DISCUSSION

The Behavior and Load-Carrying Capacity of Unstiffened Seated Beam Connections

Paper by W. H. YANG, W. F. CHEN and M. D. BOWMAN
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Discussion by C. J. Carter, W. A. Thornton and T. M. Murray

The authors raise a valid criticism in their assessment of the AISC procedure for the design of unstiffened seated connections. Specifically, the anomaly in the formulation of the design strength that occasionally results in a negative bearing length needs to be corrected. An additional shortcoming of the AISC procedure that was not noted in the subject paper is that, while the local web yielding limit state is an integral part of the formulation of the seat angle design strength, the web crippling limit state is not although it is checked after a seat angle has been selected. A revised procedure that addresses these concerns is detailed at the end of this discussion.

Additionally, a number of other concerns raised need clarification or comment:

1. The authors describe the AISC procedure for the design of unstiffened seated connections as irrational, primarily because it does not model the exact behavior of the seat angle. Instead, the AISC procedure is based upon a simplified cantilever bending model for bending of the outstanding angle leg. Despite the author's characterizations, the AISC procedure is entirely a rational method—one in which a simplified approach is utilized to determine the answer to a problem that is more complex. Historically, AISC has employed such simplified models in its design procedures to maintain design simplicity. Although some procedures are more truly descriptive of actual behavior, such as those for the design of flexural members, all result in design strengths that are reasonable and representative of those obtained from testing of the modeled components.
2. In several instances, the authors indicate that the AISC procedure may be unsafe. However, the authors also present substantial data that shows that the AISC procedure underestimates strength in many cases. Furthermore, the authors note that unstiffened seated connections have a very good historical performance record while stating that "... the overall LRFD procedures usually produce satisfactory and safe designs ..." and "... [the design strengths] generated from these procedures are not significantly off when compared with the results obtained using more rational and accurate models." Ultimately, the authors are assessing safety based upon comparisons of the results of the AISC procedure with those of another model that may be no more accurate. Safety is more appropriately assessed by comparing a model to physical test results.
3. The authors suggest that an interaction check is important to address the concurrent effects of shear and bending in determining the design strength of the seat angle. The Drucker criterion as recommended by the authors represents a lower-bound solution that is always conservative by some unknown amount. Without testing, its actual relevance, however, is unknown.

   An interaction check for concurrent shear and bending has not historically been made for unstiffened seated connections. Nor has it been made in other similar cases of combined shear and bending, including double-angle, single-angle, and single-plate shear connections and moment end-plate connections. It should be of surprise to no one that the actual distribution of stress in the outstanding leg of an unstiffened seated connection is much more complex than the idealized distribution that is fundamentally assumed to exist along the span of a

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beam. For this reason alone, such an interaction check is probably unwarranted. Furthermore, it must also be recognized that many beneficial aspects of the connection and system behavior are ignored in the AISC model. For example, in the AISC model, flexural design strength is assessed on the basis of the formation of a single plastic hinge in the outstanding leg, whereas true failure of the connection in flexure requires the formation of two hinges. Additionally, any contribution to the connection strength from the top flange stabilizing angle is ignored in the AISC model. Although small, this top angle does so contribute. These more than compensate for such design simplifications as the lack of an interaction check for combined shear and bending.

4. The authors assert that loading on the beam supported by the unstiffened seated connection tends to pull the seat angle away from the support, resulting in higher-order axial and flexural effects. In fact there is normally restraint in the structural system, such as that due to a floor slab, roof deck or other framing members, that prevents such deformations. The authors further contend that the deflection of the simple beam tends to pull the seat angle away from the support. This is incorrect because the bottom flange of the beam gets longer as the beam deflects, thereby pushing the seat angle into the support.
5. The authors discuss the effect of the presence of bolts connecting the beam to the seat angle and the installed tension when such bolts are present. While the strength assessment of the seat angle for the no-bolt case is technically acceptable, it should be noted that the AISC procedure requires that the beam be attached to the seat angle with two high-strength bolts. Any comparison of the AISC procedure to the no-bolt case strength is therefore inappropriate. From a practical standpoint, the no-bolt case is only of concern during erection when the connection normally is subject to an end reaction that is considerably lower than the in-service end reaction.

