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steel interchange

Headed Stud Anchor Diameter for Composite Beams

What should be taken into account for selecting headed stud anchor diameter for composite steel beams? Are there any limitations on using ¾-in.- or ½-in.-diameter studs welded through metal deck to create composite action?

There are some considerations in selecting headed stud anchor (stud) diameter and a few limitations that are independent of diameter.

Size selection: Per the AISC *Specification* (a free download at **www.aisc.org/specifications**) Section I3.2c (1)(2), studs shall be ¾-in. or less in diameter. Also, per Section I8.1, the diameter of the stud shall not be greater than 2.5 times the thickness of the beam flange unless the stud is welded directly over the beam web. Per Section A3.6, headed studs shall conform to AWS D1.1, which only addresses studs ½-in. in diameter and larger, which therefore defines the lower size limit. Those are the specific code provisions pertaining to diameter limitations. Beyond that, it becomes an engineering assessment by you as to what is the more economical or practical solution to transfer the shear for your specific beam.

The cross-sectional area of a ¾-in. diameter stud is more than double the area of a ½-in. diameter stud. Since stud shear capacity is directly proportional to stud cross-sectional area, you will need more than double the quantity of ½-in. diameter studs to provide the same strength as ¾-in. diameter studs. You could confirm with a local steel fabricator or erector in your area, but I would expect that the labor cost of installing a larger quantity of smaller-diameter studs would exceed any cost benefit associated with using smaller-diameter studs unless you have a floor system that uses small beams and doesn't demand very much in the way of shear transfer between the steel and concrete.

With respect to welding through deck, ICC-ES report ESR-1094 provides some good information. This may not be the only report available. Section 4.1 provides some guidelines for when studs can be welded through two layers of decking versus only one layer. This report does not distinguish between various stud diameters in defining the limitations on welding through deck.

Studs should not be welded through coated sheet metal other than typical steel decking. Welding through other materials, such as galvanized architectural flashing materials and even paint on the beam flange, can introduce contaminants into the weld that could affect the weld performance. Standard structural composite decks have controls in place to limit the potential of contaminants from the coatings.

Susan Burmeister, PE

Developing Flexural Strength of Spliced Wide-Flange Members

***Specification* Section J6 has always vexed me because it seems impossible to satisfy the requirement for groove-welded splices. By my calculation, for a W36×160 the weld-access hole reduces the flexural strength of the member to less than 70% of the flexural strength of the member without the weld access hole. My calculation is based on the moment being resisted only by the flanges, which are governed by yielding on the area of the flange. The shear strength is likewise reduced due to the presence of the weld access hole to 85% to 90% of the shear strength of the member. How does the spec intend for the designer to "develop the strength" of the shape?**

Relative to the flexural strength, the situation is similar to directly welded beam-to-column moment connections. The argument is sometimes made that one cannot develop the strength of the beam by connecting only the flanges while at the same time reducing the overall area by including weld access holes. An explanation is provided in the May 2012 article "Developing M_p" (available at **www.modernsteel.com**).

Though I am not aware of a document that addresses shear in this manner, a similar argument could be made. In fact, I suspect you make a similar assumption all of the time without giving it a second thought. It is not common for engineers to perform a yielding check on an uncoped beam with a bolted connection. Even a net area check is not typically considered necessary, since the flanges will tend to prevent such a failure. The only check commonly made is gross shear (yielding) based on the full area of the web. Regardless of the size of the weld access holes, they certainly remove less area than a full depth bolted connection.

It is also important to note *Specification* Section J6 is not requiring the connection to develop the actual strength of the member or the expected strength of the member (as we might in seismic design) but rather the "nominal strength of the smaller spliced section."

The case of the expected strength is an interesting one. The welded unreinforced flange-welded web moment connection in AISC 358 *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (a free download for members at **www.aisc.org/seismic**) has been shown through physical tests not only to develop the nominal strength of the beam but also to develop the actual beam strength and force hinging of the beam outside the connection. This is accomplished even with the larger weld access holes required in AWS D1.8.

