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NEHRP Seismic Design Technical Brief No. 2



Seismic Design of Steel Special Moment Frames:

A Guide for Practicing Engineers

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NEHRP Seismic Design Technical Briefs

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Seismic Design of Steel Special Moment Frames: A Guide for Practicing Engineers

Prepared for
*U.S. Department of Commerce
Building and Fire Research Laboratory
National Institute of Standards and Technology
Gaithersburg, MD 20899-8600*

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Cover photo – Steel special moment frame under construction.

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1. Introduction

Structural steel special moment frames often are used as part of the seismic force-resisting systems in buildings designed to resist earthquakes with substantial inelastic energy dissipation. They are one of a few select systems that U.S. building codes permit without restriction in buildings exceeding 160 ft in height, even in the most critical occupancies and in areas mapped as having the highest ground motions. Beams, columns, and beam-column connections in steel special moment frames are proportioned and detailed to resist flexural, axial, and shearing actions that result as a building sways through multiple inelastic displacement cycles during strong earthquake ground shaking. Special proportioning and detailing requirements are therefore essential in resisting strong earthquake shaking with substantial inelastic behavior. These moment-resisting frames are called *Special Moment Frames* because of these additional requirements, which improve the inelastic response characteristics of these frames in comparison with less stringently detailed *Intermediate* and *Ordinary Moment Frames*.

Design requirements for steel special moment frames are contained in a series of standards. ASCE/SEI 7-05, *Minimum Design Loads for Buildings and Other Structures* (ASCE 2006), referred hereafter as ASCE 7, sets the basic loading criteria for steel special moment frames together with associated lateral drift limits. ANSI/AISC 341-05, *Seismic Provisions for Structural Steel Buildings* (AISC 2005a) provides detailed design requirements relating to materials, framing members (beams, columns, and beam-column joints), connections, and construction quality assurance and quality control. In addition, AISC 341 presents requirements for columns that are not designated as part of the seismic force-resisting system. The numerous interrelated requirements for steel special moment frames are covered in several sections of AISC 341, with the primary requirements covered in Section 9 of Part I of the document. Section 9 of AISC 341 references ANSI/AISC 358-05, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications including Supplement No.1* (AISC 2005b), which is written to facilitate and standardize the design of steel special moment frame connections to allow their use without the need for project-specific testing. A series of different moment connection details is presented in AISC 358-05, and additional connections are being added for the 2010 edition of this document.

Sidebars in the guide

Sidebars are used in this Guide to illustrate key points, to highlight construction issues, and to provide additional guidance on good practices and open issues in Steel Special Moment Frame design.

Both AISC 341 and 358 are applied in conjunction with the ANSI/AISC 360-05, *Specification for Structural Steel Buildings* (AISC 2005c), and AISC 303-05, *Code of Standard Practice for Steel Buildings and Bridges* (AISC 2005d). AISC 360 is the main AISC specification that provides the design and detailing requirements for all steel buildings. In addition to these standards, American Welding Society (AWS) standards AWS D1.1 *Structural Welding Code* (AWS 2004) and AWS D1.8 *Structural Welding Code Seismic* (AWS 2005) present requirements for welding and fabrication that pertain to steel special moment frames. The 2005 edition of AISC 341 duplicated many of the requirements in AWS D1.8. However, the 2010 edition will refer to AWS D1.8 for these requirements. Another useful document is the AISC *Seismic Design Manual* (AISC 2006), which presents useful design aids and examples for moment frames and other steel seismic force-resisting systems.

This Guide is written for practicing structural engineers to assist in their understanding and application of the ASCE 7, AISC 341, and AISC 358 documents in steel special moment frame design. The material is presented in a sequence that practicing engineers have found useful, with historic and general principles for seismic design discussed first, followed by system-specific analysis and design requirements. Although this Guide is intended especially for the practicing structural engineer, it will also be useful for building officials, educators, and students.

This Guide follows the requirements of the 2005 editions of AISC 341 and 358, along with the pertinent design load requirements specified in ASCE 7. AISC 341 primarily addresses the seismic design of systems in Seismic Design Categories D, E, and F, as defined in ASCE 7. The International Building Code, or IBC, (ICC 2006), which is the code generally adopted throughout the United States, refers to ASCE 7 for the determination of seismic loads. AISC 341 was developed in conjunction with ASCE 7, so the documents are well coordinated regarding terminology, system definition and application limitations, and other issues.

The main body of text in this Guide emphasizes code requirements and accepted approaches to their implementation. It includes background information and sketches to illustrate the requirements. Additional guidance is presented in sidebars appearing alongside the main text. Sections 3 through 6 present analysis, behavior, proportioning, and detailing requirements for steel special moment frames and other portions of the building that interact with them. Section 7 presents a discussion of detailing and constructability issues to highlight unique features of steel special moment frame construction. Cited references, notation and abbreviations, and credits are in Sections 8, 9, and 10.

AISC 341 2005 versus 2010

AISC 341-05 and AISC 358-05 are currently the documents referenced for seismic design of steel special moment frame structures in ASCE 7 and the 2006 IBC. An updated version of AISC 341 is presently under development with planned publication in 2010. Most of the technical requirements for steel special moment frame design in the 2010 edition of AISC 341 will be essentially the same as in the 2005 edition. The document is being re-organized to be more consistent with AISC 360, and to more integrally incorporate seismic systems of composite construction into the document. A planned supplement to AISC 358-05 is scheduled for publication in 2009. This supplement will include prequalified connections added to those contained in the original publication.

Code Requirements versus Guide Recommendations

Building codes present minimum requirements for design and construction of buildings and are legal requirements where adopted by the authority having jurisdiction. Thus, where adopted, AISC 341, 358 and 360, in conjunction with ASCE 7, must as a minimum be followed. This Guide is written mainly to clarify requirements of the building code and these referenced standards, but it also presents other recommendations for good design and construction practices that may not be specifically required by the codes or standards. The Guide clearly differentiates between building code requirements and other recommendations.

2. The Use of Special Moment Frames

2.1 Historic Development

Although the concept of a steel special moment frame is a relatively recent development in the building codes, steel moment 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 with the Home Insurance Building in Chicago, a 10-story structure constructed in 1884 with a height of 138 ft, often credited with being the first skyscraper (**Figure 2-1**). This and other tall 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 plates, and “L” and “Z” sections. Starting with the Manhattan Building (1889), perimeter framing connections usually incorporated large stiffened triangular gusset plates, joined to the beams and columns with angles and rivets (**Figure 2-2**). 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 further assumed that framing connections had sufficient flexibility to be treated as “pinned” connections for gravity loading and “fixed” connections for lateral loading. Despite these 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 for both lateral and gravity loadings.



Figure 2-1 - The Home Insurance Building – Chicago, IL, 1885, an early skyscraper.

This basic construction style remained popular for high-rise construction through the 1930s, though 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 New York’s Empire State Building, for many years the world’s tallest structure, are of this construction type.

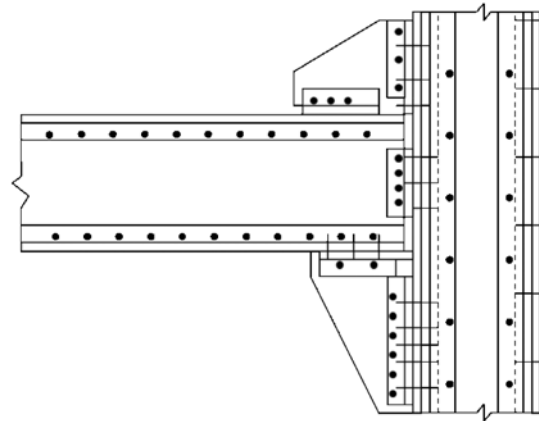


Figure 2-2 - Typical early moment connection, consisting of heavy triangular gusset plates, angles, and rivets connecting built-up columns and beams.

Following World War II, 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 large gusseted framing connections undesirable, and engineers began to design connections without gussets, using angles or split tees to connect top and bottom beam flanges to columns (**Figure 2-3**).

In the 1950s, as welding was introduced into building construction, the angles and split tees were replaced by flange plates that were 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 (**Figure 2-4**), incorporating field-welded, complete joint penetration groove welds to join beam flanges to columns, with 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 1906 San Francisco earthquake and the fires that followed it while few

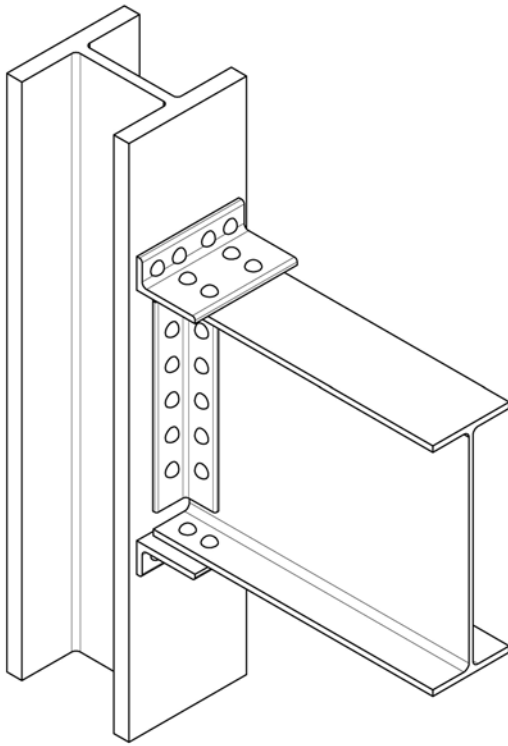


Figure 2-3 - Riveted, unstiffened seat angle connection.

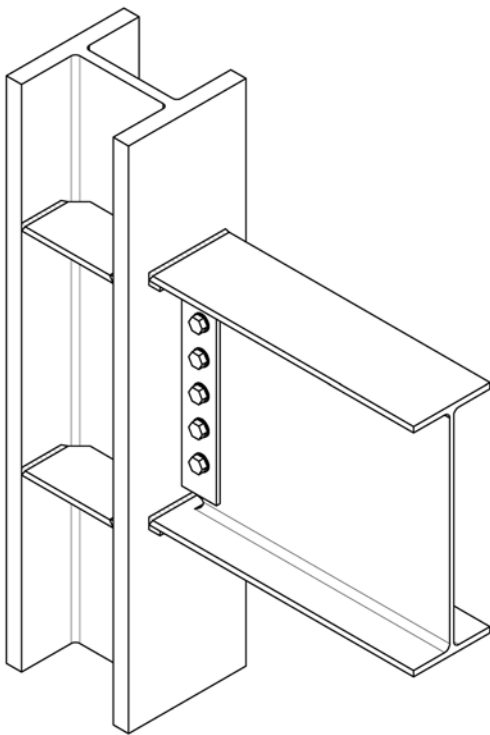


Figure 2-4 - Welded unreinforced flange – bolted web connection popularly used in the era 1970-1994.

other buildings in the central commercial district of San Francisco remained standing (**Figure 2-5**). Many of these steel frame buildings are still in service today. For nearly 90 years, as additional earthquakes shook steel structures with little apparent damage, a reputation of superior earthquake-resisting capability was created. 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, with lightweight fireproofing applied to steel members, do not have the benefit of this composite behavior.

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 systems 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 moment frames in buildings exceeding 160 ft in height.

In the 1960s and 1970s, Professor Egor Popov at the University of California at Berkeley and other researchers began to perform cyclic laboratory testing of steel moment framing and discovered 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 these researchers' recommendations 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 the 1988 Uniform Building Code, as Special Moment-Resisting Space Frames, which were assigned the highest response modification factor, R_w . The term "special" was adopted, both because special criteria applied to the design of these structures, and also because they were expected to provide special superior performance 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 welded unreinforced flange - bolted web 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 ft in height in regions with a high risk of experiencing strong ground motion, and 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,



Figure 2-5 - Steel frame buildings in downtown San Francisco performed well in the 1906 earthquake.

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 in the Los Angeles area, engineers were surprised to discover that a number of modern special moment-resisting frame structures had experienced brittle fracturing of their welded beam-to-column connections. **Figure 2-6** shows one example of such damage; however, many different types of fractures, the majority initiating where the bottom beam flange joined the column flange, were also discovered. Similar damage occurred one year later, in the 1995 Kobe earthquake in Japan. 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-resisting frame construction. The SAC research, conducted at a cost of \$12 million over eight years, resulted in the basis for the current design provisions for moment-resisting frames contained in AISC 341, AISC 358, and AWS D1.8.



Figure 2-6 - Fracturing of W14 column at welded beam-to-column connection in Northridge earthquake.

1994 Northridge Earthquake and the SAC Steel Project

In the aftermath of the 1994 Northridge, California earthquake, damage to steel special moment frame connections (in the Los Angeles area) spawned concern about the reliability of established design and construction procedures. A number of buildings experienced damage in beam-to-column connections that underwent only moderate inelastic demands. Failures included fractures of the bottom beam flange-to-column flange complete-joint-penetration groove welds, cracks in beam flanges, and cracks through the column section (FEMA 2000). 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. Research conducted by the SAC Joint Venture, published in the FEMA 350, 351, 352, 353, and 355 series of reports, underpins current code requirements for steel special moment frame design.

