CHAPTER 5 A PARAMETRIC STUDY OF THE FOUR-BOLT WIDE MOMENT END-PLATE CONNECTION

5.1 INTRODUCTION

This chapter presents the results of a parametric study of the four-bolt wide moment end-plate connection. The behavior of this connection is quite complex and the interaction between end-plate bending, prying forces, and bolt behavior is difficult to define. Currently, a design procedure is not available for the four-bolt wide moment end-plate connection. Meng (1995) shows that the connection geometry greatly influences the load paths to the bolts and for some configurations, half of the bolts are ineffective in carrying any load at all. In this chapter, the finite element method is used to determine how bolt pitch, bolt gage, and end-plate thickness affect the design and moment-rotation characteristics of the connection. The effectiveness of this connection for seismic design is then discussed.

5.2 PROCEDURE

Figure 5.1(a) shows the end-plate geometry for the four-bolt wide moment end-plate connection. The “standard case” specimen with the connection geometry detailed in Table 5.1 is a W30x173 beam with a yield strength of 50 ksi. The end-plate material has a yield strength of 36 ksi. Both of the material properties are modeled using a tri-linear stress-strain curve described in the previous chapter. The ANSYS finite element package is used to model the beam-to-column connection. Since the problem is three-dimensional in nature, solid eight-node brick elements that include plasticity effects are used extensively to model the beam. Twenty-node solid elements are used to model both the end-plate and the bolts. Contact elements are included between the end-plate and the column flange to represent the nonlinear behavior. Bolt pretensioning effects are included and prying forces are tracked throughout the loading process. The models contain 31,235 degrees of freedom.

The beam is loaded at the beam tip to generate an applied moment at the connection. Bolt gages, bolt pitches, bolt sizes, and the plate thickness are varied one at a time to determine what effect each of these variables has on the connection strength and moment-rotation characteristics.

5.3 END-PLATE BENDING

Although the purpose of this chapter is not to provide a full-fledged design procedure for the four-bolt wide moment end-plate connection, a design procedure for the limit state of end-plate bending is warranted since many of the end-plates considered are subject to this limit state. Besides, it would be difficult to provide an understanding of the effects of end-plate geometry on the response of the connection without having some idea of when the end-plate behavior becomes highly nonlinear.

Yield line analysis has proven to be a useful tool in determining the ultimate load carrying capacity of end-plates. The procedure is similar to the plastic analysis of beams where elastic deformations are considered negligible when compared to plastic deformations caused by the formation of plastic hinges. Yield lines are selected in any kinematically valid pattern to divide the end-plate into a series of rigid sections which form a mechanism. The lines are straight, and it is assumed that the moment along each line is constant and equal to the plastic moment capacity of the plate. The beam web centerline is given a virtual rotation about the bottom of the beam and virtual work is then used to find the controlling mechanism. Since the connections are to be designed such that bolt yielding is not the controlling limit state, it is assumed that all of the internal work takes place in the end-plate itself. Therefore, the plate rotates with the beam web but has no deflection at the bolt hole locations. The plate deformations are consistent with the selected yield line mechanism. For a more complete description of this method, see, for example, Szilard (1974). Borgsmiller (1995) summarizes the controlling yield line mechanisms for the most commonly used end-plate configurations.

Lastly, yield line analysis predicts what is usually called “first yield moment” or simply “first yield”. This moment is not the failure load of the connection. In fact, it can be well below the actual moment resulting in structural failure of the connection. However, in design, it is typically used as the design strength of the connection for the plate bending limit state. Theoretically, it is the moment capacity that would result in structural failure in the absence of any additional resistance. In reality, membrane stresses in the plate resist the additional applied moment at the connection and stretching of the plate takes place. Practically speaking, the first yield moment predicts the point at which plastic rotation due to inelastic plate behavior becomes significant.

