Tension Field Action in Hybrid Steel Girders

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ABSTRACT

With the advent of HPS70W (70 ksi) steel, several bridges have been built, and many more will be built, using hybrid design provisions. One limit with hybrid girder design, which decreases the beneficial aspects, is that tension field action (TFA) is not allowed when determining the shear capacity. This is a severe shear capacity penalty for using hybrid girders. This paper proposes that tension field action should be allowed for hybrid girders and presents a modified moment-shear interaction. Hybrid tension field action would allow shear capacities well above what is currently allowed by AASHTO. With an increase in shear capacity, fewer transverse stiffeners will be necessary. A decrease in stiffeners allows for a more economical design of hybrid plate girders, substantial cost savings in material and fabrication and fewer fatigue details. The paper also presents planned experimental and finite element studies that are expected to verify the development of tension field action in hybrid girders and the capability of the proposed modified moment-shear interaction equation as a lower bound strength estimate.

INTRODUCTION

Decades ago, hybrid girders were a popular choice for bridge girders. Using 50-ksi material for the flanges with a lower cost 36-ksi web material yielded more economical results while still maintaining a flexural capacity close to that of a homogeneous 50-ksi girder. Since that time, with the cost gap between the two strength materials dwindling, the economical benefit of hybrid girders also dwindled. However, with the advent of High Performance Steel, hybrid design is sure to be a common practice again. Bridge studies (Schrage, 1999) have shown that the most beneficial use of HPS70W (70 ksi) is in the flanges of hybrid girders with 50-ksi webs.

The AASHTO LRFD Specifications (AASHTO, 1998) have been updated to allow HPS70W steel in bridges. Studies have shown that the current design specifications are adequate for HPS70W and the issues of ductility and buckling are sufficiently considered (Barth, White, and Bobb, 2000). Therefore, bridges have been built, and many more will be built, with HPS70W material. A majority of these bridges will use hybrid girders.

One limit with hybrid girder design, which decreases the beneficial aspects, is that tension field action (TFA) is not allowed when determining the shear capacity. The reason is that, in hybrid girders, the web is assumed to yield near maximum moment, which may affect the tension strut assumed for TFA. Thus, hybrid girder shear capacity has historically been and currently is limited to the shear buckling capacity and any TFA occurring is ignored. This results in many more transverse stiffeners being required (closer spacing) for a hybrid girder than that for a homogeneous girder. This not only increases material costs, but also significantly increases fabrication costs. However, common sense tells one that a hybrid girder can certainly develop TFA when the moment is low (it is just a homogeneous girder with stronger flanges) and so the concern for TFA in hybrid girders is similar to the moment-shear interaction problem for homogeneous girders. The AASHTO specifications contain a design check for homogeneous girders when both the shear and moment are high when using TFA. The question is: at what point does the bending moment affect the shear capacity in a hybrid girder?

This paper presents the theory that the shear capacity of hybrid girders should include the tension field action component. The original TFA derivation (Basler, 1961a) and moment-shear interaction (Basler, 1961b) used in AASHTO is modified to account for the hybrid action. The modification has two parts: determining the shear capacity remaining (shear buckling plus TFA) with a reduced web depth due to web yielding in flexure associated with high moment; and determining the moment capacity remaining when the shear is high. The result is that, although there is a small theoretical penalty in the moment-shear interaction for using hybrid girders compared to homogeneous girders, the TFA component greatly increases the shear capacities for hybrid girders. Allowing the use of tension field action for hybrid girders would lead to more economical designs, less fabrication, and fewer fatigue details in the finished product.

SHEAR CAPACITY

Shear Buckling and Tension Field Action

The design code analyzed and discussed in this paper is the AASHTO *LRFD Bridge Design Specifications*, 2nd Edition

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(AASHTO, 1998). The shear capacity of a transversely stiffened plate girder is composed of two parts: the buckling strength of the web and the panel post-buckling strength. As a transversely stiffened plate girder is loaded in vertical shear, normal compressive and tensile stresses form in the web. The web eventually buckles between the transverse stiffeners due to the compressive stresses. In current AASHTO LRFD Specifications, the vertical shear buckling capacity, V_{cr}, is given as:

$$V_{cr} = \frac{\tau_{cr}}{\tau_y} V_{\mathrm{p}} = CV_p \tag{1}$$

where
τ_{cr} = shear buckling stress
τ_y = $0.58F_{yw}$ = shear yield stress
C = ratio of shear buckling stress to shear yield stress
V_p = $0.58F_{yw}Dt_w$ = plastic shear capacity of web
F_{yw} = yield stress of the web
D = height of web
t_w = web thickness

After the shear buckling load is exceeded, buckled waves tend to become more and more evident within the web panel. These waves are parallel to the tensile membrane stresses and are diagonal to the transverse stiffeners. Once buckled, the web theoretically cannot support any additional diagonal compressive stresses, which are assumed to transfer to the flanges and transverse stiffeners. After web buckling, additional shear forces are resisted in the manner of a Pratt Truss. The diagonal tensile forces in the web form a tension stress band that runs diagonally between the transverse stiffeners. The tension field is assumed to anchor to the transverse stiffeners. The vertical component of the tension stress band gives the web panel additional vertical shear strength beyond the point of buckling, and is called tension field action. The AASHTO TFA component, V_{tfa}, is given in U.S. Specifications in the following general form:

$$V_{tfa} = \frac{0.87V_p\left(1 - \frac{\tau_{cr}}{\tau_y}\right)}{\sqrt{1 + \left(\frac{d_o}{D}\right)^2}} \tag{2}$$

where
d_0 = transverse stiffener spacing

The additional shear strength due to tension field action, V_{tfa}, is added to the shear buckling capacity of the web panel, V_{cr}, to form the nominal shear capacity, V_n, of a transversely stiffened plate girder

$$V_n = RV_p\left[C + \frac{0.87(1 - C)}{\sqrt{1 + \left(\frac{d_o}{D}\right)^2}}\right] \tag{3}$$

where R is a reduction of shear capacity for moment-shear interaction.

