

*Engineering for Steel Construction*, just published by AISC, delineates the most recent advances in connections and detailing for structural engineers. Here's a detailed summary. . . .

# Engineering for Steel Construction

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Structural engineers are increasingly aware of the importance of details and connections in steel structures. These are not only the key to economy but also may be the source of trouble.

For this reason, AISC has just released a new 370-pg. text-book—*Engineering for Steel Construction*—for the advanced detailer and design engineer. The new book is a companion to *Detailing for Steel Construction*, which was written for the beginning detailer who might have a high school education. Since both books are self-contained, some material may be common to both. Together, the books, which replace the 2nd Edition of *Structural Steel Detailing*, are geared to the 8th Edition *Manual of Steel Construction*.

**Chapter 1:** “Structural Engineering” presents the theory and methods of application used in the book to model design procedures for details and connections. The instantaneous center method for analyzing certain types of eccentric connections is described, and references cited for simple solutions not available when the Manual was published.

Brackets are analyzed and detailed in two ways. A very conservative approach assumes the neutral axis of the bracket coincides with the center of gravity of the bolt group. A second approach determines the center of rotation by an iterative process where the static moment of the tension area equals that of the compression area. Both procedures, which result in a more realistic design, are included in the book.

Prying action is fully explained and its various applications described. The procedure in the text is based on that in the 8th Edition Manual. Prying action, essentially, is a phenomenon where the force in a bolt in a hanger-type connection is increased by the development of a prying force,  $Q$ . Several combinations of bolt capacity and plate thickness can satisfy equations and static requirements.

A new philosophy of material tear-out of tension splices is also included. In the existing *Structural Steel Detailing*, material was checked by equating the allowable shear stress of the material to the weld capacity. Thus  $0.4F_y = 0.707(0.3)F_{uw}$ . For  $F_y = 36$  ksi and  $F_u = 70$  ksi, the minimum plate thickness,  $t = 1.03w$  where  $w$  is the length of the weld leg. The new procedure recognizes that material tear-out is more of a block shear model, and new rules for this are included.

**Chapter 2:** “Metallurgy and Welding” contains practically all the information required by the structural engineer on this subject. Topics include: weldability, welding processes, types of welds, welded beam-to-column connections, electrode and process nomenclature, nondestructive testing (NDT) methods including ultrasonic and radiographic, fracture control and lamellar tearing.

The factors which affect weldability, chemical composition, geometric properties and grain size, are discussed. The advantages and disadvantages of various welding processes are outlined to help the designer and fabricator with shrinkage and distortion control problems, and the text suggests ways to minimize lamellar tearing.

**Chapter 3:** “Simple (Type 2) Connections” treats the behavior and design of all popular connections—framing angles, shear tabs, end plates, single angles and tees. The chapter examines each possible failure mode and presents rules to prevent them.

For bolted connections, this includes bolt shear, material net shear, web tear-out (block shear) and the effect of edge and end distance on bolt bearing capacity. For welded connections, new material is presented for block shear on coped beams.

The question of eccentricities on bolted connections often arises. For one-sided connections, eccentricities on the outstanding legs should be accounted for. The 8th Edition Manual Tables (Part 4) account for them. For a single vertical row on the beam web, eccentricities have been ignored traditionally, and no distress has ever been reported. For two vertical rows on the web, however, less was known at the time both the 8th Edition Manual and this new text were published, so no procedure is included in either book. The author recommends Joseph A. Yura's suggested block shear model published in the January 1983 *Journal of the Structural Division* of ASCE.

Single-plate shear connections are a relatively new Type 2 simple connection. Although it was in use for years, it was only in 1980 that Prof. Ralph M. Richard of the University of Arizona provided an analytical procedure for design, which is included in this book.

Figure 1 shows a typical shear tab connection. It has some inherent stiffness, and improper design might disqualify it as a Type 2 connection. Also, the rigidity might impose force on

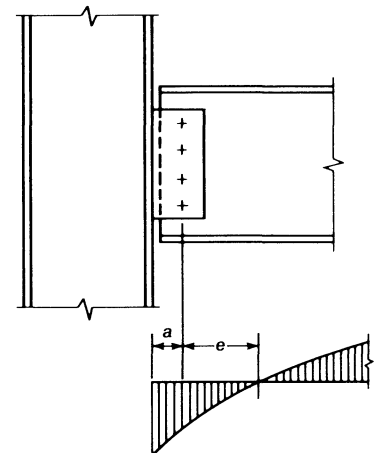


Fig.1. Typical shear tab connection

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the top part of the weld that connects the plate to the abutment. In the past, many connections have been designed with a moment equal to the reaction times the distance  $a$ .

Richard's research indicates that, for design purposes, the moment should be calculated by multiplying the reaction by  $a + e$  ( $e$  is determined as a function of connection dimensions and beam properties). Using an ultimate design procedure, the plate thickness and weld size is determined.