Larry S. Muir, PE

Tributary Length for Prying Action

The article "A Slightly Longer Look at Prying" (an online supplement to the July 2016 article "A Quick Look at Prying," available at www.modernsteel.com) states that the effective width, *p*, for prying action can be conservatively taken as 3.5*b* but cannot exceed the spacing between the bolts. This conflicts with Part 9 of the *Manual*, which indicates that *p* is limited to twice *b*. Can the effective width exceed 2 times *b*?

Yes. The statement in the article is based on the upcoming 15th Edition *Manual*, which will revise the default effective width from $p = 2b$ to $p = 3.5b$. The new default value is based on guidance provided by the South African Institute of Steel Construction that was evaluated by the AISC Manual Committee and deemed to be adequate. The new assumed distribution angle is 60°, which is conservative but not as conservative as the assumed 45° angle used in the 14th Edition *Manual*.

It should be noted that the $2p$ limit was not intended to be a requirement. Even though it is not stated, it was only a recommendation. The recommendation was established because it was brought to the attention of the Manual Committee that there was a wide range of assumed tributary lengths being used in practice. It was felt that the *Manual* should provide guidance. As is often the case when engineers are forced to provide guidance, the first pass was conservative. Given the lack of data available at the time, the committee felt that the $2b$ guidance was a safe lower bound. With a closer look at the South African Institute of Steel Construction data the $3.5b$ limit was adopted. Note that the *Manual* also allows that a larger tributary length may be justified based upon testing or rational analysis.

Carlo Lini, PE

Fatigue and Removal of Backing for Fatigue

We have received shop drawings for a steel structure with moment frame connections using complete joint penetration groove welds. It is not a high-seismic project but we do have fatigue design considerations. The contractor has indicated on the shop drawings that backing bars will be used. We requested that the backing be removed in our review comments. The contractor is asserting that this is an unusual requirement and is treating this as a change in the contract. We believe that since the structure is subjected to fatigue, the contractor should be required to remove the backing at no additional cost. How is this situation treated in AISC documents?

This situation is not directly addressed by any AISC document. However, it is addressed in AWS D1.1, which is adopted by reference in Section J2 of the *Specification*.

Clause 2.17.2 of AWS D1.1 addresses backing and directly addresses the removal of backing, which often can generally be left in place. The treatment of backing is tied to fatigue considerations as you indicate. Clause 2.17.2.1 requires the engineer to provide the fatigue stress category in the contract drawings. If you provide the applicable fatigue stress category in the contract documents and AWS D1.1 Clause 2.17.2 requires removal for that fatigue stress category, backing removal is required. Otherwise, adding a requirement to remove backing with comments during shop drawing approval or by RFI response may represent a change to the contract. Section 4.4.3 and 9.3 of the AISC *Code of Standard Practice* (a free download from www.aisc.org/specifications) addresses revisions to the contract documents.

Carlo Lini, PE

Short-Headed Stud Anchors

Chapter I of the AISC *Specification* requires that "stud shear connectors, after installation, shall extend not less than 1½ in. above the top of the steel deck." I have an existing building and the original design documents indicate the shear studs extend only 1 in. above the steel deck. When calculating the composite strength of this member, is there a reduction factor that can be used to account for shorter stud?

AISC does not have sufficient information to make a recommendation about the performance of composite flexural members when the stud projection above the deck flutes is less than 1½ in. Therefore, the *Specification* does not provide a reduction factor for use with the current equations. You would have to use your own engineering judgment. The parameter limitations noted in current Section I3.2c were established to ensure beam designs are performed within the margins of the available research data, largely summarized in the first quarter 1977 *Engineering Journal* article "Composite Beams with Formed Steel Deck" (available for free to AISC members at www.aisc.org).

Provisions for composite members with formed steel deck did not appear in the AISC *Specification* until 1978. At that time, the *Specification* required the same 1½ in. projection per the research in the above-mentioned article. However, Section 1.11.6 stated: "When composite construction does not conform to the requirements of Sects. 1.11.1 through 1.11.5, allowable load per shear connector must be established by a suitable test program." This may have permitted a designer to use a shorter stud projection if they had access to some other test data in order to establish their shear connector strength.

Susan Burmeister, PE

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If you have a question or problem that your fellow readers might help you 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:

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