   Of greater concern is the authors claim that the actual pretension present in the installed high-strength bolts affects the strength of the connection. This claim is based upon the supposition that the toe of the seat angle is initially the only point of contact with the beam flange, which causes a bending moment as the plies are brought into contact during bolt installation. In fact, due to beam camber and/or angle cross-sectional tolerances, the end of the beam is more likely to be the point of initial contact. Regardless, the small rotation required to bring the plies into firm contact is insignificant and it can be stated emphatically that the performance of unstiffened seated connections is unaffected by the installed bolt tension as long as the connected plies are in contact as required in the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*.

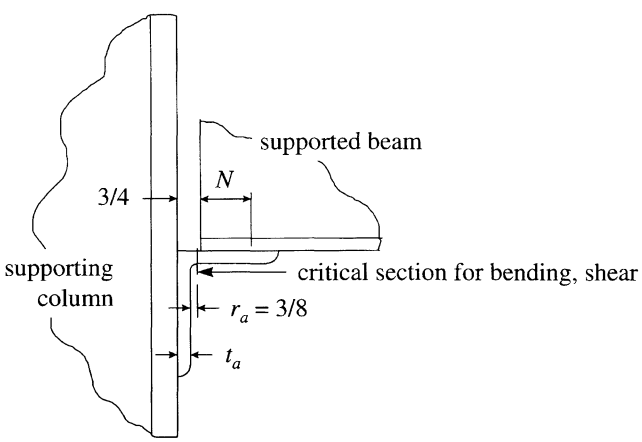

Fig. 1. Unstiffened seated connection.

The following design procedure alleviates the problem of negative bearing lengths and integrates both the local web yielding and web crippling checks into the seat-angle design. It is based upon the model and variables illustrated in Figure 1, where N is the bearing length at the beam end.

First determine the largest required bearing length for the limit states of local web yielding and web crippling of the beam. For local web yielding, from LRFD Specification Section K1.3,

$$\phi R_n = 1.0 \times (N + 2.5k) F_{yw} t_w$$

and the required bearing length N_{req} is,

$$N_{\min} = \frac{R_u}{F_{yw} t_w} - 2.5k = \frac{R_u - \phi R_1}{\phi R_2} \tag{1}$$

As indicated in LRFD Specification Section K1.3, as a lower bound,

$$N_{\min} = k \tag{2}$$

For web crippling, from LRFD Specification Section K1.4,

when $\frac{N}{d} \le 0.2$

$$\phi R_n = 0.75 \times 68 t_w^2 \left[1 + \frac{3N}{d} \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{F_{yw} t_f}{t_w}}$$

when $\frac{N}{d} > 0.2$

$$\phi R_n = 0.75 \times 68 t_w^2 \left[1 + \left(\frac{4N}{d} - 0.2 \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{F_{yw} t_f}{t_w}}$$

and the required bearing length N is,

when $\dfrac{N}{d} \leq 0.2$

$$N_{\min} = \frac{d}{3}\left[\frac{R_u t_f}{0.75(68 t_w^3 \sqrt{F_{yw}})}\left(\frac{t_f}{t_w}\right)^{1.5}\right] = \frac{R_u - \phi_r R_3}{\phi_r R_4} \tag{3}$$

when $\dfrac{N}{d} \leq 0.2$

$$N_{\min} = \frac{d}{4}\left[\frac{R_u t_f}{0.75(68 t_w^3 \sqrt{F_{yw}})}\left(\frac{t_f}{t_w}\right)^{1.5} + 0.2\right] = \frac{R_u - \phi_r R_5}{\phi_r R_6} \tag{4}$$

In the above equations,

F_{yw} = minimum specified yield stress of beam web, ksi
N = bearing length, in.
R_u = required strength (beam end reaction), kips
d = beam depth, in.
k = distance from outer face of flange to web toe fillet, in.
t_w = beam web thickness, in.
t_f = beam flange thickness, in.
ϕ = 1.0
ϕ_r = 0.75
ϕR_1, ϕR_2, $\phi_r R_3$, $\phi_r R_4$, $\phi_r R_5$, and $\phi_r R_6$ are constants tabulated in the factored uniform load tables in LRFD Manual Part 4

Because the value of N/d is initially unknown for the web crippling limit state, the larger value of $N_{\min}$ determined from Equations 3 and 4 is used (alternatively, an iterative approach can be used to determine which equation is appropriate). The required bearing length N_{req} is then the largest value of $N_{\min}$ determined from Equations 1, 2, 3, and 4. The outstanding leg dimension of the seat angle must be selected to meet or exceed the sum of the required bearing length N_{req} and ¾-in. (½-in. setback plus ¼-in. tolerance on beam length).

