APPENDIX A UNIFIED DESIGN PROCEDURE FOR THE LIMIT STATE OF COLUMN FLANGE BENDING

A.1 INTRODUCTION

The results, conclusions, and design recommendations of this study are based on the assumption that the column flange, to which the moment end-plate is bolted, behaves rigidly. A very flexible or weak column flange can alter the load distribution from the beam flange to the column flange significantly by generating large prying forces and ultimately causing a premature failure of the connection. However, this can easily be avoided by properly detailing the column flange for the limit state of column flange bending. Currently, very little literature dealing with column flange design procedures for moment end-plate connections is available, and what does exist does not consider the two connections (four-bolt wide and 16ES) that are the focus of this research. A design procedure for the eight-bolt extended stiffened and the four-bolt extended connections can be found in the *AISC Manual of Steel Construction* (1994). Hence, the purpose of this Appendix is to determine the effect of column flange bending on bolt forces as increasing moment is applied to a moment end-plate connection. This will be done two ways. First, a flexible column flange that is part of the finite element model of the entire assembly will be considered using varying column flange thicknesses. Secondly, a yield line solution will be developed for the limit state of column flange bending that is applicable regardless of the unstiffened column flange configuration (i.e., good for any moment end-plate configuration). This solution will then be examined using the finite element method.

A.2. FINITE ELEMENT ANALYSIS CONSIDERING FLEXIBLE COLUMN FLANGE

The results and conclusions of Chapter 5 deal with the design of the four-bolt wide moment end-plate connection shown in Fig. A-1. In Chapter 5, the finite element model used considers the column flange to behave rigidly. To determine the accuracy of these results, the column flange is now allowed to undergo both elastic and plastic deformations by removing the fixed boundary conditions at the back of the column flange, except at the location of the column web. A case study is performed for a

W18x130 beam-to-column connection with the end-plate geometry described in Table A1-1. In addition to the rigid column flange model, flexible column flange thicknesses of t_{fc} = 1.75 in., 1.25 in., 1.25 in. (with continuity plate), and 0.75 in. are considered. To model the continuity plate, rigid restraints are used at the continuity plate location on the back of the column flange.

Figure A-2(a) shows the end-plate separation and column flange bending at the maximum applied moment for the 1.25 in. thick column flange (i.e., t_{fc}=1.25 in.). Figure A-2(b) shows significant yielding at the same applied moment but prior to divergence due to a column flange bending mechanism. Comparing the tension zone (top of column flange) to the compression zone, lower stresses can be seen in the compression zone since full contact between the end-plate and column flange occurs in this area.

A very large increase in column flange bending strength is shown in Fig. A-3, where the stresses across the column flange are greatly reduced under the maximum applied moment. This is due to the presence of a continuity plate in the connection. The location of the continuity plate is represented by the two rows of elements of very low stress between the two rows of tension bolts in the figure. The low stress is due to the rigid boundary condition at the back of the column flange at the stiffener location.

Figures A-4 through A-7 plot bolt stress vs. applied moment for the bolts numbered in Fig. A-1 with no preload. From these figures, several conclusions can be drawn. Bolt #1 and bolt #3 are the primary load carrying bolts. Bolt #3 is the controlling bolt and it is the first bolt to fail when bolt rupture is the governing limit state. These same conclusions were drawn in Chapter 5. For a decreasing column flange thickness, the stresses in bolt #1 and bolt #3 increase. Conversely, stresses in bolt #2 and bolt #4 decrease with decreasing column flange thickness. For the 0.75 in. column flange thickness, column flange bending is the controlling limit state and the finite element method diverges at a lower load than for the thicker flanges. Although this is truly a limiting case, it is important to consider, since hardly any load is taken by bolt #4 for this case. In every other case, bolt #3 rupture is the controlling limit state. It is clear from these results that some minimum thickness should be established to ensure that significant column flange yielding does not take place under the design loading. Finally, the presence of a continuity plate causes bolt stresses that are best determined using a rigid flange. This is significant, because for seismic loading, very large flange forces must be designed for, and typical column sections will need a stiffener to satisfy, the limit state of column flange bending.

A.3. A UNIFIED DESIGN PROCEDURE FOR COLUMN FLANGE BENDING

Figures A-8 (a) and A-8 (b) show the controlling column flange bending yield line patterns for the four-bolt wide and 16ES moment end-plate connections, respectively. Utilizing symmetry, only half the column flange is shown. The half of the column web that is represented is seen in the leftmost side of each figure. The similarity between the patterns is clear. In fact, after careful examination and verification using the finite element method, it was determined that the pattern is accurate regardless of the end-plate configuration. Figure A-9 shows other yield line patterns that were included in the analysis, but provided column flange bending strengths that were not in accordance with results from the finite element model (i.e., they did not result in strengths representative of the lower bound). In this general yield line solution, “c” can be defined as the distance between the top row and the bottom row of bolts (e.g., $c = p_{fe}+t_f+p_{fi}$ for the configuration of Fig. A-1). The unknown “y” is obtained by minimizing the internal work. From this analysis, y is found to be independent of the bolt geometry and equal to $(b_f-t_{wc})/\sqrt{2}$, where b_f is the column flange width and t_{wc} is the column web thickness as indicated in Fig. A-8. In general, “g_i” can be defined as the distance from the column web centerline to the innermost column of bolts. Hence, the pattern is independent of the number of columns of bolts. Using yield line analysis, the internal work W_i for the entire flange is

