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
Attn: Keith A. Grubb, S.E., P.E.
One East Wacker Dr., Suite 2406
Chicago, IL 60601
fax: 312/670-0341
email: grubb@blacksquirrel.net

Steel Interchange Fans, Take Note!

One click from the re-designed www.modernsteel.com will take you to the *Steel Interchange* archive of published questions and answers, updated monthly. Brows and search past *Steel Interchange* columns. You may also email responses and comments directly from the web site. Columns prior to 1997 have been incorporated into the AISC Engineering and Research Department FAQ, available on the web at www.aisc.org/library.htm.

A special thanks to Charles J. Carter, Director of Engineering and Continuing Education, for this month's *Steel Interchange* column.

Seismic Requirements for Simple Buildings

I am designing a very simple building in a seismic application. Seismic loads even with R = 4 are quite within the frame capacity. Do I still have to meet the requirements of a special moment frame?

If you are not taking advantage of the higher R-factor for an SMF, you do not have to meet the detailing requirements for an SMF. The AISC *Seismic Provisions* allow a system of normal ductility (no special seismic detailing) if the R factor is not taken greater than 3. Others have indicated that 2.5 might be a better number for the R in an undetailed system, but you get the idea, and the R factor probably isn't exact enough to justify more than one significant figure anyway. In the AISC *Seismic Provisions*, there are also other systems to select from as follows: OMF R = 4, IMF R = 6, SMF R = 8. Note that those R factors are actually specified in the 1997 NEHRP Provisions.

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

When to Use ASTM A449 Bolts

While researching an issue related to bolt specifications I encountered a confusing part of the AISC Specification.

Section A3.4 states that A449 bolts are permitted only in connections requiring bolt diameters *greater* than 1.5". Table I-B in Part 4 of the *Manual* only provides tension loads for A449 bolts of 1.5" diameter or less, implying that this material is not available for diameters greater than 1.5". Am I misinterpreting something here?

ASTM A449 isn't permitted for structural bolting applications, except in diameters greater than 1.5" (for example, beyond the range of ASTM A325, which only covers up to 1.5" diameter bolts.) ASTM A449 bolts are prohibited entirely for slip-critical joints.

However, A449 bolts *can* be used for anchorage applications. In the "old" days, it should have been specified for anchor rods when you wanted a strength equivalent to A325 in an anchorage device. Why? Because A325 is specifically a steel-to-steel structural bolting specification, *not* an anchor rod specification. Today, I'd like to see everyone steer toward the new anchorage specification ASTM F1554, which includes a strength grade roughly equivalent to ASTM A325. You can read more about this and other materials recommendations here:

www.engr.psu.edu/ae/steelstuff/matls.htm

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

Checking Camber in the Field

I have had problems with inspectors in the field checking camber after beams are in place. Some beams, even of the same size and weight, deflect varying amounts, with some even losing all of their camber. What is the proper way to inspect camber in field and what are the possible reasons for beams displaying differing amounts of camber loss—even total loss of camber?

Field checking of camber is a difficult problem to address. It may be a hopeless case because of all the factors that affect the final position of the beam, including: variations in the assumptions made by the designer when the camber ordinate is selected in the design phase, the tolerances on beam camber (-0/+1/2" for most cases), camber loss during shipping and/or erection, the effects of end restraint due to the actual stiffness of connections that were assumed to be simple shear connections in the design, system effects (unanticipated sources of strength, such as incidental composite action during construction), and the variations in the actual dead and construction loads that are applied during construction. It's also possible that field variations result from errors, such as under-cambering in the shop or cambering upside down (which should be pretty obvious!)

Field-checking of camber done as an attempt to check the fabricator's work is inappropriate. The AISC *Code of Standard Practice* Section 6.6.4 (**www.aisc.org/ code.html**) indicates that the fabricator's work on beam camber must be checked in the shop with the beam in the unstressed condition. This may seem unreasonable to those who are expecting a beam with a certain amount of camber in the field, but the reality is, if you are checking the work that the fabricator did on camber, you can't do so after shipping and erection work some of the camber out, and dead and other construction loads are acting to further reduce the camber. Also, there is an effect on camber due to the actual end restraint provided by simple connections.

If field checking of camber is done as an attempt to achieve level floors, I think there are better ways to do that. Actually, I do not think beam camber should be considered as a sole or even primary means to provide a level floor. Sure, using beam camber is an important part of the process, but I think it is much more important to provide a floor slab that is thick enough to allow for the construction tolerances and variations in cambered beam profile. The cost of that extra concrete is not much in comparison to the floor leveler that would have to be brought in after the fact.

Some further recommendations on beam camber can be found in the steel economy article that I wrote with Tom Murray and Bill Thornton in the April 2000 issue of *Modern Steel Construction.*

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

Economical Composite Beam Design

What are some general guidelines for making composite beam designs economical?

The art of making composite beams economical is the right balance between beam weight savings (a pound of steel is comparatively cheap today!) and the added costs of welded studs and cambering. And you have to factor in construction loading of the non-composite cross sections. Depending on the deck details, beams can usually be considered laterally braced due to the puddle welds of the deck and the shear studs that are in place, but really light cross sections may not be enough to hold up the dead load of wet concrete.

Also requiring consideration: don't slim down that floor so much that it bounces all over the place and scares the occupants when their coffee sloshes. AISC Design Guide 11, *Floor Vibrations Due to Human Activity* by Thomas M. Murray, David Allen and Eric Ungar is an excellent reference to help you keep your floors free of perceptible vibration. You can order it through www.aisc.org or by calling AISC pubs at 800/644-2400.

In general, I equate a welded stud to about 10 pounds of steel. That assumes a stud costs $2 installed and steel costs $400 per ton, so adjust it as you see the need. I think you'll find that 50 to 75 percent composite systems make for very economical floor construction.

For camber, it's a bit trickier. I've seen some estimates on the order of $20 per beam when they can be cold cambered. Heat cambering may be more expensive since it can be more labor intensive.

Check with your favorite fabricator or two to see what they think about any issues of economy. Most of all, try to stay out of the mindset that a little extra steel weight is a big extra cost. One extra pound of steel really only costs you about a quarter, since the cost of fabrication and erection do not change much with changes in weight that don't change the basic nature of the fabrication or erection.

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

Technical Note:
Large-Diameter HSS

Large-diameter HSS are produced by a few of the many HSS manufacturers listed at the Steel Tube Institute website, www.steel-tube-institute.com.

Note that HSS with periphery dimensions that exceed 64" do not meet the ASTM A500 specification, which is limited to a periphery equal to or less than 64". If HSS with larger peripheries are specified, the product provided likely will be similar to ASTM A500, but when you are inquiring with the producers, ask them to define exactly what you would be getting.