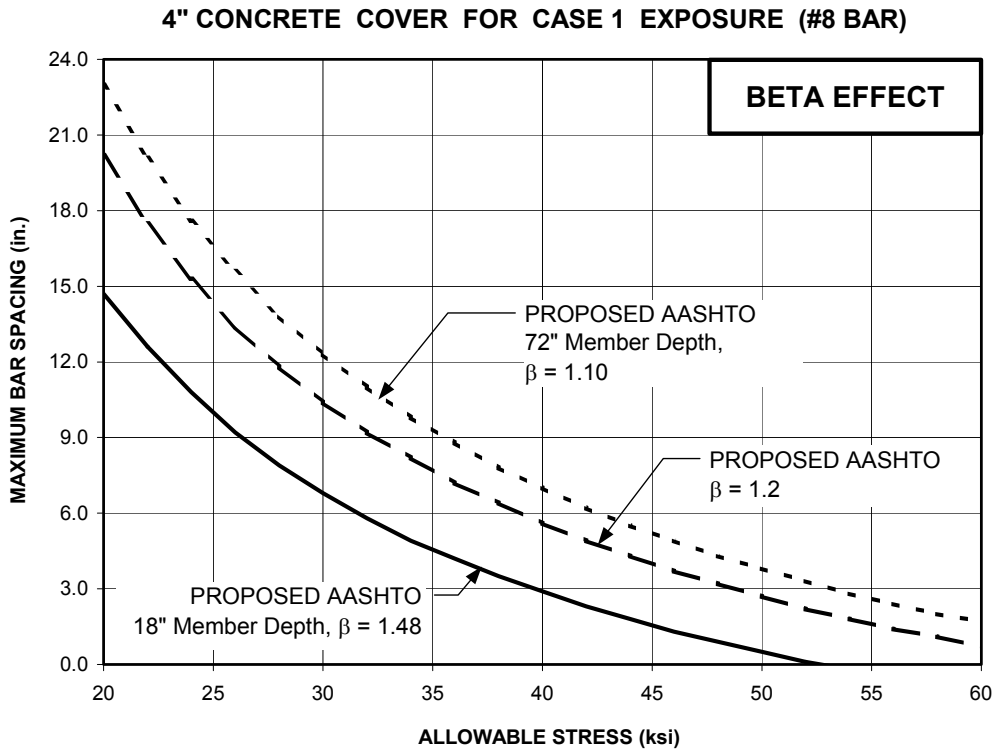


Fig. 7: Allowable Stress vs. Bar Spacing Demonstrating β Effect — 4" Concrete Cover

DISCUSSION OF EXPOSURE FACTOR

Another significant variation from the ACI 318-99 equation being proposed for AASHTO is continuing to differentiate between exposure conditions. ACI 318 decided to eliminate the previously held distinction between interior and exterior exposure conditions. Maintaining some form of distinction between different conditions was considered important for bridge construction because:

- Not all engineers are convinced there is no correlation between crack width and long-term durability.
- Some states definitely have much harsher exposure and environmental conditions (such as deicing chemical and salt spray) compared to other states. To eliminate all such distinction does not seem rational and is certainly inconsistent with past practice.
- Even if corrosion is not highly correlated to cracking, some situations can certainly tolerate higher levels of cracking compared to others. For example, cracking can be less tolerated for piers that are highly visible to the traveling public simply from a perception standpoint, even if it is known from a structural standpoint that the cracking will have no detrimental effect.

Instead of using the current terminology of 'normal' and 'severe' exposure conditions, the terms "Case 1" and "Case 2" are proposed. The intent of changing the terminology is to better reflect the purpose of the factor, which is to simply differentiate between situations in which more or less cracking can be tolerated by the owner.

The proposed value of 0.75 for the exposure factor, γ_e , was derived simply by taking the ratio of the current 130 and 170 Z-factors. Figures 8, 9, 10 and 11 show how the proposed crack control equation compares against the current AASHTO equation as well as the ACI 318-99 equation for the Case 2 exposure condition for concrete covers of 1.5", 2", 3" and 4" respectively. As can be seen, the ACI curve is the same as for the Case 1 exposure condition, again, because ACI does not make any distinction.

The largest deviation between the proposed equation and the current equation occurs for the 2-inch concrete cover case. Only for relatively wide bar spacings (exceeding approximately 13.5") does the current AASHTO equation permit a higher allowable stress than the proposed equation. For bar spacings less than 9", the proposed equation results in significantly higher allowable stresses compared to the current equation, yet, the allowable stresses remain significantly less than the ACI 318 equation.