The yield line pattern shown in Fig. 5-1(b) is the controlling yield line pattern for the four-bolt wide connection. It is identical to the four-bolt extended unstiffened pattern given in Borgsmiller (1995). Using virtual work, neglecting the thickness of the beam web and the internal work in the proximity of the bottom flange, and letting $p_{f,e}$ equal $p_{f,i}$, the nominal end-plate bending strength M_n is found to be

$$M_n = 4M_p\left\{\left[\left(\frac{b_f}{2}+b_{ext}\right)\left(\frac{1}{p_f}+\frac{1}{s}\right)+\left(\frac{p_f+s}{g_i}\right)\right](d-t_f-p_f)+\left(\frac{b_f}{2}+b_{ext}\right)\left(\frac{1}{2}+\frac{d}{p_f}\right)\right\} \quad (5-1)$$

where $M_p = F_{yp}t_p^2/4$ is the plastic moment capacity of the yield line per unit length, and the unknown length s is obtained by minimizing the internal work which results in

$$s = \frac{1}{2}\sqrt{2g_i(2b_{ext}+b_f)} \quad (5-2)$$

5.4 RESULTS OF THE PARAMETRIC STUDY

Figure 5-2 shows bolt stresses vs. applied moment at the connection for the “standard case.” Throughout the loading, the bolt stress curves for all four bolts stay relatively close to each other, indicating that each bolt takes a significant amount of load from the beam flange. This is expected since the outside bolts are well inside the beam flange (i.e., g_o = 1.25 in.) for the standard case. Bolt #3 is the controlling bolt and will be in all practical design cases when following the design recommendations in the conclusions section of this chapter. Bolt #2 lags behind bolt #1 until inelastic bolt behavior is reached at 90 ksi. At this point, bolt 2 catches up via stress redistribution. Bolt #4 is the least effective bolt in resisting the flange force. However, in this case it does take a significant amount of load.

Figures 5-3 through 5-5 plot the bolt stress vs. applied moment for the standard case with one modification. The distance g_0, from the centerline of the outside column of bolts to the beam flange tip, is varied. In Fig. 5-3, the bolt centerline is 0.50 in. outside the beam tip. Note how the outside bolts, bolt #2 and bolt #4, take much less load than the critical bolts, bolt #1 and bolt #3. As a result, this is a very poorly designed specimen, because some of the bolts do not resist a significant portion of the flange force. In Fig. 5-4, the bolt centerline is just at the beam tip. Again, the outside bolts are quite ineffective. Finally, Fig. 5-5 plots the bolt stresses for the case when the bolt centerline is 0.50 in. inside the beam tip. As expected, a significant increase in resistance is provided by the outside bolts.

In Figures 5-6 and 5-7, p_f, the bolt pitch to beam flange distance (p_{fi} and p_{fe} are equal in this study and are simply called p_f), is increased from 1.75 in. for the standard case to 2.25 in. and 2.75 in., respectively. Note that in these figures, the horizontal-axis scale has changed to account for the significant decrease in connection strength. In Fig. 5-6, it is clear that bolt #3 takes a much larger portion of the flange force. This is expected, because increasing p_f moves all of the bolts away from a source of very high stiffness (i.e., beam web or beam flange) except bolt #3, which remains close to the beam web. In Fig. 5-7, the problem is even more magnified as bolt #3 takes a much greater portion of the load. Also, as bolt #3 reaches inelastic behavior at 90 ksi, it is apparent that little or no stress redistribution can occur. This is shown by the flattening of the bolt plots for bolts #1, #2, and #4 as bolt #3 reaches 90 ksi. Although it may seem logical to attempt to increase the amount of inelastic connection behavior by increasing the bolt pitch, it is clear by the strength reduction due to an unfavorable load path that this is not a good design idea.

Figures 5-8 and 5-9 plot bolt stresses for increased end-plate thicknesses of 0.875 in. and 1.00 in., respectively. Comparing these plots to Figure 5-2, it is clear that the thickness influences the bolt behavior. Bolt #3, the critical bolt, reaches its ultimate stress of 100 ksi at a smaller applied moment for the 0.75 in. end-plate than for the thicker ones. Also note that at the maximum applied moment, all the bolt stresses are larger for the thinner end-plate. This is expected due to the presence of larger prying forces that increase with increasing inelastic plate behavior.

