steelwise

TAKE A MOMENT TO CONSIDER THIS MOMENT CONNECTION

Sometimes, simple questions aren't as simple as they appear.

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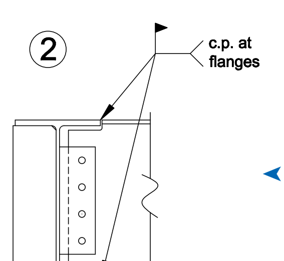
REGULAR READERS OF *MODERN STEEL CONSTRUCTION* are familiar with the Steel Interchange column, where we provide responses to questions we receive through AISC's Steel Solutions Center.

Generally, each installment focuses on a single topic, which can be addressed in a relatively brief manner. But on occasion, we receive inquiries that involve more complex topics or touch on multiple aspects of a broader topic, and whose answers we feel will prove useful to the design community—in some cases useful and detailed enough to evolve from Steel Interchange to SteelWise. What follows is one such case. The names have been withheld and the details modified to protect the innocent.

The Question

The question we received involves a relatively small steel structure. The seismic force-resisting system (SFRS) in each orthogonal direction is an ordinary moment frame (OMF). The design base shear is 20 kips. The beams in the north-south direction run continuously over the tops of the columns, which are connected to the underside of the beams with CJP (complete joint penetration) groove welds. The beams in the east-west direction frame into the north-south beams and are also CJP groove welded. See Figure 1.

The inquirer referenced the provisions of AISC *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (a free download for members from **www.aisc.org/seismic**) in particular, sections 3.3.2 and 3.3.3.



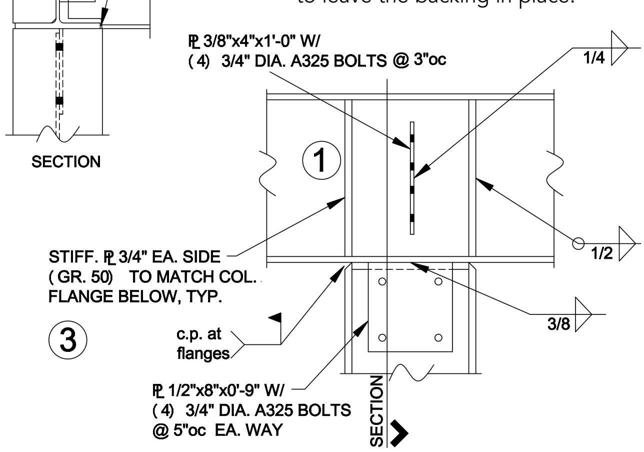


Figure 1. The engineer specified that all backing must be removed. However, the contractor requested to leave the backing in place.

Their question: Must the backing be removed?

There are several ways to interpret this question. Presumably, the intent is to determine whether the AISC *Specification* or *Seismic Provisions* require removal of the backing. The answer in this case is no. (We'll get to further information related to the requirements of the *Seismic Provisions*, their intent and how they relate to this condition, in a minute.)

An alternative interpretation is contractual. The contract documents required the removal of all backing. Therefore, the backing must either be removed or the contract must be altered. Section 4.2 of the AISC *Code of Standard Practice* describes the appropriate process as follows: "When the fabricator submits a request to change connection details that are described in the contract documents, the fabricator shall notify the owner's designated representatives for design and construction in writing in advance of the submission of the shop and erection drawings. The owner's designated representative for design shall review and approve or reject the request in a timely manner."

The Commentary provides further information: "When the fabricator intends to make a submission of alternative connection details to those shown in the contract documents, the fabricator must notify the owner's designated representatives for design and construction in advance. This will allow the parties involved to plan for the increased effort that may be required to review the alternative connection details. In addition, the owner will be able to evaluate the potential for cost savings and/or schedule improvements against the additional design cost for

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review of the alternative connection details by the owner's designated representative for design. This evaluation by the owner may result in the rejection of the alternative connection details or acceptance of the submission for review based upon cost savings, schedule improvements and/or job efficiencies."