Next, select the angle length L_a and thickness t_a such that Equations 5 and 6 are satisfied. For flexural yielding of the angle,

$$t_a \geq \sqrt{\frac{4 R_u e}{\phi F_{ya} L_a}} \tag{5}$$

For shear yielding of the angle,

$$t_a \geq \frac{R_u}{\phi(0.6 F_{ya}) L_a} \tag{6}$$

In the above equations,

$$e = \frac{N_{req}}{2} + b_s - t_a - r_a = \frac{N_{req}}{2} + 3/8 - t_a$$

and

F_{ya} = minimum specified yield stress of seat angle, ksi
L_a = seat angle length, in.
N_{req} = largest required bearing length from Equations 1, 2, 3, and 4
R_u = required strength (beam end reaction), kips
b_s = ¾-in., (½-in. beam setback plus ¼-in. tolerance on beam length)
r_a = seat angle fillet radius taken as ⅜-in.
t_a = seat angle thickness, in.
ϕ = 0.90

The design of bolts or welds connecting the seat angle to the support is unchanged from that in the current AISC procedure.

Tables 1 and 2 simplify the selection of the unstiffened seated connections illustrated in Figure 2 according to the foregoing procedure. Table 1 is for unstiffened seated connections that are bolted to both the supported beam and the supporting member. Table 2 is for unstiffened seated connections that are welded to both the supported beam and the supporting member. Unstiffened seated connections that utilize a combination of bolting and welding can also be designed with the applicable sections of Tables 1 and 2.

With the value of N_{req} calculated as above, enter the appropriate table and select a combination of angle length L_a and angle thickness t_a that provides a design strength that equals or exceeds the required strength (beam end reaction). The

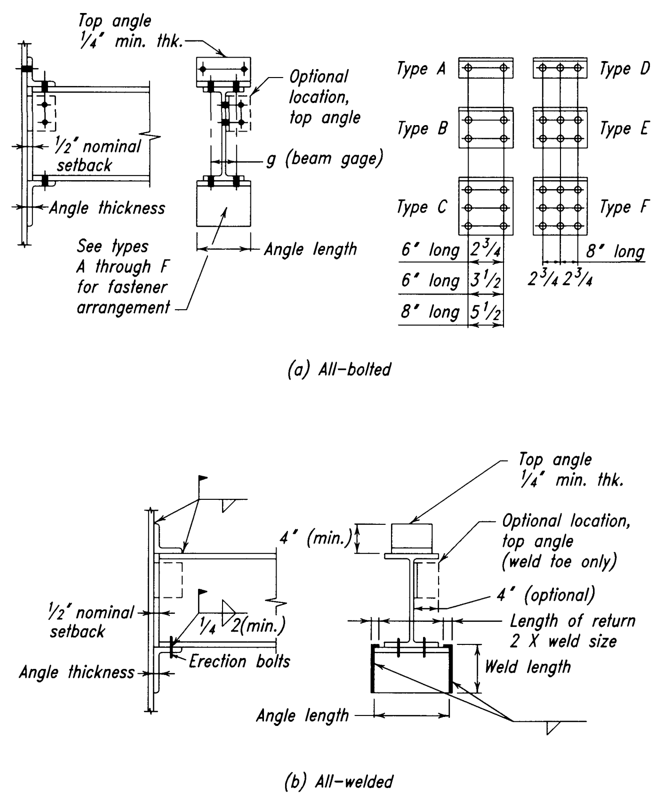

Fig. 2. Unstiffened seated connections.

outstanding angle leg size must be selected greater than the minimum angle leg tabulated in the right-hand column.

When a bolted connection to the supporting member is used, the design strength is tabulated in Table 1. When a welded connection to the supporting member is used, the design strength is tabulated in Table 2. The design strength of the supporting member must be checked independently.

Some common angle sizes with available ranges of thickness are indicated in both Tables. This is not intended to preclude the use of alternative angle sizes and thicknesses. The use of a longer outstanding angle leg than that indicated is permitted.

