Shear Design of Stemmed Bridge Members: How Complex Should It Be?

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Shear design methods for prestressed concrete beams are becoming unnecessarily complicated because of the complex procedures to determine the concrete contribution, V_c. Using a two-span I-beam and a single span box beam example bridge, it is shown that varying the V_c value from the simplest 1979 AASHTO Interim method of calculating V_c to the most complicated AASHTO LRFD iteration procedure changes the cost of the beam by not more than 1 percent.

The determination of the shear strength of prestressed concrete beams has today become an extremely complicated process. Available design methods attempt to offer semi-empirical procedures of varying theoretical rigor, which attempt to account for the parameters that influence design. Such parameters include material properties, level of prestress, shear span/depth ratio, amount of reinforcement, moment/shear ratio at cross section in question, impact of support conditions, and other factors.

Most of the complexity comes in determining the concrete contribution, V_c, to the overall shear resistance. This is clearly demonstrated by reviewing Ref. 1 and the discussion that followed its publication.

This article presents a quick review of some of the available methods for the design of bridge beams. These methods are then applied to a typical two-span I-beam and a single span box beam to establish their economic impact on precast products. It is shown that when the prediction of V_c varies from the simplest AASHTO 1979 Interim Specifications (1979 Interim)[2] to the most elaborate procedure of the AASHTO-LRFD Specifications (AASHTO-LRFD),[3] the overall beam cost changes by not more than 1 percent.

A much more important issue than refining the value of V_c is the maximum allowed shear reinforcement. The AASHTO Standard Specifications (AASHTO-STD)[4] and AASHTO-LRFD give very different limits. A maximum limit that is too low could result in an unnecessary increase in member depth, web width or concrete strength. It may, in extreme situations, result in the designer or owner abandoning a precast, prestressed concrete alternate in favor of structural steel.

This situation has become possible recently due to the economies achieved in using high strength concrete shallow bridge I-beams at relatively wide beam spacings. The issue of maximum reinforcement is currently under study at the University of Nebraska and will be the subject of a future report. In the meantime, PCI JOURNAL readers are encouraged to contribute their experiences on the issue of shear design in general and maximum shear reinforcement in particular.

SUMMARY OF DESIGN PROCEDURES

Bridge design in the United States is mostly done in accordance with AASHTO-STD. However, the new AASHTO-LRFD is gaining in acceptance and is expected to eventually replace AASHTO-STD. Shear design according to AASHTO-LRFD is based on the Modified Compression Field Theory (MCFT) developed by Collins and Mitchell.[5] It was modified from the Original Compression Field Theory (OCFT)[6] to more rigorously account for the concrete contribution to shear resistance, V_c.

Shear design based on the 1979 Interim is an acceptable alternative in AASHTO-STD. Also included in the discussion is the 1989 AASHTO Segmental Concrete Bridges Guide Specifications (AASHTO Segmental Bridges),[7] which have recently been proposed for inclusion in AASHTO-LRFD as an acceptable alternate design approach. The ACI 318-95 Building Code is not covered here because the discussion is limited to bridge design. However, the ACI Code formulas are similar to those of AASHTO-STD and their application would be expected to produce similar results.

Table 1 provides a summary of the formulas used to predict the concrete contribution, V_c, the steel contribution, V_s, and also the minimum and maximum shear reinforcement requirement using each of the above referenced methods. For definitions of the symbols, see the Notation section.

Note that f'_c in the formulas given in Table 1 has the pounds per square inch unit. This is a modification of the AASHTO-LRFD formulas, which require that f'_c be in kips per square inch. The effective depth is defined somewhat differently between AASHTO-STD (d) and AASHTO-LRFD (d_v). In AASHTO-LRFD, it is the distance between the resultants of tensile and compressive forces due to flexure but not less than the greater of $0.9d$ or $0.72h$.

The V_c formula in AASHTO-LRFD appears simple enough if one could have a direct determination of the value of β, which is a factor indicating the ability of diagonally cracked concrete to transmit tension. Unfortunately, this is not the case because β has to be determined iteratively as a function of concrete shear stress and tensile reinforcement strain.

