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!

100-Year-Old Steel

I am trying to determine the load-bearing capacity of a roof on a building that is about 100 years old. The steel has been identified as S9×19.75 (purlins) and S15×33 (girders). Is there any way, for purposes of calculations, to determine the yield strength of the members? I'm guessing it's unlikely that the members are ASTM A36 steel. What was standard for the time?

Question sent to AISC's Steel Solutions Center

If the building is about 100 years old, it is definitely not ASTM A36 steel. The ASTM A36 Standard was not issued until 1962.

The year 1900 represented the issuance of the first ASTM *Standard for Structural Steel*, which was intended to bring uniformity to the various steel materials being produced at the time. It is only a guess as to what the actual characteristics of a specific material may represent for that time period; that is, if it was produced to an ASTM Standard. The 1900 ASTM A9 *Standard for Buildings* listed tensile strength of 60,000 to 70,000 psi with a minimum yield point of 35,000 psi. The 1909 ASTM A9 *Standard* listed slightly lower tensile strength (T.S.) at 55,000 to 65,000 psi and minimum yield point at half of the T.S. AISC *Design Guide 15: AISC Rehabilitation and Retrofit Guide* is a reference for historic shapes and specifications. Therein you will find a historical summary of ASTM specifications for structural steel.

Unless there is good documentation as to what was specifically specified for the project, it would be prudent to undertake a testing program to determine reasonable material parameters for the structure. For further guidance refer to Appendix 5 of the 2005 AISC *Specification for Structural Steel Buildings* (a free download at **www.aisc.org/2005spec**), which covers evaluation of existing structures. Section 5.2 of this appendix covers material properties.

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

Shear Tab Edge Distance

Table 10-9a of the 13th edition *Manual* appears to incorporate a vertical edge distance of 1¼ in. instead of 1½ in.

I had read that the hole in shear tabs can get rounded up to ¼ in. when the bolt goes into bearing. Also, when the plate is sheared, there is ¼ in. of material that may be "mushed," and not counted as part of the material, which is the reason why you are permitted an edge distance of 1 in. for flame or saw cut edges, as compared to 1¼ in. for sheared edges. If this is true, then the final calculable vertical edge distance for the bottom bolt on these plates will be ¾ in., after removing ¼ in. for the "mush" from the shear, and then ¼ in. for the vertical elongation from the bolt in bearing.

Question sent to AISC's Steel Solutions Center

It used to be that a 1½ in. vertical edge distance was required, but the new procedure allows use of the 1¼ in. value more typical of other shear connections.

The ¼ in. bearing elongation relied upon for connection ductility and accommodation of the simple beam rotation is a horizontal deformation of the hole, not a vertical deformation. That is why the procedure requires two times the bolt diameter for horizontal edge distance.

A vertical deformation of ¼ in. is possible if the connection were loaded to its full shear capacity, and bearing were the critical limit state. But that is a separate issue entirely, as the rotational ductility has already occurred at that point. We are just assessing the ultimate performance capability of the connection, if we are concerned about the vertical bearing deformation.

Also, I don't think there is a need to deduct ¼ in. for the effect of shearing. We deduct only an additional 1⁄16 in. when calculating net area for a bolt hole to recognize that the hole may be punched (sheared), to account for the possibility of damage to the edge of the hole. But that means the depth of that damage is only about 1⁄32 in.

Charlie Carter, S.E., P.E.
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Seismically Braced Frame

I have a building in which I used X-braces to transfer the lateral loads to the foundations. In a few bays, I have to move the bottom of the braces up three feet from the finish floor elevation to allow access for doors. This building is in a high seismic area (Seismic Design Category E), and is a one-story building (approx. 18 ft to bottom of steel). Can this still be considered an Ordinary Concentrically Braced Frame?

Question sent to AISC's Steel Solutions Center

If the bracing members are designed as tension-only, neglecting the strength in compression, the K-configuration is not appropriate for an OCBF system. In other cases, see Section 14.3 of the 2005 AISC *Seismic Provisions* for OCBF special bracing configuration requirements.

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

Grade 50 Angle Availability

Are angle shapes produced in Grade 50 material?

