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! Send your questions or comments to solutions@aisc.org.

Restrained Beam

What constitutes a "restrained beam" for fire-rating purposes?

You can find a good listing of what constitutes "restrained" or "unrestrained" ratings in Table X3.1 of ASTM E119, Appendix A. For steel framing, this table classifies steel beams welded, riveted or bolted to the framing members as "restrained."

A good article titled "Restrained Fire Resistance Ratings in Structural Steel Buildings" by Gewain and Troup, appeared in *Engineering Journal*, Second Quarter, 2001, and can be found online at **www.aisc.org/ej**. Search under either author's name and the year. The download is free for AISC members.

Section 4.3.2 in Appendix 4 in the AISC *Specification for Structural Steel Buildings* (AISC 360-05) states, "Steel beams, girders and frames supporting concrete slabs that are welded or bolted to integral framing members (in other words, columns, girders) shall be considered restrained construction."

Kurt Gustafson, S.E., P.E.

Double-Angle Compression Member

The term r_{ib} in Equation E6-2 of the AISC 360-05 *Specification* is defined as "radius of gyration of individual component relative to its centroidal axis parallel to member axis of buckling." Does this mean that r_{ib} will be equal to r_x of individual angle for LLBB angles, while it is equal to r_y of the individual angle for SLBB angles?

No. I presume that since you are evaluating a built-up angle member with an LLBB configuration, the bolts are through the long legs. These bolts will be subjected to shear when the built-up member buckles about the Y-axis, which lies in the plane between the long legs. Thus, r_{ib} is equal to r_y of the individual angle, since that angle axis is parallel to the Y-axis of the built-up member.

Kurt Gustafson, S.E., P.E.

Peak Stresses

Does the AISC *Specification* define acceptance criteria for steel when using finite-element modeling to assess the stress distribution?

No. Assessment of results employing finite-element modeling techniques is really a matter of engineering judgment. The AISC *Specification* limit states are based on use of average stresses in most cases; not peak stresses as may result from a finite element analysis. When such is used, engineering judgment is involved as to how this may relate to the *Specification* parameters, and is beyond the scope of the *Specification*. Localized stresses in members are assumed to redistribute through inelastic deformation thus justifying the use of average values. Where such localized stresses can be cause for failure, such as at net sections, the *Specification* accounts for them separately. Please note that AISC typically deals in member strength values that correspond to the entire member cross-section, while finite element programs are likely to give stress values that vary across the member cross-section.

Kurt Gustafson, S.E., P.E.

ASTM A307 Bolts

Why are ASTM A307 bolts not recommended for slip critical connections? Can they be used in low demand slip-critical connections?

ASTM A307 bolts are a carbon steel fastener with lower strength and not suitable for pretensioning; the bolt would just stretch without much residual pretension if you were to try to pretension it. Because you can't induce a pretension of any significance, you can't develop the clamping force necessary to accommodate either a pretensioned or slip-critical installation.

Kurt Gustafson, S.E., P.E.

Weld for Single-Plate Shear Connection

On page 10-101 of the 13th edition AISC *Steel Construction Manual*, it is indicated that the leg size of the double fillet welds for a single-plate shear connection is required to be ⅝ t_p. Is there a check required of the support base metal to which the weld is applied?

Yes. Any base metal to which a fillet weld is applied must be capable of developing the required shear being transmitted through the fillet weld. The procedure for making the connecting element rupture strength at the welds is shown on page 9-5 of the 13th edition AISC *Manual*. In the case of a single-plate shear connection being welded on only one side of the support element, each weld will have a unique shear plane, and thus the $t_{min} = 3.09\ D\ /\ F_u$ equation for F_{EXX} = 70ksi welds will apply. If there are single-plate shear connections with equal leg sizes being applied on exactly opposite sides of the supporting element, the $t_{min} = 6.19\ D\ /\ F_u$ equation for F_{EXX} = 70ksi welds will apply.

Note that these checks are based upon the support thickness developing the strength of the fillet weld(s). If the actual thickness did not meet the minimum required thickness, it is permissible to use a more exact approach to determine the actual loading of the web and resulting required thickness. Most webs will meet the above checks, however.

Kurt Gustafson, S.E., P.E.

Flexure of Flat Plate

How can I determine the strength of a flat plate bent about the strong axis?

Section F11 of the AISC *Specification* (a free download at **www.aisc.org/2005spec**) defines the limit states of yielding and lateral-torsional buckling of rectangular bars and rounds in flexure. The limit state of lateral-torsional buckling will generally define the flexural strength of relatively thin, laterally unbraced plate members bent about the strong axis.

