Evaluation of the Inelastic Rotation Capability of Flush End-Plate Moment Connections

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(ABSTRACT)

An experimental investigation was conducted to study the inelastic rotation capability of flush end-plate moment connections. Seven specimens representing two-bolt and four-bolt flush end-plate configurations were tested under cyclic loading. “Quasi-static” or “slow-cyclic” loading histories suggested by SAC and the Applied Technology Council were used to load the specimens. Experimental results for maximum moment resisted by the connections were compared with analytical predictions. Moment strengths of the connections were calculated using yield-line theory to predict end-plate yielding and maximum bolt force calculations including prying action. Experimental results were also compared to previous research with regards to strength and stiffness. The inelastic rotation of connections was calculated and conclusions were drawn on the compliance of these connections with current AISC specifications.
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CHAPTER I

INTRODUCTION AND LITERATURE REVIEW

1.1 INTRODUCTION

The moment end-plate connection is one of the fully restrained moment connections as defined by the American Institute of Steel Construction (AISC) Manual of Steel Construction, Load and Resistance Factor Design (LRFD) (Manual 1994). Bolted, flush end-plate connections have been used extensively in the metal building industry as splices between beams, and beam-column connections in portal frame construction. The popularity of these connections stems from their ease of fabrication and erection in the field.

The beam-to-column flush moment end-plate connection consists of a steel plate welded to the end of a beam flange. The end-plate is then bolted to the column flange using various rows of fully tensioned high-strength bolts. The flush end-plate moment connection is one in which the end-plate does not extend above the beam flange. One or two rows of bolts at each flange can be used. Figure 1 shows the two and four-bolt flush configurations. This study is limited to these two configurations.

During the 1994 Northridge earthquake, many weld cracks were found in flange-welded moment connections. Because of this, bolted moment connections have seen a rise in popularity as engineers seek alternatives to the direct welded connection. The flush moment end-plate is considered a fully restrained connection but as of yet has not
been approved for usage in seismic zones due to lack of research into its inelastic rotation capability.

The purpose of this research is to investigate the usage of the flush moment end-plate connection in seismic areas. Experimental research was conducted to determine the inelastic rotation capability of the flush moment end-plate connection, and to determine its effectiveness as a fully restrained connection under seismic loading.

1.2 Previous Research on Flush End-Plate Moment Connections

Zoetermeijer (1981) analyzed an infinitely long plate bounded by two fixed edges and one free edge, and loaded with a concentrated force using yield-line theory. He
developed a chart which may be used to calculate the ultimate load of a stiffened column flange or a flush end-plate once the distance from the bolt to the beam flange and web are known. He noted, however, that the analysis was only and approximation.

Phillips and Packer (1981) conducted a series of tests to determine the influence of the end-plate thickness on moment-rotation characteristics and the end-plate collapse mechanism. They also studied end-plate connections with two rows of tension bolts in order to study the influence of the second row on the stiffness of the connection. They suggested two failure mechanisms for end-plates with two rows of bolts, by which they determine the required thickness of the end-plate. They concluded that flush end-plates with two rows of bolts in the tension region are suitable for semi-rigid construction, and that the second row of tension bolts is effective but to a much lesser extent than previously estimated.

Srouji et al. (1983) investigated the behavior of four types of end-plate moment connections including the two-bolt, and four-bolt flush connection included in this study. Srouji investigated the use of yield-line analysis for predicting the failure moment, and the Kennedy method, modified for use with end-plates, to predict bolt forces for these connections. Srouji used a combination of experimental results, finite element analysis, and comparison with previous research to verify the analytical predictions.

Hendrick et al. (1985) performed more experiments on flush end-plates and developed a unified design methodology for flush end-plates. Hendrick developed an empirical equation to estimate the distance from the bolt line to the location of prying forces.
Borgsmiller and Murray (1995) performed experiments on flush, extended stiffened, extended unstiffened, and multi-row extended end-plate moment connections. They devised a simplified method for design of end-plate moment connections based on two limit states, end-plate yielding, and bolt rupture. Yield line analysis was used to predict the strength due to end-plate yielding, and a modified version of the Kennedy method was used to calculate the connection strength due to bolt rupture. Proposed in the study was a simplified method to calculate connection strength due to bolt rupture. This method has its origin in the Kennedy method, but a simplifying assumption is made to greatly reduce the time and complexity of calculation involved in using the Kennedy method.

1.3 Previous Cyclic Testing of End-Plate Moment Connections

The tests discussed in the previous section were all performed under static loading conditions. Up to this point, there has been very little research done on the performance of flush end-plate moment connections under seismic loading. Research on the performance of four-bolt-extended-unstiffened and four-bolt-extended-stiffened end-plate moment connections under cyclic loading was performed by Tsai and Popov (1990). While they only did a limited number of tests, Tsai and Popov found the four-bolt-extended-unstiffened end-plate connection to perform well under cyclic loading. They did however have to increase bolt and end-plate sizes over what was required by the current design methods at the time.
Ghobarah et al. (1990) investigated extended end-plates under cyclic loading to investigate the effects of design parameters such as end-plate thickness, column flange stiffener and bolt pre-tension force on the overall behavior. They found that end-plate connections are able to dissipate energy induced in the connection from cyclic loading without loss of strength. This energy dissipation can be achieved through proper detailing of the connection.

Meng (1996) addressed the issue of seismic design of end-plate moment connections. Through the use of full scale moment end-plate connection tests, tee stub tests, and finite element modeling, Meng developed an LRFD based design procedure for four-bolt extended, shimmed four-bolt, and 8-bolt (four-bolt wide) moment end-plate connections. The presence of weld access holes in some of the tests were determined to be the cause of flange fractures during these tests. He deemed the four-bolt extended end-plate moment connection satisfactory for use in seismic zones. He also indicated that the 8-bolt (four-bolt wide) connection exhibits satisfactory performance under cyclic loading, but also reports that more research and analysis is needed before this connection can be considered as an alternative to flange welded connections.

Kline et al. (1989) investigated the performance of several end-plate configurations, including flush end-plate connections, under wind loads, subjecting them to numerous cycles. Kline et al. concluded that end-plate moment connections using snug tight bolts are adequate in resisting expected wind loads, but provide reduced stiffness when compared to moment end-plates with fully tightened bolts.
1.4 **Inelastic Rotation Requirements**

Current design recommendations (FEMA 1995) and the *AISC Seismic Provisions for Structural Steel Buildings (Seismic 1997)* divide moment frames into three broad categories: special, intermediate, and ordinary. Beam-to-column connections used in each of the three categories must meet strength and inelastic rotation requirements. Only tested connections are permitted for use in the Seismic Force Resisting System.

Special moment frames are expected to withstand significant inelastic deformations during the design seismic event and their beam-to-column connections must exhibit an inelastic rotation of at least 0.03 radians. Beam-to-column connections shall demonstrate a flexural strength at the column face of at least the plastic moment strength of the beam. When beam local buckling rather than beam yielding limits the flexural strength of the beam or when reduced flange width beam sections are used, the connections are allowed to be designed to withstand $0.8M_p$ of the tested beam. Connections that provide the required rotation through deformation of the various connection elements while maintaining the required design strength are permitted as long as the effects on drift and overall frame stability are considered.

Intermediate moment frames are expected to withstand moderate inelastic deformations during the design seismic event and their beam-to-column connections must exhibit an inelastic rotation of at least 0.02 radians. Fully-restrained (FR) connections in an intermediate moment frame must attain the required inelastic rotation through the plastic deformation of members in the frame. Partially-restrained (PR) connections are allowed for usage in an intermediate moment frame and may obtain the required inelastic
rotation through the plastic deformation of the connections themselves, such as plate yielding and bending of bolts. As with special moment frames, beam-to-column connections must have at least the same moment strength at the column face as the plastic moment strength of the beam. Like special moment frames, a connection strength of 0.8M_p is permitted if beam local buckling is the controlling limit state, or if a reduced beam section is used.

Ordinary moment frames are expected to withstand limited inelastic deformations during the design seismic event and their beam-to-column connections must exhibit an inelastic rotation of at least 0.01 radians. FR connections in the seismic force resisting system must be designed for a moment strength of at least 1.1R_yM_p of the beam or the maximum moment that can be delivered to the system, whichever is larger. The Parameter R_y is the ratio of the expected yield strength to the minimum specified yield strength. PR connections are also permitted for use as long as certain strength and frame stability conditions are met.

1.5 Objective and Scope of Research

The primary purpose of this research is to investigate the performance of flush end-plate moment connection under seismic loading. Seven full-scale tests were conducted to determine the inelastic rotation capability of these connections. The specimens were designed using a combination of yield-line theory and maximum bolt force predictions which include an estimate of prying forces. The design methodology for flush end-plate moment connections is described in Chapter II. The test specimens,
set-up, and procedures are described in Chapter III. Results of the full-scale testing is
given in Chapter IV, with summary, conclusions and recommendations presented in
chapter V.
CHAPTER II

FLUSH END-PLATE MOMENT CONNECTION DESIGN

2.1 OVERVIEW

The flush end-plate moment connections in this study were designed taking into account the two limit states of end-plate yielding and bolt rupture. The strength of the connection due to end-plate yielding was found using yield line theory, and the connection strength due to bolt rupture was found using a modified version of the Kennedy method including prying forces.

Included in this chapter is the development of the yield line mechanisms to calculate the connection strength due to end-plate yielding, and the development of the modified Kennedy method to calculate the connection strength due to bolt rupture.

2.2 END-PLATE YIELD-LINE STRENGTH

Yield-line theory was first introduced as a way to analyze the behavior of concrete slabs. A yield line is a continuous formation of plastic hinges along a straight or curved line. A failure mechanism is said to exist when the yield lines form a kinematically valid collapse mechanism. The following guidelines are followed when establishing yield line patterns:

1. Axes of rotation generally lie along lines of support.
2. Yield lines pass through the intersection of the axes of rotation of adjacent plane segments.
3. Along every yield line, the bending moment is assumed to be a constant and is taken as the plastic moment of the plate.

The following is a discussion of the yield-line analysis of flush end-plate performed by Srouji et al (1983). The analysis of a yield-line mechanism can be performed by two different methods, the equilibrium method and the virtual work or energy method. Srouji et al. analyzed yield line mechanisms for flush end-plates using the virtual work method. In this method, the external work done by the applied load from a small arbitrary virtual deflection is set equal to the internal work done as the plate rotates at the yield lines to accommodate this virtual deflection. For a specified yield-line pattern and loading, a certain plastic moment will be required along the hinge lines. For the same loading, other patterns may result in a larger required plastic moment capacity. Hence, the controlling pattern is the one which requires the largest required plastic moment. Or, for a given plastic moment capacity, the controlling mechanism is the one which produces the lowest failure load. This implies that the yield-line theory is an upper bound procedure and the least upper bound must be found.

To determine the required plastic moment capacity or the failure load, a number of possible yield-line mechanisms must be investigated. By equating the internal and external work, the relationship between the applied loads and the ultimate resisting moment is obtained. The resulting equation is then solved for either the unknown loads or unknown moments, and by comparing the different values obtained from the various mechanisms, the controlling minimum load (or maximum required plastic moment) is obtained.
The internal energy stored in a particular yield-line mechanism is the sum of the internal energy stored in each yield line forming the mechanism. The internal energy stored in any given yield line is obtained by multiplying the normal moment on the yield-line with the normal rotation of the yield-line. Thus the energy stored in the nth yield-line of length $L_n$ is:

$$W_{in} = \int_{L_n} m_p \theta_n \, ds$$  \hspace{1cm} (2.1)

where $\theta_n$ is the relative rotation of line n, and $ds$ is the elemental length of line n. The internal energy stored in a yield-line mechanism can be written as:

$$W_i = \sum_{n=1}^{N} \int_{L_n} m_p \theta_n \, ds$$ \hspace{1cm} (2.2)

$$= \sum_{n=1}^{N} m_p \theta_n L_n$$

where $N$ is the number of yield lines in the mechanism.

