FURTHER INFORMATION ABOUT EXTENDED SHEAR TABS

Since the introduction of Extended Single Plate Shear Connections (extended shear tabs) in the 2005 AISC Manual, I have addressed a lot of concerns about these relatively common connections. I have compiled some of my responses here.

Let's Twist Again 10/08/2008
A discussion by Larry S. Muir

I am not sure we know who first said that a "A picture is worth a thousand words", but a picture can certainly create a lasting impression that may take more than a thousand words to dislodge. This is the case with the picture that appears on the front page of the report "Design of Extended Shear Tabs" (Sherman and Ghorbanpoor 2002) - which can be downloaded from the AISC Research Library page at www.aisc.org. The picture shows a startlingly twisted extended shear tab.

Table 5.3: Ultimate shear forces, failure modes, and limit states

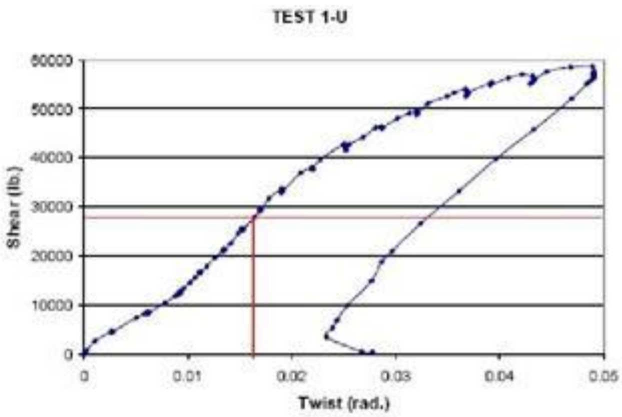
	Experimental		AISC Critical				AISC Typical
			V_n		V_e		V_3
Test	V_{exp} (kips)	Failure* Mode	(AISC e) (kips)	Limit State	(Exp. e) (kips)	Limit State	(kips)
Unstiffened							
1-U	58.7	B (A,D)	45.1	A	65.4	C	85.3
2-U	82.9	F (A,B,E)	94.8	A	100.6	C	142.2
3-U	54.8	E (A)	45.2	A	41.0	E	69.5
3-UM	58.6	E(B,D)	45.0	A	38.3	E	68.5
4-U	98.7	F (A,E)	89.9	A	92.2	A	178.4
6-U	138.0	E,J(B,F,C,K,G)	151.2	A	143.2	E	186.1
6-UB	135.8	E,J(B,F,C,K,G)	151.2	A	151.2	E	186.1
8-U	173.6	E(B,C,K)	213.0	E	194.2	A	279.5
Stiffened							
1-A	58.3	C (F,B)	39.9	B	58.3	C	61.0
1-B	54.6	C (F,B)	39.9	B	61.0	C	61.0
2-A	89.0	C (F,B)	98.3	B	98.3	C	98.3
2-B	92.6	C (F,B)	98.3	B	94.0	C	98.3
2-C	83.3	C (F,B)	98.3	B	95.3	---	---
3-A	53.2	C,F	47.0	B	59.0	C	59.0
3-B	53.1	C,F	47.0	B	59.0	C	59.0
3-C	22.1	C,F	---	---	---	---	---
3-D	51.1	C,F	47.0	B	59.0	C	59.0
3-E	48.1	C,F	39.1	B	59.0	---	---
3-F	68.4	C,B(D)	53.0	B	76.5	D	76.5
3-G	65.1	C,B	53.0	B	76.5	D	76.5
3-H	67.8	C,E	53.0	B	76.5	D	76.5
4-A	103.0	C,F	101.0	B	102.0	C	101.6
4-B	107.0	C,F	92.1	B	102.0	C	101.6
4-C	107.0	C,F	101.0	B	102.0	C	101.6
5-A	122.9	H(B,C,)	146.7	A	171.0	D	176.1
5-B	140.7	B,C,A(E,)	151.6	A	146.3	D	176.1
6-B	124.5	F,B	156.4	A	176.0	D	176.1
7-B	224.2	F,C (B,E,K)	236.0	D,C	218.0	D	235.5
7-C	204.2	H,F(B,C,I,K)	236.0	D,C	218.0	D	235.5
8-A	196.3	B(H,C,K)	260.0	C	261.0	A	250.2
8-B	227.4	F(C,B,K)	260.0	A,C	261.0	C	260.5

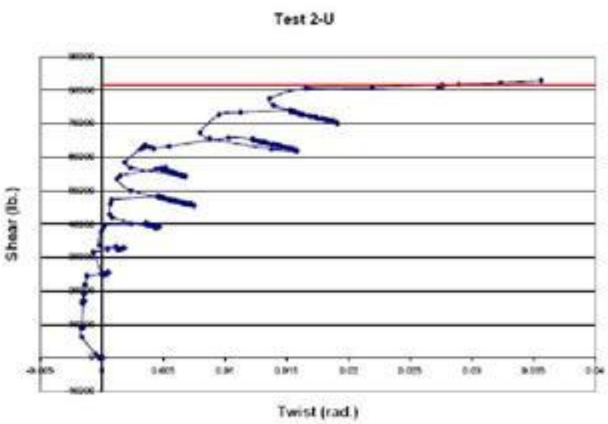
* Limit States and other failure modes:
A = bolt shear B = bolt bearing C = shear yield
D = shear rupture E = web mechanism F = twist G = weld
H = plate buckling I = tearing J = bolt fracture K = web shear

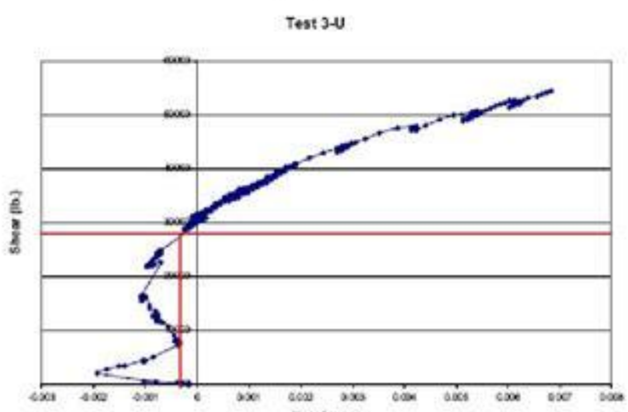
The report states, "Twisting of the shear tab was also observed in a majority of the tests. The twisting was of special concern for the deep tab tests using connections having 5 bolts or more."

Some have suggested that stiffeners must be provided to prevent this twist. However, an examination of Sherman and Ghorbanpoor's Table 5.3 quickly disproves this theory. One quarter of the unstiffened shear tabs are indicated as having failed primarily due to twist. However, more than half the stiffened tabs are shown with twist as a primary failure mode. (Twisting is listed as a failure mode for most of the tests – though not always the primary failure mode.) From this it is clear that the presence or absence of stiffeners has no appreciable effect on twist of the shear tab.