As the concrete cover increases beyond 2", the current and proposed equations converge only because of the artificial 2" maximum cover limitation allowed by the current AASHTO LRFD specifications. Such an arbitrary limitation is not used for the proposed method. The actual cover is to be always used except for cases in which additional clearance is stipulated for construction tolerances. (An example of this condition in Pennsylvania occurs for the

bottom mat of reinforcement in footings. Because the concrete is cast directly on the earth subgrade that may be unevenly prepared, an additional 1" of cover is stipulated as a precautionary measure to ensure that 3" minimum cover is obtained.)

Only for the 4" concrete cover case does the current AASHTO equation result in a higher allowable stress than the proposed equation for the full range of bar spacings. However, for the normal range of bar spacings between 4" and 9", the difference in allowable stress is relatively small and considered acceptable especially considering the fact that the proposed equation does not use the arbitrary 2" maximum cover limitation. If the owner considers this reduction in allowable stress unacceptable, a possible viewpoint that could be taken is that by providing 4 inches of concrete cover, a Case 1 exposure condition could be used for computation purposes instead of the Case 2 exposure condition. It is this type of flexibility in interpretation of the crack control provisions that is considered to be a positive aspect of the proposed equation.

Under no condition is the allowable stress obtained from the proposed AASHTO equation greater than the allowable stress obtained from the ACI 318 equation.

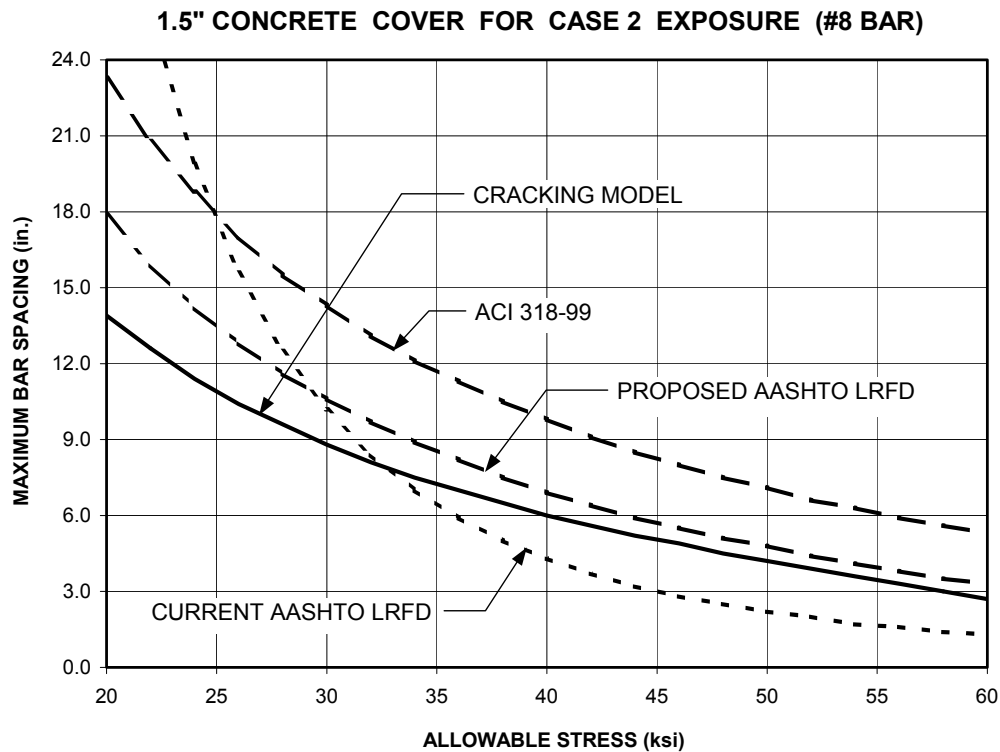


Fig. 8: Allowable Stress vs. Bar Spacing for Case 2 Exposure — 1.5" Concrete Cover