Question sent to AISC's Steel Solutions Center

The base grade for angle shapes is ASTM A36. The steel availability search function on the AISC web site (**www.aisc.org/availability**) will list producers of various shapes based on the base grade for that shape. Some mills may produce ASTM A992 or A572 Grade 50 angles, but you would need to inquire as to specific availability. If you are looking for a specific shape, you may try contacting one or more of the steel service centers or producing mills. Contact information is listed for the various mills and service centers on the same web site.

Kurt Gustafson, S.E., P.E.
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Seismic Requirements for Composite SLRS

I would like some clarification on the seismic design provisions of AISC 341-02, AISC LRFD-99 and the provisions of IBC 2003. Table 1617.6.2 of IBC permits designing steel structures as "Structural Steel Systems not Specifically Designed for Seismic Resistance." IBC 2205.3, "Seismic requirements for composite construction," states that in Seismic Design Category B or above, the design of composite systems shall conform to AISC 341, Part II. If I'm in a SDC C, and want to avoid "detailing for seismic", if I use $R = 3$, $\Omega_0 = 3$ and $C_d = 3$, can I design a composite system per AISC LRFD-99, without following AISC 341 Part II?

Question sent to AISC's Steel Solutions Center

No, Section 2205.3 of IBC 2003 requires the use of AISC 341 Part II for SDC B or higher if you plan to use a composite lateral system like those provided in AISC 341 Part II. Composite lateral framing systems are not categorized as "Structural Steel Systems not Specifically Designed for Seismic Resistance."

Structural Steel Systems not Specifically Designed for Seismic Resistance are permitted in SDC C or lower if you use $R = 3$, $\Omega_0 = 3$ and $C_d = 3$.

Sergio Zoruba, Ph.D., P.E.
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Bent Anchor Rods

I recently received an RFI stating that one of the four 1¼ in. diameter ASTM A307 anchor rods at one braced column was bent out-of-plumb by 22° and asking for a fix solution.

In the past I have seen steel workers swinging big sledgehammers to straighten crooked anchor rods. However, I am hesitant to recommend this practice. I would like to recommend that they heat the offending rod and bend it gently back into place using a large piece of pipe as a lever. Is this an acceptable way to straighten slightly bent anchor rods or is there a preferred or published methodology? Some related questions are "If heat is used, how much should they heat the rod?" and "Could this procedure be utilized for various grades of anchor rods?"

Question sent to AISC's Steel Solutions Center

AISC *Design Guide 1: Base Plate and Anchor Rod Design*, Second Edition, which was just recently released, contains a discussion on repairing bent anchor rods. The authors have made some recommendations as to grade of rods, maximum angles of bend and diameters that should be heated for bending rather than cold bent. A pipe bending device called a "hickey" should be used in the straightening process. It is recommended that ASTM F1554 Grade 36 rods over 1 in. diameter be heated to a maximum of 1200 °F to make bending easier. ASTM A307 material is similar to this grade. All AISC design guides are available at **www.aisc.org/epubs** (free downloads for AISC Members).

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

Metric Bolts

I would like to know the industry standard conversion for 1" ASTM A325 Imperial bolt to a Metric bolt. Please indicate the standard metric size bolt.

Question sent to AISC's Steel Solutions Center

There are two separate ASTM Standards for these bolts, namely ASTM A325 and ASTM A325M. Similarly, ASTM A490 and A490M also exist. ASTM A325 addresses Imperial A325 bolts while ASTM A325M covers Metric A325s. Please note that there is no conversion between these standards. Each contains a different set of bolts with different physical size characteristics.

For example, the ASTM A325 Standard allows a 1 in. nominal diameter bolt. However, the ASTM A325 Standard does not contain a 25.4 mm (1 in.) nominal diameter bolt; rather, it contains an M24 bolt (i.e. 24 mm).

Using an M24 bolt in a standard hole sized for a 1-in.-nominal diameter bolt would make the hole oversized for the 24 mm bolt diameter. Therefore, a slip-critical joint will now be required by the specification. The next larger metric bolt is M27. Using an M27 in a standard 1¹⁄₁₆ in. hole would create erection problems during bolting, as the typical ¹⁄₁₆ in. play in a standard hole is gone. It would be a very tight fit, not practical under normal construction tolerances.

As such, there is no conversion between the systems. Any attempts to convert must consider the aforementioned issues. Either the entire design should be in Imperial units, or Metric units, to avoid these pitfalls.

Please refer to ASTM standards at **www.astm.org** for additional information on these bolt specifications.

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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.

If you have a question or problem that your fellow readers might help you 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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