



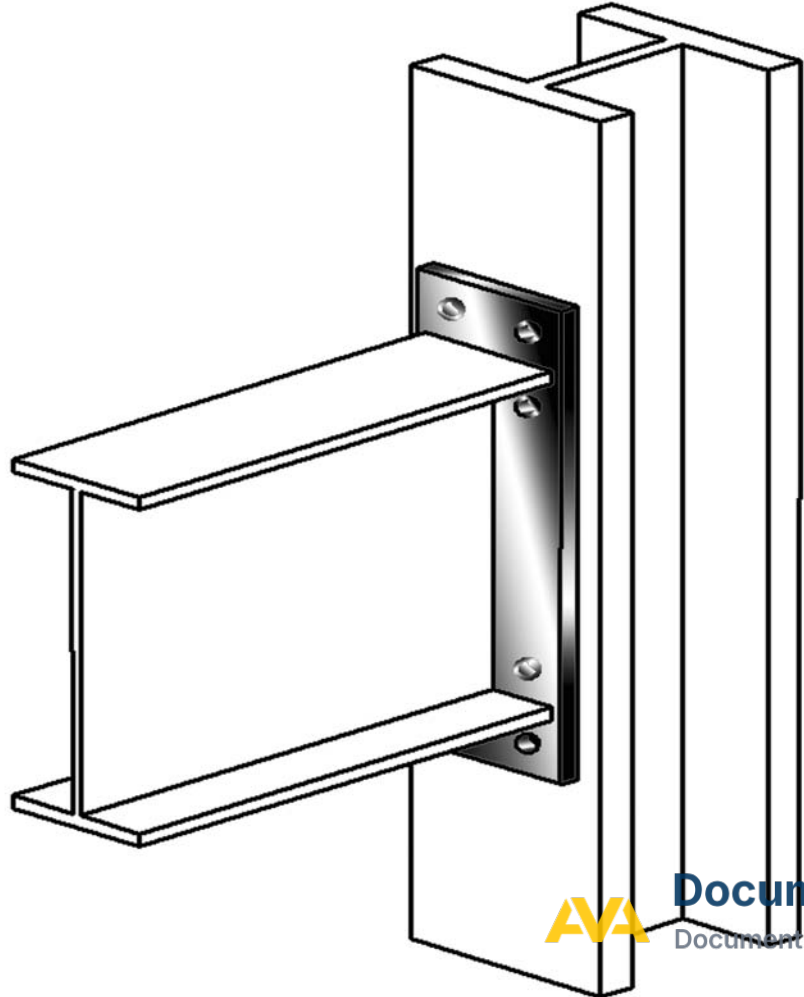
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Steel Design Guide

Extended End-Plate Moment Connections

Seismic and Wind Applications

Second Edition





4

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Second Edition

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Chapter 1

Introduction

1.1 Background

A typical moment end-plate connection is composed of a steel plate welded to the end of a beam section with attachment to an adjacent member using rows of fully tensioned high-strength bolts. The connection may join two beams (splice plate connection) or a beam and a column. end-plate moment connections are classified as either flush or extended, with or without stiffeners, and further classified depending on the number of bolts at the tension flange. A flush connection is detailed such that the end plate does not appreciably extend beyond the beam flanges and all bolts are located between the beam flanges. Flush end-plate connections are typically used in frames subject to light lateral loadings or near inflection points of gable frames. An extended connection is detailed such that the end plate extends beyond the tension flange a sufficient distance to allow a location of bolts other than between the beam flanges. Extended end plates may be used with or without a stiffener between the end plate and the tension beam flange in the plane of the beam web. Extended end plates are used for beam-to-column moment connections.

The three extended end-plate configurations shown in Figure 1.1 have been tested for use in seismic applications. The intent of this edition of the Guide is to present complete design procedures and examples of the three moment end-plate configurations, which have been shown to be suitable

for fully constrained (FR or Type I) construction in seismic applications. The design procedures can be used for other than seismic applications with proper adjustments for the required connection design moment. The four-bolt unstiffened configuration shown in Figure 1.1(a) is probably the most commonly used in multi-story frame construction. Adding a stiffener as shown in Figure 1.1(b) can reduce the required end plate thickness. Assuming the full beam moment strength is to be resisted and a maximum bolt diameter of $1\frac{1}{2}$ in., these connections, because of tensile bolt strength, will be sufficient for less than one-half of the available hot-rolled beam sections. The stiffened eight-bolt connection shown in Figure 1.1(c) is capable of developing the full moment capacity of most of the available beam sections even if bolt diameter is limited to $1\frac{1}{2}$ in. Design procedures and example calculations for these connections are given in the following chapters.

Non-seismic design procedures for the connection configurations shown in Figure 1.1(a) and (c) were presented in the first edition of this guide (Murray 1990). These procedures are also found in the *AISC ASD Manual of Steel Construction*, 9th Edition (AISC 1989) and the *LRFD Manual of Steel Construction*, 3rd Edition (AISC 2001).

New design procedures for the configurations shown in Figure 1.1(a) and (b) plus seven other configurations are available in the American Institute of Steel Construction/Metal Building Manufacturers Association *Steel*

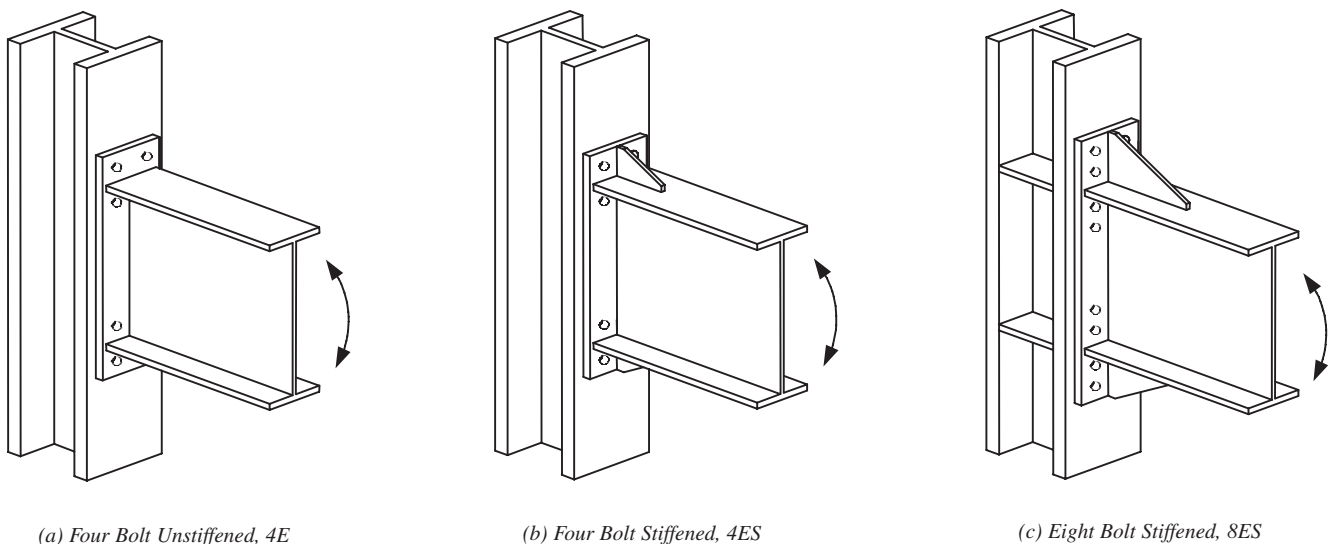


Fig. 1.1. Extended end plate configurations.

Design Guide 16 Flush and Extended Multiple-Row End-Plate Moment Connections (Murray and Shoemaker 2002). The design procedures in *Design Guide 16* permit the use of snug tightened bolts, but the procedures have not been verified for high seismic applications.

As with any connection, end-plate connections have certain advantages and disadvantages.

The principal advantages are:

- a) The connection is suitable for winter erection in that only field bolting is required.
- b) All welding is done in the shop, eliminating problems associated with field welding.
- c) Without the need for field welding, the erection process is relatively fast and generally inexpensive.
- d) If fabrication is accurate, it is easy to maintain plumbness of the frame.
- e) Competitive total installed cost, for most cases.

The principal disadvantages are:

- a) The fabrication techniques are somewhat stringent because of the need for accurate beam length and “squareness” of the beam end.
- b) Column out-of-squareness and depth tolerance can cause erection difficulties but can be controlled by fabrication of the beams $\frac{1}{4}$ in. to $\frac{3}{8}$ in. short and providing “finger” shims.
- c) End plates often warp due to the heat of welding.
- d) End plates are subject to lamellar tearing in the region of the top flange tension weld.
- e) The bolts are in tension, which can result in prying forces.
- f) A portion of the stiffened end plate may extend above the finished floor requiring a larger column closure and reduced useable floor area.

A number of designers and fabricators in the United States have successfully used moment end-plate connections for building frames up to 30 stories in height in low seismic regions and up to 10 stories in height in high seismic regions. In spite of the several disadvantages, moment end-plate connections can provide economic solutions for rigid frame construction.

1.2 Overview of the Design Guide

The remainder of this chapter is a brief survey of literature pertinent to the recommended design procedures. Chapter 2 presents the basic design procedures and recommended detailing and fabrication practices. Chapter 3 contains a design procedure for all three connections. Chapter 4 has complete design examples. Nomenclature is found in the Appendix A. Appendix B has a preliminary design procedure and design aids.

1.3 Brief Literature Overview

There is a great deal of literature available on the analysis and design of end-plate moment connections. Publication has been almost continuous since the first known paper over 40 years ago (Disque 1962). The 1st Edition of this guide contains a summary of the literature through the 1980s. Literature, which is relevant to the scope of this edition, is briefly summarized in the following five sub-sections: end-plate design, bolt design, column-side design, cyclic testing of end-plate moment connections, and finite element analysis of end-plate moment connections.

1.3.1 End Plate Design

Research starting in the early 1950s and continuing to the present has resulted in refined design procedures for both flush and extended end-plate connections. The earlier design methods were based on statics and simplifying assumptions concerning prying forces. These methods resulted in thick end plates and large diameter bolts. Other studies have been based on yield-line theory, the finite element method, and the finite element method together with regression analysis to develop equations suitable for design use. The latter method was used to develop the design procedures in the 1st Edition of this guide. The resulting design equations involve terms to fractional powers, which virtually eliminates “structural feel” from the design. The design procedures in this edition are based on yield-line theory and have been verified for use in high seismic regions by experimental testing. Reviews of relevant literature follows.

Murray (1988) presented an overview of the past literature and design methods for both flush and extended end-plate configurations, including column-side limit states. Design procedures, based on analytical and experimental research in the United States, were presented.

Murray (1990) presented design procedures for the four-bolt unstiffened, four-bolt wide unstiffened, and the eight-bolt extended stiffened end-plate moment connections. The end plate design procedures were based on the works of Krishnamurthy (1978), Ghassemieh and others (1983), and Murray and Kukreti (1988).

Chasten and others (1992) conducted seven tests on large extended unstiffened end-plate connections with eight bolts

at the tension flange (four-bolts wide). Both snug and fully tensioned bolts were used in the testing. End-plate shear fractures, bolt fractures, and weld fractures were the observed failure modes. Finite element modeling was used to predict the distribution of the flange force to the tension bolts and to predict the magnitude and location of the prying force resultants. It was shown that the end-plate shear and bolt forces, including prying, can accurately be predicted using finite element analysis. In addition, simple design rules that complemented the existing procedures were presented.

Graham (1993) reviewed the existing design methods and recommended a limit state design method for the design of rigid beam-to-unstiffened column extended end-plate connections.

Borgsmiller and others (1995) conducted five tests on extended end-plate moment connections with large inner pitch distances—the distance from the inside of the flange to the first row of inside bolts. Results showing end plate, bolt, and connected beam behavior were presented.

Borgsmiller (1995) presented a simplified method for the design of four flush and five extended end-plate moment connection configurations. The bolt design procedure was a simplified version of the modified Kennedy method (see Section 2.2.3) to predict the bolt strength including the effects of prying. The end plate strength was determined using yield line analysis. Fifty-two end-plate connection tests were analyzed and it was concluded that the prying forces in the bolts become significant when ninety percent of the yield-line end plate strength is achieved. This established a threshold for the point at which prying forces in the bolts can be neglected. If the applied load is less than ninety percent of the plate strength, the end plate is considered to be ‘thick’ and no prying forces are considered; when the applied load is greater than ninety percent of the end plate strength, the end plate is considered to be ‘thin’ and the prying forces are assumed to be at a maximum. This distinct threshold between ‘thick’ and ‘thin’ plate behavior greatly simplified the bolt force determination because only the case of no prying or maximum prying must be determined. Good correlation with past test results was obtained using the simplified design procedure.

Sumner and Murray (2001a) performed six, three row extended end-plate connection tests to investigate the validity of the current design procedures for gravity, wind and low seismic loading. In addition, the tests investigated the effects of standard and large inner pitch distances and the connections utilized both ASTM A325 and ASTM A490 bolts. Good correlation between the experimental and analytical results was observed.

Sumner and Murray (2001b) investigated extended end-plate connections with four high strength bolts per row instead of the traditional two bolts per row. The eight-bolt

extended, four-bolts wide and three row extended, four-bolts wide end-plate moment connections were investigated. Seven end-plate connection tests were performed and a modified design procedure, similar to the procedure presented by Borgsmiller (1995) was proposed. It was concluded that the modified design procedure conservatively predicts the strength of the two connection configurations.

Murray and Shoemaker (2002) presented a guide for the design and analysis of flush and extended end-plate moment connections. The guide includes provisions for the design of four flush and five extended end-plate connection configurations. The design provisions are limited to connections subject to gravity, wind and low-seismic forces; moderate and high seismic applications are not included. A unified design procedure, based on the simplified method presented by Borgsmiller (1995) was employed. The procedure is based on yield line analysis for the determination of the end plate thickness and the modified Kennedy method for determination of the bolt forces. A stiffness criterion for flush end-plate moment connections was also included in the procedure.

Sumner (2003) presented a unified method for the design of eight extended end-plate moment connection configurations subject to cyclic/seismic loading. The design procedure uses yield line theory to predict the end plate and column flange strength. The bolt forces are determined using the simplified method developed by Borgsmiller (1995). Results of ninety end-plate moment connection tests were used to evaluate the unified design method. Good correlation with the experimental results was obtained using the unified design method.

1.3.2 Bolt Design

Numerous studies have been conducted to investigate the behavior of the bolts in end-plate moment connections. The primary focus of the studies has been to measure and predict possible prying forces. The majority of the bolt force prediction methods were developed using an analogy between a tee-stub in tension and the end-plate connection. Douty and McGuire (1963, 1965), Kato and McGuire (1973), Nair and others (1974), and Agerskov (1976, 1977) conducted early studies on tee-stubs to evaluate the bolt forces including the effects of prying. All assumed the location of the prying force to be at or near the edge of the end plate. For connections with a large degree of prying action, this results in large bolt diameters and thick end plates. Fisher and Struik (1974) present a comprehensive review of the then available design methods.

Kennedy and others (1981) developed a design procedure for tee stub connections. The procedure identifies three stages of tee stub flange plate behavior. The first stage of plate behavior occurs at low load levels and is identified by purely elastic behavior. The flange plate is said to be ‘thick’,

and it is assumed that there are no prying forces. As the load increases and a plastic hinge forms in the flange plate at the base of the tee stem, a second stage of behavior exists. The plate is said to be of intermediate thickness, and prying forces are present. The third stage of plate behavior occurs as a subsequent plastic hinge forms at the bolt line. The plate is classified as thin, and prying forces are at a maximum. The analytical method correlated well with the two tee-stub tests conducted as a part of their study.

Srouji and others (1983a, 1983b), Hendrick and others (1984, 1985), Morrison and others (1985, 1986), and Borgsmiller (1995) use a modified Kennedy approach to predict the bolt forces in flush, extended, stiffened, and unstiffened end-plate moment connection configurations. The primary modification to the Kennedy method is an adjustment to the location of prying force and modification of the distribution of the flange force to the particular bolt rows.

Ahuja and others (1982) and Ghassemieh and others (1983) used regression analysis of finite element results to predict the bolt forces of the eight-bolt extended stiffened end-plate moment connection configuration.

Fleischman and others (1991) studied the strength and stiffness characteristics of large capacity end-plate connections with snug-tight bolts. They showed that the initial stiffness is slightly reduced in the snug tight connections but the ultimate strength is the same.

Murray and others (1992) investigated the behavior of end-plate moment connections with snug-tight bolts subject to cyclic wind loading. Eleven tests representing six different connection configurations were tested. The results were consistent with the analytical predictions. It was concluded that end-plate moment connections with snug-tight bolts provide slightly reduced stiffness when compared to fully-tightened end-plate connections.

1.3.3 Column-side Design

There is a relatively small amount of literature on the column-side design of end-plate moment connections. Numerous papers make observations about the behavior of the column during testing but no specific design criteria are discussed. The few papers that are available consider only the limit states of column web yielding and column flange bending.

Web Yielding. Mann and Morris (1979) investigated the column web strength at end-plate moment connections. An evaluation of results from several research projects was conducted. It was recommended that the connecting beam flange force be distributed at a slope of 1:1 through the end plate and then on a 2.5:1 slope through the column flange and web.

Hendrick and Murray (1983, 1984) conducted a series of tests and an analytical study to determine the column web

compression yielding strength at end-plate moment connections. A design equation was developed and good correlation with the finite element and experimental results was observed. It was recommended that the connecting beam flange force be distributed through the end plate at a slope of 1:1 and then on a slope of 3:1 through the column.

Flange Bending. Mann and Morris (1979) conducted an extensive study on the design of end-plate moment connections. Included in their study was the development of column-side design provisions. The column-side provisions were primarily based on the work of Packer and Morris (1977). They describe three possible modes of column flange failure and provide equations to predict the strength of each. For relatively thin column flanges, the effects of prying forces are accounted for by limiting the bolt tensile capacity.

Witteveen and others (1982) studied welded flange and bolted end-plate connections and identified three possible column flange failure modes similar to the findings of Mann and Morris (1979). Design equations to predict the three modes and comparisons with experimental testing were presented.

Tarpy and Cardinal (1981) conducted an experimental and analytical study of the behavior of unstiffened beam-to-column end-plate connections. The experimental tests were conducted with axial load applied to the columns. The analytical study included the development of finite element models, which were used to develop regression equations for predicting the end plate and column flange strength.

Hendrick and others (1983) evaluated the existing methods for predicting the column flange bending strength. They conducted limited experimental testing and concluded that the method presented by Mann and Morris (1979) was most suitable for the design of the tension region of the four-bolt extended unstiffened end-plate moment connections. In addition, they modified the end plate design procedure presented by Krishnamurthy (1978) by substituting the end plate width with an effective column flange width. This procedure was calibrated to provide the same results as the Mann and Morris (1979) equations.

Curtis and Murray (1989) investigated the column flange strength at the tension region of the four-bolt extended stiffened and eight-bolt extended stiffened end-plate connections. Their design procedure is based on the Ghassemieh and others (1983) end plate design procedure with an effective column flange length substituted for the end plate width.

Murray (1990) presented column-side design procedures for the four-bolt unstiffened, four-bolt wide unstiffened, and the eight-bolt extended stiffened end-plate moment connections. The column-side procedures were based on works by Hendrick and Murray (1984), and Curtis and Murray (1989).