Table 1. All-Bolted Unstiffened Seated Connections

Outstanding Angle Leg Design Strength, kips

Required Bearing Length N_{req}	Angle Length 6 in.: Angle Thickness $3/8$	$1/2$	$5/8$	$3/4$	1	Angle Length 8 in.: Angle Thickness $3/8$	$1/2$	$5/8$	$3/4$	1	Min. Angle Leg
in.											in.
$1/2$	27.3					36.5					
$9/16$	24.3					32.4					
$5/8$	21.9	58.3				29.2	77.8				
$11/16$	19.9	55.5				26.5	74.1				
$3/4$	18.2	48.6				24.3	64.8				
$13/16$	16.8	43.2				22.4	57.6				
$7/8$	15.6	38.9				20.8	51.8				
$15/16$	14.6	35.3				19.4	47.1				
1	13.7	32.4	72.9			18.2	43.2	97.2			
$1\,1/16$	12.9	29.9	67.5			17.2	39.9	90.0			
$1\,1/8$	12.2	27.8	60.8			16.2	37.0	81.0			
$1\,3/16$	11.5	25.9	55.2			15.3	34.6	73.6			
$1\,1/4$	10.9	24.3	50.6			14.6	32.4	67.5			
$1\,5/16$	10.4	22.9	46.7			13.9	30.5	62.3			$3\,1/2$
$1\,3/8$	9.94	21.6	43.4	87.5		13.3	28.8	57.9	117		
$1\,7/16$	9.51	20.5	40.5	79.5		12.7	27.3	54.0	106		
$1\,1/2$	9.11	19.4	38.0	72.9		12.2	25.9	50.6	97.2		
$1\,5/8$	8.41	17.7	33.8	62.5		11.2	23.6	45.0	83.3		
$1\,3/4$	7.81	16.2	30.4	54.7		10.4	21.6	40.5	72.9		
$1\,7/8$	7.29	15.0	27.6	48.6		9.72	19.9	36.8	64.8		
2	6.83	13.9	25.3	43.7	117	9.11	18.5	33.8	58.3	156	
$2\,1/8$	6.43	13.0	23.4	39.8	111	8.58	17.3	31.2	53.0	148	
$2\,1/4$	6.08	12.2	21.7	36.5	97.2	8.10	16.2	28.9	48.6	130	
$2\,3/8$	5.76	11.4	20.3	33.6	86.4	7.67	15.2	27.0	44.9	115	
$2\,1/2$	5.47	10.8	19.0	31.2	77.8	7.29	14.4	25.3	41.7	104	
$2\,5/8$	5.21	10.2	17.9	29.2	70.7	6.94	13.6	23.8	38.9	94.3	
$2\,3/4$	4.97	9.72	16.9	27.3	64.8	6.63	13.0	22.5	36.5	86.4	
$2\,7/8$	4.75	9.26	16.0	25.7	59.8	6.34	12.3	21.3	34.3	79.8	4
3	4.56	8.84	15.2	24.3	55.5	6.08	11.8	20.3	32.4	74.1	
$3\,1/8$	4.37	8.45	14.5	23.0	51.8	5.83	11.3	19.3	30.7	69.1	
$3\,1/4$	4.21	8.10	13.8	21.9	48.6	5.61	10.8	18.4	29.2	64.8	

Bolt Design Strength, kips

Bolt Diameter, in.	ASTM Desig.	Thread Cond.	Connection Type from Figure 2a: A	B	C	D	E	F
$3/4$	A325	N	31.8	63.6	95.4	47.7	95.4	143
		X	39.8	79.5	119	59.6	119	179
	A490	N	39.8	79.5	119	59.6	119	179
		X	49.7	99.4	149	74.6	149	224
$7/8$	A325	N	43.3	86.6	130	64.9	130	195
		X	54.1	108	162	81.2	162	244
	A490	N	54.1	108	162	81.2	162	244
		X	67.6	135	203	101	203	304
1	A325	N	56.5	113	—	84.8	170	—
		X	70.7	141	—	106	212	—
	A490	N	70.7	141	—	106	212	—
		X	88.4	177	—	133	265	—

Available Angles

Connection Type	Angle Size	t, in.
A, D	4×3	$3/8$-$1/2$
	4×$3\,1/2$	$3/8$-$1/2$
	4×4	$3/8$-$3/4$
B, E	6×4	$3/8$-$3/4$
	7×4	$3/8$-$3/4$
	8×4	$1/2$-1
C, F[b]	8×4	$1/2$-1

[b]Not suitable for use with 1-in. diameter bolts.

