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# steel interchange

## Ordinary Moment Frame Truss Systems

**Relative to ordinary moment frame (OMF) truss systems, the Commentary to Section E1.2 of the 2010 AISC *Seismic Provisions* states: "…to design the truss and the truss-to-column connection for the maximum force that can be transferred by the system... The maximum force that can be delivered to the truss and truss-to-column connection can be based on the flexural capacity of the columns..." However, neither the beam nor the column is typically designed for the connection force when designing an OMF. Why does a truss, acting as the beam in an OMF, need to be designed for the connection force?**

In the beam-column system, yielding can occur in either the beam or the column. The Commentary to Section E1.4 states: "Unlike SMF [simple moment frames], there is no beam-column moment ratio (i.e., strong column-weak beam) requirement for OMF. Consequently, OMF systems can be designed so that inelasticity will occur in the columns." If the connection develops the expected strength of the beam, then this will cause yielding in either member, the beam or the column, and the expected behavior is achieved. You are also okay if you design the connection for the maximum moment that can be delivered by the system, which might be governed by the flexural strength of the column.

For a truss system I think the typical situation would be for the truss to have greater flexural strength than the column. The Commentary implicitly assumes this to be the case. Based on this assumption, the guidance relative to the truss itself becomes more of a logic check than a design requirement. The process might be: (1) Design the truss and columns per the building code, (2) Design the connections for the strength of the column and (3) Check the strength of the truss against the strength of the column. In most instances, I think the third check will be satisfied. However, if the truss is not stronger than the column, then the assumed model is wrong and the truss will yield and the connections in Step 2 have been overdesigned.

You might then also want to consider other factors. For instance, the Commentary to Section E1.5 states: "There are no special restrictions or requirements on member width-to-thickness ratios or member stability bracing, beyond meeting the requirements of the *Specification*. Although not required, the judicious application of width-to-thickness limits and member stability bracing requirements, as specified for moderately ductile members in Section D1, would be expected to improve the performance of OMF." Even without explicit width-to-thickness limits and member stability bracing requirements, it is likely that the typical rolled column or beam will behave "better" than a truss, which could take on many different configurations, some of which might not be especially ductile.

The Commentary language has the implicit assumption that the column and not the truss will yield. Though not a requirement, a frame in which the column yields is probably the more common condition and likely the simpler one.

*Larry S. Muir, P.E.*

## Expansion Joint References

**Can you provide any references that address the need for and placement of expansion joints?**

Here are a few references that discuss the layout of expansion joints:

- Federal Construction Council (1974), "Technical Report No. 65, 1974, Expansion Joints In Buildings," National Research Council, Washington D.C.
- Fisher, James M. (2005), "Expansion Joints: Where, When and How." *Modern Steel Construction*, April.
- Fisher, James M. (2004), *Design Guide 7: Industrial Buildings – Roofs to Anchor Rods*, Second Edition, AISC.
- Saunders, C. (2006), "Seismic Joints in Steel Frame Building Construction," *Practice Periodical on Structural Design and Construction*, 11(2), 71-75.
- Brady, Matthew D. (2011), "Expansion Joint Considerations for Buildings." *Modern Steel Construction*, May.

The engineer of record will ultimately need to decide if and where to locate expansion joints, and the foregoing guidance should help in doing that.

*Susan Burmeister, P.E.*

## Higher-Strength Steels

**Can the AISC *Specification* be used to design members composed of steels with yield strengths in excess of 65 ksi?**

Yes. The AISC *Specification* considers steels with yield strengths greater than 65 ksi. Section A3.1a states: "Structural steel material conforming to one of the following ASTM specifications is approved for use under this *Specification*."

ASTM A913 and A514 are listed with no specific reference to a permitted yield stress. So 70 ksi A913 shapes or A514 plate—which has a yield stress of 90 ksi or 100 ksi, depending upon thickness—would both be considered approved for use under the *Specification*.