The TFA component can be used if the panel containing the tension strut is not an end panel (anchoring of the strut), if the stiffeners are spaced close enough (d_0/D ratio), and if the girder is not hybrid. The factored applied shear, V_u, is limited to the nominal shear capacity, V_n.

Moment-Shear Interaction

A concern when using TFA in the shear capacity is if flexural web stresses are large due to moment (moment-shear interaction). AASHTO (1998) uses the moment-shear interaction shown in Figure 1. If the moment exceeds 75 percent of the flexural yield capacity, the web is not able to obtain its nominal buckling and TFA shear capacity. The R factor is used to reduce the shear capacity for a given moment according to Figure 1:

$$R = 0.6 + 0.4\left(\frac{M_n - M_u}{M_n - 0.75M_y}\right) \tag{4}$$

where
M_n = factored flexural resistance
M_u = factored flexural moment
M_y = yield moment based on the flange yield stress

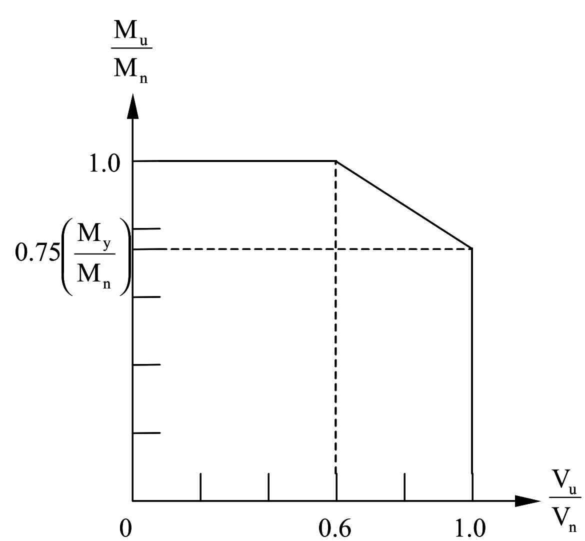


Fig. 1. AASHTO Interaction Curve.

Equation 4 is the moment-shear interaction reduction R, AASHTO Equation 6.10.7.3.3a-3 simplified for use here, and is multiplied by the nominal shear capacity when

$$M_u \geq 0.75 M_y \tag{5}$$

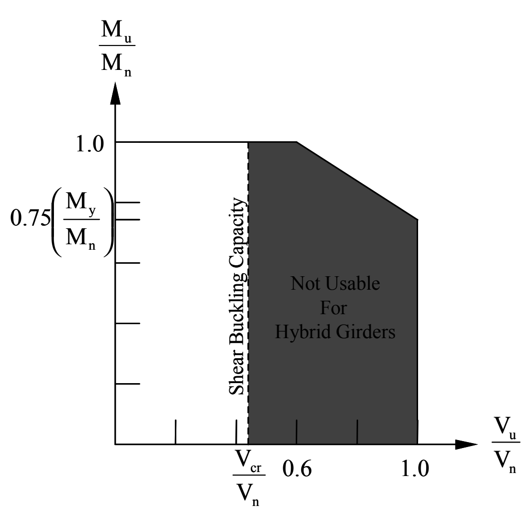
The R equation for noncompact sections (AASHTO Equation 6.10.7.3.3b-3) uses stresses instead of moments.

Current Shear Capacity of Hybrid Girders

AASHTO C6.10.7.3.4 states: "Tension-field action is not permitted for hybrid sections. Thus, the nominal shear resistance is limited to either the shear yield or the shear buckling force." This is a severe shear capacity penalty for using hybrid girders. Whereas homogeneous girders can significantly increase their shear capacities by using tension field action, hybrid girders are limited to the shear buckling capacity. Figure 2 is the moment-shear interaction for a hybrid girder assuming tension field action is applicable. Current specifications limit the useable shear capacity to the shear buckling capacity as shown. It is clear from this figure that significant benefits could be attained if TFA could be used for hybrid girders.

USING TENSION FIELD ACTION FOR HYBRID GIRDERS

AASHTO's exclusion of TFA for hybrid girders is conservative. It is true that at high moment, a portion of the web yields, but at low moment no web yielding occurs under flexural stresses and full tension field action should be possible. Even at maximum moment, with the flange at maximum stress, the web surely has significant shear capacity.



Note: V_n represents shear capacity if TFA was apllicable.

Fig. 2. Hybrid vs. Shear-Moment Interaction Curve.

Therefore, the question becomes at what moment are there adverse affects from web yielding that would cause a reduction in the shear capacity. Thus, it is the same question that prompted the moment-shear interaction requirements for homogeneous girders. The objective of this section is to develop a new moment-shear interaction requirement, which would allow tension field action for hybrid girders.

Basler developed the shear capacity (Basler, 1961a) and moment-shear interaction (Basler, 1961b) equations adopted by the AASHTO Specifications. Although there have been many studies since his work, the Basler equations have been used for decades and have resulted in good performance. Therefore, staying with common practice, the equations have been used to develop design criteria for hybrid girders. Hurst (2000) fully develops the modifications to Basler's original work to determine the shear capacity of hybrid girders and examines other's work as it relates. The Basler work is summarized below in order to present the modifications in later sections. The shear buckling component (Equation 1) is not discussed since it does not play a part in the modifications.

Basler's Shear Capacity Theory

Basler (1961a) assumes that, in a post-buckled state, a field of uniform inclined tension stresses σ_t flows through a bandwidth s of the web's cross section at an angle ϕ from the bottom flange as shown in Figure 3a.

