CHAPTER 2 LITERATURE REVIEW

The published research by other authors on topics that are closely related to moment end-plate connection design, analysis, modeling techniques, and behavior prediction is presented in this chapter. Most of the sources deal with finite element analysis. However, it seems warranted to include some papers that present the results of analytical studies and PR connection analysis techniques, given the practical applications of these topics to connection moment-rotation characteristics in general.

2.1 DESIGN PROCEDURES

Design procedures for certain configurations of end-plates under static loading are readily available. In the United States, extended end-plate design procedures (which includes the largest end-plate configurations commonly used) are presented in the AISC Manual of Steel Construction (1994). Connection types include the four-bolt extended and the eight-bolt extended stiffened configurations (see Fig. 1-2). The four-bolt wide connection is discussed, but a design procedure for this connection is not included.

The four-bolt extended connection design procedure is based on Krishnamurthy (1978), Hendrick and Murray (1984), and Curtis and Murray (1989). Prying forces are neglected in this procedure and an end-plate thickness is determined using the tee-stub analogy. The eight-bolt extended stiffened design procedure is based on Murray and Kukreti (1988), Hendrick and Murray (1984), and Curtis and Murray (1989). Prying forces are included in this method, which is based on the tee-stub analogy and a regression analysis of finite element results.

Grundy et al. (1980) discusses the general behavior of moment end-plate connections. Included are perpendicular beam-to-column connections and those at angles other than 90 degrees. A practical design procedure is presented, but the theory behind it is limited. Griffiths (1984) gives background information on the original development of moment end-plate design procedures and provides practical insights into the use and misuse of these connections.

In considering the limit states of end-plate yielding and bolt rupture, numerous design procedures utilize yield line theory to determine the strength of the end-plate and a modified version of the “split-tee analogy” considered by Kennedy et al. (1981) to predict bolt forces which include prying action. A unification of these procedures has been ongoing for around 15 years, with initial results summarized in Murray (1988). Murray traces the development of design procedures for moment end-plate connections that are based on experimental results, analytical studies, and the finite element method. Also, Murray presents a summary of moment end-plate research at the University of Oklahoma that includes design procedures for several end-plate configurations.

Some of the earliest research in this unification effort was performed by Srouji et al. (1983). Here the authors use yield line theory to predict end-plate yielding and Kennedy's split-tee model (slightly modified) to predict bolt forces. The only modification of Kennedy's model is for the four-bolt (four bolts at the tension flange) flush end-plate configuration. The authors determined that the row of two bolts furthest from the flange only carried 1/6 of the total flange force. Hendrick et al. (1984) continued to modify Kennedy's model by determining an empirical equation to more precisely determine the location of prying forces, and changing the fraction of the flange force taken by the bottom row of bolts to 1/8. Hendrick et al. (1985) completed the unification procedure for four different flush end-plate configurations. The same procedure is considered for stiffened extended end-plates by Morrison et al. (1985). Other configurations are considered by Rodkey and Murray (1993) and Borgsmiller et al. (1995). Kline et al. (1989) considers the effect of wind loading on the behavior of end-plate connections made with snug tight bolts by experimentally subjecting them to numerous cycles of loading. Meng (1996) also uses cyclic loading, but to evaluate the effectiveness of these connections under seismic loading. He includes a simplified method for the design of four-bolt wide moment end-plate connections. Sumner et al. (2000) presents experimental results of large moment end-plate connections. These results will be used to develop design procedures for seismic loading. To date, the most complete and updated version of the unification procedure is contained in Borgsmiller (1995). A more recent discussion of the unification effort is presented in Mays and Murray (1999).

Curtis and Murray (1989) presents a design procedure for the column side of a moment end-plate connection for the limit state of flange flexural yielding. Other column side limit state design procedures are presented in Faella et al. (2000), Zoetemeijer

(1974), Packer and Morris (1977), and Mann and Morris (1979). All of these procedures use some form of yield line solution.

2.2 FINITE ELEMENT ANALYSIS

The literature is filled with papers that use the finite element method to predict the behavior of different types of steel connections under static loading. In fact, there are several that consider only moment end-plate connections. However, the limitations of these works are readily apparent and can be listed as general limitations present in most current research on this topic. First, the end-plate behavior, and not bolt forces, is the prime concern. The end-plate strength for most end-plate configurations has been well defined in the literature. However, most bolt force prediction schemes have been shown to be impractical for design applications. Almost all the papers use truss elements to represent the entire bolt and the results are extremely limited. Second, all of the papers on this topic only consider small end-plate configurations (i.e., flush or four-bolt extended). The main reason for this is that these smaller connections provide more flexibility than larger ones. This is needed for efficient PR connection design applications. Finally, the theme of most papers is the adequacy of the finite element method in determining the connection's behavior. Very few applications are made. To date, this author has yet to find any papers on dynamic finite element analysis of moment end-plate connections.

