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. If you have a question or problem that your fellow readers might help you to solve, please forward it to *Modern Steel Construction*. At the same time, feel free to respond to any of the questions that you have read here. Please send them to:

Steel Interchange
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Chicago, IL 60601-2001

Answers and/or questions should be typewritten and double-spaced. Submittals that have been prepared by word-processing are appreciated on computer diskette (either as a Word file or in ASCII format).

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Information on ordering AISC publications mentioned in this article can be obtained by calling AISC at 800/644-2400.

**** **Questions and answers can now be e-mailed to: rokach@aiscmail.com** ****

The following responses from previous Steel Interchange columns have been received:

(From December 1998)

When stiffening extended end-plate moment connections, if bolts are located on the usual gauge line of the column flange then stiffeners are often required due to column flange bending opposite the tension flange of the beam. Is it a legitimate practice to locate the bolts on a narrower gauge line to avoid needing stiffeners? If this is done, can the full effective width of the end-plate still be used for the end-plate thickness calculation?

James M. Gleason, P.E.
George Koch Sons, Inc.
Evansville, IN

Refer to the "AISC Steel Design Guide 4: End-Plate Moment Connections", by Thomas M. Murray, PhD, P.E. In Chapter 2, "Recommended Design Procedures", a list of assumptions or conditions is given (2.1 Basis of Design Recommendations) that is inherent to the design procedures. Number 7, which states "the gage of the tension bolts (horizontal distance between vertical bolt lines) should not exceed the bea, tension flange width, again based on engineering judgement."

One page 11 of this design guide, for Example 3.3 (using the LRFD design procedure) it is determined that column stiffeners are required opposite the beam tension flange (see design stage D, third check) because of $t_{fr}>t_{fc}$. For this example, if the designer does not desire column stiffeners, what bolt gage could be used instead of 5.5"?

Letting the values remain the same, aspect p_e (which becomes g/2-.875/4-7/8) and letting $t_{fr}=t_{fc,}$ solving for g gives 3.69". Using 3.5" as g, letting the values remain the same under design stage B (end-plate design) and recalculating the end-plate thickness (t_p) gives .5781", try .625".

Checking the bolt bearing on the end-plate: $V_u/2=52/2=26$ kips < $\phi(2.4 \times F_u) \times d_b \times t_p= .75(2.4 \times 58) \times 8 \times .625 = 57.1$ kips

Checking end-plate shear:
$F_{fu}= 135.3/2= 67.6$ kips < $\phi(.6 \times F_y) \times b_p \times t_p= .8(.6 \times 36) \times 8 \times .625 = 86.4$ kips

For design stage D, the column web yielding (i) is recalculated and is found to be ok (check for ii and iv need not be recalculated).

Because of the possibility of interference with weak axis framing and their expense column, web stiffeners are not recommended. Murray gives three possible solutions, which are (1) use an 8-bolt stiffened end-plate, (2) increase the column flange thickness by using a heavier column, and (3) increase the bolt pitch which increases the effective column flange length and decreases the required column flange thickness (this may require a thicker end-plate).

In closing, the 1999 North American Steel Construction Conference (NASCC) in Toronto will host a session on May 20 at 8:45 a.m. and May 21 at 10:45a.m. entitled "Column Stiffening at Moment Connections". This would be an excellent reason to get out of the office. (For more information on the conference call Scott Melnick at (312) 670-5407)

Timothy M. Young
Structural Innovations Plus
Cumberland, VA

The following answer is in response to the above answer.

Dr. Tom Murray in "AISC Design Guide 4: Extended End Plate Moment Connections" clearly gives the limitations that apply to this design method. The limitations restrict the bolt gage to between 5½" to 7½" . The pitch is also limited to a maximum of 2½" to the first bolt and 3d between rows. It is my understanding that these were the limits that the research was done on. While reducing the gage might be okay for lighter sections there could be a different load distribution due to the change in relative stiffness.

Use the minimum gage permitted and the maximum pitch for the most efficient design and the best solution when checking for column stiffeners.

An alternative to using stiffeners would be to increase the column size or perhaps increase the beam depth the reduce the flange force. Either of these might be better then using stiffeners.

Lawrence A. Kloiber
Le Jeune Steel Company
Minneapolis

What are the commonly used methods of non-destructive examination?

The most commonly used NDE methods in structural steel fabrication is visual (VT). Other examination methods are also used: dye penetrant (DT), magnetic particle (MT), radiographic (RT), and ultrasonic (UT). The method to be used is established after consideration of the importance of the weld as well as the defect identification capability and relative cost of each method. When NDE is required, the process, extent, techniques and standards of acceptance must be clearly defined in the contract documents.

What NDE inspection beyond visual should be specified? What acceptance criteria should apply?

The SER should identify members and connections that must be inspected and specify how they should be inspected. Inspection requirements can be specified, if desired, by the SER as some percentage, with subsequent testing requirements identified if a significant defect rate is discovered. For example, 15 percent initial inspection might be deemed acceptable for an AISC Quality Certified fabricator, with no further testing required if all inspected joints are found to be compliant; if a significant defect rate were found, the inspection of an additional 15 percent might be required.

NEW QUESTIONS

The following questions were taken from the Steel Forum on the Modern Steel Construction web site (www.modernsteel.com).

LRFD Manual (Vol II) on page 9-194 states that "If provision is made for ductility and stability, it follows from the lower bound theorem of limit states analysis that the distribution which yields the greatest strength is closest to the true strength." Can someone help to explain just how one "provides for ductility and stability" with an extended shear tab?
Scott Dunham

I have received our client specifications and they indicate that they require all bolt holes that are in galvanized steel to be 3/32" over sized and not the standard 1/16". Our fabricator has indicated that this is a very costly request when the codes indicated that only 1/16" is required, and the 3/32" would require re-tooling the fabrication line which in turn delays the fabrication. Is there any economic justification on the constructor's side that the holes be oversized 3/32" for mechanically galvanized bolts? Hot dipped galvanized bolts? In addition to the higher cost of fabrication all engineering standard detail drawings would have to be updated to indicated this over sized hole.
Keith Webb

"Mill to bear" is a term often used in contract drawings and specifications. What precisely is the definition of "mill to bear", especially as it relates to AWS D1.1 and the 16th Edition of AASHTO Standard Specification for Highway Bridges?
While our drawings do not call the parts "stiffeners", the closest we can come is paragraph 5.23.10 of D1.1(1996). Because our contract drawings do not reference AISC, Paragraph M4.4 of the 2nd Edition of LRFD Spec is not being recognized by our customer.
Any assistance or insight would be most appreciated.
Jim Tyvand

What is the process and/or requirements for welding to A7 and/or A9 steel. Our office is working with a number of existing buildings of this era and would appreciate an answer, idea or reference.
Matthew Johnson

I am analyzing a structure which was fabricated in 1924. Am I correct in assuming that the material is ASTM A7? Where can I find the section properties for this material? In particular, I need the design properties for ship building channels, standard channels and angles from this era. Also, what is a good reference to use for determining the strength of riveted connections?
Gary A. Clark, P.E.

What is the status/prognosis of the OSHA requirement for a minimum of four bolts in column base plates?
Norman Golinkin