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.

Open Parking Garages

What is the maximum number of stories allowed for a steel parking garage structure with no fireproofing applied to the structural members? IBC 2006 is the referenced building code.

If the parking structure is considered "open" (see IBC Section 406.3), Table 406.3.5 defines the area and height limitations based on the type of construction. As long as all the provisions described in Section 406.3.6 are met, an open parking garage with eight tiers or less of parking and a limited area per tier (as described in 406.3.6) is allowed to be classified as Type IIB Construction. In Type IIB Construction, the structural frame has a zero-hour fire-resistance rating requirement.

Erin Criste

Lateral Bracing for Cantilevers

I have been told that bracing the tension flange of wide-flange cantilevers is more effective than bracing the compression flange in order to prevent lateral-torsional buckling. What is the rationale behind this statement?

This is correct. Section 6.3.1 in Appendix 6 of the AISC *Specification* requires that cantilever bracing must be attached near the tension flange.

The center of rotation for a cantilevered beam is located at a point below the bottom flange. Accordingly, an unbraced cantilevered end will undergo greater rotational displacement at the top flange than at the bottom flange. Bracing the bottom flange will move the center of rotation upward, but the top flange will still have the tendency to rotate out of plane. Therefore, in the case of cantilever beams, the best way to restrain the out-of-plane displacements associated with lateral-torsional buckling is to brace the top flange.

Heath Mitchell, P.E.

Minimum Composite Shear Connection

When designing a composite beam, what is the significance of 25% composite action? I don't find any requirement in the AISC *Specification* about 25% being a minimum, but some software design programs always assign this as a minimum requirement.

This is discussed in the Commentary to Section I3.3 of the 2005 AISC *Specification* as follows:

"There is no minimum requirement for the amount of shear connection. Design aids in the U.S. often limit partial composite action to a minimum of 25% for practical reasons, but two issues arise with the low degree of partial composite action. First, less than 50% composite action requires large rotations to reach the available flexural strength of the member and can result in very limited ductility after the nominal strength is reached. Second, low composite action results in an early departure from elastic behavior in both the beam and the studs. The current provisions, which are based on ultimate strength concepts, have eliminated checks for ensuring elastic behavior under service load combinations, and this can be an issue if low degrees of partial composite action are used."

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

One-Half of Uniform Load Capacity

In the 9th edition AISC *ASD Manual*, it stated that if the design drawings did not indicate beam end reactions, the connections *must* be designed for one half the total uniform load capacity shown in the Allowable Uniform Load Tables. Does the AISC 13th edition *Manual* include this same statement?

No. The use of the "uniform load capacity" approach for connection design is generally frowned upon, and often results in a ridiculous representation of actual beam end reactions. It is highly recommended that actual beam end reactions be shown on the contract documents; rather than a one-size-fits-all requirement, which often is uneconomical and can be impossible to achieve using standard connections. The requirements for information that must be provided, including for connection design, are covered in Section 3 of the AISC *Code of Standard Practice*.

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

Historic Angle Shapes

I am analyzing steel roof trusses for a building constructed in 1928 with double angle webs and chords. I believe that the material was supplied from Bethlehem Steel. The top chord appears to be 2L5×3 with a thickness of either 9⁄16 or 5⁄8. Do you have any shape properties for this size angle? My manuals only go back as far as the 6th edition, and neither the AISC *Iron and Steel Beams 1873 to 1952* nor AISC *Design Guide 15* include properties of angle shapes.

I am also looking for information on structural steel and rivet properties for that timeframe.

Shape properties for angles have remained fairly constant over the decades, since their cross-sections do not change appreciably. I'd say just look at the section properties of the angles shown in your 6th edition *Manual*, but the thicknesses you mention have not been included in the AISC *Manual* for many decades, and were not listed in your 6th edition *Manual*. A close approximation can be calculated using basic engineering mechanics methods if the fillet is neglected, and we were able to send you the information from an older AISC *Manual* as well.

Structural steel for buildings in the 1928 timeframe generally met the ASTM A9 Standard, which had a required tensile strength of 55,000 to 65,000 psi, and a minimum yield point requirement of ½ T.S. or not less than 30,000 psi. Rivet steel had a required tensile strength of 46,000 to 56,000 psi, and a minimum yield point requirement of ½ T.S. or not less than 25,000 psi.