The information provided is consistent with the *Code of Standard Practice*. The fabricator has notified the owner's designated representative for design—the engineer of record—of a requested change in the details, and the engineer is reviewing the alternative connection details. Typically, once the owner's designated representative for *design* has determined the change is technically sound, the owner's designated representative for *construction* will work out the potential for cost savings and/or schedule improvements with the fabricator.

Treatment of Backing

A common misconception is that all backing must be removed in the SFRS. In many instances backing can be left in place, but in some instances it can be left but must be reinforced with fillet welds. The *Provisions* do a pretty good job of indicating where each condition applies—with leaving the backing in-place being the default. However, the *Seismic Provisions* cannot address every conceivable condition. In such cases, the engineer of record must determine whether the backing must be removed or reinforced. This requires an understanding of the intent of the *Seismic Provisions* and the logic underlying the requirements.

For an OMF, Section E1.6b of the *Seismic Provisions* provides three options for fully restrained moment connections. Option (a) states that the moment connections shall be designed for the expected flexural strength of the beam multiplied by 1.1 (to account for strain hardening) and provides a corresponding shear. Option (b) requires the moment connection to be designed for the maximum moment and corresponding shear that can be transferred to the connection by the system. And option (c) provides three further options: the use of a connection qualified through physical tests, a prequalified connection per AISC 358 or a connection meeting prescriptive requirements laid out in this section.

While the middle option (choose a prequalified connection per AISC 358) seems like a plausible solution to the inquirer's issue, the connections shown in Figure 1 cannot be considered prequalified. Prequalified connections must conform to all the requirements of the connection type chosen, and the connections shown do not. Therefore, it must be assumed that some other option is being exercised.

The other two options available under option (c) can also be eliminated for the following reasons: Presumably, the connections have not been tested per Chapter K of the *Seismic Provisions*, and the requirements for the prescriptive connection described in Section E1.6b(c) indicate that "beam flanges shall be connected to column flanges using CJP groove welds." In the presented case, the beam flanges are not connected to the column flanges.

Therefore either option (a) or (b) must be applied. Neither requires backing to be removed, and there is no other requirement in the *Seismic Provisions* that applies to OMF connections and that explicitly requires the removal of backing.

However, by forgoing option (c) the design of these connections is left to the judgment of the engineer, who may wonder that while there is no requirement to remove the backing, whether the backing should still be removed.

This is where a deeper understanding of the intent is beneficial. To some extent, this deeper understanding can be derived from the Commentary to the *Seismic Provisions*. To some extent it can be inferred or deduced from the prescriptive requirements contained in the codes.

The following statements are made in the Commentary:

- "The presence of backing may affect the flow of stresses within the connection and contribute to stress concentrations. Therefore, backing removal may be required at some locations…" See Figure 2 (page 20).
- "Where steel backing remains in place in tee and corner joints with the load applied perpendicular to the weld axis, a fillet weld between the backing and the flange element of the tee or corner joint reduces the stress concentration at the weld root…" See Figure 2.
- "The requirement for removal of weld tabs and weld backing at column to base plate connections made with groove welds has been added to Section D2.6 as it is applicable to all SFRS systems in Sections E, F, G and H. The use of weld backing for a CJP groove weld of a column to a base plate creates a transverse notch. Consequently weld backing must be removed. For OMF, intermediate moment frame (IMF) and special moment frame (SMF) systems, weld backing is allowed to remain at the groove CJP welds of the top flange of beam-to-column moment connections if a fillet weld is added per Chapter 3 of ANSI/AISC 358 (AISC, 2010b). Similarly, an exception has been added for column bases to permit weld backing to remain at the inside flanges and at the webs of wide flange shapes when a reinforcing fillet weld is added between the backing bar and the base plate." See Figure 2.
- "Weld backing for groove welds in column splices may remain. The justification for this is that unlike beam-to-column connections, splices of column flanges and webs using weld backing result in no transversely loaded notch." See Figure 2.
- "At the root of groove welds between beam flanges or continuity plates and column flanges, the inherent lack of a fusion plane between the left-in-place steel backing and the column flange creates a stress concentration and notch effect, even when the weld has uniform and sound fusion at the root. Further, when ultrasonic testing is performed, this left-in-place backing may mask significant flaws that may exist at the weld root. These flaws may create a more severe notch condition than that caused by the backing itself (Chi et al., 1997)."
- "The stress and strain level at the groove weld between a continuity plate and column flange is considerably different than that at the beam flange-to-column flange connection; therefore it is not necessary to remove the backing. The addition of the fillet weld beneath the backing makes the inherent notch at the interface an internal notch, rather than an external notch, reducing the notch effect. When backing is removed, the required reinforcing fillet weld reduces the