2.2.1 *KRISHNAMURTHY AND GRADDY* (1976)

Krishnamurthy and Graddy (1976) is one of the earliest papers on the appropriateness of the finite element method for studying moment end-plate connections. The paper has many features that can be criticized, but given the computer resources of the time, the limitations in the analysis are expected. The authors attempt to correlate the results from an elastic, three-dimensional, finite element analysis to those from an elastic, two-dimensional, finite element analysis. The main reason for this correlation is that at the time, computer resources that were needed to consider the problem three-dimensionally were not economically feasible.

Thirteen commonly used four-bolt unstiffened moment end-plate connections are modeled using two-dimensional three-node constant strain elements and eight-node brick elements, respectively. The connections are examined under pretension alone and pretension plus some fraction of the expected service load. Plate separation between the end plate and the support, which is modeled as a rigid column flange, and vertical plate bending stresses are compared between the two models using correlation factors. The intent is that these factors could be used to extrapolate results for other connection configurations without the expense of a three-dimensional model. The solutions for the connections are obtained by trial and error in order to represent the contact problem between the end-plate and the rigid flange. Since contact algorithms were not available at the time, the user had to turn off spring elements upon each iteration if contact was not made at some location on the end-plate.

The authors admit that the mesh refinement of their model is very limited. This is important since three-node constant-strain elements provide highly inaccurate results for a coarse mesh. The correlation factors they develop may be arbitrary if the model is yielding improper results. In conclusion, the idea of correlating a two-dimensional model to a three-dimensional model is no longer necessary, as three-dimensional models are easily constructed.

2.2.2 *BURSI AND JASPART* (1997a)

Bursi and Jaspart (1997a) summarizes part one of a two-part investigation of finite element modeling of bolted connections. Unlike its companion paper (Bursi and Jaspart, 1997b), this paper does not consider moment end-plate connections themselves. It does, however, present a validation for the purpose of the study, which is to show that finite element programs can be used to accurately predict the behavior of moment end-plate connections. Hence, it is included here.

Tee stub connections are first modeled to determine the accuracy and/or calibration required when using finite elements to model connection behavior. Using the LAGAMINE software package, the models are constructed using both hexahedron (more commonly called brick) and contact elements. The contact elements utilize what is called a penalty technique. Here, a value is chosen as a penalty parameter and is similar to placing a spring between two bodies. Contact is simulated only for displacements within this given penalty value. Friction caused by the sliding and sticking between bodies is modeled with an isotropic Coulomb friction law. Nonlinear finite element analysis that considers large displacements, large rotations, and large deformations is used. Loads are applied using displacement as the controlling parameter. When considering the bolts, the additional flexibility provided by the nut and threaded region of the bolt are taken into account by using an effective length of the bolt. Due to the symmetry of the tee stub connection, only a quarter of the connection is modeled. Preloading forces in the bolts are taken into account by using applied initial stresses. The material properties are modeled using piece-wise linear constitutive laws for the material from experimentally tested connections. For several of these experimentally tested connections, a finite element analysis is performed. The finite element results compare quite nicely to experimental results. There is a slight difference in deflection values at the onset of yielding, which is primarily due to the presence of residual stresses in the actual tee stubs which is neglected in the finite element models of these members.

2.2.3 *BURSI AND JASPART* (1997b)

Bursi and Jaspart (1997b) is the second part of the two-part investigation by the authors. This paper uses the finite element method and the ABAQUS finite element code to analyze four-bolt unstiffened extended moment end-plate connections under static loading. The purpose of the study is to examine the stiffness and strength behavior of these connections. The finite element results are compared to those from an experimental study. End-plate rotation and bolt forces are both considered. The authors' intent is to show the feasibility of using the finite element method via commercial codes to determine moment–rotation characteristics of semi-rigid connections. Although dynamic characteristics of these connections are not considered, the authors do consider thin end-plates mainly for their ability to behave in a ductile manner when plate yielding occurs.

The finite element model considered by the authors is quite complex. The bolt and bolt head are modeled using beam elements. Both preloaded and non-preloaded bolts are considered, but only bolts in the tension region are included. The end-plate and beam elements are eight-node brick elements that allow plasticity. Contact elements are used to describe the interaction between the end-plate and the rigid column flange. Around the bolt holes, nodes are constrained in the direction perpendicular to the face of the end-plate. This assumption is very limiting, as tests and other finite element studies have shown that end-plates tend to pull away from the column flange even at the bolt locations. Other than friction forces taken by the contact elements, there are no lateral constraints mentioned in the paper. However, results are obtained even for the zero-friction case, which should result in divergence due to a singular stiffness matrix. Thus it is assumed that some other boundary condition is provided, but not discussed.

