If you've ever asked yourself "why" about something related to structural steel design or construction, *Modern Steel Construction's* monthly *Steel Interchange* column is for you!

Anchor Rods Under Cyclic Loads

I am looking for the tensile capacity of an ASTM F1554 anchor rod grade 36. In LRFD, its nominal strength is listed in Table J3.2 as $0.75(0.75F_u)A_b$. However it is noted at the bottom of the page that this is only for static loads. What value should be used if anchor rods are exposed to fatigue loads?

Question sent to AISC's Steel Solutions Center

If you are only concerned with fatigue on the steel rod portion of the anchorage design, you should refer to Appendix K3 of the 1999 *LRFD Specification*, "Design for Cyclic Loading (Fatigue)." The specific case is shown in Table A-K3.1, Example 8.5 on page 16.1-139 of the third edition *LRFD Manual*. For other conditions of dynamic loading on anchor rods or for the design of the full anchorage you should refer to ACI 318 as referenced in Section J10 – Anchor Rods and Embedments (note that the current applicable ACI Appendix is D rather than C as referenced in Section J10). ACI 318 Appendix D does not cover load applications that are predominantly high-cycle fatigue or impact loads. However it does discuss and present requirements for design when seismic loads are included.

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Fillet Weld Terminations

I am trying to clear up some confusion about fillet weld termination. We are fabricating beams for a 20-story building which have 1" gusset plates welded to top flanges of various large W shapes. The shop we are subletting some of the work to is terminating the fillet welds anywhere from 1/4" to 3/4" from the ends of the beams (welds vary from 5/16" in size to 5/8"). I say when a designer says to put fillet welds on both sides he means the total length of welds sides with no end termination. Could you please show me specifically where this issue is addressed and what exactly is acceptable?

Question sent to AISC's Steel Solutions Center

According to Section J2.2b (fillet weld terminations) in the 1999 *LRFD Specification* (a free download from www.aisc.org/lrfdspec) and Section 2.8.3 of the AWS D1.1:02 Welding Code, fillet weld terminations may extended to the ends or sides of parts, may be stopped short or may have end returns except for a handful of specific cases. Those cases are:

1. For lap joints in which one part extends beyond an edge subject to calculated tensile stress, fillet welds shall terminate not less than the size of the weld from the edge.
2. For connections and structural elements with cyclic forces, normal to the outstanding legs, of frequency and magnitude that would tend to cause a progressive fatigue failure initiating from a point of maximum stress at the end of the weld, fillet welds shall be returned around the corner for a distance not less than the smaller of two times the weld size or the width of the part.
3. For connections whose design requires flexibility of the outstanding legs, if end returns are used, their length shall not exceed four times the nominal size of the weld.
4. Fillet welds joining transverse stiffeners to plate girder webs shall end not less than four times nor more than six times the thickness of the web from the web toe of the web-to-flange welds, except where the ends of stiffeners are welded to the flange.
5. Fillet welds, which occur on opposite sides of a common plane, shall be interrupted at the corner common to both welds.

For your gusset plate configuration, none of these cases would apply. Hence, the fillet welds can be either extended to the ends or stopped short and still be in compliance with the *Specification* and *Code*. It is common for detailers to stop one to two weld sizes short of the part end. An inspector should not find issue with this as it is clearly stated in the *Code*, unless the weld lengths are too short to develop the required design strength of the connection.

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ASTM A490 Bolts and Fatigue Loads

Can I use A490 bolts in structures subjected to fatigue loads, such as bridges? I thought I remembered the answer was no.

Question sent to AISC's Steel Solutions Center

I believe that your reference to not using A490 bolts in structures subject to fatigue loading refers back to the era when these bolts were first introduced. The seventh edition of the *Manual of Steel Construction* limited the use of A490 bolts to static applications only for tension loading. Subsequent information and test data have led to the lifting of this particular restriction from current specifications. However, there are still special requirements applicable to bolted joints subject to fatigue loading which can be found in the 2000 *Specification for Structural Joints Using ASTM A325 or A490 Bolts*. This specification is available from the Research Council on Structural Connections at www.boltcouncil.org. The current AASHTO specification, paragraph 10.56.1.3 permits both A325 and A490 bolts.

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Rehabilitation of ASTM A9 Steel and Rivets

We are reinforcing connections of existing building from 1925 and have a few questions regarding the design approach that should be taken:

1. **We have done coupon tests for a beam and a column and the lab qualified the structural steel as ASTM A36 with F_y = 36 ksi. Can we assume that the angles and plates forming the different connections are made of the same material?**
2. **The majority of the connections are riveted. Others are bolted. Can we assume that the riveted connections are slip-critical and therefore can be combined with new weld to enhance the connection capacity? In this respect we assume that the bolts are in bearing and their capacity should be ignored when reinforcing the existing connection with weld. Please advise.**
3. **Based on an old AISC manual we have found that the maximum 3/4″ diameter hand driven rivet capacity in shear is 4.42 kips. We assume this is a slip-critical value. Is there any corresponding bearing capacity or does it not exist in rivets.**
4. **The rivet capacity in shear based on the old AISC manual is controlled by bearing (subject to the supporting member thickness) indicating different values for single and double shear. Are those values governed by the bearing on the rivet hole or bearing on the rivet itself. We are uncertain since the rivet material is weaker than the structural steel.**
5. **We are considering reinforcing the riveted shear connections in two ways:**
   (a) Replace old rivets with new high strength bolts.
   (b) Add weld around the connecting angles.
   If both ways are acceptable to the contractor, which solution is more cost effective?

Question sent to AISC's Steel Solutions Center

We can offer you some facts and suggestions:

1. Based upon the year, you probably have ASTM A9 material. Note that ASTM A36 was not released until the early 1960's. Did the lab check strength, but not chemical composition? Also, were enough samples tested to ensure the results are representative of all the steel? If the steel does in fact meet all requirements in ASTM A36, that would give sufficient basis to consider it as A36 for design purposes. Otherwise, the ASTM A9 minimum specified yield strength should be used for design calculations. Refer to AISC *Design Guide 15: Rehabilitation and Retrofit Guide* for historical strength information (a free download from **www.aisc.org/epubs** for AISC members.) A few additional considerations: In 1925, ASTM A9 steel was used and typically contained high sulfur and phosphate levels—something to avoid in a base metal if welding will be a consideration. This is not the case with steel today. Also, ASTM A9 had a lower minimum yield strength than ASTM A36.
2. Riveted connections are not slip-critical. Rivets were assumed to fill the hole (even if they rarely did) and slip issues were not considered. Nor was pretension applicable, since the rivet did not have a means to induce it. In contrast, bolts in slip-critical connections can be installed with pretension to develop the required clamping force during installation and their faying surfaces prepared for the needed slip-resistance. Refer to Section J1.9 of the 1999 *LRFD Specification* for combining welds and bolts (*www.aisc.org/lrfdspec*.) It mentions that:
   ...existing rivets and high-strength bolts tightened to the requirements for slip-critical connections are permitted to be utilized for carrying loads present at the time of alteration and the welding need only provide the additional design strength required.
3. Refer to Table 1.3b in AISC *Design Guide 15*. It contains rivet allowable tension, shear and bearing values over many years. The allowable strengths did change over time several times, as did the rivet material.
4. It is based on bearing at the edge of the hole of the connected part.
5. Rivets can be replaced with ASTM A325 high-strength bolts. If a suitable welding procedure for the old steel can be confirmed, a welded solution can also be used.

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Steel Interchange is a 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.

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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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