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!

Repairing Bent Anchor Rods

I am currently working on a project where an installed anchor rod was bent during backfilling against a concrete wall. The anchor rod projection from the concrete was bent to a 45 degree angle and the contractor would like to "slowly heat the rod and straighten it." Is this an acceptable repair? It seems that this may weaken the rod. Is it better to remove the anchor rod in its entirety from the concrete? What is the typical repair for this type of damage?

Question sent to AISC's Steel Solutions Center

The subject of remedial repair for bent anchor rods is not stipulated in the 2005 AISC specification. When determining how to address the subject of anchor rod remediation, two prime considerations should be the type of rod material and the function of the rods in the final structure. One of the first things to look at is if there is apparent cracking in the bend area or severe "kinks" in the rod profile. If there is, you may consider requiring replacement.

ASTM F1554, the preferred material for anchor rods, is available in three grades: 36, 55, and 105 ksi. The ASTM 1554 limitations for bending of rods (to manufacture hooked anchor rods) are a good guide for both hot and cold bending repairs. Some conditions require special attention to determine the absence of cracking in the straightened condition:

- Bends that occur in the threaded area, since the threads tend to create notch effects and reduce the bend severity required to cause cracking
- Rods made from grades with higher strength (and lower ductility for bending)
- Bends of more than 45 degrees

Another important consideration may be the function of the rod in the final structure. If the rods serve only for construction and do not resist design forces in the completed structure, repair issues may be simplified or eliminated.

There was an article in the May 2004 issue of *Modern Steel Construction* titled "An Ounce of Prevention" (**www.modernsteel.com**) by Jim Fisher and Larry Kloiber, which discusses common anchor rod installation problems with suggested fixes.

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

Translating Between ASD and LRFD

I thought the LRFD values for bending should be F_yZ_x rather than F_yS_x as shown in the web site document "Basic Design Values" (available at www.aisc.org/2005spec). How are the ASD and LRFD bending design values derived?

Question sent to AISC's Steel Solutions Center

The basic design values for bending shown on the web site document are in the correct numerical format for both ASD and LRFD, although this fact may not be readily apparent because S_x is used where the corresponding 2005 specification formula used Z_x.

For the sake of simplicity, we can re-write Z_x as the shape factor multiplied by S_x. The shape factor about the strong axis is Z_x/S_x. For wide-flange beams, the lower bound value for this shape factor is approximately 1.1, which is conservatively incorporated in the basic design tables. Hence, $Z_x/S_x = 1.1$, or $Z_x = 1.1S_x$. The factor of safety for flexure is $\Omega = 1.67$ and the resistance factor is $\phi = 0.9$. Therefore,

ASD: $M_{allowable} = F_y Z_x/\Omega = F_y(1.1S_x)/1.67 = 0.66F_yS_x$

LRFD: $M_{design} = \phi M_n = 0.9F_yZ_x = 0.9F_y(1.1S_x) = 0.99F_yS_x$

Please note that the 1989 ASD specification used $F_b = 0.66F_y$ for laterally braced compact sections. It was not readily apparent that Z_x was embedded into such a simple expression, but it was, along with a 1.67 factor of safety. Also, it is important to realize that LRFD design values are 1.5 times the ASD allowable values in the 2005 AISC specification (i.e. $1.5 \times 0.66F_yS_x = 0.99F_yS_x$). This applies not only to flexure, but to all limit states.

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

Drawings Conflict

I'm looking for some documentation that states whether the contract drawings or the project manual (or specification) dictates when there's a conflict. The architectural drawings call for painted lintels, but the spec calls for galvanized lintels. Which one takes precedence?

Question sent to AISC's Steel Solutions Center

From the 2005 AISC *Code of Standard Practice*, Section 3.3, Discrepancies:

When a discrepancy is discovered in the Contract Documents in the course of the Fabricator's work, the Fabricator shall promptly notify the Owner's Designated Representative for Construction so that the discrepancy can be resolved by the Owner's Designated Representative for Design. Such resolution shall be timely so as not to delay the Fabricator's work.

When discrepancies exist between the Design Drawings and Specifications, the Design Drawings shall govern. When discrepancies exist between scale dimensions in the Design Drawings and the figures written in them, the figures shall govern. When discrepancies exist between the structural Design Drawings and the architectural, electrical or mechanical Design Drawings or Design Drawings for other trades, the structural Design Drawings shall govern.

The first paragraph provides for a discrepancy discovered before work is completed and says you must ask for a resolution. The second paragraph provides for a discrepancy discovered after work is completed, which is really then a basis for justification of work that was performed and the extra cost associated with a change, if it is subsequently required.