Further evidence that higher-strength steels are generally permitted is that steels with $F_y$ greater than 65 ksi are specifically excluded from plastic design in Sections B3.7 and Appendix 1.2.1. There would be no reason to make such statements if higher yield strengths were generally prohibited.

*Carlo Lini*

## Web Openings in Plate Girders

**Can you provide any guidance relative to the strength of plate girders with web openings?**

This subject is not addressed in the AISC *Specification*, so you will have to use your own judgment to determine what is appropriate for your situation. I have provided some thoughts below that you may find helpful.

Practical methods exist to calculate the elastic buckling strength and fatigue resistance. The fatigue resistance can be calculated using the AASHTO *Specification* (AASHTO, 2012) requirements. For combined flexural and shear loads, the web buckling strength of non-stiffened webs can be calculated using Equations 4.12 through 4.15 in the *Guide to Stability Design Criteria for Metal Structures* (Ziemian, 2010), which were developed by Redwood and Uenoya (1979).

Hagen and Larsen (2009) published design guidance on reinforced and non-reinforced openings, including the effects of various reinforcement types and vertical eccentricities between the opening and the beam mid-depth. However, the equations are based on European codes. Further information on the buckling strength of plates with openings under various loading conditions can be found in Yettram and Brown (1987) and Paik and Thayamballi (2003).

References


- AASHTO (2012), *LRFD Bridge Design Specifications*, American Association of State Highway and Transportation Officials, Washington, D.C.
- Hagen, N.C. and Larsen, P.K. (2009), "Shear Capacity of Steel Plate Girders with Large Web Openings, Part II," *Journal of Constructional Steel Research*, Vol. 65, Issue 1, pp. 151-158.
- Paik, J.K. and Thayamballi, A.K. (2003), *Ultimate Limit State Design of Steel-Plated Structures*, John Wiley & Sons.
- Redwood, R.G. and Uenoya, M. (1979), "Critical Loads for Webs with Holes," *Journal of the Structural Division*, ASCE, Vol. 105, No. ST10, October, pp. 2053–2067.
- Yettram, A.L. and Brown, C.J. (1987), "Improving the Elastic Stability of Square Perforated Plates," *Journal of Constructional Steel Research*, Vol. 7, pp. 371–383.
- Ziemian, R.D. (2010), *Guide to Stability Design Criteria for Metal Structures*, Sixth Edition, John Wiley & Sons.


*Bo Dowswell, P.E., Ph.D.*

## Embedded Columns in Special Moment Frames

**Example 4.4.4 in the 2nd Edition AISC *Seismic Design Manual* illustrates the design of an embedded column used in a special moment frame. It seems that $M_u$ is determined, but this moment is never checked against any limit state. What is the mechanism for transferring the moment into the foundation?**

The embedded column is being designed using the connection of a coupling beam to a shear wall as the model. Therefore, the checks in Section H4 of the AISC *Seismic Provisions* are used. The Commentary to Section H4 states: "For cases in which the coupling beam embedment into the wall piers is the only mechanism of moment resistance, the embedment length has to be long enough to develop the nominal shear strength of the coupling beam. Models have been developed for connections between steel brackets and reinforced concrete columns (e.g., Mattock and Gaafar, 1982). These models are used to compute an embedment length required to prevent bearing failure of concrete surrounding the flanges of the embedded steel members… Equation H4-2 is based on the model developed by Mattock and Gaafar (1982) and recommended by ASCE (2009). The strength model in this equation is intended to mobilize the moment arm between bearing forces $C_f$ and $C_b$ shown in Figure C-H4.6."

As you point out the moment, $M_u$, is calculated in the example. However, the assertion that this moment is never checked against the limit states is not correct. The shear of 190 kips (LRFD) is calculated from this moment. Both the shear and the moment are reacted through bearing on the concrete in the embedded end of the column. The model is shown in Commentary Figure C-H4.6. The moment is resisted by the couple $C_b = C_f - V_n$.

*Larry S. Muir, P.E.*

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