If you've ever asked yourself "Why?" about something related to structural steel design or construction, *Modern Steel*'s monthly Steel Interchange is for you! Send your questions or comments to **solutions@aisc.org**.

steel interchange

Unless specifically stated, all AISC publications mentioned in the questions and/or answers reference the current edition and can be found at ***www.aisc.org/specifications****.*

Weld Inspection Acceptance Criteria

AWS D1.1 provides acceptance criteria for both statically and cyclically loaded connections. Since the criteria for cyclically loaded connections are more stringent, should they always be used unless stated otherwise? Is the inspector responsible for deciding between statically and cyclically loaded acceptance criteria?

The acceptance criteria for cyclically loaded connections should not be assumed to be the default requirement. AWS D1.1 indicates that cyclic requirements apply when the joints are "subjected to cyclic loads of sufficient magnitude and frequency to cause the potential for fatigue failure." Section 3.1 of the AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360) states: "The fatigue resistance of members consisting of shapes or plate shall be determined when the number of cycles of application of live load exceeds 20,000. No evaluation of fatigue resistance of members consisting of HSS in building-type structures subject to code-mandated wind loads is required." This means that generally welded connections in structures within the scope of the *Specification* will be subject to acceptance criteria for statically loaded connections.

The engineer of record (the owner's designated representatives for design) is responsible for defining the acceptance criteria. If the requirements are not clear, clarification should be requested. Clause 6.7 of AWS D1.1 states: "The extent of examination and the acceptance criteria shall be specified in the contract documents on information furnished to the bidder." Section 8.5.6 of the AISC *Code of Standard Practice for Steel Buildings and Bridges* (ANSI/AISC 303) states: "The inspector shall not suggest, direct or approve the fabricator or erector to deviate from the contract documents or the approved approval documents, or approve such deviation, without the written approval of the owner's designated representatives for design and construction." Both statements indicate that the requirements must be provided in the contract documents.

Larry S. Muir, PE

Connection to Supports

Section 1.4.4 of ASCE-7 states: "A positive connection for resisting a horizontal force acting parallel to the member shall be provided for each beam, girder, or truss… The connection shall have the strength to resist a force of 5% of the unfactored dead load plus live load reaction imposed by the supported member on the supporting member." I am being asked to check every connection on a current project for this axial end reaction. Is this common?

No. Note that there is a requirement that the connections must be able to resist this force. There is no requirement to provide an explicit check. Engineers commonly judge some conditions as okay by inspection based on their own engineering judgment.

The required strength (in the horizontal direction) is only 5% of the unfactored vertical loads. This is quite small. Relative to many connection-related limit states, the load described by Section 1.4.4 would be 2.5% of the vertical design load. In practice, most engineers simply conclude that typical steel connections can resist this load. I think it would be difficult to find a reasonable connection that does not satisfy this requirement.

Since these are ASCE, not AISC, requirements, you may also want to contact ASCE relative to their intent.

Larry S. Muir, PE

Galvanized Architecturally Exposed Structural Steel (AESS)

The November 2017 *Modern Steel Construction* article "Maximum Exposure" addresses changes that occurred in Section 10 of the 2016 AISC *Code* and provides other useful advice. A caption to one of the photos states: "AESS can also be galvanized. Design teams should be aware that galvanizing steel does not provide a 'chrome' finish, and no two pieces of galvanized steel will look exactly the same." An editor's note in Section 2.9 of the Sample Specification further cautions about expectations for AESS finish when hot-dip galvanizing is specified, and also explains the possible causes of such finish irregularity.

If the level of dullness/shininess is of concern, is sample/mock-up the only way to establish the acceptable level of dull or bright finish, in lieu of any other descriptive verbiage in the project specification?

A mock-up may be a means of establishing acceptable and expected finish for the galvanizing. Section 10.1.2 of the *Code* requires a mock-up for AESS categories 3, 4 and C. If a mock-up is to be used in other AESS categories, it must be specified in the contract documents.

Regardless of whether a mock-up is used, you should work with the galvanizer and fabricator to come up with specification language that will result in an end product that meets your expectations. The chemistry of the steel influences the appearance of the galvanized coating. It may be necessary to impose tighter controls on chemistry, which could impact the cost and schedule of the project. Also keep in mind that the mock-up will reflect only the appearance of the coating at a particular time. As indicated in the article (www.modernsteel.com) the appear-

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ance may change to a more uniform matte gray finish over time even if the mock-up appears bright or blotchy.

You might be able to refer to existing structures to get a better feel for the final appearance. The galvanizer may also be able to provide guidance relative to steps that were taken in an individual project to achieve a certain appearance.

You might want to reach out to the American Galvanizers Association (www.galvanizeit.org). Its site provides a good bit of information about the appearance of galvanized steel. This information is useful but should not be viewed as a substitute for direct interaction with those performing the work.

Carlo Lini, PE

Pretension in Snug-Tight Connections

I have a few questions about pretension in snug-tight connections:

1. **Do the RCSC or AISC specifications recommend that snug-tight connections not be pretensioned?**
2. **Has this recommendation changed over time?**
3. **What is the level of pretension that is expected in snug-tight connections?**

Your questions are addressed below:

1. No. There is no such recommendation. Neither the RCSC *Specification* nor the AISC *Specification* place an upper limit on installed pretension for snug-tight joints. Snug-tightened joints are defined as having all plies in firm contact and bolts tightened with a few impacts of an impact wrench or the full effort of an ironworker. To reach this condition, the bolts could be installed to the pretensions indicated in Table J3.1 of the AISC *Specification*—or even higher—in order to satisfy the requirements for a snug-tight joint.

2. No. To my knowledge, there has never been a recommended upper limit on the installed pretension for snug-tight joints.

3. As stated above, there is no upper limit on the installed pretension. AISC Design Guide 16: *Flush and Extended Multiple-Row Moment End-Plate Connections* (a free download for members at www.aisc.org/dg) does provide some guidance and states:

"The study by Kline, et al. (1989) observed that the pretension force measured in the snug-tightened bolts is directly proportional to the bolt diameter (d_b). Based on this data, a recommendation for the assumed pretension force in snug-tightened bolts to be used in the design procedure is:

$d_b \le 5/8$ in., use 75% of specified AISC full pretension
$d_b = 3/4$ in., use 50% of specified AISC full pretension
$d_b = 7/8$ in., use 37.5% of specified AISC full pretension
$d_b \ge 1$ in., use 25% of specified AISC full pretension"

This is just a guide to what pretension can be expected. It is not something to be measured when bolts are installed.

Carlo Lini, PE

Small Section Sizes

The smallest angle found in the AISC *Steel Construction Manual* is L2×2. Why doesn't the *Manual* include sizes such as L1½×1½ or L1¼×1¼?

The AISC Committee on Manuals determines what information to include in the *Manual* based on a number of factors, including the relevance to steel building structures. L1½×1½ and L1¼×1¼ are likely too small to be considered for use in building structures. If you need properties or strengths for these shapes, you may conservatively calculate the properties by hand and determine the strength manually using the provisions of the *Specification*.

Jonathan Tavarez

STEEL SOLUTIONS CENTER

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