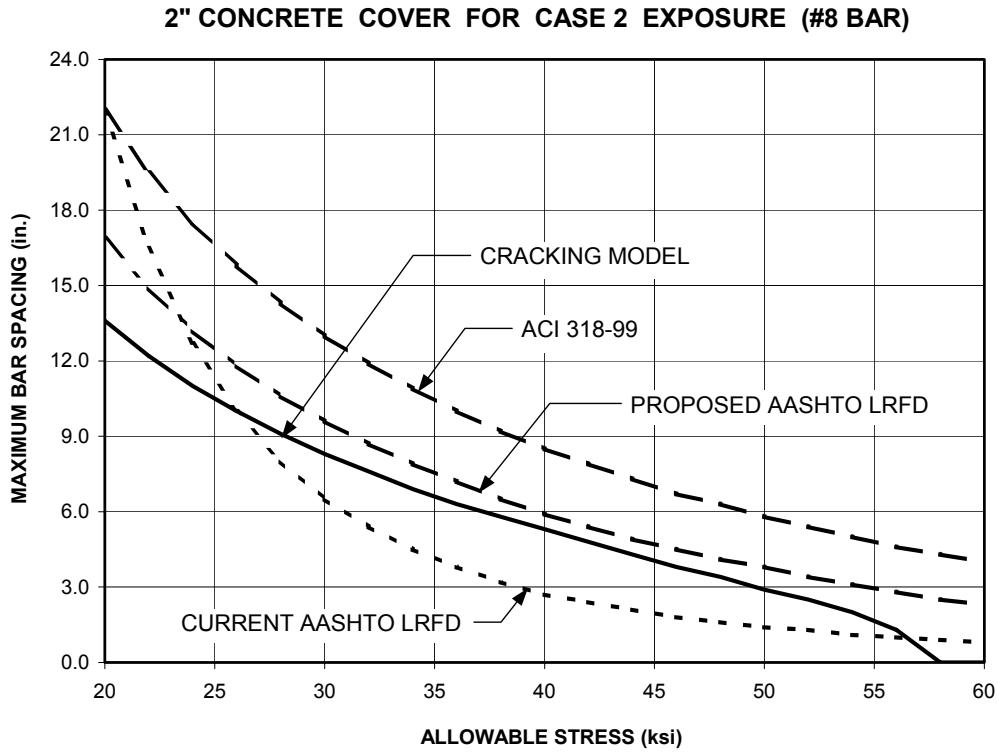


Fig. 9: Allowable Stress vs. Bar Spacing for Case 2 Exposure — 2" Concrete Cover

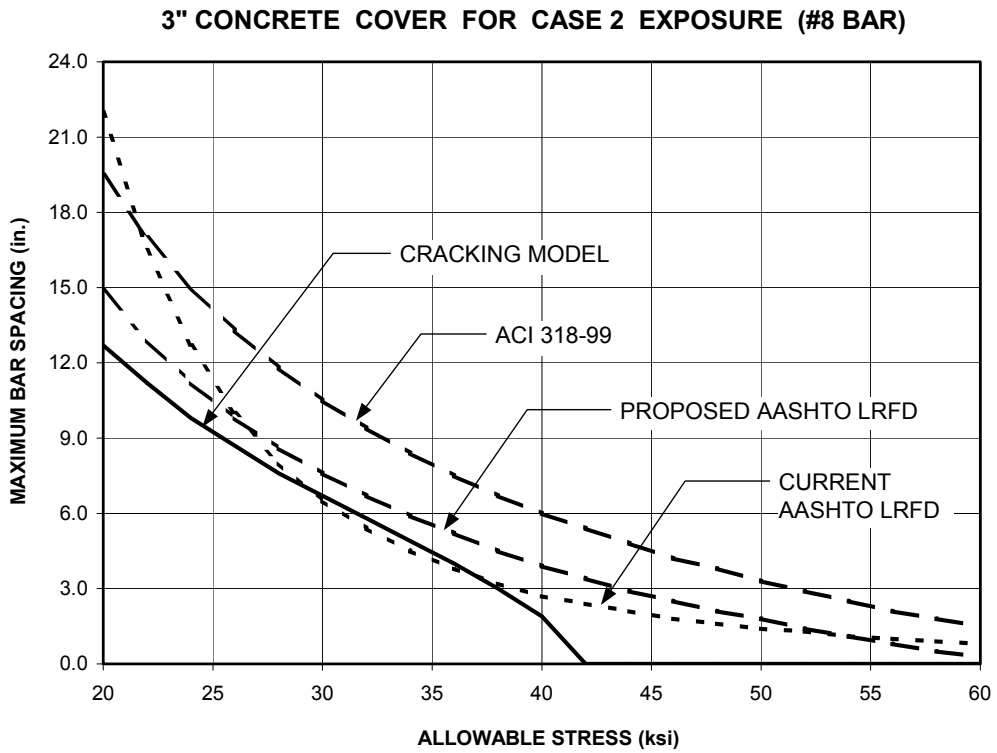


Fig. 10: Allowable Stress vs. Bar Spacing for Case 2 Exposure — 3" Concrete Cover