The concept of calculating the V_s component of the shear resistance is consistent among various methods. However, there are differences in choosing the angle of diagonal compressive stresses. The AASHTO-LRFD gives the most general formulation. With only vertical stirrups used, the term in brackets reduces to ($1/\tan\theta$). If the angle of diagonal compression is assumed to be 45 degrees, the term is further reduced to (1), which produces the familiar AASHTO-STD formula with the exception that AASHTO-STD uses the effective depth d rather than d_v as defined earlier.

The same formula as in AASHTO-STD is used in AASHTO Segmental Bridges. If the angle of diagonal compression is chosen as 26.6 degrees, the term ($\tan\theta$) is reduced to (2), which gives the 1979 Interim formula with d_v replaced by jd. The above discussion leads one to conclude that AASHTO-LRFD provides a useful general formulation for the determination of V_s. It also allows for flexibility in possible use of any combination of vertical and horizontal web reinforcement. The only complication is in the determination of the anticipated direction of the diagonal compressive stresses.

Table 1. Summary of various shear design methods.

Methods	Nominal shear strength from concrete, V_c	Nominal shear strength from shear reinforcement, V_s	Minimum area of web reinforcement, A_v	Limit on maximum area of web reinforcement, A_v
AASHTO Standard	$\min(V_{ci}, V_{cw})$, where: $V_{ci} = 0.6\sqrt{f'_c}b_w d + V_d + \dfrac{V_i M_{cr}}{M_{max}} \ge 1.7\sqrt{f'_c}b_w d$ $V_{cw} = \left(3.5\sqrt{f'_c} + 0.3 f_{pc}\right) b_w d + V_p$	$\dfrac{A_v f_y d}{s}$	$50\dfrac{b_w s}{f_y}$	$V_s \le 8\sqrt{f'_c}b_w d$
AASHTO LRFD	$\beta\sqrt{f'_c}b_w d_v$	$\dfrac{A_v f_y d_v}{s}\left(\dfrac{\sin\alpha}{\tan\theta} + \cos\alpha\right)$	$\sqrt{f'_c}\dfrac{b_w s}{f_y}$	$V_n \le 0.25 f'_c b_w d_v + V_p$
AASHTO Interim 1979	$0.06 f'_c b_w jd \le 180 b_w jd$	$2\dfrac{A_v f_y jd}{s}$	$50\dfrac{b_w s}{f_y}$ See footnote	
AASHTO Segmental Bridges	$2K\sqrt{f'_c}b_w d$, where: $K = \sqrt{1 + f_{pc}/\left(2\sqrt{f'_c}\right)} \le 2.0$	$\dfrac{A_v f_y d}{s}$	$50\dfrac{b_w s}{f_y}$	$V_n \le 10\sqrt{f'_c}b_w d$

Note: The coefficient 50 was changed from 100 in Interim 1980.

COMPARISON OF RESULTS

The above described methods are compared through design of the following two bridge examples. To make the comparison more direct, only the shear design procedures will be changed, while the HS-25 truck loading will be assumed and load factors of AASHTO-STD retained.

Example 1

A typical interior beam is shown in Fig. 1. Beam span is 130 ft (39.6 m) + 130 ft (39.6 m). Beam spacing is 8 ft (2.4 m). A total of 42 Grade 270 ksi (1862 MPa) low-relaxation, 6/10 in. (15.2 mm) diameter pretensioned strands are used in the beam. Six of the 42 strands are draped as shown in Fig. 1. The concrete strength of beams at service is $f_c' = 10{,}000$ psi (69 MPa) and at release $f_{ci}' = 7000$ psi (49 MPa). The deck thickness is 7 1/2 in. (190 mm) plus 1/2 in. (13 mm) haunch. The deck strength is 5000 psi (35 MPa).

