Meet The Experts at MSC's Online Chats

Structural Welding Issues

In keeping with its tradition of providing timely industry information, Modern Steel Construction magazine is sponsoring a series of online chats, each one featuring industry-recognized experts in the topic at hand. In this issue, we present edited transcripts of two chats. The following is the transcript from the July 24, 2001 on-line chat with Duane Miller of the Lincoln Electric Company, which focuses on welding issues for structural steel.

Duane K. Miller

Q: Why is it necessary to control heat input in welding? Does it matter if you're using D1.1 or D1.5?

A: There are two basic reasons to control the maximum heat input: steel HAZ properties and weld deposit properties. The steel HAZ might need control for Q&T steels, like A514. For weld deposits, the most important property to control is weld CVNs. Codes treat the max value somewhat differently. In D1.5, you will (in the future code) let the plate heat up until you reach the max value you will put on your WPS. Start with the preheat level; let the plate rise in temp. and then establish the max value.

Q: AWS is in the process of creating AWS D1.8 for seismic welding. How will this fit with the other documents like D1.1 and D1.5? How will they be similar? Different?

A: For D1.5, the maximum is established by test. For D1.1, there is no specific test for the maximum. Even D1.5 is somewhat confusing, and several changes have been incorporated into the D1.5-2001 (not yet published) code. Regarding D1.1 and D1.8, there is still some question as to what D1.8 will ultimately look like. Ideas include a "guide spec," a standalone code or an addition to D1.1. If it is just an addition, it will be somewhat limited in volume. There won't be a lot a changes to D1.1 in D1.8 in many areas. One key value of D1.8 will be a specific commentary that addresses specific seismic concern. I suspect in the end, D1.8 will either be a guide spec or a supplement to D1.1. Some have been asking about seismic and bridge applications; I don't think there is a lot of interest in that right now.

Q: I'm not familiar with D1.8. Can you provide some background?

A: D1.8 is the proposed title for a new welding document with the general title "Welding of Seismically Loaded Buildings." The intent of D1.8 is to capture the lessons learned from Northridge, much of which is captured in the FEMA 350-353 publications. D1.8 is only a concept right now. Very preliminary drafts have been created, but they are far from through the consensus/approval process. The spec will need to address acceptable details for various connections (backing, weld tabs, where welds and bolts can share loads), required quality levels for various welds, notch toughness levels for various welds, how to inspect various welds. Also covered will probably be welder qualification tests for various joints. The weld metal properties will probably need to follow the FEMA pattern of CVNs at 0 degrees F as well as 70 degrees F.

Q: Will this have any effect on South Dakota?

A: I suspect it might. First, much of the country is in some sort of seismic region, and many fabricators ship to the west coast. New England is already imposing some seismic provisions. Obviously, this is a big west coast issue. There are, of course, a variety of levels of seismic, but even so, there are implications from a viewpoint of design details and welding. It all depends upon what the engineers has assumed in the design. The engineer chooses an *R* factor in calculating the seismic forces for which the building is designed. If that *R* factor is chosen higher than 3 (or required by the building code to be chosen higher than 3), it is a high-seismic application. That's when the seismic design and construction requirements kick in.

Q: Has the D1.8 draft in development been scoped for high-seismic applications only (that is, applicable only when *R* is taken greater than 3)?

A: The focus has been on the high seismic but should eventually cover all zones.

Q: What types of quality levels for welds will there be?

A: There is a matrix that has been developed based upon demand on the connection and consequence of failure. Three levels exist for each variable, so there is a 3 × 3 matrix. Based on demand and consequence, the acceptance level, and the required inspection level, vary. To illustrate, a high-high (demand vs. consequence) may require UT and MT. A low-low may require only visual. This is illustrative only; the final answer may be different. These concepts are already in the FEMA publications.

Q: Some have been critical of the requirements for MT in FEMA 353. Do you think they are justified and/or beneficial?

A: The MT emphasis is new, but based on fracture mechanics, a surface crack is more severe than an internal defect. And, UT is not real sensitive to surface defects. The demands on these welds is severe so the need for assurance of crack-free welds is important. Is it too conservative? That's what the consensus process will probably sort out. In theory, it is justified. The practical element that must be considered is this: given all the other controls on weld quality (preheat, procedure control, etc.), what is the likelihood of a crack? I suspect there will a lot of discussion on this. Regarding UT vs. MT, there is probably some additional emphasis on MT due to the general dissatisfaction with recent UT results. No NDT process will catch everything. That's why an effective "in process visual inspection" process/procedure is essential. (For more information on this topic, consult Duane Miller's 2001 T.R. Higgins Paper "Effective Visual Inspection to Ensure Weld Quality for Structural Applications," which is scheduled to be published in a future issue of *Engineering Journal*).

