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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Is it a correct assumption to say that a levelness criteria should not be specified for a composite slab on unshored structural steel? From everything I've read and in my dealings with numerous contractors, this appears correct. Basically, it seems you might get excessive ponding if you specified a levelness criteria which could be a problem in the pre-composite loading condition for the beams and the metal decking.

Alan Holland

It's generally not a good idea to depend entirely on trying to level the system by placing concrete to a level condition. Depending on the flexibility of the underlying steel framing, you may end up with an impossible ponding situation—steel deflects, more concrete is added, steel deflects etc.) The best way to get a level floor is to thoroughly understand the aspects of the problem: deflections, deflection calculation procedures, connection stiffness effects, and camber practices and tolerances. Some excellent references to have on hand are:

- Allison, H., *Low- and Medium-Rise Steel Buildings, AISC Steel Design Guide Series No. 5*, AISC, 1991.
- Fisher, J., and West, M., *Serviceability Design Considerations for Low-Rise Buildings, AISC Steel Design Guide Series No. 3*, AISC, 1990.
- Ricker, D. T., "Cambering Steel Beams," *Engineering Journal*, V. 26, No. 4, 1989.
- Ruddy, J., "Ponding of Concrete Deck Floors," *Engineering Journal*, V. 23, No. 3, 1986.
- Larson, J. W., and Huzzard, R. K, "Economical Use of Cambered Steel Beams," *Proceedings*, National Steel Construction Conference, AISC, 1990, pp. 13.3-13.15.

Keith A. Grubb, P.E., S.E.
Chicago, IL

Can A490 bolts be used to resist shear loads in a cyclic application, such as for a tower crane? Can anyone direct me towards an appropriate reference?

Laura Kannady, P.E.

Yes, A490 bolts can be used to resist cyclic shear loads. However, they are required to be pretensioned. See the 1994 RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts* and the 1993 AISC LRFD *Specification* (Chapter J).

Heath Mitchell
American Institute of Steel Construction
Chicago, IL

I recently read the following note on an old drawing set—

All structural and miscellaneous steel which shall remain exposed to view shall be fabricated and erected in accordance with the AISC "Specification for Architecturally Exposed Structural Steel" without gaps or open joints.

Can anybody point me in a direction to find this or give me some insight on what the note refers to?

That's an out-dated reference to an old AISC document that is now a part of the AISC *Code of Standard Practice.* Refer to Section 10, which has all the requirements for Architecturally Exposed Structural Steel (AESS).

AISC's new *Code of Standard Practice* (2000) now available. You can download it in Adobe Acrobat .pdf format at:

www.aisc.org/documents.asp?mode=docdetail&doc=186

The code may also be purchased in printed form for $10 (item no. S303) at 800/644-2400.

Charles J. Carter, S.E., P.E.
American Institute of Steel Construction
Chicago, IL
from May 2000:

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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.
Arcadis Giffels
Southfield, MI

The allowable stress on the stem of a tee in flexural compression is limited by the AISC specifications which govern slender elements in compression. According to the 1989 *ASD Specification*, the allowable compressive stress (F_b) on slender elements must not exceed $0.6F_yQ_s$ nor the two applicable values from three equations (Eq. F1-6, F1-7, and F1-8) as specified in the 1989 *ASD Specification*.

Q_s is governed by the ratio of d/t where d is the full depth of the tee and t is the stem thickness. However, when the flange is in flexural compression, the provisions for slender elements consisting of flanges of beams governs.

Sam Babatunde, P.E.
Engineering Dynamics and Associates
Edgewood, PA

via email:

What are the design and application considerations when substituting ASTM A529 grade 50 for ASTM A572 grade 50 or ASTM A709 grade 50? What is the main difference in these grades of material as it relates to their application in construction?

Keith Woods
Pelham, AL

As far as strength (F_y) is concerned, there is no difference between the materials. Despite their similar strength characteristics, the specifications were written for distinctly different purposes:

- ASTM A529 grade 50 is only available in limited shape profiles and most commonly used in the metal buildings industry.
- ASTM A572 grade 50 (now essentially replaced by ASTM A992 for wide-flange shapes) is available in most rolled shapes and is intended for (as was developed for) building applications.
- ASTM A709 grade 50 was developed and intended for bridge applications. With supplementary requirements, it's essentially equivalent to AASHTO's M270 specification.

Keith A. Grubb, P.E., S.E.
Chicago, IL

comment on May 2000 issue:

My comments are related to the question and answer on the subject of SAE bolts. The questioner stated that the replacement bolts were to be used in a connection attaching a crane girder to a column cap plate and that the original bolts were "missing." The connection was subject to repetitive loading cycles—a fatigue condition.

This brings to mind an example emphasized in college. Connections sometimes undergo prying action that was not considered by the original designer. Under the prying load, the bolts can reach stress levels beyond the elastic limit and be plastically deformed or "stretched." When the bolts elongate, the joint reaches an equilibrium condition that can be nearer the case considered by the designer. If this is true, the connection can perform adequately if the bolt deformation is not excessive. However, sometimes the "stretched" bolts are unstressed at certain stages of the load cycle, most likely when the connection is not loaded. In this condition, it is possible for the nuts to vibrate off and the bolts fall out. Another possibility is that well meaning maintenance personnel will notice the "loose" bolts and retighten them. If this cycle (stretching and retightening) happens repeatedly, the bolts may eventually break and fall out.

The recommendation implied by Charles Carter is to replace the SAE bolts with the ASTM A325 or A490 bolts specified in the original design. I agree but would go farther to recommend some mechanical measure (not tack welds to bolt or nut) to prevent the nuts from turning relative to the bolt shanks after installation. It also would be prudent to reexamine the connection design using current day design/analysis approaches.

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