By comparison with experimental results, the results indicate that the model predicts the end-plate moment-rotation characteristics quite accurately. However, the bolt forces are not recorded experimentally and no comparison is made. The bolt axial force versus beam flange force seems reasonable in the plots provided. Bursi and Jaspart (1998) presents basically the same results as the paper discussed in this section and is not considered separately.

2.2.4 *BURSI AND LEONELLI* (1994)

Bursi and Leonelli (1994) presents some additional results that are not discussed in Bursi and Jaspart (1997b). Twenty-node brick elements are used to model the beam and plate material. Contact elements are used to represent the end-plate/column-flange interaction problem. Once again, beam elements are used to model the bolts, but here the bolts are pretensioned to a snug tight condition. The column flange is considered rigid. End-plate rotation and bolt loads are examined using the finite element model. Fairly good correlation with experimental results is obtained.

A direct application of Richard and Abbott (1975) is used to describe the analytical results obtained from the finite element model. Using the finite element method to obtain the elastic stiffness $K_{e,th}$, the inelastic stiffness $K_{p,th}$, the plastic failure moment $M_{p,th}$, and the ultimate applied moment $M_{u,th}$ for the connection, the moment-rotation plot or M-θ relationship can be described by

$$M = \frac{(K_{e,th} - K_{p,th})\theta}{\left\{1 + \left[\dfrac{(K_{e,th} - K_{p,th})}{M_{p,th}(1 - K_{p,th}/K_{e,th})}\right]^{n}\right\}^{\frac{1}{n}}} + K_{p,th}\theta \qquad (2.1)$$

where n is the shape factor.

2.2.5 *SHERBOURNE AND BAHAARI* (1997)

Sherbourne and Bahaari (1997) is the first part of a two-part study that aims to describe the moment-rotation characteristics of moment end-plate connections using results obtained from the finite element method. A three-dimensional finite element model of the four-bolt unstiffened extended end-plate is developed using the ANSYS finite element code. The bolt shank is modeled using truss elements and initial bolt strains are applied to model a snug tight condition. The bolt head and nut are made continuous with the end-plate and column flange, respectively. Contact elements are used to describe the end-plate/column-flange interaction problem. Material non-linearities are included in the analysis.

Moment-rotation curves produced by the finite element model are very similar to those of a previous experimental study. The column side of the connection is also considered, and the role played by the column flange strength in providing additional rotation is discussed. The most intriguing part of this study is that it considers the effect of geometric parameters of the end-plate configuration on the moment-rotation curve developed using the finite element method. The effects of column flange stiffening, end-plate thickness, bolt size, and bolt gage are all shown graphically. However, the effect of bolt pitch is not discussed and it is simply noted that a minimum pitch results in minimum prying forces.

2.2.6 *BAHAARI AND SHERBOURNE* (1997)

Bahaari and Sherbourne (1997) presents part two of the authors' finite element study on moment end-plate connections. Based on the parametric study found in Sherbourne and Bahaari (1997), this paper uses the Richard-Abbott power function (similar to that suggested by Bursi and Leonelli (1994)) to describe the moment-rotation behavior of four-bolt unstiffened extended moment end-plate connections of known geometrical configuration. The proposed moment-rotation relationship is

$$M = \frac{(K_i - K_p)\theta}{\left\{1 + \left[\frac{(K_i - K_p)}{M_o}\right]^n\right\}^{(1+n)}} + K_p\theta \tag{2.2}$$

where the elastic stiffness K_i, the inelastic stiffness K_p, and the plastic failure moment M_p are all obtained from the results of a finite element analysis. M_o and n are the connection-dependent reference moment and shape factor, respectively. If K_i and K_p are equal, the function becomes linear. Likewise, if K_p is zero, the curve becomes an elastic-plastic model of the connection's behavior. For large values of n, the model approaches a bilinear model of behavior. A curve fitting technique is used to determine the best set of values for the variables of (2.2) for numerous connection configurations. Using these results, an empirical equation is developed to describe the moment-rotation characteristics based on the end-plate configuration, bolt size, beam dimensions, and column dimensions. The results of this paper are eminent for the application of four-bolt unstiffened extended end-plates to semi-rigid connection philosophy. Although the moment-rotation plots given in the application examples included in the paper have decent correlation, the connection strength predicted by the method is off by as much as 75% in some cases.

