On the Need for Stiffeners for and the Effect of Lap Eccentricity on Extended Single-Plate Connections

WILLIAM A. THORNTON and PATRICK J. FORTNEY

ABSTRACT

The design procedure for extended single-plate connections presented in the 13th edition of the *Steel Construction Manual* contains many design checks to ensure satisfactory performance but does not include a check of lateral-torsional stability of the extended single plate, which resembles a double-coped beam. Research has shown that coping of beams can reduce the lateral-torsional buckling strength of beams. This paper presents a proposal to use the double-coped-beam concept to ensure the lateral-torsional stability of the extended plate. The question of stiffeners and the effect of the small eccentricity due to the lapping of the plate with the beam web will also be addressed.

Keywords: extended single-plate connections, lap eccentricity, shear tabs, stiffeners.

The design procedure for extended single-plate connections (also known as extended shear tabs) presented in the 13th edition of the AISC *Steel Construction Manual* (AISC, 2005, pp. 10-102 – 10-104), hereafter referred to as the *Manual,* contains many design checks to ensure satisfactory performance, but it does not include a check on the effect of the extended tab on lateral-torsional stability. An extended single-plate connection to a beam end resembles a beam with a double-coped end. Research (Cheng et al., 1984) has shown that coping of beams can reduce the lateral torsional buckling strength of beams below what can be achieved with the same beam without coped ends. Because of the similarity between the extended shear tab and the double-coped beam end, this paper presents a proposal to use the latter to ensure the lateral-torsional stability of the former. The question of optional stiffeners and the effect of the small eccentricity due to the lapping of the shear tab with the beam web will also be addressed.

A research study on extended tabs (Sherman and Ghorbanpoor, 2002) recommended that unstiffened tabs not be used because of excessive lateral twist. The study considered eight unstiffened extended tabs, and the beams were essentially laterally unsupported for their full span. Table 1 contains the data from these tests necessary to understand that, using the design procedure for extended tabs presented in the *Manual*, the shear tabs will not twist excessively.

The excessive twists reported by Sherman and Ghorbanpoor occurred at the ultimate capacity of the beam connection system at a load far in excess of the capacity that would be calculated using the recommended AISC design procedure. This can be seen in Table 1. For instance, for test 1-U, the reported ultimate capacity is 58.7 kips, and the AISC Allowable Strength Design (ASD) capacity of the system is 11.3 kips. The elastic twist at 11.3 kips is 0.02 radian, which is negligible considering that the calculated elastic twist corresponds to a lateral translation of the tab equal to $\theta l/2 = (0.02)(9 \text{ in.})/2 = 0.09$ in. The tab thickness for test 1-U was 0.371 in., giving a lateral translation of only 24% of the thickness of the tab. Similar observations can be made for the other seven tests presented in Table 1.

The beam strength information (uniform design load, or UDL) reported in Table 1 is given to show that the connection strengths in these eight tests were not matched to the beam strengths. These connection strengths are very low compared to the beams they are supporting and do not represent practical connections. The physical tests were effectively unbraced so the “UDL Unbraced” column of Table 1 gives the beam strength of the actual tests. These are seen to be much larger than the connection strength. Connection 1-U, for example, can only carry (11.3/68.2)(100) = 16.5% of the total load the beam can carry. Practically, one would consider this percentage to be on the order of about 50%.

THE QUESTION OF OPTIONAL STIFFENERS

Figure 10-12 of the *Manual* shows “optional” stiffeners. The purpose of this discussion is to provide some guidance as to when stiffeners may be required. Considering that a

William A. Thornton, Corporate Consultant, Cives Steel Company, Roswell, GA (corresponding). E-mail: bthornton@cives.com

Patrick J. Fortney, Chief Engineer and Manager, Cives Steel Company, Roswell, GA. E-mail: pfortney@cives.com

