Evolution of Shear Lag and Block Shear Provisions in the AISC Specification

LOUIS F. GESCHWINDNER

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The 2005 AISC *Specification for Structural Steel Buildings* (AISC, 2005), hereafter referred to as the 2005 Specification, includes provisions for both allowable strength design (ASD) and load and resistance factor design (LRFD). The overriding principle that guided the development of these new provisions was that the steel does not know what method was used in its design. Thus, there should be a single approach for determining member strength and the modification of that strength to be consistent with the ASD and LRFD loading provisions of the governing building codes. The AISC Committee on Specifications and its Task Committees were charged with evaluating the then-current ASD and LRFD provisions and incorporating the best of both standards into the new standard. In addition, research results that became available since publication of the previous standards, ASD in 1989 (AISC, 1989) and LRFD in 1999 (AISC, 1999), needed to be incorporated. Thus, this new specification is a step forward for each design approach.

The original LRFD Specification (AISC, 1986) was calibrated to the existing ASD Specification (AISC, 1978) at that time, for a live load-to-dead load ratio, $L/D = 3$, in order that the new specification produce designs that were comparable to the existing provisions. In the 2005 Specification, the ASD and LRFD design philosophies are expressed as follows, employing the most common load combinations:

$$\text{For ASD } (D+L) = R_a \leq \frac{R_n}{\Omega} \tag{1}$$

$$\text{For LRFD } (1.2D+1.6L) = R_u \leq \phi R_n \tag{2}$$

where

R_a and R_u = required strengths determined from the ASD and LRFD load combinations, respectively

R_n = nominal strength

ϕ = resistance factor for LRFD

Ω = safety factor for ASD

Louis F. Geschwindner is vice president of engineering and research, American Institute of Steel Construction, Inc., Chicago, IL.

For the load combination of $(1.2D + 1.6L)$ and $L/D = 3$, the effective load factor becomes 1.5. Thus, the relationship between ϕ and Ω can be determined by solving Equations 1 and 2 for R_n and setting them equal, thus

$$\frac{(1.2D+1.6L)}{\phi} = \frac{1.5(L+D)}{\phi} = \Omega(D+L) \tag{3}$$

which yields

$$\Omega = \frac{1.5}{\phi} \tag{4}$$

This relationship guided the development of safety factors for the ASD provisions of the unified specification, based on the LRFD resistance factors. The resulting strength provisions for tension members in the 2005 Specification are the same as they were in the ASD (AISC, 1989) and LRFD (AISC, 1999) Specifications. The provisions for shear lag have been modified slightly to account for recent research results.

Block shear has seen several changes over the years since it was first introduced into the specification. However, the strength provisions for 2005 are essentially the same as the previous ASD and LRFD provisions with a slight variation to reflect improved understanding of block shear failure.

SHEAR LAG

Shear lag provisions were first introduced in the 1978 AISC ASD Specification. This was to account for the research findings that the net section was not fully effective in providing fracture strength when all elements of the tension member section were not attached to the connecting elements. The provisions of Section 1.14.2 simply stated that the effective net area, A_e, was to be taken as the net area, A_n, times a reduction factor, C_t, thus

$$A_e = C_t A_n \tag{5}$$

Three cases were identified for determining C_t:

1. W- M- or S-shapes with flange widths not less than ⅔ the depth, and structural tees cut from these shapes, provided the connection is to the flanges and has no fewer than three fasteners per line in the direction of stress. $C_t = 0.90$.

2. W- M- or S-shapes not meeting the requirements of subparagraph 1, structural tees cut from these shapes, and all other shapes, including built-up cross sections, provided the connection has not less than three fasteners per line in the direction of stress. $C_t = 0.85$.

3. All members whose connections have only two fasteners per line in the direction of stress. $C_t = 0.75$.

The commentary to Section 1.14.2 indicates that these values are reasonable lower bounds for profile shapes and connection means described in the research of Munse and Chesson (1963). In that research the authors proposed the following equation for the reduction factor,

$$C_t \approx 1 - \frac{\bar{x}}{l} \tag{6}$$

where

$\bar{x}$ = distance from the centroid of the section to the shear plane of the connection

l = length of the connection

Although Equation 6 was not a part of the specification provisions, it was included in the commentary.