Sumner (2003) presented a unified column flange bending design procedure for eight extended end-plate moment connection configurations. The design procedure utilized yield line analysis to predict the strength of the stiffened and unstiffened column flange configurations. Results of past experimental tests were analyzed to evaluate the unified design procedure. Good correlation with the experimental results was found.

1.3.4 *Cyclic Testing of End-Plate Moment Connections*

Early investigations into the cyclic performance of end-plate moment connections were limited to small beam sections with unstiffened end plates. Subsequent studies have investigated connections between larger sections. One of the primary distinctions between the different studies is the source of inelastic behavior. Some researchers have investigated the inelastic response of the end plate and others the inelastic response of the connecting beam.

Four cruciform beam-to-column end-plate connection tests were conducted by Johnstone and Walpole (1981). The four-bolt extended unstiffened connections were designed to study the previously developed recommendations for monotonic loading together with the design rules in the New Zealand design standards. The results show that end-plate connections can transmit the necessary forces to force most of the inelastic deformations to occur in the beam. However, connections designed for less than the capacity of the beam may not provide the required ductility.

Popov and Tsai (1989) investigated cyclic loading of several different types of moment connections. The objective was to investigate realistic member size and the extent of cyclic ductility. Their results indicated that end-plate moment connections are a viable alternative to fully-welded connections in seismic moment-resisting frames. Continuing their research on end-plate connections, Tsai and Popov (1990) investigated the four-bolt extended stiffened and unstiffened end-plate connection configurations. The results from their experimental and finite element studies showed the design procedures for monotonic loading need to be modified for seismic loading.

Research by Ghobarah and others (1990) investigated the cyclic behavior of extended stiffened and unstiffened end-plate connections. Five specimens were tested, some with axial load applied to the column, to compare the performance of stiffened and unstiffened end plates, stiffened and unstiffened column flanges, and to isolate the individual behavior of the beam, column flange, stiffeners, bolts and end plate. They concluded that proper proportioning of the end-plate connections could provide sufficient energy dissipation capability without substantial loss of strength. They recommended that for unstiffened connections, the bolts and end plate be designed for 1.3 times the plastic moment capacity of the beam to limit the bolt degradation and com-

pensate for prying forces. It was also recommended that for stiffened connections, the end plate and bolts be designed for the plastic moment capacity of the beam.

As an extension of the work by Ghobarah and others (1990), Korol and others (1990) conducted seven extended end-plate moment connection tests. Design equations that consider the strength, stiffness and energy dissipation requirements of extended end-plate connections were presented. They concluded that proper design and detailing of end-plate connections will produce end-plate connections that provide sufficient energy dissipation without substantial loss of strength or stiffness.

Ghobarah and others (1992) continued their research on end-plate connections by testing four additional connections. The specimens were subjected to cyclic loading and axial load was applied to the column. They found that column panel zone yielding can dissipate large amounts of energy and that the end plate helps to control the inelastic deformation of the panel zone. They recommended that panel zone yielding be used to increase the energy dissipating capacity of the end-plate moment connections.

Fleischman and others (1990) conducted five cyclic beam-to-column tests utilizing four-bolt wide extended unstiffened end-plate moment connections. The effect of snug versus fully-tightened bolts was investigated. The connections were designed weaker than the connecting beam and column so that the inelastic behavior of the end plate could be investigated. It was observed that the connection stiffness gradually decreased in successive inelastic cycles, the energy absorption capacity increased as the end plate thickness decreased, the bolt forces were increased up to thirty percent because of prying action, and the snug-tightened connections exhibited higher energy absorption capacity.

Astaneh-Asl (1995) conducted two cyclic tests on the four-bolt extended unstiffened end-plate moment connection. The specimens were designed using the existing AISC recommendations, which were not intended for seismic applications. The first test exhibited ductile behavior and resulted in local buckling of the connecting beam flange. The second test utilized an I-shaped shim between the end plate and the column. The performance of the specimen was excellent until the shim began to yield in compression. The author concluded that the concept was sound but that a stronger shim was needed.

Adey and others (1997, 1998, 2000) investigated the effect of beam size, bolt layout, end plate thickness, and extended end plate stiffeners on the energy absorption ability of the end plate. Fifteen end-plate connections subject to cyclic loading were conducted. Twelve of the 15 connections were designed weaker than the connecting beams and columns to isolate the yielding in the end plate. The other three tests were designed to develop the nominal plastic

moment strength of the connected beam. It was concluded that the end plate energy absorption capability decreases as the beam size increases and that extended end-plate stiffeners increase the end plate absorption capability. In addition, a design procedure for the four-bolt extended unstiffened and stiffened end-plate moment connections was presented. The design procedure utilizes yield line theory for the determination of the end plate thickness. The connection bolts design procedure assumes a twenty percent increase in the bolt forces to account for the possible presence of prying forces.

Meng and Murray (1997) conducted a series of cyclic tests on the four-bolt extended unstiffened end-plate moment connections. The test specimens were designed with the connections stronger than the connecting beam and column. The end plate thickness was determined using yield line analysis and the bolt forces predicted by the modified Kennedy method. The testing identified a problem with the use of weld access holes in making the beam flange to end-plate welds. In all of the specimens with weld access holes, the flanges fractured after the first few inelastic cycles. In the specimens without weld access holes, a robust inelastic response and a large energy dissipation capacity were observed. Results from a subsequent finite element analysis study indicated that the presence of the weld access hole greatly increases the flange strain in the region of the access hole. Based on the results of their study, they recommended that weld access holes not be used in end-plate moment connections. They concluded that properly designed end-plate connections are a viable connection for seismic moment frame construction.

Meng (1996) and Meng and Murray (1996) investigated the four-bolt extended stiffened, four-bolt wide extended stiffened, four-bolt wide extended unstiffened, and shimmed end-plate moment connections. Design procedures for the connections are presented and comparisons with the experimental tests shown.

An overview of the previous research on bolted and riveted connections subject to seismic loads is presented Leon (1995). He discusses the fundamentals of bolted and riveted connection design and identifies possible extensions of the monotonic design methods to the cyclic loading case. He concludes that properly designed bolted connections can provide equal or superior seismic performance to that of fully welded ones. In addition, a new, more fundamental and comprehensive approach is needed in current codes so that bolted connections can be properly designed in areas of moderate and high seismicity.

Castellani and others (1998) present preliminary results of ongoing European research on the cyclic behavior of beam-to-column connections. The extended unstiffened end-plate moment connection tests resulted in very regular hysteresis loops with no slippage and a progressive reduc-

tion in the energy absorption. A plastic hinge formed in the connecting beams and large deformations at the plastic hinge induced cracking in the beam flange, ultimately resulting in complete failure of the section.

Coons (1999) investigated the use of end plate and tee-stub connections for use in seismic moment resisting frames. A database of previously published experimental data was created and analytical models developed to predict maximum moment capacity, failure mode, and maximum inelastic rotation. It was observed that the plastic moment strength of the connecting beams was twenty-two percent higher than predicted by the nominal plastic moment strength. He recommended that the increased beam strength be considered for the connection design, end plate thickness be determined using yield line analysis, and the bolt forces be determined without including the effects of prying.

Boorse and Murray (1999) and Ryan and Murray (1999) investigated the inelastic rotation capability of flush and extended end-plate moment connections subject to cyclic loading. The specimens were beam-to-column connections between built-up members as used in the metal building industry. The specimens were designed with the end-plate connections weaker than the connecting members to investigate the inelastic behavior of the end plate. The end plate thickness and bolt forces were determined using yield line analysis and the modified Kennedy method respectively. The experimental results were compared with the analytical results with reasonable correlation. It was concluded that the flush end plates could be designed to provide adequate inelastic rotation but the extended end plates should be designed to force the inelastic behavior into the connecting beam.

Sumner and others (2000a, 2000b, 2000c), and Sumner and Murray (1999, 2000, 2002) conducted eleven tests on extended end-plate moment connections to investigate the suitability of end-plate connections for use in seismic force resisting moment frames. Beam-to-column connection assemblies utilizing the four-bolt unstiffened, eight-bolt stiffened, and the eight-bolt four-bolt wide configurations were tested. In addition, one test of the four-bolt unstiffened connection was conducted with a composite slab cast onto the top flanges of the beams. Results showing the end plate, bolt, beam, and column behavior were presented. It was concluded that the four-bolt unstiffened and eight-bolt stiffened end-plate moment connections can be designed for use in seismic force resisting moment frames. Details of the testing procedures and results are available in FEMA-350 (FEMA 2000a) and FEMA-353 (FEMA 2000b).

1.3.5 *Finite Element Analysis of End-Plate Moment Connections*

Early finite element studies focused on correlation of results from 2-D models to 3-D models. This was important

because of the substantially higher cost of creating and running 3-D models as compared to 2-D models. With the advances in computer technology, the use of 3-D models has become more common. More recent studies have focused on the suitability of finite element method to accurately predict the inelastic behavior of end-plate moment connections.

Krishnamurthy and Graddy (1976) conducted one of the earliest studies to investigate the behavior of bolted end-plate moment connections using finite element analysis. Connections were analyzed by 2-D and 3-D programs, so that their correlation characteristics could be applied for prediction of other 3-D values from corresponding 2-D results.

Ahuja and others (1982) investigated the elastic behavior of the eight-bolt extended stiffened end-plate moment connection using finite element analysis. Ghassemieh and others (1983) continued the work of Ahuja and included inelastic behavior. Abolmaali and others (1984) used finite element analysis to develop a design methodology for the two bolt flush end-plate moment connection configuration. Both 2-D and 3-D analyses were conducted to generate correlation coefficients.

Kukreti and others (1990) used finite element modeling to conduct parametric studies to predict the bolt forces and the end plate stiffness of the eight-bolt extended stiffened end-plate moment connection. Regression analysis of the parametric study data resulted in equations for predicting the end plate strength, end plate stiffness, and bolt forces. The predictions were compared to experimental results with reasonable correlation.

Gebbeken and others (1994) investigated the behavior of the four-bolt unstiffened end-plate connection using finite element analysis. The study emphasized modeling of the non-linear material behavior and the contact between the end plate and the column flange or the adjacent end plate. Comparisons between the finite element analysis and experimental test results were made.

Bahaari and Sherbourne (1994) used ANSYS, a commercially available finite element code, to analyze 3-D finite element models to successfully predict the behavior of the four-bolt extended unstiffened end-plate moment connection. The models used plate, brick, and truss elements with non-linear material properties. They recommended that the three-dimensional models be used to generate analytical formulations to predict the behavior and strength of the connection components.

Bahaari and Sherbourne (1996a, 1996b) continued their investigation of the four-bolt extended unstiffened end-plate

connection by considering the effects of connecting the end plate to a stiffened and an unstiffened column flange. ANSYS 3-D finite element models of the end plate and the column flange were developed. The finite element results were compared with experimental results with good correlation. Once again, it is concluded that 3-D finite element analysis can predict the behavior of end-plate connections.

Choi and Chung (1996) investigated the most efficient techniques of modeling four-bolt extended unstiffened end-plate connections using the finite element method.

Bose and others (1997) used the finite element method to analyze flush unstiffened end-plate connections. The two and four-bolt flush end-plate configurations were included in the study. Comparisons with experimental results were made with good correlation.

Bursi and Jaspart (1998) provided an overview of current developments for estimating the moment-rotation behavior of bolted moment resisting connections. In addition, a methodology for finite element analysis of end-plate connections was presented.

Meng (1996) used shell elements to model the cyclic behavior of the four-bolt extended unstiffened end-plate connection. The primary purpose of the study was to investigate the effects of weld access holes on the beam flange strength. The finite element results correlated well with the experimental results.

Mays (2000) used finite element analysis to develop a design procedure for an unstiffened column flange and for the sixteen-bolt extended stiffened end-plate moment connection. In addition, finite element models were developed and comparisons with experimental results for the four-bolt extended unstiffened, eight-bolt extended stiffened, and the four-bolt wide unstiffened end-plate moment connections were made. Good correlation with experimental results was obtained.

Sumner (2003) used finite element analysis to investigate the column flange bending strength in extended end-plate moment connections. Eight and twenty node solid elements were used to model the beam, end plate, bolts, and column flange. The results of the study were compared to the yield line analysis strength predictions. Good correlation with the analytical results was observed.

Much of the literature cited was used to develop the design procedures presented in the following chapters. The procedures conform to, but are not identical to, those recommended in FEMA-350 *Recommended Seismic Design Criteria for New Steel Moment Frame Buildings* (FEMA 2002).

Chapter 2

Background For Design Procedures

2.1 Basis of Design Recommendations

The following recommended design procedures are primarily based on research conducted at the University of Oklahoma and Virginia Polytechnic Institute. Yield-line analysis is used for end plate and column flange bending. Bolt prying forces are not a consideration since the required end plate and column flange thicknesses prevent their development.

The following assumptions or conditions are inherent to the design procedures:

1. All bolts are tightened to a pretension not less than that given in current AISC specifications; however, slip-critical connection requirements are not needed.
2. The design procedures are valid for use with either ASTM A325 or ASTM A490 bolts.
3. The smallest possible bolt pitch (distance from face of beam flange to centerline of nearer bolt) generally results in the most economical connection. The recommended minimum pitch dimension is bolt diameter plus $\frac{1}{2}$ in. for bolts up to 1 in. diameter and $\frac{3}{4}$ in. for larger diameter bolts. However, many fabricators prefer to use a standard pitch dimension of 2 in. or $2\frac{1}{2}$ in. for all bolt diameters.
4. All of the shear force at a connection is assumed to be resisted by the compression side bolts. End-plate connections need not be designed as slip-critical connections and it is noted that shear is rarely a major concern in the design of moment end-plate connections.
5. It is assumed that the width of the end plate, which is effective in resisting the applied beam moment, is not greater than the beam flange width plus 1 in. This assumption is based on engineering judgment and is not part of any of the referenced end plate design procedures.
6. The gage of the tension bolts (horizontal distance between vertical bolt lines) must not exceed the beam tension flange width.
7. Beam web to end plate welds in the vicinity of the tension bolts are designed to develop the yield stress of the beam web. This weld strength is recommended even if the full moment capacity of the beam is not required for frame strength.
8. Only the web to end plate weld between the mid-depth of the beam and the inside face of the beam compression flange may be used to resist the beam shear. This assumption is based on engineering judgment; literature is not available to substantiate or contradict this assumption.

Column web stiffeners are expensive to fabricate and can interfere with weak axis column framing. Therefore, it is recommended that they be avoided whenever possible. If the need for a stiffener is marginal, it is usually more economical to increase the column size rather than install stiffeners. If column web stiffeners are required because of inadequate column flange bending strength or stiffness, increasing the effective length of the column flange may eliminate the need for stiffening. This can be accomplished by increasing the tension bolt pitch or by switching from a two row configuration, Figures 1.1(a) or (b), to the four row configuration Figure 1.1(c).

2.2 Overview of Theory and Mechanics

The unified design procedure for end-plate moment connections subject to cyclic loading requires careful consideration of four primary design parameters: the required connection design moment, end plate strength, connection bolt strength, and column flange strength. Details of the background theory and design models used to develop the provisions for each design parameter follow.

2.2.1 Connection Design Moment

The current design methodology in the AISC *Seismic Provisions* (AISC, 2002) requires that the specified interstory drift of a steel moment frame be accommodated through a combination of elastic and inelastic frame deformations. The inelastic deformations are provided through development of plastic hinges at pre-determined locations within the frame. When end-plate connections are used, the plastic hinges are developed through inelastic flexural deformations in the connecting beams and in the column panel zone. This results in a strong column, strong connection and weak beam design philosophy.

The location of the plastic hinge formation within the connecting beams is dependent upon the type of end-plate connection used. For end-plate moment connections, the hinge location is different for unstiffened and stiffened configurations. For unstiffened end-plate moment connections, the plastic hinge forms at a distance equal to approximately the minimum of one half the beam depth and three times the

beam flange width from the face of the column. For stiffened end-plate moment connections, the plastic hinge forms at the base of the end plate stiffeners. Figure 2.1 illustrates the locations of hinge formation for end-plate connections. The expected locations of the plastic hinges within the frame should be used to properly model the frame behavior, and to determine the strength demands at the critical sections within the connections.

From AISC Seismic Provisions (2002), the Required Strength of a connection is determined from the Expected Yield stress $R_y F_y$ where R_y is the ratio of the expected yield stress to the specified minimum yield stress (equal to 1.5 for $F_y = 36$ ksi and 1.1 for $F_y = 50$ ksi) and F_y is the specified minimum yield stress of the grade of steel. The expected moment at the plastic hinge is then

$$M_{pe} = 1.1 R_y F_y Z_x \quad (2.1)$$

The critical section for the design of end-plate moment connections is at the face of the column flange. The moment at the face of the column, M_{fc} , is the sum of the expected moment at the plastic hinge, M_{pe} , and the additional moment caused by the eccentricity of the shear force present at the hinge location. Figure 2.2 illustrates this concept.

Applying the distances to the expected hinge locations for stiffened and unstiffened end-plate moment connections results in the following expressions for the connection design moments.

For unstiffened connections:

$$M_{uc} = M_{pe} + V_u \left(\min \left(d/2, 3 b_f \right) \right) \quad (2.2)$$

For stiffened connections:

$$M_{uc} = M_{pe} + V_u \left(L_{st} + t_p \right) f \quad (2.3)$$

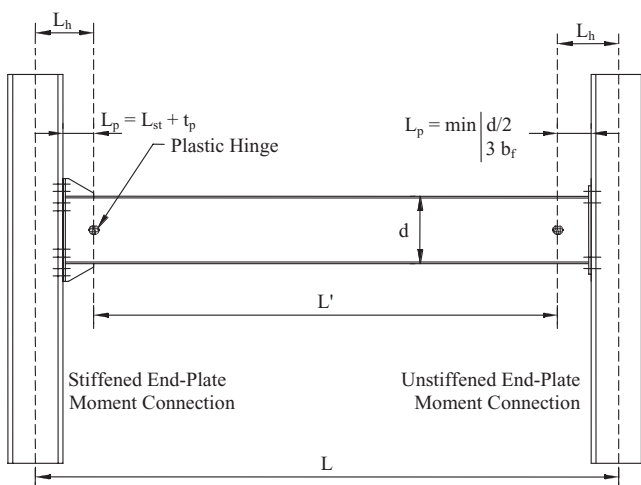


Fig. 2.1. Location of plastic hinges.

where V_u is the shear at the plastic hinge, d is the depth of the connecting beam, b_f is the beam flange width, L_{st} is the length of the end plate stiffener, and t_p is the thickness of the end plate.

2.2.2 Yield Line Theory

In the recommended design procedures, the end plate and column flange bending strengths are determined using yield line analysis. Yield line analysis can be performed by two different methods: the virtual work or energy method, and the equilibrium method. The virtual work method is the preferred method for analysis of steel plates and was used to develop the prediction equations for end plate and column flange bending strength. The virtual work method is an energy method that results in an upper bound solution for the plate strength. To determine the controlling yield line pattern for a plate, various yield line patterns must be considered. The pattern that produces the lowest failure load controls and is considered the lowest upper bound solution.

The application of yield line theory to determine the strength of an end plate or column flange requires three basic steps: assumption of a yield line pattern, generation of equations for internal and external work, and solution of internal and external work equality.