Basler assumes a succession of equal web panels all subjected to the same shear force as shown in Figure 3b. A cut is made along sections A, B, and C to obtain the free body diagram in Figure 3c. At the face of A in the web, an unknown resultant is acting which Basler decomposes into a normal component F_w and a shear force component which, because of the symmetry of the chosen cut, must be $V_\sigma/2$. The force acting in the flanges is denoted as F_f with changes of ΔF_f. At cut C, the compressive stiffener force F_s is acting along with the tension field stresses, σ_t, inclined at an angle ϕ from the horizontal. The angle ϕ is the optimum angle that gives the maximum value of tension stresses. Basler uses equilibrium to obtain the vertical component of the tension field

$$V_\sigma = \frac{D}{2}\sigma_t t_w \left(\frac{1}{\sqrt{1+\left(\frac{d_o}{D}\right)^2}} \right) \tag{6}$$

Basler states that, in plate girders with slender webs, neither pure beam action (τ) nor pure tension field action (σ) occur alone, but rather that shear is resisted by a combination of both. Since the ultimate shear load is a combination of pure beam action and pure tension field action, Basler gives

$$V_n = V_\tau + V_\sigma \tag{7}$$

In order to compute these two shear contributions, Basler makes two assumptions. The first is that the superposition of pure beam action and pure tension field action are limited by the Hencky von-Mises yield criterion (using a conservative value of 45° for ϕ in Figure 3). The second assumption is that, up to V_τ, shear is carried in a beam type manner, but after the web buckles, V_τ remains constant. Additional shear resistance above the buckling shear is carried by tension field action. Thus,

$$V_\tau = V_{cr} = \tau_{cr} D t_w = V_p \left(\tau_{cr} / \tau_y \right) \tag{8}$$

Using a linear approximation for the Hencky-von Mises Yield Criterion in the region in question yields the following result for the maximum possible tension field stress σ_t as

$$\frac{\sigma_t}{F_{yw}} = 1 - \frac{\tau_{cr}}{\tau_y} \tag{9}$$

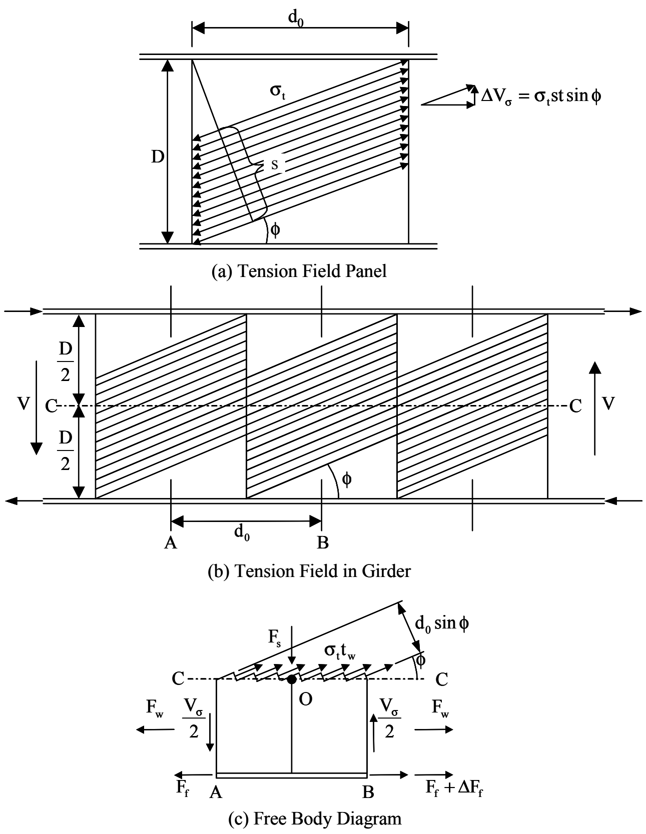


Fig. 3. Basler's Tension Field Action Theory.

which, when combining Equations 6, 7 and 8 to calculate the ultimate shear force for a plate girder, gives

$$V_n = V_p \left[\frac{\tau_{cr}}{\tau_y} + \frac{\sqrt{3}}{2} \frac{1 - \frac{\tau_{cr}}{\tau_y}}{\sqrt{1 + \left(\frac{d_0}{D} \right)^2}} \right] \tag{10}$$

This is the AASHTO shear capacity as shown in Equation 3.

Reduced Web Depth Modification

One concern with hybrid girders is that the shear capacity may be reduced if the web yields. Hurst (2000) modifies Basler's TFA Equation 10 using a reduced web depth that considers web yielding due to flexure. Figure 4 illustrates the modified tension field action panel. In Figure 4,

F_{yw} = yield stress of the web material,
σ_f = stress in flange due to moment, and
D_r = reduced web depth in which the TFA is assumed to form.

Using an approximation for the reduced TFA depth of the web,

$$D_r = \frac{F_{yw} D}{\sigma_f} \tag{11}$$

and the flange stress,

$$\sigma_f = \frac{M_u}{S_x} \tag{12}$$

where
S_x = elastic section modulus, the web depth for TFA, D_r, becomes

$$D_r = \frac{F_{yw} S_x D}{M_u} \tag{13}$$

This is a conservative estimate of the web yielding due to the approximation in Equation 11. There is yielding in the web anytime,

$$\sigma_f \geq F_{yw} \tag{14}$$

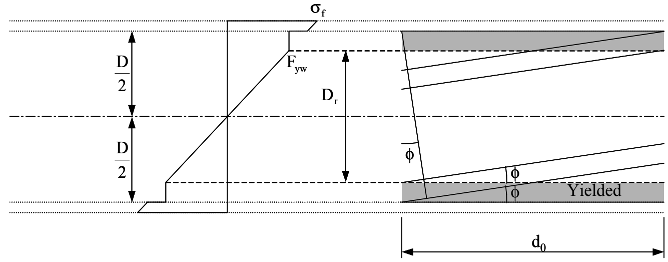


Fig. 4. Web Panel Showing Yielded Portion of the Web.

