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Steel Moment Frames – History and Evolution

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Steel-frame structures were developed during the 19th century in response to the limitations of masonry-bearing wall structures, which were the common method of commercial construction at the time. These bearing wall structures were limited to about 10 stories high and allowed for only small openings because of the strength of the masonry materials. Since property owners back then wanted the same things that developers and architects want today—the maximum amount of rentable space on their land and plenty of natural light—new solutions were explored.

Steel-frame building construction using rigid frames, or moment frames, was the answer to these demands. While originally intended for use in high-rise construction, the use of steel moment frames has evolved to encompass a large number of residential and light-framed, mixed-use projects because of the flexibility they give designers.

A Brief History

The earliest steel moment frame buildings used riveted connections with angles or T-sections connecting the beam flanges to the columns to create moment connections. The development of new welding technologies throughout the 20th Century and the introduction of high-strength bolts in the 1950s ended the use of riveting in building and bridge construction in the United States. The *Welded Flange* connection became one of the most common moment frame joints, using either a bolted or welded shear tab for vertical loads, and complete joint penetration welds for the beam flange-to-column flange moment connection. *(See Figure 1)*

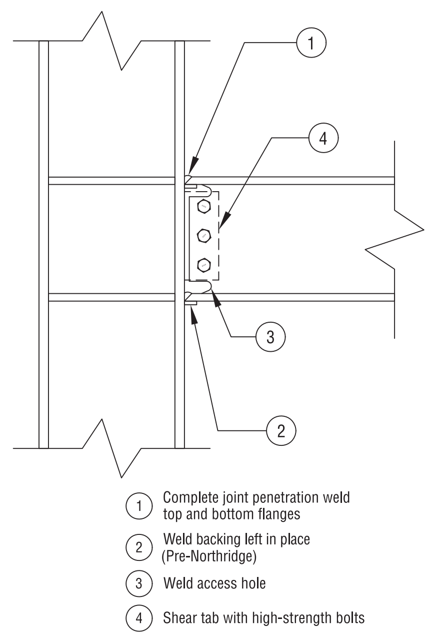

FIGURE 1 - Welded Flange Connection

Steel moment frames are expected to achieve ductility through yielding beams or columns, and the connections must be capable of remaining intact through several cycles of inelastic rotation due to seismic loading. The 1994 Northridge Earthquake demonstrated that the standard connection (shown in Figure 1) did not perform as expected, in many cases fracturing at low levels of plastic deformation. Failures in the moment frame connections were attributed to multiple causes and are well documented, but are beyond the scope of this article to discuss. In mid-1994, the SAC Joint Venture was established, bringing together SEAOC, ATC and CUREE. Funded by FEMA, SAC was tasked with developing new design recommendations for welded steel moment frames. Several documents were published to address the design of moment frames in new structures, repair and evaluation of existing structures, and quality control guidelines.

The period after the Northridge Earthquake held some uncertainty for structural engineers. Many new design requirements were developed in a relatively short time frame and were not uniformly adopted by building jurisdictions. Fortunately, the recommendations for design and quality control contained in the numerous FEMA publications have been incorporated into the current 2005 AISC Seismic Design Provisions. In addition, the American Welding Society has published AWS D1.8, a supplement to AWS D1.1, that addresses welding requirements for high seismic applications. With these two documents, engineers now have clear seismic design requirements in one place.

So Moment Frame Design is Simple Now?

The code requirements are clearly given in the AISC and AWS specifications, but the application of these design requirements remains a difficult task. This can be especially true for engineers who specialize in light-frame construction, where structural steel is sometimes required to resist high lateral loads in a small space. While the actual design and detailing of a moment frame may only take a few hours to a day's work, this is only the first stage of the process. In addition to designing the foundation anchorage, the engineer will need to produce steel and welding specifications, and also review steel shop drawings and welding procedure specifications. A steel sub-contractor will need to install the frame, and the general contractor will need to coordinate between the iron workers and the framers to make sure everything fits together.

