Steel Special Moment Frames: A Historic Perspective

By Scott M. Adan, Ph.D., P.E., S.E. and Ronald O. Hamburger, S.E., SECB

Many modern buildings use steel special moment frames for their seismic lateral force-resisting system. A moment frame is comprised of a rectangular system of rigidly connected columns and beams that resist moment and shear forces developed during earthquake ground shaking. The building code considers the system extremely ductile and assigns it the highest allowable response modification coefficient. It is one of only a few systems permitted in Seismic Design Categories D, E and F for buildings exceeding 160 feet in height. With the absence of diagonal braces or structural walls, the system facilitates architectural versatility for interior space layout and aesthetic exterior expression. Because earthquake motions can induce multiple inelastic displacement cycles, special proportioning and qualification requirements are essential for robust moment frame performance. The numerous interrelated code provisions that address these special requirements are not necessarily arranged in a logical sequence, making their application challenging for all but the most experienced designers.

As part of its support for the National Earthquake Hazards Reduction Program (NEHRP), the National Institute of Standards and Technology (NIST) is developing a series of technical briefs to assist in improving seismic design and construction quality. Technical Brief No. 2, *Seismic Design of Steel Special Moment Frames: A Guide for Practicing Engineers*, addresses the design, specification, and construction of steel special moment frames. This article, the first of two, contains excerpts from the brief summarizing the development of the steel special moment frame.

Moment Frame Historic Development

Although the steel special moment frame is a relatively recent development in building codes, steel frames have been in use for more than one hundred years, dating to the earliest use of structural steel in building construction. Steel building construction with the frame carrying the vertical loads initiated in Chicago in the 1880s. One of the first of these, the Home Insurance Building in Chicago, a 10-story structure constructed in 1884 with a height of 138 feet, is often credited with being the first skyscraper. This, and other multi-story buildings in Chicago, spawned an entire generation of tall buildings constructed with load bearing steel frames supporting concrete floors and non-load bearing unreinforced masonry infill walls at their perimeters. Framing in these early structures typically utilized "H" shapes built up from plate, "L" and "Z" sections. Starting with the Manhattan Building (1889), perimeter framing connections typically incorporated large stiffened triangular gusset plates, joined to the beams and columns with angles and rivets (*Figure 1*). Typically, steel framing was completely encased by masonry, concrete, or a combination of these, to provide fire resistance. Anecdotal evidence suggests that designers of these early moment frame structures neglected the structural contributions of concrete and masonry encasement, and assumed that framing connections had sufficient flexibility to be treated as *pinned* connections for gravity loading and *fixed* connections for lateral loading. Despite these design assumptions, the steel framing in these structures was substantially stiffened and strengthened by composite behavior with their encasements, and exhibited significant fixity at framing connections both for lateral and gravity loadings.

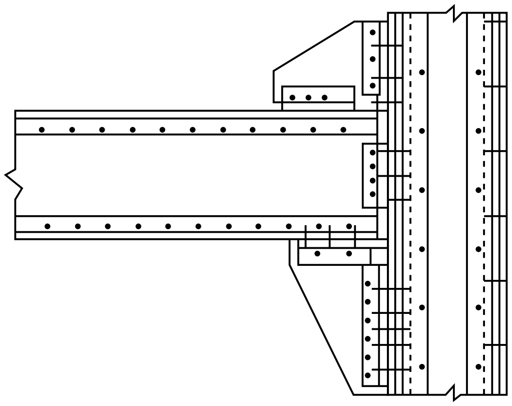

Figure 1: Typical early built-up and riveted moment connection.

This basic construction style remained popular for high-rise construction through the 1930s. By the early 1900s, rolled "H" shape sections began to see increasing use in place of the built-up sections, in particular for lighter framing. Many very tall structures, including the Empire State Building in New York, for many years the world's tallest structure, are of this construction type.

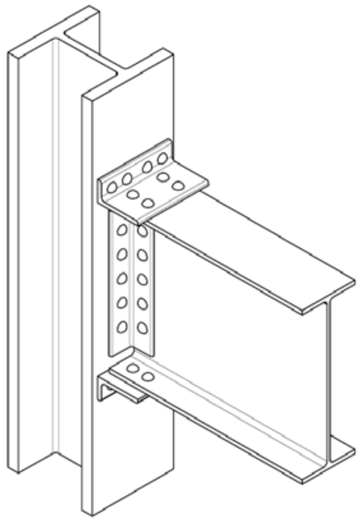

Figure 2: Riveted, unstiffened seat angle connection.

Following WWII, it became uneconomical to construct perimeter walls out of infill unreinforced masonry, particularly for tall buildings, and more modern glass and aluminum curtain wall systems were adopted as part of the new modernist architectural style. The larger windows possible with these new curtain wall systems made the large gusseted framing connections undesirable, and engineers began to design connections using unstiffened angles or split tees to connect top and bottom beam flanges to columns (*Figure 2, page 14*). In the 1950s, as welding was introduced into building construction, the angles and split tees were replaced by flange plates, shop welded to the column flanges, then riveted to the beam flanges. By the 1960s, riveting had become uneconomical and was replaced by high strength bolting. Finally, in the early 1970s, engineers began to use the connection type known today as the welded unreinforced flange – bolted web (WUF-B) (*Figure 3, page 14*) incorporating field-welded, complete joint penetration (CJP) groove welds to join beam flanges to columns, and shop-welded, field bolted shear plates joining beam webs to columns.