Table 1. Evaluation of Data Reported by Sherman and Ghorbanpoor (2002)									
Test Case	Reported Ultimate Load (kips)	Lap Eccentricity (in.)	Loads		Strength	M_t*** (kip-in.)	Calculated Angle of Twist, ϕ (rads)	Calculated Lateral Translation (in.)	Percentage of Lateral Translation** (%)
			ASD UDL (CLB)* (kips)	ASD UDL (unbraced) (kips)	AISC Conn. Strength R_a(ASD) (kips)				
1-U	58.7	0.443	43.9	34.1	11.3	5.0	0.020	0.090	24.3
2-U	82.9	0.433	72.8	42.8	33.1	14.7	0.032	0.243	65.4
3-U	54.8	0.443	43.9	34.1	11.3	4.5	0.018	0.081	22.0
3-UM	58.6	0.443	43.9	34.1	11.3	4.5	0.018	0.081	22.0
4-U	98.7	0.501	72.8	42.8	22.0	10.6	0.015	0.110	21.7
6-U	138	0.578	126	80.0	31.2	17.4	0.020	0.181	35.8
6-UB	136	0.578	126	80.0	31.2	17.4	0.020	0.181	35.8
8-U	174	0.578	152	68.4	52.9	29.6	0.026	0.307	60.7

*CLB = continuously laterally braced
**Percentage of lateral translation of the tab relative to the tab thickness
*** $M_t = (R_a)$(lap eccentricity)

double-coped beam and a beam with an extended shear tab are similar in geometry, we can use research on double-coped beam lateral-torsional stability to estimate the lateral-torsional stability of beams with extended tabs. Cheng et al. (1984) show that the lateral-torsional buckling (LTB) mode of double-coped beams occurs primarily in the double-coped sections (the extended tab), and the uncoped beam acts essentially as a rigid body. In their research, this is apparent for beams that are laterally unbraced or braced only at the midspan load point. For a beam laterally supported for its full span—that is, $L_b \le L_p$—the authors postulate that the uncoped portion of the beam can be treated as a rigid body and that lateral-torsional buckling is dependent solely on the coped section or extended tab.

Theory

With the previous assumption, the LTB capacity of the tab (coped section) under pure moment is (see Figure 1):

$$M_{rec} = \frac{\pi}{2a}\sqrt{EGI_yJ} \qquad (1)$$

where M_{rec} is the pure moment buckling strength of the rectangular section of the coped beam with a cope length, or shear tab length, of a.

As stated by Cheng et al., the pure moment case is a conservative approximation to the actual case where the moment in the tab varies from zero at the support to a maximum at the junction with the uncoped section of the beam.

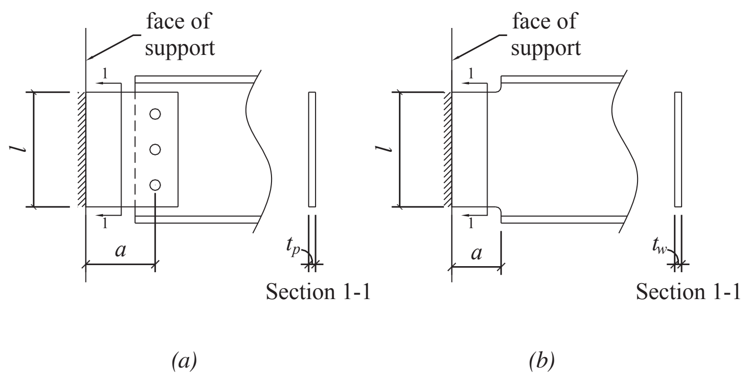

Fig. 1. (a) Shear tab and (b) coped beam.

Substituting

$I_y = \frac{1}{12} lt^3$ and $J = \frac{1}{3} lt^3$, and taking the product of EG as 324,800 ksi^2, Equation 1 becomes:

$$M_{rec} = 1500\pi \frac{lt^3}{a} \tag{2}$$

where

l = depth of shear tab
t = thickness of shear tab, t_p, or thickness of beam web, t_w
a = length of shear tab from support to first line of bolts

Equation 2 controls the lateral-torsional buckling of the shear tab.