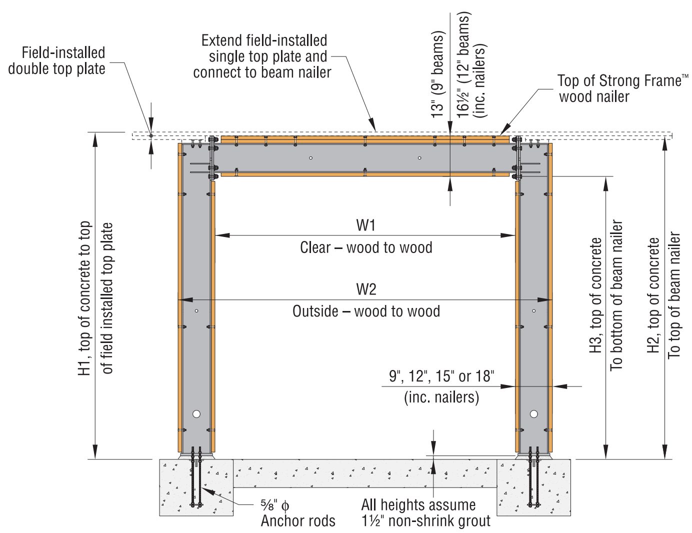

FIGURE 2 - Simpson Strong-Tie® Strong Frame™ moment frame assembly elevation

To save all this additional design work for the engineer and coordination effort for the general contractor, several manufacturers are now selling pre-fabricated moment frames that are geared specifically toward light-frame construction *(See Figure 2)*. The use of pre-fabricated, load-rated components in building designs has been evolving for the past decade, primarily with narrow shear panels that are tested to exceed code height-to-width ratios. The introduction of moment frames to resist higher loads in light-framed structures is the logical progression for pre-manufactured products. The specification of pre-manufactured moment frames can be simplified to selecting a frame model from a catalog that meets the vertical and lateral loading requirements. This is a tremendous benefit for engineers who do not have the time or budget to design a custom frame. However, there are still important design considerations that engineers must be aware of when using a moment frame (pre-manufactured or not) in light-frame construction.

All Moment Frames are Not Created Equal

Dylan Richard's article "Steel Moment Frames 101" (*Structural Engineer Magazine*, June 2008) addressed the differences in Ordinary Moment Frames (OMF), Intermediate Moment Frames (IMF) and Special Moment Frames (SMF), however, it is worth repeating the difference in the design requirements between OMF and SMF when mixing different lateral-force resisting systems.

First, Response Modification Coefficients (R-values), Deflection Amplification Factors (C_d), and System Overstrength Factors (Ω_0) used for seismic design shall be reviewed. ASCE 7-05 and other model building codes acknowledge that structures will be loaded beyond their elastic range during seismic events. Damping and ductile yielding make it unnecessary to design for the full inelastic design force, so the code divides the seismic response by the R-factor to get a lower elastic design force or base shear. Higher R-factors represent more ductile systems and, therefore, yield a lower seismic design force. Deflections must be amplified by the Deflection Amplification Factor, C_d, to obtain the expected inelastic deflections. Similarly, the System Overstrength Factor is another amplification factor that is applied to the elastic design forces for specific elements where it is necessary to prevent a brittle failure in the system.

FIGURE 3 - Select Seismic Design Coefficients from ASCE 7-05 Table 12.2-1

Seismic Force Resisting System	Response Modification Coefficient, R	System Overstrength Factor, Ω_0	Deflection Amplification Factor, C_d
Light-framed walls sheathed with wood structural panels rated for shear	6.5	3	4
Special Steel Moment Frames (SMF)	8	3	5.5
Ordinary Steel Moment Frames (OMF)	3.5	3	3
Cantilever Column System (detailed to meet SMF requirements)	2.5	1.25	2.5
Cantilever Column System (detailed to meet OMF requirements)	1.25	1.25	1.25