If a portion of the web yields near the flanges, it is assumed that the diagonal tension field stresses, which form after web buckling, are not able to utilize the entire height of the web panel. The tension strut, illustrated in Figure 4, must be reduced. Using the “effective web height” can accomplish this. In Basler’s (1961a) derivation of shear capacity, the capacity is composed of a shear buckling and tension field action component. To calculate the total shear capacity with an effective web height for the tension field, Basler’s derivation was re-executed with the effective web height D_r used for the definition of the tension strut (Hurst, 2000). Since the buckling capacity is not affected, the final equation for the shear capacity with a reduced effective web height is just a small modification to Basler’s Equation 10:

$$V_n = V_p \left[\frac{\tau_{cr}}{\tau_y} + \frac{\sqrt{3}}{2} \frac{1 - \frac{\tau_{cr}}{\tau_y}}{\sqrt{1 + \left(\frac{d_0}{D_r} \right)^2}} \right] \tag{15}$$

Thus, using this conservative modification for the tension field action component (V_p remains the full plastic shear capacity), the equation is the same as the AASHTO equation except for the reduced web height, D_r, where

$$D_r = \frac{F_{yw} S_x D}{M_u} \le D \tag{16}$$

Equation 15 is the modified shear strength of hybrid plate girders under applied moment M_u. The shear capacity will begin to decrease when the applied moment causes yielding in the web. Up to the point of web yielding, the nominal shear capacity is the combination of shear buckling and tension field action.

Basler Moment-Shear Interaction Theory

Basler (1961b) performed a study on the possible interaction between moments and shear forces in plate girders. He suggested approximations for the interaction curve, which is the basis for the moment-shear interaction requirement used in AASHTO.

Basler derives an equation for a reduced moment capacity for a given shear based on a lower-bound plastic analysis. He assumes that the section can attain the plastic moment capacity in the absence of shear and the plastic flanges, by themselves, resist moment at maximum shear (web totally yielded by shear). For design moment capacity, he assumes plastic flanges and a maximum elastic web stress at yield. These assumptions are conservative for transversely stiffened plate girders. Following is a summary of the moment-shear interaction derivation.

Figure 5 defines the moments used in the derivation. The moment carried by the flanges when fully yielded is approximated by

$$M_f = A_f F_{yf} D \tag{17}$$

where
A_f = cross-sectional area of each flange

If the flanges are assumed to be at yield, and a linear flexural stress distribution is assumed across the girder’s remaining cross section, the moment carried by the web is

$$M_w = F_{yw} S_w = \frac{F_{yw} A_w D}{6} \tag{18}$$

where
S_w = section modulus of the web, and
A_w = cross sectional area of the web.

Basler uses as the flexural capacity:

$$M_y = M_f + M_w \tag{19}$$

$$M_y = A_f F_y D \left(1 + \frac{1}{6} \frac{A_w}{A_f} \right) \tag{20}$$

Basler creates the interaction equation by assuming a central portion of the web, D_{wy}, is taken by shear yielding as shown in Figure 6. The remaining plastic section is available for moment resistance. The nominal shear strength provided by the assumed middle portion of the web is:

$$V_n' = D_{wy} t_w \tau_y \tag{21}$$

Dividing by the full plastic shear capacity of the web, and assuming that the nominal shear capacity of the web is approximately equal to the web plastic shear capacity, Basler obtains

$$D_{wy} = \left(\frac{V_n'}{V_n} \right) D \tag{22}$$

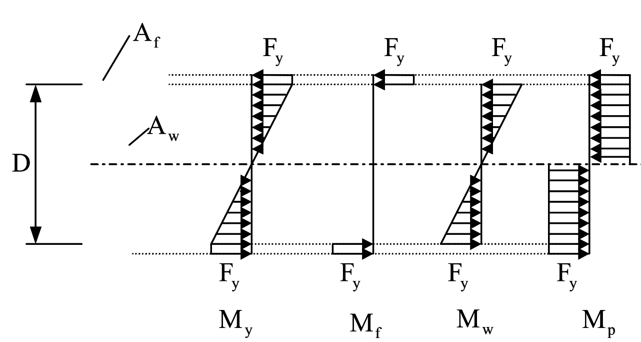


Fig. 5. Basler’s Reference Moments.

Basler's remaining flexural capacity is taken as the plastic flexural capacity of the remaining section

$$M_n' = A_f F_y D + \left(t_w \frac{D}{2}\right) F_y \left(\frac{D}{2}\right) - \left(t_w \frac{D_{wy}}{2}\right) F_y \left(\frac{D_{wy}}{2}\right) \quad (23)$$

which becomes

$$M_n' = A_f F_y D\left[1 + \frac{A_w}{4A_f}\left(1 - \left(\frac{V_n'}{V_n}\right)^2\right)\right] \quad (24)$$

Next, substitute Equation 20 into Equation 24 to obtain:

$$\frac{M_n'}{M_y} = \frac{1 + \left(\frac{A_w}{4A_f}\right)\left(1 - \left(\frac{V_n'}{V_n}\right)^2\right)}{1 + \frac{1}{6}\left(\frac{A_w}{A_f}\right)} \quad (25)$$

From Equation 25, it can be seen that Basler's interaction curve is a function of the ratio A_w/A_f. For practical girders, A_w/A_f is considered to have a practical upper limit of 2. Using this ratio, Equation 25 becomes

$$\frac{M_n'}{M_y} = 1.125 - 0.375\left(\frac{V_n'}{V_n}\right)^2 \quad (25)$$

Basler plots the parabolic interaction curve (Figure 7) that includes two critical points. The first is found by setting V_n'/V_n equal to one, which yields an M_n'/M_y value of 0.75. This point means that, as long as the moment is below 75 percent of the yield moment capacity, there is no reduction of shear capacity. It should be noted that this point corresponds to the idealized stress state in which $D_{wy} = D$, and thus $M_n' = M_f$. The second critical point is found by setting M_n'/M_y equal to one and solving for V_n'/V_n. Doing this yields a V_n'/V_n value of 0.58. This means that there is full moment capacity (i.e., the nominal moment M_y can be developed) as long as the shear is a maximum of 58 percent of the shear capacity.