Based on the AASHTO-STD method, the value of V_c changes from $2.1\sqrt{f_c'}b'd$ to $8.5\sqrt{f_c'}b'd$. This corresponds to the β range of 1.48 to 5.40 in the AASHTO-LRFD method. Obviously, AASHTO-LRFD predicts a smaller concrete contribution to the shear resistance than the AASHTO-STD method, as shown in Fig. 2.

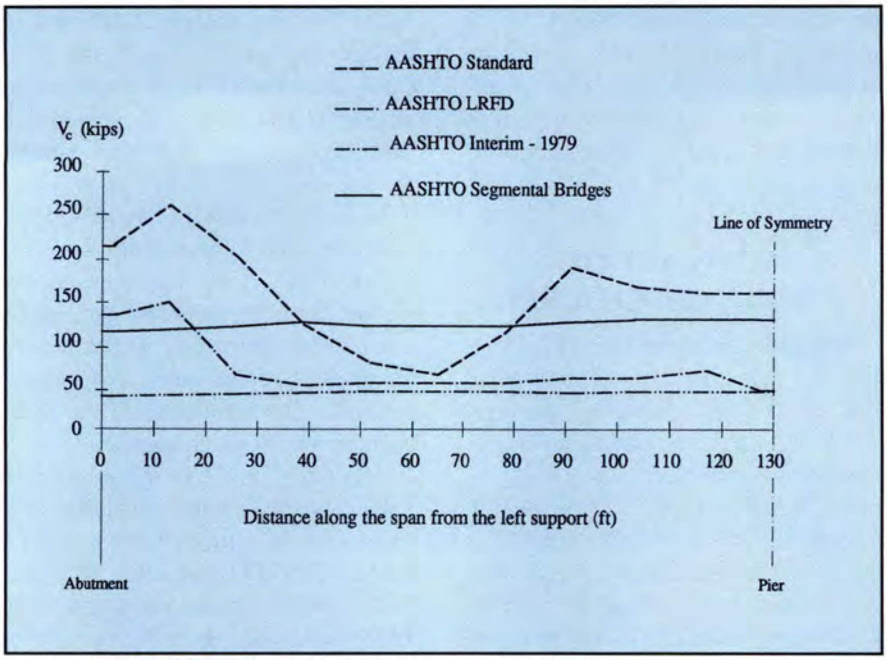

Fig. 2. Calculated V_c values by different design methods.

Except for the small change in the effective depth, d, along the span, both the 1979 Interim and AASHTO Segmental Bridges predict constant V_c values of:

$$(180)(b_w)(0.9d) = (1.62)(\sqrt{10000})b_w d = 1.62\sqrt{f_c'}b_w d$$

and

$$2K\sqrt{f_c'}b_w d = 4.0\sqrt{f_c'}b_w d$$

respectively.

