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Ordinary Moment Frame Truss Systems

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 . 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 configurations, some of which might not be especially

The Commentary language has the implicit as that the column and not the truss will yield. Tho requirement, a frame in which the column yields i the more common condition and likely the simpler c

Expansion Joint References

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." , April.
- Fisher, James M. (2004), 7: , Second Edition, AISC.
- Saunders, C. (2006), "Seismic Joints in Steel Frame Building Construction," , 11(2), 71-75.
- Brady, Matthew D. (2011), "Expansion Joint Considerations for Buildings." , May.

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

Higher-Strength Steels

Yes. The AISC 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 ."

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 .

Further evidence that higher-strength steels are generally permitted is that steels with 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.

Web Openings in Plate Girders