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

steel interchange

Fatigue Stress Range

I have a question regarding the direction of the inequality sign in Equation A-3-1 of the AISC *Specification* Appendix 3. Please explain how I am supposed to compare the actual live load stress range, allowable stress range, and threshold stress range when applying these provisions.

There are actually two limit states that come into play here:

First determine the actual live load stress range. If this is less than the threshold stress range, F_{tb}, no evaluation of fatigue is required. If it is greater, check that the actual live load stress range does not exceed the allowable stress range, F_{sr}.

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

Historic Beam Terminology

What are the shape profiles of both a "B" and a "u" section of structural steel in a building constructed in the 1950s? Members called out with B and u designations are used as beams in a roof framing plan dated 1953. The "B" profile is also called out for columns as well as beams. The "u" shape is only for beams of shorter span. The drawings are lettered neatly and the "u" is intentionally written lower case with other upper case characters near it (e.g. 6 B 16 and 8u13.75) The drawing also calls out wide-flange shapes, junior beams, angles, junior angles, and channels, so I guess it's not one of these profiles.

The common designation for a channel in the 1950s was to use a "u" with the bottom corners squared, such that it resembled a channel. The "B" designation was for a miscellaneous light beam. If you have a 5th edition AISC *Manual* (which was most prevalent in the 1950s), you will find these common designations listed in Part 1. The first number indicates the nominal depth of the shape, and the last number represents the weight in lbs/ft.

These light beams differ from the junior beams used in the same era by their size, though the distinction probably originates from the slang terms that became official designations. Light beams generally had 4-in. flange widths, while junior beams had flanges that were 3 in. or less in width.

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

Damaged Column

A steel column on our project was badly damaged and I am not sure that it can be repaired. A subcontractor is proposing to fix the column by straightening it. Do you have any information on methods to repair the column?

I'm assuming the member is bent and not torn if you are considering the option of straightening it. It is possible that your member could be heat straightened without compromising the strength of the member. This process is commonly used to repair bridges that have been struck by vehicles and members that have been dropped during erection.

Heat straightening is somewhat of an art and requires some degree of experience to perform correctly. The temperature of the steel must be maintained below that which would cause changes to its properties. FAQ 2.3.3 on the AISC website at **www.aisc.org/faq** provides further information and resources on heat cambering or heat straightening.

Larry S. Muir, P.E.

Stiffening to Resist Torsion

I am reviewing an existing beam subject to torsion. Is there a practical way to reduce the calculated angle of twist due to torsion in the beam without changing the member sizes and/or adding bracing? Would adding stiffener plates help?

Adding stiffening plates into the web of the beam would have little effect on the resulting twist about the longitudinal axis—they will just go along for the ride. Adding continuous vertical plates on each side, such that a closed box shape results, would have a greater effect on improving the torsional performance. See AISC *Design Guide No. 9: Torsional Analysis of Structural Steel Members* for further information and guidance.

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

Established Grade

What is the definition of "established grade" as used in Section 7.6 of the AISC *Code of Standard Practice*? There it states, "The variation in elevation relative to the established grade for all Bearing Devices shall be equal to or less than plus or minus ⅛ in. [3 mm]." Is the established grade the specified grade or the as-built grade?

The AISC *Code of Standard Practice* does not give an explicit definition for the term "established grade," but its usage is similar to another term that is defined: established column line. The established column line is a best-fit line based upon as-built conditions and established by the Owner's Designated Representative for Construction (usually, the general contractor).

The paragraph in Section 7.6 just previous to the one that contains the sentence you quoted talks about the lines and grades established by the Owner's Designated Representative for Construction. Thus, we can infer that the established grade is an elevation established by the Owner's Designated Representative for Construction that is to be used when applying the erection tolerances for the structural steel. This elevation can differ from the specified grade elevation, but any such difference will result from the deliberate selection of an alternative elevation by the Owner's Designated Representative for Construction.

Charles J. Carter, S.E., P.E., Ph.D.

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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.

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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