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! Send your questions or comments to solutions@aisc.org.

Turn-of-Nut Method

One of the recommended methods for installing bolts is the turn-of-nut method. The RSCS specification indicates that turn-of-bolt can be used if it is impractical to use the turn-of-nut method. How is the turn-of-bolt method different than the turn-of-nut method?

This seems to be more a question of terminology rather than requirements. Whether the nut is turned or the head is turned, the method is the same. That is, the same requirements apply.

Amanuel Gebremeskel, P.E.

Web Slenderness Ratio

Please confirm that h/t_w ratio in footnote [a] of Table B4.1 (page 16.1-18 in the 13th edition manual) is the height of the web (i.e., clear distance between flanges) over the web thickness (see Case 2). We just want to verify that this ratio should be used, and not the b/t ratio of the flange.

Yes, K_c is present to account for web slenderness h/t_w and h is the clear distance between the flanges. Note that per Section E7.1, K_c is between 0.35 and 0.76. See the Commentary to Section E7.1 for discussion.

Kurt Gustafson, S.E., P.E.

Comparison of Historic Shapes to Current Shapes

Does AISC provide any written material that compares the properties of old designated beams sizes, such as 16B26, to the current properties of a standard wide-flange (W shape)?

AISC's Design Guide 15 provides the section properties of historic shapes, as well as a summary of historic AISC specifications and applicable standards. AISC has also developed a Shapes Database v13.0 and 13.0H, where the H stands for historic. Both of these resources are available from the AISC bookstore at **www.aisc.org/bookstore** (and are free to members at **www.aisc.org/epubs**). With this data, you should be able to make comparisons between historic and current shapes of your choosing.

Kurt Gustafson, S.E., P.E.

Two-Story X-Bracing

I have a question on AISC 341 Section 14.3 regarding the unbalanced earthquake load acting on beam due to a buckled brace. We have a two-story OCBF where the V and inverted V braces meet at the second floor beams, forming a two-story X. The braces above and below the beam tend to balance each other (opposing forces). We are not clear if this requirement applies to this configuration. Does it apply only for Chevron-type configurations where the braces are located only below (inverted V configuration) or only above (V configuration)?

The intent of this section is to cover V or inverted V bracing, not the combination of these as a two-story X-braced configuration. In the case of two-story X braced systems, there is no unbalanced load on the intermediate floor beam.

Amanuel Gebremeskel, P.E.

Restrained or Unrestrained?

I am looking into the difference between "restrained" and "unrestrained" ratings for a steel-framed building. I have read the AISC engineering FAQs at www.aisc.org/faq, as well as the ASTM E119 and other literature, and all seem to point me in the direction that a standard steel building is classified as restrained.

That said, we have had some discussion in our office that the restrained classification depends on the continuity of the structure. While I understand that continuous beams spanning over more than two supports will offer more rotational restraint that a simple shear connection, are the simple shear connections enough? Could a single-bay structure (with simple shear connections) be considered restrained? Are there any special restraint requirements for perimeter beams? If one part of the structure becomes unrestrained, does that mean that the entire structure must be classified as unrestrained?

The important type of restraint is rotational restraint, not axial or in-plane expansion. Only a moderate amount of rotational restraint is needed for an assembly to perform as restrained. Beams framed with typical shear connections provide enough restraint. Other factors influence the degree of rotational restraint in large steel-framed floor assemblies. If continuity and/or composite action are part of the floor system, fire tests have shown that the concrete slab plays a significant role in providing rotational restraint and improves fire resistance.

The ability of standard shear connections to provide sufficient rotational restraint was tested by UL and independently under AISI sponsorship.

These findings are discussed in a paper "Restrained Fire Resistance Rating for Buildings" by Gewain and Troup, published in the AISC *Engineering Journal*, Second Quarter 2001 (available free to AISC members at *www.aisc.org/epubs*.)

The conclusion is that steel-framed buildings should be considered thermally restrained.

John L. Ruddy, P.E.

Shape Group Numbers

In preparation for an ICC bolting examination, one of our technicians found a question asking what structural group number a W14×426 member was. We found the answer in the AISC 9th edition ASC manual, but not in the 13th edition *Steel Construction Manual*. The ICC test now references the newer manual, and I was wondering if this table is included in it, or if it was left out intentionally?

It was left out intentionally. ASTM dropped the group number classification system several years ago. For further information on this subject, there was a Steel Interchange article in the July 2006 edition of MSC. You can browse previous Steel Interchange questions by visiting www.modernsteel.com and clicking on the Steel Interchange link on the right.

Kurt Gustafson, S.E., P.E.

Delamination

We have encountered a situation where it appears that the carbon steel framing is delaminating, for lack of a better word. I have searched the Internet, but have not found many articles or data concerning this specific issue. Do you know of the correct terminology to use when the steel is separating in layers?

Your description of laminations may be indicative of what is referred to as "rust pack." In such a case, you will see what appears to be an expansion of the material thickness, with a bulging of the plies in contact. The material thickness can be several times the thickness of the original material.

I have seen this phenomenon quite often in old masonry-clad buildings, where steel lintels were used over exterior doors and windows—in conditions where moisture could accumulate, resulting in severe rust pack after many years of exposure to the elements. When such a condition is encountered, an evaluation should be made to ascertain if the load-carrying capacity of the lintel has been severely compromised. Removal of the loose rust pack, measurement of the remaining sound material, and calculation of the resulting section properties is a common procedure to perform such an evaluation.

Kurt Gustafson, S.E., P.E.

Seismic Design for Horizontal Bracing Members

Do horizontal bracing members need to meet the requirements of Sections 13.2a and 13.2d of the 2005 AISC seismic provisions? It is not clear whether "bracing" applies to horizontal and vertical, or just vertical bracing.

Section 13 of the 2005 AISC seismic provisions is meant to address the requirements for vertical bracing. In practice, horizontal bracing is typically dealt with similar to how one would deal with diaphragms.

Amanuel Gebremeskel, P.E.

Specifying Reactions

What is the best way to specify simple shear connection reactions on the design drawings? On a non-composite steel framing system, we typically specify that "All beam connections shall develop the full uniform load capacity the member can carry..." The connection designer can then easily obtain a design load using the AISC allowable load tables. This method also ensures that the connection will not be the limiting design element. We have found this procedure to be an efficient method to specify design loads for typical framing.

We would like to specify composite beam connection design loads in a similar manner. Is there a design aid available that would allow a connection designer to easily obtain the maximum uniform load for a composite beam?

Do you have any other recommendations to efficiently provide simple shear connection design loads for composite framing?

Your stated method of specifying reactions for non-composite beams is one method used, but not necessarily the most accurate. Also, it may not always ensure that the connection will not be the limiting design element. It may generally be fairly accurate—if the beams are loaded uniformly, and if the beams are rather close to the design or allowable strength for flexure, and if the limit state of flexure controls the design of the beam. These are a lot of "ifs," and it is generally more desirable and accurate to define the actual design end reactions for the beam on the contract documents.

In the design of composite connections, the process gets more complicated, because one also needs to consider the level of composite action and resulting plastic neutral axis (PNA) location for the design of the particular beam, in order to define a flexural moment capacity for the shape and slab/deck configuration. This flexural capacity would then be correlated back to the superimposed load arrangement and span length of the beam. One may be able to make a lot of conservative assumptions in order to come up with a coefficient that may work in most cases; but again, it may be more prudent to show the actual required design reactions on the contract documents.

Why not just put the actual reactions on the drawings?

Kurt Gustafson, S.E., P.E.

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Steel Interchange is a forum 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.

If you have a question or problem that your fellow readers might help you 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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