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 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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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 wordperfect file or in ASCII format).

The opinions expressed in *Steel Interchange* do not necessarily represent an official position of the American Institute of Steel Construction, Inc. 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.

Information on ordering AISC publications mentioned in this article can be obtained by calling AISC at 312/670-2400 ext. 433.

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

Are there concerns about bending of the tube wall in shear tab type connections? When should the shear plate be carried through the tube section?

Two potential concerns pertaining to shear tabs used with tube columns are:

1. The strength of the tube wall in a yield line mechanism failure mode in the connection.
2. The effect that local distortion may have on the column strength.

Recent experimental studies have shown that due to the self limiting nature of the end slope of a simply supported beam, neither of these concerns justify the use of through-plates.

The results of a connection study were presented at the 1991 National Steel Construction Conference. (Sherman, D. R. and J. M. Ales, *The Design of Shear Tabs with Tubular Columns*, Proceedings 1991 National Steel Construction Conference, AISC) Thirteen tests were reported that included a range of b/t from 5 to 45 with fully tensioned and snug-tight bolts in the beam web connections. The failure mode was in the tube wall in only two of the tests where the wall was thin enough to produce a punching shear failure. The design guidelines presented in the paper include a criteria to prevent this failure mode. Excessive distortion of the tube wall was never a critical factor.

The design guideline included in the paper recommends that shear tabs be limited to b/t limits of 16. This was due to the limitations of previous tests on the column strength that did not include tubes with higher b/t. In the previous program, four T6x3x5⁄16" columns were tested with shear tabs, through-plates, fully tensioned bolts and snug-tight bolts. Beams framed into the tube on both 6" walls at the midheight of a 20′ column. The ultimate loads were within 10 percent with the bolt tightness being a greater factor than whether the connecting element was a tab or a through-plate.

Within the last few months a similar column test program was conducted with T8x3x¼" and T8x3x3⁄16" columns using snug-tight bolts in all connections. For the ¼" tubes the difference in the strength of the columns with tabs and through-plates was 2½ percent. The difference was 20 percent for the 3⁄16" tube columns. All failures were local or general column buckling in the lower half of the column. There was no noticeable local failure at the connection. This study also included columns with the beam connected to one side only. In these cases the failure was by excessive bending of the column and there was no clear distinction between the tabs and through-plate connecting elements.

Although the detailed data from the most recent tests are still being evaluated, it appears that through-plates are not required for tubular columns that do not exceed the b/t limit of $253/F_y$ defining a thin walled section. This conclusion is based on tests where the end rotation of the beam does not exceed that of a uniformily loaded simply supported beam.

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What procedures should be followed when assessing steel that has been exposed to a fire?

The following is taken from "Technical Committee No. 8: Fire and Blast, Discussion No. 4, *Repair of Steel Structures after Fire*" presented at the International Conference on Planning and Design of Tall Buildings:

The post-fire repair of a steel-framed structure is a situation that many designers have not been faced with. The following brief discussion of the subject provides some general recommendations, as well as an appraisal of the conditions under which structural

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damage can be expected.

Fires are unique; their effect on a building and the extent of required repairs is a special situation that has to be considered and handled for each particular circumstance. The following checklist outlines several, but not necessarily all, of the parameters that should be investigated by any designer.

- 1. An appraisal should be made of those members that have been subjected to potential damage. For convenience, this appraisal should be conducted on members grouped as to their importance:
  - a. Columns.
  - b. Primary Horizontal Members, such as girders and trusses.
  - c. Secondary Floor Members, such as beams, fillers and floor deck.
- 2. After identifying those members of potential damage, each structural member in a fire damaged area should be evaluated for individual damage. This evaluation should also include connections.
- 3. On the basis of the damage evaluation, an economic evaluation of repair or replacement of the structure should be considered.
- 4. If it is decided to repair the structure, damaged members should be divided into three categories:
  - a. Members having nominal damage and adequate structural capacity for continued service without further repair.
  - b. Members having light damage and repairable in place.
  - c. Members with severe damage that should be replaced.
- 5. Throughout all of these steps, the designer must recognize that expediency will often dictate the approach. Fires usually mean a temporary loss of business and rental income; owners and occupants will insist on a very rapid restoration of building service and availability, a situation that may lead to costly, but quick, solutions.

Fortunately, steel is a material with a very high tolerance for fire. All of the processes of its manufacture, from smelting the ore to rolling the structural shape, are done at temperatures above those that are likely to occur in an accidental building fire.

At this point, the designer needs only some guidance on evaluating the degree of structural damage. Fortunately, in steel, the rule is very simple:

Any steel member which has been distorted by fire so that it has a permanent deflection, crippled web or flange area, or damaged end connections should be considered for either in-place repair or replacement.

In practice, it may be easier to apply the corollary:

Any structural steel member remaining in place, with negligible or minor distortions to the web, flanges or end connections shall usually be considered satisfactory for further service.

There are only two exceptions which should be considered by the designer. Quenched and tempered structural steels, of which relatively small tonnages have been used, may undergo a change in properties during the heating and cooling cycle of a fire. A second area of possible departure from the above rule pertains to high strength fasteners., Under certain conditions it is possible that their properties may be altered by prolonged fire exposure. But should there be any question, it is relatively easy to remove individual fasteners for test purposes and, should replacement be necessary, to replace those that are suspected of damage.

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New Questions

Listed below are some 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. 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.

1. What can an erector and engineer do when anchor bolts are too short and the nuts are not fully engaged?

2. Can one weld to an existing structure? How does one determine if the steel is weldable?

3. Are both mechanical galvanizing and hot-dip galvanizing appropriate for bolts?