$$W_i = 4M_p\left(\frac{b_f - t_{wc}}{y} + \frac{c+2y}{b_f - t_{wc}}\right) \qquad \text{(A1-1)}$$

where t_w is the column web thickness and M_p is the plate strength along a yield line per unit length (i.e., $M_p = F_y t_f^2/4$, where F_y is the nominal yield stress of the column flange).

The external work is dependent on both the number of bolts and the bolt layout. For the four-bolt wide connection and the 16ES connection, the external work W_e is

$$W_e = F_f\left(\frac{2g_i + g_b}{b_f - t_{wc}}\right) \qquad (A1-2)$$

where F_f is the design flange force. Note that each connection has its own design flange force. This equation for external work is valid for any connection configuration with two columns of bolts per half column flange. For a connection with one column of bolts per half column flange, the second term in the equation disappears and the solution simplifies to

$$W_e = 2F_f\left(\frac{g_i}{b_f - t_{wc}}\right) \qquad (A1-3)$$

Equating the internal and external work and solving for the flange force, the nominal column flange bending strength F_n is found to be

$$F_n = \frac{4M_p[(b_f - t_{wc})^2 + y(c+2y)]}{y(2g_i + g_b)} \qquad (A1-4)$$

for configurations with two columns of bolts per half at the tension flange, and

$$F_n = \frac{2M_p[(b_f - t_{wc})^2 + y(c+2y)]}{yg_i} \qquad (A1-5)$$

for configurations with one column of bolts at the tension flange. For design using LRFD, a resistance factor of ϕ=0.9 should be used.

A.4 VALIDATION OF DESIGN PROCEDURE

To show the accuracy of the yield line solution for the limit state of column flange bending, a finite element model of the column flange side of the connection was developed using solid 20-node elements that include plasticity effects. Symmetry is utilized and only half the column flange was modeled. The column web was included but restrained at the center of the column. Bolt forces were modeled using increasing applied forces at the bolt locations. Using this model and considering a W14x193 column flange, with the four-bolt wide connection geometry indicated in Table A1-2, the finite element model was analyzed until divergence occurred due to the formation of a mechanism in the column flange.

Figure A-10(a) shows the stress distribution in the column flange at about 80% of the maximum applied load, and significant yielding can be seen. Note that all of the yielding is near the column web intersection. Figure A-10(b) shows the stress distribution just prior to divergence. The yield line pattern can be clearly seen. Using the yield line solution of Eq. (A1-5), the column flange bending strength was determined to be 541 kips. The finite element model diverges at 560 kips, indicating around 3% error.

Next, the same column flange was analyzed using the 16ES geometry of Table A1-3, and similar results were obtained. Figure A-11(a) shows the stress distribution at 80% of the maximum applied load, and yielding near the column web intersection is visible. Figure A-11(b) shows the stress distribution just prior to divergence. The mechanism is again seen. The yield line column flange bending strength of 610 kips closely approximates the finite element divergence at 635 kips.

A.5. CONCLUSIONS

It has been shown that to ensure the accuracy of the design procedures and guidelines given in this dissertation, the limit state of column flange bending must be addressed. For seismic design, this means that the column flange bending strength must be greater than or equal to $1.1R_yM_p$ or the maximum moment that can be applied to the system. If the column flange bending strength is less than the design flange force, a stiffener must be provided. For a stiffness criterion, it has been shown that a good rule of thumb is to ensure that the column flange thickness is at least as thick as the end-plate.

TABLE A-1. Connection details for four-bolt wide specimen.

Dimension	Value
g_i	2.50 in.
g_o	1.08 in.
b_{ext}	1.00 in.
p_{ext}	5.00 in.
$p_{f,e}$	2.00 in.
$p_{f,i}$	2.00 in.
t_p (plate thickness)	1.25 in.
F_{yp} (plate yield stress)	36.0 ksi
F_{yb} (beam yield stress)	50.0 ksi
b_f (beam flange width)	11.16 in.
t_f (column falgne thickness)	1.20 in.
t_w (beam web thickness)	0.67 in.

TABLE A-2. Connection details for four-bolt wide column flange.

Dimension	Value
g_i	2.50 in.
p_b	4.00 in.
c	5.00 in.
F_y (column yield stress)	50.0 ksi

TABLE A-3. Connection details for 16ES column flange.

Dimension	Value
g_i	2.50 in.
p_b	4.00 in.
c	11.00 in.
F_y (column yield stress)	50.0 ksi