Effects of the Latest LRFD Block Shear Code Change

HOWARD I. EPSTEIN

ABSTRACT

This paper examines the effects of the required change in the use of the block shear design strength equations resulting from the latest AISC-LRFD code treatment. The latest code either gives the same results as the previous code or is more conservative. The parameters of the connections for which the two treatments differ are presented, as well as the percentages of the differences. On the basis of recently reported experimental results, the need for further refinement of the codes is presented.

INTRODUCTION

Volume I of the recently published second edition of the *Load and Resistance Factor Design Manual* (LRFD)[1] contains a subtle change in the treatment of the determination of the block shear rupture design strength, ϕR_n. Both the first edition[2] as well as the second edition contain two equations for the determination of ϕR_n given by

$$\phi R_n = \phi[0.6F_y A_{gv} + F_u A_{nt}] \tag{1a}$$

$$\phi R_n = \phi[0.6F_u A_{nv} + F_y A_{gt}] \tag{1b}$$

where:

ϕ = 0.75
A_{nv} = net area subject to shear
A_{nt} = net area subject to tension
A_{gv} = gross area subject to shear
A_{gt} = gross area subject to tension
F_y = material tensile yield strength
F_u = material ultimate tensile strength

Equation 1a represents block shear strength determined by rupture on the net tensile section combined with shear yielding on the gross section on the shear plane(s). Equation 1b represents block shear strength determined by rupture on the net shear area(s) combined with yielding on the gross tensile area. These equations are based on the work of Rickles and Yura[3] as well as that of Hardash and Bjorhovde.[4] Except for slight differences in notation, these equations did not change from the first to the second edition of the LRFD *Manual*.

Howard I. Epstein is professor, department of civil and environmental engineering, University of Connecticut, Storrs, CT.

In the first edition of the LRFD *Manual*, these equations are found only in the commentary to Chapter J where it states that "the controlling equation is one that produces the *larger* force." The commentary goes on to explain that since block shear is a fracture or tearing phenomenon and not a yielding limit state, the proper formula is the one in which the fracture term is larger than the yield term. For ductile steels where F_u is considerably larger than F_y, this may be true for both equations. For brittle steels having F_y an appreciable fraction of F_u, this may not be true for either equation. The commentary goes on to state that "where it is not obvious which failure plane fractures, it is easier just to use the larger of the two formulas." In fact, the tables in the first edition of the LRFD *Manual* state that the equation to be used is the one producing the larger block shear strength.

The block shear equations in the second edition of the LRFD *Manual* are now found in Chapter J of the specifications, as opposed to the commentary. While the formulas are the same, the change in the second edition is contained in a check of the relative fracture strengths $F_u A_{nt}$ as compared to $.6F_u A_{nv}$ or since F_u is common to both terms, A_{nt} compared to $.6A_{nv}$. The code now states that when $F_u A_{nt} \geq .6F_u A_{ns}$, use Equation 1a and when $F_u A_{nt} < .6F_u A_{ns}$ use Equation 1b. The commentary now states that "the proper equation to use is the one with the larger rupture term."

In both LRFD editions, the commentary gives two extreme examples showing which of the two limiting states (shear yield/tension fracture or shear fracture/tension yield) is appropriate. One of the examples has a tension area much larger than the shear area while the other example reverses these areas. The latest interpretation of using the limiting state with the larger rupture term can certainly be justified on the basis of these examples. The same cannot easily be said of the former treatment.

The previous LRFD code treatment specified to always use the larger strength found from Equation 1a or 1b, while the current LRFD treatment may lead the designer to the same equation or possibly to the equation that yields the smaller design strength. If the smaller strength is now determined, the result is a more conservative design than found previously. The following section shows that this more conservative treatment may indeed result for certain connections. Parameters are presented for the connections for which this will occur. Implications of the change are also discussed.