The report states, "The shear value at which either the shear-displacement or shear-twist curve approached a level condition was taken as the ultimate shear capacity of the connection, otherwise, the ultimate load was taken as the highest load achieved." Using this same definition for the capacity based on twist, which seems a reasonable one, we can plot the predicted nominal capacity of the unstiffened tabs on the shear-twist graphs from the Sherman and Ghorbanpoor report. This is shown to the left and on the next page with the red horizontal line representing the AISC 13th Edition Manual predicted nominal (without omega or phi factors applied) strength. It can be seen that the twist failure occurs in all applicable cases at a load that is either equal to or greater than the strength predicted by the AISC procedures. The Virginia Tech tests conducted by Baldwin-Metzger and Murray did not exhibit twist as a failure mode. Nor did any of the tests conducted on standard shear tabs. This is important to note since the distance from the support to the bolt group is not a determining variable in Sherman and Ghorbanpoor's proposed equations for the twist limit state.

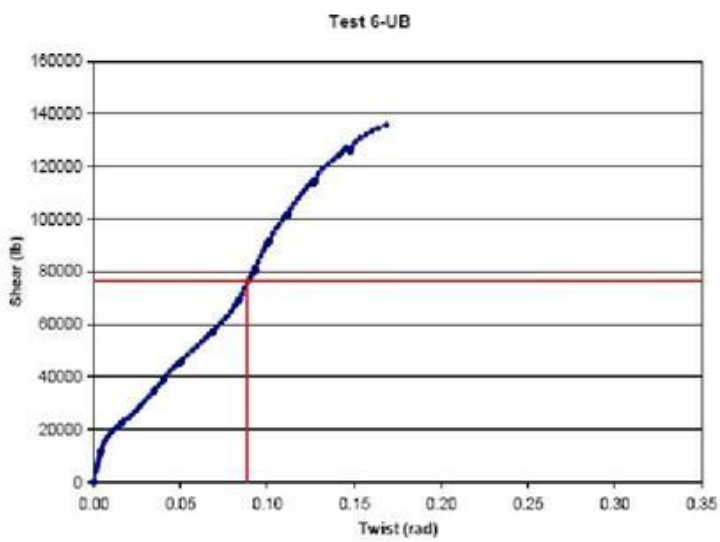


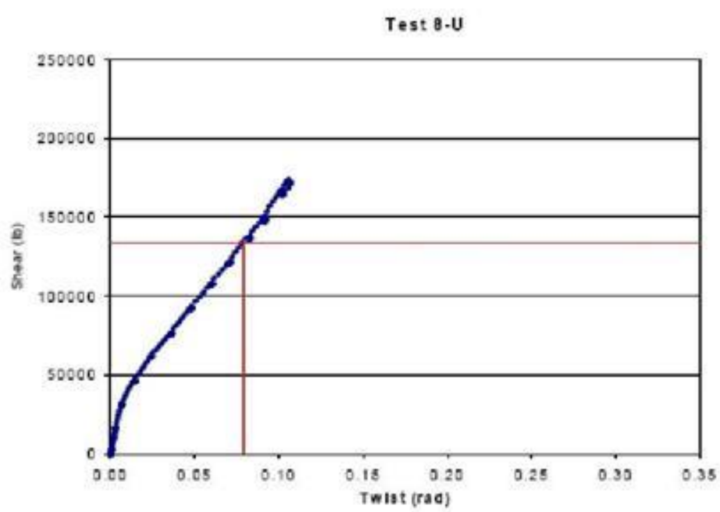




One further argument that has been raised is that the Sherman and Ghorbanpoor tests, as well as the others conducted, used relatively thin plates. It has been argued that an increased plate and/or web thickness might make the twist a greater concern. To examine this, let's assume the plate and the beam web are the same thickness. Also assume that the plate resists all the torsion induced by the shear load, a very conservative assumption. This would cause an eccentricity equal to the thickness of the tab (half the thickness of the tab + half the thickness of the web, which was assumed equal to the tab). Now we can consider the torsional resistance of the tab. The strength of a rectangular bar subjected to torsion is equal to $(F_y)(b)(t)^2/k_1$. k_1 is a parameter related to the depth to thickness ratio of the bar. For all practical cases it can be assumed to be constant and equal to 3, as shown by Sherman and Ghorbanpoor. From this it can be seen that the strength increases by the square of the increase in thickness and obviously at a much faster rate than the increase in applied torsion. Therefore, it is logical to assume than a connection will not be more susceptible to failure due to an increase in thickness.

It should also be noted that Sherman and Ghorbanpoor did not witness an applied torsion equal to the applied shear times the thickness of the tab in the tests, but rather about 1/6 this value, indicating that the beam itself resisted a lion's share of the torsion.

