3. CONNECTION QUALIFICATION

3.1 Scope

This chapter provides design procedures and qualification data for various types of connections for new steel moment-frame buildings. Included herein are criteria for design of connections and conditions that are generic to most connection upgrade types, and criteria for specific details of connections intended to be prequalified for use in seismic applications. Each of the connection prequalifications is limited to specific conditions for which they are applicable, including member size ranges, grades of material and other details of the connection. Also included in this chapter are recommended criteria for qualification of connections that have not been prequalified or are proposed for use outside the limits of their prequalification, as set forth herein, and information on several types of proprietary connections.

Commentary: The 1988 Uniform Building Code (ICBO, 1988) introduced a single prequalified ("prescriptive") moment-connection design for seismic applications, representative of prevailing west coast practice at the time. The "qualification" of this connection was based primarily on the research of Popov and Stephen in the early 1970s. The UBC prequalified connection was subsequently adopted into the 1992 AISC Seismic Provisions and then into other model building codes.

The 1994 Northridge earthquake demonstrated that this prescriptive connection, as it was being used in contemporary practice, was inadequate for the anticipated seismic demands. Following this discovery, enforcement agencies adopted emergency changes to the building codes, deleting the prescriptive connection and requiring that all connection details used in moment resisting frames for seismic application be qualified for adequacy through a program of prototype testing. The Interim Guidelines for Inspection, Evaluation, Repair, Modification and Design of Welded Moment-Resisting Steel Frames (FEMA-267) and, the companion Interim Guidelines Advisories (FEMA-267A and FEMA-267B), continued and reinforced the recommendation for permitting the use of only those connection details demonstrated as adequate by a program of prototype testing, while providing extensive guidance on how and under what conditions such testing should be required and how test results might be interpolated or extrapolated. These recommendations were adopted with some modification, by FEMA-302, the 1997 AISC Seismic Provisions, and the Uniform Building Code (ICBO, 1997), which require that connections for all types of moment frames be qualified by test. Connections for Ordinary Moment Frames (OMFs) were permitted to be designed based on calculations alone, if certain strength and detailing conditions were met.

In the time since the publication of those documents more than 150 connection assemblies have been tested, allowing new prequalifications for connection details believed to be capable of providing reliable service to be developed. Those prequalifications applicable to the design of new structures appear in these

Recommended Criteria. It is the intent of these criteria to return the design of steel moment-frame structures to the straightforward select-design-detail task, while providing the reliability that was previously incorrectly assumed to exist. For the majority of structures and conditions of use, it is intended that the designer will be able to select, design, and detail prequalified moment-frame connections appropriate for the intended structure by using the criteria of this chapter, without the need to perform project-specific prototype qualification testing. For connection details other than those included herein, prototype qualification testing must still be performed, and recommended criteria are provided for performance and acceptance of such testing.

The research supporting the connection prequalifications contained in this chapter is summarized in FEMA-355D, State of the Art Report on Connection Performance. The interested reader is referred to that report for more background on these recommendations, including complete references to specific research reports where more extensive descriptions of individual research methods and results can be found.

3.2 Basic Design Approach

This section provides recommended criteria on basic principles of connection design, including selection of an appropriate connection type, estimation of locations of inelastic behavior (formation of plastic hinges), determination of probable plastic moment at the plastic hinges, determination of shear at the plastic hinge, and determination of design strength demands at critical sections of the assembly. These basic principles apply to the recommended calculation procedures for all prequalified connection types.

3.2.1 Frame Configuration

Frames should be proportioned and detailed so that the required interstory drift angle for the frame can be accommodated through a combination of elastic deformation and the development of plastic hinges at pre-determined locations within the frame. Figure 3-1 indicates a frame in which inelastic drift is accommodated through the development of plastic flexural deformation (plastic hinges) within the beam span, remote from the face of the column. Such behavior may be obtained by locally stiffening and strengthening fully restrained connections by using cover plates, haunches and similar detailing, such that the ratio of flexural demand to plastic section capacity is maximum at these interior span locations. This condition can also be obtained by locally reducing the section of the beam at desired locations for plastic hinging to obtain a condition of maximum flexural demand to plastic section capacity at these sections. Other locations where plastic deformation may take place in frames, depending on the configuration, detailing and relative strength of the beams, columns and connections include: within the connection assembly itself, as is common for partially restrained connections; within the column panel zone; or within the column. The total interstory drift angle, as used in these criteria is equal to the sum of the plastic drift, as described here, and that portion of the elastic interstory drift resulting from flexural deformation of the individual members. Interstory drift resulting from axial deformations of columns is not included.

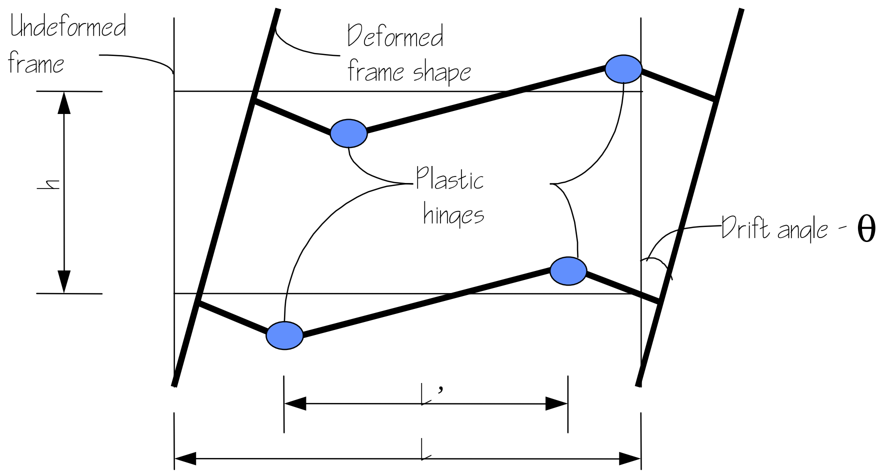


Figure 3-1 Inelastic Behavior of Frames with Hinges in Beam Span

Commentary: Nonlinear deformation of frame structures is accommodated through the development of inelastic flexural or shear strains within discrete regions of the structure. At large inelastic strains these regions can develop into plastic hinges that can accommodate significant concentrated rotations at constant (or nearly constant) load through yielding at tensile fibers and yielding and buckling at compressive fibers. If a sufficient number of plastic hinges develop in a frame, a mechanism is formed and the frame can deform laterally in a plastic manner. This behavior is accompanied by significant energy dissipation and potentially substantial damage to the highly strained elements. The formation of hinges in columns, as opposed to beams, is undesirable, as this may result in the formation of mechanisms with relatively few elements participating, so called "story mechanisms," and consequently little energy dissipation throughout the structure.

The prequalified connection contained in the building codes prior to the 1994 Northridge earthquake was presumed to result in a plastic behavior that consisted of development of plastic hinges within the beams at the face of the column, or within the column panel zone, or as a combination of the two. If the plastic hinge develops primarily in the column panel zone, the resulting column deformation may result in very large secondary stresses on the beam flange to column flange joint, a condition that, for certain types of connections, can contribute to brittle failure. If the plastic hinge forms in the beam at the face of the column, this can result in large inelastic strain demands on the weld metal and surrounding heat-affected zones. These conditions can lead to brittle failure.