RESULTS OF THE BLOCK SHEAR CODE CHANGE

In order to investigate the effects of the change in the treatment of block shear, the ratio of the two block shear rupture design strengths is defined as

$$\alpha = \phi R_n(\text{Equation 1a}) / \phi R_n(\text{Equation 1b}) \tag{2}$$

and the relative fracture strengths as

$$\beta = A_{nt} / .6A_{nv} \tag{3}$$

Figure 1 shows a plot of the relative design strengths, α, versus the relative fracture strengths, β. When $\beta < 1.0$, the code now requires the use of Equation 1b. Hence, if Equation 1b gives a larger design rupture strength, $\alpha < 1$ and there is no change from the previous LRFD treatment. On the other hand, if Equation 1a gives a larger design rupture strength, $\alpha > 1$ and the current LRFD treatment is more conservative. When $\beta \geq 1$, if Equation 1a gives the larger strength, $\alpha > 1$ and there is no change but if Equation 1a is smaller, $\alpha < 1$ and the new treatment is again more conservative. Note that there are no upper limits to α and β. The limiting values of 2, shown in Figure 1, are just for convenience.

In order to investigate which connection geometries and material parameters fall into the various regions in Figure 1, it is useful to define the material parameter

$$\gamma = F_y / F_u \tag{4}$$

and a connection geometric parameter

$$\varepsilon = A_{nv} / A_{gv} \tag{5}$$

The material parameter, γ, is obviously less than one. The geometric parameter, ε, will be 1.0 for welded connections and in the range of approximately 0.6 to 0.9 for most bolted connections. These two parameters are still not sufficient, however, to fully nondimensionalize α, the ratio of Equations 1a to 1b. Therefore, an additional ratio of $A_{gt}/.6A_{gv}$ is defined. Since this ratio will be fairly close to the ratio β defined by Equation 3, let

$$\delta\beta = A_{gt} / .6A_{gv} \tag{6}$$

where δ will be a dimensionless ratio that is close to one. With these definitions, the ratio of the two block shear equations can be written as

$$\alpha = (\gamma + \beta\varepsilon) / (\varepsilon + \gamma\delta\beta) \tag{7}$$

To show the effects that varying material properties and connection geometries have on this ratio α, specific parameters (γ, ε, and δ) were chosen for the plot shown in Figure 2. In particular, the material was chosen as A36 steel with F_y and F_u equal to 36 and 58 ksi, respectively, giving $\gamma = 36/58 = 0.62$. The net to gross shear area, ε, was chosen to be 0.9 and δ (which is approximately equal to one) was chosen to be 0.9, 1.0 or 1.1. For these parameters, the results for α versus β, for β varying from 0.5 to 1.5, are shown in Figure 2.

A comparison of Figures 1 and 2 indicates that when $\beta < 1$ and $\delta < 1$, there exists a range of β for which the new code treatment is more conservative. This translates into the net tension area less than 60 percent of the net shear area and the gross tension area even less of a percentage of the gross shear area. Similarly, there exists a range of values for $\beta > 1$ and $\delta > 1$ that also results in the new code treatment being more conservative. Of course, another way of looking at these results is that for certain parameters, the former code treatment may be somewhat unconservative.

There are three areas regarding the results of Figure 2 that will be addressed herein. First, are the results similar for other parameters. Second, are there actual connections for which the code treatments differ, and if so, by what percentage. Third, what are the implications of the change, especially regarding continuing experimental results for connections

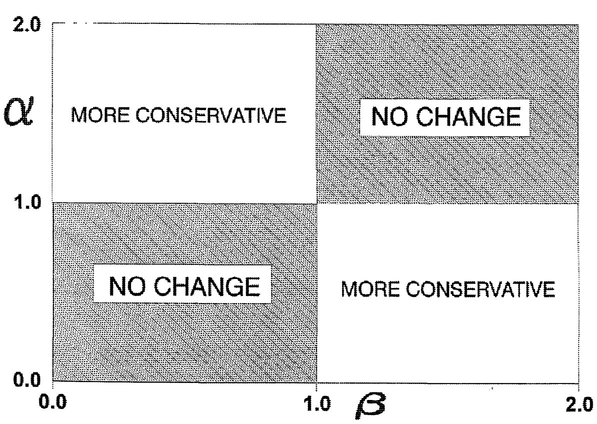

Fig. 1. Ratio of design strengths, α, vs. ratio of net to gross shear areas, β.

