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
Modern Steel Construction
One East Wacker Dr., Suite 3100
Chicago, IL 60601-2001

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

Is it acceptable to either mechanically galvanize or hot-dip galvanize high-strength bolts? Are there different requirements for the installation depending on how the bolt is galvanized?

Though no problems are reported for A36 material, hot-dipped galvanizing has caused embrittlement problems in high-strength steel, which is particularly important in fatigue bolt loadings. Embrittlement may be caused by:
1. Crack initiation sites generated in the acid cleaning done before galvanizing.
2. Hydrogen embrittlement generated during acid cleaning and the hot dip process.
3. Zinc migration along grain boundaries.
4. The temperature of the hot-dip bath which can affect the temper of the material.

Mechanical galvanizing and hot-dipped galvanizing leave threads that require an oversize nut, the mechanical process leaves clean threads where hot-dipped process will leave flakes in the threads that must be cleaned.

The mechanical process is limited to smaller weights (2kg) and lengths (0.5m) than hot dipped, so long rock bolts are not candidates for the mechanical process. Some suppliers of high-strength galvanized rock bolts have specifications for galvanized high-strength rock bolts that request the acid bath not be used for cleaning prior to hot-dip galvanizing, eliminating item 1 above. They propose instead that sandblasting be used to clean the steel.

Some good references are:
ASM Handbook, Volume 5, Surface Engineering
ASTM B-695, Standard Specification for Coating of

Zinc Mechanically Deposited on Iron and Steel.
Mike Carney, P.E.
R.W. Beck
Seattle, WA

For fire-wall construction, building codes say the wall shall have sufficient stability under fire conditions to allow for collapse of construction on either side without collapse of the wall. In a tied fire-wall application, a flexible anchor or break-away connection is recommended to laterally stabilize the wall and under fire conditions to let go and not to pull the pull down the wall due to the collapse of the structure on the fire side. What is the optimum detail (effective and economical for this type of connection?)

The illustration shown with this question is a free-standing fire wall, not a tied fire-wall. Tied fire-walls seem to be almost impossible to build, without excessive cost due to added
strength, structure, and fire protection required to withstand the force of collapse on one side of the wall.

The following are references on this subject, none of which seem enthusiastic or definitive about tied walls:


"Fire Walls in Modern Industrial Buildings", Factory Mutual Insurance Company.

It appears that free-standing fire-walls are more easily built, and can be made to perform more reliably in a fire/collapse situation.

R.W. Liebing
ITT Technical Institute
Dayton, OH

Are there special requirements for the design of high-strength A325 or A490 bolts that are going to be in a high-temperature area?

I do a lot of work with ASME pressure vessels. That code has tables of allowable strengths/temperature for most common construction alloys. A325 has no reduction at least up to 650 degrees F. If temperature is going to be a problem, I would also suggest checking the allowable for the structural steel.

E. Burrell Fisher, P.E.
Little Falls, NY

New Questions

Listed below are questions that we would like the readers to answer or discuss.

If you have an answer or suggestion please send it to the Steel Interchange Editor, Modern Steel Construction, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001.

Questions and responses will be printed in future editions of Steel Interchange. Also, if you have a question or problem that readers might help solve, send these to the Steel Interchange Editor.

In a wide-flange beam-to-girder connection with the beam on one side only, can a full depth connecting plate be adequate for load transfer or is a stiffener plate necessary on the opposite side in line with the beam web?

If the web buckling and yielding are checked and found to be OK, for a wide flange beam bearing on a column, is it still necessary to have stiffener plates in the beam in line with the column flanges? If so what are the design considerations?

Dipankar Sengupta
Sato and Associates
Honolulu, HI

If the bolts at the shear plate column connection for the simply supported beam shown in the accompanying sketch is designed to carry both shear and moment, can the connection at point A be assumed to transfer only shear to the column?

Jarvis W. Kuhlmann, P.E.
Houston, TX

When working on an old steel structure what precautions should be used before attempting to weld to this material?