Special Moment Frame (SMF) structures are expected to be capable of extensive amounts of energy dissipation through the development of plastic hinges. In order to achieve reliable performance of these structures, frame configurations should incorporate a strong-column-weak-beam design that can lead to development of column hinging and story collapse mechanisms. Further, *fully restrained beam-column connections should be configured either to force the inelastic action (plastic hinge) away from the column face, where performance is less dependent on the material and workmanship of the welded joint, or must employ optimum welded joint design and quality assurance measures. Shifting the hinge away from the column face can be done either by local reinforcement of the connection, or by locally reducing the cross section of the beam at a distance away from the connection. Plastic hinges in steel beams have finite length, typically on the order of half the beam depth. Therefore, for this approach, the location for the plastic hinge should be shifted at least that distance away from the face of the column. For situations where unreinforced connections employing optimum joint design, fabrication, and quality assurance are used, the plastic hinges will occur about one-quarter of the beam depth from the column face and will extend to the face of the column. When the plastic hinge location is shifted away from the face of the column, the flexural demands on the columns, for a given beam size, are increased. Care must be taken to ensure that weak column conditions are not inadvertently created by local strengthening of the connections.*

Connection configurations of the type described above, while believed to be effective in preventing brittle connection fractures, will not prevent structural damage from occurring. Brittle connection fractures are undesirable for several reasons. First, severe connection degradation can result in loss of gravity load carrying capacity of the framing at the connection and the potential development of local collapse. From a global perspective, the occurrence of many connection fractures results in a substantial reduction in the lateral-force-resisting strength and stiffness of the structure which, in extreme cases, can result in instability and collapse. Connections configured as described in these Recommended Criteria should experience fewer such brittle fractures than unmodified connections. However, the formation of a plastic hinge within the beam is not a completely benign event. Beams that have experienced significant plastic rotation at such hinges may exhibit large buckling and yielding deformation, as well as localized damage to floor slabs and other supported elements. In severe cases, this damage must be repaired. The cost and difficulty of such repairs could be comparable to the costs incurred in repairing fracture damage of the type experienced in the Northridge earthquake. The primary difference is that life safety protection will be significantly enhanced and most structures that have experienced such plastic deformation damage should continue to be safe for occupancy, while repairs are made.

If the types of damage described above are unacceptable for a given building, then alternative structural systems, which will reduce the plastic deformation demands on the structure during a strong earthquake, should be considered. Appropriate methods of achieving such goals include the installation of supplemental braced frames, energy dissipation systems, base isolation systems and similar structural systems. Framing systems incorporating partially *restrained connections may also be effective in resisting large earthquake induced deformation with limited damage.*

Ordinary Moment Frame structures are designed so that they will experience less inelastic deformation than Special Moment Frame structures for a given ground motion. Therefore, for Ordinary Moment Frame systems, fully restrained connections that permit development of plastic hinges at locations other than within the beam span, e.g. in the panel zone or in the column, are permitted.

Partially restrained connections are configured to form plastic hinges through yielding of the connection elements themselves. The plastic moment capacity of these connections is typically a fraction of that of the connected framing elements, encouraging the inelastic behavior to occur within the connection at relatively low force levels. These connections must be configured to ensure that inelastic behavior occurs through ductile yielding of elements, rather than brittle failure, such as shearing or elongation of bolts, or tensile fractures through weak net-sections of connection elements. Frames employing properly designed partially restrained connections can be capable of extensive inelastic response, with plastic hinges forming within the connection, adjacent to the face of the column. Because such connections are weaker and less stiff, systems using partially restrained connections typically incorporate more of the framing members into the moment-frame system than do frames using fully restrained connections.

3.2.2 Connection Configuration

A connection configuration should be selected that is compatible with the selected structural system and the sizes of the framing elements. Sections 3.5 and 3.6 present data on a series of prequalified connections, from which an appropriate connection type may be selected. Alternatively, if project-specific connection qualification in accordance with Section 3.9 is to be performed, a connection of any configuration that provides the appropriate interstory drift capacity, in accordance with Section 3.9.2 and meets the strength and stiffness demands for the structure, may be selected.

3.2.3 Determine Plastic Hinge Locations

Based on the data presented in Tables 3-2 through 3-6 and 3-8 through 3-12 for prequalified connections, or data obtained from a qualification testing program for configurations that are qualified on a project-specific basis, the location of expected plastic hinge formation s_h as indicated in Figure 3-2 should be identified. The plastic hinge locations presented for prequalified connections are valid for beams with gravity loads representing a small portion of the total flexural demand. For frames in which gravity loading produces significant flexural stresses in the members, locations of plastic hinge formation should be determined based on methods of plastic analysis.

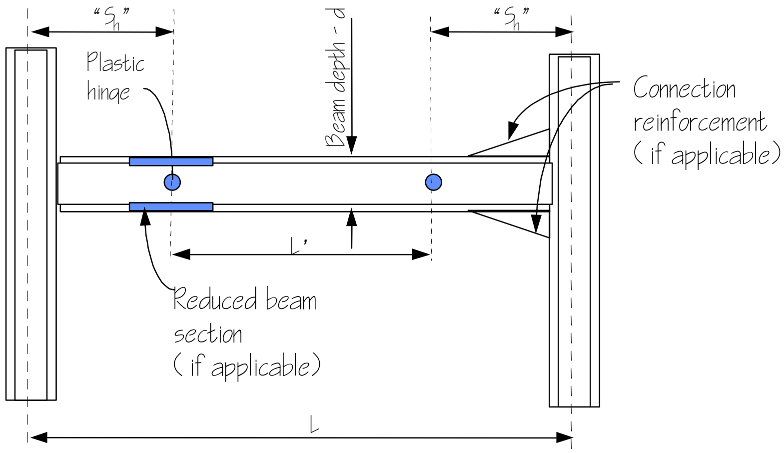


Figure 3-2 Location of Plastic Hinge Formation

Commentary: The suggested location for the plastic hinge, as indicated by the parameter s_h in the prequalification data, is valid only for frames with limited gravity loading present on the frame beams. If significant gravity load is present, this can shift the locations of the plastic hinges, and in the extreme case, even change the form of the collapse mechanism. If flexural demand on the girder due to gravity load is less than about 30% of the girder plastic capacity, this effect can safely be neglected, and the plastic hinge locations taken as indicated. If gravity demands significantly exceed this level then plastic analysis of the frame should be performed to determine the appropriate hinge locations.

3.2.4 Determine Probable Plastic Moment at Hinges

For fully restrained connections designed to develop plastic hinging in the beam or girder, the probable plastic moment at the location of the plastic hinge should be determined as:

$$M_{pr} = C_{pr} R_y Z_e F_y \tag{3-1}$$

where:

M_{pr} = probable peak plastic hinge moment,

C_{pr} = a factor to account for the peak connection strength, including strain hardening, local restraint, additional reinforcement, and other connection conditions. For most connection types, C_{pr} is given by the formula:

$$C_{pr} = \frac{F_y + F_u}{2F_y} \tag{3-2}$$

A value of 1.2 may be used for all cases, except where otherwise noted in the individual connection design procedures included with the prequalifications in later sections of these *Recommended Criteria*.

