

End-Plate Moment Connections— Their Use and Misuse

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Up until about 1960, many field-bolted moment connections were of the split tee-stub type (Fig. 1). These connections were used in power house construction, in multi-story office buildings and other structures where moment capacity was required between beams and columns. They served well, and thousands are still doing their duty reliably in some of our older structures. In later years, end

plates (Fig. 2) have been used in place of the split tee-stub connections.

It should be emphasized that the advent of end-plate connections was not something new and exotic. These connections were simply a natural development of the old split tee connection, allowed at least in part by the acceptance and greater use of shop welding. These end-

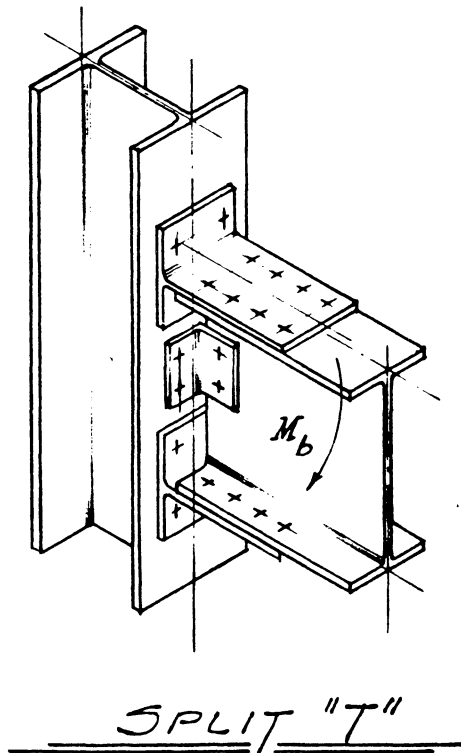


Fig. 1. Tee-stub connection with web shear connection

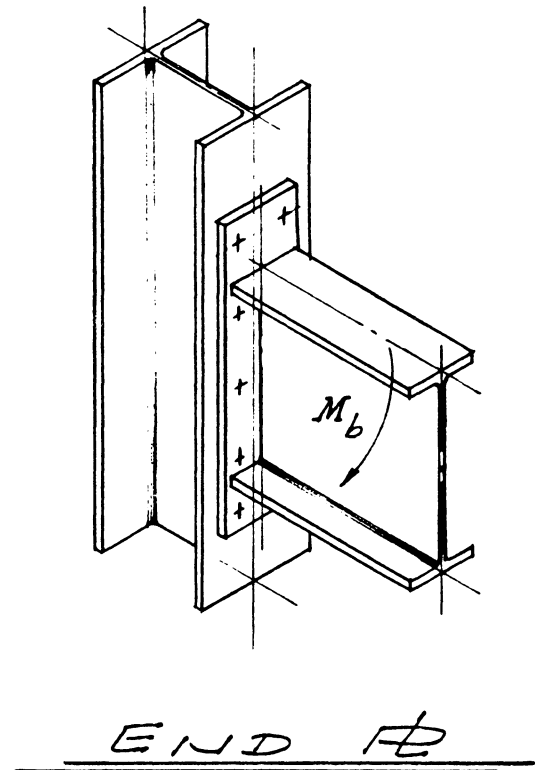


Fig. 2. End-plate connection

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plate connections have generally had a most satisfactory behavior and have provided greater economy than could be achieved with split tee connections.

One of the early papers describing end plate behavior was authored by Disque in 1962.¹ This paper encouraged the use of these connections, and their popularity continued to increase until the publication of the 7th Edition AISC *Manual of Steel Construction* in 1970. The design example in this 7th Edition was grossly conservative. The resulting increase in costs compared to results of design procedures previously in use discouraged the use of these moment connections. This situation resulted in a determination by the design profession to develop more accurate and realistic design procedures. Thus, in 1971, a research program on this subject was initiated at Auburn University under the joint sponsorship of the Metal Building Manufacturers Association and AISC. Krishnamurthy, the principal investigator, carried on this work first at Auburn and later at Vanderbilt University. His studies resulted in a paper published in the AISC *Engineering Journal*² and the design example in the present AISC Manual.³

At present, additional research is underway under AISC auspices at the University of Oklahoma, with Murray and Kukreti as the principal investigators. The Oklahoma studies include the use of more bolts in the tension region, as well as requirements for stiffeners in columns opposite both the tension and compression flanges of the connecting beams. Murray expects to complete work in the near future and many of his results will be included in the soon to be published AISC *Engineering for Steel Construction*.