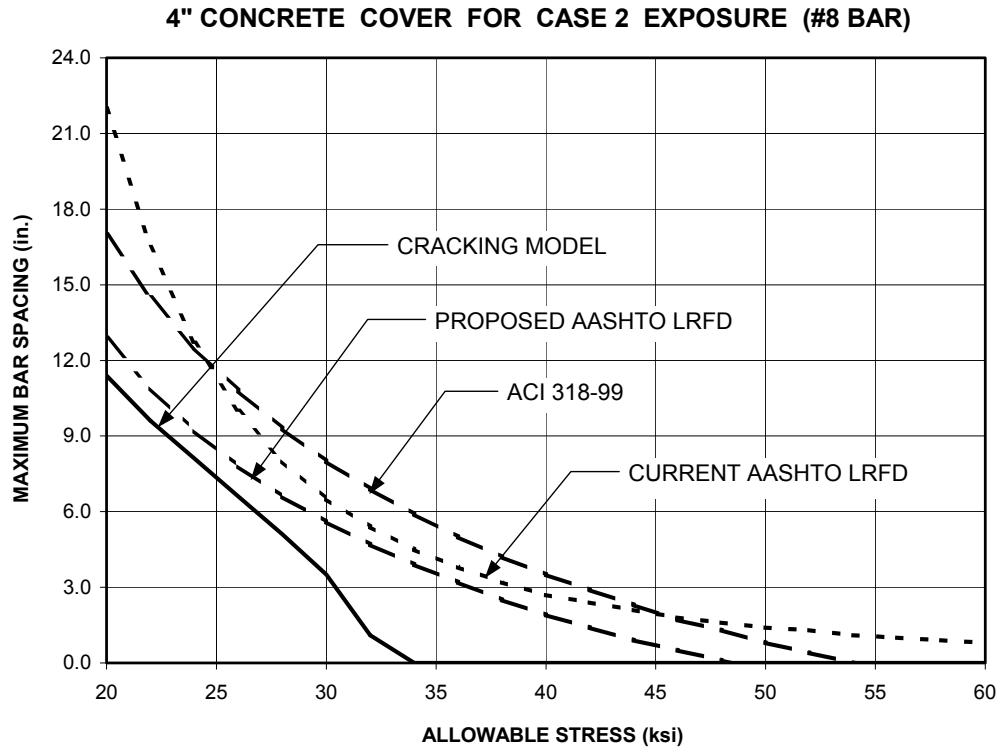


Fig. 11: Allowable Stress vs. Bar Spacing for Case 2 Exposure — 4" Concrete Cover

DISCUSSION OF CULVERTS

Of particular interest in revising the crack control equation is to treat precast and cast-in-place culverts in a consistent manner. Figures 12 and 13 show comparisons between the proposed AASHTO equation and the current equation for precast and cast-in-place culverts for member depths of 8" and 12" and for concrete covers of 1 1/2" and 2", respectively. A separate exposure condition for precast culverts, as currently implemented in AASHTO, is not proposed for the new provisions, because the β factor is explicitly included in the general crack control equation and a special case is considered unnecessary.

The results show that the proposed AASHTO equation permits higher allowable stresses than the current equation for both precast and cast-in-place culverts. The increase is particularly more for precast culverts than for cast-in-place culverts. Such increases in allowable stresses are not cause for concern, because the allowable stresses currently permitted by the AASHTO equation are unusually low and do not seem to be warranted. For example, for the 2" cover case, even with a relatively tight bar spacing of 6 inches, the allowable stress is only 24 ksi regardless of the slab thickness. Under the proposed provisions, the allowable stress would increase to 35 ksi for a 12" thick slab and 30 ksi for an 8" thick slab.