a. b.

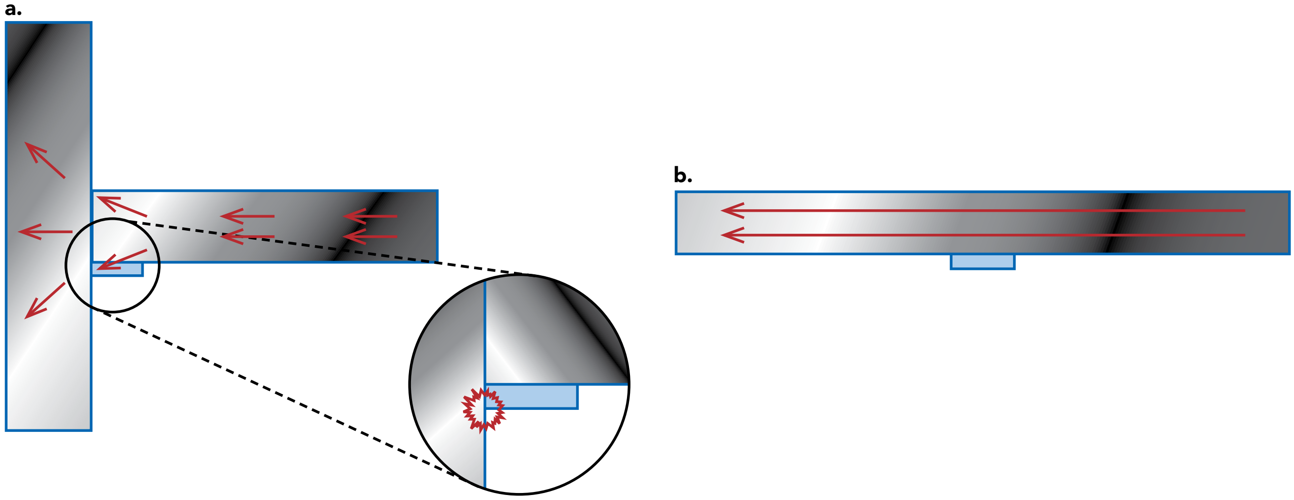

Figure 2. Effect of joint configuration on backing removal:
a. Backing to be removed at tee and corner joints since the stress may be directed to unfused area.
b. Backing can remain in place at butt joints due to less uncertainty regarding stress flow.

stress concentration at the right-angle intersection of the continuity plate and the column flange."

