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steel interchange

Thermal Cutting in the Field

Our company's standard specification prohibits thermal cutting in the field. From what we have seen, this appears to be a common prohibition for many projects. We are now facing a situation where the flanges of a beam that has been erected must be prepared to receive a complete joint penetration (CJP) groove weld. Should we enforce this prohibition? What are our other options?

No. You should not enforce this prohibition for the case described. There is likely no practical alternative.

Thermal cutting is permitted under Section M2.2 of the AISC *Specification*. This section references AWS D1.1 for further requirements. Though Section M2 is titled "Fabrication," keep in mind that this section applies to fabrication whether it occurs in the shop or the field. The situation that you describe would be classified as field fabrication. It is not uncommon for thermal cutting to take place in the field due to design changes or for the remediation of the late-discovery of fabrication and detailing errors.

Though AISC does not prohibit thermal cutting in the field, it is not unusual for project specifications to prohibit field cutting. In my experience, the prohibition typically involves one or more of three concerns:

1. Engineers and owners sometimes prohibit field cutting so that any field cutting that is unavoidable must be approved by the engineer of record. In such instances, the intent is not really to prohibit all field cutting but rather to prevent uncontrolled thermal cutting.
2. Concerns about the quality of the cut. Unguided thermal cutting in the field (or in the shop) can sometimes produce a rough cut—sometimes described as "beaver-chew." This is not an unavoidable result of field cutting, but it can be more difficult to perform quality cuts in the field. As stated in Section M2.2: "Gouges deeper than 3⁄16 in. (5 mm) and notches shall be removed by grinding or repaired by welding." In the field, the erector will typically be responsible for making the cut and addressing any quality issues through grinding or welding. The erector therefore has an incentive to make the best cut possible. In your case, the cut will ultimately be incorporated into the CJP groove weld, so any repair will essentially be by welding.

   It is possible (but not inevitable) that the quality of the cut may result in a geometry that violates the prescribed geometry for prequalified welds. For instance, the root opening may be uneven and/or exceed that permitted. If this is the case, the repair would involve buttering passes to close the opening. Again, this would be additional work for the erector, which they have an incentive to try to avoid by making the best cut possible.

   In areas of low demand, it may be acceptable, based on the judgment of the engineer, to leave a rough cut unrepaired. Leaving the cut unrepaired may reduce the cost involved in repairing the error that lead to the thermal cutting in the first place. It is best to discuss and agree to the requirements for the cutting before the work is performed; see item 1 above.
3. A third concern sometimes voiced by engineers is metallurgical effects from uncontrolled cutting. Thermal cutting will cause some local metallurgical changes, whether it is done in the shop or in the field. The controls in the AISC *Specification* make those effects negligible, so the point is to adhere to them in both the shop and the field. Additionally, in your case, the surface will be welded, so the affected area will be incorporated into the weld.

Larry S. Muir, PE

Unusual End-Plate Moment Connection Geometry

I am working on a field fix where the existing bolt gage in the column at an end-plate moment connection is greater than the flange width of the beam. The articles and design guides that I have seen all assume that the bolt gage is less than the beam flange width and that the end-plate width does not exceed the beam flange width by more than 1 in. My condition will violate both criteria.

I want to make sure I understand the basis of these checks and what modifications would be necessary to accurately reflect my condition.

If the bolt gage is wider than the beam flange width, the assumed yield line pattern may not form and an alternative pattern may need to be used. I'm not aware of any publications that provide guidelines for this case, so engineering judgment must be applied. One option would be to use extension plates welded to the flange to simulate a wider flange.

When the bolt is close to a boundary (in this case, the beam flange), the yield line pattern is defined by the boundary. At the critical distance, defined by s in AISC Design Guides 4: *Extended End-Plate Moment Connections Seismic and Wind Applications* and 16: *Flush and Extended Multiple-Row Moment End-Plate Connections* (free downloads for members at **www.aisc.org/dg**) the yield line no longer forms at the boundary. This is because the internal energy is lower for the pattern defined by s rather than the pattern defined by the boundary. Dranger (1977) derived an equation for the critical distance by minimizing the load. The equation in Design Guides 4 and 16 for s is a simplified version of Dranger's equation, based on a beam web thickness of zero.

Borgsmiller's thesis has derivations for several end plate geometries. The solutions developed by Kapp (1974) and Dranger (1977) are well-documented and easy to follow.

References:

- Borgsmiller, J.T. (1995). "Simplified Method for Design of Moment End-Plate Connections," M.S. Thesis, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, Virginia.
- Dranger, T.S. (1977), "Yield Line Analysis of Bolted Hanging Connections," *Engineering Journal*. AISC, Vol. 14, No. 3.
- Kapp, R.H. (1974), "Yield Line Analysis of a Web Connection in Direct Tension," *Engineering Journal*, AISC, Second Quarter.

AISC *Engineering Journal* papers can be downloaded from the AISC website at **www.aisc.org/ej**.

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Conflicting Requirements for Seismic Design

We have found conflicting requirements in the 2nd Edition of the AISC *Seismic Design Manual*. Section 4.2 of the *Seismic Design Manual* states: "The only system specific requirements for an OMF (ordinary moment frame) pertain to the beam-to-column moment connections." However, the commentary to Section E1.2 of the *Seismic Provisions* states: "Thus, the basic design requirement for an OMF is to provide a frame with strong connections. That is, connections should be strong enough so that, as noted above, connection failure is not the first significant inelastic event in the response of the frame to earthquake loading. This applies to all connections in the frame, including beam-to-column connections, column splices, and column base connections."

There appears to be a conflict between these two statements. The commentary states that that splices and base plates should not govern, while the *Seismic Manual* indicates that "specific requirements for an OMF pertain to the beam-to-column moment connections" only. Please provide clarification.

There is no conflict. Both documents are correct and consistent. The fact that there are "system-specific requirements for an OMF" does not mean that there cannot be general requirements that also apply.

All requirements of the *Seismic Provision* must be met. Some engineers mistakenly assume that the sections addressing each of the systems (i.e., OMFs, special moment frames, ordinary concentrically braced frames, special truss moment frames, etc.) are self-contained and apply independent of the rest of the *Seismic Provisions*. This is not the case. There are general requirements that apply to all systems provided in the chapters that precede those addressing the specific systems. Section D2.5 addresses column splices and Section D2.6 addresses column bases. D2.6 in turn references D2.5 for some of the loading requirements. All of these requirements must be met. In fact, the commentary to Section E1.2 goes on to state: "Requirements for OMF column splices and column base connections are covered in Section D2."

Though not directly related to your question, another fact that's sometimes overlooked is that the requirements of the *Specification* must also be met. Section A1 of the *Seismic Provisions* makes this clear, stating: "These Provisions shall be applied in conjunction with the AISC *Specification for Structural Steel Buildings*, hereafter referred to as the *Specification*. All requirements of the *Specification* are applicable unless otherwise stated in these *Provisions*. Members and connections of the SFRS (seismic force-resisting system) shall satisfy the requirements of the applicable building code, the *Specification* and these *Provisions*."

Larry S. Muir, PE

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