2.2 Steel Moment Frame Seismic Behavior

Even in regions of very high seismic risk, like California and Alaska, severe earthquakes are rare events, affecting typical building sites at average intervals of hundreds of years. Given this, it is economically impractical to design structures to resist such severe but rare earthquakes without damage. Instead, the building codes have adopted a design philosophy intended to provide safety by avoiding earthquake-induced collapse in severe events, while permitting extensive structural and nonstructural damage.

Inelastic behavior in steel special moment frame structures is intended to be accommodated through the formation of plastic hinges at beam-column joints and column bases. Plastic hinges form through flexural yielding of beams and columns and shear yielding of panel zones.

In addition to the behaviors discussed above, research and common sense suggest that a number of other failure modes should also be considered when designing steel special moment frame structures, some of which have not necessarily been observed in past earthquakes. These modes, associated with frame behavior and not that of other elements such as diaphragms and foundations, include the following criteria:

Beam behavior. It is expected that beams will undergo large inelastic rotations at targeted plastic hinge locations, which might be at the ends of beams, at deliberately weakened portions of the beams with reduced beam section designs, or within the beam span if large gravity moments are present. Failure modes can include excessive local buckling (**Figure 2-7**) and lateral-torsional buckling. Each mode by itself, or the combination of both, leads to a continuous decrease in strength and stiffness and is very costly to repair after an earthquake.

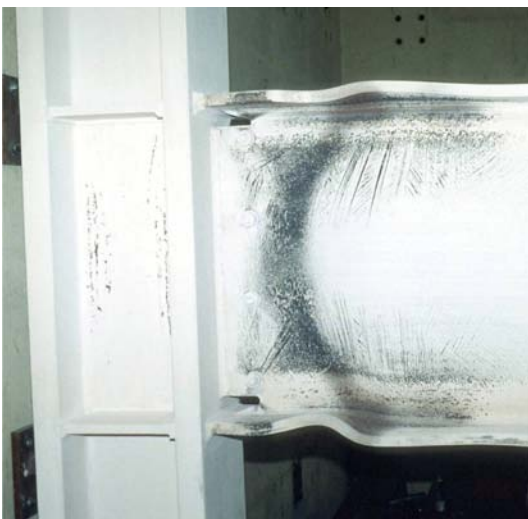


Figure 2-7 - Typical local buckling of beam flanges and web in zone of plastic hinging at high levels of inelastic rotation.

Beam-to-column connections. The connections must be capable of transferring the moment and shear forces that can be developed in the beam to the column. As a result of material overstrength, and strain hardening effects, these moment and shear forces can be substantially larger than the design forces obtained from analysis, using code-specified loads. Depending on the type of connection used, this might trigger any of the following failure modes:

- Fracture in or around welds
- Fracture in highly strained base material
- Fractures at weld access holes
- Net section fracture at bolt holes, shearing and tensile failure of bolts, bolt bearing and block shear failures

Joint panel zone behavior. The joint panel zone resists significant shear forces from the beams framing into a column. Acting as part of the column, it can also be subjected to significant compressive stresses. Potential failure modes include shear buckling and, if doubler plates are used to reinforce the panel zone, fracture at welds. Failure modes associated with the direct transfer of forces from the beam flange to the column can include column flange bending, web crippling, and web buckling.

Column behavior. The intention is to keep inelastic deformations out of most columns to minimize detrimental effects of high axial loads on bending behavior and potential formation of single-story mechanisms (**Figure 2-8**). Regardless, many columns designed in accordance with the strong-column/weak-beam requirements in AISC 341, §9.6 might experience significant inelastic rotations in a major seismic event. Therefore, excessive local buckling and lateral-torsional buckling are potential failure modes, in addition to basic flexural buckling of columns.

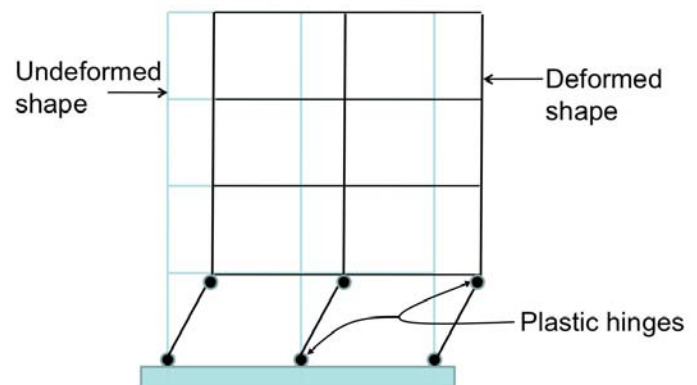


Figure 2-8 - Formation of a single story frame mechanism, also termed a "weak story" mechanism.

Column splices. Failure modes at column splices are similar to those enumerated for beam-to-column connections. Failure of column splices will not only reduce or eliminate bending and tension resistance, it will also reduce or eliminate the ability of the column to transfer shear forces. Since gravity load-carrying columns in steel special moment frame structures can experience

substantial lateral deformations and related seismic forces, column splices in these columns also can be subject to such failures.

Column bases. Failure modes depend on the connection between the column and the foundation. They include anchorage stretching or pull-out, fracture in base plates or in column-to-base plate connections, and/or excessive local and lateral torsional buckling if inelastic deformations are concentrated in the region above the base connection.

Structure P-delta Effects. Amplification of internal forces and lateral displacements, known as the P-delta effect, occurs when a structure is simultaneously subjected to gravity loads and lateral sidesway. This effect reduces frame lateral resistance and stiffness, might cause a negative effective lateral tangent stiffness once a mechanism has formed, and can lead to collapse.

Sidesway Collapse. Frame collapse can occur when the effective story shear due to inertial forces and P-delta effects exceeds the story shear resistance.

Much of this Guide focuses on design principles and analysis checks intended to assure that none of the aforementioned failure modes are likely to occur in an intolerable manner.

2.3 When To Use Steel Special Moment Frames

The principal advantage of moment frame structures is that they do not have structural walls or vertically oriented diagonal braces. They therefore provide architectural freedom in design, permitting open bays and unobstructed view lines. The tradeoff for these benefits is that moment frames can be more costly to construct than braced frame or shear wall structures. The added cost results from the use of heavier sections in the moment-resisting frames, requiring increased steel usage and more labor-intensive connections than is common in braced structures. However, moment frames typically impose smaller forces on foundations than do other structural systems, resulting in somewhat more economical foundation systems.

Once a steel moment frame solution is selected for a project, designers may be able to choose from several types, including special moment frames, intermediate moment frames, ordinary moment frames, and moment frames not specifically designed for seismic resistance.

Moment frames not specifically detailed for seismic resistance have no special detailing criteria and need only comply with the strength and drift limits of ASCE 7 and the design requirements of AISC 360. These frames are not permitted as seismic force-resisting systems in Seismic Design Categories D, E, or F. Ordinary moment frames, designed in accordance

with limited requirements specified by AISC 341, §11 are permitted in light, single-story structures and low-rise residential structures in all Seismic Design Categories, and are permitted without restriction in Seismic Design Categories A, B, and C. Intermediate moment frames, designed to somewhat more restrictive criteria in AISC 341, §10, are permitted for structures up to 35 ft in height in Seismic Design Category D, and for light, single-story structures in Seismic Design Categories E and F. Intermediate moment frames are permitted without restriction in Seismic Design Categories A, B, and C, and are permitted in structures 35 ft or less in height in Seismic Design Category D. Steel special moment frames conforming to the criteria in AISC 341, §9 are permitted without restriction in all Seismic Design Categories, and are required in Seismic Design Categories D, E, and F for most structures exceeding 160 ft in height. For structures that meet certain regularity criteria, the requirement to incorporate steel special moment frames is triggered at a height of 240 ft.

In recent years, many tall buildings in higher Seismic Design Categories have used Dual Systems, in which steel special moment frames capable of providing at least 25 % of the required lateral strength are used in combination with shear walls or braced frames. From frames not detailed for seismic resistance to ordinary moment frames, intermediate moment frames, and steel special moment frames, the seismic provisions of the building code require successively less strength. However the added level of detailing required for the better performing systems, also typically increases construction cost.

2.4 Frame Proportioning

Except for a steel special moment frame used as part of a Dual System, base shear strength is not usually the primary design consideration. The primary factors affecting steel special moment frame member size selection are the need to control design drifts below permissible levels, the need to avoid P-delta instabilities, and the need to proportion structures to comply with the strong-column/weak-beam criteria of AISC 341, §9.6. While many designers find that the use of deep section columns (W24s, W36s, and built-up box sections) is an economical choice that facilitates achievement of both drift control and strong-column/weak-beam requirements, deep wide flange sections, particularly those with lighter weights, are susceptible to undesirable local and lateral-torsional buckling; they should be avoided. It is usually advantageous to limit the widths of bays in moment-resisting frames, as long-span frames tend to be flexible, driving up section sizes required to control drift. Frame spans exceeding 40 ft are rarely practical. However, short bay widths also must be avoided; they can result in inelastic behavior dominated by shear yielding, as opposed to flexural yielding, of beams. Also, most connections prequalified for use in steel special moment frames have limits on beam span to depth ratio that prevent use of excessively short bays. Bay widths less than 20 ft are rarely economical.

The ability of steel framing members to accommodate large inelastic deformations is in part dependent on section depth and weight. Lighter, shallower sections and their connections that meet AISC 341, §8.2b compactness requirements tend to have larger inelastic deformation capacity than do deep, heavy sections. For this reason, it is desirable to distribute lateral resistance in steel special moment frame structures among many bays of framing, providing high redundancy, and reduced framing sizes. In some cases, these smaller members can offset the cost of the additional framework.

2.5 Strength and Drift Limits

Although stiffness usually controls the proportioning of most steel special moment frame members, strength also must be considered. ASCE 7, §12.2.1, Table 12.2-1 allows design of steel special moment frames using a response modification coefficient, R , of 8. That is, they are allowed to be designed for a base shear equal to one-eighth that obtained from elastic response analysis, so long as this base shear does not fall below minimum levels applicable to all structures. Base shear calculations are frequently controlled by the approximate upper limit period defined in ASCE 7, §12.8.2. Wind loads also must

be checked, and may govern strength requirements, particularly in taller structures. It is not uncommon for seismic loads to govern drift requirements while wind loads govern strength requirements. Regardless of whether gravity, wind, or seismic forces govern, proportioning and detailing provisions for steel special moment frames apply wherever they are used.

Frame stiffness must be sufficient to control lateral drift at each story within specified limits. ASCE 7, §12.1, Table 12.12-1 provides the allowable story drift, Δ_a , as a function of structure type. The redundancy coefficient, ρ , determined in accordance with ASCE 7, §12.3.4.2, also affects the permissible drift. ASCE 7, §12.12.1.1 limits the design story drift, Δ , to Δ_a/ρ .

Regardless of whether Allowable Strength Design or Load and Resistance Factor Design procedures are used to evaluate strength, drift is calculated using strength-level seismic forces, amplified by the deflection coefficient C_d . The code does not specify drift limits for wind loads; however, many designers of tall buildings limit wind-induced drift to enhance occupant comfort during wind storms. In some buildings, it may be desirable to limit design drift in order to protect cladding, stairs, and other nonstructural elements that span vertically from one level to another from damage.

3. Principles for Special Moment Frame Design

The ASCE 7, §12.8 design base shear equations incorporate a seismic response modification coefficient, R , that reflects the degree of inelastic response expected for design-level ground motions, as well as the ductility capacity of the framing system. Steel special moment frames, permitted to be designed using a value of $R = 8$, are expected to sustain multiple cycles of significant inelastic response when subjected to design-level ground motion. However, many steel special moment frame structures have substantial overstrength. This overstrength results from a number of factors, including oversizing of columns to meet strong-column/weak-beam criteria, use of oversize sections to provide sufficient stiffness for drift control, and variability in the strength of the steel material itself. As a result, although the R value of 8 specified by the code would imply initiation of inelastic behavior at shaking with an intensity $1/8$ that of the design earthquake, many steel special moment frame structures will remain elastic for shaking with an intensity as large as $1/3$ that of the design earthquake, or even more intense shaking.

The proportioning and detailing requirements of AISC 341 are intended to provide ductile inelastic response. The primary goals are as follows: (1) achieve a strong-column/weak-beam condition that distributes inelastic response over several stories; (2) avoid P-delta instability under gravity loads and anticipated lateral seismic drifts; and (3) incorporate details that enable ductile flexural response in yielding regions.

3.1 Design a Strong-column / Weak-beam Frame

In order to avoid development of P-delta instability in multi-story structures, it is desirable to achieve a relatively uniform distribution of lateral drift over the structure's height. To achieve this, it is important to avoid early formation of single-story mechanisms in which inelastic response is dominated by formation of plastic hinges at the tops and bottoms of columns within a single story (**Figure 2-8**). When such single story mechanisms form, most of the inelastic portion of a structure's drift will occur within these stories, resulting in very large P-delta effects at those locations. In order to avoid this, building codes require designs intended to promote formation of multi-story sidesway mechanisms dominated by hinging of beams, as opposed to column hinging, like the idealized sidesway mechanism of **Figure 3-1**. These requirements are termed strong-column/weak-beam design.