AASHTO uses Basler's limiting points to define a straight-line interpolation for the moment-shear interaction as shown in Figure 1.

Modified Basler Moment-Shear Interaction for Hybrid Girders

To find a modified interaction curve for hybrid girders, the calculations Basler (1961b) made to find his original interaction curve were used, modified to accommodate different yield strengths of the flange and the web. To begin, a ratio is formed to relate the yield strengths of the web and flange:

$$\beta = \frac{F_{yf}}{F_{yw}} \geq 1 \quad (27)$$

Figure 5 defines the moments, although the flange yield stress is larger, used in the modified derivation. The moment carried by the flanges when fully yielded is

$$M_f = A_f \beta F_{yw} D \quad (28)$$

If the outermost fibers of the web near the flanges are assumed to be at yield, and a linear flexural stress distribution is assumed across the girder cross section, the moment carried by the web is

$$M_w = F_{yw} S_w = \frac{F_{yw} A_w D}{6} \quad (29)$$

The yield moment M_y is approximated in the same manner as Basler (1961b) with the exception that, to account for the web yielding at high moment, the AASHTO hybrid reduction factor R_h is applied

$$M_y = R_h S_x F_{yf} = R_h S_x \beta F_{yw} \quad (30)$$

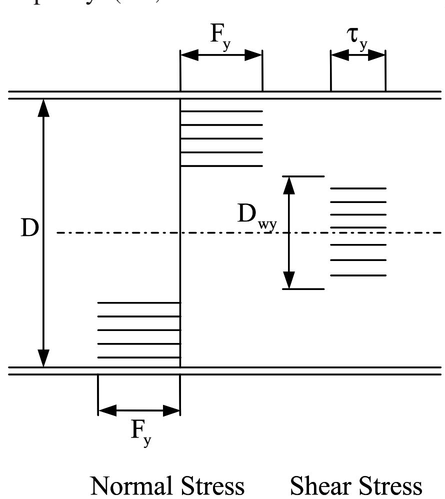


Fig. 6. Basler's Assumed Stress Distribution.

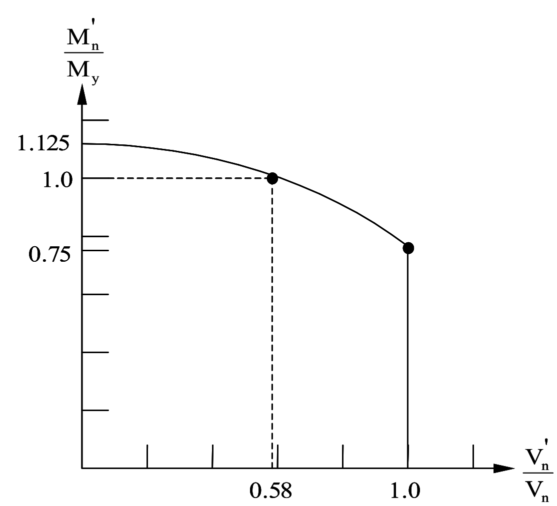


Fig. 7. Basler's Moment-Shear Interaction.

where

$$S_x = A_f D + \frac{A_w D}{6} \tag{31}$$

Combining Equations 30 and 31 yields

$$M_y = A_f D \beta F_{yw} R_h \left(1 + \frac{1}{6}\frac{A_w}{A_f}\right) \tag{32}$$

Assuming a similar stress diagram as in Figure 6, although with a larger flange yield stress, the yielded central portion of the web, D_{wy}, is still Equation 22. The remaining flexural capacity, which based on Basler's approach, is taken as the plastic capacity of the remaining section is

$$M'_n = A_f \beta F_{yw} D + \left(t_w \frac{D}{2}\right) F_{yw}\left(\frac{D}{2}\right) - t_w\left(\frac{D_{wy}}{2}\right) F_{yw}\left(\frac{D_{wy}}{2}\right) \tag{33}$$

which becomes

$$M'_n = F_{yw} A_f D\left[\beta + \frac{A_w}{4A_f}\left(1 - \left(\frac{V'_n}{V_n}\right)^2\right)\right] \tag{34}$$

Next, substitute Equation 32 into Equation 34 to obtain:

$$\frac{M'_n}{M_y} = \frac{\beta + \left(\frac{A_w}{4A_f}\right)\left(1 - \left(\frac{V'_n}{V_n}\right)^2\right)}{\beta R_h\left(1 + \frac{1}{6}\left(\frac{A_w}{A_f}\right)\right)} \tag{35}$$

Equation 35 is the modified Basler interaction curve for hybrid girders (compared to Equation 25 for homogeneous girders). Using the same assumption of $A_w/A_f = 2$ for the interaction curve, Equation 35 simplifies to

$$\frac{M'_n}{M_y} = \frac{3}{4R_h}\left[1 + \frac{1}{2\beta}\left(1 - \frac{V'_n}{V_n}\right)^2\right] \tag{36}$$

Equation 36 represents the modified Basler interaction equation for hybrid girders. This compares to Equation 26 for homogeneous girders. The next section will examine these equations and develop a hybrid interaction requirement.

