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:

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A special thanks once again to Charles J. Carter, AISC's Director of Engineering and Continuing Education, for this month's *Steel Interchange* column.

Heat Straightening vs. Mechanical Realignment

Are there any guidelines as to when one might use heat straightening? I gather that there are relatively few people who are adequately experienced at heat straightening so I assume that this is a very expensive proposition and, therefore, not appropriate for the typical damaged column in an industrial structure.

What degree of straightening might be achieved by each method? Would it be possible to reasonably determine in advance when additional reinforcing might be required?

In a complex application, heat straightening is an art and there are a few experts who can serve as consultants (Richard Avent, Dan Holt, Charles Roeder and Jeff Post come to mind). An expert will likely be needed on a very complex job, unless the fabricator's personnel have experience.

I know of one case where a long-span bridge girder got mangled during the rollover, separation, fall and plunge into the river below where it was being installed. (Other than being twisted, distorted and generally all banged up, there was nothing wrong with the girder.) The schedule, needs and economics of the job were such that heat-straightening the distorted girder (after they fished it out of the river) was much more feasible than fabricating a new girder. When the straightening process was complete, you would have sworn it was a new girder.

However, for a job that's as simple as taking a dimple out of a column flange, the various papers that have been written by those experts can be used quite successfully by capable fabrication personnel. A few of the papers that are available are as follows:

"Engineered Heat Straightening," R. Richard Avent, Proceedings of the 1995 AISC National Steel Construction Conference, AISC, Chicago, IL.

"Flame Straightening Technology," Daniel J. Holt, Proceedings of the 1995 AISC National Steel Construction Conference, AISC, Chicago, IL.

"Designing Heat Straightening Repairs," R. Richard Avent, Proceedings of the 1992 AISC National Steel Construction Conference, AISC, Chicago, IL.

Regarding the potential need for reinforcement, I wonder why any reinforcement would be required if the flange has been realigned as successfully as it can be. You probably would only have to consider adding reinforcement if it could be use in place of (and were cheaper than) heat straightening; or if the straightened flange still had waviness outside of ASTM A6 tolerances after straightening, and calculations could not be used to show that the out-of-straightness would be acceptable.

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Bent Clip Plates for Connections

A contractor has requested that I take a look at some bent clip plates (used to transmit beam gravity loads) due to the presence of cracks at the outside corner (they radiate from the neutral axis to the outside corner) of the bend. The cracks (two) also occur at the top and bottom edges of the bent corner of the plate. They are almost "surface" cracks in nature in that they are approximately 0.03" wide and ⅛" deep.

The plate is 5⁄16" thick and 5½" wide. The one outstanding leg is oriented approximately 120 degrees relative to the other. I've requested what bending radius was used and if the plate was bent parallel or perpendicular to its "roll."

Are these type of cracks typical for bent plates?

There are recommended minimum bend radii for bent plates in the AISC *Manual* that are intended to eliminate cracking problems that can occur when plates are bent to too tight a radius. See page 9-129 in

LRFD 2nd ed. or page 4-174 in ASD 9th ed. The table is the same in either book.

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Load Transfer in Bolted Connections

How do bearing connections and slip-critical joints differ in terms of load transfer mechanisms?

The following is based upon the explanation of bearing (snug-tightened or pretensioned) and slip-critical connections in the Commentary to the RCSC Specification for Structural Joints Using ASTM A325 or A490 Bolts. The only real difference between a bearing connection and a slip-critical connection is that we can define at what load slip is likely to occur for a slip critical connection because the faying surfaces are bare steel or painted with a paint that has been shown by testing to provide the required frictional resistance. From RCSC Specification:

If non-pretensioned bolts are used in a shear connection, load is transferred by shear in the bolts and bearing stress in the connected material. At the ultimate limit state, failure will occur by shear failure of the bolts, by bearing failure of the connected material or by failure of the member itself. On the other hand, if pretensioned bolts are used in such a joint, the frictional force that develops between the connected plies will initially transfer the load. Until the frictional force is exceeded, there is no shear in the bolts and no bearing stress in the connected components. When the frictional force is exceeded, slip takes place and the slip limit state is attained. Further increase of load places the bolts into shear and the connected material has bearing stresses present, just as was the case when non-pretensioned bolts were used. Since it is known that the pretension in bolts will have been dissipated by the time bolt shear failure takes place (Kulak et al., 1987; p. 49), the ultimate limit state of a pretensioned bolted joint is the same as an otherwise identical joint that uses non-pretensioned bolts.

Because the consequences of slip into bearing vary from application to application, the determination of whether a joint can be designated as snug-tightened, as pretensioned, or designated as slip-critical is best left to the judgment of the Engineer of Record.

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Use of "Jam" Nuts

I have not seen much technical information on the use and functionality of jam nuts on A307 grade A bolts. Jam nut applications do frequently appear, especially where there is vibration. I have noticed on many occasions seeing in detail drawings a jam nut specified on top of the normal hex nut. I take these to be in error. The jam nut should be tightened on the bolt first, followed by the normal hex nut. Can you clarify the proper use of jam nuts and their functionality?

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A jam nut is just like a regular nut, but only half as thick. It is there to "jam" against the other structural nut to prevent the nut from turning (in cases where you cannot pretension the bolt, such as A307s). They are not mentioned in the RCSC Bolt Specification because they are non-structural. The positioning of the nut (under or on the hex nut) isn't significant.

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Technical Note:

ASTM A992 now costs less than A36 for W-shapes

Nucor-Yamato Steel Company and TXI-Chaparral Steel, the two largest sources of W-shapes in the United States, have instituted a $10-per-ton surcharge on ASTM A36 W-shapes. But don't despair! The cost of ASTM A992 W-shapes has not increased.

AISC recommends that ASTM A992 (Fy = 50 ksi, Fu = 65 ksi) be specified for W-shapes instead of either ASTM A572 Grade 50 or A36. In the past, this recommendation was based upon the rationale that ASTM A992 offers better material definition, including: an upper limit on yield strength of 65 ksi, a minimum tensile strength of 65 ksi, a specified maximum yield-to-tensile ratio of 0.85 and a specified maximum carbon equivalent of 0.47%. Now, Nucor-Yamato and TXI-Chaparral have added a financial incentive to specify ASTM A992 for W-shapes.

You can find more information on ASTM A992 in the article "Are You Properly Specifying Materials" (Part 1) in the January 1999 issue of AISC's *Modern Steel Construction* magazine. This article can be downloaded by visiting the back issues feature at **www.modernsteel.com**.