R_y = A coefficient, applicable to the beam or girder material, obtained from the 1997 *AISC Seismic Provisions*.

Z_e = The effective plastic modulus of the section (or connection) at the location of the plastic hinge.

F_y = the specified minimum yield stress of the material of the yielding element.

F_u = the specified minimum tensile stress of the material of the yielding element.

For connections that do not develop plastic hinges in the beam, the hinge strength should be calculated, for the pertinent yield mechanism as confirmed by tests, considering the variation in material properties of the yielding elements. For prequalified connections, calculation methods to determine the yield strengths of the various active mechanisms are given in Sections 3.5, 3.6, and 3.7.

Commentary: The 1997 AISC Seismic Provisions use the formulation $1.1R_yM_p$ *for calculation of the expected plastic moment capacity of a beam As described in FEMA-355D, State of the Art Report on Connection Performance, research has shown that, for most connection types, the peak moment developed is somewhat higher than the 1.1 factor would indicate. Therefore, in these Recommended Criteria, the factor* C_{pr} *is used for individual connections, with a default value of 1.2 applicable to most cases.*

3.2.5 Determine Shear at the Plastic Hinge

The shear at the plastic hinge should be determined by methods of statics, considering gravity loads acting on the beam. A free body diagram of that portion of the beam between plastic hinges is a useful tool for obtaining the shear at each plastic hinge. Figure 3-3 provides an example of such a calculation. For the purposes of such calculations, gravity load should be based on the load combinations indicated in Section 3.4.1.

3.2.6 Determine Strength Demands at Each Critical Section

In order to complete the design of the connection, including, for example, sizing the various plates, bolts, and joining welds which make up the connection, it is necessary to determine the shear and flexural strength demands at each critical section. These demands may be calculated by taking a free body of that portion of the connection assembly located between the critical section and the plastic hinge. Figure 3-4 demonstrates this procedure for two critical sections, for the beam shown in Figure 3-3.

Commentary: Each unique connection configuration may have different critical sections. The vertical plane that passes through the joint between the beam flanges and column (if such joining occurs) will typically define at least one such critical section, used for designing the joint of the beam flanges to the column. A second critical section occurs at the center line of the column. Moments calculated at this point are used to check strong-column-weak-beam and panel zone shear conditions. Other critical sections are described in the design procedures for each connection type.

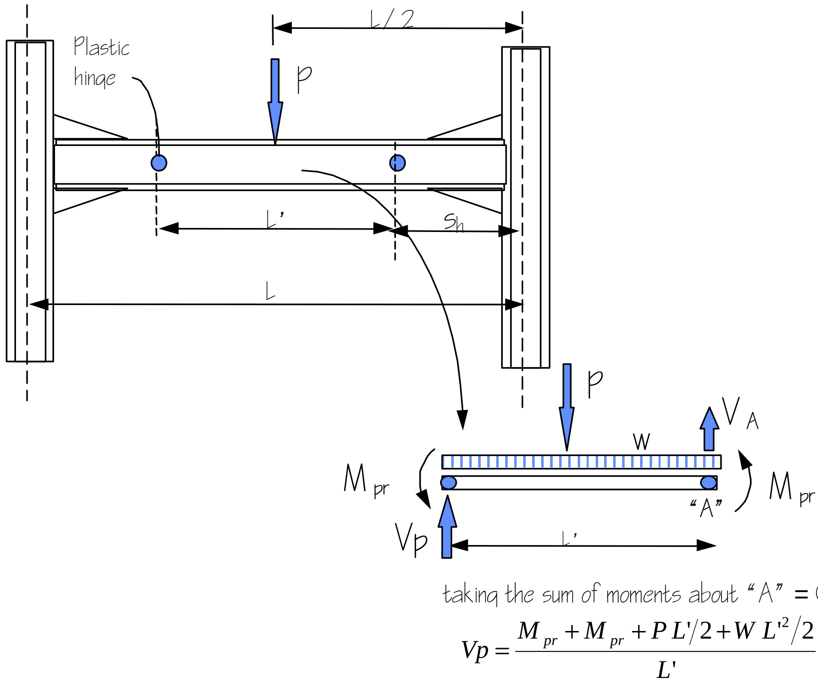


Figure 3-3 Sample Calculation of Shear at Plastic Hinge

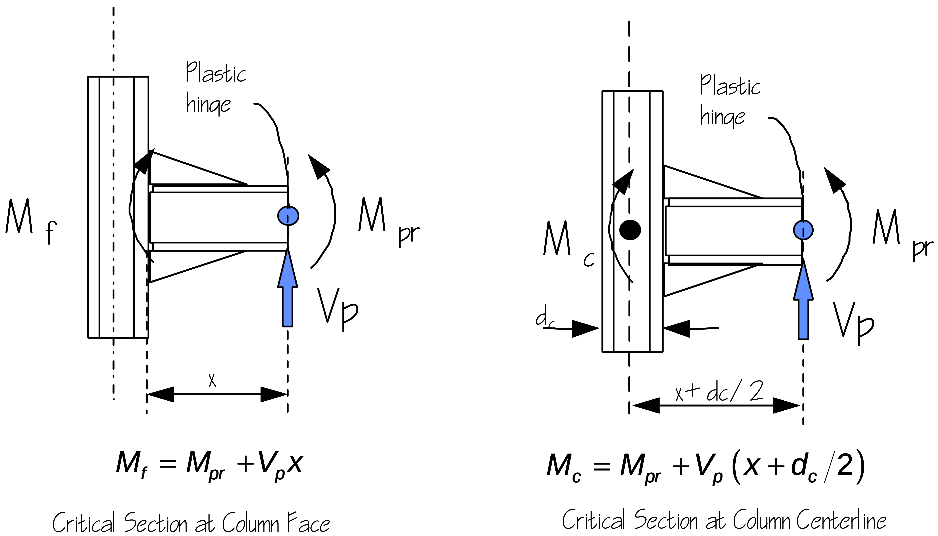


Figure 3-4 Calculation of Demands at Critical Sections

3.2.7 Yield Moment

The design procedures for some prequalified connections contained in these *Recommended Criteria* require that the moment at the face of the column at onset of plastic hinge formation, M_{yf}, be determined. M_{yf} may be determined from the following equation:

$$M_{yf} = C_y M_f \qquad (3\text{-}3)$$

where:

$$C_y = \frac{1}{C_{pr}\,\frac{Z_{be}}{S_b}} \tag{3-4}$$

C_{pr} = the peak connection strength coefficient defined in Section 3.2.5

S_b = the elastic section modulus of the beam at the zone of plastic hinging

Z_{be} = the effective plastic section modulus of the beam at the zone of plastic hinging.

3.3 General Requirements

This section provides criteria for connection design conditions that are considered to be general, that is, those conditions which, when they occur in a connection, are considered to perform in a similar way, or at least to have the same requirements for successful performance, irrespective of the connection type being used. The designer should employ these criteria in the design of all connection types, except when specific testing has been performed that qualifies the connection for use with different conditions, or unless otherwise specifically indicated in these *Recommended Criteria*.