TENSION FIELD ACTION DESIGN FOR HYBRID GIRDERS

50 ksi Web and HPS70W Flange Example

An example section will be used to demonstrate the two modified Basler theories developed in the previous sections for hybrid girders: (1) the reduced web depth modification developed by Hurst (2000) and (2) the modified Basler moment-shear interaction theory based on a lower-bound plastic analysis. A symmetric girder with D/t_w = 140 and A_w/A_f = 2 will be used with a 50-ksi web and HPS70W flanges. Figure 8 presents the moment-shear interaction curves corresponding to each of the above theories (i.e., Equation 15 and Equation 36) and the current AASHTO interaction requirement (Equation 4) assuming tension field action is applicable for the 50-70 hybrid girder.

The reduced effective web height theory shows no loss of shear capacity until M_u/M_n reaches approximately 0.8. At values of M_u/M_n above 0.8, the web experiences yielding due to flexure, according to the reduced effective web height theory. The shear capacity would be allowed to include tension field action, but would be diminished as the moment increases and flexural yielding of the web begins. However, if V_u/V_n is less than about 0.9, this approximation suggests that the moment-shear interaction is negligible.

The modified Basler interaction theory is shown by the parabolic-shaped curve in the figure. This approximation suggests that significant moment-shear interaction occurs at values of M_u/M_n above about 0.75. Furthermore, this estimate predicts that there is significant moment-shear interaction for V_u/V_n as small as approximately 0.45.

The AASHTO interaction requirement, assuming tension field action is applicable for hybrids, is shown with the above theoretical predictions. It can be seen that this interaction equation is fairly conservative compared to the prediction based on the reduced web depth modification. However, it is evident that the current AASHTO requirement is slightly more liberal than the modified Basler interaction theory at V_u/V_n between about 0.45 and 0.75. This is due to the fact that based on Basler's lower-bound plastic analysis approach, the hybrid flexural behavior (flexural

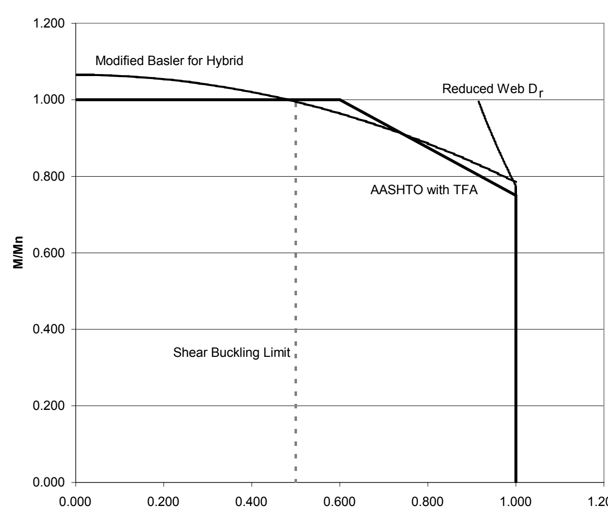


Fig. 8. Moment-Shear Interaction Curves for a 50-70 Hybrid Plate Girder.

yielding in the web at high moment) decreases the shear capacity more than for a homogeneous girder. Therefore, if Basler's approach is adopted, to be consistent with the development of the current AASHTO requirements for homogeneous girders, the interaction requirement must be adjusted to account for the hybrid action.

Parametric Study of Hybrid Combinations

Figure 9 illustrates the two previously developed hybrid girder moment-shear interaction theories for the above girder with a 50-ksi web and varying yield strengths in the flanges. The current AASHTO interaction (i.e., Equation 4) would be adequate relative to the modified Basler theory for the point of maximum shear and 75 percent of the yield moment for the practical range of web and flange strengths. Hurst (2000) shows that this practical range is

$$\beta = {F_{yf}}/{F_{yw}} \leq 1.4 \tag{37}$$

However, the AASHTO requirement would overestimate the shear capacity at maximum moment. Therefore, a modified interaction is shown in Figure 9 for hybrid girders. At maximum moment, the shear capacity is chosen as 45 percent of the nominal strength.

Figure 10 shows a similar plot for the previous 50-70 hybrid girder with varying A_w/A_f ratios. The modified interaction is adequate as long as

$$\frac{A_w}{A_f} \leq 2.5$$

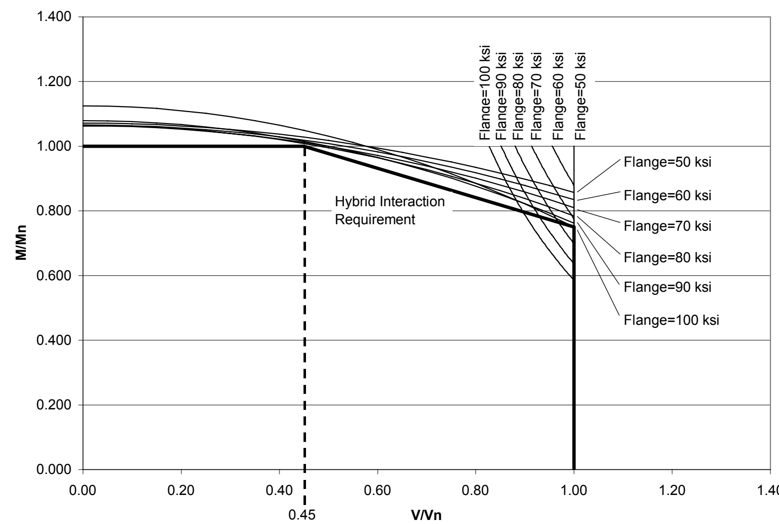


Fig. 9. Modified Basler Theory: 50 ksi Web; Varying Flange Strength.

