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steel interchange

Composite Beams

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 *Specification* 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 *Specification* 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. Purely from a strength perspective, the concrete cover over the top of the stud provides no recognized additional capacity. In the above referenced language, the phrase "specified concrete cover" was carefully chosen and deliberated over within the technical committee that maintains this section of the *Specification*. The intent is to ensure that designers specify a minimum of ½ in. concrete coverage to account for some of the field inaccuracies, but it was recognized that the in the final, as-built condition, the coverage could be less. The Commentary to this section of the *Specification* discusses ways an engineer can mitigate the potential for exposed studs in their slab system which are obviously more critical in a thin-slab system.

So, to answer your first question, it is acceptable to encroach into the ½ in. cover if necessary, but the 1½ in. minimum stud projection should be maintained.

As to whether or not two different stud lengths are required, I think that is a question that should be posed to the engineer of record. If the design specifies a uniform slab thickness of 2 in. everywhere, regardless of whether or not the final floor is level, then I would be inclined to say two different stud lengths are not necessary. However, if the design specifies a level floor finish then it is possible that if beam cambers do not come out, you could have exposed studs and that extra 3⁄16 in. of stud length could become very important.

Susan Burmeister, PE

Web Compactness for Singly Symmetric I-Sections

I am designing a singly symmetric I-shaped member in flexure. The plastic neutral axis for this section falls within the compression flange resulting in a negative value for h_p/2. How can I determine whether the web is compact, non-compact or slender? Note that if the web is not compact, then Section F4 of the *Specification* applies and since λ_p is equal to λ_r the denominators in Equations F4-9b and F4-16b become zero—again resulting in a result that is difficult to interpret.

Table B4.1b of the AISC *Specification* applies to compression elements of members subject to flexure. If h_p/2 is within the flange, then, under a plastic stress distribution, the web is in tension and therefore doesn't need to be classified. If h_c/2 is not within the flange, then, under elastic stress, some portion of the web will be subjected to a linearly varying compression load. In such a case, the magnitude of the compression stress will be relatively small when the section is elastic. As more and more of the section is strained beyond the elastic limit, the length of web in compression will decrease. Both of these trends tend to indicate that the stability of the web will not be a concern.

There are several possible approaches. First, the limits could be calculated based on Case 15, the doubly symmetric case, with the length of the web, h, assumed to be h_c. I believe this would be a conservative approach. The coefficient of λ_r is the same for the doubly symmetric and singly symmetric cases. Now consider the calculation of λ_p. If the equation for Case 16 is applied to a doubly symmetric I-shape h_c/h_p is 1.0. A reasonable value for the shape factor of a rolled wide flange is 1.12. This value produces a coefficient of 3.77—pretty close to the coefficient for Case 15, 3.76. So Case 16 produces about the same result as Case 15 assuming the same parameters.

There are two ratios that determine the value of λ_r for Case 16. The first is h_c/h_p. For a case like yours, with the larger flange in compression this ratio will always be greater than one. A negative value for hp does not make sense physically relative to checking the stability of the web. However, as h_p approaches zero, it can be seen that the value for h_c/h_p becomes very large. This again tends to indicate that buckling of the web becomes less and less of a concern. The other factor is related to the shape factor, Z_x/S_x, which is obviously in the same proportion as M_p/M_y. 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 *Specification*. The following statement has been added: "In extreme cases where the plastic neutral axis is located in the compression flange, h_p = 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 *Specification*. 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.

Larry S. Muir, PE

Not Qualified vs. Not Approved in ASTM F3125

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 in accordance with IFI 144. Hydrogen embrittlement testing required by IFI 144 shall be performed in accordance with F1940 for internal hydrogen embrittlement and F2660 for environmental hydrogen embrittlement."

A footnote to the table in the Annex states:

> "Other metallic and nonmetallic coatings may be used on 120 ksi/830 MPa minimum tensile fasteners upon agreement between the purchaser and user. Performance requirements shall be specified by the purchaser and agreed to in writing. Coatings for 150 ksi/1040 MPa bolts must be qualified. See A1.1."

So the requirements for F1852 and F2280 are different. The standard does not prohibit any coating to be used with F1852 "upon agreement between the purchaser and user" with "performance requirements…specified by the purchaser and agreed to in writing." For F2280, coatings must be must be qualified.

The difference seems to involve hydrogen embrittlement. Galvanizing of A490 bolts (150 ksi) has been prohibited for some time. This is because the process can lead to hydrogen embrittlement, which can lead to failure of the bolt and is therefore a safety concern. The same concerns do not exist for 120 ksi bolts.

I take "not approved" as meaning that this combination has not been explicitly considered, but there is no reason to believe there is an inherent safety concern. Therefore, if you are going to do it you are on your own, relying on your own judgment and knowledge.

I take "not qualified" as meaning that there are known safety concerns with this combination, and it should not be used.

Larry S. Muir, PE

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