**Chapter 4:** "Moment Connections" includes the design and detailing of welded beam-to-column connections, end plates, flange plates, cap and seat angles and flexible wind connections. The text emphasizes that, with a welded beam-to-column connection, the web of the beam need not be welded directly. A better method is to weld a shear tab to the columns and bolt the beam web to the shear tab. Flanges of the beam are then field-welded. This connection has been tested thoroughly at Lehigh University and the University of California/Berkeley. Those tests have determined the connection can easily achieve the plastic moment of the beam.

The procedure for designing the detailing end-plate connections was developed by Krishnamurthy, as presented in the 8th Edition Manual. The procedure is empirical, and is based on physical tests and a finite element analysis. The result is a method to determine bolt size and end-plate thickness. However, no method to determine column stiffener requirements was included. Stiffener requirements for end-plate connections is the subject of a major research project at the University of Oklahoma under the direction of Thomas Murray. At publication time of *Engineering for Steel Construction*, research was not complete. But, sufficient progress has been made to recommend these tentative, conservative rules:

1. In the region of the beam compression, flange stiffeners are not required when the column web  $t_w$  exceeds:

$$t_w \geq \frac{P_{bf}}{F_y \{t_b + 5k + 2t_p + 2w\}}$$

where  $P_{bf}$  = factored beam flange force, kips;  $F_y$  = specified yield strength of the columns, ksi;  $t_b$  = beam flange thickness, in.;  $k$  = distance from outer face of flange to web toe of fillet of column section, in.;  $t_p$  = end-plate thickness, in.;  $w$  = end-plate fillet weld leg dimension, in. It is anticipated that final research will permit  $5k$  to increase to  $6k$  or even  $7k$ . However, until then,  $5k$  is recommended.

2. In the tension region, a rule of thumb is proposed until research is completed. This rule is conservative, and states that if the column flange thickness is as thick as the bolt diameter (determined by the Krishnamurthy, or Manual, procedure), then stiffeners are not required. Otherwise, they are.

A welded beam connection into the weak axis of a column has to be designed and detailed with special care. Research is presently underway at Lehigh University to determine how this connection can best be designed to insure it will behave in a ductile fashion once the plastic moment of the beam is attained. Preliminary tests indicate that ductility could be impaired if the flange connection plates are terminated at the toes of the column flange. Although final results are not yet in, several recommendations have been made to improve the connection:

1. Use connection plates slightly thicker than the beam flange thickness.

2. Use backup stiffener.

3. Extend the connection plate beyond the tips of column flanges.

Flexible moment connections designed to carry only the wind moment are also treated in this new AISC text. Several types discussed include cap and seat angles and flange plates.

**Chapter 5:** "Skewed, Sloped and Canted Beam Connections" is an update of a similar chapter in the 2nd Edition *Structural Steel Detailing*. It will be of more interest to the detailer than the designer. However, the designer should be aware of the various eccentricities sometimes associated with these connections. One change from previous detailing practice is that all working points will be located on the material. Previously, they could be located in space.

**Chapter 6:** "Columns" includes detailing procedures, stiffener requirements and design, built-up members, lifting lugs, splices and base plates. Column stiffeners are required because of a local deficiency in column web or column flange thickness. In the area of beam compression, flange stiffeners might be required to prevent column web yielding or buckling. In the tension area, stiffeners prevent web yielding and provide a uniform stress distribution to the column flange. If a non-uniform flange distribution exists, the beam-to-column flange weld could be overstressed. The text notes that determination of column web stiffeners should be the responsibility of the designer, rather than of the detailer. The reason is that, for economy, the designer might prefer a column with a larger web or flange to avoid adding stiffeners.

As in the 2nd Edition of *Structural Steel Detailing*, the new text includes details of recommended column splices. Several are shown, but the one which may need special attention is the direct field-welded splice. A clean detail with no extra pieces, it is economical to prepare in the shop, and uses minimal field labor.

This chapter also discusses the design of column base plates. The procedure in the 8th Edition Manual is recognized as ultraconservative in the case of small base plates—those large enough in plan to include just the section profile. The procedure in *Engineering for Steel Construction*, based on a yield line theory developed by R.S. Fling, results in reasonably sized base plates. However, since the material was included in the book, a third method, developed by Thomas Murray of Oklahoma University, has been proposed. This method was published in AISC's *Engineering Journal*, 4th Quarter, 1983. It is relatively simple and is expected to be very popular.

**Chapter 7:** "Framing for Heavy Construction" includes bracing, crane girders, trusses and heavy bracing connections. Except for the material on heavy bracing connections, it is primarily an update of that in *Structural Steel Detailing*. A heavy bracing connection is defined as a connection involving a column, beam and diagonal brace. Usually, very heavy loads are involved, such as those expected in power plants, industrial structures and high-rise buildings.

Designers differ widely in the models they use to design these connections. For this reason, AISC has underway a major research project at the University of Arizona, which should be completed in a year or so. In the meantime, William A. Thornton of Cives Corporation has developed a procedure that is included in this new text.