To ensure that the tab does not buckle, the required moment strength, $M_{req'd}$ (the load or demand) needs to be less than the LTB strength of the tab, M_{rec}.

$$M_{req'd} \leq M_{rec} \tag{3}$$

It is more convenient to deal with a reaction than a moment. Assuming a case of constant shear over length a and moment varying from zero at the support to maximum at the end of the cope or tab, $M_{req,d}$ (an approximation of the common case with uniform loading along the length of the beam), the required reaction strength, $R_{req'd}$, at the support is

$$R_{req'd} = \frac{M_{req'd}}{a} \tag{4}$$

and

$$R_{req'd} \leq \frac{M_{rec}}{a} = 1500\pi \frac{lt^3}{a^2} \tag{5}$$

In specification notation, stiffeners are not required when Equation 6, as follows, is satisfied.

$$R_{req'd} \leq \phi R_n \text{ (LRFD)}$$

$$R_{req'd} \leq \frac{R_n}{\Omega} \text{ (ASD)}$$

where

ϕ = 0.9
Ω = 1.67

$$R_n = 1500\pi \frac{lt^3}{a^2} \tag{6}$$

It may be convenient to check the need for stiffeners by evaluating the ratio of available shear to required shear as shown in Equation 7, as follows.

When $\eta \geq 1.0$, stiffeners are not required, calculate η as follows:

$$\eta = \frac{\phi R_n}{R_u} \text{ (LRFD)} \tag{7a}$$

$$\eta = \frac{R_n/\Omega}{R_a} \text{ (ASD)} \tag{7b}$$

The following are examples of implementation of the proposed limit state as given in Equation 6. In the first example, an artificial problem is presented, and the need for stiffening a shear tab is determined using the proposed procedure. The second example problem compares the strength predicted using the proposed procedure to the measured strength of a test specimen as reported by Cheng et al. (1984). In the third example problem, the measured strength of one of Cheng et al.'s coped beams is compared to the strength of the same beam converted from a coped beam to an extended tab of similar proportions, and then its strength predicted using the proposed theory given in Equation 6. Finally, the procedure proposed in this paper is compared to the results of a design example reported by Brockenbrough and Merritt (2006).

EXAMPLES

Example 1: A shear tab connection at the end of a W30×90 beam is used with the following properties (see Figure 2); L = 28 ft = 336 in., l = 24 in., t_p = 0.5 in., a = 9 in., and shear stud spacing is 12 in. o.c. over the length of the beam. Check both the ASD and LRFD methods. The Load and Resistance Factor Design (LRFD) required shear, R_u, is 115 kips, and assume that the ASD required shear, R_a, equals R_u/1.5.

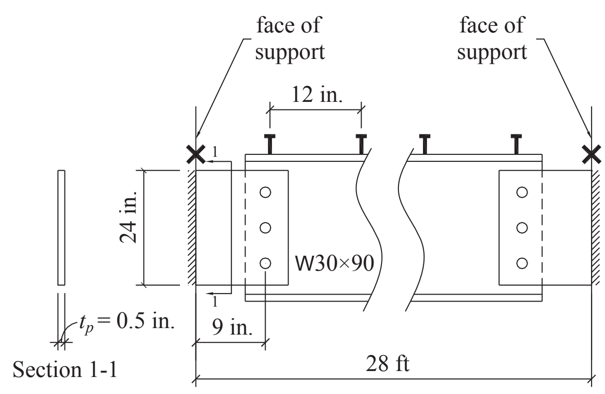

Fig. 2. Single-plate connection for Example 1.

Solution

L_b = 12 in. (stud spacing) < L_p = 7.38 ft
= 88.6 in. **o.k.**

$$R_n = \left(1500 \text{ kip/in.}^2\right)(\pi)\left(\frac{24 \text{ in.}}{(9 \text{ in.})^2}\right)(0.5 \text{ in.})^3$$
$$= 175 \text{ kips}$$

$$R_a = R_u/1.5 = 115 \text{ kips}/1.5 = 76.7 \text{ kips}$$

$$\phi R_n = 0.9(175 \text{ kips}) = 157 \text{ kips} > R_u = 115 \text{ kips (LRFD)} \quad \textbf{o.k.}$$

$$\frac{R_n}{\Omega} = \frac{175 \text{ kips}}{1.67} = 105 \text{ kips} > R_a = 76.7 \text{ kips (ASD)} \quad \textbf{o.k.}$$

Thus, for this example, the tab does not need the optional stiffeners.