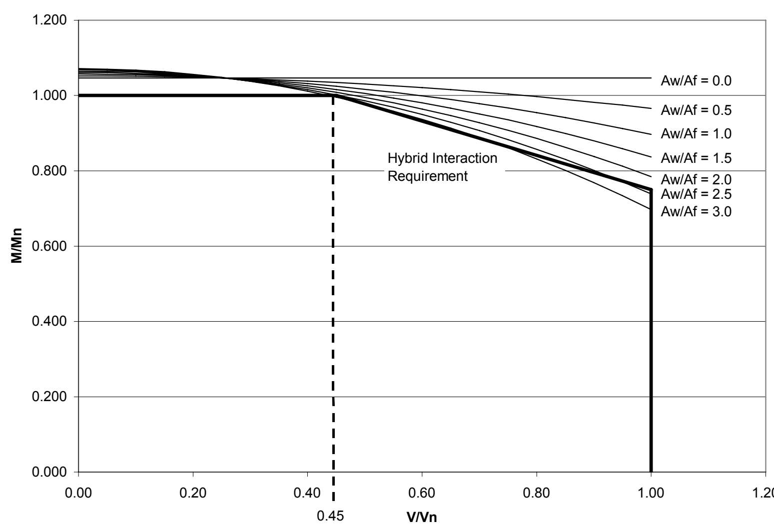


Fig. 10. Modified Basler Theory: 50-70 Girder with Varying A_w/A_f.

Hurst (2000) conducted parametric studies with differing web and flange relative strengths and web-to-flange area ratios to confirm the modified hybrid interaction shown in Figures 9 and 10. The hybrid girder modified equation (compared to Equation 4 for homogeneous girders) for the shear reduction can be written as

$$R = 0.45 + 0.55\left(\frac{M_n - M_u}{M_n - 0.75M_y}\right) \tag{38}$$

IMPACT OF HYBRID TFA WITH MODIFIED INTERACTION EQUATION

The introduction of TFA along with the hybrid moment-shear interaction requirement of Equation 38 would have a significant positive impact on the design of hybrid plate girders. The shear capacity of hybrid girders would no longer be limited by the shear buckling capacity. Figure 11 illustrates the benefits graphically. Although there is a small penalty for hybrid design compared to homogeneous girders, the benefits from the additional shear capacity would be large.

The planned experimental test girders discussed in the next section are used here to expand on the advantages of tension field action in hybrid girders. The test girders are 50-70 hybrid with dimensions proportional to typical girders. Each of the test girders is designed with one test panel with an aspect ratio (d_0/D) of 1.5 that will be tested to failure. According to AASHTO, the shear buckling capacity of the 50 ksi, 35 in. by 0.25 in. test panel is 82 kips, which is the current design capacity. If TFA is used, however, and if the moment in the test girder is below 75 percent of the flexural yield capacity, the contribution of TFA raises the design shear capacity to 165 kips.

Increases in shear capacity lead to wider stiffener spacing to achieve required shear capacities. For example, assume a specific design situation requires a shear capacity of 165 kips; the 50-70 hybrid girder previously described requires an aspect ratio of 0.73 according to AASHTO. However, if TFA were used, the required aspect ratio would be 1.5. A larger aspect ratio results in larger transverse stiffener spacing and thus fewer stiffeners; fewer stiffeners results in less material, less fabrication and fewer fatigue details.

The example advantages presented are only in the region of the interaction where M_u/M_n is less than 0.75. However, even if both moment and shear are of high magnitudes, the hybrid interaction requirement still holds advantages over the previous AASHTO limitations. Any allowance of tension field action will raise the shear capacity of hybrid girders creating more efficient hybrid designs.

VERIFICATION OF MODIFIED HYBRID INTERACTION

Finite Element & Experimental Verification

Finite element and experimental tests will be used to validate the hybrid moment-shear interaction. The girder geometry will be as discussed for Figures 9 and 10. The test

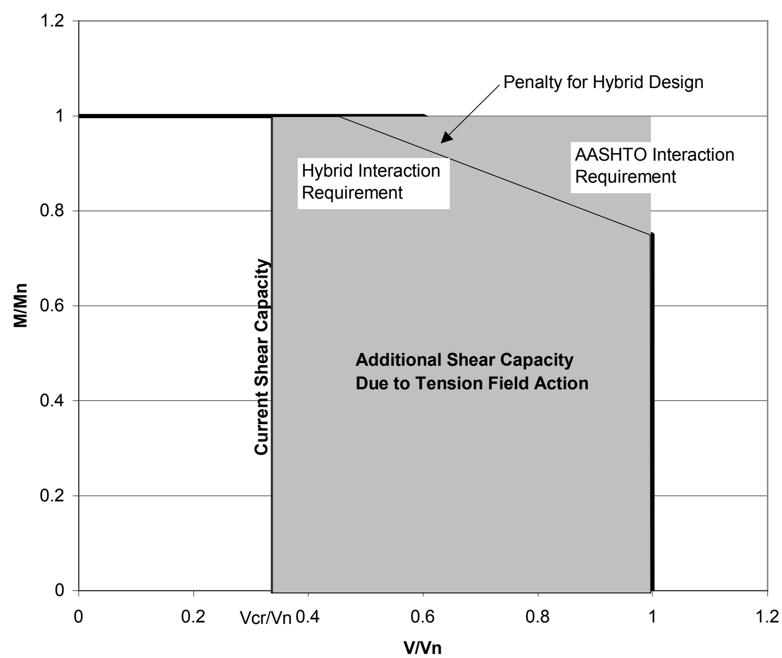


Fig. 11. Proposed Hybrid Shear Moment Interaction and AASHTO Interaction.

girders will consist of 50-ksi steel for the lower strength material and 70-ksi (HPS70W) steel for the high strength material. To verify tension field action in hybrid girders and the hybrid interaction requirement, target shear and moment combinations were chosen to validate critical areas of the interaction as shown in Figure 12.

