CHAPTER 4 CORRELATION OF FINITE ELEMENT RESULTS WITH EXPERIMENTAL VALUES OBTAINED FROM THE SAC STEEL PROJECT SPECIMENS

4.1 INTRODUCTION

The previous chapter of this dissertation examined the effectiveness of the finite element method in predicting the behavior of statically loaded specimens, where emphasis was placed on correlating bolt forces and end-plate bending strength with two experimentally tested end-plate configurations. This chapter presents some of the findings of an ongoing study to correlate test results of large moment end-plate connections under cyclic loading with those of finite element models under identical loading. The need for such a correlation is readily apparent. Current design practice requires that steel connections which are part of a seismic lateral force resisting system be tested experimentally. Although limited interpolation is acceptable, proper use of the finite element method provides a basis for justifying a particular beam-to-column connection configuration based on a rational interpolation process. The accuracy of the finite element method in predicting the behavior of moment end-plate connections under static loading has been well established in the literature. However, only smaller connections such as two-bolt flush (two bolts at the tension flange) and four-bolt extended (four bolts at the tension flange) configurations have been considered.

In this chapter, a four-bolt extended unstiffened, a four-bolt wide, and an eight-bolt extended stiffened moment end-plate configuration are considered. The connections are made using thin end-plates to ensure that the inelastic behavior is primarily from plate bending and bolt yielding. Although beam hinging caused by designing strong connections using thick end-plates and large bolts is an effective energy dissipation mechanism, this form of inelastic behavior is arguably connection independent and will be considered in Chapter 6. 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 components. The bolts and end-plate are modeled using 20 node solid elements. Contact elements are included between the end-plate and the column flange to represent the nonlinear behavior of this complex interaction problem. Large, 1 ¼ in. diameter A325 bolts are used in the tests, and the finite element analysis effectively models the pretensioning effects on bolt stresses and contact surfaces. Prying forces and plastic behavior in the bolts are tracked throughout the loading process. The inelastic moment-rotation characteristics of each connection are considered and it is shown that the finite element method can be used to accurately predict the envelope of the behavior of such connections under cyclic loading.

4.2 PROCEDURE

For the purposes discussed above, three moment end-plate configurations from the SAC Steel Project, shown in Fig. 4-1, are considered. They include: (a) a four-bolt extended connection (W24x68-A572Gr50 beam); (b) a four-bolt wide connection (W30x99-A572Gr50 beam); and (c) an eight-bolt extended stiffened connection (W30x99-A572Gr50 beam). The details of each specimen are obtained from Sumner et al. (2000) and are discussed in section 4.3. Although not included here, other SAC connections are modeled and results are presented in Sumner et al. (2000).

To obtain the highest precision finite element results in the most efficient manner, symmetry about the beam web centerline is assumed. In real engineering practice, this is sometimes not the case, since actual dimensions can sometimes radically vary, particularly when using built-up sections (Boorse, 1999). Nonetheless, the differences in the geometric layout of the end-plate across the beam web centerline are usually quite small. Here, average values are used to obtain better correlation. For example, for the bottom flange of the eight-bolt stiffened configuration, the actual b_{ext} dimension shown in Fig. 4-1(a) is 3/16” on the left side and 13/16” on the right side. The average, or 1/2”, was used in the finite element model.

In contrast to Chapter 3, tri-linear stress-strain curves are used to model all material properties. The curves shown in Fig. 4-2 illustrate the models used to represent the nonlinear stress-strain properties of the material making up the moment end-plate connection. These models are slight modifications of the material property representations of Sherbourne and Bahaari (1997) and account for high strength bolt testing results from Kulak et al. (1987) and Abel (1993). The specimens are loaded at the beam tip until the model becomes unstable due to excessive inelastic behavior of the structure. Static loading is used and a comparison is made between the envelope obtained from the experimental hysteretic behavior of the specimen and the finite element results.

4.3 SPECIMEN DETAILS

Tables 4-1 through 4-3 provide the details of the specimens considered here and shown in Figure 4-1. The values listed are those used in the finite element model. Since symmetry is utilized in the analytical study, averages (from experimental values) are sometimes required since small fabrication errors exist in these specimens. The nominal dimensions for the beam sections were used. Also, t_p is the thickness of the end-plate and F_{yp}, F_{yb}, and F_{ys} represent the measured yield stress of the end-plate, beam, and stiffener, respectively. Note that the tension flange (top flange) bolts are numbered in Fig. 4-1. Individual bolts for each connection will be referred to by these numbers in future sections of this chapter.