From the test of Equation 7,

$$\eta = \frac{157}{115} = 1.4 \text{ (LRFD)}$$

$$\eta = \frac{105}{76.7} = 1.4 \text{ (ASD)}$$

Because $\eta = 1.4 \geq 1.0$, the shear tab does not affect the beam strength and the optional stiffeners noted in Figure 10-12 are not required. If the extended tab affects beam strength, $\eta < 1.0$, the term a can be reduced by adding the optional stiffeners, or a thicker tab plate can be used, and then again checking if η is ≥ 1.0.

For this example, η is greater than 1.0, so stiffeners are not required.

Example 2: This example is from Cheng et al. (1984), Illustrative Example 2, page 127. The beam is a W12×14 beam, the span L = 223.2 in., loaded at the center, and laterally supported at the ends and at the point of loading, which is approximately at the beam center. The cope length is a = 12 in., the depth is d_c = 1.19 in.

From the measured properties of the test beam (beam LTB3), l = 9.504 in., and the web thickness t_w = 0.212 in. The test beam had flange and web yield stresses, F_y, of 57.4 and 55.3 ksi, respectively. Considering that the flanges are dominant in flexure, use F_y = 57.4 ksi. Also, Cheng et al. used G = 11,600 ksi rather than the AISC-recommended nominal value of 11,200 ksi and reported a shear strength of R_n = 2.75 kips. See Figure 3 for details of the problem.

Solution

Applying the proposed Equation 6,

$$R_n = 1500\pi\left(\frac{(9.504)(0.212)^3}{12^2}\right)\sqrt{\frac{11{,}600}{11{,}200}} = 3.02 \text{ kips}$$

(cope or tab LTB strength)

Assuming the more accurate value is that obtained by Cheng et al., the proposed theory given by Equation 6 is unconservative by 9.82% as shown in the following calculation. However, continuous lateral support of the beam is assumed in the derivation of Equation 6 (i.e., $L_b \leq L_p$ = 31.92 in. for a W12×14 beam), whereas Cheng et al. provided lateral support only at midspan and the ends of the beam (L_b = 119.3 in.).

$$\frac{3.02 - 2.75}{2.75}(100) = 9.82\%$$

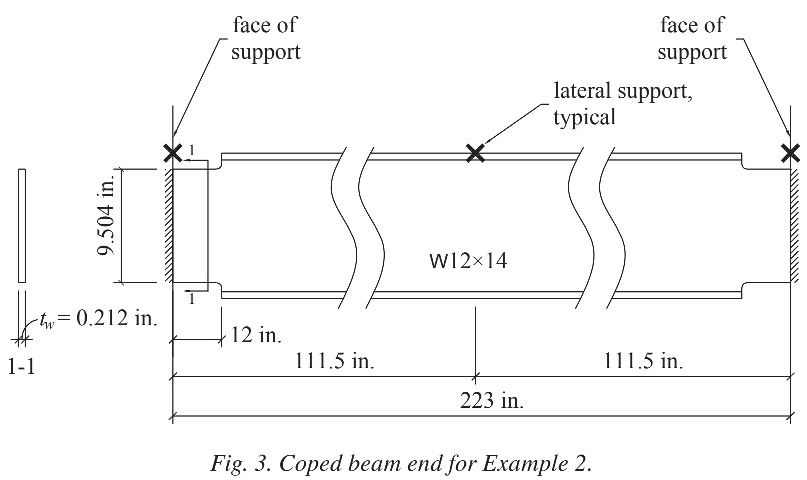

Fig. 3. Coped beam end for Example 2.

