Steel Interchange is an open forum for *Modern Steel Construction* readers to exchange useful and practical professional ideas and information on all phases of steel building and bridge construction. Opinions and suggestions are welcome on any subject covered in this magazine.

The opinions expressed in *Steel Interchange* do not necessarily represent an official position of the American Institute of Steel Construction, Inc. and have not been reviewed. It is recognized that the design of structures is within the scope and expertise of a competent licensed structural engineer, architect or other licensed professional for the application of principles to a particular structure.

If you have a question or problem that your fellow readers might help you to solve, please forward it to us. At the same time, feel free to respond to any of the questions that you have read here. Contact *Steel Interchange* via AISC's Steel Solutions Center:

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QUALITY OF STRUCTURAL DRAWINGS

Where can I find published information regarding the requirements for completeness of structural design drawings?

Question sent to AISC's Steel Solutions Center

This information is contained in Section 3 of the 2000 AISC *Code of Standard Practice for Steel Buildings and Bridges* (a free download from **www.aisc.org/code**), which is likely referenced in the contract documents. The *Code* mandates that specific information appear on Structural Design Drawings and Specifications, including working points, sizes, locations, material grades, elevations, connection-type restrictions, loads and whether LRFD or ASD is to be used in completing connection details.

Also, the Council of American Structural Engineers (CASE) recently released "A Guideline Addressing Coordination and Completeness of Structural Construction Documents", Case Document 962 D. It addresses the quality of structural construction documents, including aspects of design relationships, communications, coordination and completeness, guidance for dimensioning of structural drawings, effects of various project delivery systems, document revisions, and recommendations for the development and application of quality management procedures. This document can be ordered from **www.acec.org/publications**.

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MAKING FILLET WELDS

How many passes does it take to make a 3/8" fillet weld? I want to tell the Engineers in our office to limit fillet weld sizes to 5/16" (except where larger welds are absolutely needed) and I would like to show them that while a 3/8" fillet weld has 20% more strength than a 5/16" fillet weld, it takes ___% more time to install. I'm assuming that a two-pass weld takes twice as long as a one-pass weld, etc.

Question sent to AISC's Steel Solutions Center

Welds of course must be large enough to transfer the loads imposed upon them, and when a 3/8" fillet weld is required, that must be specified, regardless of the production difficulties, costs, etc.

When welding in the horizontal position, where in the case of a tee joint, one member is horizontal, and the other is vertical, the largest fillet weld that can be made in a single pass is typically 5/16". Larger welds, such as 3/8", become difficult to make because gravity tends to pull the molten weld metal down, resulting in an unequal fillet weld size (the vertical leg being smaller).

The AWS D1.1 *Structural Welding Code*, Table 3.7 provides maximum single pass fillet weld sizes for various welding processes and positions of welding. This table applies to prequalified Welding Procedure Specifications (WPSs). For SMAW, the maximum size in the horizontal position is 5/16". The same is true for SAW with single electrode, or parallel electrodes. For GMAW and FCAW, the largest single pass fillet weld in the horizontal position, for prequalified WPSs, is one size larger — 3/8".

All of the above has lead to the general rule-of-thumb that 5/16" is a reasonable maximum weld size for single pass welds, made in the horizontal position.

All through this discussion, emphasis has been placed on the "horizontal position". When welding in the flat position (for example, a tee joint where each piece is at a 45 degree angle to horizontal), larger weld puddles are possible since in this position, gravity helps hold the weld metal in place. Accordingly, larger single pass welds may be made. D1.1 Table 3.7 reflects this reality by permitting 3/8" fillet welds in the flat position with prequalified WPSs for SMAW. For FCAW and GMAW, this dimension is 1/2", and for SAW, the size is unlimited. Of course, many structural applications do not lend themselves to positioning the weld in the flat position, and thus the horizontal constraints discussed above will control.

In a very general way, welding time, and cost, is proportional to the volume of weld metal involved. Thus, with everything else being equal, an ideal, flat faced, equal legged 3/8" weld will theoretically take 44% more time and money to make than an equivalent ideal 5/16" weld. But, assuming a 3/8" weld will take two passes, there will be an additional cleaning cycle involved, so the real cost will probably be more than a 44% increase.