Figure 2.3 illustrates the controlling yield line pattern and assumed virtual displacement for the four-bolt extended unstiffened end-plate connections. The internal work stored within a yield line pattern is the sum of the internal work stored in each of the yield lines forming the mechanism. For the complex patterns observed in end-plate moment connections it is convenient to break the internal work components down into Cartesian (x - and y -) components. The

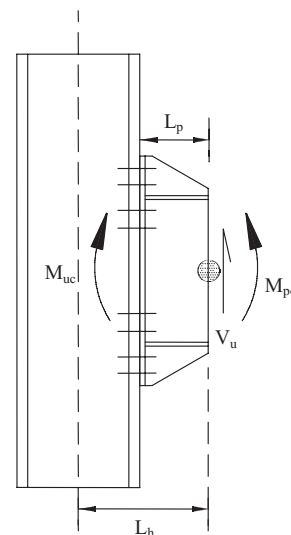


Fig. 2.2. Calculation of connection design moment.

general expression for internal work stored by the yield line pattern is

$$W_i = \sum_{n=1}^N (m_p \theta_{nx} L_{nx} + m_p \theta_{ny} L_{ny}) \quad (2.4)$$

where θ_{nx} and θ_{ny} are the x- and y-components of the relative rotation of the rigid plate segments along the yield line, L_{nx} and L_{ny} are the x- and y- components of the yield line length, and m_p is the plastic moment strength of the end plate per unit length,

$$m_p = F_{yp} Z_p = F_{yp} \left(\frac{1}{4} t_p^2 \right) \quad (2.5)$$

The internal work, W_i , includes the distance from the inner bolts to the edge of the yield line pattern, for example, the distance s in Figure 2.3. Minimization of W_i with respect to the s -distance results in the least internal energy for the yield line pattern.

The external work due to the unit virtual rotation is given by

$$W_e = M_{pl} \theta = M_{pl} \left(\frac{1}{h} \right) \quad (2.6)$$

where M_{pl} is the end plate flexural strength and θ is the applied virtual displacement. The applied virtual displacement is equal to $1/h$, where h is the distance from the cen-

terline of the compression flange to the tension side edge of the end plate.

The flexural strength of the end plate is found by setting W_i equal to W_e and solving for M_{pl} . Or, by rearranging the expression, the required end plate thickness can be determined.

To reduce the complexity of the yield line equations, the following simplifications have been incorporated into their development. No adjustment in end plate or column flange strength is made to account for the plate material removed by bolt holes. The width of the beam or column web is considered to be zero in the yield line equations. The width of fillet welds along the flange or stiffeners and web is not considered in the yield line equations. Finally, the very small strength contribution from yield lines in the compression region of the connections is neglected.

There have been relatively few studies conducted to determine the column flange strength in beam-to-column end-plate moment connections. In a beam-to-column end-plate moment connection, the beam flange tension forces are transmitted directly to the column flange by the connection bolts. The column flange must provide adequate strength to resist the applied bolt tensile forces. The column flanges can be configured as stiffened or unstiffened. A stiffened column flange has flange stiffener plates, often called continuity plates, installed perpendicular to the column web and in-line with the connecting beam flanges. An unstiffened column flange does not have stiffener or continuity plates.

Yield line analysis has been used to develop solutions for the stiffened and unstiffened column flange configurations

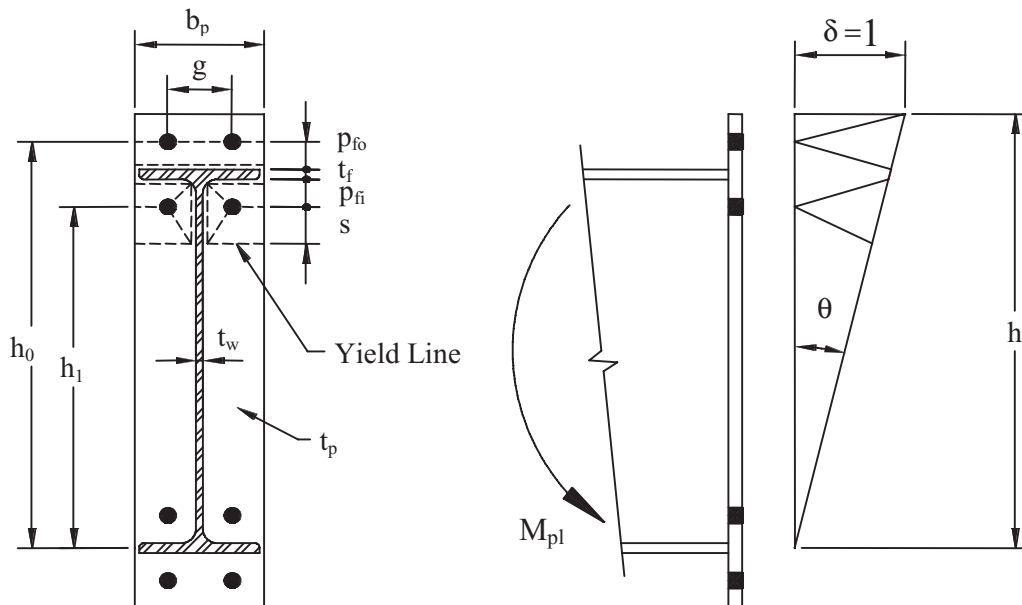


Fig. 2.3. Yield line pattern and virtual displacement of a four-bolt extended unstiffened connection.

for the end-plate moment connection configurations shown in Figure 1.1 (Sumner 2003). For example, the column flange unstiffened and stiffened yield line pattern for the eight-bolt extended stiffened end-plate connection is shown in Figure 2.4

Yield line solutions for the three end plate configurations shown in Figure 1.1 and for the corresponding unstiffened and stiffened column flanges are found in Chapter 3.

2.2.3 Bolt Force Model

Numerous studies have been conducted to investigate the behavior of the bolts in end-plate moment connections. The primary focus of the studies has been to measure and predict the possible prying forces within end-plate connections. The majority of the bolt force prediction methods were developed using an analogy between an equivalent tee-stub in tension and the end-plate connection. The design model developed by Kennedy and others (1981) is the most commonly used procedure for determining the bolt forces

in end-plate moment connections. Srouji and others (1983a, 1983b), Hendrick and others (1984, 1985), Morrison and others (1985, 1986), and Borgsmiller (1995) all used a modified Kennedy approach to predict the bolt forces in flush, extended, stiffened, and unstiffened end-plate moment connection configurations. The primary modification to the Kennedy method is an adjustment to the location of prying force and modification of the distribution of the flange force to the particular bolt rows.

The Kennedy design procedure identifies three stages of tee stub flange plate behavior. The first stage of plate behavior occurs at low load levels and is identified by purely elastic behavior. The flange plate is said to be 'thick' and it is assumed that there are no prying forces. As the load increases and a plastic hinge forms in the flange plate at the base of the tee stem, a second stage of behavior exists. The plate is said to be of intermediate thickness and prying forces are present. The third stage of plate behavior occurs as a subsequent plastic hinge forms at the bolt line. The

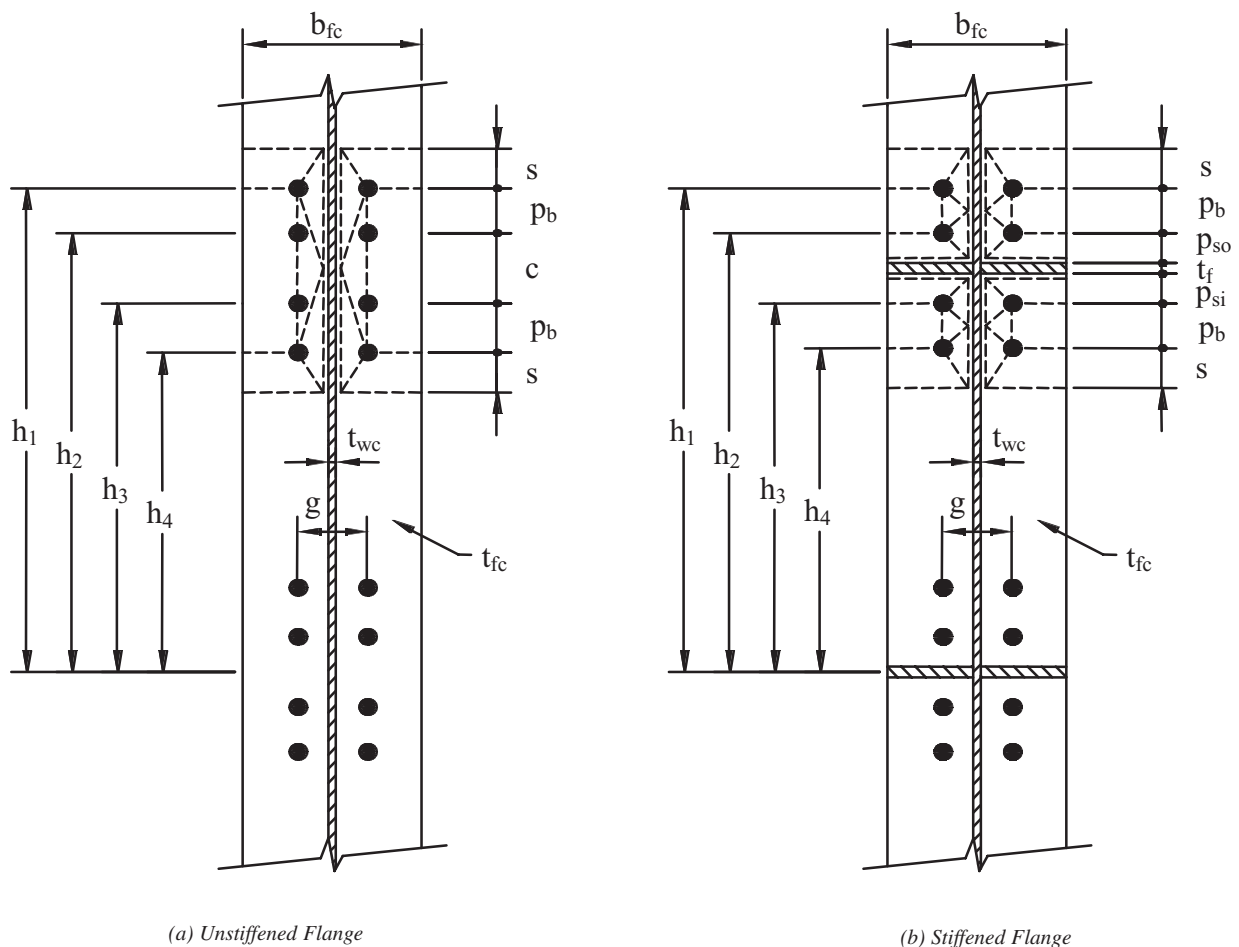


Fig. 2.4. Column flange yield line patterns of eight-bolt extended stiffened end-plate moment connections.

plate is classified as thin and prying forces are at a maximum. Figure 2.5 illustrates the three stages of plate behavior.

The Kennedy model was modified by Srouji and others (1983a, 1983b), Hendrick and others (1984, 1985), Morrison and others (1985, 1986) to adjust the location of the prying forces and to modify the distribution of the flange tension force to the various bolt rows. Borgsmiller (1995) presented a simplified version of the modified Kennedy method to predict the bolt strength including the effects of prying. The simplified method considers only two stages of plate behavior; thick plate behavior with no prying forces, and thin plate behavior with maximum prying forces. The intermediate plate behavior, as defined in the Kennedy model, is not considered. This simplification allows for direct solution of the bolt forces.

The threshold between thick and thin plate behavior was established as the point where the bolt prying forces are negligible. Based upon past experimental test results, Borgsmiller (1995) determined this threshold to be when ninety percent of the end plate strength is achieved. If the applied load is less than ninety percent of the plate strength, the end plate is considered to be ‘thick’ and no prying forces are considered; when the applied load is greater than ninety percent of the end plate strength, the end plate is considered to be ‘thin’ and the prying forces are assumed to be at a maximum.

The modified Kennedy and the simplified Borgsmiller method were developed to predict the bolt forces in tee stub and end-plate moment connections subject to monotonic loading. The application of cyclic (seismic) loading to the end-plate connections requires careful consideration. The previously discussed design philosophy is to have a strong column, strong connection and a weak beam. This forces the inelastic behavior into the connecting beams and column panel zone, and requires that the connection and column remain elastic. Applying this philosophy to the

connection requires that the end plate and column flange be designed to exhibit ‘thick’ plate behavior. This will ensure that the end plate and column flange remain elastic and that the bolts are not subject to any significant prying forces.

For thick plate behavior, the bolt forces are determined by taking the static moment of the bolt forces about the centerline of the compression flange. The connection strength, based upon bolt tension rupture, then becomes the static moment of the bolt strengths about the centerline of the compression flange. Figure 2.6 illustrates this concept for the eight-bolt stiffened end-plate connection. The no-prying moment for the bolt strength, M_{np} , is expressed by:

$$M_{np} = nP_t \sum_{i=1}^N h_i \quad (2.7)$$

where n is the number of bolts in each row, N is the number of bolt rows, and h_i is the distance from the centerline of the compression flange to the centerline of the bolt row. The bolt tension strength, P_t , is the bolt tensile strength and is expressed as follows:

$$P_t = F_t A_b \quad (2.8)$$

where F_t is the specified tensile strength (90 ksi for ASTM A325 bolts and 113 ksi for ASTM A490 bolts) in the *LRFD Specification* (AISC 1999) and A_b is the nominal area of the bolt.

The no-prying bolt moment utilizes the full tensile strength of each bolt within the connection. A common assumption that plane sections remain plane indicates that the outermost bolts will reach their tensile strength first. The underlying assumption in the Borgsmiller model is that the outer bolts will yield and provide enough deformation to develop the full tensile force in each of the inner connection

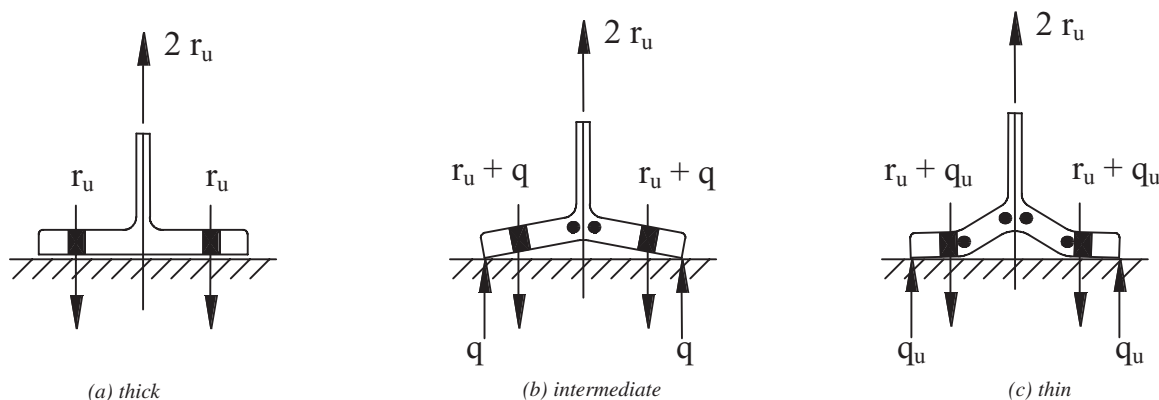


Fig. 2.5. Three stages of plate behavior in Kennedy model.

bolt rows. This assumption has been investigated in multiple row extended connections by Sumner and Murray (2001a) and was determined to be valid.

The no-prying bolt strength, calculated using Equation 2.7, implies that the end plate and column flange will exhibit thick plate behavior. To ensure thick plate behavior, the no prying strength of the bolts must be less than or equal to ninety percent of the end plate and column flange strength. Another way to state the requirement is that the end plate and column flange strength must be greater than or equal to one hundred and eleven percent of the strength of the bolts. Equations 2.9 and 2.10 are equivalent expressions defining the thick plate design requirements.

$$M_{np} \leq 0.9 M_{pl} \quad \text{and} \quad M_{np} \leq 0.9 M_{cf} \quad (2.9)$$

$$M_{pl} \geq 1.11 M_{np} \quad \text{and} \quad M_{cf} \geq 1.11 M_{np} \quad (2.10)$$

where M_{np} is the no prying moment, given in Equation 2.7, M_{pl} is the end plate flexural strength, and M_{cf} is the column flange flexural strength.

2.3 Limit States Check List

Limit states (or failure modes) that should be considered in the design of beam-to-column end-plate moment connections are:

1. Flexural yielding of the end plate material near the tension flange bolts. This state in itself is not limiting, but yielding results in rapid increases in tension bolt forces.

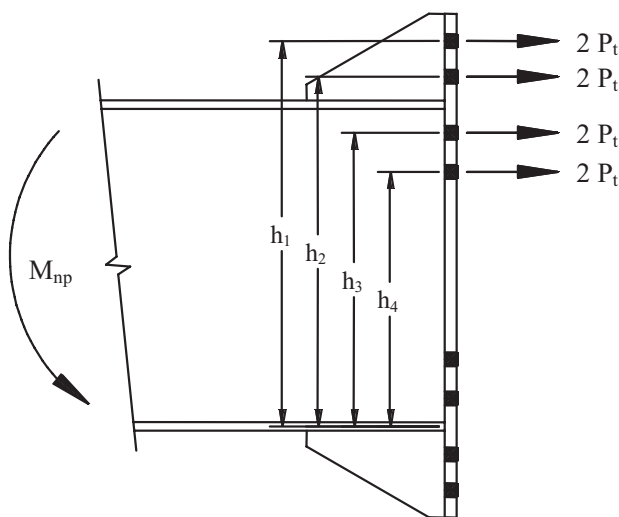


Fig. 2.6. Thick plate bolt force design model (8ES).

2. Shear yielding of the end plate material. This limit state is not usually observed, but shear in combination with bending can result in reduced flexural capacity and stiffness.
3. Shear rupture of an unstiffened end plate through the outside bolt hole line.
4. Bolt tension rupture. This limit state is obviously a brittle failure mode and is the most critical limit state in an end-plate connection.
5. Bolt shear rupture due to shear at the interface between the end plate and column flange.
6. Plate bearing failure of end plate or column flange at bolts.
7. Rupture of beam tension flange to end plate welds or beam web tension region to end plate welds.
8. Shear yielding of beam web to end plate weld or of beam web base metal.
9. Column web yielding opposite either the tension or compression flanges of the connected beam.
10. Column web crippling opposite the compression flange of the connected beam.
11. Column web buckling opposite the compression flange of the connected beam.
12. Flexural yielding of the column flange in the vicinity of the tension bolts. As with flexural yielding of the end plate, this limit state in itself is not limiting but results in rapid increases in tension bolt forces and excessive rotation at the connection.
13. Column transverse stiffener (continuity plate) failure due to yielding, local buckling, or weld failure.
14. Column panel zone failure due to shear yielding or web plate buckling.

2.4 Detailing and Fabrication Practices

Proper detailing of an end-plate connection is necessary to ensure that the load path and geometric assumptions integrated into the design procedure are properly observed. It is recommended that beams with end-plate connections not be cambered since the resulting beam end rotation will cause field fit up problems. A critical aspect of end-plate connection design is the welding procedure used to install the welds that connect the end plate to the connected beam. As

observed in the 1994 Northridge earthquake, inadequate welding procedures and details used in the direct welded beam-to-column connections caused premature failure of the connection. The importance of proper weld detailing of end-plate connections is presented by Meng and Murray (1996,1997). They observed premature beam flange fractures in end-plate connections that utilized weld access holes to install the end plate to beam flange welds. The following are end-plate connection detailing guidelines and welding procedures that are required to satisfy the load path and geometric assumptions integrated into the design procedures.

