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:

Steel Interchange
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Question from June 2000:

If fixed-base columns are used, should seismic overstrength factors be applied to the design of the column connection to the foundation? In other words, if fixed-base steel columns were used, would the seismic overstrength factor need to be applied to the base plate and anchor rod design? For columns at braced bays, should the seismic overstrength factor be used in the design of the anchor rods?

Paul H. Lind, P.E.
Butler Construction
Kansas City, MO

Per section 1633A.2.12, 1998 California Building Code, the connection of the superstructure elements to the foundation shall have the strength to resist the seismic loads considering either the strength of the superstructure elements or the overstrength factor. This requirement shall be considered regardless of the pinned or the fixed base connections. Even though UBC 1997 doesn't address this requirement, the connection of the column to the foundation shall be capable of transferring the seismic load used for the design of superstructure base connection. In other words, the base plate and anchor rods may be considered as a portion of the base connection.

Shiping Feng, S.E., P.E.
Sacramento, CA

Another response:

The seismic force amplification factor to account for structural overstrength. Then it was known as $3/8 \times R_w$. Today, in the UBC, IBC, and NEHRP Provisions, we know it as Ω_0. The AISC Seismic Provisions do not presently address the issue, leaving it up to the judgment of the design engineer.

Section 1809.3 of the 1997 UBC requires that for Seismic Zones 3 and 4, "The connection of superstructure elements to the foundation shall be adequate to transmit to the foundation the forces for which the elements were required to be designed." This appears to require the use of Ω_0 for the design of the column base connection to the foundation. Note that some engineers interpret the phrase "adequate to transmit" to mean that Ω_0 is not required if the anchor rod is designed to be governed by a ductile limit state.

Section 1604.8.1 of the 2000 IBC requires that "Anchorage of...columns to foundations shall be provided to resist the sliding and uplift forces that result from the application of the prescribed loads." Section 1605.2.3 of the 1997 UBC has similar requirements. This appears to require the use of Ω_0 for the design of the column base connection to the foundation. Regardless of applicable codes and their interpretation, it is conservative to apply the factor to the design of base plates and anchor rods.

Rick Drake, S.E.
Fluor Daniel, Inc.
Aliso Viejo, CA

Question from June 2000:

I am retrofitting some existing building trusses to handle additional loading. These trusses have continuous top and bottom chords and all welded connections. Even under existing loads, the only way that the truss members can be made to work is to assume that all members have pinned ends. Obviously, this eliminates all member bending moments from the analysis.

Clearly this is what the original designer assumed. But, given the as-built conditions, I don't see how I can legitimately make the same assumption. Are there any references that discuss this?

John Brock

Truss analysis, HSS cap plates & drainage holes

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The continuity in the top and bottom chord members does not change the concept of designing truss members for axial forces only. If applicable, secondary stresses due to eccentricities and local bending members from uniform loading between the panel points of the top chord should be considered. Also, unless one sliding support is considered in a simple span truss, the axial forces in the members will be transformed. To consider pin-ended truss members for retrofitting work would be a fair assumption. If a web member is to be cut, that panel should be analyzed as for a Vierendeel truss and reinforced accordingly, if necessary.

Isaac Gordon, P.E.
Ang Associates, Inc.
Philadelphia, PA

Another response:

A good reference regarding the analysis of a truss assuming pinned joints is:

Shedd, Thomas Clark and Jamison Vawter, "Theory of Simple Structures," 2nd ed., John Wiley & Son, New York, 1941, Article 78, Page 165.

In spite of its copyright date, the assumptions and their limitations are well defined. Any text describing the analysis of a truss using the method of joints, method of sections or graphical analysis should define this assumption as well. All of this assumes panel point loads only.

Trusses have been successfully designed assuming pinned joints for nearly a century. Regardless of fabrication method, some small moments develop at the joints. Because the truss is made of a ductile material, localized yielding will redistribute the loads (this is termed shakedown).

When using finite element analysis, the model should be consistent with the design assumptions—always! For a truss, either use the truss analysis option or release the joints. The resulting axial loads will be somewhat larger than with rigid joints.

Using the member loads determined above using pinned joints, connection eccentricities should be considered in the member design. There should be no eccentricities in the chords, only in the web members. For example, WT sections connected on their flanges and single angles must be designed considering eccentricity. For double angles connected on the gage line, the eccentricity is customarily neglected.

A reference for welded truss connections is:

Blodgett, Omer W., "Design of Welded Structures," James F. Lincoln Arc Welding Foundation, Cleveland, Ohio, June 1966, Section 5.9

Bob Leffler

What guidelines are available to determine the thickness and welding details for cap plates on HSS columns? Is the main purpose of the cap plate to keep water out, or to add structural integrity to the column? Or is there some other purpose? What is the current thinking regarding drain or weep holes for non-galvanized products? Years ago we used to provide drain holes in HSS columns.

A-1 Detailing
Millville, NB Canada

Cap plates for HSS columns are covered in detail in the AISC/STI *HSS Connections Manual.* You can order it at 800/644-2400 if you don't already have one.

In compression, the thickness of the cap plate is controlled by flexure of the plate or a limit state in the HSS wall, like compression yielding or crippling. In tension, it's flexure of the plate, tension yielding of the HSS wall or tension rupture of the weld. Welding is usually a fillet weld, just like you'd probably use for a base plate.

The main purpose of the cap plate is to provide a means of attachment between the HSS column and the member that rests on the top of the column (the most typical detail). It transfers the forces that occurs at the joint. It does keep water out, though.

From Section 10 of the AISC *Specification for the Design of Steel HSS* (1997), "When water can collect inside an HSS, either during construction or during service, HSS shall be sealed, provided with a drain hole at the base, or protected by other suitable means." The associated commentary gives further insight. The Specification and the Commentary are printed in the HSS *Manual.*

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