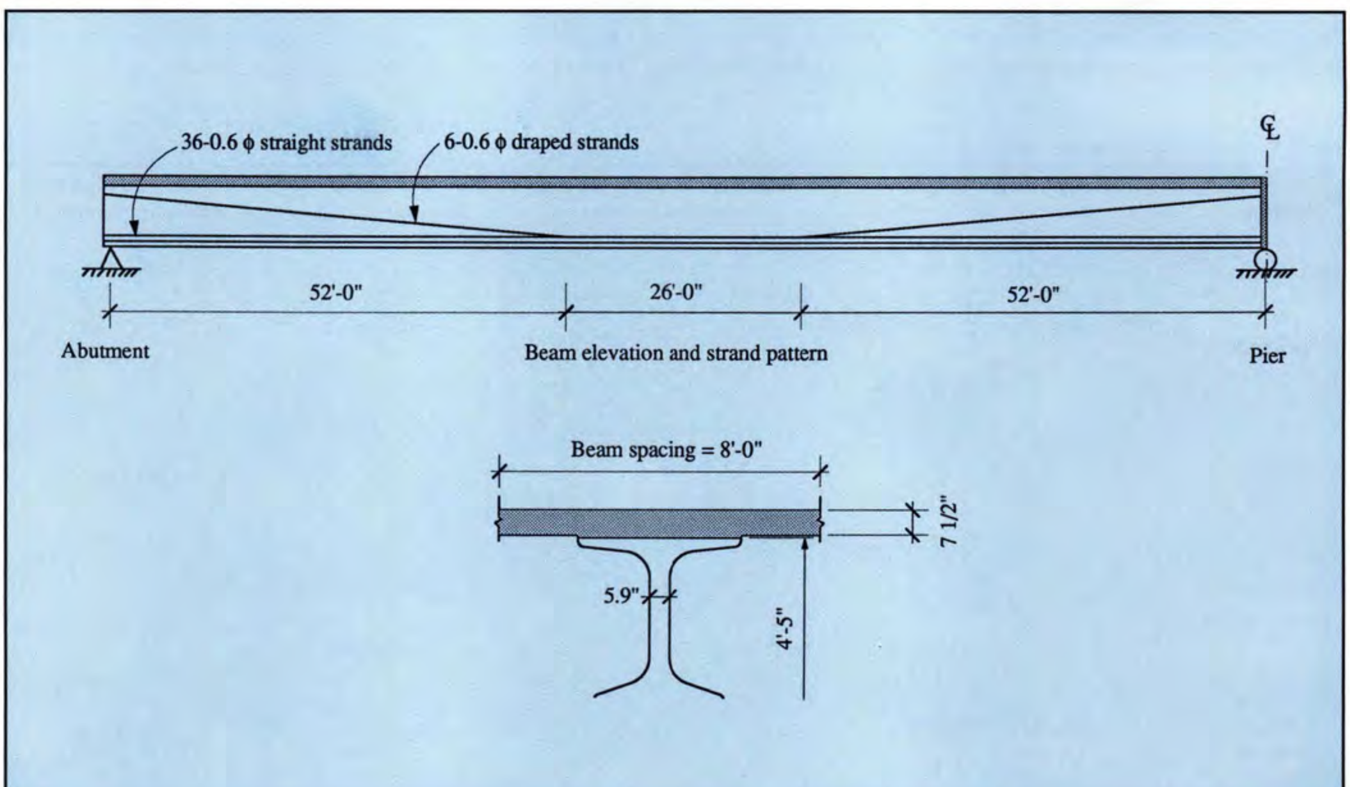

Fig. 1. Elevation and cross section of an example bridge beam.

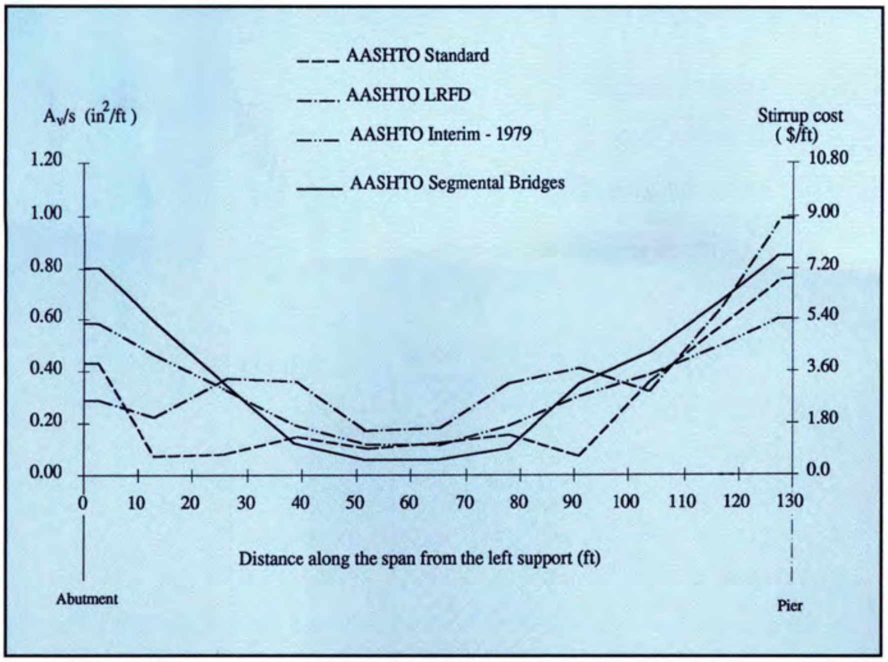

Fig. 3. Shear reinforcement requirement by different design methods.