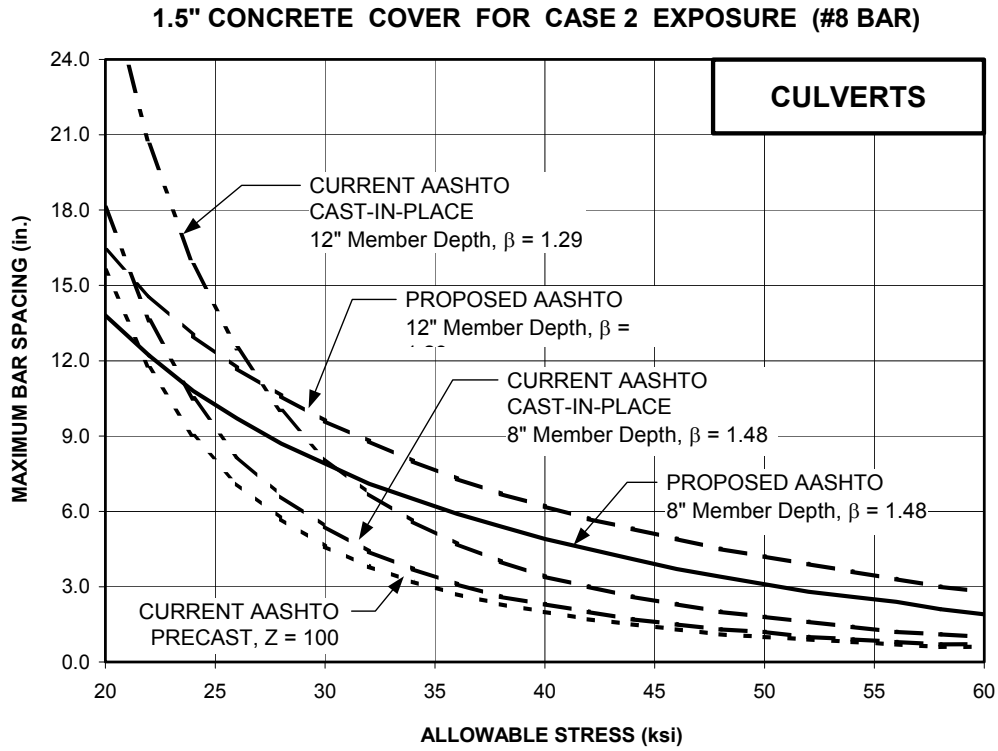


Fig. 12: Allowable Stress vs. Bar Spacing for Culverts — 1.5" Concrete Cover

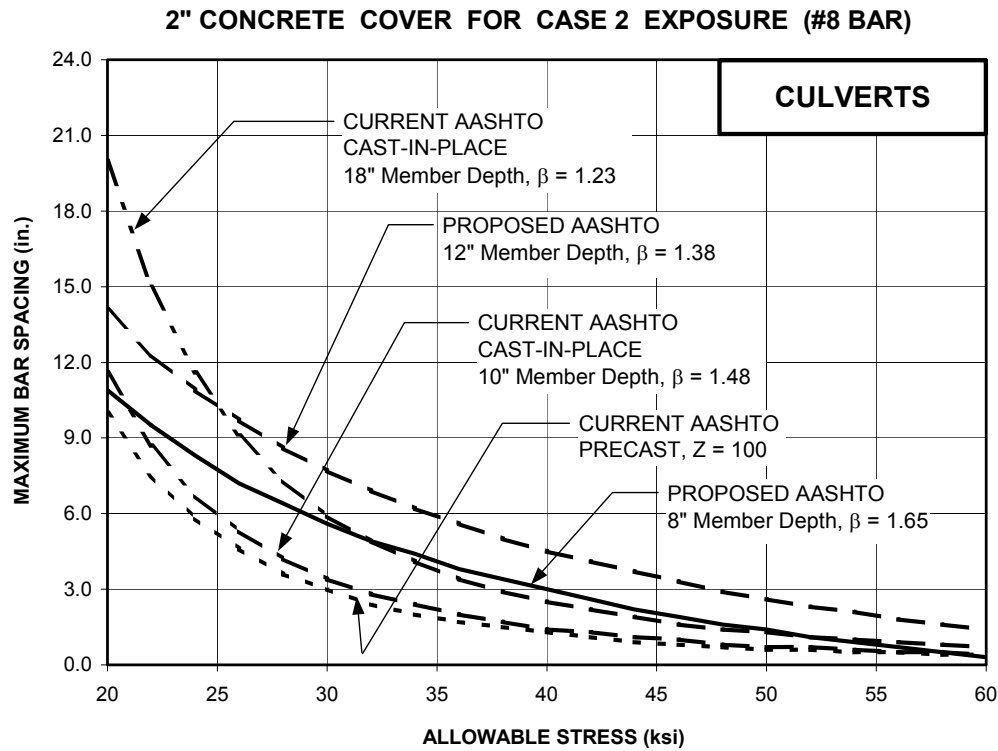


Fig. 13: Allowable Stress vs. Bar Spacing for Culverts — 2" Concrete Cover

DESIGN EXAMPLES

Several design examples are provided for the most common serviceability computations encountered in bridge design. The intent is to demonstrate the simplicity of the proposed method and to provide a direct comparison in the allowable stress results between the current AASHTO method and the proposed method.