3.3.1 Beams

3.3.1.1 Beam Flange Stability

Beam flange slenderness ratios $b_f/2t_f$ (*b/t*) should be limited to a maximum value of 52/ ÖF_y, as required by the 1997 *AISC Seismic Provisions*. For moment frame beams with RBS connections, it is recommended that the $b_f/2t_f$ be determined based on the flange width (b_f) measured at the ends of the center 2/3 of the reduced section of beam unless gravity loads are large enough to shift the hinge point significantly from the center point of the reduced section.

Commentary: The AISC Seismic Provisions require that beam flange slenderness ratios $b_f/2t_f$ (b/t) be limited to a maximum of 52/ ÖF_y. This specific value is intended to allow some plastic rotation of the beam to occur before the onset of local buckling of the flanges, a highly undesirable phenomenon. Buckling of most of the beam flanges in a moment resisting frame results in development of frame strength degradation increasing both story drifts and the severity of P-D effects and therefore should be avoided. Local flange buckling results in large local straining of the flanges and the early on-set of low-cycle fatigue induced tearing of the beam flanges, which ultimately limits the ability of the assembly to withstand cyclic inelastic rotation demands. Further, severely buckled beam flanges can be even more difficult to repair than fractured beam connections.

Notwithstanding the above, under large plastic rotation demands, buckling of beam flanges will inevitably occur. The value of the b/t of the beam involved in a specific connection can have a major effect on how the beam column assembly *performs. Beams and girders used in moment frames should comply with the limits specified by AISC, except as specifically modified by individual connection prequalifications or qualification tests. It should be noted that under this program, many assemblies with W30x99 beams conforming to ASTM A572 were tested. Although this section has* $b_f/2t_f$ *equal to 54/Ö*F_y *they performed acceptably.*

3.3.1.2 Beam Web Stability

Web height-to-thickness ratios, h_c/t_w for beams in moment resisting frames should not exceed $418/\sqrt{F_y}$.

Commentary: The 1997 AISC Seismic Provisions permit use of beams with web h_c/t_w *ratios as high as 520/Ö*F_y*, for beams without axial load. Most of the testing conducted in support of the development of these Recommended Criteria utilized either W30x99 or W36x150 beam sections. Both of these structural shapes have* h_c/t_w *ratios that conform to the recommended 418/Ö*F_y *ratio, as do nearly all commonly rolled shapes. Since many of the specimens exhibited significant web buckling in the area of plastic hinges, it is not considered prudent to utilize beams with thinner webs in moment resisting frames. Although stiffening of the webs could be done to limit web buckling, it is possible that such stiffeners could be detrimental to connection performance. Since connections with web stiffeners were not tested, such connections have not been prequalified. Refer to FEMA-355D, State of the Art Report on Connection Performance, for further discussion of web buckling of moment-frame beams.*

3.3.1.3 Beam Depth and Span Effects

The prequalified connections contained in Sections 3.5, 3.6, and 3.7 of these R*ecommended Criteria* are limited in application to specific beam depths and span-to-depth ratios. These limitations are noted in the tabulated data for each connection. For frames designed using project-specific connection qualifications, connection tests used in the connection qualification program should employ beams of similar or greater depth than those used in the frame and similar or smaller span-to-depth ratio.

Commentary: Both beam depth and beam span-to-depth ratio are significant in the inelastic behavior of beam-column connections. At a given induced curvature, deep beams will undergo greater straining than shallower beams. Similarly, beams with shorter span-to-depth ratio will have a sharper moment gradient across the beam span, resulting in reduced length of the beam participating in plastic hinging and increased strains under inelastic rotation demands. Most of the beam-column assemblies tested under this project used configurations approximating beam spans of about 25 feet and beam depths varying from W30 to W36 so that beam span-to-depth ratios were typically in the range of 8 to 10.

Additional information may be found in FEMA-355D, State of the Art Report on Connection Performance.

3.3.1.4 Beam Flange Thickness Effects

The prequalified connections contained in Sections 3.5, 3.6, and 3.7 of these *Recommended Criteria* are limited in application to specific beam flange thicknesses. These limitations are noted in the tabulated data for each connection. For frames designed using project-specific connection qualifications, connection tests used in the connection qualification program should employ beam flanges of similar or greater thickness than those used in the frame.

Commentary: In addition to controlling the stability of the flange under compressive loading, as described in Section 3.3.1.1, beam flange thickness also affects the size of welds in welded connections. Although it is not a given that larger welds will be less reliable than smaller welds, greater control may be necessary to assure their performance, and quality control may be more difficult. Additionally, residual stresses are likely to be higher in thicker material with thicker welds.

3.3.1.5 Lateral Bracing at Beam Flanges at Plastic Hinges

Plastic hinge locations that are remote from the column face in beams that do not support a slab should be provided with supplemental bracing, as required by the 1997 *AISC Seismic Provisions*. Where the beam supports a slab and is in direct contact with the slab along its span length, supplemental bracing need not be provided.

Commentary: The 1997 AISC Seismic Provisions require that beam flanges be braced at plastic hinge locations. Because plastic hinges have been moved away from the column face for some of the connection types in this section, a strict interpretation of the provisions would lead to a requirement that flanges at such hinges be laterally braced. Limited testing conducted as part of this project (FEMA-355D) suggests that, as long as the hinging beam is connected to a concrete slab, excessive strength deterioration due to lateral buckling will not occur within the ranges of drift angle normally considered important. Therefore, these Recommended Criteria do not require supplemental bracing of plastic hinge locations adjacent to column connections of beams supporting slabs.

For those cases where supplemental bracing of beam flanges near plastic hinges is appropriate, care must be taken in detailing and installation of bracing to assure that detrimental attachments are not made directly within the area of anticipated plastic behavior. This is because of the inherent risk of reducing plastic deformation capacity for the beam by introducing stress concentrations or metallurgical notches into the region of the beam that must undergo plastic straining. See FEMA-355D, State of the Art Report on Connection Performance, for further discussion of flange bracing.

3.3.1.6 Welded Shear Studs

Welded shear studs, or other attachments for composite action with slabs or for diaphragm shear transfer, should not be installed within the hinging area of moment frame beams. The hinging area is defined as the distance from the column flange face to one half the beam depth beyond the theoretical hinge point. Shot-in, or screwed attachments should not be permitted in this area either.

Commentary: It has been shown in some tests that welded shear studs and the rapid increase of section caused by composite action can lead to beam flange fractures when they occur in the area of the beam flange that is undergoing large cyclic strains. It is not certain whether the welding of the studs, the composite action, or a combination of the two is the cause, but, based on the limited evidence, it is judged to be prudent to permit no studs in the hinging area. It is also prudent to permit no attachments, which involve penetration of the flanges in the hinging region.

3.3.2 Welded Joints

3.3.2.1 Through-Thickness Strength

The through-thickness strength of column material conforming to *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications*, need not be explicitly checked in connection design, except where required by the design procedures for a specific prequalified connection.

Commentary: Early investigations of connection fractures in the 1994 Northridge earthquake identified a number of fractures that appeared to be the result of inadequate through-thickness strength of the column flange material. As a result of this, in the period immediately following the Northridge earthquake, a number of recommendations were promulgated that suggested limiting the value of through-thickness stress demand on column flanges to a value of 40 ksi, applied to the projected area of the beam flange attachment. This value was selected to ensure that through-thickness yielding did not initiate in the column flanges of fully restrained connections and often controlled the overall design of a connection subassembly.