Considering the fact that the shear capacity allowed in the AASHTO Segmental Bridges is $10\sqrt{f_c'}\,b_w d$, the maximum shear reinforcement contribution, V_s, has to be limited to $6\sqrt{f_c'}\,b_w d$, which is even smaller than the $8\sqrt{f_c'}\,b_w d$ limit in the AASHTO-STD.

The authors feel that this limit on the maximum shear reinforcement lacks experimental support; therefore, this limit is ignored in the example. The required shear reinforcement from the four methods is shown in Fig. 3. This figure shows that the required average shear reinforcement does not vary as significantly as V_c. If the unit cost of shear reinforcement is assumed as \$0.50 per lb, the calculated average shear reinforcement cost changes from \$2.07 to \$3.35 per ft of beam length.

Example 2

A typical interior AASHTO box beam of a 95 ft (29.0 m) single span is shown in Fig. 4. The superstructure consists of seven beams butted together. A 3 in. (76.2 mm) bituminous surfacing will be placed on the beams as a wearing surface. Beams are transversely post-tensioned through 8 in. (203 mm) thick full-depth diaphragms located at quarter points. A total of 31 Grade 270 ksi (1862 MPa) low-relaxation, 1/2 in. (12.7 mm) diameter pretensioned strands are used in the beam. The concrete strength of beams at service is $f_c' = 5000$ psi (35 MPa) and at release $f_{ci}' = 4000$ psi (28 MPa).

The variation of the concrete contribution, V_c, and A_v/s along the span is similar to the I-beam example bridge among the various shear design approaches. Although V_c varies significantly from one method to the other, the overall cost of the beam caused by the variation of the total amount of shear reinforcement is affected by a