- "The removal of backing, whether fusible or non-fusible, followed by back-gouging to sound weld metal, is required so that potential root defects within the welded joint are detected and eliminated, and the stress concentration at the weld root is eliminated. The influence of left-in-place steel backing is more severe on the bottom flange, as compared to the top flange, because at the bottom flange, the stress concentration from the backing occurs at the point of maximum applied and secondary tensile stresses in the groove weld, at the weld root, and at the outer fiber of the beam flange. A reinforcing fillet weld with a 5/16-in. (8-mm) leg on the column flange helps to reduce the stress concentration at the right-angle intersection of the beam flange and column flange, and is placed at the location of maximum stress."
- "Because of differences in the stress and strain conditions at the top and bottom flange connections, the stress/strain concentration and notch effect created by the backing/column interface at the top flange is at a lower level, compared to that at the bottom flange. Therefore, backing removal is not required. The addition of the reinforcing fillet weld makes the inherent notch at the interface an internal notch, rather than an external notch, further reducing the effect."
- "Tack welds for beam flange-to-column connections should be made within the weld groove. Tack welds or fillet welds to the underside of beam at the backing would direct stress into the backing itself, increasing the notch effect at the backing/column flange interface. In addition, the weld toe of the tack weld or fillet weld on the beam flange would act as a stress concentration and a potential fracture initiation site."
- "After non-fusible backing is removed, back-gouging to sound metal removes potential root flaws within the welded joint. A reinforcing fillet weld with a 5/16-in. (8-mm) leg on the column flange helps reduce the stress concentration at the right-angle intersection of the beam flange and column flange."

We can use these statements to understand why the backing is being removed or reinforced. It can be distilled down to a few basic reasons:

1. The presence of backing may affect the flow of stresses within tee and corner joints.
2. The presence of backing may contribute to stress concentrations due to transversely loaded notches.
3. The presence of backing may mask significant flaws that may exist at the weld root when the weld is ultrasonic tested.
4. The attachments made to the backing to hold it in place may direct stress into the backing itself.

We can also see that there are mitigating factors that permit backing to be left even when the concerns listed above exist:

1. Weld backing at butt joints (splices) results in no transversely loaded notch.
2. The stress and strain level at the groove weld between a continuity plate and column flange is considerably different than that at the beam flange-to-column flange connection.
3. Because of differences in the stress and strain conditions at the top and bottom flange connections, the stress/strain concentration and notch effect created by the backing/column interface at the top flange is at a lower level, compared to that at the bottom flange.

A further consideration is the magnitude of inelastic stress and strain. Backing typically is not removed when designing structures that do not need to meet the *Seismic Provisions*. As stated previously, there are no explicit requirements to remove backing in OMF. Per the *Provisions*, OMFs are "expected to provide minimal inelastic deformation capacity in their members and connections," IMFs are "expected to provide limited inelastic deformation capacity through flexural yielding of the

IMF beams and columns, and shear yielding of the column panel zones," and SMFs are "expected to provide significant inelastic deformation capacity through flexural yielding of the SMF beams and limited yielding of column panel zones." The descriptions are more qualitative than quantitative, but there are clearly different expectations for these systems.

Ultimately, the engineer of record must decide what is appropriate for a given condition based on his or her own engineering judgment and knowledge, and this should never be forgotten. A process is described below that might be aid in these deliberations. The process described reflects the author's approach to the condition shown. The underlying logic and principles can be generalized but ultimately must be applied to the specifics of the condition at hand. The different parts of the connection are numbered in both the discussion below and in Figure 1.

1. First, let's consider the welds to the continuity plates. Here, backing would not be required to be removed even if for IMF or SMF because "the stress and strain level at the groove weld between a continuity plate and column flange is considerably different than that at the beam flange-to-column flange connection."
2. Now let's consider the connection to the "weak-axis of the column." The weld is made between the flanges of the two beams. This is a butt joint not a tee or corner joint. There is a smoother path for the stress flow, similar to a column splice. So again, removal of backing would not be required—even for IMF or SMF.
3. Finally, let's consider the connection to the "strong-axis of the column." Here, the column and beam are welded to form a tee joint. All four of the concerns listed above apply. It is not a butt joint and is not a weld to a continuity plate. The welding could be accomplished from either side of the flange. If the weld is made from the outside of the column, then this is similar to the condition that typically exists at the top flange condition of a beam-to-column moment connection, and for an IMF or SMF the tendency would be to leave the backing but apply a reinforcing fillet. If the weld is made from the inside of the column, then this is similar to the condition that typically exists at the bottom flange condition of a beam-to-column moment connection, and for an IMF or SMF the backing would have to be removed. However, this is neither a SMF nor an IMF. The system is expected to provide minimal inelastic deformation capacity. So again, removal of backing would not be required.

