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steel interchange

Concrete Cover for Steel Headed Stud Anchors

AISC 360-10 Section I3.2c(1)(2) requires ½-in. clear cover between the top of the headed stud and the top of the concrete slab. We have a project with composite beams composed of steel beams with 3-in. metal deck, 3 in. of concrete topping (a total slab thickness of 6 in.) and studs with a length of 5½ in. After the concrete pour, the total slab depth at certain locations has been determined to be 5¾ in. This depth does not allow for the ½ in. clear cover over the studs. What is the basis of the ½-in. clear cover requirement? Is there a tolerance on this requirement? Is there any way to calculate a reduced beam composite action capacity when the studs only have ¼ in. of concrete cover above them?

The ½-in. minimum reflects the minimum clear cover that was used in the research that has been conducted on composite beams, and also is oriented toward preventing studs that protrude above the surface of the slab due to variations, such as in camber and slab thickness. I am not aware of any research that addresses the behavior of studs with less cover. The determination of appropriate reductions, if any at all, for studs with less cover is not addressed by the AISC *Specification* and is a matter of engineering judgment. From a pure strength standpoint, the stud strength is based upon the concrete cone below the head, not the concrete above the head of the stud.

On future projects you might consider specifying shorter studs to allow for the usual variations. The minimum height in this application is 4½ in., but a 5-in. stud will give you tolerance both on the minimum height above the deck and the minimum cover over the top of the stud.

Erin Criste

Minimum Dry Film Thickness

If the contract documents for a project do not specify the dry film thickness required for standard shop primer, is there a "standard" rule of thumb that we can use to meet minimum specifications?

The minimum dry film thickness for shop primer is 1 mil unless noted otherwise in the contract documents. This topic is addressed in AISC *Code of Standard Practice* Section 6.5.3, which states:

"Unless otherwise specified in the *contract documents*, paint shall be applied by brushing, spraying, rolling, flow coating, dipping or other suitable means, at the election of the *fabricator*. When the term "shop coat," "shop paint" or other equivalent term is used with no paint system specified, the *fabricator's* standard shop paint shall be applied to a minimum dry-film thickness of one mil [25 µm]."

Heath Mitchell, S.E., P.E.

Connection Design Loads

We typically delegate the design of connections on our projects. On one particular project, there is a bolted connection that is part of the seismic force resisting system and is required to be pretensioned with Class A surfaces per AISC 341-10 Section D2.2(4). Under normal operating conditions the connection will experience significant load reversal and we would like to designate it as a slip-critical connection. However, we want to avoid the fabricator having to design this as slip-critical for the full amplified seismic load that we specify on the drawings. Is it an acceptable practice to provide multiple load cases (i.e., a serviceability load case for which the connection is designed as a slip-critical joint and an amplified seismic load case for which the connection is designed as a bearing joint)?

Yes. The AISC *Seismic Provisions* language recognizes exactly what you are describing when it requires preparation for slip resistance but design for bearing values. Assuming you really do need a slip-critical connection for the service-level loading, it would be reasonable to give a separate loading condition for each case. The key is to make your intent clear in the contract documents. Discuss it to be sure at the pre-detailing meeting, too.

I should point out that Section 4.2 of the RCSC *Specification* (a free download from **www.boltcouncil.org**) requires pretensioned, not slip-critical, connections for "joints that are subject to significant load reversal." Section 4.3 requires slip-critical connections for "joints that are subject to fatigue load with reversal of the loading direction." Therefore, the connection need not be designed as slip-critical unless this is a fatigue condition. In my experience talking with engineers there is a lot of confusion on this point.

Larry S. Muir, P.E.

Weld Tabs

Does AISC require the use of weld tabs in CJP-groove welded joints? What if the welds are specified as demand-critical?

Weld tabs are used when specified by contract documents or when deemed necessary by the contractor to provide ease in terminating welds at the end of a joint. Often they are needed, but not always. Accordingly, weld tabs are not mandated for use by AISC 360, AISC 341, AWS D1.1 or AWS D1.8. Further, none of these documents require the use of weld tabs for demand-critical welds.

AISC 341-10 Section I2.3 and AWS D1.8 Clause 6.11 contain provisions related to weld tab removal. These provisions should not be viewed as implying that weld tabs must be used. They are simply provisions that apply to weld tab removal if weld tabs happen to be used.

Keith Landwehr

Heat Straightening

Can heat be used to straighten steel members that are bent or damaged during erection? Are there any precautions that should be taken if the member is part of the seismic force resisting system (SFRS)?

Heat straightening is an acceptable method for repairing bent members, as well as for inducing shop camber or sweep. The AISC *Specification* section that contains the requirements for the use of heat in this way is M2.1 Cambering, Curving and Straightening, which states:

> "Local application of heat or mechanical means is permitted to be used to introduce or correct camber, curvature and straightness. The temperature of heated areas shall not exceed 1,100 °F (593 °C) for ASTM A514/A514M and ASTM A852/A852M steel nor 1,200 °F (649 °C) for other steels."

The idea of providing an upper limit on temperature is to provide a margin of safety against changing the metallurgical structure in the heated region. Some engineering firms may desire to be informed and provide approval before heat straightening is performed. This stipulation should be communicated in the contract documents, since it is otherwise allowed by code.

The proper use of heat straightening is acceptable for steel members in general, whether or not they are part of the SFRS. The AISC *Seismic Provisions* does not contain an exception to AISC 360 Section M2.1. There is not any published guidance that I am aware of that is specifically related to heat straightening members of the SFRS, so the use of heat straightening becomes a matter of engineering judgment. The concern related to members of the SFRS is ductility and not strength. Bending something back and forth uses up some of the available ductility. In gravity loaded elements, this is usually not a concern, but it may be for elements in which significant inelastic strain demand is expected.

If the member or element to be straightened is part of an ordinary-type frame that is expected to have limited inelastic deformation capacity, then you may be able to justify treating it like any other steel member. In support of this, ordinary-type frames do not have protected zones. Pins, welds, etc. are allowed in the zone wh( )3vhis iimited iuctility as ielaid mupn. oHicaz5l5 Tw 0 oHicartenina9ecomesi.tify trl.(orc0.01 (a1.333 -1.elem).)36(oHic-)Tj15e frale tis ilo mechanica (-)T0.1ed is p