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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* via AISC's Steel Solutions Center at:

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CAMBERING GALVANIZED MEMBERS

from August Steel Interchange

Are there any special considerations that need to be made in specifying camber in a beam that is also to be hot-dip galvanized?

Question submitted anonymously

There does not seem to be much information on the subject. Nevertheless, I would offer the following:

In 1991, in an attempt to gain a better understanding of this subject, Bethlehem Steel personnel monitored thirteen 54-foot span W24x62 grade 50 beams. The beams were mechanically (cold) cambered at the mill and then transported to a hot-dip galvanizing facility. Measurements were taken immediately prior to and after galvanizing. Before galvanizing, cambers measured from 2⁷⁄₈" to 3³⁄₄", with an average of 3³⁄₁₆". After galvanizing, cambers measured from 2⁷⁄₈" to 3¹⁄₂", with an average of 3¹⁄₈". Similarly, in a paper distributed by the American Galvanizers Association entitled "Construction of Maintenance-Free Hot Dip Galvanized Bridges" by Ichikawa and Shimomura of the Japan Highway Public Corporation, a difference of only 5 mm (³⁄₁₆") before and after galvanizing was observed for girders with spans of 13.9 m (45.6'). In both studies there were members that lost no camber. In the Japanese report, some girders actually gained camber.

From the above, it could be concluded that hot-dip galvanizing has little effect on camber. In addition, since it is usually better if the camber in a member is a little less than specified, rather than greater, it is probably best to ignore the effects of hot-dip galvanizing when specifying camber. However, the above data is far from exhaustive. Depending on how critical this issue may be on a specific project, it might be worthwhile to monitor, at least initially, to make sure that the desired results are realized.

Jay W. Larson, P.E.
Bethlehem Steel
Bethlehem, PA

Galvanizing any steel members will probably result in deformations. These deformations will often require cold bending the members to straighten them. The mill rolling process locks in residual compressive stresses in structural steel shapes. The process of galvanizing releases these stresses in much the same manner as flame cambering a hot rolled member.

Galvanizing a cambered steel member may or may not require re-cambering. It depends on the size of the members, and the hot dipping process.

Harold Sprague
Black & Veatch

ANCHOR RODS TOO SHORT

from September Steel Interchange

Are there any guidelines or recommendations concerning the repair of anchor rods without adequate projections? This question applies particularly to applications in rigid frames and braced frames where tension is a limiting design condition. Also these are applications where epoxy anchors are not applicable. We know of several methods of repair - couplers or cutting and welding bolt projections. Could you supply some information on the applicability of each repair—minimum/maximum size of anchor, minimum/maximum projection, minimum/maximum plate size?

Kurt Swensson
KSI Structural Engineers
Atlanta, GA

AISC's *Steel Design Guide No. 1—Column Base Plates*, page 45, illustrates some suggested repairs. The use of a coupler may be precluded if there is insufficient space to thread the anchor bolt and fully engage the coupler. Welding may be precluded if the anchor rod is not weldable. Determine what the carbon equivalence of the anchor rod is prior to welding.

If epoxy bolts cannot be used, you might be forced to use a mechanical anchor. If a mechanical anchor is used, I would consider a mechanical anchor using an under-reamed hole.

If the nut is partially engaged, you can also calculate the amount of effective bolt engagement using the *Fastener Design Manual* by Richard T. Barrett, NASA Reference Publication 1228.

Harold Sprague
Black & Veatch

I ran into a similar situation once with a project involving the base of a tall tower where a couple of anchors did not have sufficient projection to accommodate two nuts for locking action. In this case, the columns were subjected to mild vibrations and loosening of the nuts had to be prevented. I did not want to weld an extension anchor or weld the nut to the bolts for the obvious reasons. The solution involved tack welding retainer bars to the column base plate adjacent to each of the anchor bolts.

While in your case, this solution may not be effective due to controlling tension, welding to an existing anchor should be avoided if at all possible.

Vijay P. Khasat, P.E.
Senior Project Manager

STIFFENER REQUIREMENTS

from September Steel Interchange

Regarding Chapter K of the 1989 ASD *Specification,* reference page 5-82, Section K1-8, Paragraph 3:

If Sections K1.4 or K1.6 require stiffeners, the stiffeners shall be designed as axially compressed members (columns) in accordance with the requirements of Section E2.

If Section K1.4 requires the stiffener, I would design the stiffener as a compression member with an axial load of *R* from section K1.4. If Section K1.6 requires the stiffener, should the stiffener be designed for an axial load of Pb_f from section K1.6 or from the computed force delivered by the flange? If Pb_f is used, often the stiffener (assuming the same width as the flange) will be thicker than the flange and this appears odd to me.

Paul Howell

Equation K1-8 in the 1989 *ASD Specification* is lifted unchanged from the 1986 *LRFD Specification*. Thus the capacity calculated from the equation must be compared to a factored load, which is what Pb_f is. When designing the stiffener, the un-factored load should be used, since the ASD design incorporates the safety factor in the allowable stress. Equation K1-1 in the *ASD Specification* is a restated version of K1-1 in the 1986 *LRFD Specification,* so the same situation would occur when stiffeners are required for tensile loads on flanges.

Malcolm A. Carter, P.E.
Needham and Associates
Overland Park, KS

WELDING ON EXISTING MEMBERS

from October Steel Interchange

It is a general rule that welding on an existing structural member is not permitted, unless provisions are made to unload the member first (if the member is being reinforced), and that the weld not degrade the properties of the material?

Is there a written reference that discusses this, both from a code perspective, and a practical approach?

Alan L. Blosser P.E.

The only "code" that you will find on this is the AWS D1.1 Chapter 8, but it places all of the responsibility on the EOR. There is nothing that requires you to totally or partially unload a structure. In fact more often than not, shoring and unloading is not required.

On the practical side I would reference "Field Welding of Existing Steel Structures" by David T. Ricker, AISC *Engineering Journal*, 1st Quarter 1988; "Reinforcing Steel Members and the Effects of Welding", by R. H. R. Tide, AISC *Engineering Journal*, 4th Quarter 1990; and "Strengthening of Steel Structures Under Load" by Terence Patrick O'Sullivan, The Institution of Civil Engineers, Vol. 2, 1953.

Each article has some wisdom to offer and some caveats, and the article by Tide has some good references.

Harold Sprague
Black & Veatch

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