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!

Maximum Spacing

In Section J3.5 of the AISC 2005 *Specification* (www.aisc.org/2005spec), the maximum distance from the center of any bolt to the nearest edge of parts in contact shall be 12 times the thickness of the connected part under consideration, but shall not exceed 6". What is the basis of this requirement?

Question sent to AISC's Steel Solutions Center

The requirement is intended to provide for the exclusion of moisture in the event of paint failure, thus preventing corrosion between the parts, which might accumulate and force these parts to separate. More restrictive limitations are required for connected parts of unpainted weathering steel exposed to atmospheric corrosion.

For more detailed information, you can download the *Engineering Journal* paper "Considerations in the Design of Bolted Joints for Weathering Steel" (1983) by Roger Brockenbrough from www.aisc.org/ej.

Sergio Zoruba, Ph.D., P.E.
American Institute of Steel Construction

Continuous vs. Intermittent Welds

I am analyzing an existing steel truss constructed of various sizes of W14 beams. A few of the top chord members are over-stressed. Could plates be added between the flanges of the existing section to increase the effective r_y? This would lower the *kl/r* value in the weak-axis and increase the capacity of the member. In a recent *Modern Steel Construction*, the issue of continuous welding vs. intermittent welding was addressed. Could intermittent welding be used in this case, or would a continuous weld be required?

Question sent to AISC's Steel Solutions Center

Let me take the issue of continuous versus intermittent welds first. The *Steel Interchange* article to which I believe you are referencing was intended as a cautionary note for high fatigue cases where the allowable weld stress range may be severely limited. It was intended to convey the idea that many times the engineer will select continuous welds in high fatigue situations to reduce the effective stress on the weld throat. Properly sized intermittent welding is OK for static load conditions, but is generally avoided in high-cycle fatigue applications.

The adding of plates to reinforce structural members is a common method to achieve an increased capacity. Often truss chords are not clear along the length, and thus reinforcing could possibly involve problems of continuity at the joints. These are details that will likely have to be reviewed in the design process.

Kurt Gustafson, S.E., P.E.
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What are $k_{detailing}$ and k_{design} values?

In the 3rd edition LRFD *Manual*, there are two different k values listed for a given wide-flange shape. One is $k_{detailing}$ and the other k_{design}. Why is this and which one should I use?

Question sent to AISC's Steel Solutions Center

The k-dimension of wide-flange shapes were changed in the 3rd Edition LRFD *Manual* to reflect the way the mills roll shapes. Each of the mills has a different radius for each shape and our values are calculated so that shapes from any producer can be used. The largest fillet is generally of interest to the detailer, while the smallest fillet is generally of interest to the designer.

For detailing, use $k_{detailing}$ values, which are the larger values in fractional form. For structural calculations, use the k_{design} values, which are the smaller values in decimal form. As you can tell, there is variation between the design and detailing values for any given wide-flange shape, hence these two values represent the upper and lower bounds of current fillet radii used by mills in wide-flange shape production. The idea is to use an upper-bound value for detailing to ensure that the connection angles do not encroach upon the fillets of the shape, and to use a lower-bound value for design calculations (i.e. such as web local yielding checks, etc.) to ensure we do not account for more capacity than might really exist.

Bill Liddy
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Seismic Provisions: Application of R_y

Can you please expand on what is meant by the statement found in AISC *Seismic Provisions for Structural Steel Buildings* dated May 21, 2002 in Section 6.2 regarding when R_y can be applied to F_y in the determination of the design strength. The statement reads as follows: "When both the Required Strength and the Design Strength calculations are made for the same member or connecting elements, it is permitted to apply R_y to F_y in the determination of the Design Strength."

Question sent to AISC's Steel Solutions Center

Specific provisions for some Seismic Load Resisting Systems stipulate that the Required Strength be determined by multiplying the Nominal Strength of a certain member or connecting element by the value of R_y for the corresponding material grade. This overstrength is primarily of interest when evaluating the Design Strength of another connecting element or member.

We want to know and control which element or member will fail first in order to insure that the Seismic Load Resisting System will behave in a predictable manner when deforming to dissipate seismic energy. This requires that the overstrength in the ductile element(s) or member(s) be considered when comparing against other members and connections.

However, it is not of interest when evaluating the Design Strength of the same member to which the value of R_y was applied in the determination of the Required Strength.

Therefore, when both the Required Strength and Design Strength calculations are made for the same member or connecting element, it is also permitted to apply R_y in the determination of the Design Strength. In effect, doing so cancels the R_y on both sides of the equation.

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Splices and CVN Toughness

If we weld a heavy cross-section beam to column using a partial-joint-penetration (PJP) groove weld, does the member still need to have the required CVN values in the "*k*" area or is the CVN testing from the flange specimen good enough?

Question sent to AISC's Steel Solutions Center

AISC *Specification* Section A3.1 only stipulates that CVN testing is required for such members that are spliced using CJP groove welds. The subject of using PJP welds in this situation is not discussed.

CJP welds correlate with high stresses in welded joints both due to the increased available strength of CJP welds and potential residual stresses. In response to issues arising many years ago AISC implemented provisions to alleviate problems arising from these stresses. Those provisions include access hole dimensions, weld requirements and this Core CVN requirement. The anecdotal record since the provisions were implemented indicates they were successful in eliminating problems. PJP welds do not have the same available strength nor do they have the same size that results in the residual stresses. The AISC requirement for Core CVNs is not written as a requirement for PJP welded members.

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Weak-Axis Moment Connection Plates

We have a situation where connection plates are required at beam to column web moment connections. The beam flanges are attached to the connection plates with complete-joint-penetration groove welds. The connection plates are the same thickness and are the same grade of steel as the beam flanges. The bottom of the lower connection plates were detailed to be flush with the bottom of the bottom flange of the beam. The beams were overrun at the mill. The as-built condition is that the bottom of the bottom flange is lower than the bottom of the lower connection plate by 1/4 to 3/8 inch. Is there a tolerance allowance for a condition such as this? Can a small eccentricity between the center of the flange and the center of the connection plate be allowed?

Question sent to AISC's Steel Solutions Center

This is a common case of tolerances that is addressed in the Appendix of AISC *Design Guide 13: Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications* (**www.aisc.org/epubs**). Therein, a design procedure is outlined for such connections based on the NASCC conference paper "Moment Connections to Column Webs" (1998) by Tom Ferrell (**www.aisc.org/epubs**.) It states:

"The top connection plate thickness is equal to t_f plus ¼ inch. This additional thickness is necessary to accommodate tolerances for fabrication and beam flange tilt. Note that the bottom of this connection plate is aligned with the bottom of the beam top flange."

"Also, the bottom connection plate tolerance should be t_f plus 3/8 inch."

Based on your description of the beam flange being CJP welded to an equal thickness continuity plate with a ¼" to 3/8" offset, it does not appear that the problem is as much one of eccentricity of the center of the plate versus flange, as one of developing the CJP weld. You may want to look at Section 2.7.1 of AWS D1.1-2004 for requirements related to thickness transitions.

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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.

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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:

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