Connection Detailing

Proper selection of the bolt layout dimensions is a critical part of end-plate connection design. Smaller bolt spacing will result in connections that are more economical than ones with larger bolt spacing. However, small bolt spacing can cause difficulties with fit-up and bolt tightening during erection. The three primary dimensions that must be selected when designing and detailing end-plate moment connections are: the bolt gage (g), bolt pitch to the flange (p_f), and bolt pitch to adjacent bolt row (p_b). The bolt gage and pitch distances are illustrated in Figure 2.7.

The bolt gage should be selected to allow for adequate clearance to install and tighten the connection bolts. In addition, for beam-to column connections, the gage must be large enough for the bolts to clear the fillets between the column web and flange. The “workable gage” (minimum gage) for a connection to a column flange is tabulated along with the section properties for each hot-rolled shape in Section 1 of the *Manual of Steel Construction* (AISC, 2001). Regardless of the flange width, the maximum gage dimension is limited to the width of the connected beam flange. This restriction is to ensure that a favorable load path between the beam flange and the connection bolts is provided.

The pitch to flange and pitch to adjacent bolt row distances should be selected to allow for adequate clearance to install and tighten the connection bolts. The bolt pitch to the flange distance, p_f , is the distance from the face of the flange to the centerline of the nearer bolt row. The absolute minimum pitch dimension for standard bolts is the bolt diameter plus $\frac{1}{2}$ in. for bolts up to 1 in. diameter, and the bolt diameter plus $\frac{3}{4}$ in. for larger diameter bolts. For tension control bolts, a larger pitch to flange dimension may be required because of wrench size.

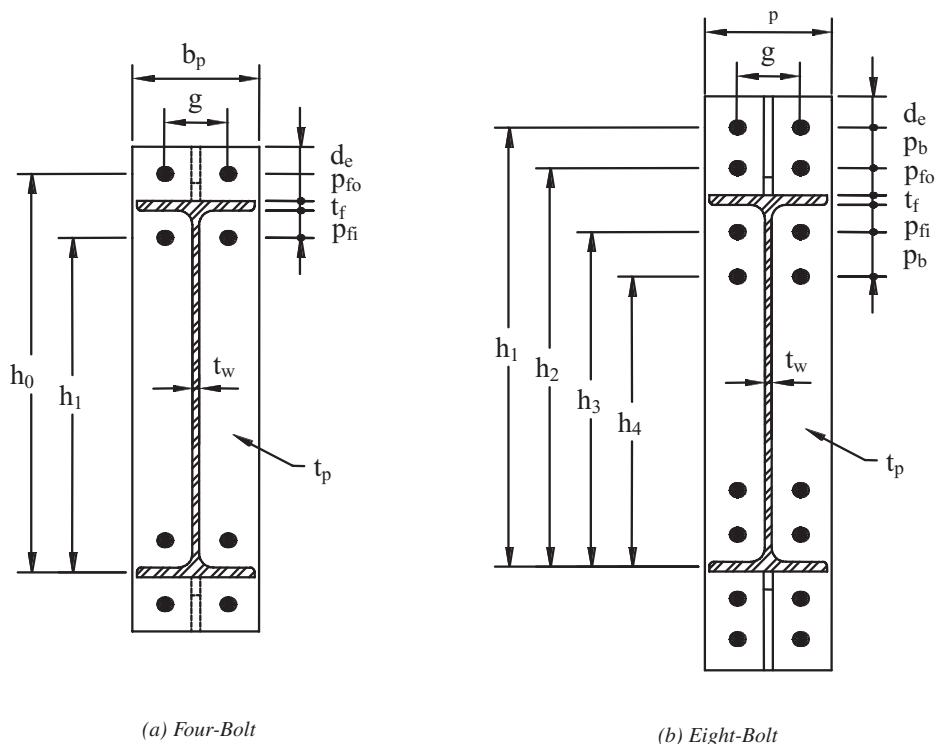


Fig. 2.7. End plate geometry.

The bolt pitch to adjacent bolt row, p_b , is the distance from the centerline of bolt row to the adjacent bolt row. The spacing of the bolt rows should be at least $2\frac{2}{3}$ times the bolt diameter. However, a distance of three times the bolt diameter is preferred (AISC, 1999).

The width of the end plate should be greater than or equal to the connected beam flange width. Typically, the width of the end plate is selected by adding 1 in. to the beam flange width and then rounding the width up or down to the closest standard plate width. The additional end plate width allows tolerance during fit-up of the end plate and an area for welding “runoff” in the fabrication shop. In design calculations, the effective end plate width should not be taken greater than the connected beam flange plus 1 in. This provision ensures that the excess end plate material outside the beam flange width, which may not be effective, is not considered in the end plate strength calculations.

The two extended stiffened end-plate connections, Figures 1.1(b) and (c), utilize a gusset plate welded between the connected beam flange and the end plate to stiffen the extended portion of the end plate. The stiffening of the end plate increases the strength and results in a thinner end plate when compared to an equivalent unstiffened connection. Use of the eight-bolt connection, Figure 1.1(c), may also eliminate the need for column stiffeners because of the wider distribution of the beam flange force at the column flange. The end plate stiffener acts like a portion of the

beam web to transfer part of the beam flange tension force to the end plate and then to the connection bolts. To ensure a favorable load path, the detailing of the stiffener geometry is very important.

Analytical and experimental studies have shown that a concentrated stress applied to an unsupported edge of a gusset plate is distributed out from that point towards the supported edge at an angle of approximately 30° . This force distribution model is commonly referred to as the “Whitmore Section”. The same force distribution model is applied to the detailing of the end-plate stiffeners. The portion of the flange force that is transferred to the stiffener is assumed to distribute into the stiffener plate at an angle of thirty degrees. Using this model the required length of the stiffener along the outside face of the beam flange is

$$L_{st} = \frac{h_{st}}{\tan 30^\circ} \quad (2.11)$$

where h_{st} is the height of the end plate from the outside face of the beam flange to the end of the end plate (see Figure 2.8).

To facilitate welding of the stiffener, the stiffener plates should be terminated at the beam flange and at the end of the end plate with landings approximately 1 in. long. The landings provide a consistent termination point for the stiffener plate and the welds. The stiffener should be clipped where it meets the beam flange and end plate to provide clearance between the stiffener and the beam flange weld. Figure 2.8 illustrates the recommended layout of the end-plate stiffener geometry.

The end-plate stiffener must have adequate strength to transfer a portion of the beam flange force from the beam flange to the bolts on the extended portion of the end plate. To provide a consistent load path through the end-plate connection, the end-plate stiffener should provide the same strength as the beam web. When the beam and end-plate stiffeners have the same material strengths, the thickness of the stiffeners should be greater than or equal to the beam web thickness. If the beam and end-plate stiffener have different material strengths, the thickness of the stiffener should be greater than the ratio of the beam-to-stiffener plate material yield stress times the beam web thickness.

Beam length and column depth tolerances are a concern in the fabrication and erection of structural steel moment frames utilizing end-plate moment connections. The end plates are welded to the beam or girder in the fabrication shop and the column flanges are drilled to match the end plate bolt pattern. This results in a connection with very little adjustment.

According to the *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2000) the allowable fabrication tolerance for the length of a beam connected on both

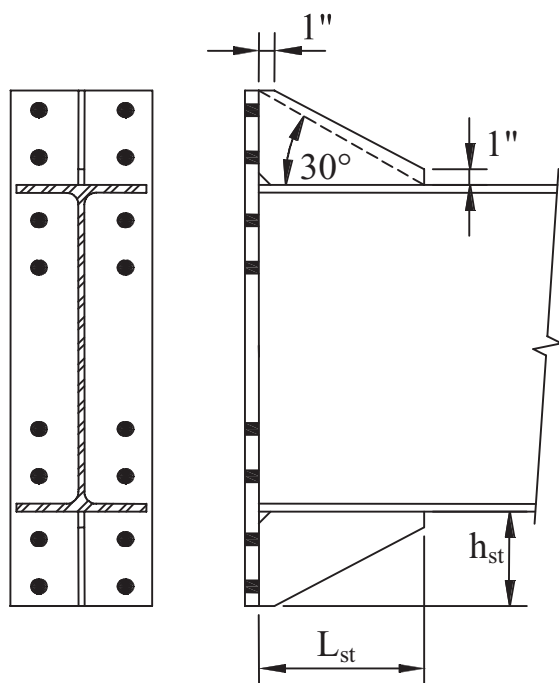


Fig. 2.8. End plate stiffener layout and geometry (SES).

ends is $\frac{1}{16}$ in. for members less than 30 ft. and $\frac{1}{8}$ in. for all others. The *Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling, ASTM A6* (ASTM, 2001) specifies that the maximum hot-rolled section depth variation and flange out of straightness tolerances are $\pm \frac{1}{8}$ in. and $\pm \frac{5}{16}$ in. respectively for sections less than or equal to 12 in. in depth and $\pm \frac{1}{8}$ in. and $\pm \frac{1}{4}$ in. for section depths greater than 12 in.

To solve the tolerance problem the beam or girder may be detailed and fabricated $\frac{3}{16}$ in. to $\frac{3}{8}$ in. short and then any gaps between the end plate and column flange filled using finger shims. Finger shims are thin steel plates, usually $\frac{1}{16}$ in. thick, that are cut to match the connection bolt pattern so that they can be inserted between the column flange and the end plate. Figure 2.9 illustrates the use of finger shims. A skewed column flange or end plate can be corrected by inserting more shims on one side of the connection than the other. Experimental tests have been performed with finger shims and no adverse consequences or differences in connection behavior were observed (Sumner and others 2000a).

Composite Slab Detailing

When beams and girders are connected to the concrete slab using headed shear studs, the composite action greatly increases the strength of the beams and girders. However, this additional strength is not considered in the design of the

members of the seismic force resisting moment frames (FEMA 1997). The assumption has been that the composite concrete slab will crack, the concrete will crush around the column, and the strength added by the composite slab will be reduced to an insignificant level before the large inelastic deformations of the beam will occur. This philosophy has been incorporated into the current design criteria for beam-to-column moment connections, which consider only the strength of the connected bare steel beams. However, it is possible for the composite slab to contribute to the strength of the connected beams unless proper detailing is used.

To eliminate the composite action of the slab and beam in the regions of the beam where plastic hinges are expected to form, the following slab and shear stud detailing is recommended (Sumner and Murray 2001):

- Shear studs should not be placed along the top flange of the connecting beams for a distance from the face of the column, one and a half times the depth of the connecting beam.
- Compressible expansion joint material, at least $\frac{1}{2}$ in. thick, should be installed between the slab and the column face.
- The slab reinforcement in the area within two times the depth of the connecting beam from the face of the column should be minimized.

These recommendations are based on engineering judgment and have not been substantiated for moment end-plate connections by testing. However, Yang and others (2003) have conducted tests of flange-welded connections subjected to positive moment and with composite beams. The concrete slab detailing was very similar to that recommended above and the tests were considered successful in that there was not a significant increase in bottom flange force.

Welding Procedures

The welding procedures outlined in this section are designed to provide welded connections between the connected beam and the end plate that can meet the demands of inelastic cyclic loading. Although not absolutely necessary, the same procedures are recommended for low seismic and wind controlled applications. The detailing and fabrication requirements have been developed from the experience of fabricators across the country and from experimental testing programs conducted at Virginia Polytechnic Institute over the past ten years. All welds specified in the forthcoming procedures should be made in accordance with the American Welding Society (AWS), *Structural Welding Code, AWS D1.1* (AWS, 2002). The welding electrodes used to make the welds specified in the procedures should conform to the

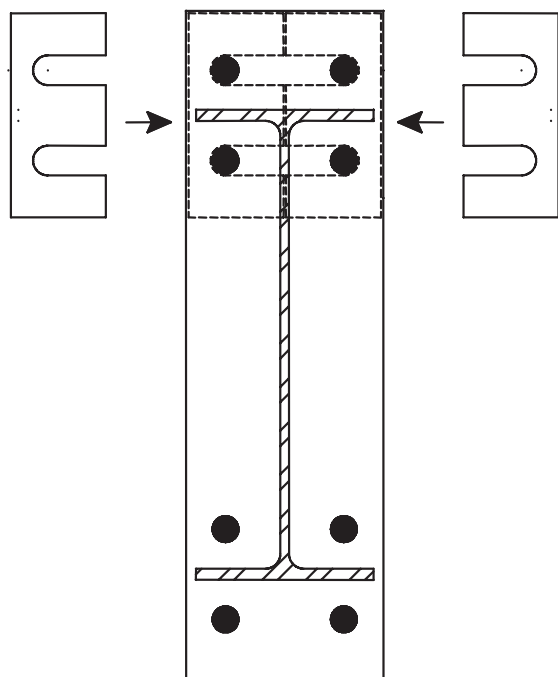


Fig. 2.9. Typical use of finger shims.

requirements of the *Seismic Provisions for Structural Steel Buildings* (AISC, 2002). The *Specification* requires that the weld filler metal have a minimum Charpy V-Notch (CVN) toughness of 20 ft-lbs at minus 20 degrees F. The procedures have also been published in the *Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications*, FEMA-353 (FEMA, 2000b).

The beam web to end-plate connection may be made using either fillet welds or complete joint penetration welds. The fillet welds should be sized to develop the full strength of the beam web in tension near the inside bolts (see Section 2.1). If the fillet weld size becomes large, a complete joint penetration weld may be more economical. The beam web to end-plate weld should be installed before beam flange to end-plate welds. This sequence is used to avoid inducing additional stresses in the beam flange to end-plate welds due to shrinkage of the web welds.

The beam flange to end-plate connection should be made using a CJP weld if the flange thickness is greater than $\frac{3}{8}$ in. Fillet welds on both sides of the beam flange may be acceptable for thinner flanges. The CJP weld should be

made such that the root of the weld is on the beam web side of the flange. The flange weld is similar to the AWS pre-qualified TC-U4b-GF with a full depth 45-degree bevel and a minimal root opening. The root of the weld should be backed by a $\frac{5}{16}$ in. fillet weld installed on the web side of the flange. Most importantly, weld access holes in the beam web should not be used. Once the backing weld is installed, the root of the groove weld should be backgouged to solid weld metal and the groove weld placed. One exception to this procedure is welds in the area of the flange directly above the beam web, backgouging of the root is not required. This exception is necessary because, in the area above the beam web, the backing fillet weld is not present. A summary of the welding procedure is presented in Figure 2.10.

End-Plate Stiffener Welds

The connection of the end-plate stiffener to the outside face of the beam flange and to the face of the end plate may be made using complete joint penetration groove welds or fillet welds. The CJP welds can be single or double bevel groove welds. Fillet welds should be used only if the stiffener plate is $\frac{3}{8}$ in. or less in thickness.

- Prepare the flanges of the beam with a 45 degree, full depth bevel.
- Fit up the end-plate and beam with a minimum root opening.
- Preheat the specimens as required by AWS specifications.
- Prepare the surfaces for welding as required by AWS specifications.
- Place the web welds (1).
- Place the $\frac{5}{16}$ in. backing fillet welds on the beam web side of the beam flanges (2).
- Backgouge the root of the bevel to remove any contaminants from the $\frac{5}{16}$ in. backer fillet welds (3).
- Place the flange groove welds (AWS TC-U4b-GF).

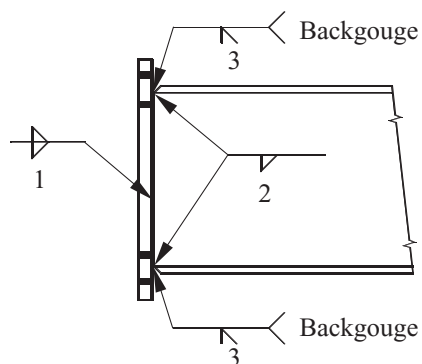


Fig. 2.10. Summary of recommended welding procedure.

Chapter 3

Design Procedure

3.1 Overview

The four primary design parameters for the design of extended end-plate moment connections subject to cyclic loading are:

1. The required connection design moment
2. Connection bolt strength
3. End plate strength
4. Column flange bending strength

Design procedures for the design of the four bolt unstiffened (4E, Figure 1.1 (a)), four-bolt stiffened (4ES, Figure 1.1(b)), and eight-bolt stiffened (8ES, Figure 1.1(c)) end-plate moment connections follow. The procedures use yield-line theory for determination of the end plate strength and a simplified method to determine the bolt forces as described in Chapter 2.

Tables 3.1, 3.2, and 3.3 at the end of this chapter include expressions for the end plate flexural strength and no prying bolt moment strength for the 4E, 4ES, and 8ES moment

end-plate connections. Tables 3.4 and 3.5 have similar expressions for the corresponding unstiffened and stiffened column flange flexural strengths. The end plate design flexural strength, ϕM_{pl} , includes the distance s . The yield line patterns in the tables show s measured from the innermost tension bolt row and, for the stiffened connections, from the outermost tension bolt row. If a large inside pitch distance, p_{fi} , is used, a horizontal yield line between the beam flange and the first inner bolt row may form. Therefore, if $p_{fi} > s$, then p_{fi} is set equal to s when calculating the flexural strength of the end plate.

3.2 Design Steps

The following steps are recommended to design a bolted end-plate moment connection subject to cyclic/seismic forces. If the connection is subject to other than cyclic/seismic forces, the required connection moment, M_{uc} , in Step 1 should be determined from the frame analysis. Alternately, the design procedures in the AISC/MBMA Design Guide Series 16 (Murray and Shoemaker 2002) may be used.

Connection geometry is shown in Figure 3.1, 3.2, and 3.3 for the 4E, 4ES, and 8ES connections, respectively.

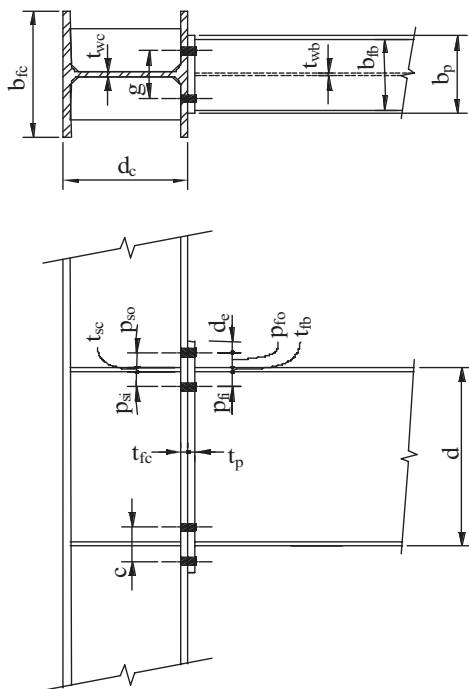


Fig. 3.1. Four-bolt unstiffened extended, 4E, end plate geometry.