EXAMPLE 1 — CAST-IN-PLACE BOX CULVERT

Given:

Member depth, $h = 12''$

Concrete cover, $c_c = 2''$

Reinforcement, #6 @ 8''

Current AASHTO Method:

$$d_c = 2'' + 0.75'' / 2 = 2.375''$$

$$\text{Effective depth, } d = 12'' - 2.375'' = 9.625''$$

$$Z = \frac{155}{\beta}$$

$$\beta = 1 + \frac{d_c}{0.7d} = 1 + \frac{2.375}{0.7 \times 9.625} = 1.35$$

$$Z = \frac{155}{1.35} = 114.8$$

$$A = 2 d_c s = 2 \times 2.375 \times 8 = 38.0 \text{ in}^2$$

$$f_{sa} = \frac{Z}{\sqrt[3]{d_c A}} = \frac{114.8}{\sqrt[3]{2.375 \times 38.0}} = 25.6 \text{ ksi}$$

Proposed AASHTO Method:

$$d_c = 2'' + 0.75'' / 2 = 2.375''$$

$$\text{Effective depth, } d = 12'' - 2.375'' = 9.625''$$

Exposure factor, $\gamma_e = 0.75$ for culverts.

Rebar factor, $\gamma_r = 1.00$ for deformed reinforcement

$$\beta = 1 + \frac{d_c}{0.7(h - d_c)} = 1 + \frac{2.375}{0.7(12 - 2.375)} = 1.35$$

$$f_{sa} = \frac{700 \gamma_e \gamma_r}{\beta(s + 2d_c)} = \frac{700 \times 0.75 \times 1.00}{1.35(8 + 2 \times 2.375)} = 30.5 \text{ ksi}$$

EXAMPLE 2 — PRECAST BOX CULVERT

Given:

Member depth, $h = 12''$
 Concrete cover, $c_c = 1.5''$
 Reinforcement, #6 @ 8''

Current AASHTO Method:

$$d_c = 1.5'' + 0.75'' / 2 = 1.875''$$

$$\text{Effective depth, } d = 12'' - 1.875'' = 10.125''$$

$$Z = 100$$

$$A = 2 d_c s = 2 \times 1.875 \times 8 = 30.0 \text{ in}^2$$

$$f_{sa} = \frac{Z}{\sqrt[3]{d_c A}} = \frac{100}{\sqrt[3]{1.875 \times 30.0}} = 26.1 \text{ ksi}$$

Proposed AASHTO Method:

$$d_c = 1.5'' + 0.75'' / 2 = 1.875''$$

$$\text{Effective depth, } d = 12'' - 1.875'' = 10.125''$$

$$\text{Exposure factor, } \gamma_e = 0.75 \text{ for culverts.}$$

$$\text{Rebar factor, } \gamma_r = 1.00 \text{ for deformed reinforcement}$$

$$\beta = 1 + \frac{d_c}{0.7(h - d_c)} = 1 + \frac{1.875}{0.7(12 - 1.875)} = 1.26$$

$$f_{sa} = \frac{700 \gamma_e \gamma_r}{\beta(s + 2d_c)} = \frac{700 \times 0.75 \times 1.00}{1.26(8 + 2 \times 1.875)} = 35.5 \text{ ksi}$$

EXAMPLE 3 — ABUTMENT STEM, REAR FACE STEEL

Given:

Member depth, $h = 30''$
 Concrete cover, $c_c = 3''$
 Reinforcement, #10 @ 12''

Current AASHTO Method:

AASHTO Art. 5.7.3.4 stipulates using a maximum concrete cover of 2'' regardless of the actual cover when computing d_c .

$$d_c = 2.0'' + 1.27'' / 2 = 2.64''$$

$$Z = 130 \text{ (severe exposure condition)}$$

$$A = 2 d_c s = 2 \times 2.64 \times 12 = 63.4 \text{ in}^2$$

$$f_{sa} = \frac{Z}{\sqrt[3]{d_c A}} = \frac{130}{\sqrt[3]{2.64 \times 63.4}} = 23.6 \text{ ksi}$$