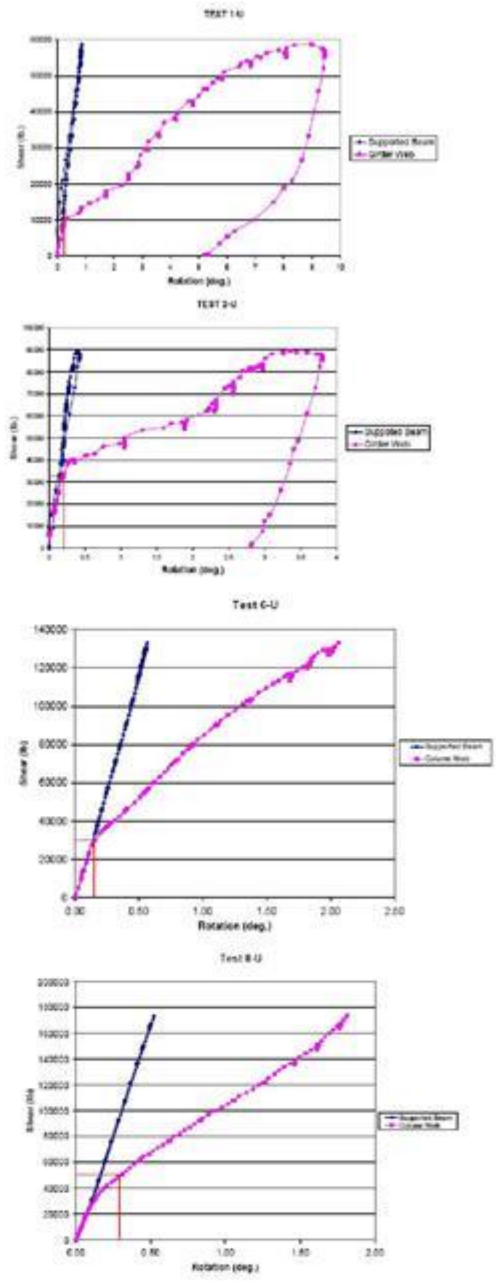




Why then do Sherman and Ghorbanpoor recommend that stiffeners be provided? It was not to prevent twist, since in their proposed twist limit state equation shortening the unbraced length of the tab plate would have no effect. As a matter of fact Sherman and Ghorbanpoor state that this sort of twist, lateral-torsional buckling was not a problem. The report states: “As it is for coped beams, lateral-torsional buckling was initially considered as a limit state associated with twist [3]. However, torsional bracing of the beam near the connection had no effect on the ultimate experimental shear in Phase I and again in Phase III. For this reason, lateral-torsional buckling was not considered to be a limit state. In support of this assumption, load-deflection and load-twist curves (Figures 5.1 and 5.2) indicate that there was no drop in load as is indicative of a stability failure. However, it must be recognized that the shear tab, especially the extended shear tab, has very little torsional stiffness at high loads and rigid body rotation of the beam at the tab must be prevented. The history of using shear tabs in practice indicates that typical lateral bracing and floor/roof framing near the connection are satisfactory for this purpose.”

It should be noted, however, that the potential problem of lateral-torsional beam buckling as related to the need for a torsional simple end support should be considered. In some conditions it may be necessary to add stiffeners for this reason. When a slab or a deck is present that provides continuous lateral support, this is not a concern.

However, back to the question at hand: Why do Sherman and Ghorbanpoor recommend that stiffeners be provided?



It was not to address twist or lateral-torsional buckling, but rather to address excessive deformations at the support web, what is referred to as plastification in Chapter K of the Specification. The report states "It was found in Phase II by examining strain gage data that the stiffeners in the connection provided significant resistance to column web mechanism for the extended shear tab throughout the entire test."

Again we can use graphs, this time the shear-rotation graphs, from the Sherman and Ghorbanpoor report to conclude that this also is not a concern using the AISC procedure. As before, the horizontal red line indicates the predicted AISC connection strength. Significant divergence of the supported beam curve and the girder web curve does not occur until we have approached the predicted capacity. The predicted capacity is also in every case far less than the load at which the test was stopped and failure was assumed to have occurred. It should also be noted that the capacities plotted (the red lines) represent the nominal strength these must be divided by either 2 or 1.67 to obtain the design strength (ASD). This would obviously further limit the rotation on the support in service.

In conclusion I feel that the Extended Tab procedure adopted by the Manual Committee is sound. All of the concerns and possible limit states presented in the Sherman and Ghorbanpoor report have been adequately considered.

Q: AISC *Steel Construction Manual* 13th Edition page 10-103 item 4. Is the equation $M_n = F_{cr}Z$ correct or should it be $M_n = F_{cr}S$?

A: The gross plastic section modulus is the best predictor of the ultimate bending strength of these connections. The buckling check shown in step 5 of the procedure assumes a plastic bending distribution on the plate and conservatively assumes the plastic neutral axis at the mid-depth of the plate. The elastic section modulus, S, is used when checking copes to avoid confusion, since the factors used in other cope checks are all empirical and calibrated to S.

2019 UPDATE: The assertion above that “The gross plastic section modulus is the best predictor of the ultimate bending strength of these connections” was based on work at Virginia Tech by Mohr and Murray. An early and unpublished report of the work stated, “F_y x Z_{gross} shows the best overall agreement with test results. The average agreement value for this model is 1.02, with a standard deviation of 0.09.”

This work was ultimately published in AISC’s Engineering Journal (2nd Quarter 2008). In the published paper the authors conclude:

> “This study indicates that design models used prior to the publication of the 13th Edition AISC Steel Construction Manual (AISC, 2005b) for determining bracket plate and web splice nominal moment strength are overly conservative. From the test results, the available moment strength in LRFD, φM_n, can safely be calculated as the minimum of $0.9F_yZ_{gross}$ and $0.75F_uZ_{net}$, or in ASD as the minimum of $F_yZ_{gross}/1.67$ and $F_uZ_{net}/2.0$, which are the current AISC Manual design models. Consequences of large deflections and supported member or plate instability must be considered when these values are used. If deflection is a concern, the factored loads should also be checked against $0.9F_yS_{gross}$.
>
> Lateral stability is extremely important to reach the maximum plastic moment; therefore, these results are not recommended for coped beams or unbraced bracket plates.”

This conclusion ultimately formed the basis for recommendations in the AISC *Manual*.

I encourage engineers to look at the data and come to their own conclusion, though ultimately it will likely be less controversial to simply conform to the guidance in the AISC *Manual*, which is to calculate the nominal flexural strength as the minimum of F_yZ_{gross} and F_uZ_{net}.

I will also state the question, as presented, considers that the plate may be governed by buckling. The Mohr and Murray did not examine buckling and therefore is not directly applicable. However, the stability checks in the *Manual*, where the double cope checks are adopted for extended shear tabs, also utilize the plastic section modulus as does the check proposed by Muir and Hewitt ("Design of Unstiffened Extended Single-Plate Shear Connections" AISC Engineering Journal. 2nd Q 2009).

Q: AISC *Steel Construction Manual* 13th Edition page 10-103 item 4. In the calculation of Z or S should the bolt holes be deducted?

A: The answer is no. Z_{gross} should be used in this calculation. This was a conscious decisions made by the Manual Committee. The reasoning is based first on research conducted by Tom Murray at Virginia Tech and his graduate student Ben Mohr that concluded that F_yZ_{gross} was the best predictor of the ultimate bending strength of bracket plates. The use of F_yZ_{gross} produced a mean prediction of strength that was 2% greater than the experimental with a standard deviation of 0.09.

The test of extended shear tabs run by Murray and Baldwin-Metzer also confirmed that the net section bending was not a good predictor of strength. Including the shear-bending interaction based on the net section modulus of the plate increases the conservatism by an average of 50%, for those conditions where it controls. This cannot necessarily be taken as conclusive proof that it is appropriate to neglect the net bending section check, since these tests were all conducted with the plates attached to column flanges. Therefore, it might be assumed that the reserve capacity of the column resisted some amount of the eccentricity. However, there were several tests in which the capacity of the eccentrically bolt group was the best predictor of strength and for which the net section interaction was grossly conservative, in most cases 40-80% conservative. This would add credence to using these tests to justify the procedure.

Unfortunately a similar analysis cannot be run on the Sherman and Ghorbanpoor data, since the bolts we considerably weaker than the plate in these tests.

2019 UPDATE: I believe that the response above was accurate at the time it was written. I believe that in 2005, based on an early and unpublished report of the work by Mohr and Murray, the AISC Committee on Manuals decided that Z_{gross} should be used in the determination of flexural strength for extended tabs without consideration of net section rupture.

This guidance was subsequently changed and clarified.

Also see 2019 Update above.