Srouji et al. investigated a number of yield line mechanisms for two-bolt and four-bolt flush end-plates. They found that for each end-plate configuration one yield-line mechanism controlled. The controlling yield line mechanism for the two bolt flush end-plate is shown in Figure 2.1. The yield line mechanism for the four-bolt flush endplate is shown in Figure 2.2. The geometric parameter are also defined in the figures. The internal energy stored in the yield-line mechanism shown in Figure 2.1 is:
FIGURE 2.1 Yield Line Pattern for Two-Bolt Flush End-Plate

a) Dimensions

b) Yield Line Mechanism
a) Dimensions

b) Yield Line Mechanism

**Figure 2.2** Yield Line Pattern for Four-Bolt Flush End-Plate
\[ W_i = 4m_p \frac{(h - p_i)}{h} \left[ \frac{b_f}{2} \left( \frac{1}{p_f} + \frac{1}{s} \right) + (p_f + s) \left( \frac{2}{g} \right) \right] \]  \tag{2.3}

where \( p_f \) is the distance from the bolt centerline to the face of the flange, equal to \( p_i - t_f \), and \( s \) is the distance between parallel yield lines. The quantity \( s \) in Equation 2.3 is obtained by differentiating the internal work equation with respect to \( s \) and equating to zero, resulting in:

\[ s = \frac{1}{2} \sqrt{b_fg} \]  \tag{2.4}

The internal energy stored in the straight yield-line mechanism for a four-bolt flush end-plate shown in Figure 2.3 is given by:

\[ W_i = \frac{4m_p}{h} \left[ (h - p_i) \left[ \frac{b_f}{2} \left( \frac{1}{p_f} + \frac{1}{s} \right) + \frac{2}{g} (p_f + p_b + s) \right] - \frac{b_fp_b}{2s} \right] \]  \tag{2.5}

where \( p_b \) is the pitch between bolt rows, and \( s \) is the distance between parallel yield lines. The quantity \( s \) is determined in the same manner as \( s \) in Equation 2.3 e.g., by differentiating the internal work equation with respect to \( s \) and equating to zero, resulting in:

\[ s = \frac{1}{2} \sqrt{b_fg} \frac{(h - p_i - p_b)}{(h - p_i)} \]  \tag{2.6}
For all the end-plates, the external work done due to a unit displacement at the top of the tension beam flange, resulting in a rotation of the beam flange, resulting in a rotation of the beam cross-section about the outside of the compression flange is given by:

\[ W_e = M_u \theta \]  \hspace{1cm} (2.7)

Where \( M_u \) is the factored beam moment at the end-plate, and \( \theta \) is the rotation at the connection, equal to \( 1/h \), where \( h \) is the beam depth.

On equating the internal and external work terms and canceling \( \theta \), the expression for the ultimate moment, \( M_u \), can be obtained. Then by rearranging the expression for \( M_u \), the equation for \( t_p \), the end-plate thickness, can be written in terms of \( M_u \). To obtain equations for \( M_u \) and \( t_p \) in terms of \( F_{py} \) and \( t_p \), \( m_p \) can be replaced with the following:

\[ m_p = \frac{F_{py} t_p^2}{4} \]  \hspace{1cm} (2.8)

where \( m_p \) is the plastic moment strength of the end-plate per inch of width, \( F_{py} \) is the yield strength of the end-plate, and \( t_p \) is the end-plate thickness. The resulting equations for required end-plate thickness from those yield-line patterns are as follows.

For a two-bolt flush end-plate:

\[ t_p = \sqrt{\frac{M_u}{F_{py}}} \left[ \frac{b_f}{2} \left( \frac{1}{p_f} + \frac{1}{s} \right) + (p_f + s) \frac{2}{g} \right] \]  \hspace{1cm} (2.9)
For a four-bolt flush end-plate:

\[
\frac{M_o}{F_{py}} = \sqrt{\left( h - p_t \right) \left[ \left( \frac{b_t}{2} \left( \frac{1}{p_t} + \frac{1}{s} \right) + \frac{2}{g} (p_t + p_b + s) \right) \right] - \frac{b_t p_b}{2s}}
\]  
(2.10)

From these equations connection strength according to yield-line theory can be determined as follows:

For a two-bolt flush end-plate.

\[
M_{pl} = t_p^2 F_{py} \left( h - p_t \right) \left[ \frac{b_t}{2} \left( \frac{1}{s} + \frac{1}{p_t} \right) + (p_t + s) \left( \frac{2}{g} \right) \right]
\]  
(2.11)

For a four-bolt flush end-plate:

\[
M_{pl} = t_p^2 F_{py} \left[ \left( h - p_t \right) \left[ \frac{b_t}{2} \left( \frac{1}{s} + \frac{1}{p_t} \right) + \frac{2}{g} (p_t + p_b + s) \right] \right] - \frac{b_t p_b}{2s}
\]  
(2.12)

### 2.3 Prediction of Bolt Forces Including Prying Action

Kennedy, et al. (1981), used a split-tee analogy to predict bolt forces for tee-hanger connections. In this study, three types of plate behavior was identified. The first case, to which all plates belong under low applied load, is identified by the absence of
plastic hinges in the plate. Bolt prying force is assumed to be zero, and the plate is said to be “thick”. The next stage that the plate passes through is an “intermediate” stage. This is exemplified by a plastic hinge that forms at the flange. The plate is said to be of intermediate thickness and the prying force is somewhere between zero and a maximum possible value. As the load increases, a second plastic hinge forms at the bolt lines and the prying forces are considered to be at a maximum. When the plate is in this stage it is considered to be “thin”. The model for this theory is shown in Figure 2.3.

![Kennedy Split-Tee Model](image)

**Figure 2.3** Kennedy Split-Tee Model

This model was applied to flush end-plate moment connections by Srouji et al. For the flush two-bolt connection, the model is simply one half of the Kennedy Split-Tee
Model and is shown in Figure 2.4. For a flush four-bolt connection, Kennedy’s Split-Tee Model was modified. This model is shown in Figure 2.5.

To determine which stage of behavior the end-plate is in, the thick plate limit and, if necessary, the thin plate limit must be calculated. Using a flange force calculated from a pre-determined moment, the thick plate limit, \( t_1 \), is calculated using the following equation:

\[
t_1 = \frac{4p_1 F_f}{\sqrt{b_f \left( F_{py}^2 - 3 \left( \frac{F_f}{b_f t_1} \right)^2 \right)}}
\]

(2.13)

Where \( F_f = \) total flange force. Because this equation is iterative, the following approximation is given to find a preliminary value for \( t_1 \) with which to start iterations:

\[
t_1 = \frac{4.21 p_1 F_f}{\sqrt{b_f F_{py}}}
\]

(2.14)

Once the thick plate limit is determined, it is compared to the actual end-plate thickness. If \( t_p > t_1 \), then the end-plate is considered to be “thick” and the prying force is taken to be zero. If \( t_p < t_1 \), then the prying force is not zero and the thin plate limit, \( t_{11} \), must be calculated using the following equation:

\[
t_{11} = \frac{2 \left( F_f p_1 - \frac{\pi d_s^3 F_{yb}}{16} \right)}{b_f \sqrt{2 F_{py}^2 - 3 \left( \frac{F_f}{b_f t_{11}} \right)^2 + w' \sqrt{F_{py}^2 - 3 \left( \frac{F_f}{2w t_{11}} \right)^2}}}
\]

(2.15)
**Figure 2.4** Modified Kennedy Model for Two-Bolt Flush End-Plate

**Figure 2.5** Modified Kennedy Model for Four-Bolt Flush End-Plate
where \( w' \) = width of end-plate per bolt line minus the diameter of the bolt. Because the equation for \( t_{11} \) is iterative, an approximation is given to find a preliminary value for \( t_{11} \) with which to start iterations:

\[
t_{11} = \sqrt{\frac{2 F_t F_{py} \pi d_k^3 F_{yb}}{16 F_{py} \left( \frac{0.85 b_f}{2} + 0.8 w' \right)}}
\]

(2.16)

To insure that the end-plate is not failing due to shear effects, the following equation should be satisfied prior to starting iterations with Equation 2.5.

\[
F_t < \frac{2 w' t_{11} F_{py}}{\sqrt{3}}
\]

(2.17)

Where \( t_{11} \) is found from Equation 2.16. If Equation 2.17 is not satisfied, the beam capacity must be increased so that shear failure does not occur. The thin plate limit is then compared to the actual end-plate thickness. If \( t_p > t_{11} \), then the plate is considered to be intermediate, the prying forces are not equal to zero, and can be calculated using the appropriate equations. If \( t_p < t_{11} \), then the end-plate is considered to be thin, the prying forces are at a maximum and are calculated using the appropriate equations.

Kennedy et al. (1981) considered bolt forces to be the sum of a portion of the flange force and the prying forces. The prying action varies according to what stage of behavior the end-plate is in. For thin end-plates the prying force is at a maximum and is given by the following equation:
\[ Q_{\text{max}} = \frac{w't_p^2}{4a} \sqrt{\frac{f_{\text{py}}^2}{3} \frac{F'}{w't_p^2}} \]

(2.18)

where \( Q_{\text{max}} = \) maximum prying force, \( a = \) distance from the prying force to the bolt line, and \( F' = \) flange force per bolt. Where \( F' \) is taken as the lesser of \( F_{\text{limit}} \) or \( F_{\text{max}} \) as calculated by the following:

\[ F_{\text{limit}} = \frac{t_p^2 F_{\text{py}} \left( \frac{0.85b_f}{2} + 0.80w' \right)}{2} + \frac{\pi d_b^3 F_{y_b}}{8} + \frac{F_{p} + F_{p}'}{4p_f} \]

(2.19)

or \( F_{\text{max}} = \frac{(F_f)_{\text{max}}}{2} \)

(2.20)

An equation for the calculation of the “\( a \)” distance was developed by Hendrick et al. (1985) and verified by experimental results. This equation is a function of the end-plate thickness and the bolt diameter and is given as:

\[ a = 3.682 \left( \frac{t_p}{d_b} \right)^3 - 0.085 \]

(2.21)

Srouji et al. (1983) give two equations for prying forces of flush end-plates in the intermediate stage, one for two-bolt flush, and one for four-bolt flush. The prying force for an intermediate two-bolt end-plate is:
where $F = \text{flange force per bolt} = F_d/2$ and “a” is found from Equation 2.21. The prying force equation given by Srouji et al. for an intermediate four-bolt flush end-plate is:

$$Q = \frac{F_{p_f}}{a} - \frac{b_t t_p}{8a} \sqrt{F_{py}^2 - 3 \left( \frac{2F}{b_t t_p} \right)^2} - \frac{\pi d_b^3 F_{yb}}{32a}$$  \hspace{1cm} (2.22)

where $F = F_d/2$. For two-bolt flush end-plate connections, half of the flange force plus prying forces is assumed to be distributed to each of the bolts.

