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!

Single-Angle Shear Connection Eccentricity

Regarding single-angle connections with the "girder" leg welded and the "beam" leg bolted, the strength is apparently based on eccentric shear in the plane of the weld. Why is the eccentricity normal to the plane of the weld (distance to the bolt line) not also considered?

Question sent to AISC's Steel Solutions Center

The weld group on the supporting member side is designed for only in-plane eccentricity. The out-of-plane eccentricity is ignored for the weld group because the top of the angle at the supported member is able to flex (i.e. there is a weld termination of two to four times the weld size from the toe of the angle to allow for this flexibility.) The flexibility gives the connection rotational ductility to accommodate beam end rotation (unlike a moment connection.)

Because of this flexibility, the moment that would otherwise be present is relieved. Hence there is no out-of-plane eccentricity in single-angle shear connections.

Sergio Zoruba, Ph.D.
American Institute of Steel Construction

Edge Distance and Load Direction

When shear is considered in a direction perpendicular to a bolt holes edge distance can the edge distances of table J3.4 of *Specification* Chapter J be reduced? If so by how much can it be reduced and is there a reduction in load capacity.

Question sent to AISC's Steel Solutions Center

The AISC *LRFD Specification* is not specific as to minimum edge distance requirements other than that the requirements of bearing strength checks of Section J3.10 must be met in order to reduce the edge distance from those shown in Table J3.4. This reduction only applies to the edge distance in the direction of the applied load. There is presently no reduction mentioned for the edge distance perpendicular to the direction of load.

Kurt Gustafson, S.E., P.E.
American Institute of Steel Construction

Bottom Flange Bending Capacity

(reprinted from December 1999)

How do you calculate the lower flange loading capacity of a steel beam to be used to support an underhung crane? Are there any published ASD or LRFD design procedures?

Question sent to AISC's Steel Solutions Center

The bottom part of the crane beam must be checked for:

1. Tension in the web.
2. Bending of the bottom flange.

Most underslung cranes will have each end supported by two pairs of wheels. Each individual wheel load will include a portion of the lifted load (in its most critical position), the dead loads, and impact. Impact is usually about 25% of the lifted load but will depend on the speed and braking ability of the hoist. Allowable stresses must be reduced due to the cyclical nature of the applied load.

The wheels must be purchased to suit the profile of the supporting crane beam, either an S-shape or a W-shape. The web tension at each pair of wheels is checked at the intersection of the web and fillet (at the *k* distance).

Referring to Figure 1, the length of resistance is seen to be 3.5 k. The 30° angle is a consensus figure used for many years. Assuming four wheels (two pairs) at each end of the crane, each wheel will support $P/4$ delivered to the supporting crane beam. In Figure 1, two wheels cause the web tension, so the load is $P/2$.

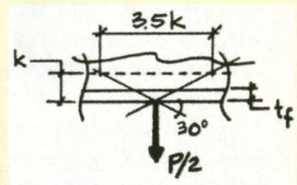

Figure 1.

The tensile stress in the web becomes:

$$f_t = P/2A = P/(2t_w)(3.5k) = P/(7k)t_w$$

Flange bending depends on the location of the wheels with respect to the beam web. Referring to Figure 2, this is dimension *e*. As stated previously, each wheel load is $P/4$.

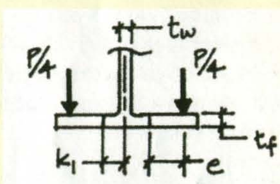

Figure 2.

The longitudinal length of flange participating in the bending resistance can be taken as $2e$ per yield-line analysis. See Figure 3.

The section modulus at the plane of bending is $(bd^2)/6$ which translates to $e(t_f)^2/3$. From Figure 2 the bending moment is $eP/4$. The bending stress is:

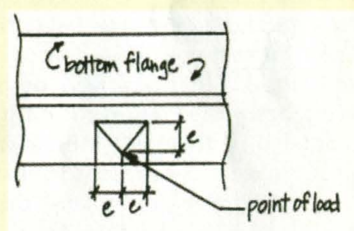

Figure 3.

$$f_b = M/S = 3eP/(4e)(t_f)^2 = 0.75P/(t_f)^2$$

Local loadings such as this often result in biaxial and triaxial stresses. These stress combinations are quite common, and designers must design accordingly. For more information on crane loading, refer to my paper in the fourth quarter 1982 *Engineering Journal* called "Tips for Avoiding Crane Runway Problems."

David T. Ricker, P.E.
Javelina Explorations

Rehabilitation of ASTM A9 Steel and Rivets

(from September 2004)

We are reinforcing connections of an 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 ¾″ 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

Whether or not riveted connections are considered to be slip-critical is an interesting question, but perhaps only of academic interest now. In support of the position that they were slip-critical is that they were designed that way when repetitive loads were present, if only by implication. Putting it another way, there is a fatigue category for riveted connections. Are such connections in bearing or are they slip-critical? If they are indeed in bearing, then the fatigue category customarily selected for this case is significantly non-conservative. The "stress category" used in the AISC *LRFD Specification* (which is typical) is D. If the rivets are truly in bearing, the stress category will be much less than D. The selection of D comes from test results and those test results are from riveted joints that in fact transmit the load by a combination of friction and bearing; some rivets are in bearing and there is enough tensile force in the rivet to provide slip resistance as well. By the way, whatever rivet tension is present arises as the heated rivet shrinks against the connected material as cooling takes place.

Geoff Kulak, Professor Emeritus
University of Alberta

Preheat and Stud Welding

(from November 2004)

I have two questions:

1. **When welding headed stud to beams or plates using the standard welding gun, are there any requirements to preheat the base material in cases where the base material is, say, 2″ to 3″ thick or more? I know that when the stud is fillet welded, AWS specifies the base material to be preheated but there is no mention of preheat when the stud gun is used.**
2. **Lastly, what problems can be expected if proper preheat is not provided and conventional fillet welds are used? Cracks in base metal? Cracks in weld? Other?**

Question sent to AISC's Steel Solutions Center

We have investigated the effects of weld power input (current and time) on stainless steel Nelson studs in the ¾″ to 1″ size range, welded to ¾″ thick ASTM A572 Gr. 50 plate. Good ductility in the base metal and the weld is dependent on using more power than is required to simply fuse the materials and is closely correlated with through heating of the base metal which may reach 400°F or 500°F opposite the weld. Preheating of the plate is thus accomplished before the weld or heat affected zone falls through the martensite start temperature during cooling. Based on the sensitivity of weld reliability to temperature conditions with ¾″ plate, for 2″ or 3″ thick material we would recommend a combination of preheat and generous weld currents (just shy of what would melt too much of the stud) in combination with rigorous testing per AWSD1.1-96 Sec 7, especially the 7.6.6.1 bend test. Stud welding involves high currents compared to stick welding. We specify a bolt tightened grounding clamp clamped at a specified torque to a clean ground (all mill scale removed) surface. We disallow spring clamps.

Henry K. Obermeyer, P.E.
Obermeyer Hydro, Inc.

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:

Steel SolutionsCenter
Your connection to ideas + answers

One East Wacker Dr., Suite 3100
Chicago, IL 60601
tel: 866.ASK.AISC
fax: 312.670.9032
solutions@aisc.org