Q: The weld is supposed to develop the strength, but when I calculate the weld required I get $^3/_4t_p$ not $^5/_8t_p$ as listed in the Manual. Is $^5/_8t_p$incorrect?

A: No. The derivation of the $5/8t_p$ weld size requirement is available in Muir and Hewitt ("Design of Unstiffened Extended Single-Plate Shear Connections" AISC Engineering Journal. 2nd Q 2009).

The $^3/_4t_p$ weld can be derived as:

$$0.9(F_y)(t_p)d^2/4= (1.5)(1.329)(D)\ d^2/4$$
$$0.9(50)(t_p)/[\ .75(1.5)(1.329)]= D$$
$$10.8t_p= D$$

$0.675\ t_p = w$

However, there are several areas where this derivation differs from the derivation used to justify the $^5/_8 t_p$ recommendation in the AISC *Manual*.

1. This is a ductility check, not a strength check. There is nothing in the AISC *Specification* that requires this check be done at all. The *Specification* requires that the elements possess sufficient strength to resist the design loads, not the capacity of any of the elements. The *Specification* also requires that simple connections “have sufficient rotation capacity to accommodate the required rotation determined by the analysis of the structure”. Providing a weld that develops the strength of the plate is a means of ensuring sufficient rotation capacity.

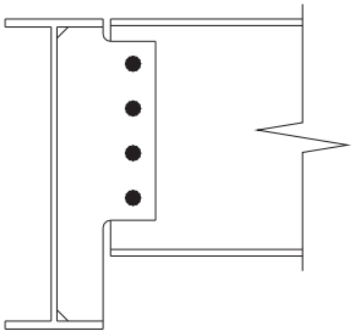
2. The *Specification* contains a resistance factor, ϕ, of 0.9 for limit states that do not lead to joint separation, a resistance factor of 0.75 for limit states that lead to joint separation, but does not provide a resistance factor for ductility checks.

3. The weld size derivation is based on developing the plate, not because this is an empirically based requirement that is plainly evident from the testing, but rather because it is convenient and conservative. There is no need to develop the strength of the plate; it is merely a construct. The goal is to limit the unintended moment and its effect on the connection, including the weld. The weld size requirement is derived using the assumption that the plate must hinge. This assumption is clearly an incorrect, but conservative, assumption. We know the shear applied with certainty, or at least any uncertainty in the shear is directly related to uncertainties concerning the relative stiffnesses of the end connections. The large moment that we fear will develop in the weld will only develop in the weld if there is stiffness there to attract it. This stiffness comes from two sources. First, the support must be sufficiently rigid. This is easily ascertained. However, the connection must also be stiff enough to attract the moment. The unyielded plate may be stiff enough to do this, but as the plate yields the stiffness will quickly diminish. When the plastic hinge forms, the rotational stiffness will be, by definition, zero. However, since yielding initiates at the outer fibers, from which much of the stiffness is derived, the stiffness of the connection will rapidly diminish as yielding progresses from the extremes of the plate towards the center. We do not require zero stiffness to shed excess moment and protect the weld; we only need a stiffness that is low enough to stop attracting the moment. As soon as the plate yields, the effective modulus of elasticity of the yielded portion of the plate drops off significantly, and as an effect reduces the stiffness of the plate. This yielding occurs at a moment that is about 2/3 the moment required to hinge the plate.

4. The $5/8t_p$ recommendation is supported by the Virginia Tech (Murray/Metzger) tests conducted with 50 ksi plate, to rigid supports, and with welds sized to $1/2t_p$. The theoretically derived and empirically substantiated weld requirement is closer $1/2t_{p.}$

Q: Can the *Manual* design procedure be used when the extended tab plate is extended to attach to the bottom flange of the supporting beam?

A: I will preface my response by stating that in general it is a good idea to use a design model that as closely as possible represents the actual condition. If the tab plate is extended to attach the bottom flange, presumably this is done for some reason, which involves the transfer of force between the shear tab and the bottom flange. The design model should reflect this intent.



However, there are times when the tab plate is (or has been) arbitrarily connected to the bottom flange and no force transfer is intended or required. For such cases in my opinion the *Manual* design procedure can still be used.

The single-page first chapter of AISC Design Guide 29 states:

> "All structural design, except for that which is based directly on physical testing, is based either explicitly or implicitly on the principle known as the lower bound theorem of limit analysis. This theorem is important because it allows structural engineers to be confident that 1) their assumptions about the internal force field will not over-predict the strength of an indeterminate structure, and 2) different methodologies for determining an admissible force field, while they may vary significantly in their predictions of the available strength, are nonetheless all valid. This theorem, which was first proven in the form given in the following in the 1950s (Baker et al., 1956), states that:
>
> > Given: An admissible internal force field (i.e., a distribution of internal forces in equilibrium with the applied load)
> >
> > Given: Satisfaction of all applicable limit states
> >
> > Then: The external load in equilibrium with the internal force field is less than, or at most equal to, the connection capacity.
>
> The lower bound theorem is applicable to ductile limit states, and most connection limit states have some ductility."

Since the *Manual* design procedure is based on a statically admissible force distribution that requires all applicable limit states (based on the assumed statically admissible force distribution) to be satisfied, it will produce a safe design – assuming that the condition remains ductile. I suspect that generally these conditions should possess sufficient ductility to justify the use of the *Manual* design procedure.

Fortunately this assumption can be examined against empirical results. The paper "Flexural buckling of extended shear tab connections under gravity induced shear force" (Proceedings of the Annual Stability Conference, Structural Stability Research Council. Nashville, Tennessee, March 24-27, 2015) provides load-deformation curves for several conditions as shown here.

Similar to what was done above in the discussion of twist, the predicted strengths from the AISC *Manual* design procedure as reported by the authors is plotted in red. The authors conclude, “After comparing the observed and expected buckling strength based on the AISC equation for buckling of the doubly coped beam, it is concluded that this equation overestimates the buckling strength for beam-to-column shear tabs. Similar findings hold true for the extended beam-to-girder shear tabs.” Though their conclusion may be factual, it is also likely irrelevant in terms of practical design. As can be seen each of the connections either reached its predicted strength while still exhibiting significant stiffness or was still exhibiting ductile behavior with no indication of imminent connection failure when the test was stopped. There is nothing in these results that indicates that applying the *Manual* design procedure to such conditions would result in an unsafe prediction of the in-service strength of these conditions.