4.4 FINITE ELEMENT ANALYSIS OF BEAM-TO-COLUMN CONNECTIONS

As in Chapter 3, the specimens are loaded at the beam tip until some maximum moment occurs at the connection. The maximum moment is calculated at the onset of finite element divergence due to numerical instability. For these specimens in particular, instability is caused by significant yielding of one or more bolts. Part of the finite element model for the four-bolt extended specimen is shown in Figure 4-3(a). For an applied moment of 840 k-ft (103% of the beam yield moment), the Von Mises stress distribution at the tension flange region can be seen. The beam yield stress is 55 ksi, and Figure 4-3(a) indicates that there is no yielding of the tension flange. Although the moment at the connection is 3% larger than the beam yield moment, a hinge does not form, because the hinge location is a distance (about one foot) away from the connection and the moment is less at this point. However, Figure 4-3(b) shows stresses in excess of the plate yield stress of 37.9 ksi.

At an applied moment of 1100 k-ft (76% of the beam yield moment), Figure 4-4(a) shows the stress distribution in the tension flange region of the finite element model for the four-bolt wide connection. Note that bolt #2 and bolt #4 are indeed outside the flange in this figure (in Fig. 4-1(b), they are shown inside the beam flange to indicate a positive g_0 distance from the outside bolt centerline to the beam flange tip). As a result, these bolts do not carry loads much larger than the pretension load. This puts excess stress on the interior bolts, #1 and #3, and causes an alarming load path. The tensile stresses in the beam flange must reach the supporting member via the interior bolts. This results in a large stress concentration from the beam flange-to-end-plate intersection to bolt #1. This is shown more clearly in Figure 4-4(b). Also note the excessive stresses below bolt #3 that are caused by significant end-plate bending. To better describe the load path of this configuration, Fig. 4-5 shows the horizontal displacements of the finite elements under the same applied moment. Figure 4-5(a) shows the view looking at the beam web centerline. The maximum flange displacement occurs here. Conversely, Fig. 4-5(b) shows the same plot looking in the opposite direction. Note that the exterior part of the flange (the flange tips) have virtually no displacement. Hence it is clear that very little of the flange force is carried here.

At an applied moment of 1600 k-ft (112% of the beam yield moment), Figure 4-6(a) shows the stress distribution in the tension flange region of the finite element model for the eight-bolt extended stiffened connection. The stiffener previously indicated in Figure 4-2(c) is shown more clearly here. The presence of a stiffener changes the load path considerably. As seen in Fig. 4-6(a), yielding of the beam flange does occur, but only near the end of the stiffener. This is important for seismic design, since the location of inelastic behavior should be known. Also, part of the stiffener has completely yielded and provides no additional stiffness to the connection. This greatly alters the force taken by bolt #1, which will be discussed in the next section. The part of the stiffener that hasn't yielded takes very little load. Thus, it is typical practice to remove the corner of the stiffener during fabrication. This was done for the actual SAC specimen, but not here. Figure 4-6(b) shows the stress distribution across the end-plate for the same loading condition.

4.5 COMPARISON OF FINITE ELEMENT TO EXPERIMENTAL RESULTS

Figures 4-3 to 4-6 show that the finite element method can be used to better understand the complex behavior of different moment end-plate connection configurations. In this section, bolt forces, plate separation, and inelastic rotation capabilities of these connections are considered.

Figures 4-7 and 4-9 show the bolt forces vs. applied moment for the four-bolt extended connection. Comparing the finite element results with the experimental results, good correlation is obtained for bolt #1 and bolt #2. Bolt #2 yields prior to bolt #1. This is expected, as both are equally distant from the beam flange, yet bolt #2 is close to the beam web, which provides considerable stiffness. Figure 4-10 plots the applied moment at the connection vs. plate separation. Excellent correlation is obtained here also. This is very important for seismic design, since the amount of inelastic rotation provided by the end-plate is usually directly determined from the measured plate separation value. Finally, applied moment vs. total inelastic rotation is shown in Fig. 4-10. Good correlation is obtained throughout the loading. Since the finite element model does not consider the column side of the connection, most of the plastic behavior of this connection must come from the end-plate, bolts, and/or beam elements. If the panel zone of an experimentally tested specimen were to undertake considerable plastic deformations, the finite element model would not predict this source of inelastic rotation.