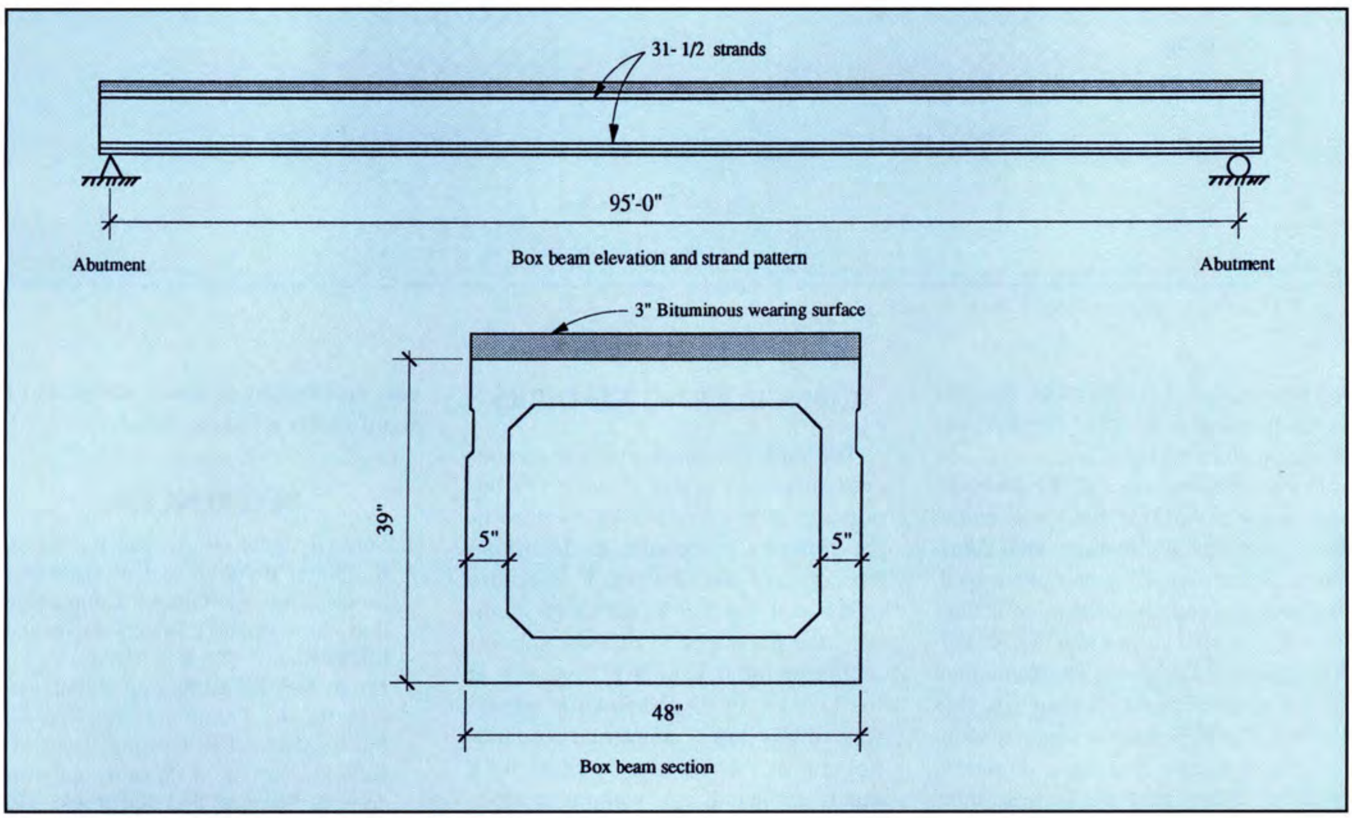

Fig. 4. Elevation and cross section of single span box beam.