It is important to prevent the inelastic behavior of connections from being controlled by through-thickness yielding of column flanges. This is because it would be necessary to develop very large local ductilities in the column flange material in order to accommodate even modest plastic rotation demands on the assembly. However, the actual cause for the fractures that were initially identified as through-thickness failures of the column flange are now believed to be unrelated to this material property. Rather, it appears that this damage occurred when fractures initiated in defects present in the CJP weld root, not in the flange material (FEMA-355E). These defects sometimes initiated a crack, that *under certain conditions, propagated into the column flange, giving the appearance of a through-thickness failure. Detailed fracture mechanics investigations conducted under this project confirm that damage initially identified as through thickness failures are likely to have occurred as a result of certain combinations of material strength notch toughness, conditions of stress in the connection, and the presence of critical flaws in the welded joint.*

As part of the research conducted in support of the development of these Recommended Criteria, extensive through-thickness testing of modern steels, meeting the ASTM A572, Gr. 50 and ASTM A913, Gr. 65 specifications has been conducted to determine the susceptibility of modern column materials to through thickness failures (FEMA-355A, State of the Art Report on Base Metals and Fracture). This combined analytical and laboratory research clearly shows that due to the restraint inherent in welded beam flange to column flange joints, the through thickness yield and tensile strengths of the column material is significantly elevated in the region of the connection. Further, for the modern materials tested, these strengths significantly exceed those that can be delivered to the column by beam material conforming to these same specifications. For this reason, no limits are suggested for the through thickness strength of modern steel materials.

3.3.2.2 Base Material Toughness

Material in rolled shapes with flanges 1-1/2 inches or thicker, and sections made from plates that are 2 inches or thicker, should be required to have minimum Charpy V-notch (CVN) toughness of 20 ft-lbs at 70° F. Refer to *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications*.

Commentary: The 1997 AISC Seismic Provisions specify minimum notch toughness for rolled shapes with flanges 1-1/2 inches thick or thicker and sections made from plates 1-1/2 inches thick or thicker. These Recommended Criteria relax the requirement for toughness of plate material to apply to plates 2 inches or thicker as this was the original intent of the 1997 AISC specification, and it is believed that the AISC document will be revised to this requirement.

Research has not clearly demonstrated the need for a specific value of base metal notch toughness. However, it is judged that base metal toughness is important to prevention of brittle fracture of the base metal in the highly stressed areas of the connection. A number of connection assemblies that have been tested have demonstrated base metal fractures at weld access holes and at other discontinuities such as at the ends of cover plates. In at least some of these tests, the fractures initiated in zones of low notch toughness. Tests have not been conducted to determine if higher base metal notch toughness would have reduced the incidence of such fractures.

The CVN value of 20 ft.-lbs at 70° F recommended here was chosen because it is usually achieved by modern steels, and because steels meeting this criterion have been used in connections which have performed successfully. Since current studies (FEMA-355A, State of the Art Report on Base Metals and Fracture) have indicated that rolled shapes produced from modern steels meet this requirement almost routinely, even in the thicker shapes that currently require testing, it has been suggested that the requirement for this testing could be eliminated and replaced by a certification program administered by the mills. However, such a program is not currently in existence. Until such time as such a certification program is in place, or a statistically meaningful sampling from all major mills has been evaluated, it is recommended that the AISC requirement for testing be continued. According to the Commentary to the 1997 AISC Seismic Provisions, thinner sections are judged not to require testing because they "are generally subjected to enough cross-sectional reduction during the rolling process that the resulting notch toughness will exceed that required." In other words, the toughness is desired, but testing to verify it on a project basis is not judged to be necessary as it is routinely achieved.

3.3.2.3 k-Area Properties

The k-area of rolled wide-flange shapes, which may be considered to extend from the mid-point of the radius of the fillet from the flange into the web, approximately 1 to 1-1/2 inches beyond the point of tangency between the fillet and web, as defined in Figure C-6.1 of 1997 *AISC Seismic Provisions*, is likely to have low toughness and may therefore be prone to cracking caused by welding operations. Designers should detail welds of continuity plates and web doubler plates in columns in such a way as to avoid welding directly in the k-area. Refer to Section 3.3.3 for more information.

Fabricators should exercise special care when making welds in, or near to, the k-area. Where welding in the k-area of columns cannot be avoided, special nondestructive testing is recommended. Refer to *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications.*

Commentary: Recent studies, instigated in response to fabrication problems have shown that, for rotary-straightened W-shapes, an area of low material toughness can occur in the region of the web immediately adjacent to the flange. In some instances, cracking has occurred in these areas during welding. The Commentary to the 1997 AISC Seismic Provisions provides a figure (Fig. C-6.1) that defines the k-area.

The low toughness of the k-area seems to be associated only with rotary-straightened sections. Which sections are rotary straightened varies among the mills. One major domestic supplier rotary-straightens all shapes weighing less than 150 pounds per linear foot. Larger sections are often straightened by other means that do not result in as much loss of toughness in the k-area. Because *rolling mill practice is frequently changed, it is prudent to assume that all rolled sections are rotary-straightened.*

3.3.2.4 Weld Metal Matching and Overmatching

The use of weld metal with tensile strength that is significantly less than the expected strength of the connected base steel material is not recommended. Welding consumables specified for CJP groove welds of beam flanges and flange reinforcements should have yield and tensile strengths that are approximately the same as, or slightly higher than, the expected yield and tensile strength of the beam or girder flanges being welded. Significant overmatching of weld metal should not be required unless overmatching is specified in the connection prequalification or is used in the prototypes tested for project-specific qualification of the connection being used. Flux-Cored Arc Welding and Shielded Metal Arc Welding filler metals commonly used in structural construction and conforming to the E70 specifications provide adequate properties for joining most material conforming to ASTM A36, A572, Grades 42 and 50, A913 Grade 50 and A992. Weld splices of columns conforming to ASTM A913, Grade 65 steel should be made with filler metals capable of depositing weld metal with a minimum tensile strength of 80 ksi.

Commentary: Undermatched weld metals, that is, weld metals with lower strength than the connected base metals, are beneficial in some applications in that they tend to limit the residual stress state in the completed joint. This can be achieved by employing balanced, or slightly undermatched filler metals. However, in applications where yield level stresses are anticipated, it is desirable to minimize the amount of plasticity in the welded joint. This can be achieved by employing balanced, or slightly overmatched filler metals. There is significant variation in the yield and tensile strengths of typical structural steels. Although E70 filler metals will produce matching or slightly overmatching conditions for most structural steel conforming to grade 36 and grade 50 specifications, they will not always provide these conditions. The new A992 specification for grade 50 structural steel has controlled upper limits on the strength of the material and should produce, with E70 filler metals, more closely matching conditions in most cases. Notwithstanding the above, the majority of the successful connection tests performed under this project have used weld metals with yield and tensile strengths in the nominal range of 58 and 70 ksi respectively, and these have performed in an adequate manner. Therefore, it is believed that the use of E70 filler metals with grade 50 structural steels is acceptable. For additional information, refer to FEMA-355B, State of the Art Report on Welding and Inspection.

