Calculate Loads Transferred Through Welds

By Duane K. Miller, P.E.

This is the third in a series of articles focusing on welding and the practicing engineer

THE MOST BASIC FUNCTION OF A WELD IS TO TRANSFER LOADS BETWEEN SEPARATE ELEMENTS OF A CONNECTION. A properly sized weld is one that is sufficient in size (i.e., area) so that loads or forces transferred through the weld do not cause the joint, or the adjacent base materials, to be over-loaded. Furthermore, a properly sized weld is no larger than necessary. Undersized welds may fail, while unnecessarily large welds needlessly increase fabrication cost. Oversized welds also result in higher residual stresses and greater distortion, as well as an increased tendency toward weld metal and base metal cracking. Welds that are specified to be either too large or too small are obviously inefficient designs.

Several mistakes can result in the specification of too-large welds. Some of these errors can be traced back to a violation of the basic principal: to properly determine the weld size, the load transferred through the weld must be known. Perhaps the most common mistake is to assume that the transferred loads will always be equivalent to the maximum capacity of the adjoining material. This approach assumes that the base material has been optimally designed with respect to strength and size, and that it will be loaded to its maximum capacity. If the selection criteria for the base material and weld sizing are different, or if the weld is not loaded to its maximum capacity, an oversized weld may result.

A second error is to assume that all the force transferred between two members must be transferred directly through the weld. For connections that are subject to compression, welds can share loads with the surrounding material if it is finished to bear. Failure to recognize this alternate load path will result in a weld of sufficient size to absorb the full compressional load.

A final, all too common example of over-sizing welds results when a complete joint penetration (CJP) groove weld is unnecessarily specified. This can happen "by accident" when the designer or detailer neglects to specify the weld size on a welding symbol.

As stated in AWS A2.4-93, *Standard Symbols for Welding Brazing and Nondestructive Examination*, 4.2.2: "Omitting the depth of bevel and groove weld size dimensions from the welding symbol requires complete joint penetration only for single-groove welds and double-groove welds having symmetrical joint geometries......

This is repeated in AWS Dl.1-92, *Structural Welding Code-Steel*, Paragraph 2.1.3.1, which explains that the original intent may be for a partial joint penetration (PJP) groove weld, but if the effective size (E) is omitted from the groove weld symbol, it defaults to a CJP groove weld.

Sometimes, a CJP groove weld is specified "just to be safe". Such an approach neglects the engineer's responsibility to provide the client with a safe and economical product. Finally, CJP may be specified simply to avoid the need to fully design the welded connection. This raises questions of professional ethics.

Case Study

A building that utilized large, fabricated box columns was recently erected. For the field-welded column splices, the engineer had specified CJP groove welds. This decision was extremely costly, and alternate PJP groove welds could have been employed.

For joining compression members with bearing joints, the AISC *LRFD Specification for Structural Steel Buildings* states in Section Jl.4: When columns bear on bearing plates, or are finished to bear at splices, there shall be sufficient connectors to hold all the plates securely in place. When other compression members are finished to bear, the splice material and its connectors shall be arranged to hold all parts in line and shall be proportioned for 50% of the factored strength of the member. All compression joints shall be proportioned to resist any tension developed by the factored loads supplied in the formula A4-6.

Any potential uplift (tension) is addressed by equation A4.6 which states that the following load combinations and the corresponding load factors shall be investigated:

$$0.9D + (1.3W \text{ or } 1.0E)$$

For this particular project, no uplift was calculated. However even if some uplift existed, neither AISC nor AWS would prohibit the use of PJP groove welds for these applications in statically loaded structures. Some engineers incorrectly apply the provision of AWS Dl.1-92, paragraph 2.5, which states: "Partial joint penetration groove welds subject to tension normal to their longitudinal axis shall not be used where design criteria indicate cyclic loading could produce