A properly detailed SMF is among the most ductile lateral-force resisting systems. This ductility is recognized in ASCE 7 by the relatively high R-value of 8, which yields lower design forces, smaller foundation forces and reduced diaphragm forces in the system. However, the lower design forces for SMF come with a relatively high Deflection Amplification Factor, C_d, which can result in frames that need to be much heavier to meet drift limits than if just designed for strength. In addition, the AISC Seismic Provisions have special beam bracing requirements which are not simple to satisfy, especially in light-frame construction. The beam bracing and the additional steel weight required to meet drift requirements for an SMF in light frame can outweigh the potential cost savings in the foundation and diaphragms.

OMF is a less ductile system than SMF, being described in the 2005 AISC Seismic Provisions as a system *expected to withstand minimal inelastic deformations in their members and connections when subjected to forces resulting from the design earthquake.* ASCE 7 addresses the lower ductility of OMF by assigning it an R-factor of 3.5, resulting in an increased seismic design force compared to SMF. OMF is allowed for use in high seismic regions, but the code places some limits on allowable structure heights and building weights in Seismic Design Categories D, E, and F. The AISC Seismic Provisions do not have OMF beam bracing requirements beyond what is required by the AISC Specification, which allows engineers to design beams as unbraced.

Combination of Systems

While steel moment frames are generally less cost effective than shearwall or braced-frame systems, the open space and design flexibility they provide continue to drive their demand in modern structures. This holds true in light-frame construction as well. Moment frames can accommodate the demand for more windows and grander, taller entryways in custom single-family homes that simply would not work with code aspect ratio shearwalls. Steel moment frames also are needed for mixed-use developments where several stories of light-frame housing are constructed over retail space and require open storefronts and open floor plans.

ASCE 7-05 addresses combinations of lateral systems that commonly occur in many designs. Sections 12.2.3.1 and 12.2.3.2 provide direction for dealing with vertical and horizontal combinations of lateral force resisting systems, but each of these sections requires interpretation and judgment when establishing design loading requirements.

For both vertical and horizontal combinations in the same direction, the code is clear that the lowest R factor for calculating seismic loading is used. For example, for structural panel shearwalls (R=6.5, C_d=4.0) and ordinary moment frames (R=3.5, C_d=3.0), the design shear would be based on the R=3.5 in that direction. However, in detached 1- and 2-family dwellings of light-frame construction two-stories or less, the respective R-values along each wall line can be used, so an R=6.5 along the shear wall line and an R=3.5 along the OMF line are permitted. What becomes less clear is what to do for the deflection amplification, C_d. The language in 12.2.3.1 and 12.2.3.2 is somewhat unclear and could be interpreted in two ways:

1) The largest C_d and Ω_0 values of all the individual structural systems shall be used.
2) The C_d and Ω_0 values shall correspond to the least R-value of all the individual structural systems.

While the first interpretation is conservative, the second interpretation is more appropriate since, fundamentally, as the design R-factor goes down, so too does the ratio between the calculated elastic design displacement and the expected inelastic displacement, which the code refers to as C_d. Therefore, the C_d factor should always be linked to the R-factor used for calculation of the design forces.

Beam Bracing

Special Moment Frames are expected to withstand significant inelastic deformation during a design earthquake. To preclude undesirable beam buckling failure modes that may occur during the formation of plastic hinges in the beam, Section 9.8 of the AISC Seismic Provisions has the following requirement for SMF: *Both flanges of beams shall be laterally braced, with a maximum spacing of $L_b = 0.086 r_y E/F_y$. Braces shall meet the provisions of Equations A-6-7 and A-6-8 of Appendix 6 of the Specification….*

This section goes on to require bracing at concentrated forces, changes in cross section or other locations where plastic hinges may occur. It also has higher brace strength requirements for bracing adjacent to plastic hinges. These requirements are summarized in *Figure 4*.