AISC 341, §9.6 adopts a strong-column/weak-beam design approach that requires that the sum of column flexural strengths at each joint exceed the sum of beam flexural strengths. When determining available column flexural strength, it is important to consider the axial loads that will be simultaneously present

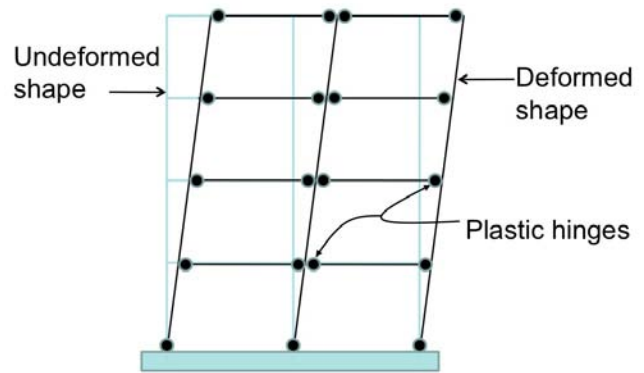


Figure 3-1 - Idealized sidesway mechanism intended for columns with strong-column / weak-beam design.

in the column along with flexural demands. The provisions provide an expression to determine the column-beam strength ratio and acknowledge that the design requirement is not adequate to completely avoid flexural hinging of columns. The provisions require supplemental lateral bracing of beam-column connections, unless it can be shown that the columns will remain elastic. Section 5.4 of this Guide discusses this additional bracing requirement. When the column-beam moment ratio is 2 or greater, AISC 341, §9.7a permits an assumption that columns will remain elastic. Recent research has found that in cases involving larger ground motions, the strong-column/weak-beam provisions presently contained in AISC 341, §9.6 may not be adequate to avoid formation of story mechanisms in all cases. Designers may wish to increase column sizes, beyond the code requirements in order to obtain better performance in severe earthquake events. This also has the advantage of reducing the need to provide costly web stiffener and doubler plates but will increase the total weight of steel used on the project.

3.2 Proportion for Drift

Sizing of beams in steel special moment frames typically is controlled by consideration of drift. As a consequence, the sizes of many columns also are drift-controlled because the strong-column/weak-beam provisions discussed earlier demand larger columns if larger beams are used. Exceptions are end columns in steel special moment frames, which often have high axial load demands and in most cases are controlled by strength design criteria.

ASCE 7, §12.12.1, Table 12.12-1 limits story drift under seismic loading to a fraction of the story height. This design story drift may be determined by multiplying the story drift obtained from the lateral analysis by the factor C_d / I , where the design lateral forces may be determined based on the computed fundamental period of the structure without the upper limit ($C_u T_a$) specified

for determination of required strength. When modal response spectrum analysis is used, forces obtained from the analysis must be scaled such that the total base force is not less than 80% of that obtained using the equivalent lateral force procedures. This requirement does not apply to drifts. Necessary amplifications of story drift due to real and accidental torsion and due to P-delta effects (see Section 3.3) are stated explicitly in ASCE 7, §12.8.7 and are treated equally in modal response spectrum and equivalent lateral force designs. Design for stiffness due to story drift limitations is often an iterative process, because the design lateral forces depend on the computed fundamental period of the structure.

Consideration of minimum base shear in design for drift

The ASCE 7 treatment of minimum design forces for drift determination may lead to much larger stiffness requirements for structures designed in accordance with the equivalent lateral force procedure than the modal response spectrum procedure, if the computed period exceeds the period at which minimum base shear requirements control the design lateral forces. It is expected that this inconsistency will be eliminated in ASCE 7-10, and that for both procedures the use of the minimum base shear (Eq. 12.8-5) for drift design will be removed. However, the modal response spectrum procedure will almost always result in more economical designs for steel special moment frames than the equivalent lateral force procedure provides.

Story drifts are caused by flexural and shear deformations in beams and columns, and by shear deformations in joint panel zones, causing a shear (racking) mode of drift, and by axial deformations in the columns causing a flexural mode of drift.

The contributions to the shear mode of drift vary with configuration, but in general beam bending is the largest contributor while column bending is the smallest. Panel zone shear deformations contribute on the order of 15% to 30% to the total shear mode of drift. Estimates of the contributions to story shear drift can be obtained from the equations presented in Section 4.2.1. ASCE 7, §12.7.3b requires that the contribution of panel zone deformation to story drift be included when checking drift limits. Section 4.2 provides additional discussion on this topic.

The flexural mode of drift becomes important for relatively slender frames with a height-to-width (aspect) ratio of about 1.5 or larger. For symmetrical frames, this portion of the total story drift is approximately equal to the rotation of a cantilevered steel column having a moment of inertia $I = A_c D^2$, where A_c is the area of a single end column in the frame and D is the distance between the frame's end columns. The total story drift is the

sum of shear and flexural mode drifts. If the flexural mode of drift contributes significantly to the story drift, the remedy is to increase the size of the exterior steel special moment frame columns or to provide tighter limits on the shear mode drift, so that shear drift plus flexural drift does not exceed the allowable story drift. For slender steel special moment frames, optimal sizing of members to meet drift requirements can lead to the use of larger beam sections near the frame's mid-height than at lower levels.

3.3 Frame Stability

In a severe earthquake, frame structures have the potential to collapse in a sidesway mode due to P-delta effects. These effects are caused by vertical gravity loads acting on the deformed configuration of the structure. For design purposes, the P-delta effect is assessed in codes by means of elastic and static concepts, even though in reality the response of the structure in a severe earthquake is inelastic and dynamic. The simple P-delta provisions in ASCE 7, §12.8.7 provide some protection against sidesway failures, but do not provide accurate information on the susceptibility of a structure to such failure.

ASCE 7, §12.8.7, Equation 12.8-16 requires explicit consideration of P-delta effects in each story in which the elastic stability coefficient $\theta = (P\Delta) / (VhC_d)$ exceeds 0.1. In this equation Δ is the story drift computed per ASCE 7, §12.8.6, using T_1 limited to $C_u T_a$, which means it contains the deflection amplification factor, C_d , i.e., $\Delta = \Delta_e C_d$. This implies that the elastic stability coefficient can be computed under any level of lateral load, provided that the structure remains elastic and Δ_e and V come from the same lateral load condition. The load P is the total unfactored vertical load, including the load tributary to gravity framing. When computer analysis is performed, these elastic P-delta effects usually can be accounted for automatically in the analysis; however, the user usually must specify that the software perform this calculation.

ASCE 7, §12.8.7, Equation 12.8-17, places an upper limit of $\theta_{max} = 0.5 / (\beta C_d) \leq 0.25$ on the permissible stability coefficient, where β is the ratio of shear demand to shear capacity for the story under consideration. Shear demand is the Load and Resistance Factor Design story design shear force, and shear capacity is the maximum shear force that can be resisted by the story. This shear capacity cannot be defined uniquely because the capacity in one story depends on the load pattern applied to the full structure. An estimate of the story shear capacity can be obtained by dividing the average of the "floor moment" capacities of the two floors bounding the story by the story height. The "floor moment" capacity is the sum of the maximum beam or column moments that can be developed at the intersection of all beam-to-column centerlines at the floor level. For connections at which the strong-column/weak-beam concept is followed, this amounts to the quantity ΣM_{pb}^* employed in AISC 341, §9.6, Equation 9-3, divided by 1.1 to

eliminate the strain hardening effect. For connections with weak columns, the quantity ΣM_{pc}^* from AISC 341, §9.6, Equation 9-3, should be used.

It is not uncommon for the story stability coefficient to exceed 0.1, particularly in regions in which the design spectral values S_{DS} and S_{D1} are relatively small. In such cases, the seismic design forces are small, and steel special moment frames may become very flexible unless wind criteria control member design. It is also not uncommon in such cases that the θ_{max} criterion controls, and P-delta considerations will require an increase in member stiffness. Most computer analysis programs will not check for θ_{max} , so this criterion must be checked manually.

When the stability coefficient, θ , exceeds a value of 0.1, ASCE 7, §12.8 requires evaluation of P-delta effects, either using a first order approach in which computed deflections are amplified by the quantity $1/(1-\theta)$, or by second order analysis in which geometric nonlinearities are explicitly considered. Many structural software packages commonly used by engineers to analyze and design steel structures have the ability to perform these second order analyses. However this software generally does not evaluate whether θ exceeds θ_{max} as required by ASCE 7, §12.8. Therefore, engineers must manually check that this condition is satisfied.

P-delta evaluations should be performed for each frame so that torsional effects, which cause displacement amplification, are considered. AISC 341, §C3, contains a suggested provision, Equation C3-1, that is more stringent in most cases than required by ASCE 7, §12.8.7. However, that commentary is not a binding part of the code requirements.

3.4 Strength Verification

Columns and beams are required to have adequate strength to resist the ASCE 7, §2.3 and 2.4 load combinations, considering axial-flexural interaction effects. In addition, columns are required to have adequate strength to avoid global buckling or tensile fracture under maximum axial forces, and beam-column connections are required to have adequate strength to develop the probable flexural strength of the beams. The provisions of AISC 341 and AISC 360 govern the calculation of design strength for both Allowable Strength Division and Load and Resistance Factor Design procedures.

3.5 Connection Type Selection

Since the 1994 Northridge earthquake, AISC 341, §9.2a has required that steel special moment frame moment connections be demonstrated through qualification testing to be capable of developing at least 0.04 radians of interstory drift without

excessive strength loss, when subjected to the cyclic loading protocol specified in AISC 341, Appendix S. Qualification testing must be conducted on full-size specimens using sections, materials, and fabrication procedures comparable to those to be incorporated in the actual construction. Relatively few laboratories have the capability to perform such tests, and the tests are expensive. If initial connection designs fail the testing, it may be necessary to perform multiple iterations of the design and testing, adding months of delay and hundreds of thousands of dollars of expense to projects. To avoid these difficulties, AISC 341, §9.2b(a) permits the use of prequalified connections. Prequalified connections have been demonstrated by extensive testing and analysis, acceptable to an expert review panel, to be capable of reliable service when used within specified application limits. There are several sources of connection prequalifications, described below.

3.5.1 AISC Prequalified Connections

AISC maintains a Connection Prequalification Review Panel that develops an American National Standards Institute (ANSI)-approved standard, AISC-358 *Prequalified Connections for Special and Intermediate Moment Resisting Frames for Seismic Applications including Supplement No. 1*. AISC 358 presents materials, design, detailing, fabrication, and inspection requirements for a series of prequalified moment connection details. This standard is referenced by AISC 341, and connection prequalifications contained in the standard are acceptable to most building officials. AISC updates and reissues this standard from time to time, as additional research becomes available. The connections in AISC 358 are not interchangeable; there are limits of applicability for each. **Figures 3-2 through 3-6** show the configurations of connection technologies currently included in AISC 358.

3.5.2 Other Prequalified Connections

In addition to AISC 358, several other sources of prequalification exist. The federally funded SAC Joint Venture that performed the post-Northridge earthquake research into steel special moment frame behavior published FEMA 350, *Recommended Design Criteria for Moment Resisting Steel Frames*, which contains a number of connection prequalifications. Many of the FEMA 350 prequalifications have since been updated and adopted into AISC 358. Some have not, either because AISC's Connection Prequalification Review Panel has deemed that there is insufficient research to support prequalification, or has not yet had time to review the connection and include it in AISC 358. FEMA 350, like AISC 358, includes design, materials, fabrication, and inspection criteria for prequalified connections. When both AISC 358 and FEMA 350 have criteria for a particular connection type, the information in AISC 358 should be considered to supersede that in FEMA 350. Some, but not all, building officials will accept FEMA 350 prequalifications. An on-line database of the tests that were performed as part of the FEMA/SAC project can be accessed at www.sacsteel.org/connections/.

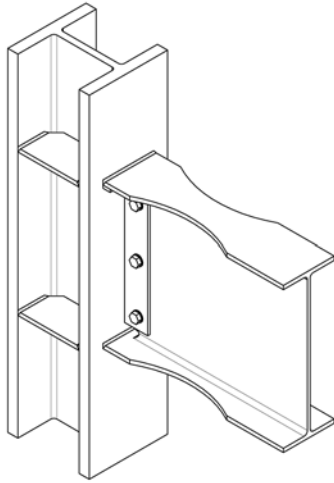


Figure 3-2 - Reduced beam section connection.

Special Conditions

All of the existing prequalification tests have been conducted using specimens in which the beams and columns were within a single plane and in which the beams intersected the columns orthogonally. Prequalifications do not exist presently for connections in which the beams are skewed relative to the axis of the column, are connected at other than orthogonal conditions, or are part of a column subjected to bi-axial frame behavior. For these conditions, the code requires connection-specific qualification. Therefore, it is probably best to avoid these conditions when laying out the structure's seismic force-resisting system.

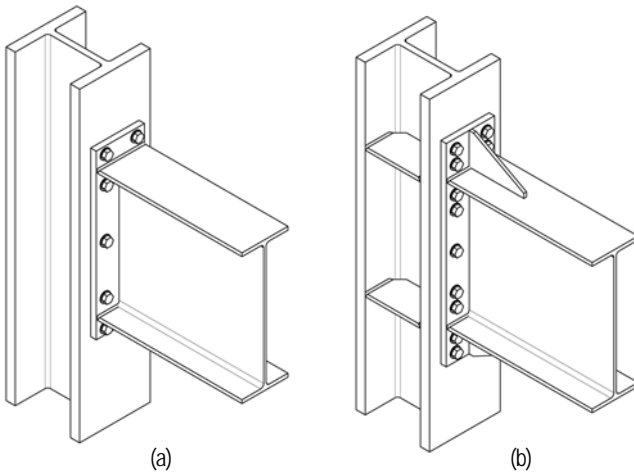


Figure 3-3 - (a) Bolted unstiffened extended end plate and (b) Bolted stiffened extended end plate connections.