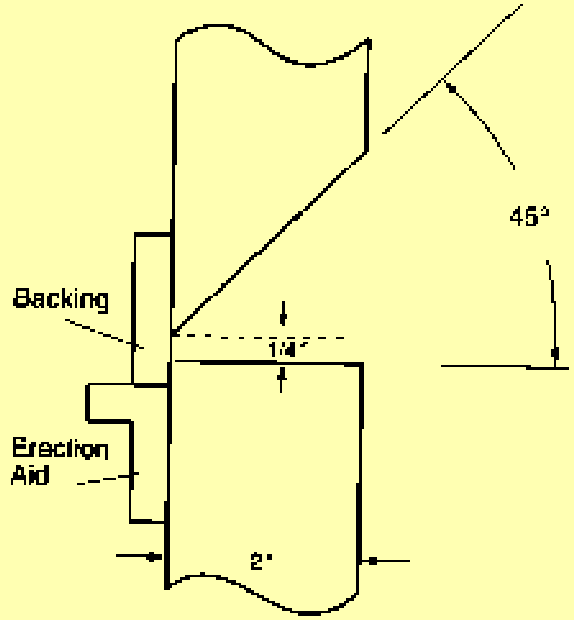

Figure 1 — B-U4a-GF

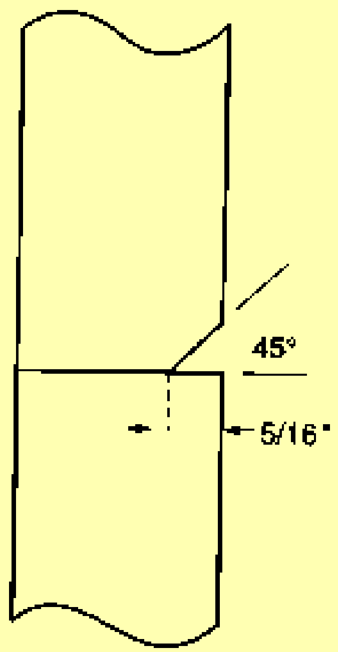

Figure 2—BTC-P4-GF

fatigue failure. Joints containing such welds, made from one side only, shall be restrained to prevent rotation." The first provision does not apply to statically loaded structures. The second requirement would be automatically met when splices are made on box columns. It should be noted that until the application of cyclic loads exceeds 20,000 cycles, the structure should be treated as being subject to static load. With this criteria, wind loading or even seismic loading would not constitute fatigue-type service.

For this project, the erector chose to use an AWS prequalified joint, namely, B-U4a-GF with a ¼" root opening and a 45 degree included angle (see Figure 1). For 2" thick steel, that detail requires 9.15 pounds of weld metal per foot (see Figure 2). Jl.4 provisions would only require "sufficient connectors to hold all the plates securely in place." A minimum sized PJP groove weld would have been sufficient. The minimum PJP weld size for 2" plate is ⅜", according to AWS Dl.1-92, Table 2.3. This would have permitted the use of a prequalified BTC-P4-GF groove weld detail, requiring only 0.30 pounds of weld metal per foot (see Figure 2). This constitutes a reduction of 96% in the weld metal required, with correspondingly less residual stress. The cost analysis on the following page (Table 1) demonstrates the dramatic savings that are possible.

The preceding calculations do not include the additional cost increasing elements associated with the use of CJP groove welds such as the requirement for backing bars and greater difficulty in fit up. Experience has shown these welds to be very difficult to make in the field. Double-sided CJP's require back gouging, which is impossible with box sections, and very difficult in the case of wide-flange columns. With the increased volume of weld metal, and the corresponding increases in weld metal shrinkage, vertical alignment is more difficult. The costs associated with these activities are real, although difficult to quantify.

In the actual project, some cracking was experienced. To overcome the cracking tendencies, the required preheat levels were increased. It can be argued that with the lower volume of weld shrinkage associated with the PJP groove weld, as well as the lower degree of restraint associated with this type of joint, the cracking would not have occurred if the proper weld had been specified.

For connections in heavy sections, AISC provides helpful information in Section Jl.5. The last paragraph of this section states: "Alternatively, splicing of such members subject to compression, including members which are subject to tension due to wind or seismic loads, shall be accomplished using splice details which do not induce large weld shrinkage strains; for example partial joint penetration flange groove welds with fillet-welded

Table 1: Cost Analysis

Data

Electrode Purchased (for column splices)	25,000 lbs
Electrode Efficiency	86 %
Weld Metal Deposited	21,500 lbs
Deposition Rate	18 lbs/hr
Operating Factor	40%
Labor & Overhead Rate	$65/hr

Cost Computation (As Built)

Labor:	Time to deposit weld = 21,500/(18)(0.40) =	2,986 hrs
	Cost to deposit weld = 2,986 hrs x 65 $/hr =	$194,000
Material:	(Purchase of electorde)	
	25,000 lbs x 1.00 $/lb =	$25,000
Total Cost:		**$219,000**

Cost Computation (Alternative)

Total Cost = (219,000)(0.30/9.15) =	**$7,000**

Savings: $212,000

surface lap plate splices on the web, bolted lap plate splices, or combination bolted/fillet-welded lap plate splices."

When details appropriate for tension connection in heavy shapes are applied to compression connections, many of the extra precautions and requirements for tension applications must be employed to eliminate cracking.

Conclusion

This case study once again proves that good design will facilitate both quality and economy. Unfortunately, it also proves that welded joint design continues to be plagued by the incorrect, unnecessary and wasteful specification of the complete joint penetration groove weld.

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