Charlie Carter, S.E., P.E.
American Institute of Steel Construction

Shop and Erection Drawings

Is there an AISC requirement that structural steel shop and erection drawings be prepared under the supervision of and stamped by a licensed professional engineer in the applicable jurisdiction, or that the drawings are to be reviewed and stamped by a P.E.? Is either of these criteria a requirement of AISC?

Question sent to AISC's Steel Solutions Center

There is no AISC requirement that shop drawings be prepared under the supervision of and stamped by a licensed professional engineer. The AISC *Code of Standard Practice* (COSP) addresses the case of a design that is completed entirely by the Structural Engineer of Record (SER), as well as a case where the connections standardized in AISC manual tables or similar information are delegated for selection and completion based upon the information provided by the SER in accordance with Section 3.1.2. If other arrangements are desired, they must be specified in the contract documents. A copy of the COSP is available to download free from **www.aisc.org/code**.

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

Beam Size

I am working on a remodeling project in an existing building that was built in the 1905 to 1915 time range. I field measured an I-shaped beam as follows: d = 15″, b_f = 5.5″ and t_f = 0.5″. Do you know its properties and designation? Its F_y?

Question sent to AISC's Steel Solutions Center

AISC *Design Guide 15* contains a reference for historic shapes and specifications. The document is titled *AISC Rehabilitation and Retrofit Guide* and is available at **www.aisc.org/epubs**.

From your description of 15″ depth and 5.5″ flange width, it sounds as though the section may be an American Standard Beam, commonly called an I-beam, and today referred to as an S-shape. However, these shapes have tapered flanges. If this profile accurately describes your cross-section, and the average flange thickness is 0.5″, this shape likely corresponds to the designation S15×35. There were several manufacturers of shapes during the 1905 to 1915 era, and similar sections from different manufacturers may have slightly different section properties. You may want to search *Design Guide 15* for other possible shapes that fit your measurements.

Design Guide 15 also includes a historical summary of ASTM specifications for structural steel that will give you an idea of the standards for steel tensile and yield strengths available at the time. The common grades indicated in this book for the time period in question have yield strengths that range from 24 ksi to 30 ksi. Given this, you may consider performing limited testing to verify the yield strength and that the minimum requirements of the time were actually used in the project.

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

Gusset Plate Yield Line (from July 2005)

We are currently detailing our first Special Concentric Braced Frame (SCBF) in Seismic Design Category D and have a question. We understand the concept of creating a yield line through the gusset plate. Our issue concerns the column base at the floor slab. The gusset plate will need to be huge to establish the yield line above the concrete slab because the slab would confine the brace and gusset, preventing the yield zone from forming. The best idea we can come up with is to wrap the gusset and brace end with a layer of compressible material, such as rigid insulation, prior to pouring concrete. Is this something that has been discussed or written about already? Does our idea seem reasonable? We are assuming that slab confinement is an issue here and would like to know if it is typically dealt with or simply ignored.

Question sent to AISC's Steel Solutions Center

Reader Rob Marostica, P.E., offered this additional response:

I read the question in the July 2005 *Steel Interchange* regarding gusset plate yield lines at column bases and would like to add the following:

When a brace buckles, a hinge will form near the end of the brace. There are three possible outcomes to consider:

1. The hinge forms in the brace while the gusset plate remains elastic.
2. The hinge forms in the gusset plate and the gusset accommodates the rotation in a ductile manner.
3. The hinge forms in the gusset plate, but a fracture occurs because the gusset is unable to accommodate the rotation.

The connection must be detailed to prevent the third outcome from occurring. Fracture is most likely to occur when the assumed yield line is partially embedded in concrete. This can be prevented by extending the gusset plate, elevating the connection by placing the frame column on a concrete pier, or providing a compressible material between the yield line and the top of slab. The first two allow the yield line to occur above the top of slab elevation. The third allows the yield line to occur below the top of slab, but the compressible material permits the brace and gusset to rotate.

Another option is to locate the entire connection below grade and rely on the restraint provided by the concrete closure pour and slab on grade to force the hinge to occur above grade in the brace. The engineer may reinforce the slab on grade, bond the gusset to the closure pour with headed studs/deformed bar anchors, or enclose the entire connection within a concrete pier to ensure that the hinge is forced into the brace. The buckling strength of the brace and geometry of the connection will determine the most practical approach.

Rob Marostica, P.E.
BHB Consulting Engineers
Salt Lake City, UT