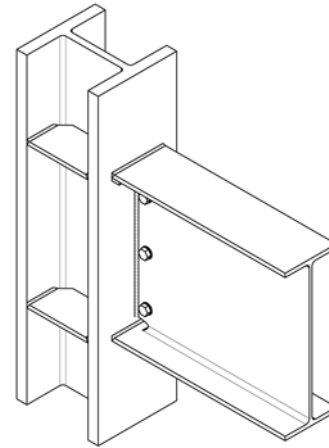


Figure 3-5 - Welded unreinforced flange – welded web connection.

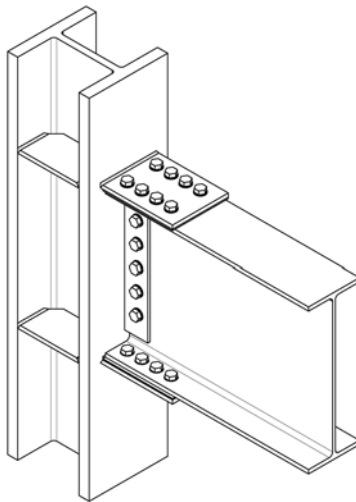


Figure 3-4 - Bolted flange plate connection.

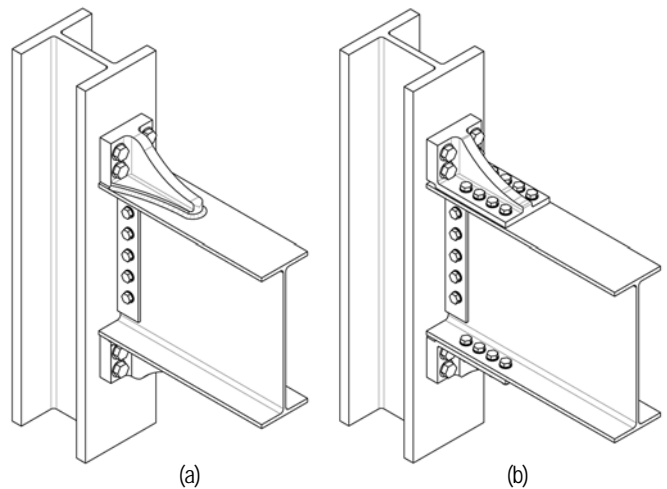


Figure 3-6 - Kaiser bolted bracket connection (a) welded to beam (b) bolted to beam.

In addition, there are several code agencies that operate evaluation services to qualify the use of proprietary products and procedures as meeting the criteria contained in the building code. These evaluation services publish connection prequalifications for proprietary connection technologies in the form of evaluation reports, and building officials typically accept these reports as evidence of code conformance. However, engineers relying on these evaluation reports should be aware that the rigor of review does not always match that performed by AISC's Connection Prequalification Review Panel. Therefore, the performance capability of connections that have been included in these evaluation reports may not match that of connections contained in AISC 358.

Some individual patent holders for proprietary connections maintain their own library of test data and analysis to substantiate the performance capability of their connections. Strictly speaking, these connections are not prequalified. However, some building officials will accept their use, sometimes requiring independent review as a condition of such use.

3.5.3 Project Specific Qualification

In some cases, the prequalifications available in AISC 358, FEMA 350, and evaluation service reports may not be adequate to cover the design conditions for a particular project. One reason this may occur is that the sizes of frame elements selected for a steel special moment frame may fall outside the limits contained within the prequalifications. Another reason this may occur is that presently there are no prequalifications associated with connections to the minor axis of wide-flange columns. If no prequalified connections meet the requirements of a particular design condition, AISC 341, §9.2b (ii) requires project-specific testing. At least two specimens must be tested and must pass the criteria specified in AISC 341, Appendix S. Since the required size of specimens needed to comply with the AISC 341, Appendix S requirements can be quite large, often only universities have the capability to perform such testing. Scheduling use of these facilities can be difficult. Therefore, if project-specific testing will be required, early planning for this effort is recommended. Because of specimen fabrication, shipping, and set-up costs, testing can be expensive. Consideration should be given to using framing configurations that will enable the use of prequalified connections.

3.6 Details for Ductile Behavior

As a highly ductile system, it is expected that steel special moment frames will undergo significant inelastic behavior in numerous members when subjected to severe seismic shaking. The primary source of this inelastic behavior is intended to occur in the form of plastic hinging in the beams, adjacent to the beam-column connections. In a properly configured system, this hinging should occur over multiple stories to spread the total displacement demand and limit the local deformations and

member strains to a level that the members can withstand. In addition to the hinging of beams, inelastic behavior can be expected to occur in beam-column joint panel zones and at column bases.

A number of features are incorporated into steel special moment frame design to achieve the intended ductility level. One primary feature is the level of compactness required of beam and column members. In addition, steel special moment frame members also must be laterally braced for stability. AISC 341, §9.8 prescribes a maximum spacing distance for lateral bracing of steel special moment frame beams and specifies stiffness and strength criteria for this bracing to avoid lateral-torsional buckling. In most applications where the framing supports a concrete floor slab, the lateral bracing is provided for only the bottom beam flange. Lateral bracing of columns at the floor levels is also required. This bracing is especially important for deep column sections that, while efficient for frame stiffness because of their high moment of inertia to weight per ft ratio, are more susceptible to lateral-torsional buckling than stockier W14 column shapes.

As mentioned in previous sections, implementing a strong-column/weak-beam design philosophy is important to good steel special moment frame performance. While it is desirable to avoid column hinging, under very intense shaking, columns will invariably form hinges at the frame base. Frame design should explicitly consider this inelastic demand. Generally, the design of steel special moment frame column bases should be strong enough so that inelastic deformation is limited to a region that can exhibit significant ductility, such as the column member just above the base connection. Another approach, if the steel special moment frame extends to the foundation, is to design and detail anchor bolts to yield as a means of limiting demand on other elements of the connection, or through the formation of yielding in supporting foundation elements. In some cases, engineers may wish to design columns assuming the bases are "pinned." In those cases, it is important to detail the bases to accommodate the large anticipated rotations without failing the anchorage and attachment to foundations.

3.6.1 Seismically Compact Sections

Reliable inelastic deformation requires that width-thickness ratios of compression elements be limited to a range that provides a cross section resistant to local buckling into the inelastic range. AISC 360, §B4 uses the term "compact" for steel cross sections that are expected to be able to achieve the full plastic section capacity. In AISC 341, §8.2b, a higher level of compactness (termed "seismically compact") is required of both steel special moment frame beams and columns. Seismically compact sections are expected to be able to achieve a level of deformation ductility of at least 4. To be seismically compact, AISC 341, §8.2b requires member flanges to be continuously connected to the web(s) and the width-thickness ratios of the compression elements must be less than or equal to those that are resistant to local buckling when stressed into the inelastic

range. Limiting width-thickness ratios for compression elements are provided in AISC 341, §8.2 b, Table I-8-1.

3.6.2 Demand-Critical Welds

AISC 341, §7.3b defines demand-critical welds as those that require increased quality and toughness requirements based upon inelastic strain demand and the consequence of failure. Unless otherwise designated by AISC 358, or as determined in either prequalification or qualification testing, welds designated demand-critical in steel special moment frames are identified specifically in AISC 341, §9.2c as complete – joint penetration groove welds of beam flanges, shear plates, and beam webs to columns. Other complete joint penetration groove welds considered demand-critical by AISC 341, §7.3b include those at a column splice, those joining a column and base plate, and those in built-up members joining a web plate to a flange in the plastic hinge region.

3.6.3 Protected Zones

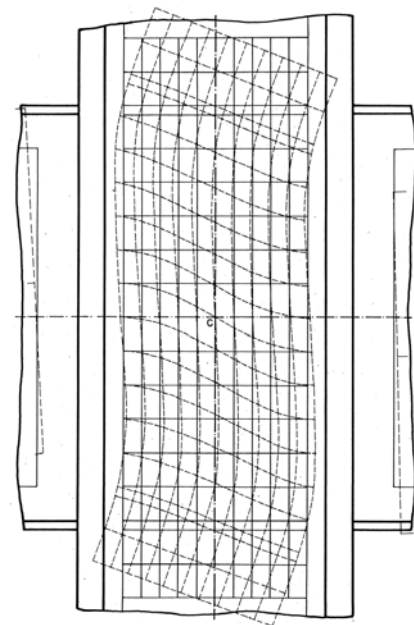
AISC 341, §9.2d requires designation of the region at each end of a steel special moment frame beam subject to inelastic straining as a protected zone. Protected zones must meet the requirements of AISC 341, §7.4. AISC 358 designates the location and extent of protected zones for prequalified connections. For connections not contained in AISC 358, engineers should specify protected zones based on the inelastic behavior exhibited in connection assembly qualification tests. In beams carrying heavy gravity loads, plastic hinging may occur within beam spans remote from connections. When such conditions are anticipated, engineers should designate protected zones in these additional areas of anticipated plastic hinging.

3.6.4 Panel Zone of Beam-to-Column Connections

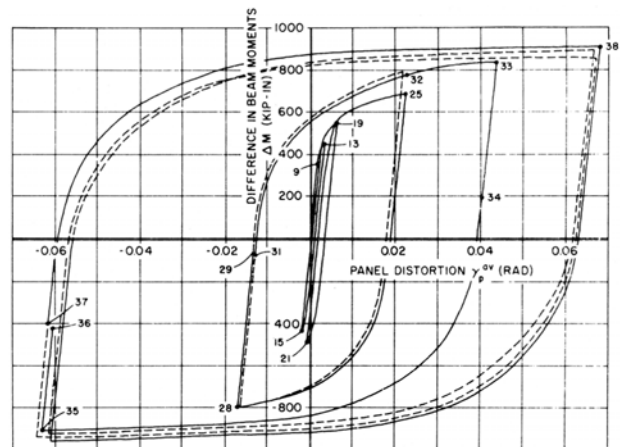
Panel zones experience large shear forces due to the transfer of moments from beams to columns. As these shear forces increase, a panel zone starts to yield at its center. Subsequently, yielding propagates towards the panel zone corners, with deformations of the panel zone as shown (greatly amplified) in **Figure 3-7a**. Tests have shown that shear yielding in panel zones is a very ductile mode of deformation. Panel zones can undergo many

cycles of large inelastic distortions without deterioration in strength, while exhibiting cyclic hardening (**Figure 3-7b**). It is desirable, in most cases, to have panel zones participate in dissipating energy through inelastic deformations and by doing so decrease the energy dissipation demands on plastic hinge regions in beams. To accomplish sharing of inelastic deformations, one can try to tune the relative magnitude of beam bending strength and panel zone shear strength. The extent to which sharing of inelastic deformations will be accomplished in an earthquake will depend on the accuracy of design equations and on actual material properties of beams, columns, and doubler plates, which might be quite different from those assumed in the design process.

Global shear yielding of the panel zone occurs when the average shear stress in the panel zone is about $0.6F_y$, where F_y is the



(a)



(b)

Figure 3-7 - Panel zone behavior; (a) mode of deformation, (b) typical shear force – shear deformation relationship

Protected Zone

Testing conducted by the FEMA/SAC project demonstrated that the regions of beams undergoing significant inelastic strains are sensitive to discontinuities caused by welding, rapid change of section, penetrations, or construction-related flaws. Connections, attachments, notches, or flaws may interfere with the formation of the hinge or initiate a fracture. For this reason, areas of anticipated inelastic straining are designated as protected zones and are not to be disturbed by other building construction operations.

specified minimum yield stress. Tests and analyses have shown that the actual shear strength of the panel zone might be considerably larger than the shear force that causes global shear yielding, because of strain hardening and the additional resistance provided by the column flanges bounding the panel zone. AISC 360, §J6.6 permits utilization of this additional shear resistance in design when the flexibility of the panel zone is considered in analysis, as required by ASCE 7, §12.7.3. Designers should be aware, however, that significant inelastic deformations might be associated with this increase in resistance. For a connection in which beam flanges are directly welded to column flanges, large inelastic shear distortion of the panel zone might contribute to the initiation and propagation of fracture at welded beam-to-column connections. In such cases sharing of inelastic deformations between beams and panel zones is not encouraged.

3.6.5 Lateral Bracing and Stability

Stability bracing is required to inhibit lateral buckling or lateral-torsional buckling of primary framing members. Beam flange continuity plates are required in columns, except in certain cases where the column web and flange thicknesses can address the limit states associated with concentrated beam flange forces.

Post-Yield Increase in Strength of Panel Zones

Panel zones in columns with thick column flanges exhibit a sizeable increase in strength beyond global shear yielding of the panel zone web area. In large part this increase is attributed to the observation that panel zones deform in a parallelogram shape and the column flanges at the corners of the panel zone provide resistance to this deformation mode until plastic hinging occurs. Attainment of this post-yield strength increase, as expressed in AISC 360, §J6.6, Equations J10-11 and J10-12, is associated with an inelastic distortion on the order of three times the distortion causing global shear yielding of the panel zone web.

4. Analysis Guidance

4.1 Analysis Procedure

ASCE 7, §12.6 permits three types of analysis procedures to determine member design forces and design drifts. These include: equivalent lateral force, modal response spectrum, and seismic response history analysis. Equivalent lateral force analysis is the simplest procedure. However, it can lead to excessively conservative designs. ASCE 7, §12.6, Table 12.6-1 prohibits this analysis procedure for structures having periods greater than $3.5T_s$ or structures with certain horizontal or vertical building irregularities.