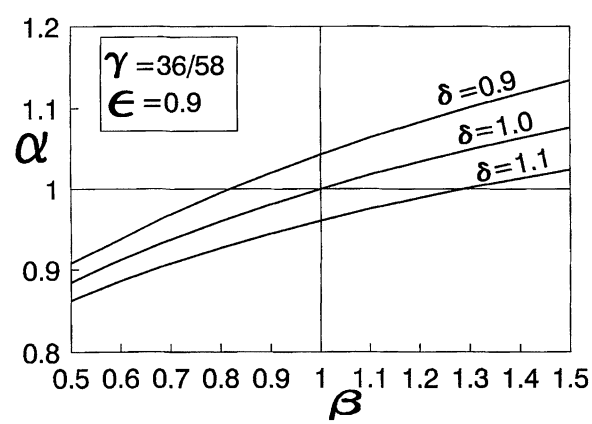

Fig. 2. α vs. β for some specific connection parameters.

where a possible block shear failure occurs in conjunction with shear lag.

Other Parameters

For $\delta = 1$, which means that the ratio of shear to tension areas is the same for net and gross areas, note that Equation 7 gives $\alpha = 1$ when $\beta = 1$ regardless of the values of ε and γ. This means that the $\delta = 1$ curve will always cross the point $\alpha = \beta = 1$. Further, with everything else equal, as δ decreases, Equation 7 shows that α increases. Therefore, portions or all of the curves for $\delta < 1$ will always be found in the upper left quadrant of Figure 1. For $\delta > 1$, portions or all of the curves will always be found in the lower right quadrant.

As can be seen from Equation 7, the intersection of any of the curves with the line $\beta = 0$ will be at the point $\alpha = \gamma / \varepsilon$ regardless of the value of δ. As β becomes large, each curve becomes asymptotic to the line $\alpha = \varepsilon / \gamma\delta$. Figure 3 shows plots of α versus β for varying values of the ratio γ / ε. These ratios, however, will usually be lower than 0.9. For instance, the ratio γ / ε for the curves in Figure 2 was approximately 0.7, and this is a fairly typical value. Larger values of γ / ε are therefore possible for materials that are relatively brittle or where large portions of the gross shear area missing, or a combination of the two.

Actual Beam Connections

Volume II of the second edition of the recently published *Load and Resistance Factor Design Manual* (LRFD)[5] contains tables with coefficients for various materials and geometric parameters. Material strengths include yield strengths $F_y = 36$ and 50 ksi and ultimate strengths F_u = 58, 65 and 70 ksi. Therefore, the common ratios of F_y/F_u = 36/58, 50/65 and 50/70 (γ = 0.62, 0.77 and 0.71) are included. The tables are set up for coped beam-to-beam connections, as shown in Figure 4. There is one vertical row of bolts (with bolt diameter $= d_b = \frac{3}{4}, \frac{7}{8}$, or 1 inch) with varying number of bolts (n = 2 to 12), varying horizontal edge distance (L_{eh} = 1 to 3 inches in ¼-in. increments) and varying vertical edge distance (L_{ev} = 1¼ to 3 inches in ¼-in. increments).