For four-bolt flush end-plate connections, the amount of flange force distributed to each bolt is dependent on the stage of end-plate behavior. For a thick end-plate, the outer row of bolts is assumed to carry all of the flange force, while the force in the inner row of bolts is assumed to be zero. For an intermediate end-plate, the outer bolt force, $B$, is given by:

$$B = \frac{F_l}{2.5} + Q$$  \hspace{1cm} (2.24)

For a thin end-plate the outer bolt force, $B$, is given by:

$$B = \frac{3F_l}{8} + Q_{\text{max}}$$  \hspace{1cm} (2.25)
By setting the bolt force, (B), to the unfactored tensile strength of the bolt, these equations can thus be solved for a flange force, which in turn can be used to calculate the moment strength of the connection due to bolt fracture.
CHAPTER III

TESTING PROGRAM

3.1 TESTING PROGRAM

Full scale testing was conducted using built up sections and flush end-plate moment connections. A325 bolts and A572 Gr50 steel were used for the connection components. Two flush end-plate configurations were tested in the study: 2-bolt and 4-bolt unstiffened. The nominal geometry is listed in Table 3.1, and the details of the end-plate connections tested are shown in Figure 3.1. The test designations shown in the Table 3.1 are to be interpreted as follows: F1-3/4-3/8-16 designates a flush end-plate with one row of ¾ in. diameter bolts at each flange, the end-plate thickness is 3/8 in. and the beam depth is 16 in. Because the testing was cyclic, the same bolt pattern was used at both flanges. The connections were designed so that failure would occur within the connection elements and not in the test beam or column. This was achieved by computing an end-plate strength from yield line theory and the bolt force prediction method described in Chapter II, and then designing beams and columns to be approximately ten percent stronger than the connections. The same end-plate was welded to both sides of the test beam allowing two tests for each end-plate configuration.
3.2 Test Set-Up

The physical test setup for the evaluation of the connections was a cantilevered beam connected to a column section as shown in Figures 3.2 and 3.3. A section cut through the test set-up is shown in Figure 3.2. The test setup was erected upright in the vertical plane, eliminating any moment caused by gravity forces.

<table>
<thead>
<tr>
<th>Test Designation</th>
<th>Bolt Diameter (in)</th>
<th>End-Plate Thickness (in)</th>
<th>Beam Depth (in)</th>
<th>Flange Width (in)</th>
<th>Pitch (in)</th>
<th>Gage (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1-3/4-3/8-16</td>
<td>3/4</td>
<td>3/8</td>
<td>16</td>
<td>6</td>
<td>1 1/2</td>
<td>3 1/2</td>
</tr>
<tr>
<td>F2-3/4-3/8-16</td>
<td>3/4</td>
<td>3/8</td>
<td>16</td>
<td>6</td>
<td>1 1/2</td>
<td>3 1/2</td>
</tr>
</tbody>
</table>

perpendicular to the loading plane. The test column was bolted to the test set-up in a way to create what was essentially a simple connection. Whether or not this in fact was a truly simple connection was not a concern in this study, because the rotation was measured in a manner that accounts for both rigid body motion of the test frame and test column rotation at the connection. Lateral bracing of both beam flanges was provided at intervals such to prevent failure by lateral torsional buckling.
Figure 3.1 End-Plate Details
Figure 3.2 Section Through Test Set-Up Diagram
FIGURE 3.3 Test Setup
The specimen was loaded by means of a 200 kip hydraulic ram placed 6 ft from the center of the test column. No axial loads were applied to the column or beam sections.

3.3 INSTRUMENTATION

Instrumentation for recording of test data included a 200 kip capacity tension-compression load cell, a linear displacement transducer to measure beam deflection at a distance of 1 ft offset from the load point, potentiometers to measure the end-plate and column flange separation at the top and bottom of the end-plate, strain gauges to measure beam flange strain and instrumented bolts to measure bolt strain. Linear transducers were also used to measure the vertical displacement and rotation of the test column. This allowed for the determination of the rotation at the end-plate, taking into account the rotation and vertical displacement of the test column.

The end-plate connection bolts used in the tests were instrumented with an internal strain gauge to measure real time strain within each bolt. In addition to recording bolt strain during these tests, the instrumented bolts allowed for accurate determination of pre-tension force during tightening.

To prepare the instrumented bolts, a 2 mm hole was drilled in the head of the bolt to a depth such that a special strain gauge could be inserted below the head of the bolt but above the threaded portion of the shank. This is done such that the 2 mm hole does not reduce the strength of the bolt. This is possible because the reduction of the bolt shank area is much less than the reduction of area in the threaded portion of the bolt. After insertion of the strain gauge, an epoxy was injected into the hole which, upon curing,
formed a tight bond between the gauge and bolt material. Each bolt was then calibrated for tension load versus strain using a tensile test machine. Data from each test was recorded from test setup to eventual failure, through the use of the System 4000 data acquisition system. The data was transferred via disk media to commercial software for analysis in spreadsheet and graphical analysis software.

### 3.4 Loading Protocol

The specimens were tested using one of two loading protocols. The first three tests (F1-3/4-3/8-16 tests 1 and 2, and F2-3/4-3/8-16) were conducted using the loading protocol outlined in ATC-24, *Guidelines for Cyclic Seismic Testing of Components of Steel Structures* (1992). The procedure is for use in slow cyclic load application which is significantly less than the real time frequency of an earthquake. This method of testing is used for its cost effectiveness and allows for more observation of damage to the structural components as they occur. A major limitation of this method is that it severely distorts time and its material property effects. ATC-24 states, however, that results from tests using this methodology can be considered conservative because the slow cyclic loading results in a small decrease in strength and an increase in the rate of deterioration. Although this may seem to limit the value of the results, slow cyclic testing does provide valuable data for overall connection performance.

To gain an understanding of the nature and process of cyclic testing of connections, the following definitions are given from ATC-24.
Cycle: a load or deformation history unit consisting of two sequential excursions, one in the positive and one in the negative loading direction.

Deformation: a generic quantity, $\delta$, including strains, angles of shear distortion, rotations, axial deformations and displacements.

Ductility Ratio: a ratio of peak deformation over yield deformation.

Excursion: a load or deformation history unit that starts and finishes at zero load, and contains a loading and unloading branch.

Force: a generic quantity, $Q$, including internal forces and externally applied loads.

Hysteretic Area: the area enclosed by a force-deformation diagram.

Load or Deformation Step: a load history unit consisting of a series of cycles with constant peak load or deformation.

Peak Deformation: the deformation at a load reversal point.

Total Deformation Range: the total deformation between the peak of an excursion and the peak of the preceding opposite excursion.

Yield Force of Deformation: the predicted or measured force or deformation at which significant yielding occurs.

While using the ATC-24 loading protocol two methods of loading control are utilized during each specimen test, force and deformation. As their names imply, the control of loading by the tester is either by the force applied to the specimen or by the measured deformation of the specimen. For the tests performed in this study, the force
control is monitored from values given by the load cell attached to the hydraulic ram used to apply load. The deformation method is in reference to the deflection of the specimen at a distance inboard one foot from the load point.

The ATC-24 loading history is shown as Figure 3.4. ATC-24 makes several recommendations on the numbers of cycles and peak deformations in each load step. They are as follows:

- The number of cycles $n_0$ with a peak deformation less than $\delta_y$ should be at least six.
- The number of cycles $n_1$ with a peak deformation $\delta_1$ equal to $\delta_y$ should be at least three.
- The number of cycles $n_2$ with peak deformation $\delta_2 = \delta_y + \Delta$ should be at least three unless a lower number can be justified.
- The number of cycles $n_3$ with peak deformation $\delta_3 = \delta_y + 2\Delta$ should be at least three

![Figure 3.4 ATC-24 Loading Protocol](image-url)
unless a lower number can be justified.

- The number of cycles $n_4$ to $n_m$ with peak deformation $\delta_4 = \delta_y + 3\Delta$ to $\delta_m = \delta_y + (m - 1)\Delta$ should be at least two unless a lower number can be justified.

To use this loading protocol, an accurate calculation of the yield load of the specimen is needed. All of the cycles with a peak deformation below the yield deformation were performed using load control. Once the specimen reaches the yield load, deformation control is used exclusively. In this study, the specimen was loaded to a force, $Q$, equal to $0.5Q_y$ for the first three cycles, then the specimen was loaded to a force, $Q$, equal to $0.75Q_y$ for the second three cycles, where $Q_y$ is equal to the yield load of the connection. Once the specimen reached the yield load, the deformation was recorded. This then became the yield deformation, $\delta_y$, described in ATC-24. This deformation also becomes the $\Delta$ that the specimen is loaded in multiples of for the rest of the experiment.

The analytical calculation of the yield deflection needed for use of the ATC-24 loading protocol proved to be somewhat problematic. Therefore, the remaining four tests (F1-5/8-3/8-16 tests 1 and 2, and F2-5/8-3/8-16 tests 1 and 2) were performed using the loading protocol defined by the SAC Protocol for Fabrication, Inspection, Testing, and Documentation of Beam-Column Connection Tests and Other Experimental Specimens (1997). This will be hereafter defined as the SAC Protocol. (SAC is a joint venture involving the Structural Engineers Association of California (SEAOC), Applied Technology Council (ATC), and the California Universities for Research in Earthquake Engineering (CUREe)). Like the ATC-24 loading protocol, the SAC protocol is a multiple step test. The main difference between the ATC-24 and the SAC protocol is that
the SAC protocol is completely deformation controlled. The deformation parameter used to control the loading history is the interstory drift angle, $\theta$, defined as interstory displacement divided by the story height. In the test specimen, this angle is defined as the beam deflection divided by the beam span (to the column centerline) if the vertical beam deflection is controlled, or as the column deflection divided by the column height if the horizontal column deflection is controlled. As in the ATC-24 loading protocol, the cycles are symmetric in peak deformations. The history is divided into steps and the peak deformation of each step, $j$, is given as $\theta_j$, a predetermined value of the interstory drift angle. The SAC protocol proved far easier to use than the ATC-24 protocol because the values of $\theta_j$ remain the same for all experiments, making it unnecessary to calculate the “first yield” deflection of the specimen. The SAC protocol is shown graphically in Figure 3.5, and in tabular form in Table 3.2.

![Figure 3.5 SAC Loading Protocol](image-url)
### Table 3.2 SAC Loading Protocol

<table>
<thead>
<tr>
<th>Load Step #</th>
<th>Peak Deformation, $\theta$</th>
<th>Number of Cycles, $n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.00375</td>
<td>6</td>
</tr>
<tr>
<td>2</td>
<td>0.005</td>
<td>6</td>
</tr>
<tr>
<td>3</td>
<td>0.0075</td>
<td>6</td>
</tr>
<tr>
<td>4</td>
<td>0.01</td>
<td>4</td>
</tr>
<tr>
<td>5</td>
<td>0.015</td>
<td>2</td>
</tr>
<tr>
<td>6</td>
<td>0.02</td>
<td>2</td>
</tr>
<tr>
<td>7</td>
<td>0.03</td>
<td>2</td>
</tr>
</tbody>
</table>

Continue with increments in $\theta$ of 0.01 rad., and perform two cycles at each step

Results, including comparison of predicted and tested moment strengths, total and inelastic rotation capabilities, and condition at the end of the test are presented in Chapter IV. Also included is a comparison of the current study with previous research.
CHAPTER IV

EXPERIMENTAL RESULTS

4.1 TWO-BOLT FLUSH TESTS

4.1.1 COMPARISON OF PREDICTED AND ACTUAL MOMENT STRENGTHS

Four tests were conducted on two different flush two-bolt end-plate configurations. Two tests were performed on the two-bolt configuration with 3/4 in. bolts (F1-3/4-3/8-16), and two tests were performed on the two-bolt configuration with 5/8 in. bolts (F1-5/8-3/8-16). Gage and pitch for these connections are given in Table 3.1. A nominal design yield stress of 55 ksi was used for design of the connections. The connections were designed without weld access holes.