Proposed AASHTO Method:

$$d_c = 3.0" + 1.27" / 2 = 3.64" \text{ (Note: the actual concrete cover is used.)}$$

$$\text{Exposure factor, } \gamma_e = 0.75 \text{ (Case 2 condition selected.)}$$

$$\text{Rebar factor, } \gamma_r = 1.00 \text{ for deformed reinforcement}$$

$$\beta = 1 + \frac{d_c}{0.7(h - d_c)} = 1 + \frac{3.64}{0.7(30 - 3.64)} = 1.20$$

$$f_{sa} = \frac{700 \gamma_e \gamma_r}{\beta(s + 2d_c)} = \frac{700 \times 0.75 \times 1.00}{1.20(12 + 2 \times 3.64)} = 22.7 \text{ ksi}$$

Change Reinforcement Design:

Try #7 @ 6"

Current AASHTO Method:

$$d_c = 2.0" + 0.875" / 2 = 2.44"$$

$$A = 2 d_c s = 2 \times 2.64 \times 6 = 29.3 \text{ in}^2$$

$$f_{sa} = \frac{Z}{\sqrt[3]{d_c A}} = \frac{130}{\sqrt[3]{2.44 \times 29.3}} = 31.3 \text{ ksi}$$

Proposed AASHTO Method:

$$d_c = 3.0" + 0.875" / 2 = 3.44"$$

$$\beta = 1 + \frac{d_c}{0.7(h - d_c)} = 1 + \frac{3.44}{0.7(30 - 3.44)} = 1.19$$

$$f_{sa} = \frac{700 \gamma_e \gamma_r}{\beta(s + 2d_c)} = \frac{700 \times 0.75 \times 1.00}{1.19(6 + 2 \times 3.44)} = 34.3 \text{ ksi}$$

EXAMPLE 4 — PIER CAP

Given:

Member depth, $h = 96"$; Cap width = 48"

Concrete cover, $c_c = 2" + 5/8"$ stirrup = 2.63"

Reinforcement,

Row 1 → 6 – bundled (2 bar) #10's

Row 2 → 6 – single #10's with 3" clear distance from Row 1.

Current AASHTO Method:

$$d_c = 2" \text{ max cover} + 5/8" + 1.27 = 3.90"$$

Centroid of reinforcement:

$$\bar{y} = 12 \times 3.90" + 6 \times (3.90" + 1.27 + 3" \text{ clr} + 1.27/2) / 18 = 5.54"$$

Effective concrete area per bar:

For bundled bars, use 1.4 equivalency factor.

$$A = 48'' \text{ cap width} \times (2 \times 5.54'') / (6 \text{ bundles} \times 1.4 \text{ eq. factor} + 6) = 36.9 \text{ in}^2$$

$$f_{sa} = \frac{Z}{\sqrt[3]{d_c} A} = \frac{130}{\sqrt[3]{3.90 \times 36.9}} = 24.8 \text{ ksi}$$

Proposed AASHTO Method:

$$d_c = 2.0'' + 5/8'' \text{ stirrup} + 1.27'' = 3.90''$$

Exposure factor, $\gamma_e = 0.75$ (Case 2 exposure condition selected.)

Rebar factor, $\gamma_r = 1.00$ for deformed reinforcement.

Reinforcement spacing,

$$s = \frac{48'' \text{ wide cap}}{6 \text{ bar bundles in row nearest face}} = 8''$$

$$\beta = 1 + \frac{d_c}{0.7(h - d_c)} = 1 + \frac{3.90}{0.7(96 - 3.90)} = 1.06$$

$$f_{sa} = \frac{700 \gamma_e \gamma_r}{\beta(s + 2d_c)} = \frac{700 \times 0.75 \times 1.00}{1.06(8 + 2 \times 3.90)} = 31.3 \text{ ksi}$$

EXAMPLE 5 — PIER FOOTING

Given:

Member depth, $h = 48''$

Concrete cover, $c_c = 3''$ (4'' actual but 1'' considered construction tolerance)

Reinforcement in bottom mat,

Bottom layer → #10 @ 9''

Top layer → #10 @ 9''

Bottom Layer — Current AASHTO Method:

AASHTO Art. 5.7.3.4 stipulates using a maximum concrete cover of 2'' regardless of the actual cover when computing d_c .

$$d_c = 2.0 + 1.27'' / 2 = 2.64''$$

$Z = 170$ (normal exposure condition)

$$A = 2 d_c s = 2 \times 2.64 \times 9 = 47.5 \text{ in}^2$$

$$f_{sa} = \frac{Z}{\sqrt[3]{d_c} A} = \frac{170}{\sqrt[3]{2.64 \times 47.5}} = 34.0 \text{ ksi}$$

Bottom Layer — Proposed AASHTO Method:

$$d_c = 3.0'' + 1.27'' / 2 = 3.64''$$

Exposure factor, $\gamma_e = 1.00$ (Case 1 exposure condition selected.)