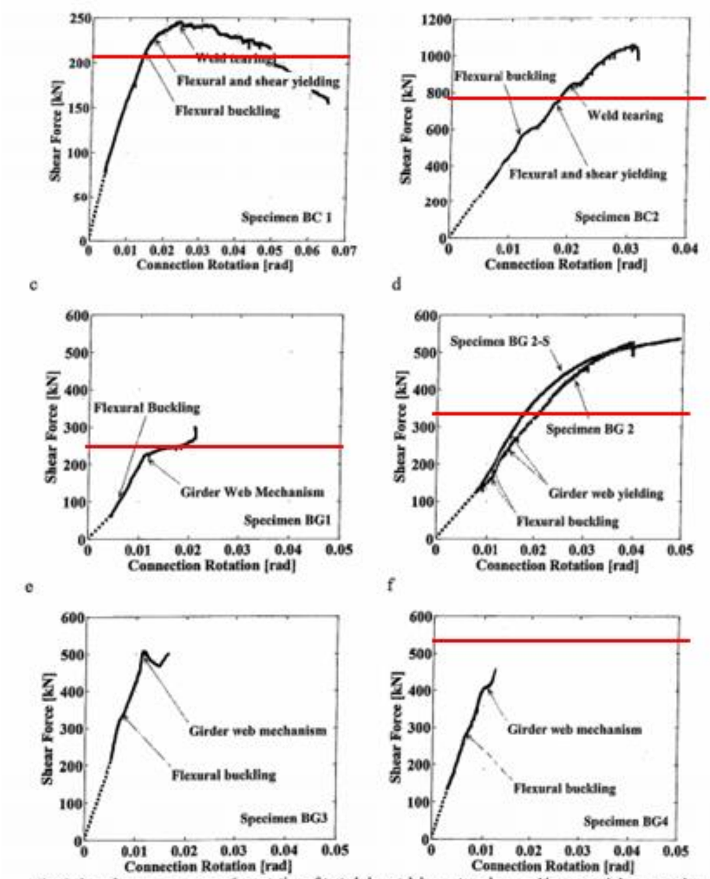


Fig. 5-shear force versus connection rotation of tested shear tab beam-to-column and beam-to-girder connections

It might be noted that two of the tests, BG3 and BG4, did not reach their predicted strengths. Of the six tests shown only two were tested to failure, and these involved beam-to-column conditions. None of the beam-to-girder conditions were tested to failure, as was presumably intended by the researchers. Tests BG1, BG3, and BG 4 were “stopped due to binding of the bottom flange of beam to the shear plate”. Test BG2 “was stopped due to limitation of vertical displacement of lateral bracing system”.

It is worth considering the behaviors that led to the failures in these tests. I think the “limitation of vertical displacement of lateral bracing system” was simply a deficiency in the test setup and reflects conditions that are unlikely to exist in practice. On the other hand bottom flange binding should be given some additional consideration. Binding of the connection, a state in which the elements bear upon each other in a manner that is not intended or anticipated in the design, commonly results in a significant increase in stiffness and strength followed by rupture due to the increased demand associated with the sudden increase in stiffness. It is a known issue and has been reported on many times. It typically is a concern for deep connections or connections to beam subjected to large end rotations.

As can be seen in the plots for Tests BG1, BG3, and BG 4 binding, and the subsequent failure, occurred at relatively modest end rotations. This is not especially surprising as previous researchers have also struggled with these issues. The tests in the article were conducted without a slab. Full-scale test incorporating concrete slabs are costly, so it is not unexpected that the tests would omit the slab. However, it must be recognized that the presence of a slab will prevent (or delay) binding in typical conditions. Some previous researchers have elected to include horizontal plates joining the top flange of the beam and girder to simulate the effect of a slab and better represent typical in-service conditions. The article indicates that only test BG2 was replicated with such a simulated slab. BG2 apparently did not experience binding wither with or without the simulated slab, so it appears that no conclusion can be drawn from these tests relative to the effect of the slab.

Returning to the effect of extending the shear tab to the girder's bottom flange, it is true that the *Manual* procedure does not accurately predict the behavior of the extension to the bottom flange. This is not surprising, since the *Manual* procedure assumes no such extension exists. However, in terms of practical design this is not necessary, as seems to be confirmed by the empirical data and as would be predicted by the lower bound theorem of limit analysis. A corollary to the lower bound theorem indicates that a structure cannot be weakened through additional strength or restraint – again assuming sufficient ductility. By extending the shear tab to the bottom flange we are adding restraint. But how do we explain what has been observed?

From discussions with one of the researchers and based on inspection of the conditions tested, it seems reasonable to assume that during the early stages of loading the extension to the bottom flange behaves in a manner similar to a stiffened seat. Early in the loading the un-yielded and un-buckled extension is stiff and the load is attracted to stiffness. Since the extension has not been designed to carry any portion of the load, it is perhaps not surprising that at some point its capacity to do so is exceeded and the extension "fails". It is also not surprising that the governing mode is buckling (stability-based).

What may be surprising to some, though not unexpected based on the lower bound theorem, is that this failure is of little consequence. The extension buckles and loses stiffness with little more than a hiccup being observed in the load-deformation curve. The loss of strength and stiffness simply allows the connection to revert to a condition that in many respects more closely resembles the assumed design model. The design model assumes that the extension does not exist, and at some point the extension buckles, loses strength and stiffness, and in effect ceases to exist.

The paper (Muir and Hewitt 2009) introducing what was to become the *Manual* design procedure states, "Fearing that the plate might buckle or that the weld might fracture, many designers have chosen to detail the connections with top and bottom stiffening plates or to extend the plate and connect it to the top and bottom flanges of a supporting girder (Figure 2). Ironically, testing has shown that extending the plate vertically in this manner could actually result in a lower plate buckling strength (Sherman and Ghorbanpoor, 2002) and is, in many cases, unnecessary… Plate buckling was not listed as a failure mode for any of the unstiffened, extended single-plate connection tests. Interestingly when the plate was extended to the bottom flange of the supporting girder, as illustrated in Figure 2, the plate was more likely to buckle." I believe the authors' statements are misleading, which I find unfortunate since I am one of the authors. Like in the McGill University tests, that have been here discussed, the Sherman and Ghorbanpoor do not exhibit a reduction in overall connection strength when the shear tab is extended to the bottom flange in the girder. It is true that "the plate was more likely to buckle" and that this could be classified as ironic since the goal of the extension seems often to be to prevent buckling, but again the buckling is inconsequential in terms of practical design.

Though providing the extension to the bottom flange will likely not be detrimental in terms of strength, doing so commonly provides no benefit and incurs additional cost relative to fabrication. If an extension to the bottom flange is provided to achieve some specific behavior, then (obviously) the presence of the extension should be explicitly considered and the *Manual* design procedure is not applicable. Therefore the discussion above is primarily useful in evaluating existing conditions where the extension has been provide but serves no real purpose. In new construction it is probably best to omit the extension, and simplify both the design and the fabrication.

Q: Can the *Manual* design procedure be used when the extended tab plate is connected to bottom and bottom stiffeners?

A: The considerations involved with this configuration are very similar to the considerations related to the extension of the shear tab to the girder bottom flange discussed above.