The engineer of record can require that the backing be removed even if it is not explicitly required by the *Seismic Provisions*. If this is the intent then per Section A4.2 of the *Seismic Provisions*, then this must be conveyed in the structural design drawings and specifications. When requiring something beyond the requirements of the *Seismic Provisions* or anything unusual, it makes sense to highlight the project-specific requirement so that it is not overlooked during the bidding.

Other Considerations

Beams continuous over columns in moment frames resisting lateral loads. Though it is not uncommon to stop the column short and attach it to the underside of the beams, it may be the less economical arrangement. Beams tend to be deep with thin webs. Columns tend to be shallow, with stout webs. Given a constant moment, a deeper member will produce less flange force than a shallower member. A stouter web will be less likely to require reinforcing than a thinner, deeper web. Both of these observations lead to the conclusion that framing a beam to a column is more likely to eliminate the need for reinforcing and therefore result in a more economical project. Also, when the column is stopped short, the gravity loads must also be transferred through the column flanges into the beam web—again increasing the likelihood that reinforcing will be required.

It is a common misconception that continuity plates are required at all moment connections designed under the *Seismic Provisions*. This is not true. For each system, including those satisfying AISC 358, checks are provided to determine whether or not reinforcing must be provided. Eliminating reinforcing, especially doubler plates, through the use of proper framing details and member choices can significantly improve the economy of structural steel framing. Uncertainty about the need for and the magnitude of reinforcing can also lead to ongoing contractual and cost issues for the project. A choice that reduces costs, disputes and RFIs seems like an all-around winner. In this case, the base shear (20 kips) seems pretty low, making the reinforcing an unnecessary belt-and-suspenders approach.

Other considerations related to the presence of continuity plates. The presence of continuity plates in this joint also causes problems with the beam-to-beam connection in the "weak-axis direction." Assuming a similar condition exists at each end of the beam it may be nearly impossible to erect the beam. An extended single-plate shear connection could be used to make erection easier or possible. Again, running the column through may be the better choice.

Welding considerations. Again, referring to Figure 1, in the "weak-axis direction" the weld between the beam flanges does not seem to account for tolerance in beam depth or flange tilt. We regularly receive inquiries from engineers looking for field fixes where the CJP groove weld is either not complete or is kinked due to such tolerances. Again, running the column through eliminates this problem, provided that the guidance in Part 12 of the AISC *Manual* and Appendix D of AISC Design Guide 13: *Wide-Flange Column Stiffening at Moment Connections* (a free download for members at **www.aisc.org/dg**) is followed.

At the top of the column, there is a ½-in. plate that acts as an erection aid and transfers shear. There appears to be a ⅝-in. weld on each side of this plate—and 1¼ in. of weld on a ½-in. plate seems very excessive. In the *Manual*, where we intend to develop the strength of the plate in a single-plate shear connection for rotational ductility, we recommend a weld that is ⅝ of the plate thickness. This would be 5⁄16-in. fillets on either side of a ½-in. plate. Per Table 8-12 of the *Manual*, a 5⁄16-in. fillet weld can be made in a single pass, while a ⅝-in. fillet weld can require six passes. This means the weld specified will cost more than six times the weld required to develop the plate, discounting safety factors. This detail contrasts starkly with the detail of the single-plate connection to the "broken" beam, which uses a ¼-in. weld for a ⅝-in. plate.