Quasi-static or “slow cyclic” loading was applied to the cantilevered beam using a hydraulic actuator. The loading history prescribed by ATC-24 and presented in Chapter III was used for the two F1-3/4-3/8-16 tests. The loading history prescribed by SAC and presented in Chapter III was used for the two F1-5/8-3/8-16 tests. Detailed presentations of the data for all of the tests are located in Appendix B.

The F1-3/4-3/8-16 tests used instrumented bolts, allowing them to be tightened to the minimum bolt tension of 28 kips as prescribed by the AISC LRFD Design Specification for Structural Steel Buildings (Load 1993). Instrumented bolts were not used for the F1-5/8-3/8-16 tests because the shanks of the instrumented bolts originally intended for use in the test were too long, requiring the use of an excessive amount of
washers. Uninstrumented bolts were used for these tests and tightened according to the “turn of the nut” method.

The maximum moment strength of the connections from the tests correlated poorly with the analytical strength predictions from yield-line analysis and bolt force predictions including prying action. The methods used for the analytical predictions are described in Chapter II. Comparison of predicted versus tested failure moments are given in Table 4.1. In Table 4.1, $M_{pl}$ is the moment strength of the connection calculated using yield line theory, and $M_q$ is the moment strength of the connection calculated using bolt force predictions including prying action. The shaded boxes are the predicted controlling limit state for the connection. The end-plate yield stresses, $(F_{py})$, from coupon tensile tests are shown in the last column. All yield stresses are approximately 53 ksi for the end-plates.

**Table 4.1** Two-Bolt Flush Test Results – Strength Data

<table>
<thead>
<tr>
<th>Connection</th>
<th>Test</th>
<th>Maximum Moment (ft-kips)</th>
<th>Predicted Strengths</th>
<th>Applied</th>
<th>$M_{pl}/M_{app}$</th>
<th>$M_q/M_{app}$</th>
<th>$F_{py}$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1-3/4-3/8-16</td>
<td>1$^1$</td>
<td>51.5</td>
<td>78.0</td>
<td>44.7</td>
<td>1.15</td>
<td>1.74</td>
<td>53.6</td>
</tr>
<tr>
<td>F1-3/4-3/8-16</td>
<td>2$^1$</td>
<td>50.0</td>
<td>78.3</td>
<td>44.5</td>
<td>1.12</td>
<td>1.76</td>
<td>52.6</td>
</tr>
<tr>
<td>F1-5/8-3/8-16</td>
<td>1$^2$</td>
<td>56.2</td>
<td>57.1</td>
<td>43.6</td>
<td>1.29</td>
<td>1.31</td>
<td>53.8</td>
</tr>
<tr>
<td>F1-5/8-3/8-16</td>
<td>2$^2$</td>
<td>60.4</td>
<td>58.0</td>
<td>45.3</td>
<td>1.33</td>
<td>1.28</td>
<td>53.5</td>
</tr>
</tbody>
</table>

1  ATC Loading Protocol used to load specimen.
2  SAC Loading Protocol used to load specimen.
The condition at the end of the tests are reported instead of the failure mode. Because the rotation applied to the connections in this study is far beyond what is normally required for other than earthquake loads, the failure mode or limit state does not have the same significance as for monotonic tests. For an end-plate connection, if enough rotation is applied, the connection will exhibit either bolt fractures, weld fractures or end-plate fractures. Because of this, the yield-line solution can not be used to predict the final condition of the specimen, it can only predict the moment strength of the connection.

The yield-line predicted strengths were calculated using actual measurements of gage and pitch of the connections and end-plate yield stress from tensile coupon tests. Nominal bolt material strength was used to determine $M_q$. The tested maximum moment was taken as the maximum moment at the end-plate that was resisted by the connection during the entire test. For F1-3/4-3/8-16 tests 1 and 2 the moment strength given by yield line theory is the controlling limit state but overpredicts by 15% and 12%, respectively. For F1-5/8-3/8-16 test 1 the moment strength given by yield line theory is the controlling limit state but is unconservative by 29%. For F1-5/8-3/8-16 test 2 the moment strength from bolt force predictions including prying action is the controlling limit state but is unconservative 28%.

### 4.1.2 Rotation Capability

Rotation results of the two-bolt tests are summarized in Table 4.2. All rotations given
In Table 4.2 are rotations at the end-plate. The number of completed cycles refers to the number of cycles of the loading protocol that were completed prior to termination of the test. Rotations were calculated by taking the deflection of the test beam at a point inboard 1 ft from the applied load, dividing that deflection by the distance to the face of the column and subtracting the measured rotation of the test column and the measured rigid body motion of the load frame. A schematic of the test set-up is shown in Figure 4.1. The net or reported rotation is shown schematically in Figure 4.2.

Inelastic rotation was calculated using a simple linear interpolation from the moment versus total rotation curve from the test. Two points were chosen on the unloading curve of one of the first few cycles. It is during these first cycles that the

<table>
<thead>
<tr>
<th>Connection</th>
<th>Test</th>
<th>Completed Cycles</th>
<th>Maximum Rotation</th>
<th>Condition at End of Test</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Total</td>
<td>Inelastic</td>
</tr>
<tr>
<td>F1-3/4-3/8-16</td>
<td>1</td>
<td>28\textsuperscript{1}</td>
<td>0.041</td>
<td>0.038</td>
</tr>
<tr>
<td>F1-3/4-3/8-16</td>
<td>2</td>
<td>17\textsuperscript{1}</td>
<td>0.045</td>
<td>0.041</td>
</tr>
<tr>
<td>F1-5/8-3/8-16</td>
<td>1</td>
<td>26\textsuperscript{2}</td>
<td>0.016</td>
<td>0.0093</td>
</tr>
<tr>
<td>F1-5/8-3/8-16</td>
<td>2</td>
<td>26\textsuperscript{2}</td>
<td>0.018</td>
<td>0.013</td>
</tr>
</tbody>
</table>

\textsuperscript{1} ATC Loading Protocol
\textsuperscript{2} SAC Loading Protocol
**Figure 4.1** Schematic of Test Setup

**Figure 4.2** Basis of Reported Rotation
connection exhibits completely elastic behavior. An equation was then written for the line joining these two points. This was used as the equation representing the elastic deformation of the connection. This allowed the elastic deformation of the connection to be calculated for any moment. The inelastic rotation, $\theta_i$, was then found by simply subtracting the calculated elastic rotation, $\theta_e$, from the total rotation, $\theta_t$, data from the test using the equation given below:

$$\theta_i = \theta_t - \theta_e$$

where $\theta_i$ is the inelastic rotation, $\theta_t$ is the total rotation, and $\theta_e$ is the calculated elastic rotation. Typical total and inelastic rotation curves are shown as Figures 4.3 and 4.4 respectively.

F1-3/4-3/8-16 was designed with a “wide” configuration and F1-5/8-3/8-16 was designed with a “tight” configuration. “Tight” configurations have smaller gages and pitches than “wide” configurations. This helps explain why both F1-3/4-3/8-16 tests exhibited much greater inelastic rotation capability than F1-5/8-3/8-16 tests 1 and 2. The wide configuration allows for a more flexible connection with more end-plate yielding. Bolt fracture occurred in F1-5/8-3/8-16 tests 1 and 2 without any significant end-plate yielding, while the F1-3/4-3/8-16 tests showed significant end-plate yielding. The tight configuration combined with the use of smaller bolts in the F1-5/8-3/8-16 tests led to less end-plate yielding, bolt fracture, and therefore, less inelastic rotation capability.
FIGURE 4.3  Typical Moment Versus Total Rotation Curve

FIGURE 4.4  Typical Moment Versus Inelastic Rotation Curve
4.1.3 CONDITION AT END OF TEST

The condition at the end of each test for the two-bolt connections is given in Table 4.2. The F1-3/4-3/8-16 connections had end-plate fractures, and bolt fractures occurred in the F1-5/8-3/8-16 tests.

All of the end-plate fractures occurred at the weld locations. This means that the end-plate cracked at the interface of the end-plate and the weld. There were no true fractures of the welds, that is, where cracks formed in the welds.

Specimen F1-3/4-3/8-16 test 1 at failure is shown in Figure 4.5. The deformation of the end-plate at the end of the test is shown in the photograph. The top and bottom flanges have “peeled” away from the face of the column and the end-plate has deformed significantly at the center. From the photograph it is difficult to see the end-plate fractures at the interface of the web-to-end-plate weld and the end-plate. They became more evident when the connection was disassembled and inspected. The fractures started at the inside of the flanges and emanated toward the center of the web, 3-3/8 in. from the top flange and 2-1/4 in. from the bottom flange. At the outer edge of the top and bottom flange, a crack formed at the interface of the flange and the flange-to-end-plate weld and the flange peeled away from the end-plate. This fracture is shown in Figure 4.6 as highlighted by the red oval. It is evident that a crack has formed and the flange has peeled away from the end-plate.

Specimen F1-3/4-3/8-16 test 2 at failure is shown in Figure 4.7. The photograph shows the deformation of the end-plate at the end of the test. As in F1-3/4-3/8-16 test 1, the top and bottom flanges have peeled away from the face of the column and the end-
plate has deformed significantly at the center. It is possible to see the end-plate fractures at the interface of the end-plate and the web-to-end-plate weld in the photograph. The fractures started at the inside of the flanges and emanated toward the center of the web, 3-3/8 in. from the top flange and 2-3/8 in. from the bottom flange. As in F1-3/4-3/8-16 test 1, cracks formed at the interfaces of the top and bottom flanges and flange-to-end-plate weld. This occurred at the edges of both flanges and the flanges “peeled” away from the flange-to-end-plate weld.

Bolt rupture occurred at the end of F1-5/8-3/8-16 tests 1 and 2. Both tests showed very little deformation of the end-plate prior to bolt fracture. The only noticeable yielding of the end-plate was around the bolts. This was noticed by flaking of the whitewash around the bolts.

![Figure 4.5 F1-3/4-3/8-16 Test 1 at End of Test](image-url)
**Figure 4.6** Fracture at Flange-to-End-Plate Interface – F1-3/4-3/8-16 Test 1

**Figure 4.7** F1-3/4-3/8-16 Test 2 at End of Test
In test 1, one bolt ruptured, and in test 2 both bolts ruptured, both at the tension side of the connection.

4.2 FOUR-BOLT FLUSH TESTS

4.2.1 COMPARISON OF PREDICTED AND ACTUAL MOMENT STRENGTHS

Three tests were conducted on two different flush four-bolt end-plate configurations. One test was performed on the four-bolt configuration with 3/4 in. bolts (F2-3/4-3/8-16), and two tests were performed on the four-bolt configuration with 5/8 in. bolts (F2-5/8-3/8-16). Gage and pitch for these connections are given in Table 3.1. A nominal design yield stress of 55 ksi was used for design of the connections. The connections were designed without weld access holes.

Quasi-static or “slow cyclic” loading was applied to the cantilevered beam using a hydraulic actuator. The loading history prescribed by ATC-24 and presented in Chapter III was used for the F2-3/4-3/8-16 test. The loading history prescribed by SAC and presented in Chapter III was used for the second two tests, namely F2-5/8-3/8-16 tests 1 and 2. Detailed presentations of the data for all of the tests are located in Appendix C.

The F2-3/4-3/8-16 test used instrumented bolts, allowing them to be tightened to the minimum bolt tension of 28 kips prescribed by the AISC LRFD Design Specification for Structural Steel Buildings (Load 1993). Instrumented bolts were not used for the F2-5/8-3/8-16 tests. This was because the shanks of the instrumented bolts originally intended for use in the test were too long, requiring the use of an excessive amount of
washers. Uninstrumented bolts were used for these tests and tightened according to the “turn of the nut” method.