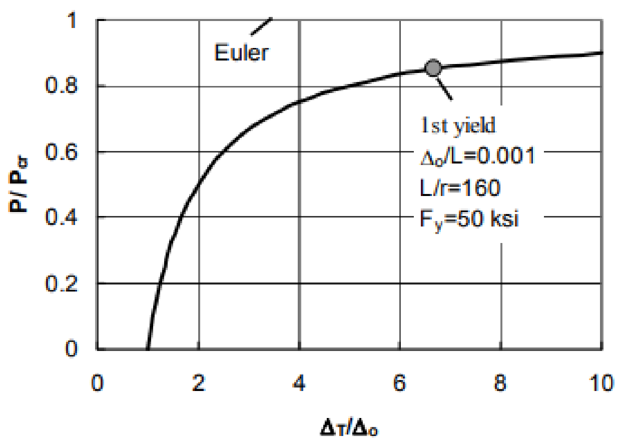
Again, I will preface my response by stating that in general it is a good idea to use a design model that as closely as possible represents the actual condition. The design procedure in the *Manual* is for an unstiffened plate, so ultimately this again becomes a question of whether it is reasonable to neglect the effect of the stiffeners. In other words whether there is sufficient ductility to overcome the difference between the assumed design model and the actual condition.

The primary concern here is likely not the strength of the connection but rather the strength of the column. Common practice is to assume that the column is pinned between points of restraint. Tab plates, whether of the conventional or extended configuration, tend to be stiffer than other shear connections. When attached to the weak-axis of a column the inherent stiffness of the plate is countered by the flexibility of the column web – unless of course it is connected by stiffeners. With stiffeners in place considerable moment may be transferred to column, and the concern becomes that this moment, not accounted for in the design, may cause failure of the column.

This issue has been addressed since the 2nd Edition of the LRFD Manual (1994), which presents a model in which "the partial restraint of the connection… actually stabilizes the column and reduces its effective length factor, K, below the originally assumed value of 1." In other words the demand on the column is increased, but so too is its strength. (See also Ioannides "Minimum Eccentricity for Simple Columns", ASCE Structures Congress Proceedings, Volume 1, 1995.) However, this does not represent the entire underpinnings of the *Manual* the argument. Ultimately the *Manual* argument that it is conservative to design the connection for the shear, R, and the eccentric moment, Re, while designing the connection as pinned-pinned without moment relies on the lower bound theorem. The Manual states, “If provision is made for ductility and stability, it follows from the lower bound theorem of limit states analysis that the distribution which yields the greatest strength is closest to the true strength.”

The *Manual* model might be challenged based on the need to ensure ductility in order to apply the lower bound theorem at all. The strength of the column is likely to be governed by buckling, and (as we were all taught in college) buckling is a sudden sideways deflection accompanied by an equally sudden decrease in strength – behavior that can be described as anything but ductile.

However, in his excellent discussion “Five Useful Stability Concepts” Yura argues that such sudden failures that we typically associate with buckling will only occur in theoretical columns that are perfectly straight. “Since all members in a structure have some initial out-of-straightness, flexural displacements will get very large near the theoretical buckling load thus providing ample warning of impending disaster.” In the discussion Yura provides the plot to the left. Though demonstrating no clear “yield point” the plot does resemble in some respects the softening that often accompanies yielding and which not only provides ample warning of impending disaster but perhaps enough ductility to justify the use of the lower bound theorem.



Plastic Methods of Structural Analysis (Neal 1956) provides plots taken from von Karman (1910) that indicate that slender columns exhibit buckling behavior consistent with the use of the lower bound theorem, while for “lower slenderness columns where yield occurs simultaneously with or even before buckling… the load reaches a maximum and then drops off more or less abruptly to much smaller value – a condition that would not lend itself to the use of the lower bound theorem.

In the paper, “Analysis and Design of Stabilizer Plates in Single-Plate Shear Connections” (Fortney and Thornton, Engineering Journal, 1st Quarter 20016) the authors recommend that “the weak axis rotational demand imparted to the column, Mc, should be accounted for when sizing the column.” This statement is consistent with the position presented here that “in general it is a good idea to use a design model that as closely as possible represents the actual condition”. However, while certainly a good idea, this may not always be necessary.

The best advice also from Fortney and Thornton is “only use a stabilizer plate when all other alternatives, such as changing the geometry and proportioning of the connection, have been exhausted to no avail.” There will generally be no reason to engage stiffeners when designing extended shear tabs. Though providing stiffeners may not be detrimental in terms of strength to either the connection or the column, it does incur additional cost relative to fabrication. If stiffeners are provided to achieve some specific behavior, then (obviously) the presence of the stiffeners should be explicitly considered and the *Manual* design procedure is not applicable. Therefore the discussion above is primarily useful in evaluating existing conditions where stiffeners have been provide but serves no real purpose. In new construction it is probably best to omit the stiffeners, and simplify both the design and the fabrication.

Q: Must the weld always be sized to 5/8t$_p$ as recommended in the *Manual*?

A: No. The following provides further guidance:

Proper Application of the $^5/_8$tp Weld Size Criterion

Part 10 of the *Manual* contains recommended design procedures for single-plate shear connections. For both the conventional and extended configurations the *Manual* recommends that “the weld between the single plate and the support should be sized as $(^5/_8)t_p$, which will develop the strength of either a 36-ksi or 50-ksi plate.” The weld is sized such that the plate will yield prior to the weld fracturing, allowing the plate to act as a fuse that accommodates the beam end rotation in a ductile manner (Muir and Hewitt 2009). It should be noted that this is only a recommendation. There is no provision of the *Specification* requiring that the weld be stronger than the plate. Instead the $(^5/_8)t_p$ requirement is used as a means of satisfying Sections B3.4a and J1.2 of the Specification. Section B3.41 requires that, “A simple connection shall have sufficient rotation capacity to accommodate the required rotation determined by the analysis of the structure.” Section J1.2 requires that, “Flexible beam connections shall accommodate end rotations of simple beams. Some inelastic but self-limiting deformation in the connection is permitted to accommodate the end rotation of a simple beam.”

Rather than requiring engineers to determine the simple beam end rotation for every beam receiving a single-plate shear connection, the *Manual* procedure is intended to accommodate rotations of about 0.03 radians, a rotation that exceeds the end rotation required of serviceable beams. In other words the recommended $(^5/_8)t_p$ weld size reflects a conservative simplification.

Consideration of Other Mechanisms

The design procedures for single-plate shear connections provided in the Manual assume that plate yielding in some form accommodates simple beam end rotation. The conventional configuration relies on bolt plowing, local yielding due to bearing at the plate (or potentially the beam web). The extended configuration primarily relies on flexural yielding of the plate, with bolt plowing considered in some cases. These are not the only mechanisms that can accommodate simple beam end rotation. In reality a combination of mechanisms will be mobilized to accommodate the rotation. It may not be necessary to adhere to the $(^5/_8)t_p$ weld size recommendation when other mechanisms are available. Some commonly encountered conditions are discussed below.

