Welding Solutions: Put Things In Perspective With The AWS D1.1 Code

By Jeffrey W. Post, P.E.

"Those who do not study History are doomed to repeat it," warned George Santayana. These wise words speak to the engineering profession as much as to any other field of human endeavor. To some extent, the Codes and Standards that govern engineering practice are really works of history. For example, the AWS D1.1 Structural Welding Code – Steel is a consensus document that represents the collective wisdom of designers, fabricators, inspectors, educators and consultants, acquired over decades of experience with welded fabrication. It is not just a book of good advice; it is a book of requirements. Sometimes a provision of the Code is ignored, and there are no negative consequences. This can lead to complacency. In another instance, however, failure to observe that same provision can lead to major problems, sometimes with dire results in terms of human safety and/or financial consequences.

Technique

Backing

The use of steel backing provides our first example of the type of problem that may arise when provisions of D1.1 are inadequately followed. Paragraph 5.10.2 states: "Steel backing shall be made continuous for the full length of the weld. All joints in the steel backing shall be complete joint penetration welded butt joints meeting all of the requirements of Section 5, Fabrication, of the Code." In practice, this provision is sometimes ignored or overlooked when fabricators fit segments of bars into the back of groove preparations.

In one case involving long box girders, the corner welds were fitted with ½"x1" flat bars. The bars came in 20' lengths, so there was a natural butt splice every 20'. For whatever reason, the butt splices were only tack welded or partial joint penetration groove welded. Clearly, this did not satisfy the Code provision. One cold winter night, a brittle fracture originated at an unfused portion of a butt splice in one of the small backing bars and propagated completely through the bottom flange and portions of the webs of the box girder. The cost of the repairs, modifications, and litigation went into the millions. All this resulted from a mere butt weld that did not meet Code requirements!

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In the case of longitudinal corner welds in box girders, the need for continuity of backing seems obvious. But the same requirements apply to intersecting corners such as those found inside a box tube. For example, the bottom chord of a truss is to be made from TS12"x 8"x5/8", and requires several butt splices with complete joint penetration groove welds. Fabricators tend to fit four individual pieces of flat bar into the inside of one of the tubes, tack them in place with no regard to producing 100% sound welds at the corner intersections of the flat bars, slip the mating tube over the backing bar, and weld the tubular butt joint. This practice places a severe notch at the root of the butt weld at each of the four corners. The corners happen to be where local stresses are the highest.

Sometimes fabricators will form a continuous length of flat bar into the necessary backing, which leaves only one butt splice to make. This is much better, but still not acceptable. A variety of alternate solutions were given in a paper entitled "Box-

Tube Connections; Choices of Joint Details and Their Influence on Costs" I presented at the National Steel Construction Conference of AISC in 1990.

Groove Tolerances

One of the reasons for the popularity of the D1.1 Code is that it provides prequalified joint details. These have a long history of satisfactory performance and, when applied correctly, will produce sound welded joints. The fitup tolerances are given to the nearest 1/16" and 5o, which is close enough for structural applications. Those who are using robotics and mechanized welding apparatus might find these tolerances too liberal.

When the fitup of a weld groove allows the root opening to become too tight or the preparation of the groove angles is too tight, it will be difficult to obtain a satisfactory weld. Even the best welders cannot produce a sound weld if the fitup conditions are too tight for the welding procedure.

These issues may seem rudimentary, but all too often, the failure to pay attention to the simplest of details renders a weld unacceptable.

Weld Termination

Quality groove welds require good starts and stops of each weld bead. On simple butt joints, this is readily achieved by using temporary weld extension tabs or "run-off" tabs. AWS D1.1, Section 5.31.1 requires that "weld tabs (be) aligned in such a manner (as) to provide an extension of the joint preparation." In static applications like steel-framed buildings, these tabs may be left in place unless the engineer specifies removing them. The Northridge Earthquake provided an example of a case where engineers, contractors, and inspectors had misunderstood and misapplied this provision. In many instances, the investigation of damaged welded connections revealed welded joints where floor beams were framed into columns and run-off tabs were used. The welders simply installed the tabs across both ends of the groove, which created a dam effect, as if preventing run-off of the liquid weld metal and slag was the main purpose of the tabs; in fact, this created major fusion problems at both ends of the joint.

Prohibited Types of Joints and Welds

For dynamic or cyclically loaded welds or structures, AWS D1.1, Section 2.27 provides some specific prohibitions that grew out of experience with bridge fabrication. For instance, partial joint penetration butt joints in tension are not permitted, nor are intermittent groove welds or intermittent fillet welds. The Code recognizes that these types of welds are not adequate to perform satisfactorily under cyclic or dynamic loading. However, Section 6.8 (Commentary C6.8) permits the designer to use engineering judgment to exclude some of these Code provisions where appropriate. For example, if the stresses are low enough, a partial joint penetration butt joint in tension may prove to be perfectly adequate.