3.3.2.5 Weld Metal Toughness

For structures in which the steel frame is normally enclosed and maintained at a temperature of 50^{o} F or higher, critical welded joints in seismic-force-resisting systems, including CJP welds of beam flanges to column flanges, CJP groove welds of shear tabs and beam webs to column flanges, column splices, and similar joints, should be made with filler metal providing CVN toughness of 20ft-lbs at -20° F and 40ft-lbs at 70° F and meeting the Supplemental Toughness Requirements for Welding Materials included in *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications*. For structures with lower service temperatures than 50° F, these qualification temperatures should be reduced accordingly.

Commentary: Principles of fracture mechanics demonstrate the importance of notch toughness to resist fracture propagation from flaws, cracks, and backing bars or other stress concentrations that may be pre-existing or inherent, or that may be caused by applied or residual stresses. The 1997 AISC Seismic Provisions require the use of welding consumables with a rated Charpy V-Notch toughness of 20 ft.-lbs. at -20° F, for Complete Joint Penetration groove welds used in the seismic-force-resisting system. The 1997 AISC Seismic Provisions, Supplement No. 1, February 15, 1999 (AISC, 1999), changes this requirement to include "all welds used in primary members and connections in the Seismic-Force-Resisting System". The rating of the filler metal is as determined by AWS classification or manufacturer certification.

Studies conducted under this project have indicated that not all weld consumables that are rated for 20 ft-lbs of toughness at –20°F will provide adequate toughness at anticipated service temperatures. The supplemental toughness requirements contained in FEMA-353 are recommended to ensure that weld metal of adequate toughness is obtained in critical joints.

Most of the beam-column connection tests conducted under this project were made with filler metal conforming to either the E70T6 or E70TGK2 designations. These filler metals generally conform to the recommended toughness requirements. Other filler metals may also comply.

3.3.2.6 Weld Backing, Weld Tabs, and Other Welding Details

Weld backing and runoff tabs should be removed from CJP flange welds, unless otherwise noted in the connection prequalification or demonstrated as not required by project-specific qualification testing. Refer to *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications*, for special requirements for weld backing, weld tabs and other welding details for moment-frame joints.

Commentary: It was originally hypothesized, following the 1994 Northridge earthquake that weld backing created an effective crack equal to the thickness of the backing and that this phenomena was responsible for many of the fractures that had occurred. Finite element analyses of welded joints (Chi, et. al., 1997) have shown that although the backing does create some notch effect, far more significant is the fact that when backing is left in place, it obscures effective detection of significant flaws that may exist at the weld root. These flaws represent a significantly more severe notch condition than does the backing itself.

It is recommended that backing be removed from beam bottom-flange joints, to allow identification and correction of weld root flaws. This is not required for top-flange joints because the stress condition at the root of the top flange weld is less critical and less likely to result in initiation of fracture, even if some weld root flaws are present. Also, as a result of the more favorable position, it is far less likely that significant flaws will be incorporated in top-flange joints. Special welding of backing for top-flange welds is recommended.

Weld tabs represent another source of potential discontinuity at the critical weld location. Additionally, the weld within the weld tab length is likely to be of lower quality and more prone to flaws than the body of the weld. Flaws in the runoff tab area can create stress concentrations and crack starters and for this reason their removal is recommended. It is important that the process of removal of the weld tabs not be, of itself, a cause of further stress concentrations, and therefore, FEMA-353 requires that the workmanship result in smooth surfaces, free of defects.

3.3.2.7 Weld Access Holes

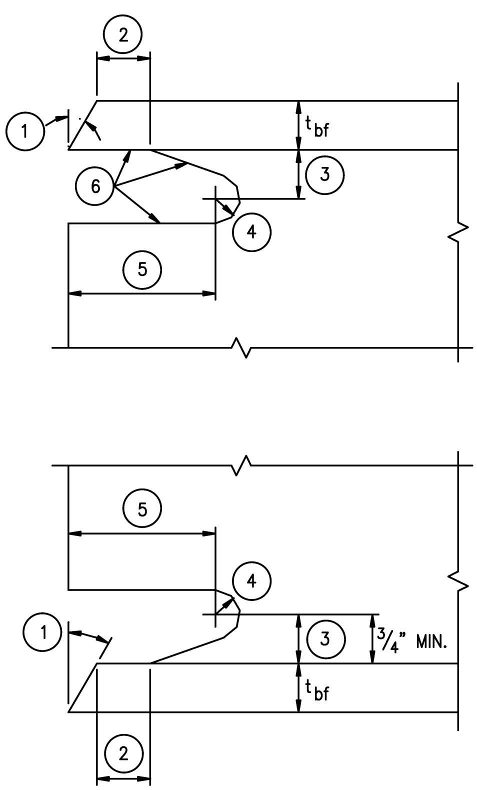
New welded moment-resisting connections should utilize weld access hole configurations as shown in Figure 3-5, except where otherwise noted in specific connection prequalifications. Criteria for forming and finishing of weld access holes are provided in *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications.*

Commentary: The size, shape and workmanship of weld access holes can affect the connection in several different ways. If the hole is not large enough, this restricts welder access to the joint and increases the probability of low quality joints. Depending on the size and shape of the weld access hole, plastic strain demands in the welded joint and in the beam flange at the toe of the weld access hole can be significantly affected. Laboratory tests of unreinforced connections fabricated with tough filler metals have indicated that these connections frequently fail as a result of low cycle fatigue of the beam flange material at the toe of the weld access hole, resulting from the strain concentrations introduced by this feature. The configuration shown in Figure 3-5 was developed as part of a program of research conducted in support of the development of these Recommended Criteria and appears to provide a good balance between adequate welder access and minimization of stress and strain concentration. For further discussion of weld access holes, see FEMA-355D, State of the Art Report on Connection Performance.

3.3.2.8 Welding Quality Control and Quality Assurance

FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications, includes recommendations for quality control and quality assurance for steel moment frames and connections intended for seismic applications. Recommended inspections are divided into two categories: Process and Visual Inspection, and Nondestructive Testing. For each category, different levels of quality assurance measures are specified depending on the anticipated severity of loading, or demand (Seismic Weld Demand Category) and the consequence of welded joint failure (Seismic Weld Consequence Category). All welded joints in the seismic-force-resisting system should be categorized according to the applicable Consequence and Demand Categories, using the following form: "QC/QA Category – BH/T", where the first letter (in this case B) indicates the Demand Category, the second letter (in this case H) indicates the Consequence Category and the third letter, either T or L, indicates that primary loading is either transverse or longitudinal, respectively. The various categories are described in detail in the referenced documents. For the prequalified connections included in these *Recommended Criteria*, the appropriate categories have been preselected and are designated in the information accompanying the prequalfication.

olerances shall not
ccumulate to the extent
at the angle of the access
ole cut to the flange
urface exceeds 25°.