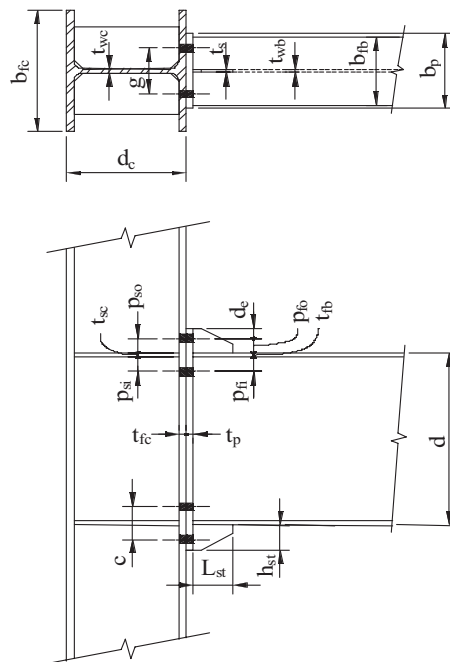


Fig. 3.2. Four-bolt stiffened extended, 4ES, end plate geometry.

End Plate and Bolt Design

1. Determine the sizes of the connected members (beams and column) and compute the moment at the face of the column, M_{uc} .

$$M_{uc} = M_{pe} + V_u L_p \quad (3.1)$$

where

$$M_{pe} = 1.1 R_y F_y Z_x \quad (3.2)$$

V_u = shear at the plastic hinge

L_p = distance from the face of the column to the plastic hinge

for unstiffened connection (4E):

$$= \min \left\{ \begin{array}{l} d/2 \\ 3 b_f \end{array} \right. \quad (3.3)$$

for stiffened connections (4ES, 8ED):

$$= L_{st} + t_p \quad (3.4)$$

R_y = the ratio of the expected yield strength to the specified minimum yield strength

= 1.1 for $F_y = 50$ ksi, and 1.5 for $F_y = 36$ ksi (from AISC Seismic Provisions, 2002),

d = depth of the connecting beam,

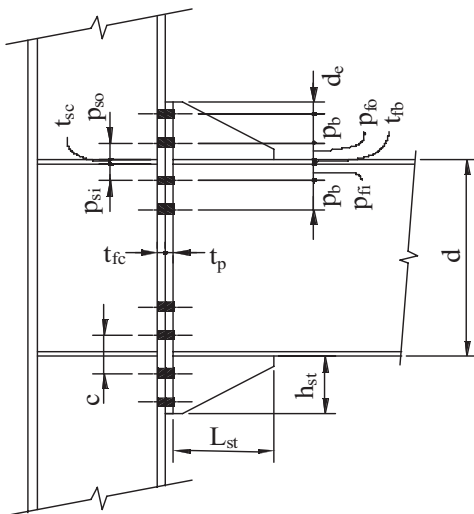
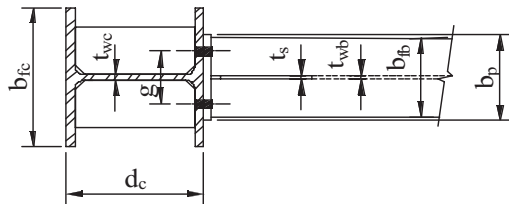


Fig. 3.3. Eight-bolt stiffened extended, 8ES, end plate geometry.

b_f = width of the beam flange,

L_{st} = length of the end plate stiffener, and

t_p = thickness of the end plate.

2. Select one of the three end-plate moment connection configurations and establish preliminary values for the connection geometry (g , p_{fv} , p_{for} , p_b , etc.) and bolt grade.
3. Determine the required bolt diameter, $d_{b \text{ Req'd}}$, using one of the following expressions.

For four-bolt connections (4E, 4ES):

$$d_{b \text{ Req'd}} = \sqrt{\frac{2 M_{uc}}{\pi \phi F_t (h_0 + h_1)}} \quad (3.5)$$

For eight-bolt connections (8ES):

$$d_{b \text{ Req'd}} = \sqrt{\frac{2 M_{uc}}{\pi \phi F_t (h_1 + h_2 + h_3 + h_4)}} \quad (3.6)$$

where

ϕ = 0.75

F_t = specified LRFD bolt tensile strength (90 ksi for ASTM A325 bolts and 113 ksi for ASTM A490 bolts),

h_i = distance from the centerline of the beam compression flange to the centerline of the i^{th} tension bolt row.

Equations 3.5 and 3.6 were derived by equating the factored moment at the face of the column, M_{uc} , equal to the no prying bolt strength moment, M_{np} , and solving for the required bolt diameter.

4. Select a trial bolt diameter, d_b , greater than that required in Step 3 and calculate the no prying bolt moment strength, M_{np} .

For four-bolt connections (4E, 4ES):

$$M_{np} = 2 P_t (h_0 + h_1) \quad (3.7)$$

For eight-bolt connections (8ES):

$$M_{np} = 2 P_t (h_1 + h_2 + h_3 + h_4) \quad (3.8)$$

where

P_t = Bolt tensile strength

$$= F_t A_b = F_t \left(\frac{\pi d_b^2}{4} \right) \quad (3.9)$$

A_b = the nominal cross sectional area of the selected bolt diameter

d_b = selected nominal bolt diameter

5. Determine the required end plate thickness, $t_{p \text{ Req'd}}$.

$$t_{p \text{ Req'd}} = \sqrt{\frac{1.11 \phi M_{np}}{\phi_b F_{yp} Y_p}} \quad (3.10)$$

where

$$\begin{aligned} \phi &= 0.75 \\ \phi_b &= 0.9 \\ F_{yp} &= \text{the end plate material yield strength} \\ Y_p &= \text{the end plate yield line mechanism parameter from Table 3.1, 3.2, or 3.3.} \end{aligned}$$

Equation 3.10 was derived by equating 111% (1/0.9 × 100%) of the no prying bolt moment strength to the end plate flexural strength and solving for the required end plate thickness.

6. Select an end plate thickness greater than the required value.
7. Calculate the factored beam flange force

$$F_{fu} = \frac{M_{uc}}{d - t_{fb}} \quad (3.11)$$

d = depth of the beam

8. Check shear yielding resistance of the extended portion of the four-bolt extended unstiffened end plate (4E):

$$F_{fu} / 2 < \phi R_n = \phi 0.6 F_{yp} b_p t_p \quad (3.12)$$

where

$$\begin{aligned} \phi &= 0.9 \\ b_p &= \text{width of the end plate} \end{aligned}$$

If Inequality 3.12 is not satisfied, increase the end plate thickness until it is satisfied.

9. Check shear rupture resistance of the extended portion of the end plate in the four-bolt extended unstiffened (4E):

$$F_{fu} / 2 < \phi R_n = \phi 0.6 F_{up} A_n \quad (3.13)$$

where

$$\begin{aligned} \phi &= 0.75 \\ F_{up} &= \text{minimum tensile strength of the end plate} \\ A_n &= \text{net area of the end plate} = [b_p - 2(d_b + 1/8)] t_p \text{ when standard holes are used} \\ d_b &= \text{diameter of the bolts} \end{aligned} \quad (3.14)$$

If Inequality 3.13 is not satisfied, increase the end plate thickness until it is satisfied.

10. If using either the four-bolt extended stiffened (4ES) or eight-bolt extended stiffened (8ES) connection, select the end plate stiffener thickness and design the stiffener-to-beam flange and stiffener-to-end plate welds.

$$t_{s, req'd} = t_{wb} \left(\frac{F_{yb}}{F_{ys}} \right) \quad (3.15)$$

where

$$\begin{aligned} t_{wb} &= \text{thickness of the beam web} \\ F_{yb} &= \text{specified minimum yield stress of beam material} \\ F_{ys} &= \text{specified minimum yield stress of stiffener material} \end{aligned}$$

The stiffener geometry should be selected in accordance with the recommendations presented in Section 2.4. In addition, to prevent local buckling of the stiffener plate the following width-to-thickness criterion should be satisfied.

$$\frac{h_{st}}{t_s} \leq 0.56 \sqrt{\frac{E}{F_{ys}}} \text{ or } t_s \geq 1.79 h_{st} \sqrt{\frac{F_{ys}}{E}} \quad (3.16)$$

where

$$h_{st} = \text{the height of the stiffener}$$

The stiffener-to-beam flange and stiffener-to-end-plate welds should be designed to develop the stiffener plate in shear at the beam flange and in tension at the end plate. Either fillet or CJP welds are suitable for the beam flange welds. If the stiffener plate thickness is greater than $\frac{3}{8}$ in., CJP welds should be used for the stiffener-to-end plate weld. Otherwise, fillet welds may be used.

11. The bolt shear rupture strength of the connection is conservatively assumed to be provided by the bolts at one (compression) flange, thus

$$V_u < \phi R_n = \phi (n_b) F_v A_b \quad (3.17)$$

where

$$\begin{aligned} \phi &= 0.75 \\ n_b &= \text{number of bolts at the compression flange, four for 4ES, and eight for 8ES connections} \\ F_v &= \text{nominal shear strength of bolts from Table J3.2 of the AISC LRFD Specification (AISC, 1999)} \\ A_b &= \text{nominal gross area of bolt} \end{aligned}$$

If Inequality 3.17 is not satisfied, increase the bolt diameter or number of bolts.

12. Check bolt bearing / tear out failure of the end plate and column flange:

$$V_u < \phi R_n = (n_i) \phi R_n \text{ (Inner Bolts)} + (n_o) \phi R_n \text{ (Outer Bolts)} \quad (3.18)$$

where

$$\begin{aligned} \phi &= 0.75 \\ n_i &= \text{number of inner bolts (two for 4E and 4ES, and four for 8ES connections)} \\ n_o &= \text{number of outer bolts (two for 4E and 4ES, and four for 8ES connections)} \\ R_n &= 1.2 L_c t F_u < 2.4 d_b t F_u \text{ for each bolt} \quad (3.19) \\ L_c &= \text{clear distance, in the direction of force, between the edge of the hole and the edge of the adjacent hole or edge of the material} \\ t &= \text{end plate or column flange thickness} \\ F_u &= \text{specified minimum tensile strength of end plate or column flange material} \\ d_b &= \text{diameter of the bolt} \end{aligned}$$

If Inequality 3.18 is not satisfied, increase the end plate thickness.

13. Design the flange to end plate and web to end plate welds.

Column Side Design

14. Check the column flange for flexural yielding

$$t_{fc \text{ Req'd}} = \sqrt{\frac{1.11 \phi M_{np}}{\phi_b F_{yc} Y_c}} \leq t_{fc} \quad (3.20)$$

where

$$\begin{aligned} \phi_b &= 0.9 \\ F_{yc} &= \text{specified yield stress of column flange material} \\ Y_c &= \text{unstiffened column flange yield line mechanism parameter from Table 3.4 or 3.5.} \\ t_{fc} &= \text{column flange thickness} \end{aligned}$$

If Inequality 3.20 is not satisfied, increase the column size or add web stiffeners (continuity plates).

If stiffeners are added, Inequality 3.20 must be checked using Y_c for the stiffened column flange from Tables 3.4 and 3.5.

15. If stiffeners are required for column flange flexural yielding, determine the required stiffener force.

The column flange flexural design strength is

$$\phi M_{cf} = \phi_b F_{yc} Y_c t_{fc}^2 \quad (3.21)$$

Therefore, the equivalent column design force is

$$\phi R_n = \frac{\phi M_{cf}}{(d - t_{fb})} \quad (3.22)$$

Using ϕR_n , the required force for stiffener design is determined in Step 19.

16. Check the local column web yielding strength of the unstiffened column web at the beam flanges.

$$\text{Strength requirement: } \phi R_n > F_{fu} \quad (3.23)$$

$$\phi R_n = \phi C_t (6k_c + N + 2t_p) F_{yc} t_{wc} \quad (3.24)$$

where

$$\begin{aligned} \phi &= 1.0 \\ C_t &= 0.5 \text{ if the distance from the column top to the top face of the beam flange is less than the depth of the column} \\ &= 1.0 \text{ otherwise} \\ k_c &= \text{distance from outer face of the column flange to web toe of fillet (design value)} \\ N &= \text{thickness of beam flange plus two times the groove weld reinforcement leg size} \\ t_p &= \text{end plate thickness} \\ F_{yc} &= \text{specified yield stress of the column web material} \\ t_{wc} &= \text{column web thickness} \\ d &= \text{depth of the beam} \\ t_{fb} &= \text{thickness of beam flange} \end{aligned}$$

If the strength requirement ($\phi R_n > F_{fu}$) is not satisfied, then column web stiffener plates (continuity plates) are required.

17. Check the unstiffened column web buckling strength at the beam compression flange.

Strength requirement:

$$\phi R_n > F_{fu} \quad (3.25)$$

When F_{fu} is applied a distance greater than or equal to $d_c / 2$ from the end of the column

$$\phi R_n = \frac{\phi 24 t_{wc}^3 \sqrt{E F_{yc}}}{h} \quad (3.26)$$

When F_{fu} is applied a distance less than $d_c / 2$ from the end of the column

$$\phi R_n = \frac{\phi 12 t_{wc}^3 \sqrt{E F_{yc}}}{h} \quad (3.27)$$

where

- $\phi = 0.9$
 $h =$ clear distance between flanges less the fillet or corner radius for rolled shapes; clear distance between flanges when welds are used for built-up shapes

If the strength requirement ($\phi R_n > F_{fu}$) is not satisfied, then column web stiffener plates (continuity plates) are required.

18. Check the unstiffened column web crippling strength at the beam compression flange.

Strength requirement:

$$\phi R_n > F_{fu} \quad (3.28)$$

When F_{fu} is applied a distance greater than or equal to $d_c / 2$ from the end of the column

$$\phi R_n = \phi 0.80 t_{wc}^2 \left[1 + 3 \left(\frac{N}{d_c} \right) \left(\frac{t_{wc}}{t_{fc}} \right)^{1.5} \right] \sqrt{\frac{E F_{yc} t_{fc}}{t_{wc}}} \quad (3.29)$$

When F_{fu} is applied a distance less than $d_c / 2$ from the end of the column

For $N/d_c < 0.2$,

$$\phi R_n = \phi 0.40 t_{wc}^2 \left[1 + 3 \left(\frac{N}{d_c} \right) \left(\frac{t_{wc}}{t_{fc}} \right)^{1.5} \right] \sqrt{\frac{E F_{yc} t_{fc}}{t_{wc}}} \quad (3.30)$$

For $N/d_c > 0.2$,

$$\phi R_n = \phi 0.40 t_{wc}^2 \left[1 + \left(\frac{4N}{d_c} - 0.2 \right) \left(\frac{t_{wc}}{t_{fc}} \right)^{1.5} \right] \sqrt{\frac{E F_{yc} t_{fc}}{t_{wc}}} \quad (3.31)$$

where

- $\phi = 0.75$
 $N =$ thickness of beam flange plus 2 times the groove weld reinforcement leg size
 $d_c =$ overall depth of the column

If the strength requirement ($\phi R_n > F_{fu}$) is not satisfied, then column web stiffener plates (continuity plates) are required.

19. If stiffener plates are required for any of the column side limit states, the required strength is

$$F_{su} = F_{fu} - \min \phi R_n \quad (3.32)$$

where $\min \phi R_n =$ the minimum design strength value from Steps 15 (column flange bending), 16 (column web yielding), 17 (column web buckling), and 5 (column web crippling).

The design of the column stiffeners (continuity plates) requires additional consideration. Details of the design requirements are provided in *AISC Design Guide 13 Wide-Flange Column Stiffening at Moment Connections—Wind and Seismic Applications* (Carter, 1999).

20. Check shear yielding and plate buckling strength of the column web panel zone. For further information, see the *AISC Design Guide 13, Wide-Flange Column Stiffening at Moment Connections—Wind and Seismic Applications* (Carter, 1999) and the *Seismic Provisions for Structural Steel Buildings* (AISC, 2002).

3.3 Analysis Procedure

For a given end plate geometry, bolt diameter, beam and column geometry, and material properties, the design moment strength, ϕM_n , can be determined using the following procedure:

- Calculate the end plate bending strength, $\phi_b M_{pl}$, column flange bending strength, $\phi_b M_{cf}$, and the no-prying bolt tension rupture strength, ϕM_{np} , using the equations presented in the summary tables (Tables 3.1 through 3.5).
- Determine the behavior, ‘thick’ or ‘thin’, of the end plate and column flange using the following.

For the end plate

$$\text{If } M_{pl} > 1.1 M_{np} \rightarrow \text{Thick plate} \quad (3.33)$$

$$\text{If } M_{pl} < 1.1 M_{np} \rightarrow \text{Thin plate} \quad (3.34)$$

For the column flange

$$\text{If } M_{cf} > 1.1 M_{np} \rightarrow \text{Thick flange} \quad (3.35)$$

$$\text{If } M_{cf} < 1.1 M_{np} \rightarrow \text{Thin flange} \quad (3.36)$$

If the end plate and the column flange are exhibiting thick plate behavior, then the connection design strength, ϕM_n , is equal to the no prying bolt strength, ϕM_{np} .

If the end plate and/or the column flange are exhibiting thin plate behavior, then the connection does not comply with the requirements of the design procedure. The connection strength cannot be calculated using the procedures outlined herein because an additional limit state, bolt rupture with prying, is induced by the thin plate behavior.

- c. Procedures for determining the strength of end plates that exhibit thin plate behavior are available in the *AISC/MBMA Design Guide 16 Flush and Extended Multiple-Row Moment End-Plate Connections* (Murray and Shoemaker, 2002).

3.4 Limitations

The design and analysis procedures presented in this guide were verified through experimental tests, Packer and Morris (1977), Ghassemieh (1983), Morrison and others (1985),

Tsai and Popov (1990), Ghobarah and others (1990, 1992), Abel and Murray (1992a), Borgsmiller and others (1995), Meng and Murray (1996), Ryan and Murray (1999), Adey and others (1997), Sumner and others (2000). Geometric parameters of the connections were varied among the test configurations. Significant variance outside the ranges of geometric relationships could affect the failure mechanism and thus the predicted strength. The applicable range of tested parameters for cyclic/seismic applications are shown in Table 3.6 and for other applications in Table 3.7.