If Cheng et al.'s proposed equation (Equation 4.3 of their report) is used with $L_b = L_p = 31.92$ in., $R_n = 2.87$ kips and the difference is 4.97%.

$$\frac{3.02-2.87}{3.02}(100)=4.97\%$$

The two theories thus give approximately the same answer to the same problem. Note that neither theory is an exact solution. However, the theory proposed in this paper provides a much simpler approach in that the only assumption required is that the uncoped portion of the beam acts like a rigid body. This assumption is validated by the results presented by Cheng et al. Cheng et al. use an interaction equation that is also an approximation to the exact differential equation formulations.

Continuing with Example 2, suppose the required shear strength is $R_u = 14$ kips. The uncoped beam has sufficient strength to produce an end reaction of 14 kips when the beam is uniformly loaded. The η test gives $\eta = (0.9)(3.02)/14 = 0.194 < 1.0$. Because this is less than 1.0, the strength of the beam is reduced by the copes and, hence, for an extended tab connection of similar geometry. If the beam must carry a load greater than $\phi(2R_n) = 2(0.9)(3.02) = 5.44$ kips, the optional stiffeners are required, or perhaps a thicker shear tab will suffice. If the optional stiffeners are used, the *a* dimension will be reduced from the outer edge of the stiffeners to the first line of bolts. Note that the original *a* is still used in the design of the extended tab. Regardless, if the parameters remain unchanged, stiffeners would be required, or the available strength of the beam would have to be taken as 5.44 kips.

Example 3: Assume the coped beam presented in Example 2 of Cheng et al. is converted to a shear tab configuration with similar proportioning. The beam is W12×14 with a span of 223 in. = 18.6 ft. The length of the tab plate, from the face of the support to the first column of bolts, is taken as the same as the cope length of Example 2 (i.e., a = 12 in.). The depth of the tab is l = 9 in., the reaction is Ru = 14 kips (assuming maximum total uniform load), and Zx = 17.4 in.3 (from the AISC Manual). Using the procedure for extended shear tabs from the Manual (p. 10-103), a plate thickness of *tp* = 0.375 in. will suffice.

Solution

From Equation 6:

$$\phi R_n = (0.9)\left(\frac{(1500)(\pi)(9)(0.375)^3}{(12)^2}\right)$$
$$= 14.0 \text{ kips} \geq 14 \text{ kips} \qquad \textbf{o.k.}$$

From Equation 7:

$$\eta = \frac{14}{14} = 1.0$$

The connection is perfectly proportioned so as not to reduce the LTB strength of the beam, and therefore, stiffeners are not required. In the event η was less than 1.0, the designer would need to make a decision to provide stiffeners, or increase the plate thickness while considering the maximum thickness prescribed by the *Manual* procedure.

Example 4: Consider a beam with a shear tab connection discussed by Brockenbrough and Merritt (2006), Section 3.4.3. The beam is W16×45 with $R_u = 51$ kips (as given by Brockenbrough and Merritt), $L = 24$ ft, $a = 10.5$ in., $t_p = 0.625$ in., $l = 12$ in., and all material has a yield strength of $F_y = 50$ ksi. See Figure 4 for a similar connection.

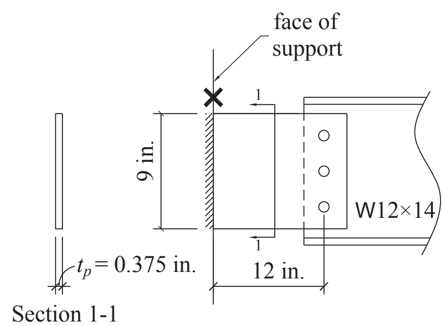

Fig. 4. Example 3; beam used in Example 2, but with a shear tab connection.

Solution

From Equation 6:

$$\phi R_n = (0.9)\frac{(1500)(\pi)(12)(0.625)^3}{(10.5)^2} = 113 \text{ kips}$$

Because ϕR_n = 113 kips > R_u = 51.0 kips, this design is satisfactory for lateral-torsional buckling.