There is also a lot of welding in this detail, and some thought should be given to the effect of all of this welding on the members and connecting elements. Welding involves heat, and heat can lead to distortion. We have received a number of inquiries where excessive reinforcement involving large welds has led to distortion of columns that is so extreme that the column becomes unserviceable. In some instances, even with little information about the actual loads on the connection, it can be determined that the size of the welds is excessive and cannot be required per any requirements of the *Specification* or *Seismic Provisions*. Distortion due to the welds shown in this detail will likely not render the condition unserviceable, but there could be some distortion and its effects might be a consideration.

The requirements for the welds joining the continuity plates to the underside of the top flange of the beam are not clear, and we'll address this below, assuming that assumed that ½-in. fillet welds are intended. Per Table 8-12 of the *Manual*, this might require four passes of weld. However, there can be little load delivered to the stiffeners at this point. Couldn't the stiffeners simply be stopped short of the top flange?

Weld Symbols and Details

Figure 1 has several issues related to the use welding symbols and details.

The first involves probably the most misused symbol contained in AWS A2.4, *Standard Symbols for Welding, Brazing, and Nondestructive Examination*, the weld all-around symbol. The weld all-around symbol indicates that the weld continues completely around the perimeter of the joint. Figure 1 shows an all-around symbol at the welds to the continuity plates. However, in order to accomplish this weld, the continuity plate would have to be fitted and welded continuously at the flange-to-web fillet and set back from the toe of the flange sufficiently to allow welding across the edge of the continuity plate. This would be a very unusual and costly detail and would also involve wrapping the weld around the corners. Wrapping welds is not strictly prohibited but it can be problematic, as there is a tendency to gouge the corner. The bigger issue is that the detail simply isn't clear. Presumably the ½-in. weld applies to the welds at both the beam web and flanges, but this is an assumption as this is not what the symbol conveys. When you find yourself tempted to use the all-around symbol, please reconsider; chances are you will be using it incorrectly.

Another oddity of Figure 1 is that the column flange is wider than the beam flange. A CJP groove weld is called out between these members. Presumably, the intent is to provide a CJP groove weld only for the width of the beam flange. However, unless the maximum force that can be delivered to the system has been determined, the *Seismic Provisions* require the connection to develop the expected strength of the beam times 1.1 to account for strain hardening. It is unclear whether the intent here is to develop the expected strength of the column or whether the weld provided is felt to be consistent with the requirements of the *Seismic Provisions*. Adding to the confusion is the fact that the continuity plate shown in the elevation overhangs the beam flange terminating at the same location as the toe of the column. Why is this done? Is it assumed that the column flange will be CJP groove welded to the continuity plate beyond the end of the beam flange? This is certainly not what is shown in the detail and requiring such a weld would make for a very complex and expensive detail.

A final consideration is that the welds at the top of the column will have to be made overhead in the field from some sort of scaffolding, increasing both the cost and the difficulty of this weld.

It is probably not reasonable, or at least realistic, to assume that engineers have the same proficiency with weld symbols as detailers and fabricators. However, the engineer must communicate in a manner that facilitates fabrication and erection that is consistent with the intent of the design. Engineers and contractors should work together to ensure that what is built reflects what was designed.

The Steel Solutions Center receives a fair number of inquiries from contractors asking us to interpret the intent of the engineers on their projects. We cannot. If a detail is unclear to the contractor, it will likely be just as unclear to the Steel Solutions Center staff. Fabricators and their detailers should not hesitate to seek clarification. As we've all experienced, it is far better to clarify things up front rather than to correct things on the back end.

On the other hand, engineers should not feel obliged to represent every detail using standard symbols. When in doubt, draw it out. AWS A2.4 defines standard weld symbols. However, not all welds can be effectively described using these symbols alone. Clause 1.6 of AWS D1.1 states: "Special conditions shall be fully explained by notes or details." This indicates that notes or details must be used where the limitations of the available symbols prevent adequate communication of the intent. However, this clause can also be used to overcome lack of proficiency in the use of the standard symbols.

If, as the engineer, you are not sure your details are clear, then supplement them with sketches and/or notes. You may also want to highlight these items during project meetings to make sure what gets done is what you expected to get done. ■