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 over-sized 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. × $1^1/_8$ in. and $^{15}/_{16}$ × $2^3/_{16}$ 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").

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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.

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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.*)

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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.

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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 $0.5R$