The equivalent lateral force procedure is based on an approximate fundamental period, unless the period of the structure is determined by more exact analysis. In many cases, exact analysis will determine a substantially longer building period than that determined by the approximate methods. As a result, substantial reduction in base shear forces often can be obtained by calculating building periods using the more exact methods. However, ASCE 7, §12.8.1.1, Equations 12.8-3 and 12.8-4 place an upper limit on the period that can be used.

Modal response spectrum analysis is the preferred procedure, as it more accurately accounts for a building's dynamic behavior, takes advantage of a calculated rather than approximated period, and accounts for modal participation, which can result in a lower response than that calculated using the equivalent lateral force procedure. ASCE 7, §12.9.4 requires scaling the modal base shear and all corresponding element forces to a minimum of 85 % of the base shear determined using the equivalent lateral force procedure. This provision is intended to guard against the use of analytical models that underestimate a structure's stiffness and produce unrealistically low estimates of design forces.

For structures with periods calculated to exceed limits specified in ASCE 7 or for irregular structures, either the modal response spectrum or seismic response history analysis procedures are required. However, elastic response history analysis is more difficult than modal response spectrum analysis, does not provide significant design advantage.

Analysis can use either 2-D or 3-D computer models. Three-dimensional models are recommended, and sometimes required, because they are effective in identifying the effects of any inherent torsion in the lateral system, as well as combined effects at corner conditions.

ASCE 7, §12.5 specifies requirements for the combination of seismic forces along different building axes. The design forces for the beams and columns are calculated independently for response in each orthogonal direction. It is common to combine the resulting seismic forces using the orthogonal combination

procedure in which 100 % of the seismic force in one direction is combined with 30 % of the seismic force in the perpendicular direction. Multiple load combinations are required to bound the orthogonal effects in both directions. The design of each beam and column is based on an axial and biaxial flexural interaction for each load combination. However, this orthogonal force combination procedure is not required in all structures. The requirements should be reviewed and the frame designed accordingly.

4.2 Connection Modeling

Traditional structural analyses often model moment frames as line representations of horizontal and vertical members, with the lines intersecting at dimensionless nodes. Such representations are referred to as centerline models. Modifications to centerline models might be advisable, and in some cases are required to account for the effects of panel zones and non-uniform beam stiffness on horizontal displacements caused by seismic design forces.

4.2.1 Effect of Panel Zone Stiffness on Drift

ASCE 7, §12.7.3b requires inclusion of the contribution of panel zone deformation in drift calculations. This is also required as a precondition to the use of the panel zone design shear strength equations, J10-11 and J10-12 of AISC 360, §J6.6, which allows for the increased panel zone strength that can be mobilized by including the effect of column flanges.

Elastic panel zone deformation contributions to story drift can be accounted for by either explicit modeling of panel zone shear behavior or by adjusting the lengths of beams and columns in a manner that accounts implicitly for the contributions of panel zone deformations to drift. Many analysis programs permit the insertion of rigid offsets at the ends of beams and columns as a means of accounting for panel zone stiffness. The use of rigid offsets is not recommended unless the dimensions of the offsets are obtained by rational analysis.

A practical way of accounting implicitly for the contribution of panel zone deformations to story drift is through the use of centerline dimensions for beams and columns. In this approach, the contributions of beam and column flexural deformations to drift are overestimated, while the contributions of panel zone shear deformations are ignored. In most practical cases, the resulting story "racking" drift is larger than that obtained from incorporating elastic panel zone shear deformations explicitly. Equations 4-1 through 4-3, which provide estimates of the contributions of beam and column flexure and of panel zone shear deformations to drift, can be used to check the validity of this observation. These equations are based on the subassembly freebody shown in **Figure 4-1**, which assumes points of

$$\delta_r = \frac{h^2 \left(1 - \frac{2d_c}{\ell_1 + \ell_2} \right)}{6E \left(\frac{I_1}{\ell_1 - d_c} + \frac{I_2}{\ell_2 - d_c} \right)} V_{col} \quad \text{Equation 4-1}$$

$$\delta_c = \frac{(h - d_b)^3}{12EI_c} V_{col} \quad \text{Equation 4-2}$$

$$\delta_p = \frac{(h - d_b) \left(\frac{h}{d_b} - 1 \right)}{G t_p d_c} V_{col} \quad \text{Equation 4-3}$$

Where:

- δ_r = story drift due to beam flexure, in
- δ_c = story drift due to column flexure, in
- δ_p = story drift due to panel zone shear deformations, in
- V_{col} = column shear force, kip
- h = story height (centerline dimension), in
- ℓ_1 and ℓ_2 = beam spans (centerline dimensions), in
- I_1 and I_2 = beam moment of inertia, in⁴
- I_c = column moment of inertia, in⁴
- d_b = depth of beam, in
- d_c = depth of column, in
- t_p = thickness of joint panel zone, in

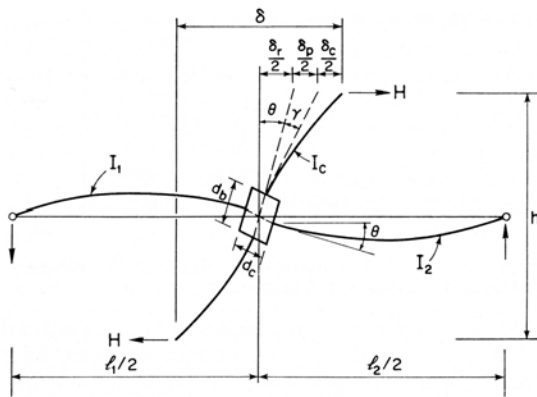


Figure 4-1 - Contributions of beams, columns, and panel zone to shear mode of drift.

inflection at beam midspans and column midheights. These assumptions should be reasonable except in the bottom story where a larger story height and column base boundary conditions have a large effect on drift.

The equation for δ_p is based on the following simplifying assumptions: panel zone shear force, $V_{pz} = \Sigma M_b/d_b - V_{col}$ (see Section 5.4.3), where $\Sigma M_b = V_{col}h$, and angle of shear distortion of the panel zone, $\gamma = V_{pz}/(t_p d_c G)$.

The total drift obtained from summing all three equations provides a baseline estimate of drift. If centerline dimensions are used and joint panel zone deformations are ignored (two

compensating errors), the panel zone parameters d_b and d_c in Equations 4-1 and 4-2 become zero, which greatly simplifies these equations, and Equation 4-3 is not needed. In this manner, these equations can be used to assess the accuracy obtained from drift estimates based on centerline dimensions. If centerline dimensions are used, the required bending strength for column design is not obtained directly from a computer analysis, but it can be obtained by interpolation from the column moments at beam centerlines or the ends of offsets to the moments at the beam flange levels.

One desirable option is to incorporate the effects of panel zone shear deformations directly in the analytical model. In a frame analysis program that consists only of line elements, panel zone behavior can be modeled in an approximate manner by means of scissors elements (**Figure 4-2**) or more accurately by creating

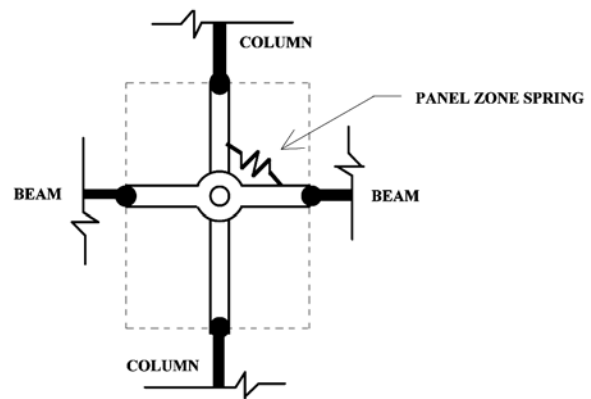


Figure 4-2 - Panel zone scissor model.

a panel zone with rigid elements linked by hinges at three corners and by a rotational spring in the fourth corner, as illustrated in **Figure 4-3**. In the scissors model, the rotation is controlled by a spring that relates the sum of moments in the beams to the spring rotation. The sum of moments can be related to the joint shear force (see Section 5.4.3), and the spring rotation is equal to the panel zone shear distortion angle. In this model, the right angles between the panel zone boundaries and the adjacent beams and columns are not maintained, which results in approximations in deflections. The parallelogram model (**Figure 4-3**) avoids this approximation, but requires the addition of eight rigid elements per panel zone. These eight rigid elements create a panel zone that deforms into a parallelogram. The strength and stiffness properties of the panel zone can be modeled by a rotational spring located in one of the four panel zone corners. In this case, the elastic spring stiffness is defined as $K_s = (V_{pz}/\gamma)d_b = t_p d_c d_b G$.

4.2.2 Beam Stiffness Reductions

For Reduced Beam Section connections, the beam flange width is reduced near the beam-ends, where curvature effects are at a maximum under lateral frame loading. It is important to account

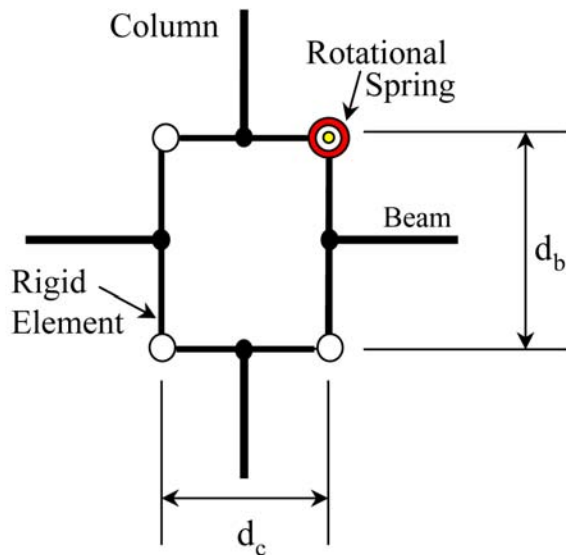


Figure 4-3 - Panel zone parallelogram model.

for the resulting reduction in beam stiffness in analytical models. Some software packages used for such analysis have explicit elements that can model the reduced stiffness of beams having Reduced Beam Section cutouts. Alternatively, use of 90 % of the beam section properties is typically a reasonable approximate representation of this effect, when 50 % reductions in flange width are used and is a conservative approximation when smaller reductions in beam section are used.

4.3 Base Fixity

Base restraint can have a significant effect on moment frame behavior. ASCE 7, §12.7.1 permits consideration of the columns to be either fixed or flexible at the base, as suits the conditions of construction. Therefore, the designer is required to determine the appropriate analytical restraint conditions for column bases.

Since most column bases and foundations will provide some restraint against rotation, an assumption that column bases are pinned will tend to overestimate column flexibility, building period and 1st story drift, all of which are conservative design assumptions. When pinned bases are assumed, the column base anchorage must be designed with adequate capacity to transfer the shear and axial forces to the foundation, while accommodating the rotations that will occur at the column bases. Some of this rotation can occur in the foundation itself.

Similarly, since few real column bases provide true fixity, fixed base assumptions tend to result in underestimates of column flexibility, building period and 1st story drift, which can be unconservative. Column bases should not be modeled as fixed unless the bases themselves and the foundation elements they are attached to can effectively provide sufficient restraint. Column bases should not be modeled as fixed unless foundations are designed with adequate strength and stiffness to provide such fixity. The extent of fixity present at column bases can be determined by explicit modeling of the base and foundation conditions.

5. Design Guidance

5.1 Design Procedure

The three basic steel special moment frame design components are beams, columns, and beam-column connections. Beams span the horizontal clear distance between protected zones; columns span the vertical clear distance between panel zones; and the beam-column connections encompass both protected and panel zone regions at the beam-column intersections.

AISC 341 permits the use of either Load and Resistance Factor Design or Allowable Strength Division design approaches to proportion beams and columns in steel special moment frames. However, AISC 358, §1.3 permits the use of Load and Resistance Factor Design procedures only for design of prequalified connections. The corresponding nominal strengths, resistance and safety factors, and available strengths of the components must be determined in accordance with the provisions of AISC 360, unless noted otherwise in AISC 341 or AISC 358.

5.2 Beam Design

5.2.1 Limitations

To provide for reliable inelastic deformations, AISC 341, §9.4 requires seismically compact beam cross sections. Beams should not be designed as composite with supported concrete slabs as the composite behavior is not available when the top flange is in tension and also because AISC 341, §9.2d prohibits placement of shear studs in the zone of anticipated plastic hinging (see Section 7.2). AISC 341, §9.2d requires the designation of the region at each end of the beam that is subject to inelastic straining as a protected zone. AISC 341, §9.4b prohibits abrupt changes in beam flange area in this region. The drilling of flange holes or trimming of the flanges is permitted only if qualification testing demonstrates that the resulting configuration can develop a stable plastic hinge. Welds connecting the web(s) and flanges in the expected regions of plastic hinging in built-up beams must be made using complete joint penetration groove welds with a pair of reinforcing fillets in accordance with AISC 358, §2.3.2a.

AISC 358, §2.3 requires both rolled wide-flange and built-up beams to conform to the cross-section profile limitations applicable to the specific connection type. These limitations include restrictions on beam depth, weight, flange thickness, and clear span-to-depth ratio. The limitations do not apply when project-specific qualification testing is performed using beams of the proposed cross section.

