Steel Interchange is an open 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.

The opinions expressed in *Steel Interchange* do not necessarily represent an official position of the American Institute of Steel Construction, Inc. and have not been reviewed. It is recognized that the design of structures is within the scope and expertise of a competent licensed structural engineer, architect or other licensed professional for the application of principles to a particular structure.

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

Steel SolutionsCenter
One East Wacker Dr., Suite 3100
Chicago, IL 60601
tel: 866.ASK.AISC
fax: 312.423.4651
solutions@aisc.org

NET AREA CALCULATION

According to Section B2 of the 1999 *LRFD* and 1989 *ASD Specifications*, the width of a bolt hole must be taken as $^1/_{16}$ in. greater than the nominal dimension of the hole when calculating the net area. Please clarify whether this provision should read as $^1/_{16}$ in. greater than the nominal bolt diameter, as the nominal hole dimension tables in the manuals are based on adding a $^1/_{16}$ in. to the nominal diameter of the bolt.

Question sent to AISC's Steel Solutions Center

In reference to Section B2 of the AISC *Specifications*, net area is calculated by taking the nominal hole size as $^1/_{16}$ in. greater than the nominal dimension of the hole. This accounts for potential damage to the periphery of the hole in the hole-making process.

As an example, a $^7/_8$-in. diameter bolt can be used with a $^{15}/_{16}$-in diameter standard hole or a $1^1/_{16}$-in. diameter oversized hole. Adding the $^1/_{16}$ in. as indicated above, the dimensions used for calculating net area would be 1 in. and $1^1/_8$-inch, respectively.

The same bolt used in short- and long-slotted holes would require $^{15}/_{16}$ in. $\times$ $1^1/_8$ in. and $^{15}/_{16} \times 2^3/_{16}$ in. hole dimensions. A hole width of 1 in. is used in the net area calculation for these two cases (the "short way") and $^{13}/_{16}$ in. and $2\,^1/_4$ in., respectively (the "long way").

Sergio Zoruba, Ph.D.
American Institute of Steel Construction
Chicago, IL

ORDER OF PRECEDENCE WHEN DISCREPANCIES EXIST

Comment sent to AISC's Steel Solutions Center

When discrepancies exist between Design Drawings and Specifications, the 2000 AISC *Code of Standard Practice* (*COSP*) Section 3.3 states that the Design Drawings shall govern.

In the Engineers Joint Contract Documents Committee (EJCDC) and American Institute of Architects (AIA) documents, there is no order of precedence between the Contract Documents. Refer to EJCDC 1910-8, Paragraph 3.01.A. and AIA A201, Paragraph 1.2.1. The Conditions of the Contract require the Contractor to ask for a clarification from the Architect or Engineer before proceeding with the Work. The *COSP* should be revised to reflect current industry documents.

The wording in the *COSP* implies there is an order of precedence in the Contract Document, although this approach was changed several years ago. It is much better to require the Contractor to question the Architect or Engineer to determine which is correct before proceeding with the Work. If the Contractor proceeds based on an order of precedence, the order of precedence may be incorrect and any corrective changes made after the Work has proceeded can result in higher costs.

The *COSP* is consistent with AIA and EJCDC provisions. When discrepancies are discovered, all three require the discoverer to inform the responsible people so that the discrepancies can be resolved. Each of the three indicates it is not the responsibility of the contractor to discover discrepancies.

If a discrepancy is discovered before work is performed, this works as you wish. However, consider the following scenario. After the structural overhang framing was completed, the architect walks onto the job and says that overhang was supposed to span 15 feet as shown on the architectural drawings, not 10 feet as shown on the structural drawings. Should the fabricator have to provide and the erector erect new steel for free? Absolutely not, so the *COSP* gives an order of precedence for these cases.

I believe the first paragraph above is clearly stated in the text of the *Code*. It does not allow a fabricator to proceed before the resolution is returned. However, it does require the resolution to be returned in a timely manner. This item has been forwarded to the AISC Code Committee to consider if additional clarification is warranted.

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

COLUMN ERECTION TOLERANCES

In the 2000 AISC *Code of Standard Practice*, Section 7.13.1.1 states "the angular variation of the working line from a plumb line shall be equal to or less than 1/500 of the distance..." Is this the angle between the working line and plumb line, or is it the ratio of the displacement to the column length?