Q: I have started training some new detailers fresh out of school. Could you recommend a book that would teach them the basics of welding including the dos and don'ts of shop and field welding?

A: There's a lot of good information out there. Of course, I think everyone should have a copy of *The Procedure Handbook of Arc Welding and Design* from the J. F. Lincoln Arc Welding Foundation. Then, there is Omer Blodgett's classic *Design of Welded Structures*. Also, from the JFLAWF is a booklet called *The Fabricators and Erectors Guide to Welded Construction*. The Foundation makes publications available at minimum cost, approximately 1.5" thick books for $15. Not quite free, but nearly so. It's far from free, but D1.1 is important for detailers, as is the AISC *Manual*.

Q: Could you comment about the use of impact testing and general toughness requirements for structural welding?

A: In a nutshell, just like a strength related design, there is demand, and there is resistance. The calculations are more complicated, but demand must be less than the resistance. The resistance side is toughness, usually measured indirectly by CVN specimens. Just as "higher strength steel" doesn't solve ever design problem, so higher toughness material (weld or base metal) won't solve all problems. One must look at the three variables in fracture toughness evaluations: the material resistance, the stress levels and the notch or "crack" size. Thus, the question of "how much toughness" is needed for structural questions becomes an impossible one. It all depends on the stress level and the size of undetected cracks. There are examples (see Rolfe and Barsom's book) where 2 ft.-lbs. at service temperature is OK, and others where 55′ lbs is not. So, again, it depends on stress levels and details.

Q: My background is pressure vessels. Pressure vessel safety has benefited greatly from toughness requirements for welds, and it seems like buildings would, too.

A: The pre-Northridge standards had no specified minimum notch toughness levels for the welds or the steels. The designs forced high stresses into the connections, and there were notch like details incorporated into the designs. And many connections broke. Post Northridge specs usually call for notch tough weld metal, AND improved detailing (such as backing removal, weld tab removal) AND basic changes to the connection detail (dogbones, haunches, the "free flange" design).

Q: That's what I heard. I think the 15'lb. requirement that I've heard is applicable to the sorts of stress raisers associated with ordinary welded construction. Rolfe and Barsom have a pretty good empirical correlation between Kic and Cv energy for carbon steel. The 15 ft.-lb. value is a good start for constructional steel.

A: Improved performance has been seen, but what effect has the singular change in weld metal notch toughness had? It's impossible to separate all the variables. You're right. Weld metal is only part of it. Tough base metal prevents crack propagation and design details allow yielding that wouldn't happen otherwise. The 15 ft.-lb. value has an interesting history. It is summarized in the Rolfe and Barsom book. It was based on WWII Liberty ships. Steel was taken out of the ships that failed. As I recall, the cracks initiated in plates with less than 1 ft.-lb. Cracks transferred through steel with 10-15, and didn't initiate or propagate in greater than 15. In short, that's where the 15 value originated. And while 15 is a good value for ships, ship stresses and ship details, it may or may not be valid for other applications.

Q: We recently had some very large girders, 4" thick flanges and 2" thick webs. The fabricator wanted to weld the flanges to the webs using double bevel welds and then splice the flanges and the webs, again using double bevel welds. I would not allow this procedure because of the residual stresses that could occur in the girders due to the flange and web splices being made after the flanges were already welded to the webs. Is there any way of calculating what these residual stresses might be? Even an approximation would be helpful.

A: I am surprised at the requested sequence. I would prefer splicing the web and flanges first, then doing the web to flange. I agree with you that residual stresses can be minimized with a proper sequence.

Q: I agree, and that is what I required them to do, but the fabricator was unhappy because they felt they had better control over distortion by welding the flanges to the webs first. Proper sequence and preheat help reduce residual stresses.

A: Doing the butt splices first allows for shrinkage transverse to the weld. After that shrinkage takes place, then the shrinkage of the web to flange weld occurs. If the fabricator was concerned, re: angular distortion, I'd have to agree but still opt for the sequence you suggested. Also, there's going to be the issue of weld access holes, etc., with their sequence.

Q: The fabricator was staggering the splices in the web and flanges. I would have accepted all the splices in the same location splicing the girder as if it were a beam with web access holes.

A: Preheat can held "spread out" residual stresses. It allows more material to absorb the shrinkage strains and thus reduces residual stresses somewhat. Sequence can be critical. The staggering approach is an old fashioned notion that really doesn't offer value today and maybe never did in the past. The idea was to preclude a crack from initiating in a weld and propagating in the same plane into the base metal. Well, the crack will go anywhere it wants.

Q: Would staggering (of welds) be beneficial depending on compression or tension loading conditions?

A: I don't know any benefit of staggering welds. And, it usually makes the work harder to do.