Comparison of predicted versus tested maximum moments are given in Table 4.2. As in Table 4.1, $M_{pl}$ is the moment strength of the connection calculated using yield line theory, and $M_{q}$ is the moment strength of the connection calculated using bolt force predictions including prying action. The end-plate yield stresses, ($F_{py}$), from coupon tensile tests are shown in the last column. All yield stresses were approximately 53 ksi for the end-plates.

**Table 4.3 Four-Bolt Flush Test Results – Strength Data**

<table>
<thead>
<tr>
<th>Connection</th>
<th>Test</th>
<th>Maximum Moment (ft-kips)</th>
<th>Predicted Strengths</th>
<th>Applied</th>
<th>$M_{pl}/M_{app}$</th>
<th>$M_{q}/M_{app}$</th>
<th>$F_{py}$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$M_{pl}$</td>
<td>$M_{q}$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>F2-3/4-3/8-16</td>
<td>1</td>
<td>65.1</td>
<td>104.6</td>
<td>66.8</td>
<td>0.97</td>
<td>1.57</td>
<td>52.7</td>
</tr>
<tr>
<td>F1-5/8-3/8-16</td>
<td>1</td>
<td>73.0</td>
<td>76.8</td>
<td>67.7</td>
<td>1.08</td>
<td>1.13</td>
<td>53.0</td>
</tr>
<tr>
<td>F1-5/8-3/8-16</td>
<td>2</td>
<td>73.0</td>
<td>76.8</td>
<td>67.9</td>
<td>1.10</td>
<td>1.13</td>
<td>53.0</td>
</tr>
</tbody>
</table>

1  ATC Loading Protocol used to load specimen.
2  SAC Loading Protocol used to load specimen.

The yield-line predicted strengths were calculated using actual measurements of gage and pitch of the connections and end-plate yield stress from tensile coupon tests.
Nominal bolt material strength was used to determine $M_q$. The tested maximum moment was taken as the maximum moment at the end-plate that was resisted by the connection during the entire test. For all three tests, the predicted limit state was from the yield-line analysis of the end-plate. The correlation between the predicted and tested moment strengths for the four-bolt connections was slightly better than for the two-bolt connections. The moment strength calculated using yield-line theory was conservative by 3% for F2-3/4-3/8-16 and, for F2-5/8-3/8-16 tests 1 and 2, overpredicted by 8% and 10% respectively.

4.2.2 Rotation Capability

Results for rotation of the four-bolt tests are given in Table 4.4. All rotations given in Table 4.4 are rotations at the end-plate. The number of completed cycles refers to the number of cycles of the loading protocol that were completed prior to termination of the test.

**Table 4.4** Four Bolt Flush Test Results – Rotation Data

<table>
<thead>
<tr>
<th>Connection</th>
<th>Test</th>
<th>Completed Cycles</th>
<th>Maximum Rotation</th>
<th>Condition at End of Test</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Total</td>
<td>Inelastic</td>
</tr>
<tr>
<td>F2-3/4-3/8-16</td>
<td>1</td>
<td>12(^1)</td>
<td>0.052</td>
<td>0.048</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>End-Plate and Weld Fractures</td>
</tr>
<tr>
<td>F2-5/8-3/8-16</td>
<td>1</td>
<td>26(^2)</td>
<td>0.028</td>
<td>0.020</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Bolt Fracture</td>
</tr>
<tr>
<td>F2-5/8-3/8-16</td>
<td>2</td>
<td>28(^2)</td>
<td>0.033</td>
<td>0.018</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Bolt Fracture</td>
</tr>
</tbody>
</table>

1 ATC Loading Protocol
2 SAC Loading Protocol
Rotations were calculated in the same manner as the two-bolt tests, by taking the
deflection of the test beam at a point inboard 1 ft from the applied load, dividing that
deflection by the distance to the face of the column and subtracting the rotation of the test
column and the rigid body motion of the load frame. The inelastic rotation was
calculated using the same method that was used for the two-bolt tests.

F2-3/4-3/8-16 was designed with a “wide” configuration and F2-5/8-3/8-16 was
designed with a “tight” configuration. This helps explain why the F2-3/4-3/8-16
connection exhibited greater inelastic rotation capability than the F2-5/8-3/8-16
connection. The wide configuration allows for a more flexible connection with more
end-plate yielding. Bolt fractures occurred in both F2-5/8-3/8-16 tests 1 and 2 without
any significant end-plate yielding, while the F2-3/4-3/8-16 connection showed significant
end-plate yielding. The tight configuration combined with the use of smaller bolts in the
F2-5/8-3/8-16 tests led to less end-plate yielding, bolt fracture, and therefore, less
inelastic rotation capability.

4.2.3 CONDITION AT END OF TEST

The condition at the end of the test for each four-bolt connection is given in Table
4.4. End-plate and weld fractures occurred in test F2-3/4-3/8-16. Bolt fracture occurred
in both F2-5/8-3/8-16 tests 1 and 2.

In the F2-3/4-3/8-16 test, the end-plate fractured at the interface of the end-plate
and the extra web-to-end-plate weld at the bottom flange. A photograph of the
connection at the end of the test is shown in Figure 4.8. The extra web-to-end-plate weld fractured on the same side as the previously described end-plate fracture and is shown in Figure 4.9 (Extra web-to-end-plate weld refers to the fabrication where one side of the web is welded to the end-plate the full depth of the web and the other side is only welded approximately 6 in. from the inside of the flanges toward the center of the web). Cracks were also found in the flange-to-end-plate welds, and at the bottom flange, the flange had completely torn away from the end-plate on the same side that the end-plate fracture occurred as shown in Figure 4.10.

Bolt fracture occurred in both F2-5/8-3/8-16 tests with very little deformation of the end-plate. The only noticeable yielding of the end-plate was around the bolts, as evidenced by flaking of the whitewash around the bolts.

**Figure 4.8** Web-to-End-Plate Weld Fracture – F2-3/4-3/8-16
**Figure 4.9** End-Plate Fracture – F2-3/4-3/8-16

**Figure 4.10** Flange-to-End-Plate Weld Fracture – F2-3/4-3/8-16
4.3 Comparison With Previous Monotonic Load Tests

The results of this study are now compared to results from studies performed by Srouji et al. (1983) and Kline et al. (1989). Predicted moment strengths and actual tested moment strengths for the three studies are presented in Table 4.5. It is evident that the results of this study do not correlate well with the results from the previous two studies.

Current predictions for maximum strength of the connections correlate well with previous results, however, current maximum applied moments are much lower than those from Srouji et al. and Kline et al.. The ratios of predicted moment strength and maximum applied moment ($M_{\text{pred}}/M_{\text{app}}$) are all near 1.0 for the two previous studies, while they are larger in the current study.

One possible explanation for the unconservative results of the current study is an out-of-calibration load cell used to measure the applied load. To test this hypothesis, moment versus deflection curves were compared with those from Srouji et al. (1983). Monotonic moment versus deflection curves were created from the cyclic tests in this study by taking points at maximum moment for each load step in the loading histories. A typical moment versus total rotation curve for the current study is shown in Figure 4.11. The darkened line shows the monotonic moment versus rotation curve for the test. The rotation can then be multiplied by any distance to obtain a deflection at the corresponding moment.
### TABLE 4.5 Comparison of Current Results With Srouji (1983), and Kline (1989) Prior to Adjustment

<table>
<thead>
<tr>
<th>Connection</th>
<th>Test</th>
<th>Current</th>
<th>Srouji</th>
<th>Kline</th>
<th>Current</th>
<th>Srouji</th>
<th>Kline</th>
<th>Current</th>
<th>Srouji</th>
<th>Kline</th>
<th>M_{pred}/M_{app}</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1-3/4-3/8-16</td>
<td>1</td>
<td>51.5</td>
<td>54.3</td>
<td>62.0</td>
<td>44.7</td>
<td>54.0</td>
<td>60.0</td>
<td>1.15</td>
<td>1.01</td>
<td>1.03</td>
<td></td>
</tr>
<tr>
<td>F1-3/4-3/8-16</td>
<td>2</td>
<td>50.0</td>
<td>54.3</td>
<td>62.0</td>
<td>44.5</td>
<td>54.0</td>
<td>60.0</td>
<td>1.12</td>
<td>1.01</td>
<td>1.03</td>
<td></td>
</tr>
<tr>
<td>F1-5/8-3/8-16</td>
<td>1</td>
<td>56.2</td>
<td>62.0</td>
<td>67.4</td>
<td>43.6</td>
<td>64.8</td>
<td>57.0</td>
<td>1.29</td>
<td>0.96</td>
<td>1.18</td>
<td></td>
</tr>
<tr>
<td>F1-5/8-3/8-16</td>
<td>2</td>
<td>60.4</td>
<td>62.0</td>
<td>67.4</td>
<td>45.3</td>
<td>64.8</td>
<td>57.0</td>
<td>1.33</td>
<td>0.96</td>
<td>1.18</td>
<td></td>
</tr>
<tr>
<td>F2-3/4-3/8-16</td>
<td>1</td>
<td>65.1</td>
<td>68.8</td>
<td>68.2</td>
<td>66.8</td>
<td>73.2</td>
<td>70.0</td>
<td>0.97</td>
<td>0.94</td>
<td>0.97</td>
<td></td>
</tr>
<tr>
<td>F2-5/8-3/8-16</td>
<td>1</td>
<td>73.0</td>
<td>81.6</td>
<td>83.0</td>
<td>67.7</td>
<td>85.5</td>
<td>91.0</td>
<td>1.08</td>
<td>0.95</td>
<td>0.91</td>
<td></td>
</tr>
<tr>
<td>F2-5/8-3/8-16</td>
<td>2</td>
<td>73.0</td>
<td>81.6</td>
<td>83.0</td>
<td>67.9</td>
<td>85.5</td>
<td>91.0</td>
<td>1.08</td>
<td>0.95</td>
<td>0.91</td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.15</td>
<td>0.97</td>
<td>1.03</td>
<td></td>
</tr>
<tr>
<td>Standard Deviation</td>
<td></td>
<td>0.117</td>
<td>0.025</td>
<td>0.105</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
The rotations from the monotonic moment versus rotation curves for this study are multiplied by a distance to make the deflections obtained analogous to those from Srouji et al. This method was used to create the moment versus deflection curves used to compare the initial stiffnesses of the tests in the current study with those from Srouji et al. Before the connections yield, the initial stiffness of the connections in the current study and Srouji et al. should be very close. The initial stiffness is the relationship between moment and deflection while the connection is still behaving elastically. Theoretically, there should be no difference between the initial stiffness of the connections in the

**Figure 4.11** Typical Moment Versus Total Rotation Curve With Superimposed Monotonic Curve
current study and the initial stiffness of the connections in the previous study (Srouji et al. 1983).

The initial stiffnesses of the connections from the current study did not correlate well with those from Srouji et al. Because all comparisons are about the same, an adjustment was made in attempt to match the initial stiffnesses of current results with Srouji et al. A common factor of 1.2 was found and adjustments were made to the current results. The monotonic moment versus deflection curves for the current study were again compared with Srouji et al., this time multiplying the moments from the current study by 1.2. The factor of 1.2 was found to produce the best correlation between the current study and Srouji et al.. The monotonic moment versus deflection curves for the current study and Srouji et al. are shown as Figures 4.12 and 4.13 for F1-3/4-3/8-16 tests 1 and 2 respectively, Figures 4.14 and 4.15 for F1-5/8-3/8-16 tests 1 and 2 respectively, Figure 4.16 for F2-3/4-3/8-16, and Figures 4.17 and 4.18 for F2-5/8-3/8-16 tests 1 and 2 respectively. The adjusted curves correlate well with those developed by Srouji et al..