Minimum Weld Size

D1.1 specifies minimum weld sizes that have been established based on the thickness of the material, to generate sufficient heat input per unit length to prevent cracking (Table 5.8). Historically, these sizes have proven to be successful with most steels. Although some engineers consider and fabricators consider these recommendations to be on the conservative side, it is my experience that consistently violating them will lead to problems.

PREQUALIFICATION

One of the main reasons for the popularity of the D1.1 Code is that it provides a fabricator or erector the opportunity to prepare written prequalified welding procedure specifications utilizing prequalified joint details, prequalified steels and filler metals, and welding techniques covering the common arc welding processes for a very wide range of applications. Followed precisely, these prequalified details and procedures preclude the need for testing, making the Code as indispensable to the fabricator/contractor as a comprehensive cookbook is to a chef.

Ignore at your peril the wisdom of the rules of prequalified procedures and joint details! They have been developed over decades of production experience, and as long as they are precisely followed, they can save even an inexperienced contractor from disaster. If they are ignored, however, the consequences may be dire.

Example: Failure to Achieve CJP

A typical example would be when a designer has called for a complete joint penetration (CJP) groove weld in order to develop full strength and full fusion across a butt joint welded from both sides. Although this was specified in the contract and/or the drawings, the contractor may push too hard and decide "We're not going to back gouge those joints. Instead, we'll turn up the amperage on the machine, run it hot, and we'll burn it out. We'll develop CJP, but we're not going to follow all the rules." From the outward appearances, the result looks like a CJP weld, but in fact only partial joint penetration (PJP) has been achieved, because there is not full fusion all the way through the middle. However, the PJP groove weld is discernible only with nondestructive testing.

I have seen this happen on storage tanks, hoppers, and silos that called for CJP, but only spot, or random, radiography. Supervisors sometimes push their welding crews to cut corners by minimizing or eliminating the back gouging, thinking that by running the back weld passes hot enough, they will have enough penetration to make the tie-in. In some cases, they get away with it, but late in the game someone may decide to do some spot X-rays and find more than inadvertent lack of penetration. Then, lawsuits tend to develop. Of course, a possible solution in such cases is to re-evaluate the joint on a fitness-for-purpose basis, which may show that the joint was over-designed, and indeed, PJP performance may prove to be adequate. Then if the owner and the design engineer can be persuaded to agree to this alternate acceptance criteria, litigation will be avoided. But they are certainly accepting a lesser standard than what had been specified in the bidding process. And it should be noted that considerable amounts of time and money may be expended to achieve this agreement.

Example: Width/Depth Pass Limitation

To give another example, prequalified joints have minimum groove angles and minimum root openings, and we know from experience that those work. In an effort to improve productivity, a fabricator or contractor might decide to try a groove angle that's tighter than those permitted by the prequalified joint. The Code allows such deviations, but only if they can be proven by qualification testing.

The subject of testing brings up the importance of ensuring that qualification tests accurately predict actual production conditions. I learned this the hard way when I ignored the advice on width-to-depth ratio given in Sections 3.7.2 and C3.7.2 of the Code in order to qualify my own procedure. In the lab, I did mock-up tests that did not model the full restraint and solidification cracks in the full production joints were the ultimate result. This experience taught me how important it is to use larger pieces, thicker sections, and massive strong-backs to add restraint to lab specimens so that I am modeling the issues that will actually occur in real life. It also showed me that the testing I was doing to convince myself that I was smarter than the Code wasn't valid. I gained even more respect for the decades of engineering wisdom distilled in the prequalified procedures of D1.1.

Z Loss Factors

I have referred to D1.1 as a "history book" that contains the collected wisdom of engineers, fabricators, educators and inspectors. Tables 2.2 and 2.8, which detail Z Loss Dimensions, provide an excellent example of how technical discoveries become a part of that history. About twenty years ago, a company took it upon itself to do a variety of tests with SMAW, FCAW (both gas-shielded and self-shielded), and GMAW (short arc). They tested all these processes, making skewed-T fillet welds. They were able then to section all those single pass welds, and show that as the groove got tighter, for different processes and positions, penetration would not necessarily be achieved all the way to the root of the weld. We expect penetration all the way to the root in a 90o case, and as that groove is tightened up to, for example, 60o, we still expect full penetration. As the angle becomes tighter than that, the likelihood of full penetration drops off. All of these test results were compiled to create the Z Loss table. "Z" simply stands for the dimension from the root of the joint to the area where full fusion can be assumed to have started. This is a measurable factor that the designer should consider when sizing a weld. The table shows that at 45o, the loss factor could be "0" or it could be ⅛", depending on the process and the position. If the angle becomes even tighter, the loss factor could be as much as ¼" or ⅜", so this starts to become a significant measurement to deduct from the effective throat of welds.