FIGURE 4 - AISC Section 9.8 SMF Bracing Requirements

$$P_u = \frac{0.06\, M_u}{h_o} \quad \textit{(Lateral bracing strength at plastic hinges)}$$

$$P_u = \frac{0.02\, M_r C_d}{h_o} \quad \textit{(A – 6 – 7, Strength at bracing)}$$

$$\beta_{br} = \frac{1}{\phi}\left(\frac{10\, M_r C_d}{L_b h_o}\right) \quad \textit{(A – 6 – 8, Stiffness at bracing)}$$

In structural steel buildings, additional steel beams connected to full-depth shear tabs with slip-critical bolts have little difficulty in satisfying SMF bracing strength and stiffness requirements. *(See Figure 5)* However, meeting the code prescribed bracing requirements is far more problematic when installing SMF in light-framed construction. There are deflections in the brace due to oversized holes in the wood, vertical deflection of the floor beam and horizontal deflection of the floor diaphragm. Each of these sources of deflection added in series make it unlikely to achieve the minimum bracing stiffness mandated by the AISC for an SMF. *(See Figure 6)*

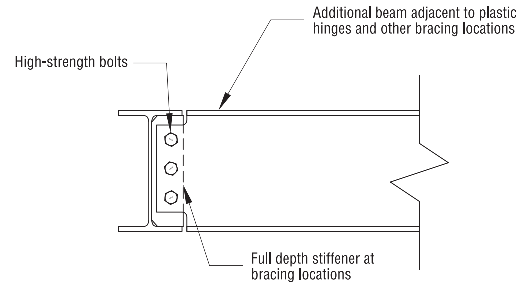

FIGURE 5 - Beam Bracing in Steel Construction

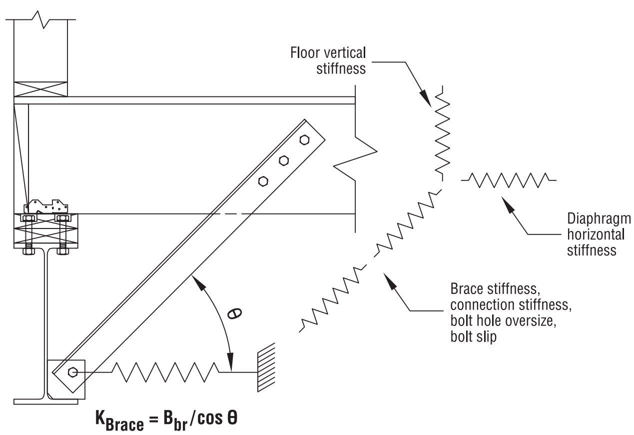

FIGURE 6 - Realities of Beam Bracing in Light-Framed Construction

The challenges of meeting the code beam bracing requirements should be carefully considered when specifying SMFs in light-framed construction for seismic designs. While the lower R-factor for OMF results in higher design forces that may increase costs due to a larger foundation or higher diaphragm forces, the benefit is that OMF can be designed without beam bracing. Engineers should weigh the added foundation and diaphragm costs associated with OMFs against the costs of trying to provide the SMFs' beam bracing that is required by the AISC Seismic Provisions.

Conclusion

Moment frames provide tremendous flexibility in meeting building lateral-load demands and are the lateral systems of choice when design constraints require a small structural footprint to accommodate large openings. Traditionally, moment frames have been time intensive to design and labor intensive to install. The introduction of pre-manufactured, load-rated moment frames offers designers and contractors very effective alternatives to site-built frames, especially for use in light-frame construction. However, while a manufacturer has taken care of the frame design and fabrication, the engineer-of-record must address several key issues to meet the code requirements. These include seismic force calculations using the appropriate design coefficients for the combination of systems used, chord and collector design as well as providing the required beam bracing if SMF is used.

References:

ASCE 7-05 Minimum Design Loads for Buildings and Other Structures
2005 Manual of Steel Construction (ANSI/AISC 360-05)
2005 AISC Seismic Provisions (ANSI/AISC 341-05)
FEMA 267, 350, 351, 352, 353, 354