Question sent to AISC's Steel Solutions Center

It is the maximum permitted deviation at one end of a column based upon a 1/500 angle between the working line and plumb line that intersect at the other end of the column.

If the world were perfect, the variation (distance) between the top and bottom working points of a column oriented in the z-direction should ideally be zero in the x- and y-directions (i.e. a perfectly vertical column.) In such an idealized case, the working line of the column and a plumb line would be identical to each other. However, the world is not perfect.

In actual construction, the line between top and bottom working points is not likely to be a plumb line. The 1/500 maximum slope defines the envelope within which the column can lean and be acceptable. Note that other tolerances apply in some cases (see *Code*.)

Keith Mueller, Ph.D.
American Institute of Steel Construction
Chicago, IL

WHITMORE SECTION

Question sent to list server at www.seaint.org

On page 7-114 of the AISC *ASD/LRFD Connections Manual*, a sketch of the Whitmore Section is shown for a particular example. Consider a connection with the brace connecting to a beam only (no column). I see the dimensions L_w, ℓ_1 and ℓ_2 remaining the same. The dimension ℓ_3 will increase in length from the corner of the section until a line parallel to the brace intersects the beam flange. Then per the last paragraph on the previous page, the column length would be the average of ℓ_3 and ℓ_1 (ℓ_2 is negative in this case). Would this be correct?

The geometry of the buckling lengths ℓ_1, ℓ_2 and ℓ_3 as you describe them sound reasonable. You can use a negative value in determining the average for the buckling check. It makes sense, because that means the gusset is very stiff there—the beam flange is restraining buckling.

Note also, however, the modification in the effective length factor for the case of a gusset supported on one edge. We recommend K = 1.2 when only one edge of the gusset is connected instead of the 0.5 used when both edges of the gusset are connected.

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

SEISMIC COLUMN SPLICES

Question sent to AISC's Steel Solutions Center

In reference to Section 8.3a(2) of 1997 AISC *Seismic Provisions* [Section 8.4a(2) in the 2002 version] states that the minimum required strength for each flange shall be $0.5R_yF_yA_f$, where A_f is the flange area of the smaller column connected. Does A_f pertain to the area of just one flange or both flanges of the smaller column?

The term A_f represents the area of one flange of the smaller column. By writing the minimum required strength of each flange as $0.5R_yF_yA_f$, the intent is that at least 50% of the flange expected strength must be developed. Note that Section 8.3a(1) requires that PJP groove welded joints satisfy 200% of the required strength as a minimum as well [Section 8.4a(1) in the 2002 version.]

Sergio Zoruba, Ph.D.
American Institute of Steel Construction
Chicago, IL

AVOIDING COLUMN CONTINUITY PLATES

Question sent to AISC's Steel Solutions Center

When designing moment frames, we will typically use continuity plates between the column flanges when a beam or girder frames into the column. Although this keeps the column size down, it also increases fabrication cost. Would you have any comments regarding the elimination of continuity plates by using a heavier column?

It may be better to use a heavier column to eliminate the transverse stiffeners (continuity plates) for moment connections attached to the flange of the column. Increasing the column size, sometimes substantially, to satisfy the Chapter K provisions governing the need for transverse stiffeners, is often more economical.

In the end, it may be more economically feasible to use a larger column then to use a smaller column requiring fabrication and installation of continuity plates. Visit www.aisc.org to download a free Excel-based script program from the Steel Tools section of the AISC website. It performs calculations to help you quickly choose a column that does not require column stiffeners based upon information in AISC *Design Guide No. 13: Stiffening of Wide-Flange Columns at Moment Connections—Wind and Seismic Applications.*

Jason Ericksen
American Institute of Steel Construction
Chicago, IL

NEW QUESTIONS

COMPOSITE FLOOR PENETRATIONS

Where can I find literature or references regarding the design of composite floors with penetrations?

NEW ASTM STANDARDS AND OLD AISC SPECIFICATIONS

ASTM A992 wide-flange shapes and F1554 anchor rods are not listed in the 9th edition ASD Manual nor in the 1989 ASD Specification. Can I use these newer materials in designs involving the ASD, or does the design need to be based on the LRFD Specification?