Rebar factor, $\gamma_r = 1.00$ for deformed reinforcement

$$\beta = 1 + \frac{d_c}{0.7(h - d_c)} = 1 + \frac{3.64}{0.7(48 - 3.64)} = 1.12$$

$$f_{sa} = \frac{700 \gamma_e \gamma_r}{\beta(s + 2d_c)} = \frac{700 \times 1.00 \times 1.00}{1.12(9 + 2 \times 3.64)} = 38.4 \text{ ksi}$$

Top Layer — Current AASHTO Method:

$$d_c = 2.0 + 1.27" + 1.27" / 2 = 3.91"$$

$$Z = 170 \text{ (normal exposure condition)}$$

$$A = 2 d_c s = 2 \times 3.91 \times 9 = 70.4 \text{ in}^2$$

$$f_{sa} = \frac{Z}{\sqrt[3]{d_c A}} = \frac{170}{\sqrt[3]{3.91 \times 70.4}} = 26.1 \text{ ksi}$$

Bottom Layer — Proposed AASHTO Method:

$$d_c = 3.0" + 1.27 + 1.27" / 2 = 4.91"$$

Exposure factor, $\gamma_e = 1.00$ (Case 1 exposure condition selected.)

Rebar factor, $\gamma_r = 1.00$ for deformed reinforcement

$$\beta = 1 + \frac{d_c}{0.7(h - d_c)} = 1 + \frac{4.91}{0.7(48 - 4.91)} = 1.16$$

$$f_{sa} = \frac{700 \gamma_e \gamma_r}{\beta(s + 2d_c)} = \frac{700 \times 1.00 \times 1.00}{1.16(9 + 2 \times 4.91)} = 32.1 \text{ ksi}$$

Table 1: Summary of Design Example Results

| Member | Specifics | Reinforcement | Current | Proposed |
|---------------------------|---|---|----------------------|----------------------|
| Cast-in-place Box Culvert | 12" thick slab, 2" concrete cover | #6 @ 8" | 25.6 ksi | 30.5 ksi |
| Precast Box Culvert | 12" thick slab, 1 1/2" concrete cover | #6 @ 8" | 26.1 ksi | 35.5 ksi |
| Abutment Stem, R.F. | 30" thick section, 3" concrete cover | #10 @ 12" #7 @ 6" | 23.6 ksi 31.3 ksi | 22.7 ksi 34.3 ksi |
| Pier Cap | 48" wide, 96" deep, 2" concrete cover, #5 stirrups | Row 1: 6 – 2 bar #10 Row 2: 6 – 1 bar #10 | 24.8 ksi | 31.3 ksi |
| Pier Footing | 48" thick, 3" concrete cover, bottom mat | Bottom layer: #10 @ 9" Top layer: #10 @ 9" | 34.0 ksi 26.1 ksi | 38.4 ksi 32.1 ksi |

CONCLUSIONS

The proposed revised distribution of flexural reinforcement equation satisfactorily addresses a number of shortcomings identified with the current Z-factor method as implemented in the AASHTO LRFD specifications. A logical and rational, yet simple, physical cracking model developed by Frosch, that has been shown to correlate well with the database of existing experimental data, is used to predict crack width. While additional concrete cover does reduce the allowable stress, the reduction is not nearly as severe as with the current method. No longer is the concrete cover arbitrarily limited to a maximum value of 2 inches in order to obtain reasonable results. Furthermore, precast box culverts are handled the same as cast-in-place box culverts in that the β effect is an integral part of the equation instead of being applied to cast-in-place boxes only. Deeper flexural members will also benefit because the β value is typically 10% – 15% smaller than the assumed value of 1.2 built into the existing equation.

An exposure factor, γ_e , defined as Case 1 or Case 2, is incorporated into the equation in order to account for varying structural applications found in bridge construction, be it from an aesthetics or corrosion standpoint. The factor provides the opportunity to restrict or relax the crack control criteria based on a particular application as determined by the owner.

Example designs were performed to compare the allowable stresses between the existing Z-method and the proposed crack control equation. The results show reasonable increases in allowable stresses, thus permitting more economical designs without sacrificing long-term durability or aesthetics.

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