Table 4.6 contains the comparison of predicted yield-line moment strengths and adjusted moment strengths for the three studies.
FIGURE 4.12 Comparison of Srouji (1983) and Adjusted F1-3/4-3/8-16 Test 1 Results

FIGURE 4.13 Comparison of Srouji (1983) and Adjusted F1-3/4-3/8-16 Test 2 Results
FIGURE 4.14 Comparison of Srouji (1983) and Adjusted F1-5/8-3/8-16 Test 1 Results

FIGURE 4.15 Comparison of Srouji (1983) and Adjusted F1-5/8-3/8-16 Test 2 Results
FIGURE 4.16 Comparison of Srouji (1983) and Adjusted F2-3/4-3/8-16 Results

FIGURE 4.17 Comparison of Srouji (1983) and Adjusted F2-5/8-3/8-16 Test 2 Results
Figure 4.18 Comparison of Srouji (1983) and Adjusted F2-5/8-3/8-16 Test 2 Results
**Table 4.6** Comparison of Results With Srouji (1983), and Kline (1989) After Adjustment

<table>
<thead>
<tr>
<th>Connection</th>
<th>Test</th>
<th>Predicted Strengths</th>
<th>Applied</th>
<th>(\frac{M_{\text{pred}}}{M_{\text{app}}})</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Current Adjusted</td>
<td>Srouji</td>
<td>Kline</td>
</tr>
<tr>
<td>F1-3/4-3/8-16</td>
<td>1</td>
<td>51.5</td>
<td>54.3</td>
<td>62.0</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>50.0</td>
<td>54.3</td>
<td>62.0</td>
</tr>
<tr>
<td>F1-5/8-3/8-16</td>
<td>1</td>
<td>56.2</td>
<td>62.0</td>
<td>67.4</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>60.4</td>
<td>62.0</td>
<td>67.4</td>
</tr>
<tr>
<td>F2-3/4-3/8-16</td>
<td>1</td>
<td>65.1</td>
<td>68.8</td>
<td>68.2</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>73.0</td>
<td>81.6</td>
<td>83.0</td>
</tr>
<tr>
<td>F2-5/8-3/8-16</td>
<td>1</td>
<td>73.0</td>
<td>81.6</td>
<td>83.0</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>73.0</td>
<td>81.6</td>
<td>83.0</td>
</tr>
</tbody>
</table>

Average: 0.96, 0.97, 1.03

Standard Deviation: 0.097, 0.025, 0.105
The moment strengths adjusted by a factor of 1.2 correlated much better with past results as Table 4.6 shows. Tables 4.5 and 4.6 show the average and standard deviation of $\frac{M_{\text{pred}}}{M_{\text{app}}}$ for all three studies. The averages of $\frac{M_{\text{pred}}}{M_{\text{app}}}$ for Srouji et al. and Kline et al. are 0.97 and 1.03, respectively, and the standard deviations are 0.025 and 0.105, respectively. Prior to adjustment, the average and standard deviation of $\frac{M_{\text{pred}}}{M_{\text{app}}}$ for the current study were 1.15 and 0.117, respectively. After adjustment the average and standard deviation of $\frac{M_{\text{pred}}}{M_{\text{app}}}$ for the current study are 0.96 and 0.097, respectively. The average of 0.96 for adjusted values of $\frac{M_{\text{pred}}}{M_{\text{app}}}$ for the current study correlates very well with averages of 0.97 for Srouji et al. and 1.03 for Kline et al.. The standard deviation of 0.097 for the adjusted values of $\frac{M_{\text{pred}}}{M_{\text{app}}}$ for the current study falls within the range of standard deviations for the previous studies (0.096 (Srouji) > 0.097 > 1.03 (Kline)). This shows the scatter of the adjusted data to be in an acceptable range. Unfortunately, the load cell was damaged beyond repair during an experiment following the completion of this study making a re-calibration impossible.

The adjustment of the moments for the current study had no affect on the total and inelastic rotation capabilities of the specimens. This occurs because the rotations remain unchanged even though the moments are increased.
CHAPTER V

SUMMARY AND CONCLUSIONS

5.1 Summary

An experimental investigation was conducted to evaluate the inelastic rotation capability of flush end-plate moment connections under seismic loading. The inelastic rotation of fully restrained connections in a steel moment frame during an earthquake is used to dissipate the energy added to the structural system by earthquake ground motions. Current design recommendations require steel moment connections in a seismic resisting moment frame to resist a predetermined amount of inelastic rotation depending on the classification of the moment frame. The three classifications of steel moment frames are special, intermediate, and ordinary as discussed in Chapter I. The inelastic rotation capability of the connection determines in which of the three types of steel moment frames that the connection may be used.

Seven specimens representing four different flush end-plate configurations were tested. The specimens were tested using slow cyclic or “quasi-static” loading. Two different loading protocols were used in the study. Three tests were performed using the loading protocol given in ATC-24 (1992), and four tests were performed using the loading protocol given by SAC (1997). The connections were loaded using one of these two loading protocols until additional load could not be applied.

The specimens were designed using yield-line theory to predict end-plate strength and maximum bolt force predictions including prying action. The methods used to
design the connections are presented in Chapter II. All connections were designed so that the test beam was stronger than the connection elements, forcing the failure mechanism to form in the connection elements. This proved successful as no failures occurred in the test beams. End-plate fracture, weld fracture, or bolt fracture were observed at the end of the tests.

Initial results for strength and stiffness did not correlate well with analytical predictions or previous research. A possible explanation for this is a load cell calibration problem. Using an adjustment factor of 1.2 the maximum moments applied to the specimens were recomputed. The average of the ratios of the adjusted \( \frac{M_{\text{pred}}}{M_{\text{app}}} \) for this study is 0.96 with a standard deviation of 0.097. These adjusted results correlated well with analytical predictions and previous research.

5.2 CONCLUSIONS

A summary of the inelastic rotation capabilities of the connections tested in this study are presented in Table 5.1 along with the type of steel moment frame in which they may be used. All of the connections with the exception of F1-5/8-3/8-16 test 1 exhibited inelastic rotations greater than the 0.01 radians which is necessary for use in ordinary steel moment frames. The two-bolt and four-bolt configurations were designed with one “tight” and one “wide” configuration each. A tight configuration is one with small gages and pitches while a wide configuration is one with larger gages and pitches. The tight configurations were F1-5/8-3/8-16 and F2-5/8-3/8-16 and the wide configurations were F1-3/4-3/8-16 and F2-3/4-3/8-16. It was found that the connections with a wide
connection allowed for more yielding of the end-plate during testing. These connections exhibited nearly double the inelastic rotation capability than their tight counterparts.

The inelastic rotation of these connections was achieved through the yielding of the connection elements while the beam remained undamaged. The AISC seismic specification (*Seismic 1997*) mandates that fully restrained connections in special and intermediate moment frames be designed to have at least the same moment strength at the column face as the plastic moment strength of the beam.

<table>
<thead>
<tr>
<th>Connection</th>
<th>Test</th>
<th>Inelastic Rotation (radians)</th>
<th>Connection Configuration (tight or wide)</th>
<th>*Type of Moment Frame for Which Connection is Qualified as Defined by AISC (<em>Seismic 1997</em>)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1-3/4-3/8-16</td>
<td>1</td>
<td>0.038</td>
<td>Wide</td>
<td>Special ($\theta_i &gt; 0.03$)</td>
</tr>
<tr>
<td>F1-3/4-3/8-16</td>
<td>2</td>
<td>0.041</td>
<td>Wide</td>
<td>Special ($\theta_i &gt; 0.03$)</td>
</tr>
<tr>
<td>F1-5/8-3/8-16</td>
<td>1</td>
<td>0.0093</td>
<td>Tight</td>
<td>None</td>
</tr>
<tr>
<td>F1-5/8-3/8-16</td>
<td>2</td>
<td>0.013</td>
<td>Tight</td>
<td>Ordinary ($\theta_i &gt; 0.01$)</td>
</tr>
<tr>
<td>F2-3/4-3/8-16</td>
<td>1</td>
<td>0.048</td>
<td>Wide</td>
<td>Special ($\theta_i &gt; 0.03$)</td>
</tr>
<tr>
<td>F2-5/8-3/8-16</td>
<td>1</td>
<td>0.020</td>
<td>Tight</td>
<td>Intermediate ($\theta_i &gt; 0.02$)</td>
</tr>
<tr>
<td>F2-5/8-3/8-16</td>
<td>2</td>
<td>0.018</td>
<td>Tight</td>
<td>Ordinary ($\theta_i &gt; 0.01$)</td>
</tr>
</tbody>
</table>

*Inelastic Rotation Requirements are Shown in Parentheses*
Partially restrained connections, however, may achieve the required inelastic rotation through the yielding of the connection elements. This requirement was written with fully welded connections in mind and is conservative for bolted connections. However, no exception for bolted connections is given in the specification. Therefore, by the definitions outlined in the AISC seismic specification, the connections tested can only be used as partially restrained connections in special and intermediate moment frames in seismic areas. However, the tested connections are permitted for use as fully restrained connections in ordinary moment frames because in ordinary moment frames the connection shall be designed for a flexural strength equal to $1.1R_yM_p$ of the beam or the maximum moment delivered to the system, whichever is less.

Two (F1-3/4-3/8-16 and F2-3/4-3/8) of the four connections tested in this study satisfy the requirements for use as fully restrained connections in ordinary moment frames in seismic areas. Because the AISC seismic specification (Seismic 1997) requires that two full scale tests be done on the same connection configuration, F1-3/4-3/8-16 cannot be approved immediately for use in ordinary moment frames. The test performed in this study shows, however, that this connection exhibits more than enough inelastic rotation capability to be used in ordinary moment frames. Two tests were performed on F1-5/8-16 with test 1 exhibiting enough inelastic rotation to be used in ordinary moment frames while F1-5/8-3/8-16 test 2 did not. At this time the results for F1-5/8-3/8-16 are inconclusive. Therefore it is recommended that further study be performed on this connection configuration.
Obviously due to cost and time constraints not every connection size and configuration can be tested under seismic loading. Therefore AISC allows interpolation and extrapolation from test data. AISC allows results of qualifying test to be extrapolated for beam depths up to 10% greater than the tested beam depths. The beam depths in this study were rather shallow, allowing the results of this study to be extrapolated for beam depths of up to $16/0.9$ or approximately 18 in.
REFERENCES


“Guidelines for Cyclic Seismic Testing of Components of Steel Structures (ATC-24)” (1992) Applied Technology Council, Redwood City, CA


APPENDIX A

NOMENCLATURE
NOMENCLATURE

\( a \) - distance from bolt line to location of prying action

AISC - American Institute of Steel Construction

ATC - Applied Technology Council

\( B \) - bolt force

\( B_1 \) - outer bolt force

\( B_2 \) - inner bolt force

\( b_f \) - beam flange width

\( ds \) - elemental length of line \( n \)

\( E \) - Young’s Modulus of Elasticity

\( F \) - flange force per bolt

\( F_f \) - total flange force

\( F_{py} \) - end-plate material yield stress

\( F_{yb} \) - yield stress of the bolt

\( g \) - end-plate bolt gage distance

\( h \) - beam depth

\( L_n \) - length of yield line \( n \)

\( M_1 \) - plastic moment at first hinge line to form

\( M_2 \) - plastic moment at second hinge line to form

\( M_3 \) - plastic moment at third hinge to form

\( M_{app} \) - applied maximum moment

\( M_b \) - moment strength of the bolt

\( m_p \) - plastic moment capacity of plate per unit length equal to \( (F_{py} t_p^2)/4 \)