Notes:

1. Bevel as required by *AWS D1.1* for selected groove weld procedure.

2. Larger of t_{bf} or ½ inch. (plus ½ t_{bf}, or minus ¼ t_{bf})
3. ¾ t_{bf} to t_{bf}, ¾" minimum (± ¼ inch)
4. 3/8" minimum radius (plus not limited, or minus 0)
5. 3 t_{bf}. (± ½ inch)
6. See *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications,* for fabrication details including cutting methods and smoothness requirements.

Figure 3-5 Recommended Weld Access Hole Detail

Commentary: FEMA-353 describes the Demand (A,B,C) and Consequence (H,M,L) Categories and indicates the appropriate levels of Visual and NDT inspection for each combination of demand and consequence category. The degree of inspection recommended is highest for the combination of high demand (Category A) with high consequence (Category H) welded joints, and conversely, less inspection is required for low demand (Category C) with low consequence (Category L) welded joints, with intermediate levels for categories in between.

3.3.3 Other Design Issues for Welded Connections

3.3.3.1 Continuity Plates

Unless project-specific connection qualification testing is performed to demonstrate that beam flange continuity plates are not required, moment-resisting connections should be provided with beam flange continuity plates across the column web when the thickness of the column flange is less than the value given either by Equation 3-5 or 3-6:

$$t_{cf} < 0.4\sqrt{1.8 b_f t_f \frac{F_{yb} R_{yb}}{F_{yc} R_{yc}}} \tag{3-5}$$

$$t_{cf} < \frac{b_f}{6} \tag{3-6}$$

where:

t_{cf} = minimum required thickness of column flange when no continuity plates are provided, inches

b_f = beam flange width, inches

t_f = beam flange thickness, inches

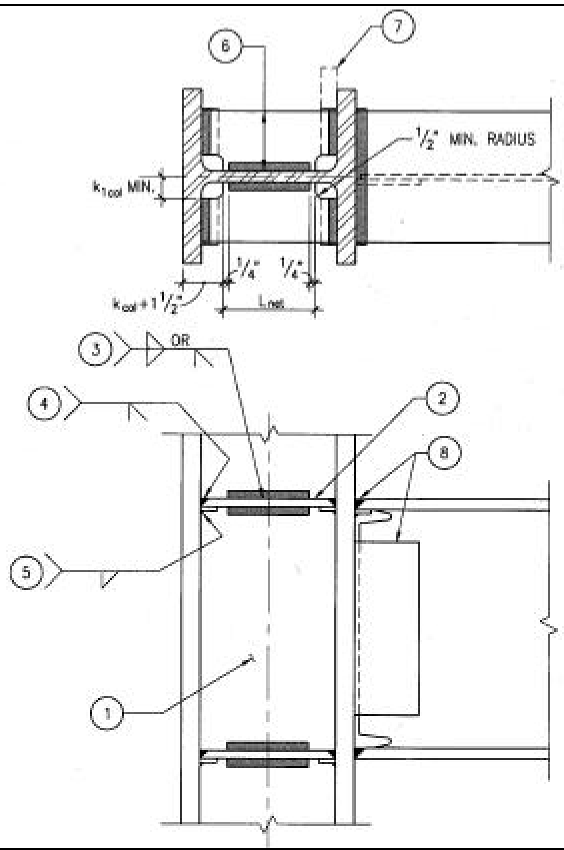
F_{yb} (F_{yc}) = Minimum specified yield stress of the beam (column) flange, ksi

R_{yb} (R_{yc}) = the ratio of the expected yield strength of the beam (column) material to the minimum specified yield strength, as in the 1997 *AISC Seismic Provisions*.

Where continuity plates are required, the thickness of the plates should be determined according to the following:

- For one-sided (exterior) connections, continuity plate thickness should be at least one-half of the thickness of the two beam flanges.
- For two-sided (interior) connections, the continuity plates should be equal in thickness to the thicker of the two beam flanges on either side of the column.
- The plates should also conform to Section K1.9 of *AISC-LRFD Specifications*.

Continuity Plates should be welded to column flanges using CJP groove welds as shown in Figure 3-6. Continuity plates should be welded to the web as required to transmit the shear capacity of the net length of the continuity plate.



Notes

1. Web doubler plate where required by Section 3.3.3.2. See 1997 *AISC Seismic Provisions* Section 9.3c, Commentary C9.3, and Figures C-9.2 and C-9.3 for options and connection requirements. Weld QC/QA Category BL/L for all welds.
2. Continuity plate as required by Section 3.3.3.1.
3. Required total weld strength = $0.6t_{pl}(L_{net})F_{y\,pl}$. QC/QA Category BL/L.
4. CJP typical. QC/QA Category BM/T. For exterior beam-column connections (beam one side only), weld of continuity plate to column flange at free side may be fillet welds at top and bottom face of plate.
5. AISC minimum continuous fillet weld under backing.
6. Minimum width to match beam flange. Preferred alternative: extend plate flush with column flanges.
7. Remove weld tabs to ¼” maximum from edge of continuity plate. Grind end of weld smooth (500 μ-in), not flush. Do not gouge column flange.
8. Beam connection, see Figures 3-7 through 3-20.

Figure 3-6 Typical Continuity and Doubler Plates

Commentary: Following the 1994 Northridge earthquake, some engineers postulated that the lack of continuity plates was a significant contributing factor to the failure of some connections. This was partially confirmed by initial tests conducted in 1994 in which several specimens without continuity plates failed while some connections with these plates successfully developed significant ductility. Based on this, FEMA-267 recommended that all connections be provided with continuity plates. The 1997 AISC Seismic Provisions, published after FEMA-267, relaxed this criteria and state that continuity plates should be provided to match those in connections tested to obtain qualification.

Research conducted in this project suggests that where the flange thickness of columns is sufficiently thick, continuity plates may not be necessary. Equation 3-5 was the formula used by AISC to evaluate column flange continuity plate requirements prior to the Northridge earthquake. It appears that this formula is adequate to control excessive column flange prying as long as the beam flanges are not too wide. Studies reported in FEMA-355D suggest that the ratio of beam-flange width to column-flange thickness is also important. Tests with a ratio of 5.3 (W36x150 beam with W14x311 column) showed little difference in performance with or without continuity plates, while tests with a ratio of 6.8 (W36x150 beam with W27x258 column) showed some difference of performance. The factor of 6 in Equation 3-6 was selected, based on these tests and engineering judgement.

3.3.3.2 Panel Zone Strength

Moment-resisting connections should be proportioned either so that shear yielding of the panel zone initiates at the same time as flexural yielding of the beam elements or so that all yielding occurs in the beam. The following procedure is recommended:

Step 1: Calculate t, the thickness of the panel zone that results in simultaneous yielding of the panel zone and beam from the following relationship:

$$t = \frac{C_y M_c \frac{h-d_b}{h}}{(0.9)0.6F_{yc}R_{yc}d_c\left(d_b - t_{fb}\right)} \tag{3-7}$$

where:

h = the average story height of the stories above and below the panel zone.

R_{yc} = the ratio of the expected yield strength of the column material to the minimum specified yield strength, in accordance with the 1997 *AISC Seismic Provisions*.

M_c and C_y are the coefficients defined in Section 3.2.6 and Section 3.2.7, respectively, and other terms are as defined in *AISC-LRFD*

Step 2: If t, as calculated, is greater than the thickness of the column web, provide doubler plates, or increase the column size to a section with adequate web thickness.

