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!

Determining I_z for Single Angles

I am searching for a method to calculate the I_z of an L6×4×3⁄8. The r_z is listed in the LRFD manual; but not the I_z.

Question sent to AISC's Steel Solutions Center

By using the r_z and A for the angle section as tabulated in the *Manual*, you can determine I_z from the relationship $I_z = A\, r_z^2$.

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Overstrength Requirement for Seismic Design of Diaphragms

When applying the amplified seismic load with a system overstrength factor of 2 as required by Table I-4-1 of the 2002 *Seismic Provisions*, what overstrength factor should be used for the roof diaphragm? Do I need to apply the same factor?

Question sent to AISC's Steel Solutions Center

The diaphragm is part of the seismic force resisting system, including the collectors and chords. The *Seismic Provisions* apply only to structural steel elements and their connections in the seismic force-resisting system and would therefore cover structural steel chords and collectors. However, the *Seismic Provisions* do not require that any element in the diaphragm be designed with an overstrength factor.

Therefore, to determine whether the diaphragm needs to be designed using an overstrength factor, you have to turn to the applicable building code or ASCE7. For example, Section 12.10.2.1 of ASCE 7-05 requires collector elements in Seismic Design Category C, D, E, or F to be designed for the load combinations with the overstrength factor.

Table I-4-1 in the 2002 seismic provisions applies only in the absence of a specific definition of overstrength factor in the applicable building code (Section 4.1 of Part I). The 2005 seismic provisions no longer include Table I-4-1; instead, they directly reference the applicable building code or ASCE 7-05 for the overstrength factors. Nothing has really changed—the intent of the provisions has just been clarified.

In general, the overstrength factor for the system, whatever it may be for your case, is applied to the components that are intended to remain elastic. A good explanation of this requirement can be found in Commentary C12.4.3 of ASCE 7-05.

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Historic Specification Issue Dates

Where can I find historic information on old AISC manuals and standards like the AISC specification, *Code of Standard Practice*, **RCSC specification, and common ASTM standards? Is there a separate list of the dates of publication of these standards?**

Question sent to AISC's Steel Solutions Center

You can find listings of historical AISC manual, specification, and code issues in *Design Guide 15: AISC Rehabilitation and Retrofit Guide—A Reference for Historic Shapes and Specifications*. Historical data on ASTM standards publication dates covering structural steel and a historical review of RCSC specifications are also included in *Design Guide 15*.

Design Guide 15 is available as a free download for AISC members at **www.aisc.org/epubs**, or from the AISC bookstore at **www.aisc.org/bookstore**.

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Bolt Bearing Resistance

On page 10-124 of the 13th edition manual, r_n = the nominal strength of one bolt in shear or bearing, kips. Please define "or bearing." Which bolt edge distance governs: the bolt closest to the edge of the connection material or the bolt in the interior of the connection? Furthermore, the specification on page 16.1-111 specifies, "For connections, the bearing resistance shall be taken as the sum of the bearing resistances of the individual bolts." Which is it?

Question sent to AISC's Steel Solutions Center

To answer the first part of your question, "or bearing" is included in the definition because the nominal bolt strength, r_n, is calculated considering bolt shear *and* bearing, including edge and end distance effects. Strictly speaking, the smallest nominal bolt strength (considering both shear in the bolt and bearing) should be used as r_n. However, the specification language highlights the intent that all of the bolts should not be penalized due to the bearing condition of the bolt closest to an edge (called the "poison" bolt). Accordingly, Section J3.10 states, "for connections, the bearing resistance shall be taken as the sum of the bearing resistances of the individual bolts."

The simple presentation of the calculation in the *Manual* can be adapted for this as follows: consider a single row of four bolts, for which $C = 3.07$. This means the bolts are 77% effective ($C/n = 3.07/4 = 0.77$). If bearing controls, the poison bolt can be avoided in this case by calculating the bearing strength without eccentricity and subsequently multiplying by 0.77. In this manner, we are really summing the resistances of the individual bolts for bearing as required in AISC specification Section J3.10, and subsequently multiplying by C/n, to account for the eccentricity.

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Anchor Rod Embedment Detail

A contractor on one of my projects has substituted ASTM F1554 threaded rod with loose nuts in lieu of the specified headed anchor rods. Some of the rods are resisting tensile loads. It is my understanding that the nuts were not welded to the threaded rod as recommended by AISC *Design Guide 1: Column Base Plates*. Are there any additional recommendations for corrective action for anchor rods with loose nuts embedded in concrete that will be used to support tensile loads?

Question sent to AISC's Steel Solutions Center

Design Guide 1 suggests tack welding the embedded nut to the rod as a precaution to keep it from rotating in the lower nut in case the tightening of the top nut causes the rod to turn. There are additional precautions that we may suggest to ensure the rod does not turn during installation: make sure the exposed threads on the rod and nut are clean and lubricated to minimize resistance to the nut installation; pay close attention to the rod during nut installation to make sure it does not turn with the nut; and do not attempt to over-tighten the nut.

Although such substitutions should be done only with approval of the specifier, a threaded and nutted rod is equivalent to a headed rod for anchor rod design. Material, design, and installation of anchor rods is addressed in *Design Guide 1*. The AISC specification and ACI 318 are referenced for embedment design of the rod to resist applied forces. Pullout strength of anchors in tension is covered in Section D.5.2.5 of ACI 318 Appendix D and is based on bearing of the embedded nut or head against the concrete. Hooked rods are also covered, but their use is not recommended if there is a calculated tension force on the rod.

Most nuts used with Grade F1554 are heat-treated ASTM A563 products, and caution should be taken with regard to heat input when welding to such a material. Historically, tack welding has been done on anchor rod embedment in lieu of other alternatives. However, at the least, such tack welding should only be done on the unstressed end of the rod and done under controlled shop conditions. Today other alternatives, such as damaging the threads or using a threadlocker, are preferable to the tack welding option.

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Axial Compression Capacity in March 2006 Steel Interchange

A response given for the question "Axial Compression Capacity" in the March 2006 edition of Steel Interchange (available at www.aisc.org/steelinterchange) was, in part, as follows: "For the 2005 AISC specification, we recognized that UM plates are no longer available and eliminated them from the determination of the resistance factor and factor of safety. As a result, phi is 0.9 and omega is 1.67 in the 2005 AISC specification; hence, the difference in strength you noted."

I have checked both LRFD and ASD versions of the steel manuals cited below and here are the results:

LRFD 13th edition = 892 kips
LRFD 3rd edition = 844 kips
ASD 13th edition = 594 kips
ASD 9th edition = 608 kips

The explanation of the increased phi value for the LRFD version seems okay, as the new 13th edition manual states a higher load capacity compared to the previous LRFD manual. However, as you can see, the new ASD table in the 13th edition appears to have a lower capacity value compared to the previous ASD manual, which is the 9th edition. A decrease in omega, as explained in the article, should have resulted in a higher capacity in the new ASD table.

What explains the decrease in load capacity, comparing the new 13th edition ASD table with the old 9th edition for the member size W14×132 and effective length of 30'?

Question sent to AISC's Steel Solutions Center

The word *approximate* is used in the article in relation to the ASD factor of safety used in the 1989 specification. To be more precise, a variable factor of safety was applied in older versions of the ASD specification to the column strength estimate to obtain the allowable stress. This resulted in a different factor of safety for short columns as opposed to those for longer columns entering the Euler slenderness range. The 2005 AISC specification uses a constant factor of safety, hence the difference you noted.

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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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