Table 3.1. Summary of Four-Bolt Extended Unstiffened End Plate Design Strength

End Plate Geometry and Yield Line Pattern		Bolt Force Model	
End Plate	$\phi M_{pl} = \phi_b F_{yp} t_p^2 Y_p$ $Y_p = \frac{b_p}{2} h_1 \left(\frac{1}{p_{fi}} + \frac{1}{s} \right) + h_0 \left(\frac{1}{p_{fo}} - \frac{1}{2} + \frac{2}{g} \right) h_1 (p_{fi} + s)$ $s = \frac{1}{2} \sqrt{b_p g}$	$\phi_b = 0.90$	Note: If $p_{fi} > s$, use $p_{fi} = s$
Bolt Rupture	$\phi M_{np} = \phi 2 P_t (h_o + h_1)$	$\phi = 0.75$	

Table 3.2. Summary of Four-Bolt Extended Stiffened End Plate Design Strength

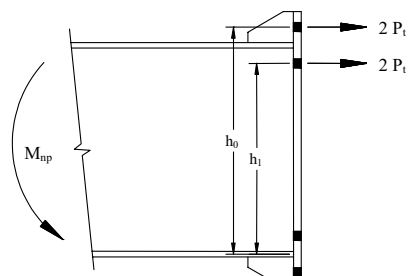
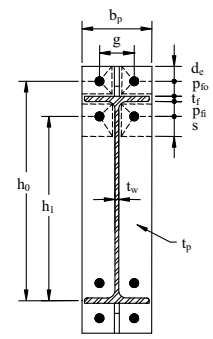
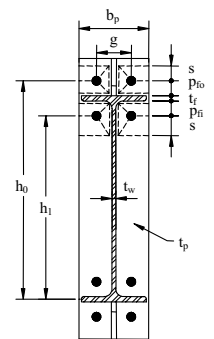
End Plate Geometry and Yield Line Pattern		Bolt Force Model	
Case 1 ($d_e \leq s$)	Case 2 ($d_e > s$)		
			
End Plate	$\phi M_{pl} = \phi_b F_{yp} t_p^2 Y_p$ Case 1 ($d_e < s$) $Y_p = \frac{b_p}{2} h_1 \left(\frac{1}{p_{fi}} + \frac{1}{s} \right) + h_0 \left(\frac{1}{p_{fo}} + \frac{1}{2s} \right) + \frac{2}{g} h_1 (p_{fi} + s) + h_0 (d_e + p_{fo})$ Case 2 ($d_e > s$) $Y_p = \frac{b_p}{2} h_1 \left(\frac{1}{p_{fi}} + \frac{1}{s} \right) + h_0 \left(\frac{1}{s} + \frac{1}{p_{fo}} \right) + \frac{2}{g} h_1 (p_{fi} + s) + h_0 (s + p_{fo})$ $s = \frac{1}{2} \sqrt{b_p g}$ $\phi_b = 0.90$ Note: If $p_{fi} > s$, use $p_{fi} = s$		
Bolt Rupture	$\phi M_{np} = \phi 2 P_t (h_o + h_1)$ $\phi = 0.75$		

Table 3.3. Summary of Eight-Bolt Extended Stiffened End Plate Design Strength

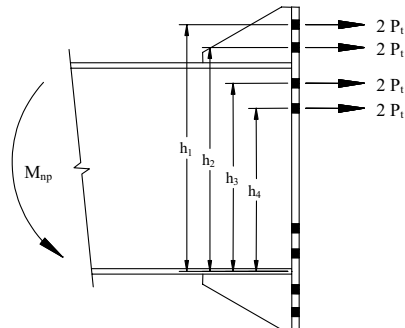
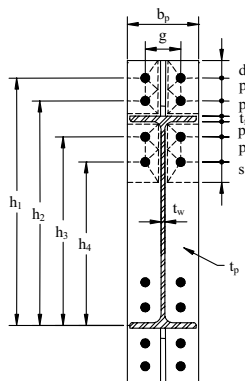
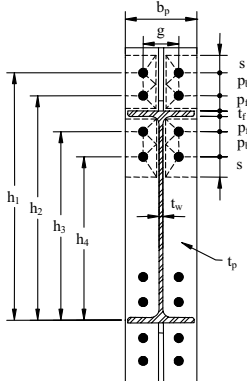
End Plate Geometry and Yield Line Pattern		Bolt Force Model
Case 1 ($d_e \leq s$)	Case 2 ($d_e > s$)	
		
End Plate	$\phi M_{pl} = \phi_b F_{yp} t_p^2 Y_p$ Case 1 ($d_e < s$) $Y_p = \frac{b_p}{2} h_1 \frac{1}{2d_e} + h_2 \frac{1}{p_{fo}} + h_3 \frac{1}{p_{fi}} + h_4 \frac{1}{s} + \frac{2}{g} h_1 d_e + \frac{p_b}{4} + h_2 p_{fo} + \frac{3p_b}{4} + h_3 p_{fi} + \frac{p_b}{4} + h_4 s + \frac{3p_b}{4} + p_b^2 + g$ Case 2 ($d_e > s$) $Y_p = \frac{b_p}{2} h_1 \frac{1}{s} + h_2 \frac{1}{p_{fo}} + h_3 \frac{1}{p_{fi}} + h_4 \frac{1}{s} + \frac{2}{g} h_1 s + \frac{p_b}{4} + h_2 p_{fo} + \frac{3p_b}{4} + h_3 p_{fi} + \frac{p_b}{4} + h_4 s + \frac{3p_b}{4} + p_b^2 + g$ $s = \frac{1}{2} \sqrt{b_p g}$ $\phi_b = 0.90$ Note: If $p_{fi} > s$, use $p_{fi} = s$	
Bolt Rupture	$\phi M_{np} = \phi 2 P_t (h_1 + h_2 + h_3 + h_4)$ $\phi = 0.75$	

Table 3.4. Summary of Four-Bolt Extended Column Flange Strength

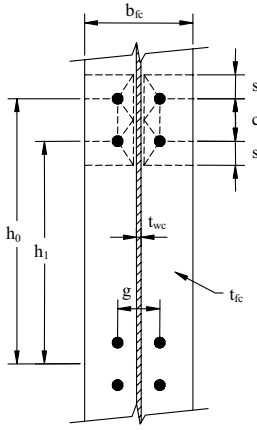
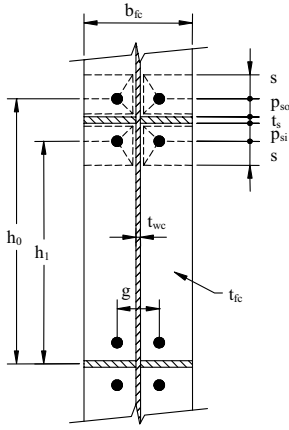
Unstiffened Column Flange Geometry and Yield Line Pattern	Stiffened Column Flange Geometry and Yield Line Pattern
	
Unstiffened Column Flange	$\phi M_{cf} = \phi_b F_{yc} t_{fc}^2 Y_c$ $Y_c = \frac{b_{fc}}{2} h_1 \left(\frac{1}{s} + h_0 \frac{1}{s} + \frac{2}{g} h_1 s + \frac{3c}{4} + h_0 s + \frac{c}{4} + \frac{c^2}{2} + \frac{g}{2} \right)$ $s = \frac{1}{2} \sqrt{b_{fc} g}$ $\phi_b = 0.90$
Stiffened Column Flange	$\phi M_{cf} = \phi_b F_{yc} t_{fc}^2 Y_c$ $Y_c = \frac{b_{fc}}{2} h_1 \left(\frac{1}{s} + \frac{1}{p_{si}} + h_0 \left(\frac{1}{s} + \frac{1}{p_{so}} \right) + \frac{2}{g} h_1 (s + p_{si}) + h_0 (s + p_{so}) \right)$ $s = \frac{1}{2} \sqrt{b_{fc} g}$ $\phi_b = 0.90$ Note: If $p_{si} > s$, use $p_{si} = s$

Table 3.5. Summary of Eight-Bolt Extended Stiffened Column Flange Design Strength

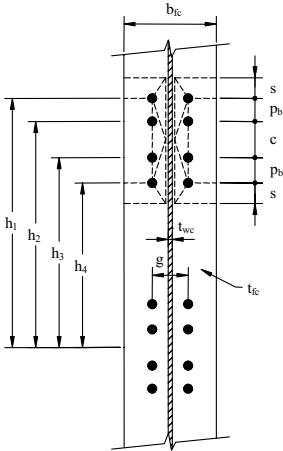
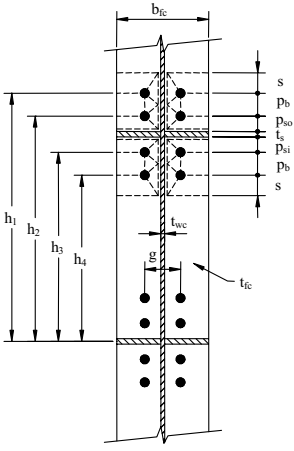
Unstiffened Column Flange Geometry and Yield Line Pattern	Stiffened Column Flange Geometry and Yield Line Pattern
	
Unstiffened Column Flange	$\phi M_{cf} = \phi_b F_{yc} t_{fc}^2 Y_c$ $Y_c = \frac{b_{fc}}{2} h_1 \frac{1}{s} + h_4 \frac{1}{s} + \frac{2}{g} h_1 p_b + \frac{c}{2} + s + h_2 \frac{p_b}{2} + \frac{c}{4} + h_3 \frac{p_b}{2} + \frac{c}{2} + h_4 (s) + \frac{g}{2}$ $s = \frac{1}{2} \sqrt{b_{fc} g}$ $\phi_b = 0.90$
Stiffened Column Flange	$\phi M_{cf} = \phi_b F_{yc} t_{fc}^2 Y_c$ $Y_c = \frac{b_{fc}}{2} h_1 \frac{1}{s} + h_2 \frac{1}{p_{so}} + h_3 \frac{1}{p_{si}} + h_4 \frac{1}{s} + \frac{2}{g} h_1 s + \frac{p_b}{4} + h_2 p_{so} + \frac{3p_b}{4} + h_3 p_{si} + \frac{p_b}{4} + h_4 s + \frac{3p_b}{4} + p_b^2 + g$ $s = \frac{1}{2} \sqrt{b_{fc} g}$ $\phi_b = 0.90$ Note: If $p_{si} > s$, use $p_{si} = s$

Table 3.6. Range of Tested Parameters (Cyclic Tests)

Parameter	Four-Bolt Unstiffened		Four-Bolt Stiffened		Eight-Bolt Stiffened	
	Maximum (in.)	Minimum (in.)	Maximum (in.)	Minimum (in.)	Maximum (in.)	Minimum (in.)
t_p	2.25	0.50	1.375	0.50	2.50	0.75
b_p	10.625	7.00	10.625	10.625	15.0	9.00
g	6.00	4.00	6.00	3.25	6.00	5.00
p_f	4.50	1.50	5.375	1.625	2.00	1.625
p_b	-	-	-	-	3.75	3.50
d	55.0	25.0	24.0	13.75	36.0	18.375
t_f	0.75	0.375	0.75	0.375	1.00	0.625
b_f	9.25	6.00	9.00	6.00	12.25	7.625
d_b	1.25	0.875	1.25	1.00	1.25	1.125

Table 3.7. Range of Tested Parameters (Monotonic Tests)

Parameter	Four-Bolt Unstiffened		Four-Bolt Stiffened		Eight-Bolt Stiffened	
	Maximum (in.)	Minimum (in.)	Maximum (in.)	Minimum (in.)	Maximum (in.)	Minimum (in.)
t_p	2.25	0.375	1.375	0.375	2.50	0.75
b_p	10.625	5.00	10.625	8.00	15.0	9.00
g	7.00	2.50	6.00	2.75	6.00	5.00
p_f	4.50	1.25	5.375	1.00	2.00	1.375
p_b	-	-	-	-	3.75	2.75
d	63.875	10.0	24.0	13.75	36.0	18.375
t_f	1.00	0.25	0.75	0.375	1.00	0.625
b_f	10.25	4.00	9.00	6.00	12.25	7.625
d_b	1.25	0.50	1.25	0.625	1.25	0.875

Chapter 4

Design Examples

4.1 Scope

The following examples illustrate design procedures for the (1) four-bolt extended unstiffened (4E), (2) four-bolt extended stiffened (4ES), and eight-bolt extended stiffened (8ES) end-plate connections. Both beam side and column side calculations are illustrated. Two examples are provided for the 4E connection: one is for cyclic/seismic design and the second for wind/gravity loading. Both beam side and column side calculations are illustrated. The connections are symmetric to accommodate load reversal, which is necessary for the cyclic/seismic designs but may not be necessary for the wind/gravity loading. Shear forces are assumed to have been determined from analysis.

4.2 Four-Bolt Unstiffened Extended (4E) End-Plate Connection

4E Example A

Using cyclic/seismic loading, a four-bolt extended unstiffened (4E) end-plate connection is to be designed to connect a W21×55 beam to a W14×109 column. The beam and column material are ASTM A992 steel and the end plate is ASTM A572 Gr. 50 steel. ASTM A490 bolts are to be used. The required shear resistance, V_u , is 40 kips.

Beam:

W21×55

$$d_b = 20.8 \text{ in.}$$

$$t_{wb} = 0.375 \text{ in.}$$

$$b_{fb} = 8.22 \text{ in.}$$

$$t_{fb} = 0.522 \text{ in.}$$

$$k_b = 1.02 \text{ in.}$$

Workable Gage = 5½ in.

$$Z_{xb} = 126 \text{ in.}^3$$

$$F_{yb} = 50 \text{ ksi (ASTM A992 steel)}$$

$$F_{ub} = 65 \text{ ksi}$$

Column:

W14×109

$$d_c = 14.3 \text{ in.}$$

$$t_{wc} = 0.525 \text{ in.}$$

$$b_{fc} = 14.6 \text{ in.}$$

$$t_{fc} = 0.860 \text{ in.}$$

$$k_c = 1.46 \text{ in.}$$

$$(h/t_w)_c = 21.7$$

Workable Gage = 5½ in.

$$Z_{xc} = 447 \text{ in.}^3$$

$$F_{yc} = 50 \text{ ksi (ASTM A992 steel)}$$

$$F_{uc} = 65 \text{ ksi}$$

Bolts:

ASTM A490

See Figure 3.1 for definition of *connection geometry*.

Beam Side Design

1. Connection Design Moment

$$M_{pe} = 1.1 R_y F_y Z_x \quad (3.2)$$

$$= 1.1(1.1)(50)(126)$$

$$= 7623 \text{ k-in.}$$

Location of Plastic Hinge

$$L_p = \min \begin{cases} d/2 = 20.8/2 = 10.4 \text{ in.} \\ 3b_{fb} = 3(8.22) = 24.66 \text{ in.} \end{cases} \quad (3.3)$$

$$= 10.4 \text{ in.}$$

Moment at the Face of Column (Connection Design Moment)

$$M_{uc} = M_{pe} + V_u L_p \quad (3.1)$$

$$= 7623 + 40(10.4)$$

$$= 8039 \text{ k-in.}$$

2. Select Connection Configuration: Four-Bolt Extended Unstiffened

Assumed Geometric Design Data

$$b_p \approx b_f + 1 \text{ in.} = 8.22 + 1 = 9.22 \text{ in.} \Rightarrow \text{Use } b_p = 9.0 \text{ in.}$$

$$g = 5\frac{1}{2} \text{ in. (same as beam and column "workable gage")}$$

$$p_{fi} = 2 \text{ in.}$$

$$p_{fo} = 2 \text{ in.}$$

$$d_e = 1\frac{5}{8} \text{ in.}$$

$$F_{yp} = 50 \text{ ksi}$$

$$F_{up} = 65 \text{ ksi (ASTM A572 Gr. 50 steel)}$$

$$F_t = 113 \text{ ksi (ASTM A490 bolts)}$$

Using assumed dimensions,

$$h_0 = 20.8 + 2 - \frac{0.522}{2} = 22.54 \text{ in.}$$

$$h_1 = 20.8 - 0.522 - 2 - \frac{0.522}{2} = 18.02 \text{ in.}$$

3. Determine the Required Bolt Diameter (ASTM A490)

$$\begin{aligned} d_{b \text{ Req'd}} &= \sqrt{\frac{2M_{uc}}{\pi \phi F_t (h_0 + h_1)}} \\ &= \sqrt{\frac{2(8039)}{\pi (0.75)(113)(22.54 + 18.02)}} \\ &= 1.22 \text{ in.} \end{aligned} \quad (3.5)$$

4. Select Trial Bolt Diameter and Calculate the No Prying Bolt Moment

Use $d_b = 1\frac{1}{4}$ in. (ASTM A490)

Bolt Tensile Strength

$$P_t = F_t A_b = 113 \left(\frac{\pi (1.25)^2}{4} \right) = 138.7 \text{ kips} \quad (3.9)$$

$$\begin{aligned} M_{np} &= 2P_t (h_0 + h_1) \\ &= 2(138.7)(22.54 + 18.02) \\ &= 11,251 \text{ k-in.} \end{aligned} \quad (3.7)$$

$$\begin{aligned} \phi M_{np} &= 0.75(11,251) = 8438 \text{ k-in.} > M_{uc} \\ &= 8039 \text{ k-in. OK} \end{aligned}$$

5. Determine the Required End Plate Thickness

End Plate Yield Line Mechanism Parameter

$$\begin{aligned} s &= \frac{1}{2} \sqrt{b_p g} = \frac{1}{2} \sqrt{9.0(5.5)} \\ &= 3.52 \text{ in.} > p_{fi} = 2 \text{ in.} \end{aligned} \quad (\text{Table 3.1})$$

$$\begin{aligned} Y_p &= \frac{b_p}{2} \left[h_1 \left(\frac{1}{p_{fi}} + \frac{1}{s} \right) + h_0 \left(\frac{1}{p_{fo}} \right) - \frac{1}{2} \right] \\ &\quad + \frac{2}{g} \left[h_1 (p_{fi} + s) \right] \quad (\text{Table 3.1}) \\ &= \frac{9.0}{2} \left[18.02 \left(\frac{1}{2.0} + \frac{1}{3.52} \right) + 22.54 \left(\frac{1}{2.0} \right) - \frac{1}{2} \right] \\ &\quad + \frac{2}{5.5} \left[18.02 (2.0 + 3.52) \right] \\ &= 148.2 \text{ in.} \end{aligned}$$

Required End Plate Thickness

$$\begin{aligned} t_p \text{ Req'd} &= \sqrt{\frac{1.11 \phi M_{np}}{\phi_b F_{yp} Y_p}} \\ &= \sqrt{\frac{1.11(0.75)(11,251)}{0.9(50)(148.2)}} \\ &= 1.19 \text{ in.} \end{aligned} \quad (3.10)$$

6. Select End Plate Thickness

USE $t_p = 1\frac{1}{4}$ in. (ASTM A572 Gr. 50 steel)

7. Calculate the Factored Beam Flange Force

$$F_{fu} = \frac{M_{uc}}{(d_b - t_{fb})} = \frac{8039}{20.8 - 0.522} = 396 \text{ kips} \quad (3.11)$$

8. Check Shear Yielding of Extended Portion of End Plate

$$\begin{aligned} \phi R_n &= 0.9(0.6 F_{yp}) b_p t_p \\ &= 0.9(0.6)(50)(9.0)(1.25) \\ &= 304 \text{ kips} \end{aligned} \quad (3.12)$$

Check Inequality 3.12

$$\frac{F_{fu}}{2} = \frac{396}{2} = 198 \text{ kips} \leq \phi R_n = 304 \text{ kips OK}$$

9. Check Shear Rupture of Extended Portion of End Plate

$$\begin{aligned} A_n &= [b_p - 2(d_b + \frac{1}{8})] t_p \\ &= [9.0 - 2(1.25 + 0.125)] 1.25 \\ &= 7.81 \text{ in}^2 \\ \phi R_n &= 0.75(0.6 F_{uf}) A_n \\ &= 0.75(0.6)(65)(7.81) \\ &= 228 \text{ kips} \end{aligned} \quad (3.13)$$

Check Inequality 3.13

$$\frac{F_{fu}}{2} = 198 \text{ kips} \leq \phi R_n = 228 \text{ kips OK}$$

10. End plate is unstiffened, therefore this step is not required.

11. Check Compression Bolts Shear Rupture Strength

$$\begin{aligned} V_u &= 40 \text{ kips} \\ V_u &\leq \phi R_n = \phi n_b F_v A_b \end{aligned} \quad (3.17)$$

$$\begin{aligned} \phi R_n &= 0.75(4)(60) \left(\frac{\pi (1.25)^2}{4} \right) \\ &= 221 \text{ kips} \end{aligned}$$

$$V_u = 40 \text{ kips} < \phi R_n = 221 \text{ kips OK}$$

12. Check Compression Bolts Bearing/Tearout

i) End Plate

$$V_u = 40 \text{ kips} \leq \phi R_n = n_i(\phi R_n)_i + n_o(\phi R_n)_o \quad (3.18)$$

$$n_i = 2$$

$$n_o = 2$$

$$\begin{aligned} \text{Bearing Strength} &= 2.4 d_b t_p F_u = 2.4(1.25)(1.25)(65) \\ &= 244 \text{ kips/bolt} \end{aligned}$$

