Steel Interchange

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* at:

Steel Interchange
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This month's* Steel Interchange *responses were provided by Charles J. Carter, S.E., P.E., Director of Engineering and Continuing Education, from questions on the Structural Engineers Association International email list-server.

Does anyone know of any clearly documented, recommended guidelines for limiting the pre-composite deflection of steel beams in composite floor systems? I've always tried to limit the total deflection to a reasonable number (1" or so, depending on the span) and the initial deflection was never a serious issue.

This is addressed in AISC Design Guide No. 5 *Low- and Medium-Rise Steel Buildings* by the late Horatio Allison and in AISC Design Guide No. 3 *Serviceability Design Considerations for Low-Rise Buildings* by Jim Fisher and Mike West. In these references, two issues are addressed:

1. *The effects of floor deflection on partitions and walls.* If you are considering beam deflection under the wet weight of concrete, this may not be a major concern because the partitions and walls will most likely be installed later. In other words, post-composite deflections may be more relevant to partition and wall considerations.

2. *The effects of concrete ponding.* The more deflection you get, the more concrete you get; the more concrete you get, the more deflection you get, and so on. In addition to the design guides, there is also an article in the 3rd quarter 1986 AISC *Engineering Journal* by John Ruddy called "Ponding of Concrete Deck Floors" that will help you with this issue.

Charles J. Carter, S.E., P.E.
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Is there a minimum radius for a cope at the top of a beam? I can't find a specific radius in either volume of the AISC ASD *Manual* or my old copy of *Detailing For Steel Construction*. These references only indicate that a radius is required but do not specify a minimum radius. I'm working on a project that calls for a 1" radius and this seems large.

Lowell McCormick

AISC doesn't require any specific radius for the corner of a beam cope, but a radius should most definitely be provided. Just say "no" to notches! The AISC LRFD *Manual*, Volume II, notes on page 8-225, *All re-entrant corners must be shaped notch-free per AWS D1.1 to a radius. An approximate minimum radius to which this corner must be shaped is ½-in.*

Let me explain some background. AISC used to say the radius had to be ½-in. Some inspectors carried a quarter with them and if the radius didn't match the curve of the quarter, they rejected it, whether it was smaller or bigger. AISC removed its requirement because any practical radius will be fine. "Practical" means a radius in size from the drill bit used to make the bolt holes for a ¾-in.-diameter bolt or the ½-in. that usually gets programmed into the CNC flame cutter. Unacceptable practice is to flame-cut a sharp corner or to band-saw to a corner, or worse yet, to overrun the cuts!

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I have an long-time industrial client who recently replaced a bunch of "missing" structural bolts with "Grade 8" (I assume SAE, Grade 8) bolts. I can only guess what happened to the original bolts. I'm sure they weren't stolen, so

that leaves the options of working loose or breaking under repetitive load cycles. This is a vertical bolt, attaching a crane girder to a column cap plate–a fatigue condition. I'm not comfortable with this replacement. As far as I can tell, the grade 8 machine bolt is hard, tough, and brittle.

I recommended replacing the new bolts with ASTM A325 bolts, but the client is reluctant to spend money twice for the same work. Thoughts?

From AISC's publication, *A Guide to Engineering and Quality Criteria: Common Questions Answered*:

Question 6.2.5: Is it acceptable to substitute SAE J429 grades 5 and 8 bolts for ASTM A325 and A490 bolts, respectively?

Answer: No. The strength properties of SAE J429 grade 5 bolts and ASTM A325 bolts are identical; likewise, SAE J429 grade 8 bolts are the strength equivalent of ASTM A490 bolts. These material specifications differ, however, in that ASTM A325 and A490 specify thread length and head size, whereas SAE J429 does not. Additionally, quality assurance and inspection requirements for ASTM A325 and A490 bolts are more stringent.

Since AISC and RCSC specifications call for pretensioned installation for your application (which involves fatigue), I would question whether the SAE fasteners that were installed were properly installed. Also, I'd question if the head size, threading and quality assurance were adequate for the application. Usually, the request to substitute an SAE fastener for A325 or A490 comes because SAE's are cheaper. Perhaps that gives you the answer right there.

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New Questions

What is the allowable bending stress for a WT member loaded in a direction parallel to the stem? I assume the allowable stress depends on whether the flange is in compression or tension. Does the AISC *Specification for Allowable Stress Design of Single-Angle Members* apply?

William B. Kussro, P.E.
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I read the *Steel Interchange* article from December 1999 regarding the bending of steel crane rails (bottom flanges) due to the applied loads from underhung crane wheels. Are there any references for the capacity of the crane rail *top* flange when it is supported using a clamp system? In other words, the loads from the crane wheels are supported by clamping the top flange of the rail at the ends. This would impart a bending in the top flange as well as tension in the web.

Stephen L. Nelson, P.E.

Can you recommend a technical paper, book, or manual on the finer points of overhead monorail design, both straight and curved?

Dick Neel
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