5.2.2 Lateral-Torsional Buckling

When subjected to inelastic deformation, steel special moment frame beams must resist member instability resulting from lateral-torsional buckling. For this reason, AISC 341, §9.8 requires

lateral bracing of top and bottom flanges near concentrated forces, changes in cross-section, locations where analysis indicates that a plastic hinge will form during inelastic deformation, and spaced at no more than $0.086r_yE/F_y$, where r_y is the beam radius of gyration about the weak axis, E is the modulus of elasticity, and F_y is the specified minimum yield stress. **Figure 5-1** shows evenly spaced lateral bracing for the bottom flange of a steel special moment frame beam in which the top flange is braced by attachment to the concrete-filled metal deck above. **Figure 5-2** shows the use of intermediate framing to provide lateral bracing of both flanges in a condition where the diaphragm is a flexible, unfilled metal deck.



Figure 5-1 - Steel special moment frame beam stability bracing with a concrete filled metal floor deck.



Figure 5-2 - Steel special moment frame beam stability bracing with a flexible unfilled metal roof deck.

Plastic hinge locations must be consistent with AISC 358, §2.4.2, or as otherwise determined with either prequalification or qualification testing. AISC 358 requires placement of lateral bracing just outside designated protected zones. Such bracing also is required at any other locations where plastic hinging

can occur, which for beams with heavy gravity loading can be within beam spans. In many cases, when a beam supports a concrete structural slab that is connected to the beam between the protected zones with welded shear connectors, supplemental lateral bracing may be eliminated at the plastic hinge. Such detail-specific exceptions are outlined in the individual connection chapters of AISC 358.

If lateral braces are provided adjacent to the plastic hinge, AISC 341, §9.8 requires a brace strength equal to at least 6 % of the expected flange capacity at the plastic hinge location. Otherwise, the brace strength must meet the provisions of AISC 360, Appendix 6, Equation A-6-7. All braces also must meet the stiffness provisions of AISC 360, Appendix 6, Equation A-6-8.

5.2.3 Strength

Required beam strength initially is determined using the specified loads of ASCE 7, §2.3 and 2.4. Although steel special moment frame story drift limits often will control the selection of the beams, the flexural and shear strengths still must be verified.

Beam nominal flexural strength, M_n , is determined in accordance with AISC 360, §F2. Since AISC 341 requires seismically compact sections with adequate lateral bracing, it is necessary to evaluate only the yielding limit state (plastic moment). When using Reduced Beam Section connections, adequacy of beam flexural strength must be evaluated both at the column face and at the reduced section. The effective Reduced Beam Section plastic section modulus is determined in accordance with AISC 358, §5.8, Equation 5.8-4.

Beam nominal shear strength, V_n , is determined from the limit states of shear yielding and shear buckling in accordance with AISC 360, §G2. Beam sections are designed for flexure and shear such that design strengths including corresponding resistance or safety factors are at least equal to required demand moments and shears.

5.3 Column Design

5.3.1 Limitations

As with beams, AISC 341, §9.4, requires seismically compact column sections. AISC 358, §2.3 also requires that both rolled wide-flange and built-up columns conform to the cross section profile limitations applicable to the specific connection type. These limitations include restrictions on depth, weight, and flange thickness.

AISC 358, §2.3.2b prequalifies a number of built-up column shapes including the following: (1) I-shaped welded columns that resemble standard rolled wide-flange shape in cross section shape and profile, (2) boxed wide-flange columns, fabricated by adding side plates to the sides of an I-shaped cross section, (3)

built-up box columns, fabricated by welding four plates together to form a closed box-shaped cross section, and (4) flanged cruciform columns. The flanged cruciform columns are fabricated by splitting a wide-flange section in half and welding the webs on either side of the web of an unsplit I-shaped section at its mid-depth to form a cruciform shape, each outstanding leg of which terminates in a rectangular flange.

The preponderance of connection tests reviewed as a basis for the prequalification of steel special moment frame assemblies consists of rolled wide-flange beam-column connections. Some limited testing has been performed with built-up box sections. Given that the predominant steel special moment frame inelastic behavior is expected to occur in the beams, additional testing has not been deemed essential to prequalify built-up column sections.

5.3.2 Stability

In most cases, steel special moment frame columns are required to be braced at beam-to-column connections to prevent rotation out of the plane of the moment frame, particularly if inelastic behavior is expected in or adjacent to a beam-column connection. In some special cases, such as when a column spans two or more stories without a supporting floor, the potential for out-of-plane buckling at the unbraced connection must be minimized. In the event such a column containing a connection is not laterally braced, AISC 341, §9.7b requires the column to conform to AISC 360, §H1, but with a number of exceptions. For example, the unbraced column segment must be designed using the distance between adjacent lateral braces as the column height for buckling transverse to the plane of the frame.

When columns are braced laterally by the floor or roof framing, column and beam webs are coplanar, and columns remain elastic outside panel zones, AISC 341, §9.7 requires only bracing at beam top flanges. Otherwise, column flange bracing is required at both the top and bottom levels of beam flanges. It is assumed that a column will remain elastic outside the panel zone when the beam-column moment ratio is greater than 2.0. Flange lateral bracing can be direct or indirect. Direct lateral support (bracing) can be achieved through the use of braces or other members, deck and slab, attached to a column flange at or near the desired bracing point. Indirect lateral support can be achieved through the stiffness of members and connections that are not directly attached to column flanges, but rather act through column web or stiffener plates.

AISC 341, §9.7a(2) specifies a required braced strength for column flange bracing that is equal to 2 % of available beam flange strength.

5.3.3 Strength

Required column strength initially is determined using the specified load combinations of ASCE 7, §2.3 and 2.4. Although

steel special moment frame story drift limits and strong-column/weak-beam requirements often will control the selection of column sections, the combined axial and flexural strengths still must be verified.

Adequacy of column strength for combined flexure and axial loads is verified using interaction equations contained in AISC 360, §H1. The interaction equations accommodate flexure about one or both principal axes as well as axial compression or tension. In these equations, the column available axial compressive strength, P_c , is determined in accordance with AISC 360, §E as the value obtained by the limit state of flexural buckling. The column available flexural strength, M_c , is determined in accordance with AISC 360, §F2 as the lower value obtained by the limit states of yielding (plastic moment) and lateral-torsional buckling.

Column nominal shear strength, V_n , is determined from the limit states of shear yielding and shear buckling in accordance with AISC 360, §G2. Column sections are designed for shear such that nominal strength including corresponding resistance or safety factors is at least equal to the required demand shear.

If the column axial demand to capacity ratio computed using required combined loads (not including amplified seismic forces) considering design seismic forces and corresponding resistance or safety factors exceeds 40 %, AISC 341 – 8.3(1) and 8.3(2) requires that columns have sufficient axial strength to avoid global buckling or tensile fracture under combined loads computed using the system overstrength factor, Ω_o . In this case, it is permitted to neglect consideration of concurrent bending moments and to use an effective length factor, $K=1.0$.

5.3.4 Column Bases

Column base connections are among the more important elements in steel special moment frame design. When design assumes a fixed base condition, the column bases must be designed and detailed to develop potential plastic hinging in the columns. AISC 341 – 8.5a, 8.5b(2), and 8.5c(2) outline the column base requirements for axial, shear, and flexural strengths, respectively.

AISC 341, §C8.5c, Figure C-1-8.5.3 shows several examples of rigid base assemblies that employ thick base plates, haunches, cover plates, or other strengthening mechanisms to develop plastic hinging in the column. In addition, AISC 341, §C8.5b, Figure C-I-8.5.1 shows examples of base assemblies that employ anchor rod bearing, shear key bearing, or grout bearing to transfer shear forces into the supporting concrete foundation. Friction should not be relied on as a means of shear transfer at column bases.

The available strength of concrete elements at column bases must be in accordance with the American Concrete Institute ACI 318, Appendix D. ACI 318, Appendix D – D.3 requires design of anchorage in Seismic Design Categories D, E, and F

to be controlled by ductile yielding of a steel element and also reduces the available capacity of such anchors by a factor of 0.75. These requirements apply for calculation of available strengths to match required strengths that are calculated for load combinations including $1.0E$. Required strengths for bases and anchors in AISC 341, §8.5 are calculated at higher force levels. Therefore, these special anchorage requirements need not apply.

5.3.5 Column Splices

Contrary to the notion that steel special moment frame columns typically will bend in double curvature with an inflection point near mid-height, nonlinear analyses have demonstrated that mid-height column bending moments can be substantial, and under some conditions single curvature bending is possible. Accordingly, AISC 341, §9.9 requires that the expected flexural strength of the smaller column cross section be developed at column splices, either through the use of complete joint penetration groove welds or through other means that can provide similar strength. In addition, it requires the shear strength of the splice be sufficient to resist the shear developed when the column nominal plastic flexural strength, M_{pc} , occurs at each end of the spliced column. AISC 341, §8.4a permits column web splices to be either bolted or welded, or welded to one column and bolted to the other. In the case of bolted splices, plates or channels must be used on both sides of column webs and single-sided connections are not permitted.

An exception in AISC 341, §9.9 permits the design of column splices based on appropriate inelastic analysis to determine required strength, coupled with the use of fracture mechanics principles to determine the available connection strength. However, this will seldom result in a more economical column splice design.

5.4 Connection Design

5.4.1 Probable Maximum Moment

AISC 358 identifies the locations of assumed plastic hinge zones in the respective provisions for each of the prequalified connection types. These plastic hinge locations are specified based on observed hinge formation during connection assembly tests. They represent the anticipated location of inelastic deformation in connection assemblies conforming to the prequalification requirements. Although AISC 358 specifies the region of primary plastic hinging, some limited inelastic behavior also may occur in other locations such as the column panel zone, and, in beams that have large gravity load demands, within the beam span.

AISC 358, §2.4.3, Equation 2.4.3-1, requires connection strength capable of developing the probable maximum moment in the beam at the location of the plastic hinge. The probable plastic moment at the plastic hinge is intended to be a conservative estimate of the maximum moment likely to be developed by the

connection under cyclic inelastic response. It includes consideration of likely material overstrength and strain hardening.

5.4.2 Column-Beam Moment Ratio

AISC 341, §9.6, requires, with some exceptions, a check on the relative bending strength of columns versus beams, using the intersection of beam and column centerlines as a reference point. At this intersection, the ratio $\Sigma M_{pc}^*/\Sigma M_{pb}^*$ should be greater than 1.0, where ΣM_{pc}^* is the sum of the plastic moment capacities of the columns above and below the panel zone, reduced for axial load effects, and ΣM_{pb}^* is the sum of beam moments obtained by “projecting” the expected flexural strengths of the beams from the plastic hinge locations to the column centerline. The term “projecting” implies that each of these moments is calculated from the flexural strength at the plastic hinge locations and amplified by a moment due to shear forces based on Load and Resistance Factor Design load combinations from the location of the plastic hinge to the centerline. Note that ΣM_{pc}^* is based on nominal strengths, and ΣM_{pb}^* is based on expected strengths (defined as nominal strength times 1.1 R_y).

The larger the ratio $\Sigma M_{pc}^*/\Sigma M_{pb}^*$, the smaller is the likelihood that undesirable plastic hinges will form in columns. AISC 341, §9.7a permits the assumption that columns remain elastic if this ratio is greater than 2.0. When a column cannot be shown to remain elastic outside the panel zone, column flanges have to be braced laterally as discussed previously in Section 5.3.2.

5.4.3 Beam-Column Panel Zone

AISC 341, §9.3, specifies that the required panel zone shear strength be determined from the summation of moments at the column faces, as determined by projecting the expected moments at the plastic hinge points to the column faces. The expected moment is the bending strength at the plastic hinge point based on expected material yield strength, i.e., based on $R_y F_y$. If the “summation of moments at the column faces” is denoted as ΣM_b , the required panel zone shear strength can be estimated as $V_{pz} = \Sigma M_b/d_b - V_{col}$, (see **Figure 5-3**). More elaborate equations should be used if V_{col} above the panel zone differs significantly from V_{col} below the panel zone, and if the depths of the two beams framing into the panel zone are different. The design shear strength is $\phi_v R_v$, with $\phi_v = 1.0$ for Load and Resistance Factor Design. The nominal shear strength R_v is determined from AISC 360–J10-6.

If the effect of panel zone deformations on frame stability is considered (see Section 4.2.1), AISC 360, §J6.6 permits an increase in the design shear strength beyond the level associated with global shear yielding of the panel zone. In this instance, Equations J10-11 and J10-12 may be used rather than Equations J10-9 and J10-10. If panel zones are modeled in analysis, as recommended previously in Section 4.2, it is permissible to use these equations.