Many plots and tables could be introduced here to show how the range of connections covered in these tables are influenced by the change in the code. Instead of doing that, however, trends will be noted and particular examples will be presented for illustration. For instance, almost every possible connection geometry covered by the tables in Volume II produce $\beta < 1$. Only some of the two-bolt connections fall into this category where the tension area is large enough compared to shear area to give $\beta > 1$. These connections have large horizontal edge distances (L_{eh}) and small vertical edge distances (L_{ev}). For every one of the connections that gives $\beta > 1$ and for all possible material properties in these tables, there are no connections having results that differ from the old code. It should be noted here that the tables in Volume II for A_{gv} are incorrect because the shear length is $d_b/2$ too large, and this is unconservative. Fortunately, few of the beam-to-beam connections shown have $\beta > 1$ for which Equation 1a, and hence A_{gv}, is required. This may not be the case for other types of connections and, therefore, care should be exercised until corrected tables are obtained.

For the vast majority of connections where $\beta > 1$, there exist ranges of parameters producing $\alpha > 1$ and, therefore, a more conservative treatment by the new code. As the number of bolts increases, β decreases to a fairly small number. At the same time, the ratio of net/gross shear area, ε, gets confined to a small range. For instance, for a 10 bolt connection, β is in the range of 0.04 to 0.21 and for ¾-in. dia. bolts, $0.706 < \varepsilon < 0.723$. Therefore, as can be seen from Figure 3, the value of α will also be confined to a small range and will most be influenced by the material property $\gamma = F_y / F_u$. For γ = 50/65, the 10 bolt connections produced the range, $1.045 < \alpha < 1.095$. The smaller numbers in this range are associated with large vertical to horizontal edge distances and the larger numbers with small horizontal to vertical edge distances.

For this same material and bolt size, fewer bolts produce larger values of β and a larger range in the values of α with

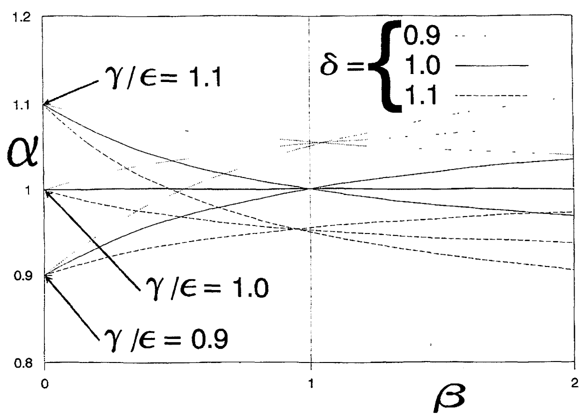

Fig. 3. Examples of trends in the plots of α vs. β.

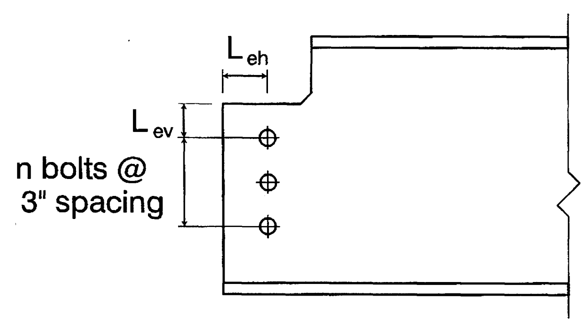

Fig. 4. Connection geometries tabulated in AISC-LRFD, 2nd Ed., Vol II.

the maximum value greater than the 1.095 above. A value of 1.105 is obtained for one of the possible three-bolt connections covered by the table. If 1-in. dia. bolts are used, the maximum α increases to over 1.2. If A572 grade 65 steel is used, $\gamma = 65/80$ and α will be greater than 1.25.

For A36 steel with $\gamma = 36/58$, the shorter connections have β closer to 1.0, and these produce the larger values of α. For instance, the parameters $n = 2$, $d_b = ¾$-in., $L_{ev} = 1½$ inches, and $L_{eh} = 2¼$ inches produce β slightly less than 1.0 and $\alpha = 1.054$. A maximum of $\alpha = 1.074$ is found for one of the possible three-bolt connections covered by the tables ($L_{ev} = 1¼$ inches and $L_{eh} = 3$ inches).