2.2.7 *GEBBEKEN ET AL.* (1994)

Gebbeken et al. (1994) investigates different finite element modeling techniques to uncover the important criteria for describing moment end-plate connection behavior. Also, the authors discuss the results of a parametric study to determine which elements of the connection provide significant amounts of connection flexibility.

The four-bolt unstiffened extended end-plate connection is considered. First, a two-dimensional model is used. The material stress/strain relationship is represented as a bilinear function. Friction between the column flange and the end-plate is neglected. The results from this analysis are poor since strength predictions are very unconservative when compared to experimental results.

The three-dimensional model used by the authors provides some limited success in predicting the moment-rotation characteristics of the connection. The description of the finite element model is vague, yet it is mentioned that brick elements are used. Also, the figures in the paper make it appear that a tee stub and not an actual end-plate is considered. In some cases the results are accurate, but in others the strength is off by 50% or more, possibly suggesting inadequate modeling assumptions. Rothert et al. (1992) presents similar results and findings based on the same research.

2.2.8 OTHER FINITE ELEMENT STUDIES

There are several other papers that consider topics dealing with finite element modeling of moment end-plate connections, or finite element modeling of steel connections for seismic design. Bahaari and Sherbourne (1994) uses the ANSYS finite element program to develop a two-dimensional finite element model of four-bolt unstiffened extended moment end-plate connections. Sherbourne and Bahaari (1994) discusses the results from a three-dimensional finite element model of the same connection constructed using ANSYS. In particular, sources of rotation are tracked for the column flange, the bolts, and the end-plate. It is shown that the column flange provides little rotation when it is stiffened. Choi and Chung (1996) investigates the most efficient techniques of modeling four-bolt unstiffened extended end-plates using the finite element method. Shi et al. (1996) and Bose et al. (1997) use the finite element method to analyze flush end-plate configurations. Troup et al. (1998) applies ANSYS to model four-bolt extended unstiffened end-plates with shell elements. Moment-rotation curves are developed. Chi et al. (1997) and El-Tawil et al. (1998) use the finite element method to examine the “pre-Northridge” connection for fracture characteristics and connection ductility, respectively. Mistikadis et al. (1998) models a column base-plate using a two-dimensional finite element model. Youssef and Lee (1998) considers dog-bone connections. Leon and Swanson (1998) discusses the effectiveness of bolted connections in moment-resisting frames.

2.3 PR CONSTRUCTION AND SYSTEM ANALYSIS

Several papers in the area of partially restrained (PR), sometimes called semi-rigid, construction are now considered. These papers do not have any direct relevance to the work herein, but they do consider the moment-rotation characteristics of steel connections. These connections are usually much more flexible than typical end-plate connections. The analysis of structural systems composed of PR connections is somewhat similar to the analysis of systems composed of moment connections that can yield during an earthquake.

Kukreti et al. (1987) provides an analytical relationship between moment and rotation for flush moment end-plate connections. The authors consider a range of connection geometry compatible to the metal building industry. Hasan et al. (1997)

presents insight about the stiffness and strength requirements of moment end-plate connections to be considered as FR connections. By analyzing frames composed of connections with known moment-rotation characteristics, the authors conclude that connections with an initial stiffness $K_I>10^6$ kip-in./rad should be considered as FR connections. A similar classification system for rigid and semi-rigid connections is proposed by Goto and Miyashita (1998) for general connections. Yee and Melchers (1986) and Foley et al. (1995) consider the typical moment-rotation characteristics of moment end-plate connections. The analysis and design of steel frames composed of semi-rigid connections is considered by Coric and Markovic (1998), Elghazouli (1998), Yu et al. (1998), Rodrigues et al. (1998), and Fu et al. (1998). Leon (1997) discusses PR composite frame design and analysis.

Faella et al. (2000) considers the theory and design of steel semi-rigid connections and contains a chapter specifically on moment end-plate connections. Design procedures presented for extended end-plates based on yield line theory are very complex and are modified to correlate with test results. Yield line patterns for the limit states of end-plate bending only consider the part of the plate above the tension flange. This is abnormal since sufficient experimental testing exists to show that most of the end-plate yielding takes place inside the beam flange.

A three-dimensional analysis of a thirteen-story steel building with weld connection damage is performed by Lobo et al. (1998). The source of plasticity (i.e., beam, column, or connection) is shown for each floor level. Song (1997) considers the seismic response of several multi-story steel frames composed of strong columns and weak beams or vice versa. Energy dissipated by the contributing elements is considered. Shimamura et al. (1998) treats the seismic response of large spatial structures. In particular, the roofing system of a large dome is analyzed three-dimensionally.