The purpose of this paper is not just to provide a brief outline of developments to date but, more importantly, to call attention to some of the inadvisable applications of end-plate construction that have recently come to the writer's attention. The most serious situations are described in the following three case histories.

CASE 1

As pointed out by Krishnamurthy, the calculated end-plate thickness is dependent to a very great degree on the bolt pitch—the distance from the top of the beam flange to the center line of the bolt row above that flange (see dimension p_f Fig. 3). A small increase in bolt pitch can result in a surprising increase in the thickness requirement for the end plate.

A number of designers use the rule that the bolt pitch be equal to the bolt diameter plus $\frac{1}{2}$ in.⁴ This provides the minimum bolt pitch dimension consistent with reasonable erection clearances. However, some fabricators establish a standard dimension of say 2 in. to provide consistency for detailing and shop practice.

A problem recently given to the writer involves a designer who used the minimum bolt pitch rule, where the detailers (and therefore the shop) used a standard dimension of 2 in. The detailer used the $\frac{5}{8}$ in. end-plate thickness as

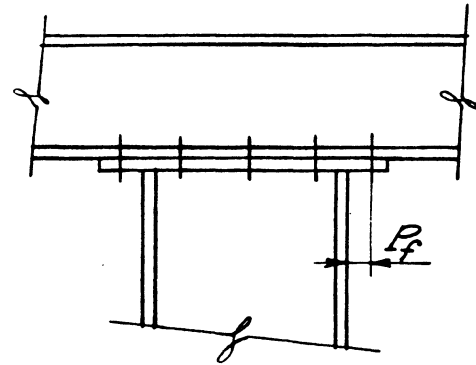


Figure 3

calculated by the designer for the smaller bolt pitch. A printout of a computer program used by the writer shows the unhappy result (Fig. 4). The strength of the $\frac{5}{8}$ in. plate is only 39% of the strength of the required 1-in. plate.

The lesson here is to be sure of proper communications between designers, detailers and checkers. It is also helpful to show the bolt pitch on the computer printout (Fig. 4) rather than trust verbal instructions or memory.

CASE 2

A fabricator was recently required to design some very heavy end-plate moment connections with A325 bolts. Four $1\frac{1}{2}$ -in. tension bolts were inadequate to carry the specified

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*****
*           A-1 PROGRAM           *
*           END PLATES           *
* A-36 MEMBERS   A-325 BOLTS *
*****

WIDE FLANGE SECTION = W12*26

BEAM END MOMENT= 54 FT. KIPS

USE 5/16  INCH FILLET WELD

USE 3/4   INCH BOLTS

PLATE WIDTH= 7 & 1/2 INCHES.

USE BOLT PITCH (P_f) = 1 & 1/4
INCHES.

END PLATE THICKNESS = 5/8
INCHES.

BOLT PITCH (P_f) IS INCREASED TO
2 INCHES.

NEW END PLATE THICKNESS = 1
INCHES

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Figure 4

moments. The fabricator simply increased the bolt size to 2 in. (even though the ASTM specification for A325 bolts does not cover sizes above 1½ in.).

Krishnamurthy, unfortunately, did not specify a maximum bolt size in his paper.² Neither Krishnamurthy nor anyone on the joint AISC-MBMA task group assigned to his project ever envisioned an attempt to use bolts larger than 1½ in. In fact, the Auburn-Vanderbilt tests were based on bolts smaller than 1½ in. Results were then extrapolated to cover the 1½-in. sizes, but no larger. There are a number of reasons to limit the bolt sizes:

1. Lack of research data for sizes above 1½ in.
2. Difficulty of tightening the larger sizes
3. Difficulty of procurement of larger sizes
4. Increased likelihood of distortion of column flanges

Luckily, the proposed connections were reviewed at the last moment by knowledgeable engineers and different connections which used a greater number of smaller bolts were designed. The lesson in this case is to limit bolt sizes to a maximum of 1½ in. In the writer's work, bolt sizes are limited to a maximum of 1¼ in. if at all possible.