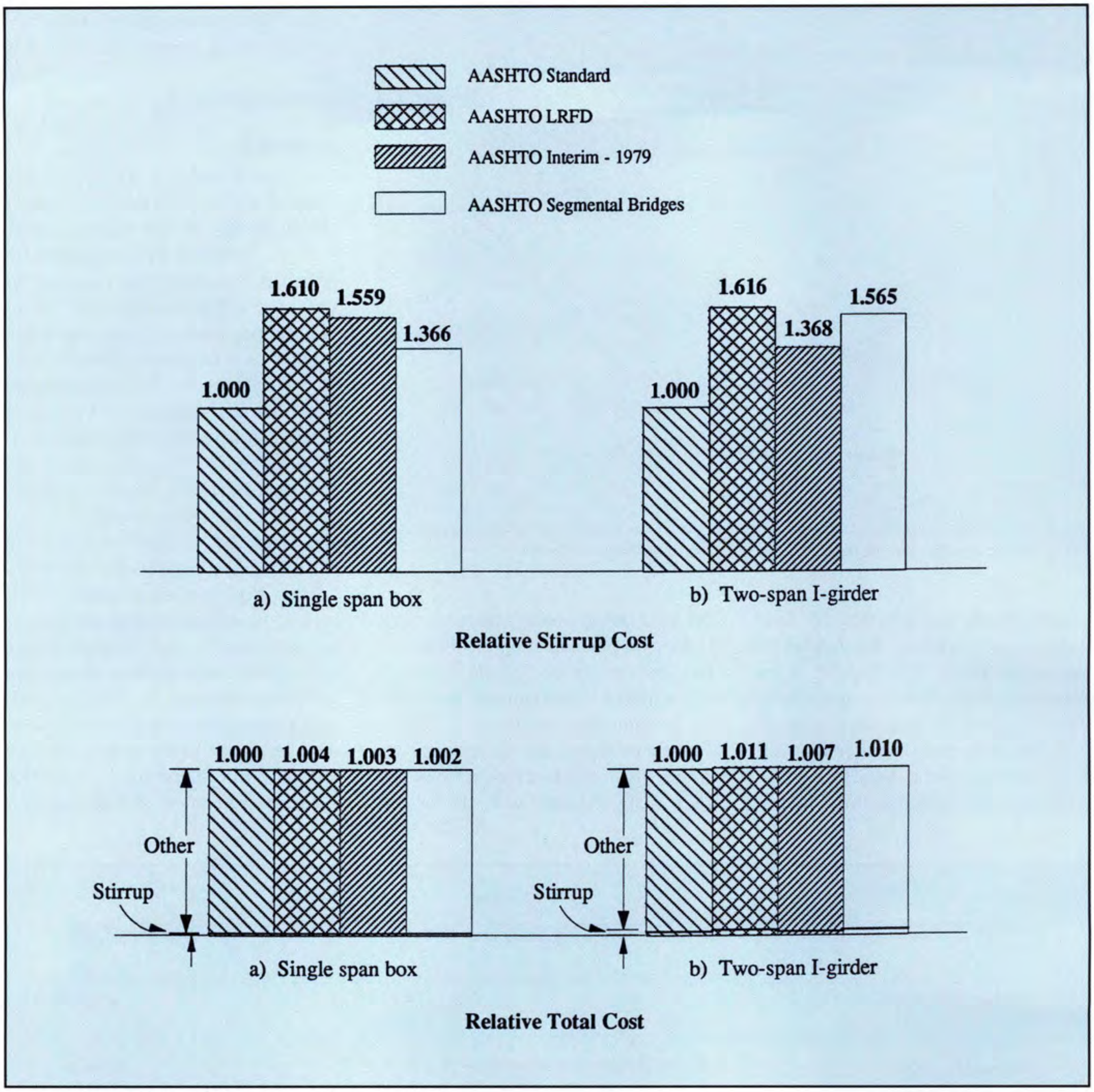

Fig. 5. Relative cost by different shear design methods.

maximum of 1.1 percent for the I-beam and 0.4 percent for the box beam, as shown in Fig. 5.

For total member cost, the base design using AASHTO-STD was calculated based on an average total fabricated beam cost of $600 per cu yd, and normalized to a value of 1.000. This figure also shows that AASHTO-STD provides the least reinforcement of all approaches. Based on this limited study, it would seem reasonable to conclude that the complexity of shear design may not be warranted for stemmed members.

CONCLUDING REMARKS

The shear design of prestressed concrete stemmed bridge members is becoming more complicated because of the complex procedures to determine the concrete contribution, V_c. Because of the thin web of the stemmed members, the impact of V_c on the total cost of the member is small. Research at the University of Nebraska is underway to develop a simplified formulation for V_c, which should result in a much simpler design without sacrificing the shear capacity of thin web precast prestressed concrete members or significantly affecting the cost.

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NOTATION

A_v = area of web reinforcement

b_w = width of web

d = distance from extreme compressive fiber to centroid of prestressing force, or to centroid of negative moment reinforcing for precast beam bridges made continuous

d_v = effective shear depth

f_c' = compressive strength (in psi) of concrete at service

f_{ci}' = compressive strength of concrete at time of initial prestress

f_{pc} = compressive stress in concrete at centroid of section resisting externally applied loads or at junction of web and flange when centroid lies within flange

f_y = yield strength of non-prestressed conventional reinforcement in tension

h = overall depth of member

j = ratio of distance between centroid of compression and centroid of tension to depth d

M_{cr} = moment causing flexural cracking at section due to externally applied loads

M_{max} = maximum factored moment at section due to externally applied loads

s = longitudinal spacing of web reinforcement

V_c = nominal shear strength provided by concrete

V_{ci} = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment

V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from excessive principle tensile stress in web

V_d = shear force at section due to unfactored dead load

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max}

V_n = nominal shear resistance of section considered

V_p = vertical component of effective prestress force at section

V_s = nominal shear strength provided by shear reinforcement

α = angle of inclination of transverse reinforcement to longitudinal axis

β = factor relating effect of longitudinal strain on shear capacity of concrete, as indicated by ability of diagonally cracked concrete to transmit tension

θ = angle of inclination of diagonal compressive stresses