For tabulated values above the heavy line, shear yielding of the angle leg controls the design strength.

Table 2.
All-Welded Unstiffened Seated Connections

Outstanding Angle Leg Design Strength, kips

Required Bearing Length N_{req}	Angle Length, in.										Min. Angle Leg
	6					8					
	Angle Thickness, in.										
in.	3/8	1/2	5/8	3/4	1	3/8	1/2	5/8	3/4	1	in.
1/2	27.3					36.5					
9/16	24.3					32.4					
5/8	21.9	58.3				29.2	77.8				
11/16	19.9	55.5				26.5	74.1				
3/4	18.2	48.6				24.3	64.8				
13/16	16.8	43.2				22.4	57.6				
7/8	15.6	38.9				20.8	51.8				
15/16	14.6	35.3				19.4	47.1				
1	13.7	32.4	72.9			18.2	43.2	97.2			
1 1/16	12.9	29.9	67.5			17.2	39.9	90.0			
1 1/8	12.2	27.8	60.8			16.2	37.0	81.0			
1 3/16	11.5	25.9	55.2			15.3	34.6	73.6			
1 1/4	10.9	24.3	50.6			14.6	32.4	67.5			
1 5/16	10.4	22.9	46.7			13.9	30.5	62.3			3 1/2
1 3/8	9.94	21.6	43.4	87.5		13.3	28.8	57.9	117		
1 7/16	9.51	20.5	40.5	79.5		12.7	27.3	54.0	106		
1 1/2	9.11	19.4	38.0	72.9		12.2	25.9	50.6	97.2		
1 5/8	8.41	17.7	33.8	62.5		11.2	23.6	45.0	83.3		
1 3/4	7.81	16.2	30.4	54.7		10.4	21.6	40.5	72.9		
1 7/8	7.29	15.0	27.6	48.6		9.72	19.9	36.8	64.8		
2	6.83	13.9	25.3	43.7	117	9.11	18.5	33.8	58.3	156	
2 1/8	6.43	13.0	23.4	39.8	111	8.58	17.3	31.2	53.0	148	
2 1/4	6.08	12.2	21.7	36.5	97.2	8.10	16.2	28.9	48.6	130	
2 3/8	5.76	11.4	20.3	33.6	86.4	7.67	15.2	27.0	44.9	115	
2 1/2	5.47	10.8	19.0	31.2	77.8	7.29	14.4	25.3	41.7	104	
2 5/8	5.21	10.2	17.9	29.2	70.7	6.94	13.6	23.8	38.9	94.3	
2 3/4	4.97	9.72	16.9	27.3	64.8	6.63	13.0	22.5	36.5	86.4	
2 7/8	4.75	9.26	16.0	25.7	59.8	6.34	12.3	21.3	34.3	79.8	4
3	4.56	8.84	15.2	24.3	55.5	6.08	11.8	20.3	32.4	74.1	
3 1/8	4.37	8.45	14.5	23.0	51.8	5.83	11.3	19.3	30.7	69.1	
3 1/4	4.21	8.10	13.8	21.9	48.6	5.61	10.8	18.4	29.2	64.8	

Weld (70 ksi) Design Strength, kips

70 ksi Weld Size, in.	Seat Angle Size (long leg vertical)				
	4×3 1/2	5×3 1/2	6×4	7×4	8×4
1/4	17.3	25.8	32.7	42.8	53.4
5/16	21.5	32.3	41.0	53.4	66.8
3/8	25.8	38.7	49.1	64.1	80.1
7/16	30.2	45.2	57.3	74.7	93.5
1/2	—	51.6	65.4	83.4	107
5/8	—	64.5	81.8	107	134
11/16	—	71.0	90.0	117	—

Available Angle Thickness, in.

	4×3 1/2	5×3 1/2	6×4	7×4	8×4
Minimum	3/8	3/8	3/8	3/8	1/2
Maximum	1/2	3/4	3/4	3/4	1

For tabulated values above the heavy line, shear yielding of the angle leg controls the design strength.