The 1986 LRFD and 1989 ASD Specifications continued the use of these specified reduction factors, although the symbol was changed to U. In addition, provisions were made for members connected through welds. For the 1993 LRFD Specification, the equation developed by Munse and Chesson (1963), with an upper limit of 0.9 added, was made a part of the Specification, and the numerical values that had been in use until this time were moved to the commentary. The background for these changes is reported by Easterling and Giroux (1993). The three previously used cases were made available for designers and continued use as reasonable lower bounds unless a higher value was determined through the provided equation. The same provisions were carried over for the 1999 LRFD Specification.

The 2005 Specification needed to consider how the combination of ASD and LRFD provisions would impact tension member strength and how, if at all, the effective net area provisions would need to change. Using the relation between ϕ and Ω presented in Equation 4, the design tensile strength, ϕP_n, and the allowable tensile strength, P_n/Ω, for the limit state of fracture can be expressed as

$$P_n = F_u A_e$$
$$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.0 \text{ (ASD)} \tag{7}$$

This is the same provision as given in the 1999 LRFD Specification, and a comparison with the 1989 ASD Specification shows that it provides the same allowable strength. Since effective net area is not a function of design approach, there will be no impact there.

There are two changes in the 2005 shear lag provisions. The first change is the removal of the upper limit on U. Research results indicate that there is no need for this limit. The work of Munse and Chesson (1963) did not include a recommendation that the shear lag reduction factor have an upper limit. In fact, the inclusion of the upper limit of 0.90 in previous editions of the AISC Specification served only to address the reliability of the shear lag calculation, thereby duplicating the function already served by the resistance factor (LRFD) and safety factor (ASD).

The second change is the addition of a requirement that single angles, double angles, and WTs be proportioned so that U is equal to or greater than 0.60. Alternatively, a lesser value of U is permitted if the tension member is designed for the effect of eccentricity through the use of the member interaction equations. However, the lower bound values of U provided in the Specification would seem to indicate that a U value below 0.60 need not be used in any practical case. That is, there is no common connection configuration in which U need be taken less than 0.60 based upon the entries in AISC Specification Table D3.1. The intent of the interaction provision is to highlight the need to be very careful with connections with low shear lag reduction factors. It is recommended that whenever the reduction factor calculated through Equation 6 is below 0.60, regardless of the lower-bound value provide in Table D3.1, the connection either be redesigned to increase U or the member be designed to accommodate the resulting moment and axial force. Recent work reported by Epstein and Stamberg (2002) suggested that for WT sections, a lower bound of 0.65 be placed on U.

Since the 2005 Specification incorporates the previously separate specifications for single angles and HSS, the number of shear lag cases presented has increased. Table D3.1 is provided in the 2005 Specification to simplify the determination of appropriate U values.

BLOCK SHEAR

As was the case for shear lag, block shear provisions first appeared in the 1978 ASD Specification. These provisions were the result of the work of Birkemoe and Gilmor (1978) that was directed at the coped beam connection. The provisions, as stated in Section 1.5.1.2.2 of the 1978 ASD Specification, indicate that the shear at beam end connections where the top flange is coped, and in similar situations where failure might occur by shear along a plane through the fasteners, or by a combination of shear along a plane through the fasteners plus tension along a perpendicular plane on the area effective in resisting tearing failure, the stress was to be limited to $0.3F_u$. This would have resulted in an allowable force equation of

$$V_a = 0.3F_u(A_{nv} + A_{nt}) \tag{8}$$

The commentary provided an alternative where the tension and shear areas could be treated separately as

$$V_a = 0.3F_uA_{nv} + 0.5F_uA_{nt} \tag{9}$$

Obviously this would have provided an increase in allowable strength.

The 1989 ASD Specification brought Equation 9 from the commentary into the specification and provided the direction that block shear should also be considered for welded connections.

Block shear provisions are not as clearly articulated in the 1986 LRFD Specification. The provision for shear fracture is given in the body of the specification but the discussion of block shear is relegated to the commentary. In the commentary presentation, the possibility of a combination of yielding on one plane and fracture on the other is introduced, and the following two equations are given

$$\phi R_n = \phi(0.6F_yA_{gv} + F_uA_{nt}) \tag{10}$$

$$\phi R_n = \phi(0.6F_uA_{nv} + F_yA_{gt}) \tag{11}$$

with $\phi = 0.75$ and the largest value of ϕR_n taken as the design strength. It is worth comparing this first introduction of LRFD block shear provisions with the ASD provisions using the relationship of ϕ and Ω presented in Equation 4. Since $\phi = 0.75$, then $\Omega = 2.00$. Thus, dividing the nominal fracture term from Equations 10 and 11 by the safety factor of 2.00 and adding them, yields Equation 9.