Equating required shear strength with design shear strength

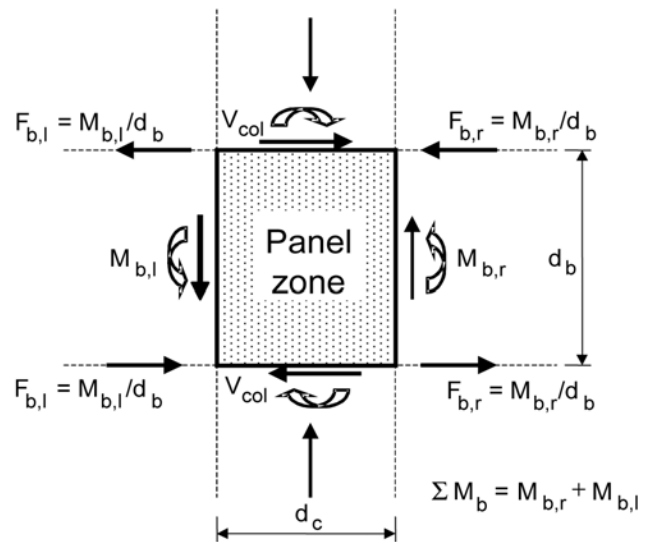


Figure 5-3 - Panel zone free body diagram.

will determine the need for panel zone doubler plates. AISC 341, §9.3b presents limits on the plate thickness of column web and doubler plates. AISC 341, §C9.3, Figure C-I-9.2 and Figures C-I-9.3a, b, and c show alternative details for connection of the web doubler plate(s) to the column panel zone, when doubler plates are required. Often it will be more economical to use a heavier column section than to provide doubler plate reinforcement in panel zones.

5.4.4 Continuity Plates

AISC 341, §9.5 requires continuity plates in steel special moment frames consistent with the prequalified connection designated in AISC 358, or as otherwise determined with either prequalification or qualification testing. AISC 358, §2.4.4 includes a provision requiring continuity plates unless the column flange thickness is adequately proportioned in accordance with AISC 358, §2.4.4 Equations 2.4.4-1 and 2.4.4-2 for wide flange or built-up I-shaped columns, or with AISC 358, §2.4.4 Equations 2.4.4-3 and 2.4.4-4 for boxed wide-flange columns.

AISC 341, §2.4.4a governs the design of continuity plate thickness. The continuity plates must be attached to the column in accordance with AISC, §2.4.4b. In addition, to avoid welding in regions of potentially low notch toughness in wide-flange sections, the corners of the continuity plates are required to be clipped in accordance with AISC 341, §7.5 and as shown in AISC 341, §C8.1 Figures C-I-7.2a and b.

5.4.5 Beam Web to Column Connection

AISC 358 provides beam web-to-column connection design and detailing requirements for prequalified connections. Two types of web connection details are used for steel special moment frame connections: a welded and a bolted detail. In the welded detail, the beam web is welded directly to the column flange using a complete joint penetration groove weld. For the bolted detail, pretensioned high-strength bolts are used to attach the beam web to a single plate shear connection. Details for this shear connection vary between different connection types.

6. Additional Requirements

6.1 Special Inspection

Steel special moment frames are complex structural systems whose performance depends on proper implementation of design requirements during construction. Therefore, wherever steel special moment frames are used, regardless of the seismic design category, IBC, §1704 requires continuous special inspections by qualified inspectors for the fabrication and construction of the steel frame. Continuous special inspection generally is interpreted to mean that the special inspector is on-site at all times observing the work requiring special inspection. This includes welding and high-strength bolting.

Generally, the special inspector is required to inspect work for conformance to approved design drawings and specifications. Under IBC, §1704.1.1, the engineer of record should designate the specific inspections and tests to be performed in a Statement of Special Inspections, submitted as part of the permit application. Contract documents should specify that the special inspector will furnish inspection reports to the building official, the engineer of record, owner, and contractor. Discrepancies should be brought to the immediate attention of the contractor for correction, then, if uncorrected, to the proper design authority and the building official. A final signed report should be submitted stating whether the work requiring special inspection was completed in conformance with the approved plans and specifications and applicable workmanship provisions of the IBC and its referenced standards.

6.2 Material Properties

Wherever steel special moment frames are used, regardless of the seismic design category, AISC 341 and ACI 318 stipulate that material properties conform to specific requirements. These requirements are intended to result in a frame capable of sustaining multiple inelastic deformation cycles without critical degradation.

6.2.1 Steel

AISC 341, §6.1 requires that structural steel used in steel special moment frames meet the requirements of AISC 360, §A3.1a, except as modified by AISC 341. The specified minimum yield stress to be used for members in which inelastic behavior is expected cannot exceed 50,000 psi, unless the suitability of the material is determined by testing in accordance with AISC 341, Appendix S or other rational criteria. This same limitation does not apply to columns for which the only expected inelastic behavior is yielding at the column base.

AISC 360, §A3.1c and AISC 341, §6.3 require structural steel hot rolled shapes with flanges 1 1/2 inches thick and thicker or plates 2 inches or more in thickness used in steel special moment

frames to have additional Charpy V-Notch toughness requirements.

6.2.2 Concrete

Concrete used in composite components and in supporting foundations must meet the requirements of ACI 318, §21.1.4 through 21.1.8. ACI 318, §21.1.4.2 requires a minimum concrete compressive strength, f'_c , of 3,000 psi.

6.3 Bolting

AISC 341, §7.2 requires fasteners used in steel special moment frames to be pretensioned high-strength bolts meeting the requirements of AISC 360, §J3.8 with a Class A surface. The faying surfaces for some types of bolted connections are permitted to be painted with coatings not tested for slip resistance, or with coatings with a slip coefficient less than that of a Class A faying surface. AISC 358, §4.1 requires bolts in prequalified steel special moment frame connections to conform to ASTM A325, A490 or F1852. AISC 360 – M1 and AISC 341, §5.3 require the locations of pretensioned bolts to be shown in the shop and erection drawings. There may be connections or applications for which details are not addressed specifically by referenced standards. If such a condition exists, the shop drawings should include appropriate requirements for that application.

Inspection of bolts and bolting operations in steel special moment frames must be performed in accordance with IBC, §1704.3.3, AISC 341, Appendix Q, and AISC 358, §4.2.

6.4 Welding

AISC 341, §7.3 requires welding of steel special moment frames to be performed in accordance with AISC 341, Appendix W. In addition, welding must be performed in accordance with an appropriate Welding Procedure Specification approved by the engineer of record, as required by AWS D1.1. The Welding Procedure Specification essential variables, including current setting, length of arc, angle of electrode, speed of travel, and filler metal specification must be within the parameters established by the filler metal manufacturer. AISC 360, §M1 and AISC 341, §5.1, 5.2, and 5.3, require that the locations of shop and field welds, particularly demand-critical welds, be included in the structural design, shop, and erection drawings and in the structural specifications.

Inspection of welds and welding operations in steel special moment frames must be performed in accordance with IBC, §1704.3.1, AISC 341, Appendix Q and AWS D1.1.

6.4.1 Filler Metal

AISC 341, §7.3b requires that welds designated as demand-critical be made with a filler metal capable of providing a minimum Charpy V-Notch toughness of 20 ft•lb at -20 °F as determined by the appropriate AWS classification test method or manufacturer certification, and 40 ft•lb at 70 °F as determined by AISC 341, Appendix X or other approved method, when the steel frame is normally enclosed and maintained at a temperature of 50 °F or higher. For structures with service temperature lower than 50 °F, the qualification temperature for AISC 341, Appendix X must be 20 °F above the lowest anticipated service temperature, or at a lower temperature.

AISC 341, §7.3a requires all other welds in members and connections in the Seismic Load-Resisting System to be made using filler metal with a minimum Charpy V-Notch toughness of 20 ft•lb at 0 °F as determined by the appropriate AWS classification test method or manufacturer certification.

6.5 Built-Up Members

The use of built-up members in steel special moment frames are permitted with prequalification or qualification testing, or if the members meet geometric, profile, and connection limitations specified in AISC 358, §2.3.2.

The flanges and webs of a built-up member must have a width, depth, and thickness profile similar to rolled wide-flanges sections meeting specific profile limitations. The webs must be connected continuously to flanges in accordance with AISC 358, §2.3.2a or 2.3.2b. Complete joint penetration groove welds joining the web plate to the flange are demand-critical in accordance with AISC 341, §7.3b. Built-up columns must also satisfy the requirements of AISC 360, §E6 except as modified by AISC 358, §2.3.2b.

6.6 Additional System Design Requirements

6.6.1 Structural Diaphragms

In steel special moment frame construction, roof and floor slabs typically consist of either composite or noncomposite concrete-filled metal deck slabs that are connected to the structural framing and provide an in-plane diaphragm that collects and distributes inertial forces. ASCE 7, §12.10 specifies required diaphragm design forces and requires design of diaphragm collectors for forces amplified by the applicable system overstrength factor, Ω_o (ASCE 7, §12.10.2.1). Shear forces are most commonly transferred through puddle welds of the steel deck to the framing. Where large loads must be transferred to a diaphragm, and concrete slabs are present, it can be advantageous to use headed shear studs. The recommended diaphragm (in-plane) shear strength and stiffness design values

for metal deck and composite diaphragms are available in code evaluation service reports and from other industry sources.

As the thickness of concrete over a metal deck is increased, the shear strength can approach that for a formed concrete slab of the same thickness, using the concrete and reinforcement above the metal deck ribs and ignoring the beneficial effect of the concrete in the flutes. When the required composite shear strengths exceed those available from industry sources, ACI 318, §21.11 outlines requirements for concrete diaphragms that are applicable in Seismic Design Categories D, E, or F.

6.6.2 Foundations

ACI 318, §21.12.1 outlines design requirements for foundations that transfer earthquake-induced forces between the steel special moment frame and the ground in Seismic Design Categories D, E, or F. As foundation damage can be extremely difficult to detect and repair, it is desirable that inelastic response during earthquake ground shaking occurs above the foundation. The requirements include specific provisions for foundation elements (footings, foundation mats, pile caps, grade beams, etc.). In addition, when concrete grade beams connect adjacent column bases, such as that shown in **Figure 6-1**, longitudinal and transverse reinforcement must meet the concrete steel special moment frame requirements of ACI 318, §21.5.



Figure 6-1 - Concrete grade beam connecting adjacent column steel special moment frame bases.

6.6.3 Members Not Designated as Part of the Seismic Load-Resisting System

Due to the inherent flexibility of steel special moment frame systems, columns that are not part of the seismic load-resisting system still may develop significant bending moments and shears when the frame is subjected to the design displacements. Even though the connections of beams framing to columns are often considered to be pins, the columns typically will bend in double curvature with the inflection point near mid-height. AISC 341, §8.4b(1) requires the splice location for such columns be located away from the beam-to-column connection and near the expected inflection point.

At the splice, if partial joint penetration groove welds are used, AISC 341, §8.4a requires that the available column strength be at least 200 % of the required strength. Otherwise, the available strength for each flange splice must be at least half the expected flange strength of the smaller column connected. Partial penetration groove welds are not recommended because such welds are susceptible to fracture under tensile forces, and fracture of a flange splice can lead to fracture of the web splice.

In addition, AISC 341, §8.4b(2) requires the shear strength of the splice be sufficient to resist the shear developed when the column nominal plastic flexural strength, M_{pc} , occurs at the smaller end of the spliced column in both orthogonal axes. As with other column splices, the column web splices can be either bolted or welded, or welded to one column and bolted to the other. In the case of a bolted splice, plates or channels must be used on both sides of the column web. In the event of a flange splice fracture, the plates are expected to maintain alignment of the column.

7. Detailing and Constructability Issues

This section of the Guide addresses a number of issues related to documentation of the design engineer's information, connection detailing, and construction quality and control that are essential to achieving the expected seismic performance of steel special moment frames.

7.1 Structural Drawings, Specifications, Shop, and Erection Drawings

Clear documentation of the expectations of the Structural Engineer of Record is essential to convey the design intent to the general contractor, fabricator, and erector. For steel special moment frame projects, this is especially important, since the design is intended to result in significant inelastic response when subjected to shaking equal to or greater than the design earthquake. This documentation manifests itself in the form of complete drawings and project specifications, with special emphasis on the unique aspects of steel special moment frame connections design, details, and joining via welding and/or bolting.

Recognizing the importance of this documentation, AISC 341, §5.2, 5.3, Appendix W2.2, and Appendix W2.3 specifically list items that are required for documentation, beyond the standard documentation listed in the AISC *Code of Standard Practice for Steel Buildings and Bridges* (AISC 2005). For structural design drawings and specifications, this list includes the following:

- Designation of the members and connections that make up the Seismic Load Resisting-System.
- Configuration, materials and sizes of connections, including gusset plates that are intended to accommodate inelastic rotation.
- Location of demand-critical welds and protected zones.
- In cold weather regions, definition of lowest anticipated service temperature for exposed steel members of the Seismic Load Resisting-System.
- Welding requirements that go beyond standard AWS D1.1 requirements as outlined in AISC 341, §7.3.

7.2 Protected Zones

The majority of steel seismic force-resisting systems not designated as “ordinary” in AISC 341, §11 have specific elements that are intended to be the primary source of inelastic response when the structure is subjected to severe ground shaking. In steel special moment frame structures, the primary inelastic behavior is intended to occur in the beams near the beam-column connections, either just outside a region of strengthening, or within the length of the reduced section for a Reduced Beam Section connection. In most cases, this inelastic

behavior can be expected to concentrate over a length approximately equal to the beam depth, or slightly longer. Because large inelastic strains are expected to occur at these locations, any discontinuities in the material in the steel beam in the hinge zones could become fracture initiation points. In an attempt to avoid these fractures, AISC 341, §9.2d has designated that these zones be protected from discontinuities to the greatest extent possible. Specifically, AISC 341, §7.4 requires the following measures for these “Protected Zones”:

- Any discontinuities created by fabrication or erection procedures must be repaired.
- Attachments for support of perimeter edge angles, facades, ductwork, piping or other construction are to be located outside the protected zone.
- Welded shear studs are not permitted within the protected zone.