\( M_{pl} \) - moment strength of the connection calculated using yield-line theory

\( M_{pred} \) - predicted moment strength
\(M_q\) - moment strength of the connection calculated using maximum bolt force predictions

\(M_{U}\) - end-plate ultimate moment capacity at beam end

\(N\) - number of yield lines in a mechanism

\(n_j\) - the number of cycles to be performed in load step \(j\)

\(p_b\) - distance between centerline of upper and lower rows of bolts

\(P_{ext}\) - distance from face of beam flange to end of end-plate in Kennedy model

\(p_f\) - distance from bolt centerline to near face of beam flange

\(p_t\) - distance from bolt centerline to far face of beam flange

\(Q\) - prying force

\(Q_{\text{max}}\) - maximum prying force

\(s\) - distance from bolt centerline to outermost yield-line

SAC - joint venture involving: the Structural Engineers Association of California (SEAOC), Applied Technology Council (ATC), and the California Universities for Research in Earthquake Engineering (CUREe)

\(t_1\) - thick plate limit

\(t_{11}\) - thin plate limit

\(t_p\) - end-plate thickness

\(w'\) - width of end-plate per bolt at bolt line minus bolt hole diameter

\(W_e\) - total external work done on the connection

\(W_i\) - total internal energy stored

\(\delta\) - deformation, a generic quantity including strains, angles of shear distortion, rotations, axial deformations and displacements

\(\delta_y\) - yield deformation

\(\theta\) - interstory drift angle

\(\theta_e\) - elastic rotation

\(\theta_i\) - inelastic rotation
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\theta_j$</td>
<td>interstory drift at load step $j$ in SAC protocol</td>
</tr>
<tr>
<td>$\theta_j$</td>
<td>the peak deformation in load step $j$</td>
</tr>
<tr>
<td>$\theta_n$</td>
<td>relative normal rotation of yield-line $n$</td>
</tr>
<tr>
<td>$\theta_t$</td>
<td>total rotation</td>
</tr>
</tbody>
</table>
APPENDIX B

TWO-BOLT FLUSH RESULTS AND TEST DATA
Summary
F1-3/4-3/8-16
Test 1

Predicted Capacities:
Moment Strength Predicted by Yield Line Analysis........51.5 ft-kips
Moment Strength Predicted by Bolt Rupture...............78.0 ft-kips

Maximum Moment From Test.................................44.7 ft-kips

Maximum Total Rotation.................................0.041 rad
Maximum Inelastic Rotation.........................0.038 rad

Number of Completed Cycles..............................28

Failure mode..................................................End-plate fractures
Fractures that occurred during test:
• Top flange-to-end-plate weld, 1-3/8 in. in length on right side of the web.
• Bottom flange-to-end-plate weld 1-1/4 in. in length on left side of the web.
End-Plate fractures that occurred during test:
• At web-to-end-plate weld 3-3/8 in. in length beginning at the top flange.
• At web-to-end-plate weld 2-1/4 in. in length beginning at the bottom flange.
### Load and Displacement History

**F1-3/4-3/8-16 Test 1**

<table>
<thead>
<tr>
<th>Load Step #</th>
<th>Maximum Load (kips)</th>
<th>Peak Deformation, $\theta$</th>
<th>Number of Cycles, $n$</th>
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<tbody>
<tr>
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<td>3</td>
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<td>4</td>
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<td>2</td>
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<td>*</td>
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<td>*</td>
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<td>10</td>
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<td>0.044</td>
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<tr>
<td>11</td>
<td>*</td>
<td>0.0465</td>
<td>2</td>
</tr>
</tbody>
</table>

*Note: Loading switched from load to deflection control at the fourth load step.*

### ATC Loading Protocol Used

### Material Testing Results

**F1-3/4-3/8-16 Test 1**

<table>
<thead>
<tr>
<th>Location</th>
<th>% Elongation</th>
<th>$F_y$ (ksi)</th>
<th>$F_u$ (ksi)</th>
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</thead>
<tbody>
<tr>
<td>End-Plate</td>
<td>23.44</td>
<td>53.61</td>
<td>83.94</td>
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<tr>
<td>Beam Flange</td>
<td>22.67</td>
<td>60.59</td>
<td>85.16</td>
</tr>
<tr>
<td>Beam Web</td>
<td>21.32</td>
<td>73.77</td>
<td>82.74</td>
</tr>
<tr>
<td>Column Flange</td>
<td>24.92</td>
<td>57.61</td>
<td>85.80</td>
</tr>
<tr>
<td>Column Web</td>
<td>12.70</td>
<td>72.08</td>
<td>84.16</td>
</tr>
</tbody>
</table>
F1-3/4-3/8-16 Test 1 Connection Details
Moment at End-Plate vs. Total Rotation at End-Plate
F1-3/4-3/8-16 Test 1

Moment At End-Plate vs. Inelastic Rotation at End-Plate
F1-3/4-3/8 Test 1
Measured Bolt Strain $X E$ vs. Flange Force - Bolt 1
F1-3/4-3/8-16 Test 1

- Cycles 1-10
- Cycles 11-13
- Cycles 14-16
- Cycles 17-19
- Cycles 19-28

Measured Bolt Strain $X E$ vs. Flange Force - Bolt 2
F1-3/4-3/8-16 Test 1

- Cycles 1-10
- Cycles 11-13
- Cycles 14-16
- Cycles 17-19
- Cycles 19-28
Moment At End-Plate vs. Total Rotation at End-Plate

**F1-3/4-3/8-16 Test 1**

Yield Line Moment = 51.5 ft-kips

Moment Due to Bolt Rupture = 78.0 ft-kips
Summary
F1-3/4-3/8-16
Test 2

Predicted Capacities:
Moment Strength Predicted by Yield Line Analysis…………50.0 ft-kips
Moment Strength Predicted by Bolt Rupture…………………78.3 ft-kips

Maximum Moment From Test………………………………..44.5 ft-kips

Maximum Total Rotation……………………………………..0.045 rad
Maximum Inelastic Rotation………………………………….0.041 rad

Number of Completed Cycles………………………………...17

Failure mode…………………………………………………..End-plate fractures
Fractures that occurred during test:
• Top flange-to-end-plate weld on both sides of the web.
End-Plate fractures that occurred during test:
• At web-to-end-plate weld 3-3/8 in. in length beginning at the top flange.
• At web-to-end-plate weld 2-3/8 in. in length beginning at the bottom flange.
Load and Displacement History
F1-3/4-3/8-16 Test 2

<table>
<thead>
<tr>
<th>Load Step #</th>
<th>Maximum Load (kips)</th>
<th>Peak Deformation, $\theta$</th>
<th>Number of Cycles, $n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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<td>*</td>
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</tr>
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<td>2</td>
<td>4</td>
<td>*</td>
<td>3</td>
</tr>
<tr>
<td>3</td>
<td>6</td>
<td>*</td>
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<tr>
<td>4</td>
<td>*</td>
<td>0.022</td>
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<tr>
<td>6</td>
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</tbody>
</table>

*Note: Loading switched from load to deflection control at the fourth load step.
ATC Loading Protocol Used

Material Testing Results
F1-3/4-3/8-16 Test 2

<table>
<thead>
<tr>
<th>Location</th>
<th>% Elongation</th>
<th>$F_y$ (ksi)</th>
<th>$F_u$ (ksi)</th>
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</thead>
<tbody>
<tr>
<td>End-Plate</td>
<td>23.44</td>
<td>52.62</td>
<td>82.71</td>
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<tr>
<td>Beam Flange</td>
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<td>60.59</td>
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<td>Beam Web</td>
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<td>73.77</td>
<td>82.74</td>
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<tr>
<td>Column Flange</td>
<td>24.92</td>
<td>57.61</td>
<td>85.80</td>
</tr>
<tr>
<td>Column Web</td>
<td>12.70</td>
<td>72.08</td>
<td>84.16</td>
</tr>
</tbody>
</table>

Listing of Observations Made During Test

1. Flange-to-end-plate weld fracture noticed at cycle 17. Test Terminated
F1-3/4-3/8-16 TEST 2 CONNECTION DETAILS
Moment vs. Total Rotation at End-Plate
F1-3/4-3/8-16 Test 2

Moment At End-Plate vs. Inelastic Rotation at End-Plate
F1-3/4-3-8 Test 2
Moment At End-Plate vs. End-Plate Separation at Top Flange
F1-3/4-3/8-16 Test 2

Moment At End-Plate vs. End-Plate Separation at Bottom Flange
F1-3/4-3/8-16 Test 2
Moment At End-Plate vs. Total Rotation at End-Plate

F1-3/4-3/8-16 Test 2

Moment (ft-kips)

Rotation (radians)

Test
Prediction
Yield Line Moment=50.0 ft-kips
Moment Due to Bolt Rupture=78.3 ft-kips
Summary
F1-5/8-3/8-16
Test 1

Predicted Capacities:
Moment Strength Predicted by Yield Line Analysis.........56.2 ft-kips
Moment Strength Predicted by Bolt Rupture..................57.1 ft-kips

Maximum Moment From Test................................43.6 ft-kips

Maximum Total Rotation......................................0.016 rad
Maximum Inelastic Rotation.................................0.0093 rad

Number of Completed Cycles................................26

Failure mode.................................................Bolt Fracture
Top left bolt fractured during cycle 27
Displacement History
F1-5/8-3/8-16 Test 1

<table>
<thead>
<tr>
<th>Load Step #</th>
<th>Peak Deformation, θ</th>
<th>Number of Cycles, n</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.00375</td>
<td>6</td>
</tr>
<tr>
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<tr>
<td>3</td>
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<td>2</td>
</tr>
<tr>
<td>7</td>
<td>0.03</td>
<td>1/4</td>
</tr>
</tbody>
</table>

Material Testing Results
F1-5/8-3/8-16 Test 1

<table>
<thead>
<tr>
<th>Location</th>
<th>% Elongation</th>
<th>F_y (ksi)</th>
<th>F_u (ksi)</th>
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</thead>
<tbody>
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<td>53.79</td>
<td>84.65</td>
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<td>Beam Flange</td>
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<td>60.19</td>
<td>86.16</td>
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<tr>
<td>Beam Web</td>
<td>20.28</td>
<td>73.86</td>
<td>82.63</td>
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<td>24.14</td>
<td>57.77</td>
<td>85.21</td>
</tr>
<tr>
<td>Column Web</td>
<td>14.68</td>
<td>72.35</td>
<td>84.67</td>
</tr>
</tbody>
</table>

Listing of Observations Made During Test

1. Yielding of end-plate around bolts noticed during cycle 23.

2. At maximum positive moment of cycle 27, the top left bolt (i.e. the top left bolt at the tension flange) fractured. Test was terminated.
F1-5/8-3/8-16 TEST 1 CONNECTION DETAILS
Moment At End-Plate vs. Total Rotation at End-Plate

Test
Prediction
Yield Line Moment=56.2 ft-kips
Moment Due to Bolt Rupture=57.1 ft-kips
Summary
F1-5/8-3/8-16
Test 2

Predicted Capacities:
Moment Strength Predicted by Yield Line Analysis.........60.4 ft-kips
Moment Strength Predicted by Bolt Rupture...............58.0 ft-kips

Maximum Moment From Test.................................45.3 ft-kips

Maximum Total Rotation......................................0.018 rad
Maximum Inelastic Rotation.................................0.013 rad

Number of Completed Cycles.................................26

Failure mode....................................................Bolt Fracture
Top two bolts fractured during cycle 27
Displacement History
F1-5/8-3/8-16 Test 2

