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C B

A project on which we are installing shear studs specifies a composite steel system comprised of 2 in. concrete over 3 in. metal deck. Headed anchor studs, ¾ in. in diameter, are specified and noted to be a minimum of 1.5 in. above the deck and ½ in. below the top of the concrete. In an ideal situation, this can theoretically be achieved with 4⅞-in. studs that achieve 4½ in. of finished length. However, this only occurs where studs are installed through metal deck and ⅜-in. burn-through is theoretically achieved. At girders parallel to deck direction where the stud attaches directly to the girder flange, the theoretical burn-through is 3⁄16 in. and thus the finished length is 4 11⁄16 in. Both conditions run a high risk of being exposed when typical fabrication tolerances are considered (crown-up fabrication) even if there is no camber required. Section I3.2c of the AISC *S ec f ca* has the following requirements: 2 in. minimum slab over deck, 1.5 in. minimum length above metal deck and ½ in. minimum of concrete cover to surface. Are there permitted deviations to this rule? Are two different stud lengths required in this situation?

The system you have described satisfies the requirements of the AISC *S c ca* but, as you've noted, does not allow much room for tolerance. The specific provision in Section I3.2c(1)(2) states: "Steel headed stud anchors, after installation, shall extend not less than 1½ in. above the top of the steel deck and there shall be at least ½ in. of specified concrete cover above the top of the steel headed stud anchors." There are a couple of nuances within the wording here that are worth pointing out.

First and foremost, the 1½ in. minimum stud projection above the deck is structurally more important to the performance of the system than the ½ in. clear cover over the top.

as *M/M*. Up to a shape factor of about 2 the denominator will be less than one, tending to increase the coefficient. Beyond this shape factor, the coefficient will begin to decrease. Why should this be? The greater the shape factor, the more inelastic deformation will be required to fully yield the section. In other words, the demand becomes greater and greater. Also, at a shape factor of about 2, the shape is likely moving from a singly symmetric I-shape to something approaching a tee. It is interesting to note that there is no case addressing the web of a tee with the flange in compression. To me, this is another indication that at this extreme the stability of the web is not a concern.

This condition will be addressed in the Commentary to the 2016 *S c ca*. The following statement has been added: "In extreme cases where the plastic neutral axis is located in the compression flange, = 0 and the web is considered to be compact." This corresponds to the logic above.

If the web is compact, then Section F3, not Section F4, applies and a zero will not appear in the denominator.

I believe it is always appropriate (necessary!) to exercise engineering judgment. It is especially critical to do so when addressing conditions at the fringes of those considered in the *S c ca*. It seems there are two different extremes that can cause the plastic neutral axis to be located in the compression flange. One would be where the compression flange is very clearly compact—i.e., it is very thick and relatively narrow. In such a case, it would seem the assumption that the web is compact is uncontroversial. At the other extreme, where the compression flange is very thin but very wide, I would be hesitant to treat the condition using Case 16. The distribution of stress typically assumed when calculating hc and hp might not be appropriate when the effective flange consists of a very thin but very wide element.

La S. M , PE

. A A 3125

The new ASTM F3125, which consolidates the previous ASTM A325, A490, F1852 and F2280 standards, indicates in Table A1.1 that F1136 coatings are not approved for use with twist-off bolts (Grades F1852 and F2280). It is my understanding that this indicates that these coatings are prohibited for use with twist-off tension-control bolts. Some vendors state that these bolt-coating combinations are not prohibited. What is the intent?

You are referring to an ASTM standard. Therefore ASTM would be the appropriate source for an interpretation. I will, however, provide my own opinion.

F3125 provides two different descriptions: not approved and not qualified. These terms are defined in the standard:

- "Not qualified" in Table A1.1 means that a particular coating has not been qualified and accepted by ASTM committee F16 for use on 150 ksi/1040 MPa bolts.
- "Not approved" in Table A1.1 means that a particular coating was not approved for a particular bolt style or grade in the individual standard prior to combination into F3125.

The reason for the different designations may not be immediately clear, since both would seem to discourage the use of the coating with the fasteners listed. However, the Annex also states:

> "Coatings listed in this Annex for 150 ksi/1040 MPa bolts have been qualified and approved where indicated for use with 150 ksi/1040 MPa strength bolts. For use on 150 ksi/1040 MPa bolts, other coatings must be qualified i(An)0.5c 0 T0(ic)0.5(c)0.6 (o)0.5r.