Tearout Outer Bolts:

$$\begin{aligned} L_c &= (2.0 + 0.522 + 2.0) - (1.25 + 1/16) = 3.21 \text{ in.} \\ R_{n, \text{inner}} &= 1.2 L_c t_p F_u = 1.2(3.21)(1.25)(65) \quad (3.19) \\ &= 313 \text{ kips} > 244 \text{ kips} \end{aligned}$$

By inspection, bearing controls for the inner bolts.

$$\begin{aligned} \phi R_n &= 4(0.75 \times 244) \\ &= 732 \text{ kips} > V_u = 40 \text{ kips OK} \end{aligned}$$

ii) Column Flange

$$\begin{aligned} t_{fc} &= 0.860 \text{ in.} \\ \phi R_n &= 732 \left(\frac{0.860}{1.25} \right) \left(\frac{F_{yp} = 50}{F_{yc} = 50} \right) \\ &= 504 \text{ kips} > V_u = 40 \text{ kips OK} \end{aligned}$$

13. Design Welds

i) Beam Flanges to End-Plate Weld

Use CJP welds and the procedure in Figure 2.10.

ii) Beam Web to End-Plate Weld

Minimum weld size for 1¼ in. end plate is 5/16 in. The required weld to develop the bending stress in the beam web near the tension bolts using E70 electrodes is

$$D = \frac{0.6 F_{yb} t_{wb}}{2(1.392)} = \frac{0.6(50)(0.375)}{2(1.392)}$$

$$= 4.04 \text{ sixteenths}$$

USE 5/16 in. Fillet Welds

The applied shear is to be resisted by weld between the minimum of the mid-depth of the beam and the compression flange or the inner row of tension bolts plus two bolt diameters and the compression flange. By inspection the former governs for this example.

$$\begin{aligned} \text{Effective length of weld} &= d_b/2 - t_{fb} \\ &= 20.8/2 - 0.522 = 9.88 \text{ in.} \end{aligned}$$

$$D = \frac{40}{2(1.392)(9.88)} = 1.45 \text{ sixteenths}$$

USE 5/16 in. Fillet Welds

Column-Side Design

14. Check the Column Flange for Flexural Yielding

$$s = \frac{1}{2} \sqrt{b_{fc} g} = \frac{1}{2} \sqrt{14.6(5.5)} = 4.48 \text{ in.} \quad (\text{Table 3.4})$$

$$c = p_{fo} + t_{fb} + p_{fi} = 2.0 + 0.522 + 2.0 = 4.52 \text{ in.}$$

$$Y_c = \frac{b_{fc}}{2} \left[h_1 \left(\frac{1}{s} \right) + h_0 \left(\frac{1}{s} \right) \right] \quad (\text{Table 3.4})$$

$$+ \frac{2}{g} \left[h_1 \left(s + \frac{3c}{4} \right) + h_0 \left(s + \frac{c}{4} \right) + \frac{c^2}{2} \right] + \frac{g}{2}$$

$$= \frac{14.6}{2} \left[18.02 \left(\frac{1}{4.48} \right) + 22.54 \left(\frac{1}{4.48} \right) \right]$$

$$+ \frac{2}{5.5} \left[18.02 \left(4.48 + \frac{3(4.52)}{4} \right) + 22.54 \left(4.48 + \frac{4.52}{4} \right) + \frac{4.52^2}{2} \right] + \frac{5.5}{2}$$

$$= 170.1 \text{ in.}$$

Required Unstiffened Column Flange Thickness

$$t_{fc \text{ Req'd}} = \sqrt{\frac{1.11 \phi M_{np}}{\phi_b F_{yc} Y_c}} \quad (3.20)$$

$$= \sqrt{\frac{1.11(0.75)(11,251)}{0.9(50)(170.1)}}$$

$$= 1.10 \text{ in.} > t_{fc} = 0.860 \text{ in.}$$

∴ Add Flange Stiffeners

Assume ½ in. Stiffener Plates

$$t_s = 1/2 \text{ in.}$$

$$p_{so} = p_{si} = \frac{c - t_s}{2} = \frac{4.52 - 0.5}{2} = 2.01 \text{ in.}$$

For Stiffened Column Flange

$$Y_c = \frac{b_{fc}}{2} \left[h_1 \left(\frac{1}{s} + \frac{1}{p_{si}} \right) + h_0 \left(\frac{1}{s} + \frac{1}{p_{so}} \right) \right]$$

$$+ \frac{2}{g} \left[h_1 (s + p_{si}) + h_0 (s + p_{so}) \right] \quad (\text{Table 3.4})$$

$$= \frac{14.6}{2} \left[18.02 \left(\frac{1}{4.48} + \frac{1}{2.01} \right) + 22.54 \left(\frac{1}{4.48} + \frac{1}{2.01} \right) \right] + \frac{2}{5.5} [18.02 (4.48 + 2.01) + 22.54 (4.48 + 2.01)]$$

$$= 309.1 \text{ in.}$$

Determine Reduced Stiffened Column Flange Thickness

$$t_{fc \text{ Req'd}} = \sqrt{\frac{1.11 (0.75)(11,251)}{0.9(50)309.1}} \quad (3.17)$$

$$= 0.82 \text{ in.} < t_{fc} = 0.860 \text{ in.}$$

∴ OK with Column Flange Stiffeners

15. Calculate Strength of Unstiffened Column Flange to Determine Stiffener Design Force

$$\phi M_{cf} = \phi_b F_{yc} Y_c t_{cf}^2 \quad (3.21)$$

$$= 0.9(50)(170.1)(0.860)^2$$

$$= 5661 \text{ k-in.}$$

$$\phi R_n = \frac{\phi M_{cf}}{d - t_{fb}} = \frac{5661}{20.8 - 0.522} \quad (3.22)$$

$$= 279 \text{ kips} < F_{fu} = 396 \text{ kips}$$

16. Calculate Local Web Yielding Strength

$$C_t = 1.0 \text{ (Assume not at Top of Column)}$$

$$N = t_{fb} + 0 = 0.522$$

Note: If fillet welds are used $N = t_{fb} + 0.707 t_w$, where t_w is the fillet weld leg size.

$$\phi R_n = \phi C_t (6k_c + N + 2t_p) F_{yc} t_{wc}$$

$$= 1.0(1.0)[6(1.46) + 0.522 + 2(1.25)](50)(0.525)$$

$$= 309 \text{ kips} < F_{fu} = 396 \text{ kips}$$

∴ Column Stiffeners Required

17. Calculate Web Buckling Strength

$$h = \left(\frac{h}{t_w} \right)_c t_{wc} = (21.7)(0.525) = 11.39 \text{ in.}$$

$$\phi R_n = \frac{\phi 24 t_{wc}^3 \sqrt{E F_{yc}}}{h} \quad (3.26)$$

$$= \frac{0.9(24)(0.525)^3 \sqrt{29000(50)}}{11.39}$$

$$= 330 \text{ kips}$$

$$F_{fu} = 396 \text{ kips} > 330 \text{ kips} = \phi R_n \quad (3.25)$$

∴ Column Stiffeners Required

18. Calculate Web Crippling Strength

$$\phi R_n = \phi 0.80 t_{wc}^2 \left[1 + 3 \left(\frac{N}{d_c} \right) \left(\frac{t_{wc}}{t_{fc}} \right)^{1.5} \right] \quad (3.29)$$

$$\times \sqrt{\frac{E F_{yc} t_{fc}}{t_{wc}}}$$

$$= 0.75(0.80)(0.525)^2 \left[1 + 3 \left(\frac{0.522}{14.3} \right) \left(\frac{0.525}{0.860} \right)^{1.5} \right]$$

$$\times \sqrt{\frac{29000(50)(0.860)}{0.525}}$$

$$= 268 \text{ kips} < F_{fu} = 396 \text{ kips}$$

∴ Column Stiffeners Required

19. Determine Stiffener Design Force

$$F_{cu} = F_{fu} - \min \phi R_n$$

$$= 396 - \min \begin{array}{l} 279 \\ 309 \\ 330 \\ 268 \end{array}$$

$$= 128 \text{ kips}$$

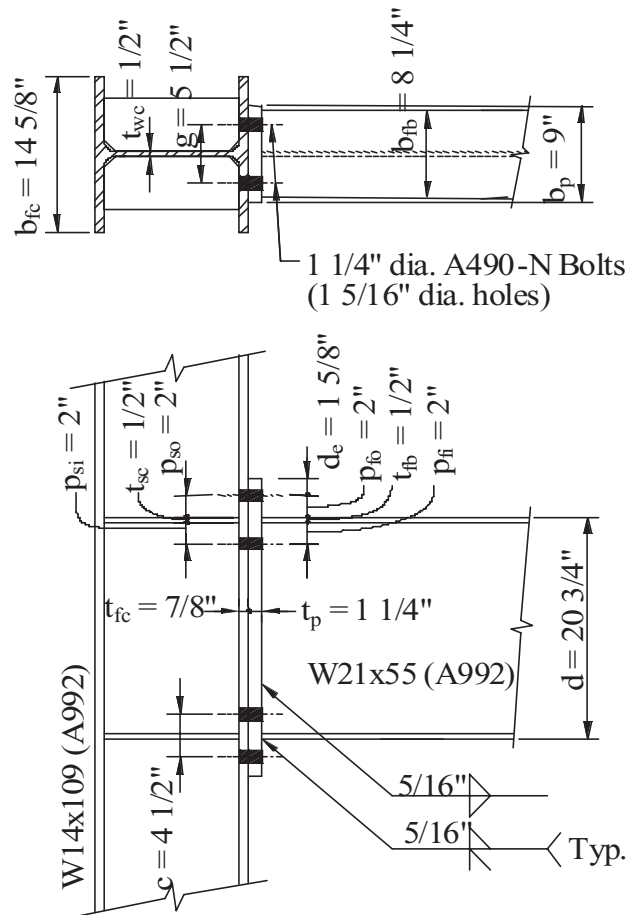
20. Stiffener Design and Panel Zone Checks

Refer to AISC *Design Guide 13 Wide-Flange Column Stiffening at Moment Connections—Wind and Seismic Applications* (Carter, 1999) for design requirements and procedures.

Summary

Beam: W21×55 (ASTM A992)
 Column: W14×109 (ASTM A992)
 4E End Plate: 1 1/4 in. by 9 in. (ASTM A572 Gr. 50)
 Bolt Diameter: 1 1/4 in.
 Bolt Grade: ASTM A490
 Column Requires Stiffeners

Final Details for 4E Moment End-Plate Connection Design



4E Example B

A four-bolt extended unstiffened (4E) end-plate connection is to be designed to connect a W21×55 beam to a W14×109 column. The connection is designed for a low seismic application ($R < 3$) with a design moment of 4000 k-in. The beam and column material are ASTM A992 steel and the end plate is ASTM A572 Gr. 50 steel. ASTM A490 bolts are to be used. The required shear resistance, V_u , is 40 kips.

Beam:

W21×55

$$d_b = 20.8 \text{ in.}$$

$$t_{wb} = 0.375 \text{ in.}$$

$$b_{fb} = 8.22 \text{ in.}$$

$$t_{fb} = 0.522 \text{ in.}$$

$$k_b = 1.02 \text{ in.}$$

Workable Gage = $5\frac{1}{2}$ in.

$$Z_{xb} = 126 \text{ in.}^3$$

$$F_{yb} = 50 \text{ ksi (ASTM A992 steel)}$$

$$F_{ub} = 65 \text{ ksi}$$

Column:

W14×109

$$d_c = 14.3 \text{ in.}$$

$$t_{wc} = 0.525 \text{ in.}$$

$$b_{fc} = 14.6 \text{ in.}$$

$$t_{fc} = 0.860 \text{ in.}$$

$$k_c = 1.46 \text{ in.}$$

$$(h/t_w)_c = 21.7$$

Workable Gage = $5\frac{1}{2}$ in.

$$Z_{xc} = 447 \text{ in.}^3$$

$$F_{yc} = 50 \text{ ksi (ASTM A992 steel)}$$

$$F_{uc} = 65 \text{ ksi}$$

Bolts:

ASTM A325

See Figure 3.1 for definition of *connection geometry*.

Beam Side Design

1. Connection Design Moment

$$M_n = 4000 \text{ k-in.} < \phi M_p = 0.9(50)(126) = 5670 \text{ k-in.} \quad (3.2)$$

2. Select Connection Configuration: Four-Bolt Extended Unstiffened

Assumed Geometric Design Data

$$b_p \approx b_f + 1 \text{ in.} = 8.22 + 1 = 9.22 \text{ in.} \Rightarrow \text{Use } b_p = 9.0 \text{ in.}$$

$$g = 5\frac{1}{2} \text{ in. (same as beam and column "workable gage")}$$

$$p_{fi} = 2 \text{ in.}$$

$$p_{fo} = 2 \text{ in.}$$

$$d_e = 1\frac{5}{8} \text{ in.}$$

$$F_{yp} = 50 \text{ ksi}$$

$$F_{up} = 65 \text{ ksi (ASTM A572 Gr. 50 steel)}$$

$$F_t = 90 \text{ ksi (ASTM A325 bolts)}$$

Using assumed dimensions,

$$h_0 = 20.8 + 2 - \frac{0.522}{2} = 22.54 \text{ in.}$$

$$h_1 = 20.8 - 0.522 - 2 - \frac{0.522}{2} = 18.02 \text{ in.}$$

3. Determine the Required Bolt Diameter (ASTM A325)

$$d_{b \text{ Req'd}} = \sqrt{\frac{2 M_{uc}}{\pi \phi F_t (h_0 + h_1)}} \quad (3.5)$$

$$= \sqrt{\frac{2(4000)}{\pi(0.75)(90)(22.54 + 18.02)}}$$

$$= 0.96 \text{ in.}$$

4. Select Trial Bolt Diameter and Calculate the No Prying Bolt Moment

Use $d_b = 1 \text{ in. (ASTM A325)}$

Bolt Tensile Strength

$$P_t = F_t A_b = 90 \left(\frac{\pi (1.0)^2}{4} \right) = 70.7 \text{ kips} \quad (3.9)$$

$$M_{np} = 2 P_t (h_0 + h_1) \quad (3.7)$$

$$= 2(70.7)(22.54 + 18.02)$$

$$= 5375 \text{ k-in.}$$

$$\phi M_{np} = 0.75(5375) = 4031 \text{ k-in.} > M_{uc} = 4000 \text{ k-in. OK}$$

5. Determine the Required End Plate Thickness

End Plate Yield Line Mechanism Parameter

$$s = \frac{1}{2} \sqrt{b_p g} = \frac{1}{2} \sqrt{9.0(5.5)} \quad (\text{Table 3.1})$$

$$= 3.52 \text{ in.} > p_{fi} = 2 \text{ in.}$$

$$Y_p = \frac{b_p}{2} \left[h_1 \left(\frac{1}{p_{fi}} + \frac{1}{s} \right) + h_0 \left(\frac{1}{p_{fo}} \right) - \frac{1}{2} \right]$$

$$+ \frac{2}{g} [h_1 (p_{fi} + s)] \quad (\text{Table 3.1})$$

$$= \frac{9.0}{2} \left[18.02 \left(\frac{1}{2.0} + \frac{1}{3.52} \right) + 22.54 \left(\frac{1}{2.0} \right) - \frac{1}{2} \right] + \frac{2}{5.5} [18.02 (2.0 + 3.52)]$$

$$= 148.2 \text{ in.}$$

Required End Plate Thickness

$$t_{p \text{ Req'd}} = \sqrt{\frac{1.11 \phi M_{np}}{\phi_b F_{yp} Y_p}} \quad (3.10)$$

$$= \sqrt{\frac{1.11(0.75)(5735)}{0.9(50)(148.2)}}$$

$$= 0.85 \text{ in.}$$

6. Select End Plate Thickness

USE $t_p = 7/8$ in. (ASTM A572 Gr. 50 steel)

7. Calculate the Factored Beam Flange Force

$$F_{fu} = \frac{M_{uc}}{(d_b - t_{fb})} = \frac{4000}{20.8 - 0.522} = 197 \text{ kips} \quad (3.11)$$

8. Check Shear Yielding of Extended Portion of End Plate

$$\phi R_n = 0.9(0.6 F_{yp}) b_p t_p \quad (3.12)$$

$$= 0.9(0.6)(50)(9.0)(0.875)$$

$$= 213 \text{ kips}$$

Check Inequality 3.12

$$\frac{F_{fu}}{2} = \frac{197}{2} = 98.5 \text{ kips} \leq \phi R_n = 213 \text{ kips OK}$$

9. Check Shear Rupture of Extended Portion of End Plate

$$A_n = [b_p - 2(d_b + 1/8)] t_p$$

$$= [9.0 - 2(1.00 + 0.125)] (0.875)$$

$$= 6.13 \text{ in}^2$$

$$\phi R_n = 0.75(0.6 F_{uf}) A_n \quad (3.13)$$

$$= 0.75(0.6)(65)(6.13)$$

$$= 179 \text{ kips}$$

Check Inequality 3.13

$$\frac{F_{fu}}{2} = 109 \text{ kips} \leq \phi R_n = 179 \text{ kips OK}$$

10. End plate is unstiffened, therefore this step is not required.

11. Check Compression Bolts Shear Rupture Strength

$$V_u = 40 \text{ kips}$$

$$V_u \leq \phi R_n = \phi n_b F_v A_b \quad (3.17)$$

$$\phi R_n = 0.75(4)(48) \left(\frac{\pi(1.0)^2}{4} \right)$$

$$= 113 \text{ kips}$$

$$V_u = 40 \text{ kips} < \phi R_n = 113 \text{ kips OK}$$

12. Check Compression Bolts Bearing/Tearout

i) End Plate

$$V_u = 40 \text{ kips} \leq \phi R_n = n_i(\phi R_n)_i + n_o(\phi R_n)_o \quad (3.18)$$

$$n_i = 2$$

$$n_o = 2$$

$$\text{Bearing Strength} = 2.4 d_b t_p F_u = 2.4(1.0)(0.875)(65)$$

$$= 137 \text{ kips/bolt}$$

Tearout Outer Bolts:

$$L_c = (2.0 + 0.522 + 2.0) - (1.0 + 1/16) = 3.46 \text{ in.}$$

$$R_{n, \text{inner}} = 1.2 L_c t_p F_u = 1.2(3.46)(0.875)(65) \quad (3.19)$$

$$= 236 \text{ kips} > 137 \text{ kips}$$

By inspection, bearing controls for the inner bolts.

$$\phi R_n = 4(0.75 \times 137)$$

$$= 411 \text{ kips} > V_u = 40 \text{ kips OK}$$

ii) Column Flange

$$t_{fc} = 0.860 \text{ in.}$$

$$\phi R_n = 411 \left(\frac{0.860}{0.875} \right) \left(\frac{F_{yp} = 50}{F_{yc} = 50} \right)$$

$$= 404 \text{ kips} > V_u = 40 \text{ kips OK}$$

13. Design Welds

i) Beam Flanges to End-Plate Weld

Minimum Fillet Weld size $5/16$ in.

$$\phi R_n = 0.6 F_y A_{fb} > F_{fu}$$

$$0.6 F_y A_{fb} = 0.6(50)(8.22 \times 0.522) = 129 \text{ kips}$$

$$\text{Use } \phi R_n = F_{fu} = 197 \text{ kips}$$

Note: For wind and low-seismic applications, it is recommended that the weld design force be taken equal to

the calculated flange force but not less than $0.6F_yA_{fb}$. This recommended minimum weld design force is based upon engineering judgement and intended to preclude small weld sizes on comparatively larger beams sized for stiffness, and to account for the variations in the distribution of the flange force across the weld length.