The limitation on shear studs in the protected zone was the result of a fracture that occurred in a connection test with a composite floor slab that had headed shear studs in the plastic hinge region to connect the deck and slab to the moment frame beam. If the connection being used in the design has been tested successfully with these shear studs or other connectors in place, AISC 341, §7.4 allows these connectors to be included at the protected zones.

AISC 341, §5.1, 5.2, and 5.3 require designation of the location and dimensions of protected zones on structural design, shop, and erection drawings. Engineers should be aware that trades that may make attachments to structural framing often may not be familiar with these requirements. As shown in **Figure 7-1**, enforcement of the protected zone provisions can be a significant challenge on construction sites. Preconstruction meetings with the general contractor should be used to emphasize the importance of these requirements. The concept needs to be



Figure 7-1 - Unauthorized attachment within the protected zone of a Reduced Beam Section connection.

clear not only to the structural steel and decking subcontractors, but also to all curtain wall, mechanical, electrical, and plumbing subcontractors. It also may be helpful to have these regions painted or otherwise designated once any fireproofing materials are applied.

When repair of a discontinuity within the protected zone is required, the repairs are subject to the approval of the engineer of record. As a reference, the engineer of record can refer to AWS D1.1 and ASTM A6, §9 for guidance in establishing repair acceptance criteria. Outside the protected zone, AWS D1.1 requirements apply for the repair of discontinuities.

7.3 Weld Access Holes

Many steel special moment frame connections include a complete joint penetration groove weld between the beam flanges and the column flange. In most cases, this joint is made with a single bevel weld that is detailed with weld backing across the width of the flange, with the weld being made in the flat position. The backing is typically a steel bar, one inch wide by 3/8 inch thick, though ceramic and copper backing can also be used. To accommodate this backing and to provide access for the welder to make the weld at the bottom flange, a weld access hole is provided. AISC 360, §J1.6 specifies the minimum permissible shape of these access holes for typical conditions.

One finding of the post-Northridge earthquake research was that the configuration and preparation of these access holes can play a critical role in the performance of steel special moment frame connections. Large inelastic strains are concentrated in these regions in connections that focus much of the inelastic behavior at the beam-column interface, for example in the Welded Unreinforced Flange – Welded Web connection. Both experimental and parametric finite element analytical studies have confirmed that modifications to the standard AWS access hole configurations are needed to achieve the levels of inelastic deformation anticipated in steel special moment frame designs. AISC 341, §11.2.a, Figure 11-1, shows the weld access hole configuration required for Welded Unreinforced Flange – Welded Web connections. Access holes for Reduced Beam Section connections must be detailed according to AISC 360, §J1.6. It also should be noted that complete joint penetration groove welds for end plate connections fabricated per AISC 358, §6.9.7 are to be detailed without weld access holes, since extensive testing of this connection indicates that eliminating the access holes significantly improves the performance.

Similar to Protected Zones, weld access holes should be free of discontinuities that could cause a premature fracture. As a result, AISC 341, §11.2a(2) and AISC 358-05, §5.7 state: "The maximum surface roughness of the thermally cut surface shall be 500 microinches (13 microns) in accordance with ANSI B46.1, as measured using AWS C4.1-77." Most automatic thermal

cutting machines used in modern fabricating facilities can achieve this level of surface roughness.

7.4 Web Doubler Plate

As discussed previously in this Guide, high shear forces occur in the joint panel zones of steel special moment frames. In many cases, in order to meet panel zone shear strength requirements, a doubler plate is needed to locally strengthen the column web. Adding doubler plates is expensive because of the significant shop fabrication time that is needed to prepare the plate and weld it into the column web. A rule of thumb that commonly applies is that for most typical moment frame configurations (story heights on the order of 15 ft and beam spans of approximately 30 ft) if the designer can increase the mass per foot of the column by less than 100 lb/ft and avoid the need for doubler plates, the cost of the frame will be reduced. Engineers should confirm this approach with the fabricators selected for a given project.

Proper detailing of the welds between the doubler plates and the column web, column flanges, and/or continuity plates is needed to ensure that force transfers through this highly stressed region can be achieved. Since the AISC 341, §9.3 design provisions are intended to result in some yielding of the joint panel zone, the design of all welds (whether groove or fillet) between the doubler plate and the other elements of a connection need to be able to develop the capacity of the plate. In most applications, the doubler plate is placed immediately adjacent to the column web plate. This location requires welding of the doubler in the region of the web-flange junction of the column, sometimes known as the "k-area." Some fabrication-induced cracking in this area of the column also has led to the suggestion of moving the doubler plate, or plates, to obtain symmetry in the connection, away from this highly stressed area closer to the mid-point of the flange half-width. This practice has not gained widespread acceptance because the need for a second plate and the increased thickness necessary for plate stability increases the cost of this detailing approach over the typical single plate placed adjacent to the column web. AISC 341, §9.3b requires that all plates have a thickness that is larger than 1/90 of the sum of the panel zone depth plus width (all terms in inches). Doubler plates that are thinner than this limit are typically brought into conformance with this requirement by the addition of a series of four plug welds at about the quarter points of the joint panel zone. AISC 341, §C9.5, Figure C-I-9.3, depicts the various configurations for web doubler plates.

7.5 Continuity Plates

As shown in **Figure 7-2**, continuity plates often are added between column flanges to help transfer beam flange forces



Figure 7-2 - Installed connection continuity plates in a bolted stiffened extended end plate connection.

through an entire connection. In steel special moment frame designs, the need for and design of continuity plates should be included in the overall connection design and testing process that is used to verify connection deformation capacity. Connections included in AISC 358 specify the procedures to be used in the design of continuity plates.

Like doubler plates, proper detailing of continuity plates is crucial to the anticipated ductile performance of steel special moment frame connections. Welds between continuity plates and the other elements of a connection typically also are required to develop the capacity of the plate. In most instances, complete joint penetration groove welds are specified between these plates and the inside face of the column flanges because the load is a direct tension that approaches the capacity of the plate. Because the force transfer between continuity plates and the column web, or doubler plate, is primarily a shear force, double fillet welds or groove welds are both options.

AISC 341, §7.5 requires that continuity plates be configured to avoid the welding in the k-area of the column. This is because the straightening process used by some mills can cause local embrittlement of the wide flange section in this area. When shapes have been made brittle by mill straightening, welding can result in fracturing of the section during fabrication. AISC 341, §C6.3, Figure C-I-6.1 illustrates the location of the k-area. Specific dimensions are provided for the clipping of continuity plates to avoid the rounded area at the web flange junction. The reader is referred to AISC 341, §C8.1, Figure C-I-7.2, for a graphical explanation of the continuity plate clips in this region.

When both continuity plates and doubler plates are included in a steel special moment frame detail, the engineer needs to decide whether or not the length of the doubler plates will be stopped at the face of the continuity plates that are adjacent to the joint panel zone. Stopping the doubler plate in this way requires

careful detailing of the two welded joints that would occupy the same space at this intersection. The option of extending the vertical length of the doubler plate beyond the extent of the beam can facilitate welding procedures, but the need to transfer continuity plate forces results in both a thicker doubler plate and a larger weld between the longer doubler plate and the column web.

7.6 Column Splices

Splices in steel special moment frame columns also can be critical to system performance. It is important to note that, in many cases, the primary demand on steel special moment frame columns is flexure, or flexure combined with axial tension, rather than axial compression. In effect, these columns act as “vertical beams” rather than classical columns. Nonlinear response history analyses of steel special moment frame designs performed for the FEMA-SAC Steel Joint Venture project demonstrated that column flexural and tension demands can approach column capacity (FEMA, 2000). These studies also indicate that the location of minimum moment in a column is not static, but moves along the length of the column, depending on the characteristics of the ground motion and the frame configuration. In some instances, no point of inflection occurs over entire story heights at various times during seismic response of the frames. As a result of these findings and the potentially dramatic consequence of column splice failures, AISC has put severe demands on steel special moment frame column splice design. In most cases, it is likely that complete joint penetration groove welds, such as those shown in **Figure 7-3**, will be required for these splices. Severe limitations are placed on the use of partial joint penetration welds.



Figure 7-3 - Steel special moment frame complete joint penetration groove weld column splice.

The preceding discussion focused on splices for Seismic Load-Resisting System columns. The SAC Steel research studies noted above also found that non-Seismic Load-Resisting

System columns that are intended primarily for gravity load resistance can provide beneficial effects to overall system seismic performance. Continuity of these columns was found to help vertically distribute inelastic demands throughout the height of a building, thereby avoiding focusing inelastic demands in a single or small number of stories. This finding caused AISC 341 to place a design requirement on shear connections of non-frame column splices as a means of providing this continuity. This requirement increases design forces on this splice substantially, but still can be accomplished by bolted connections in most cases.

7.7 Concrete Placement

AISC 358 limits the prequalification of some moment connections when a concrete structural slab is present. This is because slabs will tend to act compositely with the steel framing, whether intended to or not, and in the process shift the location of the beam neutral axis and alter stress distributions in the connection. AISC 358 permits some connections to be used with a structural slab only if the slab is not restrained by the column or, in some

cases, by other protruding elements associated with the connection. In this manner, the slab will not inhibit the expected performance of the connection. As shown in **Figure 7-4**, detailing compressible material against the protruding elements prior to the placement of the concrete is a sufficient means to address the requirement.

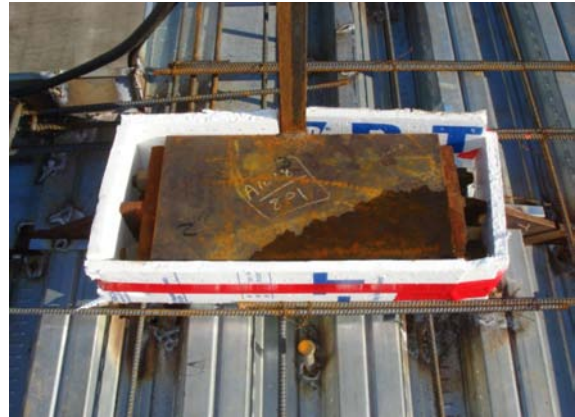


Figure 7-4 - Compressible material used to isolate a protruding bolted stiffened extended end plate connection prior to concrete placement.

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9. Notation and Abbreviations

A_c	area of the end column, in ²	R_v	panel zone nominal shear strength, kip
C_d	deflection amplification coefficient defined in ASCE 7	R_w	response modification factor
C_u	coefficient for upper limit on calculated period as defined in ASCE 7	R_y	ratio of expected strength to the specified minimum tensile strength
D	distance between end columns, in	S_{DS}	design, 5 % damped, spectral response acceleration parameter at short periods as defined in ASCE 7
d_b	overall beam depth, in	S_{D1}	design, 5 % damped, spectral response acceleration parameter at a period of 1 s as defined in ASCE 7
d_c	overall column depth, in	T_a	approximate fundamental period of building defined in ASCE 7
E	modulus of elasticity of steel, $E = 29,000$ psi	T_s	SD1/SDS as defined in ASCE 7
F_y	specified minimum yield stress, ksi	t_p	thickness of the panel zone including doubler plates, in
f'_c	specified concrete compressive strength at 28 days	V	seismic shear force as defined in ASCE 7, kip
G	elastic shear modulus	V_{col}	column shear force, kip
h	story height as defined in ASCE 7, in	V_n	nominal shear strength, in•lb
I	moment of inertia, in ⁴	V_{pz}	panel zone shear force, kip
l	beam span, in	β	ratio of shear demand to shear capacity for the story under consideration, as defined in ASCE 7
K	effective length factor	γ	unit shear deformation of panel zone
K_s	panel zone rotational stiffness	Δ	design story drift as defined in ASCE 7
M_c	column available flexural strength, in•lb	Δ_a	allowable story drift as defined in ASCE 7
M_n	nominal flexural strength, in•lb	Δ_e	story drift determined by an elastic analysis
M_{pc}	column nominal plastic flexural strength, psi	Ω_o	system overstrength factor
M_{pr}	probable peak plastic moment capacity of a structural section, considering material strength variation and strain hardening effects	θ	stability coefficient as defined in ASCE 7
ΣM_b^*	summation of moments at the column faces	δ_r	story drift due to beam flexure, in
ΣM_{pc}^*	sum of the projected column moments at the top and bottom of the panel zone, in•lb	δ_c	story drift due to column flexure, in
ΣM_{pb}^*	sum of the projected beam moments on either side of the panel zone, in•lb	δ_p	story drift due to panel zone shear deformations, in
P	total vertical design load as defined in ASCE 7, kip	ρ	redundancy factor as defined by ASCE 7
P_c	column available axial compressive strength, lb	ϕ	resistance factor for strength
r_y	radius of gyration about y-axis, in	ϕ_v	resistance factor for panel zone shear strength
R	response modification coefficient as defined in ASCE 7		

Abbreviations

ACI	American Concrete Institute
AISC	American Institute of Steel Construction
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
ATC	Applied Technology Council
AWS	American Welding Society
CUREE	Consortium of Universities for Research in Earthquake Engineering
FEMA	Federal Emergency Management Agency
IBC	International Building Code
ICC	International Code Council
SAC	SAC Steel Joint Venture (SEAOC, ATC, CUREE)
SEAOC	Structural Engineers Association of California
SEI	Structural Engineering Institute

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