CASE 3

A single-story structure, some 30 ft in height, with multiple spans of approximately 40 ft, was designed in accordance with Type 2 construction assumptions. This construction is permitted by AISC specifications and assumes pinned connections between beams and columns for the vertical load. Subsequently, the connections are assumed rigid to accommodate moments associated with horizontal loads. Type 2 construction presupposes that the connections have sufficient strength and ductility to accept moments associated with *combined* gravity and wind loadings. In this particular case, the connection moments due to horizontal loads were approximately 50% of the moments that would be generated by vertical loads for a fixed ended beam. End plates were used to provide the connection stiffness and were designed for wind moments only. This design resulted in very small welds connecting the beams to the end plates, and also relatively small bolts. Because end-plate connections are inherently very stiff, moments associated with vertical loads would have greatly overstressed the welds. Weld fracture may have occurred with the subsequent danger of catastrophic collapse. A redesign was performed to alleviate the problem.

The primary lesson in this case is to properly design the end-plate connection for maximum *combined* moments. Particular attention should be given to adequacy of the weld of the tension flange to the end plate. A failure of this weld would not just result in loss of moment resistance and excessive deformation but would most likely strip the beam from the end plate with attendant collapse of the structure.

There is another reason to be conservative in the design of this weld. The end plate will be distorted during welding as shown in Fig. 5.

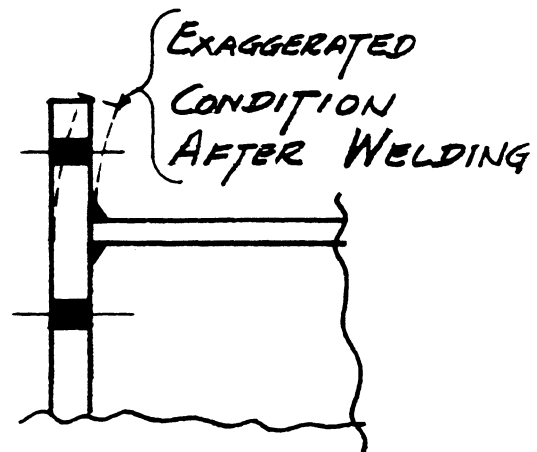


Figure 5

Tests at both Vanderbilt and Oklahoma have shown that such end-plate distortion does not decrease the over-all connection capacity. Therefore, preforming and straightening costs are normally not justified. However, it is apparent in such cases that welds will be given a severe test during the field bolt tightening operation. For reasons just outlined, the writer's firm requires these welds to be sized to develop the full strength of the beam tension flange regardless of the moments imposed. This requirement is not as costly as might first be assumed. This is true because, for reasons of economy, end plates are normally used where connection moments are equal to, or at least approach, the beam capacity. Thus, the advisability of using end plates in Type 2 construction is open to question from an economic as well as an engineering point of view. This comment applies with special emphasis to one-story structures where moments due to wind are often small compared to moments due to combined vertical and horizontal loads.

The writer wishes to thank the engineers, herein unnamed, associated with the cases described for information they provided and for their willingness to have their problems discussed so these same problems can be avoided by others.

REFERENCES

1. Disque, Robert O. End Plate Connections *Proceedings, National Engineering Conference, American Institute of Steel Construction, Chicago, Ill., 1962 (p. 30).*
2. Krishnamurthy, N. A Fresh Look at Bolted End-Plate Behavior and Design *AISC Engineering Journal, 2nd Qtr., 1978 (p. 39).*
3. *American Institute of Steel Construction Manual of Steel Construction 8th Ed., Chicago, Ill., 1980.*
4. Griffiths and Wooten Discussion of Ref. 2 *AISC Engineering Journal, 2nd Qtr., 1979 (p. 56).*