2.4 DOUBLER PLATE/PANEL ZONE BEHAVIOR AND DESIGN REQUIREMENTS

Panel zone yielding provides an efficient means of energy dissipation during an earthquake. However, there are limits to the benefits that can be gained, and an improperly designed panel zone could have a detrimental effect on the entire structure. Bertero et al. (1972) concludes that the panel zone should be proportioned within an optimal range to ensure that any yielding during an earthquake would be benign. Said another way, they conclude that a weak panel zone will not dissipate enough energy and an excessively rigid panel zone will not contribute toward energy dissipation at all. As a matter of practicality, these authors suggest that the panel zone be designed so that yielding will first occur in the connecting beams.

Krawinkler et al. (1975) bolsters these early conclusions and add some additional insight. It is determined that limited shear buckling could occur in the panel zone without any noticeable loss in strength. However, it is also shown that kinking of the column flanges at the corners of the panel zone might occur at large deformations. Other experimental tests that consider panel zone deformations include Schneider et al. (1993), Lee and Lu (1989), and Lee and Uang (1997). Roeder et al. (1993) and Schneider and Amidi (1998) have performed analytical research. Tsai and Popov (1990) and El-Tawil et al. (1999) have also done finite element studies. The results of these efforts are consistent with the findings of earlier researchers.

Slutter (1981) experimentally considers four panel zone configurations. In one case, no doubler plate is used. In the other three cases, the doubler plate is welded to the column flange with either a complete-joint-penetration groove-welded detail or a fillet-welded detail. For the latter cases, the doubler plate extends beyond the stiffener and is fillet welded to the column web. Several conclusions are made in reference to the vertical fillet welding. The panel zone tests show a large ductility with no signs of weld failure. The performance of the fillet welds is very similar to that of the groove welds except for a slight decrease in stiffness. Although no conclusions are drawn about the horizontal welding, the size used corresponds to the AISC Specification minimum fillet weld size.

Although, in general, plasticity in the panel zone of a beam-to-column connection is a benign event, limits must be established. Due to the occurrence of column flange kinking that accompanies large deformations of the panel zone, current guidelines (FEMA 267 and 267A) recommend that an additional factor be used to establish the panel zone shear strength. The *AISC Seismic Provisions for Structural Steel Buildings* (1997) incorporates this recommendation, which is presented in the Chapter 7. In addition, particular requirements regarding doubler plate welding have been established.

To allow for the desirable energy dissipation that can occur through panel zone yielding, the vertical welds (i.e., doubler plate to column flange) must be proportioned to develop the full web doubler plate thickness. Horizontal welds must be designed to carry the portion of the unbalanced force from the transverse stiffener that is transmitted to the doubler plate, if any. For the extended doubler, there is no force transfer, but for the fitted case, there might be if some of the unbalanced stiffener force has to be carried by the doubler.

2.5 NEED FOR FURTHER RESEARCH

The need for larger moment end-plate connections is manifested in Chapter 1, where it is shown that current code requirements and design recommendations are leaning away from allowing connection yielding as a structure's primary source of inelastic behavior. Also, since engineers are seeking alternatives to the flange directly welded connection that was commonplace prior to the Northridge earthquake, it is only reasonable that larger moment end-plate connections should be available for the design of larger lateral force resisting systems (i.e., larger buildings).

However, as shown in this literature survey, very little research on moment end-plate connections larger than the four-bolt extended configuration exists. What is available on larger connections usually only presents the results of experimental testing. Papers dealing with finite element modeling of moment end-plate connections deal almost exclusively with the four-bolt extended connection. This is primarily due to its common use, relatively well-understood behavior, and modeling simplicity. It is evident in the research discussed in this chapter that the finite element method has evolved significantly over the years. Crude models of moment end-plate connections have become very refined and required correlation factors have become extinct, since one can effectively model all the components of the connection accurately. However, as discussed in the previous sections of this chapter, limitations or assumptions established in prior studies should be addressed, and the research of this dissertation aims to provide the most accurate and detailed models of moment end-plate connections to date. Using this validated model (Chapter 3), the results obtained from the ongoing moment end-plate project funded by SAC are validated (Chapter 4), a parametric study of the effects of end-plate geometry on connection response is presented (Chapter 5), and a design procedure for large moment end-plate connections is developed (Chapter 6). Other finite element models are used to consider panel zone behavior (Chapter 7), single-story and multi-story building analysis (Chapter 8), and column-side behavior and design of moment end-plate connections (Appendix A).