Kurt Gustafson, S.E., P.E.

steel interchange

Historic Beam Designation

I am investigating a building designed in 1967, with the plans dated February 1, 1968. The plans indicate some roof beams as being 18B35. I have a 6th edition AISC *Manual* dated 1967 and this shape is not indicated. Can you tell me where I might find the properties of this shape?

There was a grouping of light 18-in. beams (W18×40 and W18×35) that were added in the 7th edition AISC *Manual* published in 1970. I believe that these shapes were added by some mills in the late 1960s. It is possible that this may be what is designated as an 18B35, even though it was designated as a W18×35 by the time the *Manual* was published. You may want to check the dimensions of the shape against those listed in the 7th edition AISC *Manual* to see if that is what you have.

In case you do not have a copy of the 7th edition *Manual*, AISC has developed two sources of information pertaining to historic shapes. AISC *Steel Design Guide 15* is a reference for historic shapes and specifications. There is also the AISC Shapes Database v13.1H, where the H stands for Historic. Both of these resources are available as free downloads by AISC members at **www.aisc.org/epubs** or can be purchased by others.

Kurt Gustafson, S.E., P.E.

Rivet Replacement

When removal of existing corroded rivets is required, what is the appropriate nomenclature and tightening method for high strength bolts being used as a replacement? Should slip-critical or pretensioned connections be considered? What are the differences in installation and inspection methods between the slip-critical and pretensioned options?

The bolt installation methods for pretensioned joints and slip-critical joints are identical. The only differences are the surface preparation and inspection of the faying surfaces of slip-critical joints. Additionally, slip-critical joints are intended for new construction, not the retrofit that you describe.

If your main concern is replacing the clamping force of the rivet, the bolts should be specified as pretensioned. You may need further notes on your details to ensure that the construction sequence does not result in the degradation of the faying surfaces of the joint, or the loss of clamping force in any previously pretensioned bolts. However, if a snug-tightened joint would be permitted by today's standards, there is no need to do anything more than install the bolts as snug-tightened.

Heath Mitchell, P.E.

Table B4.1 – Compression or Flexure?

Table B4.1, Case 1 description says "Flexure in flanges of rolled I-shaped sections and channels." For bending about the major axis, the stress distribution on the top flange (for a simply supported beam subject to gravity loads) is uniform compression. Therefore, should Case 3 be used for the flange classification?

No. Case 3 applies to a member that is subjected to uniform compression on the entire cross-section. The limiting lambda values are derived differently for a member subject to flexure as compared to a member subject to uniform compressive stress.

Brad Davis, Ph.D., S.E.

Maximum Bolt Tension

We are installing ASTM A325 galvanized bolts by the turn-of-nut method. We are following the preinstallation verification procedure using a tension calibration (Skidmore) unit and making sure that we meet the extra 5% over the 70% mininum tensile strength. I understand there is not an upper limit of the applied pretension on the bolt, with the upper limit in effect resulting in the bolt breaking, or threads stripped during installation. The question has come up if this is true, then why can't we reuse a bolt (A325 galvanized) if it has been previously pretensioned by the turn-of-nut method.

The intent of the RCSC *Specification* is not to allow bolts to be tightened to the point of breakage or thread stripping. Rather, the pretensioning procedures are intended to essentially yield the bolt. Because at that level of strain, the stress-strain curve has hit a plateau, some degree of strain above the target is not detrimental to the performance of the connection.

All bolts possess some degree of ductility, which allows them to reach some strain beyond this plateau without fracture. However, only ungalvanized A325 bolts have been deemed to have enough ductility to undergo repeated tensioning.

The degradation of galvanized ASTM A325 (and black ASTM A490) bolts in repeated cycles of pretensioning is illustrated in Section 4.5 of the *Guide to Design Criteria for Bolted and Riveted Joints*, 2nd edition, which is available as a free download at **www.boltcouncil.org**. This clearly shows why galvanized A325 (and black A490 bolts) are not allowed to be reused.

Larry S. Muir, P.E.

Block Shear

Should block shear failure be considered for the connection elements loaded in compression?

No. Block shear consists of a shear failure along one or more planes combined with a tension failure along one of more planes. Block shear cannot occur without a plane subjected to tension and therefore need not be checked for compression loads.

Larry S. Muir, P.E.

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Steel Interchange is a forum 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 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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