Tests 1-3 are designed to test the shear capacity of three girders under low moment. Test Girder 1 was designed as 50-50 (50 ksi web and 50 ksi flanges), test Girder 2 as 70-70, and test Girder 3 as 50-70. Tests 4-8 are designed to test the shear capacity of five girders under varying combinations of shear and moment. Four of the five test girders (4-7) are designed as 50-70 hybrid sections. The fifth is designed with a 50-ksi web and 50-ksi flanges.

Expected Results

The modified interaction Equation 38 for hybrid girders is expected to be conservative, as the current AASHTO interaction Equation 4 is for homogeneous girders (Hurst, 2000). Preliminary finite element analyses of the test specimens discussed above, from Aydemir (2000), show that the reductions in capacities at high moment and shear are minimal or nonexistent. The finite element predictions generated by Aydemir are shown in Figure 13. It is expected that the experimental test results will be similar to these predictions. The results in Figure 13 are consistent with the results from a review of existing physical tests and a large parametric study conducted by Aydemir (2000), which indicates

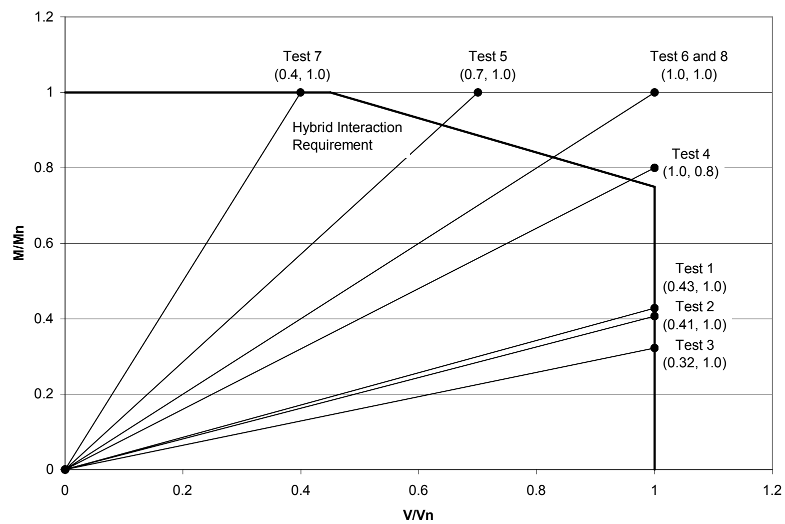


Fig. 12. Proposed Interaction Curve with Target Test Values.

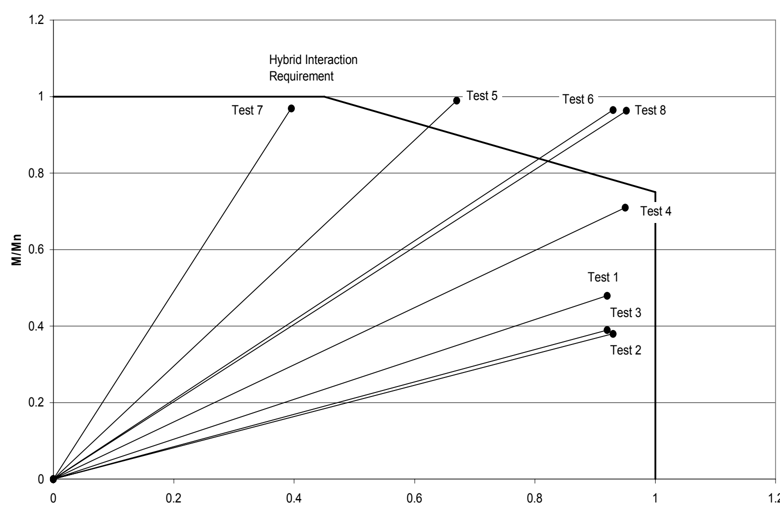


Fig. 13. FEA Results on Test Specimen Models.

that the maximum reduction in the shear capacity due to moment-shear interaction exhibited over a practical range of bridge girders is within the scatter band of the predictions of Basler's shear strength formula for high shear low moment. The liberal shear strength predictions for high-shear low-moment and the minimal moment-shear interaction shown in Figure 13 are consistent with Basler's (1961a) test data. If Aydemir's findings are confirmed by the experimental test results, there is a strong case to liberalize the moment-shear interaction requirements beyond those of Equation 38.

CONCLUSIONS

The use of tension field action with the modified hybrid moment-shear interaction requirement presented in this paper would positively impact the design and use of hybrid plate girders. With the projected use of HPS70W material hybrid with 50 ksi, hybrid tension field action would allow shear capacities well above what is currently allowed by AASHTO. With an increase in shear capacity comes an increase in transverse stiffener spacing and fewer stiffeners will need to be used. A decrease in stiffeners allows for a more economical design of hybrid plate girders, substantial cost savings in material and fabrication and fewer fatigue details.

The proposed modified interaction requirement for hybrid girders was developed using an approach that parallels Basler's work on TFA and moment-shear interaction. Although the current AASHTO moment-shear interaction equation is considered conservative, it is the standard for which AASHTO bases the shear design. The proposed modifications are acceptable for immediate use, since they are based on the same conservative theoretical development as Basler's original work, and they are written in the current AASHTO format. It is expected that further experimental and finite element studies will verify the development of tension field action in hybrid girders and the capability of the proposed modified moment-shear interaction equation as a lower bound strength estimate for cases of high-moment high-shear.

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