If you want a rule-of-thumb, tell your Engineers that the time increase will be the square of the size increase. Thus, a weld that is 20% stronger, (e.g. 1.20 times the original size) will take 44% more time to make than the smaller weld (e.g., 1.202, or 1.44).

How fast can a welder lay down a 5/16" fillet weld? How many inches per minute?

Welding travel speeds depend on the deposit rate of the process, the electrode and the procedure used. The number of possibilities is nearly endless.

Here are some examples based on actual procedures:

1. **SMAW with 1/4″ E7018 electrode** (neither this electrode or diameter are commonly used in construction today, but was at one time, was a very productive welding method), a 5/16″ fillet can be made at 6.5-7.5 inches per minute (ipm). Using the same electrode, for a two-pass 3/8″ fillet, two passes at 9.5-11.5 are required. That results in a net combined average travel speed of about 4.8-5.8 ipm. In this case, the two-pass 3/8″ fillet weld took 32% longer to make. This does not consider the weld cleaning that would take place between weld passes.
2. **FCAW with 7/64″ E70T-7 electrode**, a 5/16″ fillet can be made at 15-17 ipm, and a single-pass 3/8″ fillet made at 11-12 ipm. In this case, the single-pass 3/8″ fillet weld (which is permitted by Table 3.7 for FCAW) took 39% more time to make.

In these two examples, neither had the ideal relationship of the larger weld taking exactly 44% more time to make. The differences are likely due to weld profile differences: none of the welds were the ideal, flat faced, even legged fillets that would be required to achieve the exact 44% increase. In all probability, the 5/16″ fillet welds in these examples were probably slightly oversized.

This illustration also shows the advantages that can be achieved when using FCAW versus SMAW. The FCAW process is capable at making the 5/16″ fillet at a production rate that is 228% of SMAW, and the 3/8″ fillet at a rate that is 216% of SMAW. Additionally, because FCAW is semiautomatic, the amount of time the welder keeps the arc lit it greater, in that he/she does not need to stop and exchange electrodes as is the case with SMAW.

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FATIGUE STRESS RANGE

With respect to Equation A-K3.1 in the 1999 AISC *LRFD Specification*, a question has come up as to why the design stress range F_{sr} value has to equal to or greater than the life stress range F_{th}? Would it seem that F_{sr} should be less than or equal to F_{th}? Can you clarify?

Question sent to AISC's Steel Solutions Center

Think of F_{th} as a stress range small enough to allow for an infinite number of cycles. Fatigue becomes an issue if the design stress range happens to be larger (or equal) to Fth. When that happens, there will be a finite number of cycles before cracking and progressive failure (fatigue) occurs.

The Commentary of Appendix K in the 1999 AISC *LRFD Specification* will help explain stress ranges. The *Engineering Journal* paper "Fatigue Strength of Steel Members with Welded Details" by Fisher and Yen (fourth quarter 1977) is also recommended. Finally, if you happen to have the *Fracture and Fatigue Control in Structures* by Barsom and Rolfe, 3rd Edition, refer to Chapter 9.

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TEE STUB MOMENT CONNECTION

During my review of the 3rd Edition *LRFD Manual*, I noticed that there is no mention of the T-stub moment connection as a fully restrained moment connection. Such a connection uses tee sections to connect the top and bottom flange of the beam to the column through bolting (single shear on the beam and tension with prying on the column flange.) I am also aware that the extended end-plate appears to have take the place of this bolted connection. Is AISC discouraging the use of the T-stub connection for FR moment connections?

Question sent to AISC's Steel Solutions Center

You are correct in that end-plate moment connections have taken the place of the old T-stub moment connections. It was a natural development due to the advent of shop welding several decades ago and supplanted the all-bolted T-stub moment connection for reasons of economy (fewer pieces = fewer dollars). If desired, the T-stub configuration can still be used.

Today, for high-seismic applications ($R > 3$), FEMA 350 contains the prequalified double split-tee moment connection.

There is also progress into revisiting the tee stub moment connection for wind and low-seismic applications strictly as a flexible moment connection (FMC). The 3rd Edition *LRFD Manual* currently contains two FMC connections, namely the flange-angle and flange-plated flexible moment connections.

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