Figures 4-11 through 4-14 plot bolt force vs. applied moment for the four-bolt wide connection. Good correlation is seen for all the bolts. However, the experimental values for bolt #4 are taken from the bottom side of the experimentally tested connection due to a faulty gage in the proper bolt during the loading process. This only slightly alters the experimental results, since the connection details at the bottom of the connection are quite close to those for the top. As expected, bolt #2 and bolt #4 take very little load above the pretensioning value. Bolt #3 is the first bolt to yield. This is expected, as once again it is relatively close to the stiff beam web. Figure 4-15 plots the applied moment at the connection vs. plate separation. Initially, there is excellent correlation. However, near the maximum applied moment, experimental plate separation values exceed those from the finite element analysis. The finite element analysis diverges due to bolt rupture prior to reaching the ultimate end plate separation value found experimentally. The experimental testing of this connection culminated in a limit state of plate tearing/shearing in the proximity of the beam flange. The mechanism began in the plate near the beam web-to-beam flange intersection and extended outward until failure. Although the finite element method did not predict this type of failure, stresses near the peak of the strain hardening curve of the material did exist in the end plate near the beam web-to-beam flange intersection. Further studies (see Chapter 5) have resulted in numerical instabilities in this area prior to bolt rupture, indicating that the finite element method can predict the onset of the limit state.

Figures 4-16 through 4-19 plot bolt force vs. applied moment for the eight-bolt extended stiffened connection. Although the correlation between the finite element and experimental results are excellent initially, the results begin to vary at about 65 percent of the ultimate moment. This is because, the yield stress of the stiffener was not found by testing and was assumed to be only slightly higher than that of the end-plate. Logically, the specimen should have been designed with a stiffener that yields at a stress close to that of the beam yield stress. Since this is not the case (the stiffener is made of A36 steel and the beam is A572Gr50), the load path changes significantly once the stiffener yields. Apparently the finite element stiffener yields prior to the experimental stiffener and the bolt forces change significantly at this point. Figure 4-16 shows that at the ultimate moment, the experimental bolt #1 takes a higher load than does the finite element bolt. Since the stiffener carries the flange force to bolt #1, this result would be expected for a stiffener that has a higher yield stress than used in the finite element model. Conversely, the experimental bolt #4 should take less load than the finite element bolt as a result of load redistribution. This is shown in Fig. 4-19. The load path is altered and the stresses in bolt #2 and bolt #3 vary as shown in Figs. 4-17 and 4-18, respectively.

To test this hypothesis, the finite element method was utilized using different yield stresses for the stiffener and applying the maximum moment. It was found by trial and error that for a stiffener yield stress of 43 to 44 ksi, the maximum finite element bolt forces very closely matched those for the experimental specimen.

Figure 4-20 plots the applied moment at the connection vs. plate separation. Good correlation is shown up to the ultimate moment. Finally, Fig. 4-21 plots applied moment vs. inelastic rotation for the connection. Initially, there is good correlation. However, the experimental ultimate inelastic rotation is much larger than predicted by the finite element model. This means that another large source of inelastic rotation must exist for this connection. Local flange buckling of the beam occurred prior to the ultimate moment and provided a significant amount of inelastic rotation. Of course, this source of inelastic behavior is not predictable by the present model, since imperfections that exist in the actual specimen and the experimental set-up are not included.

TABLE 4-1. Connection details for four-bolt extended specimen.

Dimension	Value
g_i	3.00 in.
b_{ext}	0.50 in.
p_{ext}	5.01 in.
$p_{f,e}$	2.00 in.
$p_{f,i}$	2.00 in.
t_p (plate thickness)	1.15 in.
F_{yp} (plate yield stress)	37.9 ksi (measured)
F_{yb} (beam yield stress)	55 ksi (measured average)
M_b (beam yield moment)	812 k-ft

TABLE 4-2. Connection details for four-bolt wide specimen.

Dimension	Value
g_i	2.5 in.
g_o	-0.65 in. (outside)
b_{ext}	2.25 in.
p_{ext}	3.69 in.
$p_{f,e}$	1.75 in.
$p_{f,i}$	1.75 in.
t_p (plate thickness)	1.13 in.
F_{yp} (plate yield stress)	42.8 ksi (measured)
F_{yb} (beam yield stress)	56.0 ksi (measured average)
M_b (beam yield moment)	1456 k-ft

TABLE 4-3. Connection details for eight-bolt extended stiffened specimen.

Dimension	Value
g_i	2.76 in.
b_{ext}	0.33 in.
p_{ext}	7.33 in.
$p_{f,e}$	1.78 in.
$p_{f,i}$	1.84 in.
$p_{b,e}$	3.77 in.
$p_{b,i}$	3.77 in.
t_p (plate thickness)	1.01 in.
F_{ys} (plate yield stress)	38.0 ksi (measured)
F_{ys} (stiffener yield stress)	40.0 ksi (assumed)
F_{yb} (beam yield stress)	55.0 ksi (measured average)
M_b (beam yield moment)	1430 k-ft