Effective length of weld, on both sides of the flange

$$\begin{aligned} &= b_f + (b_f - t_w) \\ &= 8.22 + (8.22 - 0.375) = 16.1 \text{ in.} \\ D &= \frac{197}{1.5 \times 1.392 \times (16.1)} \end{aligned}$$

= 5.86 sixteenths

USE $\frac{3}{8}$ in. Fillet Weld

ii) Beam Web to End-Plate Weld

Minimum weld size for $\frac{7}{8}$ in. end plate is $\frac{5}{16}$ in. The required weld to develop the bending stress in the beam web near the tension bolts using E70 electrodes is

$$D = \frac{0.6F_{yb}t_{wb}}{2(1.392)} = \frac{0.6(50)(0.375)}{2(1.392)}$$

= 4.04 sixteenths

USE $\frac{5}{16}$ in. Fillet Welds

The applied shear is to be resisted by weld between the minimum of the mid-depth of the beam and the compression flange or the inner row of tension bolts plus two bolt diameters and the compression flange. By inspection the former governs for this example.

Effective length of weld = $d_b/2 - t_{fb}$

$$= 20.8/2 - 0.522 = 9.88 \text{ in.} \quad (3.25)$$

$$D = 40/2(1.392)(9.88) = 1.45 \text{ sixteenths}$$

USE $\frac{5}{16}$ in. Fillet Welds

Column-Side Design

14. Check the Column Flange for Flexural Yielding

$$\begin{aligned} s &= \frac{1}{2} \sqrt{b_{fc} g} = \frac{1}{2} \sqrt{14.6(5.5)} \\ &= 4.48 \text{ in.} \end{aligned} \quad (\text{Table 3.4})$$

$$c = p_{fo} + t_{fb} + p_{fi} = 2.0 + 0.522 + 2.0 = 4.52 \text{ in.}$$

$$\begin{aligned} Y_c &= \frac{b_{fc}}{2} \left[h_1 \left(\frac{1}{s} \right) + h_0 \left(\frac{1}{c} \right) \right] \quad (\text{Table 3.4}) \\ &+ \frac{2}{g} \left[h_1 \left(s + \frac{3c}{4} \right) + h_0 \left(s + \frac{c}{4} \right) + \frac{c^2}{2} \right] + \frac{g}{2} \\ &= \frac{14.6}{2} \left[18.02 \left(\frac{1}{4.48} \right) + 22.54 \left(\frac{1}{4.48} \right) \right] \\ &+ \frac{2}{5.5} \left[18.02 \left(4.48 + \frac{3(4.52)}{4} \right) + 22.54 \left(4.48 + \frac{4.52}{4} \right) + \frac{4.52^2}{2} \right] + \frac{5.5}{2} \\ &= 170.1 \text{ in.} \end{aligned}$$

Required Unstiffened Column Flange Thickness

$$\begin{aligned} t_{fc \text{ Req'd}} &= \sqrt{\frac{1.11 \phi M_{np}}{\phi_b F_{yc} Y_c}} \quad (3.20) \\ &= \sqrt{\frac{1.11(0.75)(5735)}{0.9(50)(170.1)}} \\ &= 0.790 \text{ in.} < t_{fc} = 0.860 \text{ in.} \end{aligned}$$

∴ Column Stiffeners Not Required

15. Calculate Strength of Unstiffened Column Flange to Determine Stiffener Design Force

Column Stiffeners Not Required

16. Calculate Local Web Yielding Strength

$$C_t = 1.0 \text{ (Assume not at Top of Column)}$$

$$N = t_{fb} + 0 = 0.522$$

Note: If fillet welds are used $N = t_{fb} + 0.707 t_w$, where t_w is the fillet weld leg size.

$$\begin{aligned} \phi R_n &= \phi C_t (6k_c + N + 2t_p) F_{yc} t_{wc} \\ &= 1.0(1.0)[(6(1.46) + 0.522 + 2(0.875))] \\ &\quad (50)(0.525) \\ &= 289 \text{ kips} > F_{fu} = 197 \text{ kips} \\ &\therefore \text{Column Stiffeners Not Required} \end{aligned}$$

17. Calculate Web Buckling Strength

$$\begin{aligned} h &= \left(\frac{h}{t_w} \right)_c t_{wc} = (21.7)(0.525) = 11.39 \text{ in.} \\ \phi R_n &= \frac{\phi 24 t_{wc}^3 \sqrt{E F_{yc}}}{h} \quad (3.26) \\ &= \frac{0.9(24)(0.525)^3 \sqrt{29000(50)}}{11.39} \\ &= 330 \text{ kips} \end{aligned}$$

$$F_{fu} \leq \phi R_n$$

$$197 \text{ kips} < 330 \text{ kips}$$

∴ Column Stiffeners Not Required

18. Calculate Web Crippling Strength

$$\phi R_n = \phi \cdot 0.80 \cdot t_{wc}^2 \left[1 + 3 \left(\frac{N}{d_c} \right) \left(\frac{t_{wc}}{t_{fc}} \right)^{1.5} \right] \quad (3.29)$$

$$\times \sqrt{\frac{E F_{yc} t_{fc}}{t_{wc}}}$$

$$= 0.75 (0.80) (0.525)^2 \left[1 + 3 \left(\frac{0.522}{14.3} \right) \left(\frac{0.525}{0.860} \right)^{1.5} \right]$$

$$\times \sqrt{\frac{29000 (50) (0.860)}{0.525}}$$

$$= 268 \text{ kips} > F_{fu} = 197 \text{ kips}$$

∴ Column Stiffeners Not Required

19. Determine Stiffener Design Force

No Column Stiffeners Required

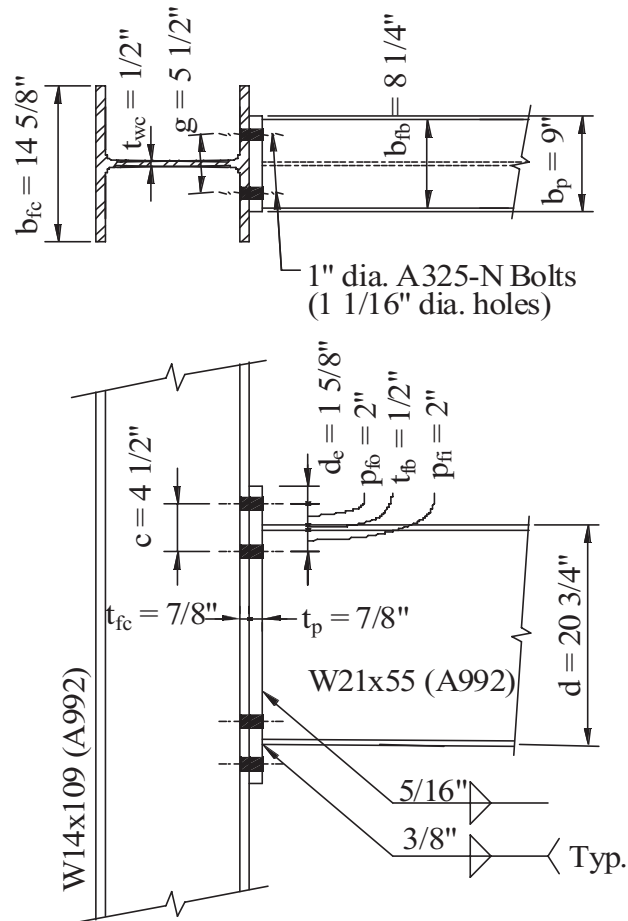
20. Stiffener Design and Panel Zone Checks

Refer to AISC *Design Guide 13 Wide-Flange Column Stiffening at Moment Connections—Wind and Seismic Applications* (Carter, 1999) for design requirements and procedures.

Summary

Beam: W21x55 (ASTM A992)
 Column: W14x109 (ASTM A992)
 4E End Plate: $\frac{7}{8}$ in. by 9 in. (ASTM A572 Gr. 50)
 Bolt Diameter: 1 in.
 Bolt Grade: ASTM A325
 Column Does Not Require Stiffeners

Final Details for 4E Moment End-Plate Connection Design



4.3 Four-Bolt Extended Stiffened (4ES) End-Plate Connection

4ES Example

Redesign the 4E Example A using a four-bolt extended stiffened (4ES) end-plate connection, subjected to cyclic/seismic loading. Use the same beam and column sections and material properties. The required shear resistance, V_u , remains at 40 kips. Conservatively, the connection design moment from the 4E Design Example will be used, which required 1 1/4 in. diameter ASTM A490 grade bolts.

This example is a continuation of the 4E Example A. The only modification is the consideration of a stiffened end plate. All other limit state checks are same as in the 4E Design Example A. Step numbers are from Chapter 3 and replace corresponding steps in the 4E Example A.

5. Determine for the Required End Plate Thickness

End Plate Yield Line Mechanism Parameter

$$s = \frac{1}{2} \sqrt{b_p g} = \frac{1}{2} \sqrt{9.0(5.5)} \quad (\text{Table 3.2})$$

$$= 3.52 \text{ in.} > p_{fi} = 2 \text{ in.}$$

$$d_e = \text{Vertical Edge Distance for Outside Bolt Holes}$$

$$= 1.625 \text{ in.}$$

Since $d_e < s$, use Case 1 from Table 3.2

$$Y_p = \frac{b_p}{2} \left[h_1 \left(\frac{1}{p_{fi}} + \frac{1}{s} \right) + h_0 \left(\frac{1}{p_{fo}} + \frac{1}{2s} \right) \right]$$

$$+ \frac{2}{g} \left[h_1(p_{fi} + s) + h_0(d_e + p_{fo}) \right] \quad (\text{Table 3.2})$$

$$= \frac{9.0}{2} \left[18.02 \left(\frac{1}{2.0} + \frac{1}{3.52} \right) + 22.54 \left(\frac{1}{2.0} + \frac{1}{2(3.52)} \right) \right]$$

$$+ \frac{2}{5.5} [18.02(2.0 + 3.52) + 22.54(1.625 + 2.0)]$$

$$= 194.6 \text{ in.}$$

Required End Plate Thickness

$$t_{p \text{ Req'd}} = \sqrt{\frac{1.11 \phi M_{np}}{\phi_b F_{yc} Y_c}} \quad (3.10)$$

$$= \sqrt{\frac{1.11(0.75)(11251)}{0.9(50)(194.6)}}$$

$$= 1.03 \text{ in.}$$

6. Select End Plate Thickness

USE $t_p = 1 \frac{1}{8}$ in. (ASTM A572 Gr. 50 steel)

10. Determine Required Stiffener Thickness and Length and Design the Stiffener Welds

$$t_{s, \text{req'd}} = t_{wb} \left(\frac{F_{yb}}{F_{ys}} \right) = 0.375 \left(\frac{50}{50} \right) \quad (3.15)$$

$$= 0.375 \text{ in.}$$

$$h_{st} = p_{fo} + d_e = 2.0 + 1.625$$

$$= 3.625 \text{ in.}$$

$$L_{st} = \frac{h_{st}}{\tan 30^\circ} = \frac{3.625}{\tan 30^\circ} = 6.3 \text{ in.}$$

Check Local Buckling:

$$\frac{h_{st}}{t_{st}} = \frac{3.625}{0.375} = 9.67 \leq 0.56 \sqrt{\frac{E}{F_y}} \quad (3.16)$$

$$= 0.56 \sqrt{\frac{29,000}{50}} = 13.5 \text{ OK}$$

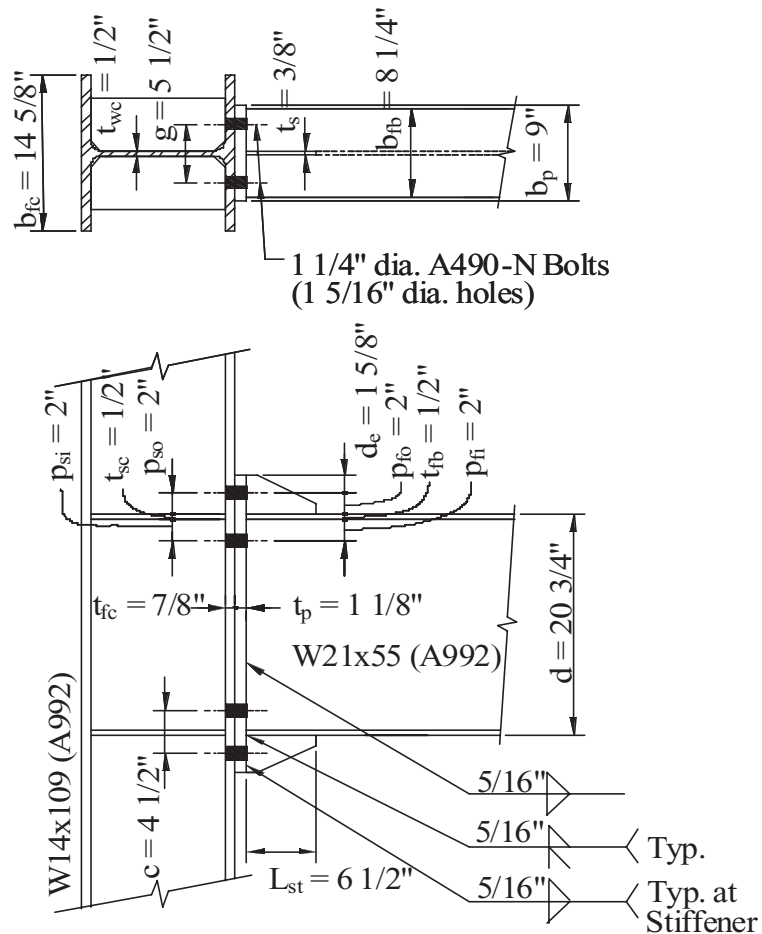
Use $\frac{3}{8}$ in. $\times$ $3 \frac{5}{8}$ in. $\times$ $6 \frac{1}{2}$ in. stiffener (ASTM A572 Gr. 50)

Fillet welds, $\frac{5}{16}$ in., are sufficient to develop the stiffener at the beam flange and at the end plate since the stiffener thickness is not greater than $\frac{3}{8}$ in.

Summary

Beam: W21x55 (ASTM A992)
 Column: W14x109 (ASTM A992)
 4ES End Plate: 1 1/8 in. by 9 in. with 3/8 in. Beam
 Flange to End Plate Stiffener
 (ASTM A572 Gr. 50)
 Bolt Diameter: 1 1/4 in.
 Bolt Grade: ASTM A490
 Column Requires Stiffeners

Final Details for 4ES Moment End-Plate Connection Design



4.4 Eight-Bolt Extended Stiffened (8ES) End-Plate Connection

8ES Design Example

Redesign 4E Example A using an eight-bolt extended stiffened (8ES) end-plate connection, subjected to cyclic/seismic loading. Use the same beam and column sections, and material properties. The required shear resistance, V_u , is 40 kips.

Beam:

W21×55

$d_b = 20.8$ in.

$t_{wb} = 0.375$ in.

$b_{fb} = 8.22$ in.

$t_{fb} = 0.522$ in.

$k_b = 1.02$ in.

Workable Gage = $5\frac{1}{2}$ in.

$Z_{xb} = 126$ in.³

$F_{yb} = 50$ ksi (ASTM A992 steel)

$F_{ub} = 65$ ksi

Column:

W14×109

$d_c = 14.3$ in.

$t_{wc} = 0.525$ in.

$b_{fc} = 14.6$ in.

$t_{fc} = 0.860$ in.

$k_c = 1.46$ in.

Workable Gage = $5\frac{1}{2}$ in.

$Z_{xc} = 447$ in.³

$F_{yc} = 50$ ksi (ASTM A992 steel)

$F_{uc} = 65$ ksi

Bolts:

ASTM A325

Beam Side Design

1. Connection Design Moment

$$\begin{aligned} M_{pe} &= 1.1 R_y F_y Z_x \\ &= 1.1(1.1)(50)(126) \\ &= 7623 \text{ k-in.} \end{aligned} \quad (3.2)$$

Estimate Location of Plastic Hinge

$$L_p = L_{st} + t_p \approx 11.5 \text{ in.} \quad (3.4)$$

Moment at the Face of Column (Connection Design Moment)

$$\begin{aligned} M_{uc} &= M_{pe} + V_u L_p \\ &= 7623 + 40(11.5) \\ &= 8083 \text{ k-in.} \end{aligned} \quad (3.1)$$

2. Select Connection Configuration: Eight-Bolt Extended Stiffened

Assumed Geometric Design Data

$$b_p \approx b_f + 1 \text{ in.} = 8.22 + 1 = 9.22 \text{ in.} \Rightarrow \text{Use } b_p = 9 \text{ in.}$$

$$g = 5\frac{1}{2} \text{ in. (same as beam and column "workable gage")}$$

$$p_{fi} = 1\frac{3}{4} \text{ in.}$$

$$p_{fo} = 1\frac{3}{4} \text{ in.}$$

$$p_b = 3 \text{ in.}$$

$$d_e = 1\frac{1}{4} \text{ in.}$$

$$F_{yp} = 50 \text{ ksi}$$

$$F_{up} = 65 \text{ ksi (ASTM A572 Gr. 50 steel)}$$

$$F_t = 90 \text{ ksi (ASTM A325 bolts)}$$

Using assumed dimensions,

$$h_1 = 20.8 + 1.75 + 3 - \frac{0.522}{2} = 25.29 \text{ in.}$$

$$h_2 = 20.8 + 1.75 - \frac{0.522}{2} = 22.29 \text{ in.}$$

$$h_3 = 20.8 - 0.522 - 1.75 - \frac{0.522}{2} = 18.27 \text{ in.}$$

$$h_4 = 20.8 - 0.522 - 1.75 - 3.0 - \frac{0.522}{2} = 15.27 \text{ in.}$$

3. Determine the Required Bolt Diameter (ASTM A325)

$$\begin{aligned} d_{b \text{ Req'd}} &= \sqrt{\frac{2 M_{uc}}{\pi \phi F_t (h_1 + h_2 + h_3 + h_4)}} \quad (3.6) \\ &= \sqrt{\frac{2(8083)}{\pi(0.75)(90)(25.29 + 22.29 + 18.27 + 15.27)}} \\ &= 0.97 \text{ in.} \end{aligned}$$

4. Select Trial Bolt Diameter and Calculate the No Prying Bolt Moment

Use $d_b = 1$ in. (ASTM A325)

Bolt Tensile Strength

$$P_t = F_t A_b = 90 \left(\frac{\pi(1.0)^2}{4} \right) = 70.7 \text{ kips} \quad (3.9)$$

$$\begin{aligned} M_{np} &= 2P_t (h_1 + h_2 + h_3 + h_4) \quad (3.8) \\ &= 2(70.7)(25.29 + 22.29 + 18.27 + 15.27) \\ &= 11,470 \text{ k-in.} \end{aligned}$$

$$\phi M_{np} = 0.75(11,470) = 8603 \text{ k-in.} > M_{uc} = 8083 \text{ k-in. OK}$$

5. Determine for the Required End Plate Thickness

End Plate Yield Line Mechanism Parameter