Also, $\eta = \dfrac{113}{51} = 2.22 > 1.0$ **o.k.**

THE EFFECT OF LAP SPLICE ECCENTRICITY

Because of the lap joint between the shear tab and the beam web, there is a small eccentricity equal to $(t_p + t_w)/2$, which produces a torsional moment $M_t = R(t_p + t_w)/2$. This torsional moment also exists for the standard shear tab, but has been found to have no effect on connection strength in research (Astaneh et al., 1998; Creech, 2005) studies and in actual practice for at least 30 years. Extended tabs are a relatively new connection, with the design procedure first introduced in the AISC *Manual* (2005). Because of a lack of experience with this connection, the purpose of this discussion is to provide some guidance in its treatment.

Theory

The moment M_t defined earlier is resisted by two parts of the connection system: (1) the torsional strength of the tab itself and (2) the local torsional strength of the beam due to the floor slab or roof deck (see Figure 5). Also, recall that only laterally supported beams with $L_b \le L_p$ are considered in this paper.

The torsional resistance of the connection assembly is the sum of the torsional resistances of the tab plate and the beam. Let these be denoted by $M_{t,tab}$ and $M_{t,beam}$, respectively. Each of the two resistance components needs to be evaluated separately. As long as the total resistance exceeds the required torsional moment, M_t, the connection is satisfactory.

Torsional Resistance of the Tab

The tab shear stress is the sum of the torsional shear stress and the vertical shear stress due directly to the load R. Thus, using the plastic strength of the tab:

$$\frac{2}{lt_p^2}M_t + \frac{R}{lt_p} \le 0.6F_{yp} \tag{8}$$

Solving for M_t and setting M_t equal to the nominal strength of the tab, $M_{t,tab}$,

$$M_{t,tab} = \left[0.6F_{yp} - \frac{R}{lt_p}\right]\frac{lt_p^{\,2}}{2} \ge 0 \tag{9}$$

Note that when $M_{t,tab} < 0$, the tab can carry no torsion and $M_{t,tab}$ should be taken as zero. However, $M_{t,tab}$ must be greater than zero in order to proceed.

Torsion of the Beam Local to the Connection

In addition to the torsional resistance of the tab, the beam will also provide some torsional resistance due to the floor or roof slab. Because the beam tends to rotate, the slab on the "high" side will resist the rotation simply due to its weight and the imposed live load. Note that because no consideration here is given to studs, puddle welds or TEK screws, this is a conservative approach. Only the total dead and live loads local to the connection are used. It is assumed that the dead and live floor load imparts a contact load that is uniformly distributed across the flange of the beam, regardless of the direction of the span of the decking. Note that these dead plus live loads give rise to the beam reaction R.

The length of the beam, l_w, effective in resisting the moment, M_t, is controlled by the beam web thickness, t_w, as

$$M_t = R\left(\frac{t_w + t_p}{2}\right) = \frac{1}{4}\left(F_{yb}\right)t_w^{\,2}l_w \tag{10}$$

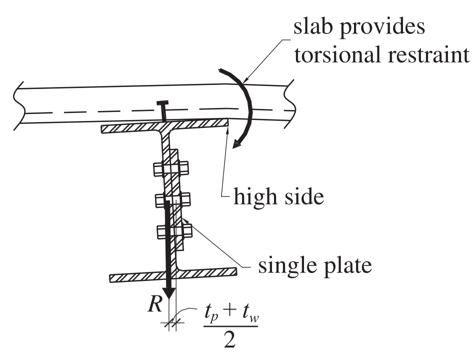

Fig. 5. Composite slab and metal roof or floor deck provides torsional resistance.

and solving for lw,

$$l_w = \frac{2R\left(t_w + t_p\right)}{\left(F_{yb}\right)t_w{}^2} \tag{11}$$

The maximum force, *F,* that can be exerted at the tip of the high-side beam flange (see Figure 5) is controlled by the beam loading. The beam loading per unit length is 2*R/L*, and for a length, l_w:

$$F = \frac{2Rl_w}{L} = \frac{4R^2\left(t_w + t_p\right)}{\left(F_{yb}\right)Lt_w{}^2} \tag{12}$$

The nominal strength of the beam is thus

$$M_{t,beam} = \frac{Fb_f}{2} = \frac{2R^2\left(t_w + t_p\right)b_f}{\left(F_{yb}\right)Lt_w^2} \tag{13}$$