Ignoring the Z Loss table can result in undersized welds. However, when welds are designed, bids have not even gone out, and the designer does not know who the fabricator will be, let alone what processes or positions the fabricator will ultimately use. Some designers may assume the worst case, ⅜", and add that dimension to their designed weld size. This approach leads to significant overwelding, with its associated extra costs and distortion. Ideally, shop drawings should be submitted for review and approval by the designer, so that processes and positions can be considered in the light of the Z Loss dimensions, and welds re-sized accordingly.

Welder Qualification

Contractors must ensure that their welders are qualified for each welding process they will use, in the position required for production, and in the direction (uphill or downhill) of welding progression. Historically, welding on conventional flat plate and rolled shapes required a relatively simple test consisting of a 5"- long, 1"-thick coupon with a 45 degree groove angle, ¼ in root opening and steel backing. However, when welding tubular connections for offshore applications became widespread, the Structural Welding Code Committee was realigned and added a separate subgroup on tubular structures in the late 1960s. As a result, the first D1.1 Code published in 1972 introduced provisions for tubular T-Y-K connections.

Welding on tubular connections is more challenging than conventional plate and rolled shape construction, because with a tubular connection, both the position of welding and the joint geometry change continuously. For complete joint penetration groove welds in tubular connections, D1.1 requires that welders pass the difficult 6GR test, which uses a 37½ degree groove angle. By this test, welders are qualified down to 30 degree. For grooves under 30 degree, welders must also pass the Acute Angle Heel Test, which covers them down to 15 degree. Too often, a contractor thinks that the Acute Angle Heel Test does not apply to the job at hand, but in fact, the heel region of a tubular brace intersecting a member at a 45 degree incline leads to a 22½ degree groove for the CJP case.

For box tubes, an additional test is required. A special corner macroetch test measures the ability of welders working on CJP groove welds to deposit sound weld metal around the relatively sharp corners (the areas of highest load transfer across the connection).

Welder testing requirements are significantly less demanding for PJP connections. Although the 6GR test may be used, it is not required; the less demanding combination of the 2G plus 5G tests with backing is an acceptable alternative. For PJP welds on box tubes, the macroetch corner test is not required. The Acute Angle Heel Test is required only if details outside the prequalified limits (less than 30 degree) are used. While the welder skill level needed to achieve a PJP groove welded connection is lower than that required for a CJP weld, this does not imply that inferior welds result, but only that greater skill is needed to handle any open root joints welded from one side.

Structural welders qualified to the 1G, 2G, 3G, and 4G levels often fail their first attempt on a 6GR coupon. Expert training, supervision, and continuing practice are essential even for those who have attained 6GR qualification. For a welder to produce sound welds under all conditions, there can be no substitute for experience.

Importance Of Preheat

In Section 3.5, D1.1 is explicit about minimum preheat and interpass temperature requirements: "The preheat and interpass temperature shall be sufficient to prevent cracking. Table 3.2. shall be used to determine the minimum preheat and interpass temperatures for steel listed in the code."

The well-understood reasons to use preheat are: 1) it drives off excess surface moisture; 2) it retards the cooling rate in both weld metal and base metal, thereby lowering hardenability, and making the weldment less susceptible to cracking; 3) it provides more time at elevated temperatures for hydrogen diffusion, which lowers the tendency toward cracking; 4) it reduces the differential temperature so residual stresses are less. The fifth reason is less often considered: the act of adding preheat, raising the temperature of the steel from room temperature to just a modest 125 or 150 degree F, can temporarily improve the toughness of even poor toughness steels (such as A36 or A572 Grade 50) enough to prevent brittle fracture during the stresses and strains of normal fabrication.

Poor toughness steels typically have a brittle-to-ductile transition temperature of 30oF or higher, so for example, consider a steel that has a transition temperature of 75oF. Typically, wintertime fabrication shop temperatures are often below this. Cracking can occur, even though the minimum required preheats of Table 3.2 were exceeded by the ambient temperature. All too frequently, examples of this have occurred when too little or no preheat has been applied prior to welding. The contractor says that the operators have been welding the same components exactly the same way for at least five months with no problem. In reality, the only thing that changed was the ambient temperature.

This article was adapted from a paper by Jeffrey W. Post, P.E., for the 1998 National Steel Construction Conference. Post is a Welding Engineering Consultant serving the fabrication community. He also has worked as a fitter, welder, boilermaker, certified welder and certified welding inspector.