Other Connections

The beam-to-beam connections shown have geometries which in general produced large shear/tension areas and, hence, values of β generally less than one. There are many connections that may have a shear/tension area ratio closer to or even less than one and thus $\beta > 1$ (e.g., shorter connections where two or more rows of bolts are used). These connections may occur for shallow, heavy beams and many connections of tension angles, where two gage lines are possible. There are many possible materials and geometric parameters for connections that result in an increased conservatism in the new codes more than that for the tabulated connections of Volume II.

Implications of the Change

The change in the latest LRFD block shear provision has been shown to be a conservative one. The figures and examples presented have shown that some connections may have their design block shear capacities reduced, according to the new code. The examples cited show this reduction to be up to approximately 20 percent, with the vast majority of cases giving considerably less, or no reduction at all.

The implications for new designs are positive. If the new treatment more closely approximates reality, it certainly is good to be conservative. For upgrading existing structures, however, the designer must be aware of the possible loss in code capacities. The AISC code equations resulted from tests of coped beam-to-beam connections. The treatment for such connections appeared to be adequate before the latest modification, and should now be even more so.

Is this Change Sufficient?

Results of experimental studies conducted by the author[6] of block shear, for angles in tension, showed (in the author's opinion) "...the need for modification in the AISC code treatment of block shear." Revisiting these results in light of the recent code modification gives 7 of the 38 connections tested with design values reduced by the new treatment. However, the average reduction for these connections is only about three percent, and does not change the trends or conclusions of the paper.

Since the results of the author's block shear experiments were published, other researchers have published additional experimental results for angles.[7,8] There continue to be serious questions raised concerning the adequacy of the AISC codes in predicting block shear for angles in tension. Angles, unlike the coped beam-to-beam connections, experience bending superimposed upon the block shear area, as a result of the eccentricity of the load. The appropriateness of including these "shear lag" effects in the treatment of block shear was previously presented by the author.[6] The experimental work conducted since the author's work reinforces, in the author's opinion, the need for further code modifications

CONCLUSIONS

This paper examined the effects of the required change in the use of the block shear design strength equations resulting from the latest AISC-LRFD code treatment. The latest code gives either the same results or, possibly, more conservative results than the previous code. The connection parameters for which the two treatments differ were presented as well as the percentage of the difference that can be expected with this latest change. On the basis of continuing experimental results reported in the literature, the need for further refinement of the codes is presented for connections where shear lag acts in conjunction with possible block shear failure.

REFERENCES

1. American Institute of Steel Construction, *Manual of Steel Construction, Load and Resistance Factor Design: Volume I—Structural Members, Specifications & Codes*, 2nd Ed., Chicago, IL, 1994.
2. American Institute of Steel Construction, *Manual of Steel Construction, Load and Resistance Factor Design*, 1st Ed., Chicago, IL, 1986.
3. Rickles, J. R., and J. A. Yura, "Strength of Double-Row Bolted Web Connections," ASCE *Journal of Structural Engineering*, No. ST1, Vol. 109, January 1983, pp. 126–142.
4. Hardash, S., and R. Bjorhovde, "New Design Criteria for Gusset Plates in Tension," AISC *Engineering Journal*, 2nd Qtr., Vol. 22, 1985, pp. 77–94.
5. American Institute of Steel Construction, *Manual of Steel Construction, Load and Resistance Factor Design: Volume II—Connections*, 2nd Ed., Chicago, IL, 1994.
6. Epstein, H. I., "An Experimental Study of Block Shear Failure of Angles in Tension," AISC *Engineering Journal*, 2nd Qtr., Vol. 29, 1992, pp. 75–84.
7. Gross, J. M., "A Study of Block Shear Failure in Bolted Connections in High Strength Steel Angles," *M. S. Thesis*, Bucknell University, May 1994.
8. Cunningham, T. J., Orbison, J. G., and Ziemian, R. D., "Assessment of American Block Shear Load Capacity Predictions," *Journal of Constructional Steel Research*, (accepted for publication).