Figures 5-10 and 5-11 plot bolt stress vs. applied moment for decreased bolt sizes of 0.75 in. and 1.00 in., respectively. Note that the horizontal-axis range is different in these figures. A decrease in ultimate strength via bolt rupture is apparent.

Finally, since end-plate separation is directly proportional to the rotation of the connection, applied moment vs. end-plate separation is shown in Fig. 5-12 for varying end-plate thicknesses. The accompanying yield line solution is shown with matching symbols. In the linear elastic range, all three connections respond identically. After significant inelastic behavior takes place, the end-plates tend to pull away from the column flange.

The inelastic rotation of the connection is directly proportional to the difference between the elastic deflection and the beam tip deflection for any applied moment. For seismic design, the ultimate inelastic rotation capacity is important. In Fig. 5-12, it is directly proportional to the horizontal distance between the elastic deflection and the finite element beam tip deflection at the maximum applied moment. From this figure, it is apparent that the 0.75 in. end-plate can provide about four times the inelastic rotation capacity of a 1.00 in. end-plate with the same connection geometry for the same applied moment. Also note the reserve strength in the end-plate after the yield line solutions are reached.

5.5 CONCLUSIONS AND SEISMIC DESIGN RECOMMENDATIONS

It is clear that each of the individual geometric parameters considered in this section can greatly influence connection behavior as well as overall performance of the four-bolt wide moment end plate connection. The distance g_o is the primary factor determining the effectiveness of the outside bolts. It has been shown that the farther inside the beam tip the outside bolts are, the more effective they are at resisting the flange force. As part of this study, numerous configurations have been analyzed and it is apparent that for the connection to be even somewhat effective, g_o should not be taken as less than 0.25 in. inside the beam tip. A value of g_o greater than 0.50 in. is recommended, but difficult to obtain given the flange width of most practical beam sizes. Not only is this recommendation important for developing some effectiveness of the outside bolts, it is also shown in Chapter 4 that high stress concentrations occur at the beam flange to beam web intersection when this condition is not satisfied. This is due to an unfavorable load path generated as the flange force attempts to reach the interior bolts only, and can result in a brittle failure via a tearing/shearing mechanism through the plate.

An increased bolt pitch to flange distance, p_f, can increase a connection's inelastic rotation capability but greatly diminishes the connection's overall strength. Bolt #3 takes too much load and an unfavorable load path is generated. Hence it is recommended that p_f be selected based on minimum distances required for bolt clearance and tightening.

Decreasing the thickness of the end-plate is the only viable option when attempting to provide an increased inelastic rotation capacity of the connection. Equation (5-1) can be used to determine an adequate thickness, but it should be remembered that the connection must be able to elastically resist all load combinations that do not include seismic effects. Hence, the end-plate bending strength should be greater than the factored moment at the connection.

As currently required for FR construction by the *AISC Seismic Provisions for Structural Steel Buildings* (1997), this connection, designed with the connection weaker than the adjoining beam, can only be used as the exception case for ordinary moment frames. It must be shown by experimental testing that the designed connection can provide 0.01 radians of inelastic rotation. Alternatively, PR detailing can be used in combination with special, intermediate, or ordinary moment frames. For a select number of beams, Eq. (5-1) can be used in combination with large bolts to provide connections that are stronger than the adjoining beam. The bolt size can be selected following Meng (1996) for example. In this case, no experimental testing is required and the connection can be used as part of an ordinary moment frame only.

TABLE 5-1. Connection details for 30x173 four-bolt wide “standard specimen”.

Dimension	Value
$b_f/2$	7.49 in.
g_i	2.5 in.
g_o	1.25 in. (inside)
b_{ext}	2.25 in.
p_{ext}	6 in.
$p_{f,e}$	1.75 in.
$p_{f,i}$	1.75 in.
t_p (plate thickness)	0.75 in.
F_{yp} (plate yield stress)	36.0 ksi
F_{yb} (beam yield stress)	50.0 ksi
d_b (bolt diameter)	1.25 in. (A325)
M_b (beam yield moment)	2522 k-ft