A design is satisfactory based on beam strength if

$$M_t \le M_{t,beam} \tag{14}$$

As mentioned earlier, the total strength is the sum of the strength of the tab and the strength of the beam. So, a satisfactory design is one for which

$$M_t \le M_{t,tab} + M_{t,beam} \tag{15}$$

In *Specification* notation,

$$M_{t,u} \le \left[\phi_v\left(0.6F_{yp}\right) - \frac{R_u}{lt_p}\right]\frac{lt_p^2}{2} + \frac{2R_u^2\left(t_w + t_p\right)b_f}{\left(\phi_b F_{yb}\right)Lt_w{}^2}$$

(LRFD) (16a)

$$M_{t,a} \le \left[\frac{0.6F_{yp}}{\Omega_v} - \frac{R_a}{lt_p}\right]\frac{lt_p^2}{2} + \frac{\Omega_b 2R_a^2\left(t_w + t_p\right)b_f}{F_{yb}Lt_w{}^2}$$

(ASD) (16b)

where

$\phi_v = 1.0$
$\phi_b = 0.9$
$\Omega_v = 1.5$
$\Omega_b = 1.67$

ADDITIONAL EXAMPLES

Example 5: This is the same example as Example 1. The given data are R_u = 115 kips, L = 336 in., t_w = 0.470 in., t_p = 0.5 in., b_f =10.4 in. and l = 24 in., From Equation 16,

$$\phi M_t = \left[(1.0)(0.6)(50) - \frac{115}{(24)(0.5)}\right]\left(\frac{(24)(0.5)^2}{2}\right) + \frac{(2)(115)^2(0.470+0.5)(10.4)}{(0.9)(50)(336)(0.470)^2}$$

$$\phi M_t = 61.25 \text{ kips} + 79.88 \text{ kips} = 141 \text{ kip-in.}$$

$$M_{t,u} = (115)\left(\frac{0.470+0.500}{2}\right)$$

$$= 55.8 \text{ kip-in.} < 141 \text{ kip-in.} \quad \textbf{o.k.}$$

From Examples 1 and 5, the W30×90 beam with an extended tab connection is o.k. for lateral-torsional buckling and lap eccentricity.

Example 6: This is the problem shown in Example 3. From the data of Example 3, R = 14 kips, l = 9 in., a = 12 in., t_p = 0.375 in., t_w = 0.20 in., L = 233 in., and b_f = 3.97 in., assuming Grade 50 plate material.

Solution

From Equation 16:

$$\phi M_t = \left[(1.0)(0.6)(50) - \frac{14}{(9)(0.375)}\right]\left(\frac{(9)(0.375)^2}{2}\right) + \frac{(2)(14)^2(0.200+0.375)(3.97)}{(0.9)(50)(223)(0.2)^2}$$

$$\phi M_t = 16.35 + 2.23$$

$$= 18.6 \text{ kip-in.}$$

The required torsional moment $M_{t,u}$ is:

$$M_{t,u} = 14\left(\frac{0.200+0.375}{2}\right)$$

$$= 4.02 \text{ kip-in.} < 18.6 \text{ kip-in.} \quad \textbf{o.k.}$$

From Examples 3 and 6, the extended shear tab is satisfactory for lateral-torsional buckling and lap eccentricity.

Example 7: This is the extended shear tab of Example 4. Using the data of Example 4, and b_f = 7.04 in., Equation 16 gives the following:

Solution

$$\phi M_t = \left[(1.0)(0.6)(50) - \frac{51}{(12)(0.625)}\right]\left(\frac{(12)(0.625)^2}{2}\right) + \frac{(2)(51)^2(0.345+0.625)(7.04)}{(0.9)(50)(288)(0.345)^2}$$

$$\phi M_t = 54.4 + 23.0$$

$$= 77.4 \text{ kip-in.}$$

The required moment, $M_{t,u}$, is:

$$M_{t,u} = 51\left(\frac{0.345+0.625}{2}\right)$$

$$= 24.7 \text{ kip-in.} < 77.4 \text{ kip-in.} \qquad \textbf{o.k.}$$

From Examples 4 and 7, the extended tab is satisfactory for lateral-torsional stability and lap eccentricity.