<table>
<thead>
<tr>
<th>Load Step #</th>
<th>Peak Deformation, $\theta$</th>
<th>Number of Cycles, n</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.00375</td>
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<tr>
<td>2</td>
<td>0.005</td>
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<td>0.0075</td>
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<td>5</td>
<td>0.015</td>
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<tr>
<td>6</td>
<td>0.02</td>
<td>2</td>
</tr>
<tr>
<td>7</td>
<td>0.03</td>
<td>1/4</td>
</tr>
</tbody>
</table>

Material Testing Results
F1-5/8-3/8-16 Test 2

<table>
<thead>
<tr>
<th>Location</th>
<th>% Elongation</th>
<th>$F_y$ (ksi)</th>
<th>$F_u$ (ksi)</th>
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</thead>
<tbody>
<tr>
<td>End-Plate</td>
<td>22.66</td>
<td>53.50</td>
<td>83.94</td>
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<td>Beam Flange</td>
<td>22.26</td>
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<tr>
<td>Beam Web</td>
<td>20.28</td>
<td>73.86</td>
<td>82.63</td>
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<tr>
<td>Column Flange</td>
<td>24.14</td>
<td>57.77</td>
<td>85.21</td>
</tr>
<tr>
<td>Column Web</td>
<td>14.68</td>
<td>72.35</td>
<td>84.67</td>
</tr>
</tbody>
</table>

Listing of Observations Made During Test

1. Yielding of end-plate around bolts noticed during cycle 22.

2. At maximum positive moment of cycle 27, the top two bolts fractured. Test was terminated.
F1-5/8-3/8-16 TEST 2 CONNECTION DETAILS
Moment vs. Total Rotation at End-Plate
F1-5/8-3/8 Test 2

Moment (ft-kips)

Moment At End-Plate vs. Inelastic Rotation at End-Plate
F1-5/8-3/8-16 Test 2

Rotation (radians)
Moment At End-Plate vs. End-Plate Separation at Top Flange
F1-5/8-3/8-16 Test 2

End-Plate Separation (in)
Moment (ft-kips)

Moment At End-Plate vs. End-Plate Separation at Bottom Flange
F1-5/8-3/8-16 Test 2

End-Plate Separation (in)
Moment (ft-kips)
Moment At End-Plate vs. Total Rotation at End-Plate
F1-5/8-3/8-16 Test 2

<table>
<thead>
<tr>
<th>Test</th>
<th>Prediction</th>
<th>Yield Line Moment=60.4 ft-kips</th>
<th>Moment Due to Bolt Rupture=58.0 ft-kips</th>
</tr>
</thead>
</table>

Moment (ft-kips)
Rotation (radians)

Moment At End-Plate vs. Total Rotation at End-Plate
F1-5/8-3/8-16 Test 2
APPENDIX C

FOUR-BOLT FLUSH RESULTS AND TEST DATA
Summary
F2-3/4-3/8-16

Predicted Capacities:
Moment Strength Predicted by Yield Line Analysis........65.1 ft-kips
Moment Strength Predicted by Bolt Rupture..................104.6 ft-kips

Maximum Moment From Test...............................66.8 ft-kips

Maximum Total Rotation.................................0.052 rad
Maximum Inelastic Rotation..............................0.048 rad

Number of Completed Cycles.........................12

Failure mode........................................End-plate and weld fractures
Weld fractures that occurred during test:
• Top flange-to-end-plate on right side of the web.
• Bottom flange-to-end-plate on left side of web.
• Bottom flange-to-end-plate on right side of web, 2-3/8 in. in length beginning at the edge of the flange.
• Web-to-end-plate 6 in. in length beginning at the top flange.
End-Plate Fractures that occurred during test:
• At web-to-end-plate 6-1/2 in. in length beginning at the bottom flange.
Load and Displacement History
F2-3/4-3/8-16

<table>
<thead>
<tr>
<th>Load Step #</th>
<th>Maximum Load (kips)</th>
<th>Peak Deformation, $\theta$</th>
<th>Number of Cycles, n</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4.6</td>
<td>*</td>
<td>4</td>
</tr>
<tr>
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<td>7.5</td>
<td>*</td>
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</tr>
<tr>
<td>3</td>
<td>9.4</td>
<td>*</td>
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<tr>
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<td>0.05</td>
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</table>

*Note: Loading switched from load to deflection control at the fourth load step.

ATC Loading Protocol Used

Material Testing Results
F2-3/4-3/8-16

<table>
<thead>
<tr>
<th>Location</th>
<th>% Elongation</th>
<th>$F_y$ (ksi)</th>
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<td>Beam Web</td>
<td>21.09</td>
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<td>86.18</td>
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<tr>
<td>Column Web</td>
<td>16.25</td>
<td>69.62</td>
<td>82.44</td>
</tr>
</tbody>
</table>

Listing of Observations Made During Test

1. Separation of end-plate noticed at top and bottom bolts during cycle 10.
2. End-plate-to-beam-web weld fracture noticed during cycle 10.
3. Test terminated after cycle 12
F2-3/4-3/8-16 CONNECTION DETAILS
Measured Bolt Strain X E vs. Flange Force - Bolt 1
F2-3/4-3/8-16

Measured Bolt Strain X E vs. Flange Force - Bolt 2
F2-3/4-3/8-16
Measured Bolt Strain X E vs. Flange Force - Bolt 3
F2-3/4-3/8-16

Measured Bolt Strain X E vs. Flange Force - Bolt 4
F2-3/4-3/8-16
Moment At End-Plate vs. Total Rotation at End-Plate

- Yield Line Moment = 65.1 ft-kips
- Moment Due to Bolt Rupture = 104.6 ft-kips

Yield Line Moment = 65.1 ft-kips
Moment Due to Bolt Rupture = 104.6 ft-kips
Summary
F2-5/8-3/8-16
Test 1

Predicted Capacities:
Moment Strength Predicted by Yield Line Analysis........73.0 ft-kips
Moment Strength Predicted by Bolt Rupture.................76.8 ft-kips

Maximum Moment From Test.................................67.7 ft-kips

Maximum Total Rotation......................................0.028 rad
Maximum Inelastic Rotation.................................0.02 rad

Number of Completed Cycles.................................26

Failure mode....................................................Bolt Fracture
Bolt fractured during cycle 27
Displacement History
F2-5/8-3/8-16 Test 1

<table>
<thead>
<tr>
<th>Load Step #</th>
<th>Peak Deformation, $\theta$</th>
<th>Number of Cycles, n</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.00375</td>
<td>6</td>
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<tr>
<td>2</td>
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<tr>
<td>5</td>
<td>0.015</td>
<td>2</td>
</tr>
<tr>
<td>7</td>
<td>0.03</td>
<td>2</td>
</tr>
<tr>
<td>8</td>
<td>0.04</td>
<td>½</td>
</tr>
</tbody>
</table>

Material Testing Results
F2-5/8-3/8-16 Test 1

<table>
<thead>
<tr>
<th>Location</th>
<th>% Elongation</th>
<th>$F_y$ (ksi)</th>
<th>$F_u$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>End-Plate</td>
<td>23.02</td>
<td>52.97</td>
<td>82.89</td>
</tr>
<tr>
<td>Beam Flange</td>
<td>23.14</td>
<td>61.07</td>
<td>85.93</td>
</tr>
<tr>
<td>Beam Web</td>
<td>20.34</td>
<td>73.88</td>
<td>83.17</td>
</tr>
<tr>
<td>Column Flange</td>
<td>23.39</td>
<td>58.47</td>
<td>86.29</td>
</tr>
<tr>
<td>Column Web</td>
<td>15.75</td>
<td>70.69</td>
<td>84.70</td>
</tr>
</tbody>
</table>

Listing of Observations Made During Test

1. Slight end-plate separation noticed at both flanges during cycle 25.
2. At maximum negative moment of cycle 27, bolt fractured. Test was terminated.
End Plate 3/8x6x16
(Gr 50)

Column:
Gr 50 Steel
Flange 5/8"
Web 1/4"
Depth 12"

Beam:
Gr 50 Steel
Flange 1/4"
Web 3/16"

5/8" dia. A-325

1/8" typ

1/4" typ

1/4" 5 3/4"

6"

1 3/8"

1 5/8" 2 3/4" 1 5/8"

6"

3/4"

1/4"

1/4"

1/4"

F2-5/8-3/8-16 TEST 1 CONNECTION DETAILS
Moment vs. Total Rotation at End-Plate
F2-5/8-3/8-16 Test 1

Moment At End-Plate vs. Inelastic Rotation at End-Plate
F2-5/8-3/8 Test 1
Moment At End-Plate vs. End-Plate Separation at Top Flange
F2-5/8-3/8-16 Test 1

Moment At End-Plate vs. End-Plate Separation at Bottom Flange
F2-5/8-3/8-16 Test 1
Moment At End-Plate vs. Total Rotation at End-Plate

F2-5/8-3/8-16 Test 1

0.000 0.005 0.010 0.015 0.020 0.025 0.030 0.035
Rotation (radians)

0 20 40 60 80 100 120
Moment (ft-kips)

- Test
- Prediction
- Yield Line Moment=73.0 ft-kips
- Moment Due to Bolt Rupture=76.80 ft-kips

Yield Line Moment=73.0 ft-kips
Moment Due to Bolt Rupture=76.80 ft-kips
Datapack
Test F2-5/8-3/8-16
Test 2

Research on the Inelastic Rotation Capability of End-Plate Moment Connections

Mark R. Boorse
Research Assistant

Thomas M. Murray
Principal Investigator
Summary
F2-5/8-3/8-16
Test 2

Predicted Capacities:
Moment Strength Predicted by Yield Line Analysis………….75.0 ft-kips
Moment Strength Predicted by Bolt Rupture…………………76.8 ft-kips

Maximum Moment From Test………………………………..67.9 ft-kips

Maximum Total Rotation……………………………………..0.033 rad
Maximum Inelastic Rotation………………………………….0.018 rad

Number of Completed Cycles………………………………...28

Failure mode…………………………………………………..Bolt Fracture
Bolt fractured during cycle 29
Displacement History
F2-5/8-3/8-16 Test 2

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<td>2</td>
</tr>
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<td>0.02</td>
<td>2</td>
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<tr>
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<td>0.03</td>
<td>2</td>
</tr>
<tr>
<td>8</td>
<td>0.04</td>
<td>¼</td>
</tr>
</tbody>
</table>

Material Testing Results
F2-5/8-3/8-16 Test 2

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<thead>
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</tr>
</tbody>
</table>

Listing of Observations Made During Test

1. Yielding of end-plate around bolts noticed during cycle 24.

2. At maximum negative moment of cycle 29, bolt fractured. Test was terminated.
F2-5/8-3/8-16 TEST 2 CONNECTION DETAILS
Moment At End-Plate vs. Total Rotation at End-Plate

<table>
<thead>
<tr>
<th>Test</th>
<th>Prediction</th>
<th>Yield Line Moment=75.0 ft-kips</th>
<th>Moment Due to Bolt Rupture=76.8 ft-kips</th>
</tr>
</thead>
</table>

Moment (ft-kips) vs. Rotation (radians)

Moment At End-Plate vs. Total Rotation at End-Plate

F2-5/8-3/8-16 Test 2
VITA

Mark R. Boorse was born in Brookfield, Wisconsin on July 30, 1974. After graduating from high school in 1992 he entered the civil engineering program at the University of Wisconsin at Platteville. In 1997 he attained a Bachelor of Science in Civil Engineering and enrolled in the graduate program in the Civil Engineering Department at Virginia Polytechnic Institute and State University, Blacksburg, Virginia.