Support flexibility

Single-plate connections framing to the webs of supports are commonly encountered in practice. When there is no connection framing to the opposite side of the web, the flexibility of the web can often be relied on to accommodate the end rotation of the supported beam. In such cases the maximum demand that can be delivered to the weld may be limited based on the weak-axis flexural strength of the supporting web, rather than the strong-axis flexural strength of the plate. The weak-axis flexural strength of the supporting web can be established using a yield-line analysis (Abolitz and Warner 1965, Stockwell 1974). Though yield-line analysis is an upper bound approach and therefore should produce a safe estimate of the maximum demand that can be delivered to the weld, a yield-line analysis typically neglects the contribution of membrane action. However, due to the deformation required to mobilize membrane action and the significant loss of stiffness likely to accompany such large deformations, the strength predicted from a yield-line analysis can be used to determine the demand on the weld.

A similar model applies to single-plate connections framing to the face of HSS members.

Slotted holes

Table 1

Number of Rows	Rotation (radians)
2	0.085
3	0.042
4	0.028
5	0.021
6	0.017
7	0.014
8	0.012
9	0.010
10	0.009
11	0.008
12	0.008

For bolt diameters most commonly used for shear connections, ¾-in. and $^7/_8$-in., short slotted holes provide ¼-in. horizontal clearance. This clearance theoretically allows unrestrained, rigid-body rotation to occur without any demand being placed on the bolts or the weld. For a single column of bolts the rotation allowed is dependent on the diameter and spacing of the bolts and the depth of the bolt group (Muir and Hewitt 2009). The rotation that can be accommodated for typical connections with short slotted holes at 3-in. spacing is shown in Table 1.

However, in practice even connections design for bearing and with the bolts installed snug-tight will possess some slip resistance. The AISC *Specification* provides no limits on the installed pretension of snug-tight bolts. In practice it is not uncommon for connections specified as snug-tightened to be installed with preload approaching, or even exceeding, the minimum bolt pretensions listed in Table J3.1 of the *Specification*. When determining the maximum demand that can be delivered to the weld, it is conservative to overestimate the pretension in the connection and the slip coefficient at the faying surface.

Theoretically, the slip resistance could equal or exceed the shear strength of the bolts. The mean slip coefficient for unpainted clean mill scale or hot-dipped galvanized and roughened surfaces is taken to be 0.30 in the *Specification*. However, the largest reported slip coefficient in the Bolt Guide () is about 0.74 for a blast-cleaned surface. If the installed pretension is assumed to be equal to the nominal tensile strength of the bolt, taken as 075F_u in the Specification, then the slip resistance is calculated as:

$$R_{slip} = \mu T_b = 0.74(0.75F_u) = 0.555F_u$$

This is nearly equal to the nominal shear strength of a bolt when threads are excluded from the shear planes, 0.563F_u, given in the *Commentary* to the *Specification*.

Section 2.5.2 of AISC Design Guide 16 provides estimates of bolt pretension for A325 snug-tightened bolts. These could be used to determine the moment that could be delivered to the weld, if steps are taken to limit the pretension introduced during installation. Using the pretension values in Design Guide 16, say 50% of the minimum specified pretension, and an estimated maximum slip coefficient for clean mill scale from the Bolt Guide, say 0.56, then the slip resistance is calculated as:

$$R_{slip} = \mu T_b = 0.56[(0.50)(0.70)F_u] = 0.196F_u$$

This might be a more reasonable estimate in practice. Consideration of the actual conditions that exist and some judgment is required.

Once an assumed slip resistance is established the flexural demand on the weld, M_{weld}, can be determined as:

$$M_{weld} = R_{slip}C'$$

A description C’ is provided in Part 7 of the *Manual*.

Required Rotation Determined by Analysis

As stated previously 0.03 radians is commonly assumed as an upper bound rotation for shear connections. The *Specification* requires only that the connection accommodate the rotation determined by the analysis of the structure, the simple beam end rotation. The end rotation of a beam can be calculated from mechanics. For a simply-supported beam with a uniformly distributed load, *w*, the end rotation is:

$$\theta_{end} = \frac{wl^3}{24EI}$$

For a simply-supported beam with a concentrated load, *P*, at mid-span the end rotation is:

$$\theta_{end} = \frac{Pl^2}{16EI}$$

To determine the flexural demand on the weld, M_{weld}, the single-plate connection can be modelled as a cantilever beam, fixed at the welded end with a length l_p. The moment delivered to the weld is:

$$M_{weld} = \frac{\theta_{end} E I_p}{l_p}$$

M_{weld} as calculated above is actually the moment at the bolted side on the single-plate connection, modeled as a cantilever. The moment at the weld will be slightly smaller as it will be reduced by the beam reaction, *R*, times the length of the connection, l_p. The reduction will generally be small and is therefore conservatively neglected here.

Substituting the plate properties for the moment of inertia of the plate, I_p, produces:

$$M_{weld} = \frac{\theta_{end} E t_p d_p^3}{12 l_p}$$

For a simply supported with a uniformly distributed load the moment delivered to the weld is:

$$M_{weld} = \frac{RL^3 t_p d_p^3}{144 I l_p}$$, where R is the beam end reaction.

The weld size need not exceed $(^5/_8)t_p$, regardless of the size calculated based on the end rotation.

Some commonly encountered conditions will produce little rotational demand on single-plate shear connections. Member commonly referred to as transfer girders transfer reactions from columns that end above the foundation to the surrounding columns. Due to their intended function sections used as transfer girders are typically deep and heavy. Transfer girders often have large end reactions and short spans. The large reactions often require thick plates. Though engineers sometimes apply the $(^5/_8)t_p$ weld size recommendation to transfer girder connections, this is generally not necessary. Rotational capacity is often not a significant concern for transfer girder connections and can usually be judged sufficient by inspection. If need be the demand can be determined as described above.

Other Conditions

Numerous conditions of welded plates are encountered in practice. The $(^5/_8)t_p$ weld size recommended in Part 10 of the *Manual* is intended to be applied only to the design of single-plate shear connections supporting simply supported beams.

Engineers may choose to apply the $(^5/_8)t_p$ weld size criterion to other conditions. However, care should be exercised, as the resulting weld will often be larger than required though some applications may actually demand an even larger weld.

Some common conditions are discussed below. Typically for the conditions discussed below, with the exception of high-ductility bracing systems, rotational ductility is not a significant consideration and rotational ductility can be judged to be sufficient based on engineering judgment. For the rare condition where a more rigorous analysis is warranted, some guidance is provided.