CONCLUSIONS AND RECOMMENDATIONS

The lateral displacement of beams with coped ends or extended plate connections is primarily due to the torsional strength of the connection or connection region. It is assumed that the main beam section, or uncoped length, displaces laterally as a rigid body, and as discussed, this provides a reasonable prediction of the lateral-torsional buckling capacity of the connected beam. The authors recommend that the need for stiffeners be evaluated using Equations 6 and 7 (repeated here for convenience). When η is less than 1.0, the "optional" stiffeners as noted in Figure 10-12 of the AISC *Manual* (2005) should be provided, or in the case of a shear tab, perhaps a thicker shear plate will suffice. When η is greater than or equal to 1.0, stiffeners need not be used.

Recommended Check for Stiffening of Extended Single-Plate Connections or Coped Ends

The following checks will be included in the revised design procedure for extended single-plate connections in the 14th edition of the *Steel Construction Manual*:

$$R_{req'd} \le \phi R_n \text{ (LRFD)}$$

$$R_{req'd} \le \frac{R_n}{\Omega} \text{ (ASD)}$$

where

ϕ = 0.9
Ω = 1.67

$$R_n = 1500\pi \frac{lt^3}{a^2} \tag{6}$$

Stiffeners are not required when η ≥ 1.0, where:

$$\eta = \frac{\phi R_n}{R_u} \text{ (LRFD)} \tag{7a}$$

$$\eta = \frac{R_n/\Omega}{R_a} \text{ (ASD)} \tag{7b}$$

The torsional resistance of lap splice shear connections is the sum of two components; the lateral resistance of the tab and the lateral resistance of the beam in the connected region. The example problems presented in this paper suggest that the effect of the eccentricity inherent in the connection is negligible relative to the total torsional capacity of the connection. However, the following is a recommended check.

$$M_{t,u} \le \left[\phi_v\left(0.6F_{yp}\right) - \frac{R_u}{lt_p}\right]\frac{lt_p^2}{2} + \frac{2R_u^2\left(t_w + t_p\right)b_f}{\left(\phi_b F_{yb}\right)Lt_w^2} \quad \text{(LRFD)} \tag{16a}$$

$$M_{t,a} \le \left[\frac{0.6F_{yp}}{\Omega_v} - \frac{R_a}{lt_p}\right]\frac{lt_p^2}{2} + \frac{\Omega_b 2R_a^2\left(t_w + t_p\right)b_f}{F_{yb}Lt_w^2} \quad \text{(ASD)} \tag{16b}$$

where

ϕ_v = 1.0
ϕ_b = 0.9
Ω_v = 1.5
Ω_b = 1.67

SYMBOLS

E Young's modulus

F Force at beam flange tip resisting beam rotation

F_{yb} Yield stress of beam web

F_{yp} Yield stress of tab plate (or coped beam web)

G Shear modulus

J Torsional constant

L Length of beam

L_b Unbraced length of beam

L_p Limiting laterally unbraced length for the limit state of yielding

M_a Design required moment strength (ASD)

M_{rec} Elastic lateral-torsional buckling moment of a rectangular beam section

M_{rec} Critical lateral-torsional moment of the shear tab or double coped portion of beam

$M_{req'd}$ Required moment strength

M_t Torsional moment due to lap eccentricity

$M_{t,a}$ Design required torsional moment strength (ASD)

$M_{t,u}$ Design required torsional moment strength (LRFD)

M_u Design required moment strength (LRFD)

R_a Design required reaction strength (ASD)

R, R_{req} Required reaction strength

R_u Design required reaction strength (LRFD)

W Total load on beam

a Length of cope or length of shear tab to first column of bolts

b_f Width of beam flange

l Depth of tab plate or cope

l_w Effective length of beam to resist rotation

t See t_p or t_w

t_p Thickness of tab plate

t_w Thickness of beam web

w Uniform load on beams

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