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.

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.

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 \( P_f \) from section K1.6 or from the computed force delivered by the flange? If \( P_f \) is used, often the stiffener (assuming the same width as the flange) will be thicker than the flange and this appears odd to me.

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?

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.


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

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 \( P_f \) 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-