<u>Moment connections</u>

Single plates are commonly used to connect the web of moment-connected beams. By definition fully-restrained moment connections permit negligible rotation between the connected members. There is no need to develop the strength of the single-plate connection when little rotation will occur.

For partially-restrained moment connections the rotation between the connected members will not be negligible. However, the rotation will be considerably less than that of simple connections. Rotational ductility of the web connection is likely not a practical concern, but it would be investigated as described in the section above, Required Rotation Determined by Analysis.

<u>Beam-columns</u>

Collectors (sometimes referred to as drag beams or drag struts) are often subjected to a combination of gravity loads (producing bending in the beam) and lateral loads (producing axial demand in the beam). Such members are referred to as beam-columns and can be connected with single plate connections. Member subjected only to axial load (columns, struts, hangers, etc.) will produce little rotational demand on the connection. The rotational demand for members subjected only to transverse loads within the beam span is addressed above. Between these extremes exist a wide range of conditions that could be encountered in practice.

Some general trends related to beam-columns can be established. Beam-columns subjected to large axial loads but modest bending will generally place little rotational demand on the end connections. Beam-columns subjected to small axial loads but significant bending will generally place larger rotational demand on the end connections. Beam-columns will generally have larger moments of inertia than beams subjected only to bending; this will tend to reduce the end rotation. Tension in the beam-column will tend to decrease the end rotation caused by bending in the member, and compression in the beam-column will tend to increase the end rotation caused by bending in the member.

The rotational demand for beam-columns subjected to combined bending and tension could be conservatively determined using the above, Required Rotation Determined by Analysis, while neglecting the axial load.

Though tending to underestimate the rotation demand an analysis neglecting the axial load might also be used to determine whether or not rotational demand is a significant consideration. In the rare instances where rotational demand is a significant consideration, a more rigorous analysis could be performed.

Vertical bracing connections

Single plate connections are often used to join beam webs and gusset plates in vertical bracing connections to columns. There is generally no need to apply $(^5/_8)t_p$ weld size recommendation at vertical bracing connections. Braces frames are inherently stiff and the elastic drift, and therefore the rotation at the column connections, will be small. Rotational demand is typically not a significant consideration for vertical bracing connections.

However, there are conditions for which the $(^5/_8)t_p$ weld size criterion may not be sufficient to ensure ductile behavior. Seismic force resisting systems that rely on highly ductile behavior, such as special concentrically braced and buckling-restrained braced frames, may experience inelastic drifts many times the elastic drifts (Thornton, 2009) combined with out-of-plane movement of the gusset plate

(Astaneh-Asl et al., 1986, Roeder et al., 2011, Lehman et al., 2008, Lopez et al., 2004). In such cases the weld can be sized to develop the maximum weak-axis moment occurring in combination with the shear, compression, and strong-axis moment that result on the gusset plate edge from the brace compression force (Carter et al., 2016).

Shear tabs welded to stiffeners

Single-plate shear connections to column webs are sometimes welded to stiffeners that have been provided as column reinforcing due to a moment connection to the strong axis of the column. When this is done, the entire C-shaped weld, instead of only the vertical weld, can be used to develop the strength of the shear plate. For conditions typically encountered in practice, the minimum size of fillet welds shown in Table J2.4 of the Specification will develop the strength of the shear plate.

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Q: Can the bolted connection to the supported member at an extended shear tab be welded?

A: There is nothing in the AISC *Specification* that explicitly prohibits the use of a welded connection.

The *Specification* requires, “A simple connection transmits a negligible moment. In the analysis of the structure, simple connections may be assumed to allow unrestrained relative rotation between the framing elements being connected. A simple connection shall have sufficient rotation capacity to accommodate the required rotation determined by the analysis of the structure, ” and, “Simple connections of beams, girders and trusses shall be designed as flexible and are permitted to be proportioned for the reaction shears only, except as otherwise indicated in the design documents. Flexible beam connections shall accommodate end rotations of simple beams. Some inelastic but self-limiting deformation in the connection is permitted to accommodate the end rotation of a simple beam.” Both of these statements recognize the need for rotational ductility.

The AISC *Manual* design procedures for shear tabs are intended to address the need to accommodate simple beam end rotations. A de facto standard for this rotation has become 0.03 radians. This is actually a very large demand and represents something akin to a beam whose length in feet is 2-3 times its depth in inches as it forms a plastic hinge at its center under a point load. If the end rotation is significantly less than this, which it may be in many practical conditions, then it makes sense that the detailing recommendations could be relaxed.

In the case of a conventional tab plate all of the end rotation is assumed to be accommodated through plowing of the bolts, which obviously would not occur if the connection were welded. There are however other mechanisms that could be used to accommodate the end rotation. For instance, for the case of a connection to a column (or beam) web, if no beam is present on the other side and the supporting web is not excessively thick, the simple beam end rotation would be accommodated through weak-axis flexure of the web.

Another mechanism that can be used to accommodate simple beam end rotations is flexure of the plate. This method is used primarily for extended tabs but there is no reason it could not be applied to a conventional single plate shear connection as well. A paper on the design of extended tab can be found here:

Muir, Larry S.; Hewitt, Christopher M. (2009). "Design of Unstiffened Extended Single-Plate Shear Connections," Engineering Journal, American Institute of Steel Construction, Vol. 46, pp. 67-80.

When applying strong-axis flexural yielding to an all-welded conventional tab I tend to have concerns about the relatively small distance that might exist between the welds at the supported beam and the welds at the face of the support. If the length is too small the plate or weld could rupture before the connection elongates enough to accommodate the rotation. For this reason I have typically tried to use only the vertical weld at the end of the plate or if horizontal top and bottom welds are used hold them back somewhat to provide a larger length over which the tab plate can yield. I would try to avoid a vertical weld between the beam end near the support. When addressing a field fit-up problem, the welds should probably be made beyond the line of holes in the tab plate. This is in some ways ideal. If you think about it what we are doing is using the plate as a fuse to protect the less ductile welds (or bolts). The open holes in the plate provide a reduced section somewhat like (though certainly not as ideal) the “dog-bone” in an reduced beam section (RBS) moment connection.

When sizing the welds I would make sure that their strength relative to a moment is greater than that of the plate's flexural strength (first yield criterion). This is similar to what is done in the paper referenced above. I would use the instantaneous center method described in Part 8 of the *Manual* to determine the weld strength. Note that as described in the paper the location of the instantaneous center is the centroid of the weld group (similar to the bolts described in the paper) so there will be no need for iterations.

What is described above has been my preferred approach. I am also aware of the research on all-welded shear tabs that employ welds all-around the shear tab at McGill University. Though I read some early reports on the work and have attended a presentation by the researchers, I have not thoroughly reviewed the conclusions presented. My impression however is that the all-welded connections have performed better than might have been expected.