The 1993 LRFD Specification (AISC, 1993) brought the two block shear equations forward into the specification but altered the way that the controlling equation was selected. For these provisions, the controlling factor was fracture. Shear fracture and tension fracture were to be calculated and the larger fracture term was to be combined with the opposite yield term. Additional research reported by Ricles and Yura (1983) for coped beams and Hardash and Bjorhovde (1985) for gusset plates confirmed that the strength could be determined by the summation of the shear and tension terms.

The 1999 LRFD Specification continued to use the previously presented block shear equations but set an upper limit on the strength that is equal to the sum of the two fracture terms which can be stated as

$$\phi R_n = \phi(0.6F_uA_{nv} + F_uA_{nt}) \tag{12}$$

This is actually a return to the commentary equation of the 1978 ASD Specification, since R_n from Equation 12 divided by the safety factor, 2.00, yields Equation 9.

For the 2005 Specification, the block shear provisions have undergone another modification to better reflect the observed mode of failure, particularly for coped beams. For shear strength, either fracture or yield, the strength equations remain unchanged. For tension strength, two revisions were made. The first recognizes the influence of nonuniform tension stress distribution, as would occur on the block shear tension face for a coped beam with two rows of bolts, as identified by Ricles and Yura (1983). The second change is the use, for all conditions, of the tension fracture strength, rather than selecting from either tension fracture or tension yield strength based on specified criteria. Close examination of the actual mode of failure indicates that tension fracture is always the critical mode. The resulting provisions for LRFD are given as

$$\phi R_n = 0.75(0.6F_yA_{gv} + U_{bs}F_uA_{nt}) \tag{13}$$

but not greater than

$$\phi R_n = 0.75(0.6F_uA_{nv} + U_{bs}F_uA_{nt}) \tag{14}$$

The comparable provisions for ASD are

$$\frac{R_n}{\Omega} = 0.3F_yA_{gv} + 0.5U_{bs}F_uA_{nt} \tag{15}$$

but not greater than

$$\frac{R_n}{\Omega} = 0.3F_uA_{nv} + 0.5U_{bs}F_uA_{nt} \tag{16}$$

When the tension stress is uniform, $U_{bs} = 1.0$, and for cases where the tension stress is not uniform, $U_{bs} = 0.5$. The commentary indicates that U_{bs} should be taken as 0.5 for coped beams with two rows of bolts. For all other cases illustrated, it gives $U_{bs} = 1.0$. This is consistent with the recommendations of Kulak and Grondin (2002). Although the committee considered a more involved approach to the calculation of U_{bs}, it was decided to simplify the term so as not to make its determination laborious for little gain.

Throughout this paper, the term *fracture* has been used. That is the term historically used to describe this mode of failure. However, the 2005 Specification has uniformly changed the term to *rupture* for consistency across various provisions and to eliminate any confusion with brittle fracture.

CONCLUSIONS

An evolution of the shear lag and block shear provisions has taken place since their first introduction in the AISC Specification in 1978. Although the changes have been slight, they reflect an improving understanding of the behavior that they are attempting to predict. The intent has always been to provide specification provisions that are sufficiently accurate so as to provide for safe and economical structures while at the same time providing design methods that are simple and economical to apply. The 2005 Specification does that.

NOTATION

A_e = effective net area

A_{gt} = gross area subjected to tension

A_{gv} = gross area subjected to shear

A_n = net area

A_{nt} = net area subjected to tension

A_{nv} = net area subjected to shear

C_t = shear lag reduction factor from the 1978 ASD Specification

D = dead load

F_u = specified minimum tensile strength

F_y = specified minimum yield stress

L = live load

P_n = nominal tensile strength

R_a = required strength (ASD)

R_n = nominal strength

R_u = required strength (LRFD)

V_a = allowable shear based on block shear

l = length of the connection

$\bar{x}$ = distance from the centroid of the section to the shear plane of the connection

ϕ = resistance factor

Ω = safety factor

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