Almost from their inception as a means of building construction, engineers began to observe that steel moment-frames seemed to exhibit superior performance in earthquakes. More than 20 such structures were subjected to and survived the great 1906 San Francisco earthquake and the fires that followed it, while few other buildings in the central commercial district of San Francisco remained

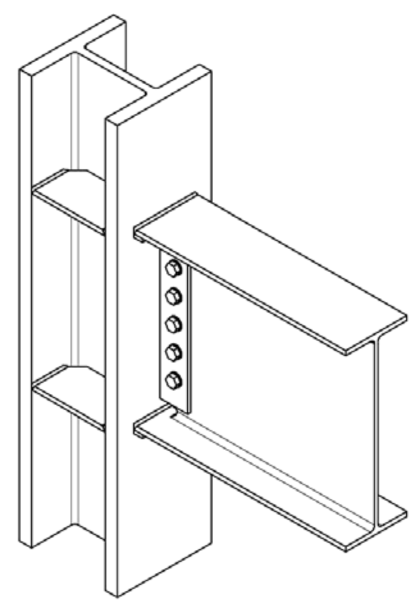

Figure 3: Welded unreinforced flange – bolted web (WUF-B) connection popular from 1970 to1994.

standing. Many of these steel frame buildings are still in service today. For nearly 90 years, engineers continued to observe apparent superior performance of these structures, building the reputation that they had superior earthquake-resisting capability. It is worth noting that much of the seismic and fire resistance possessed by these structures was a result of the composite interaction of the steel framing with the encasing masonry and concrete. Modern steel structures typically do not have the benefit of these features.

As a result of the apparent superior performance of these structures, building codes of the 1960s adopted preferential design criteria for steel moment frames. Under these codes, buildings having complete vertical load-carrying space frames as their lateral force resisting system could be designed for two thirds of the seismic forces specified for braced frames and half the forces specified for bearing wall structures. Further, these codes required such frames in buildings exceeding 240 feet in height.

In the 1960s and 1970s, researchers began to perform cyclic laboratory testing of steel moment framing. The researchers determined that some control on the proportioning and detailing of these structures was necessary to obtain superior inelastic behavior in strong earthquakes. Slowly, throughout the 1970s and 1980s, the building codes began to adopt the recommendations of these researchers and require special design, configuration, and detailing of steel moment frames used for seismic resistance in regions of high seismic risk. Frames conforming to these design criteria were first designated as Ductile Moment Resisting Space Frames, and then finally, in 1988, as Special Moment-Resisting Space Frames. The term "special" was adopted, both because special criteria applied to the design of these structures and also because special, superior behavior was anticipated of them in strong earthquakes.

Initially, the special design criteria were limited to a requirement that connections be capable of developing the strength of the connected members, with the WUF-B connection identified as a deemed-to-comply standard. Later, requirements were introduced to provide for strong-column/weak-beam behavior, balance of the shear strength of panel zones with beam flexural capacity, and addition of section compactness and lateral bracing criteria. Building codes of this era required the use of ductile moment-resisting space frames in all structures exceeding 240 feet in height in zones of high risk of experiencing strong ground motion. As a result, nearly every tall building constructed in the western U.S. in this era was of steel moment-frame construction. Such structures designed in the 1960s and 1970s tended to employ moment-resisting connections at every beam-column joint, providing great redundancy and distribution of lateral force resistance. However, by the 1980s engineers had begun to economize their designs and minimize expensive field welding by using fewer bays of moment-resisting framing that employed heavier beams and columns, resulting in less redundant structures with more concentrated lateral force resistance. In extreme cases, some tall structures were provided with only a single bay of moment-resisting framing on each side of the building.

Following the 1994 Northridge earthquake, engineers were surprised to discover that a number of modern special moment-resisting frame structures had experienced unanticipated brittle fracturing of their welded beam-column connections (*Figure 4*). Similar damage occurred one year later, in the 1995 Kobe, Japan earthquake. Following these discoveries, a consortium of professional associations and researchers, known as the SAC Joint Venture, engaged in a federally funded, multi-year program of research and development to determine the causes of this unanticipated behavior and to develop recommendations for more robust moment frame construction. Conducted at a cost of $12 million over eight years, the SAC research determined the fractures were a result of the basic connection geometry, lack of control of base material properties, the use of weld filler metals with inherent low toughness, uncontrolled deposition rates, inadequate quality control and other factors. The resulting research is the basis for the current steel special moment frame code design requirements.▪

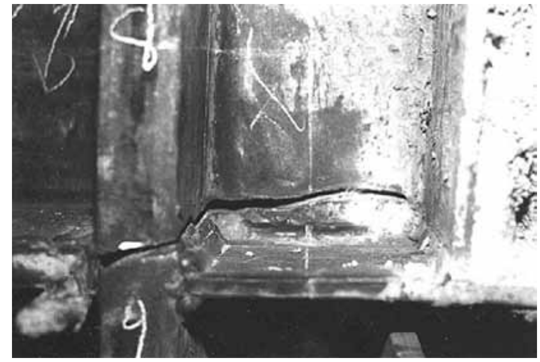

Figure 4: Fracturing of a W14 column at the welded beam-to-column connection. Courtesy of the SAC Joint Venture.

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Further historic information is contained in the NEHRP Technical Brief No. 2. The brief also provides information on the expected earthquake performance of the system in general and outlines applicable building code design criteria. The intent of the brief is to emphasis code requirements and accepted approaches to their implementation. It provides background information and illustrations to help understand the requirements. The brief was developed by the NEHRP Consultants Joint Venture (a partnership of the Applied Technology Council and Consortium of Universities for Research in Earthquake Engineering), under Contract SB134107CQ0019, Earthquake Structural and Engineering Research, issued by the National Institute of Standards and Technology. It is available as a free download at **www.nehrp.gov/pdf/nistgcr9-917-3.pdf**. The contributions of brief coauthors, Helmut Krawinkler and James O. Malley are gratefully acknowledged.