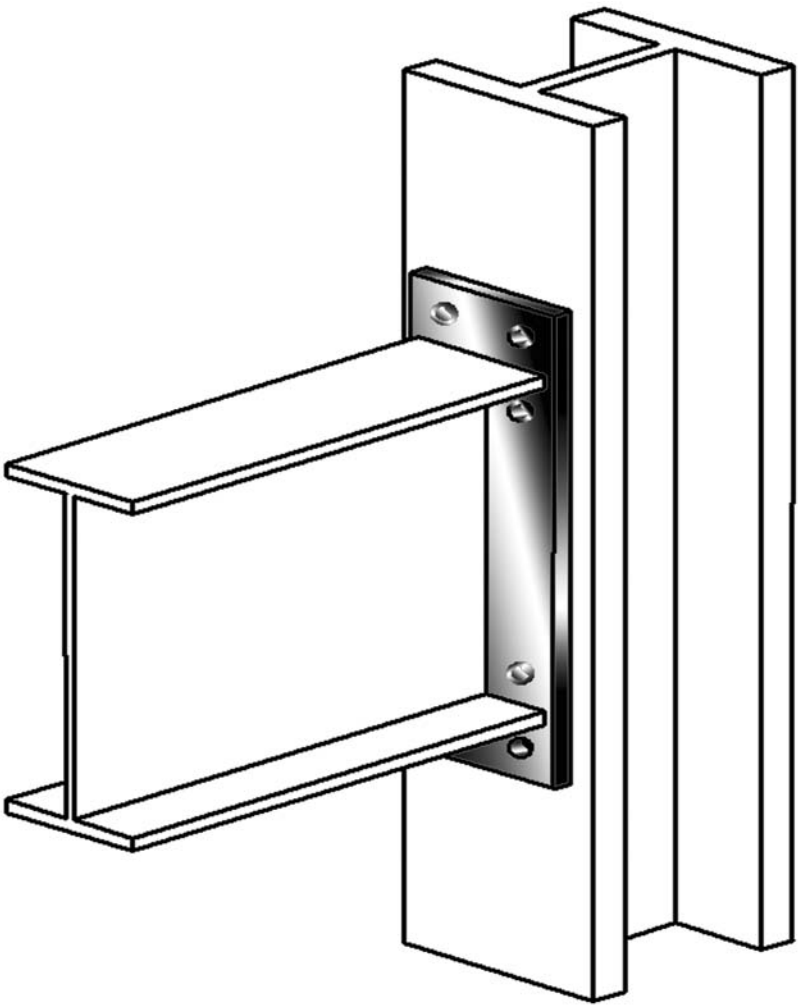
Steel Design Guide 4

Extended End-Plate Moment Connections

Seismic and Wind Applications

Second Edition



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4

Extended End-Plate Moment Connections

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Second Edition

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Table of Contents

Chapter 1
Introduction

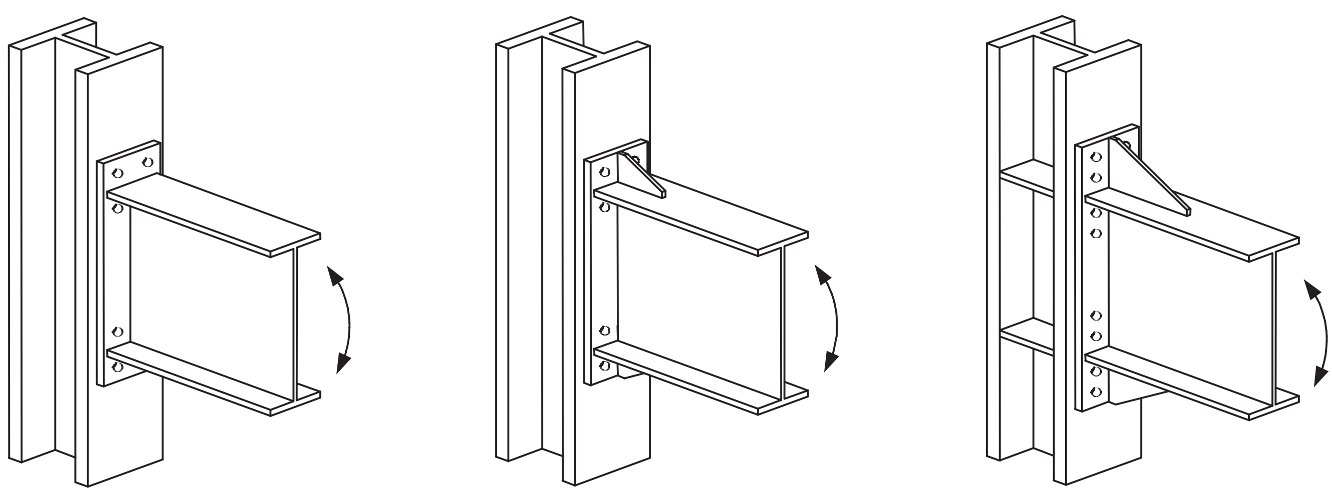
1.1 Background

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*



(a) Four Bolt Unstiffened, 4E *(b) Four Bolt Stiffened, 4ES* *(c) Eight Bolt Stiffened, 8ES*

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 1/4 in. to 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