# 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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**I just picked up the May 2000 issue of *Modern Steel Construction*. The front has photos of chevron bracing. In the matching article, sketches show yield lines not square with the brace. As I understand it, the yield line, 2" from the end of the brace, must be parallel to the cut end of the brace and to the yield line at the opposite end. Testing has shown that other assumed yield lines may result in a curved yield line and then tear the gusset plate. It looks like the shown method may not meet either criteria. What is the practice by those who are designing SCBFs?**

*David Merrick, S.E.*

You are correct, David. The yield line for an SCBF gusset is intended to be perpendicular to the axis of the brace to satisfy the "2t free" dimension requirement in the AISC *Seismic Provisions*, Part 1, Section 13. There has been a lot of discussion on this topic, particularly because the requirement drives up the gusset size whenever the brace can't be placed at a 45-degree angle.

The engineer on this project clarified that the schematic bracing details shown in the article unintentionally visually distort the severity of the tilt of the yield line relative to the brace axis. In the actual connections, the engineer indicates they are much closer to perpendicular.

I've seen details that don't satisfy this geometric requirement on several occasions. Although the writers of the 1997 AISC *Seismic Provisions* had the picture clear in their minds, readers of the resulting document did not always pick up on the nuance that the yield line should be perpendicular to the axis of the brace. To explicitly express this aspect of the intent, a clarification was made in AISC *Seismic Provisions Supplement No. 1*, which was published in 1999. You can download AISC *Seismic Provisions Supplement No. 1* here:
www.aisc.org/documents.asp?mode=docdetail&doc=153

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

*New question:*

**Does anyone have experience with Aerosol Zinc-Clad cold galvanizing compound? The fabricator wants to substitute this for hot-dip galvanizing on brick shelf angles. The Sherwin-Williams fact sheet says it is equal to or better than hot-dipped galvanizing.**

*Mark Daskilewicz*

I have salt-spray tested (ASTM B117) both hot-dip, mechanical and cold spray galvanized coatings on plate and tubing coupons. The coupons were cleaned with degreasing solvent, dried and sprayed in "shop" conditions. The cold spray actually resisted red rust corrosion twice as long (1,200 hours) as hot-dip galvanized coupons, as the paint's low surface tension did not allow the salt fog to wet the surface.

I performed this test to determine the validity of cold galvanizing finished structural products, as well as making repairs to shipping and handling damage of traditional galvanized coatings.

*Robert Hathaway*

*Via email:*

**What happened to the "standard beam gages" as listed in the *Manual of Steel Construction*, 7th edition? I can't seem to find gages for some of the newer shapes.**

You can get the gages for these new shapes from your old *Manual* by looking in ASTM A6 to see into which flange width grouping they fall. Then, apply the gages for those shapes (which are essentially constant throughout a flange-width grouping) to the new shapes. Said more simply, match the new shape flange width to the groupings of the old shapes for which you have the standard gages and you're home free.

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AISC will be adding back standard gages into the 3rd edition LRFD *Manual*. They were deleted previously because many people incorrectly insisted that only the gage listed in the AISC *Manual* could be used because that was what AISC "required." Rather than completely remove them, I would have just rewritten what was in there to educate people that the gages were recommendations, but that any gages that satisfied edge distance requirements and allowed room for bolt entering and tightening requirements could be used.

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

*Question from February 2000:*

Girts are typically designed to support the vertical tributary area weight of siding for each girt level as well as the horizontal (component and cladding) tributary area wind pressure for each girt member.

Considering that siding is necessarily erected from the base upward and that the diaphragm arching effect of the siding would certainly bridge between columns and load them directly, why does it make any sense to consider channel girts to eccentrically support siding weight on one flange? Suppose no sag rods are used?

*James G. Brooks, P.E.*
**OnBoard Engineering**
**Newark, DE**

*Another response:*

In response to the question about girts supporting the sheeting posed by James G. Brooks, P.E., in the pre-engineered building industry, the sheeting is supported by bearing on the slab and not the girts.

Typically the girts must be supported to level by blocking prior to the installation of the sheeting. That blocking is not removed until after sheeting is completed. The girts are designed for wind loads only and in some cases diagonal support rods from the columns to third points on the lower girt must be installed if the sheeting does not have bearing i.e. above door openings or projected facades. Relying on diaphragm for support can lead to field problems.

*Rod Roberts*
**Field Services Manager**
**Varco-Pruden**

## New Questions

**If fixed base columns are used, should these seismic overstrength factors similarly be applied to the design of the column connection to the foundation? In other words, if fixed base steel columns were used, would the seismic overstrength factor need to be applied to the base plate and anchor bolt design? For columns at braced bays, should the seismic overstrength factor be used in the design of the anchor bolts?**

*Paul H. Lind, P.E.*
**Butler Construction**
**Kansas City, MO**

**I am retrofitting some existing building trusses to handle additional loading. These trusses have continuous top and bottom chords and all welded connections. Even under existing loads, the only way that way the truss members can be made to work is to assume that all members are pin ended. Obviously, this eliminates all member bending moments.**

**Clearly this is what the original designer assumed. But, given the as-built conditions, I don't see how I can legitimately make the same assumption. Are there any references that discuss this?**

*John Brock*