Where doubler plates are required, the thickness should be determined as described above, and they should be proportioned and welded as described in the 1997 *AISC Seismic Provisions*. QC/QA Category BL/L procedures are defined in FEMA-353. For connections designed using project-specific qualifications, the panel zone strength should match that of the tested connections.

Commentary: Several aspects of the methodology for the design of panel zones, as contained in the 1997 AISC Seismic Provisions, are considered to require revision, based on studies conducted by this project. As described in FEMA-355D, the best performance is likely to be achieved when there is a balance of beam bending and panel zone distortion. The equations given are intended to provide panel zones that are just at the onset of yielding at the time the beam flange begins to yield.

The procedure recommended in these Recommended Criteria differs significantly from that contained in the 1997 AISC Seismic Provisions, but the results are not dramatically different. For most column sizes results will be similar to methods used in the past. For columns with thick flanges, the methods herein will result in the need for moderately thicker panel zones than in the past.

3.3.3.3 Connections to Column Minor Axis

Connections to the minor axis of a column should be qualified by testing following the procedures of Section 3.9. If minor-axis connections are to be used in conjunction with major-axis connections at the same column, the testing program should include biaxial bending effects at the connection.

Commentary: In general, the prequalified connections have not been tested for use with columns oriented so that beams connect to the minor axis of the column. Two tests of Reduced Beam Section connections in this orientation were conducted, which indicated good performance. These tests were conducted to provide a general indication of the possible performance of weak axis connections, but are not considered to comprise a sufficient database for prequalification of such connections.

3.3.3.4 Attachment of Other Construction

Welded or bolted attachment for exterior facades, partitions, ductwork, piping, or other construction should not be placed in the hinging area of moment frame beams. The hinging area is defined as one half of the beam depth on either side of the theoretical hinge point as described in the prequalification data table for each connection detail. It is recommended that bolt holes for this type of construction not be permitted between the face of the column and six inches, minimum, beyond the extreme end of the hinging area. Outside of the described area, a calculation should be made to assure sufficient net section to avoid fracture, based on moments calculated using the expected moment at the hinge point. Welding between the column face and the near edge of the hinging area should be carefully controlled to avoid creation of stress concentrations and application of excessive heat. Specifications and drawings should clearly indicate that anchorage shall not be made in the areas described and this should be coordinated with the architect and other members of the design team.

Commentary: It is common for precast panels and other facade elements, as well as other construction, to be anchored to members of the steel frame through the use of welds, bolts, powder-driven fasteners, or other fasteners. Such anchorage is often not considered by the engineer and is not performed with the same care and quality control as afforded the main building structure. Such anchorage, made in an area of high stress, can create stress concentrations leading to potential fracture.

3.3.4 Bolted Joints

The 1997 *AISC Seismic Provisions* contain requirements for bolted joints used in seismic-force-resisting systems. These requirements should be followed, as supplemented by the specific requirements given in the individual design procedures provided for the prequalified bolted connections, or where special bolting requirements are used in project-specific tested connections. QA/QC requirements for bolted joints are given in *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications*. Where these *Recommended Criteria* require, or permit, the use of bolts conforming to ASTM A325, an acceptable alternative is twistoff bolts conforming to ASTM F1852.

3.4 Prequalified Connections – General

Prequalified connection details are permitted to be used for moment frame connections for the types of moment frames and ranges of the various design parameters indicated in the limits accompanying each prequalification. Project-specific testing should be performed to demonstrate the adequacy of connection details that are not listed herein as prequalified, or are used outside the range of parameters indicated in the prequalification. Designers should follow the procedures outlined in Section 3.9 for qualification of nonprequalified connections.

Commentary: The following criteria were applied to connections listed as prequalified:

1. *There is sufficient experimental and analytical data on the connection performance to establish the likely yield mechanisms and failure modes for the connection.*

2. *Rational models for predicting the resistance associated with each mechanism and failure mode have been developed.*

3. *Given the material properties and geometry of the connection, a rational procedure can be used to estimate which mode and mechanism controls the behavior and the deformation capacity (that is, interstory drift angle) that can be attained from the controlling conditions.*

4. *Given the models and procedures, the existing data base is adequate to permit assessment of the statistical reliability of the connection.*

Some of the connections in the following sections are only prequalified for use in Ordinary Moment Frames, while others are prequalified for both Ordinary Moment Frame (OMF) and Special Moment Frame (SMF) use. In general, when a connection is qualified for use in SMF systems, it is also qualified for use in OMF systems with fewer restrictions on size, span, and other parameters than are applied to the SMF usage. For SMF application, very little extrapolation beyond the parametric values for which testing has been performed has been applied. For OMF application, some judgement has been applied to permit extrapolation for OMFs, based on the significantly lower rotational demands applicable to those systems.

Some connection types for which extensive testing has been performed have not been included as prequalified for new buildings. These include the following:

1. *Welded Cover Plated Flange (WCPF);*
2. *Welded Bottom Haunch (WBH);*
3. *Welded Top and Bottom Haunch (WTBH).*

In general, these connections are not included because they do not have any significant advantages in performance over connections that are much simpler and cost-effective. The haunched connections in particular were not studied in detail by this project, because they were not considered to be economically practical for application in new buildings. Consequently, the data base for this connection type is insufficient to permit prequalification. WCPF connections, similarly, are relatively expensive, and, although there is a fairly large data base of tests, many of them successful, there have also been some significant brittle failures of this type of connection. The fact that these connections are not listed in this guideline as prequalified is not intended to preclude their use, nor to suggest that those structures for which they have been used previously are not expected to exhibit acceptable performance. Rather, it is believed that for new construction there are connections which are equally or more reliable, yet less costly. For those desiring to use the above listed connections, information is provided in FEMA-355D, State of the Art Report on Connection Performance, *and in the specific laboratory test reports referenced therein. Design procedures are given in FEMA-351 for some of these connections for use in upgrading existing buildings.*

Prequalified connections are also recommended for use without further testing in structures having dual systems, as defined in FEMA-302, provided that attachment of bracing to the connection does not inhibit or alter the yield mechanism for the assembly.

3.4.1 Load Combinations and Resistance Factors

Design procedures for prequalified connection upgrades contained in Sections 3.5, 3.6 and 3.7 of this document are formatted on an expected strength basis, as opposed to either a Load and Resistance Factor Design basis or Allowable Stress Design basis. Loading used in these design formulations is generally calculated on the basis of the stresses induced in the assembly at anticipated yielding of the beam-column connection assembly. Where these design procedures require that earthquake loading be applied simultaneously with dead and live loading, the applicable load combinations of the 1997 *AISC Seismic Provisions* apply. Resistance factors should not be applied except as specifically required by the individual design procedure.

3.5 Prequalified Welded Fully Restrained Connections

This section provides prequalification data and design procedures for alternative types of welded, fully restrained, steel moment-frame connections, suitable for use in new construction. Table 3-1 indicates the various types of prequalified fully restrained connections, and the structural systems for which they are prequalified. Additional prequalification data on these connections are provided in the following sections.

Table 3-1 - Prequalified Welded Fully Restrained Connections

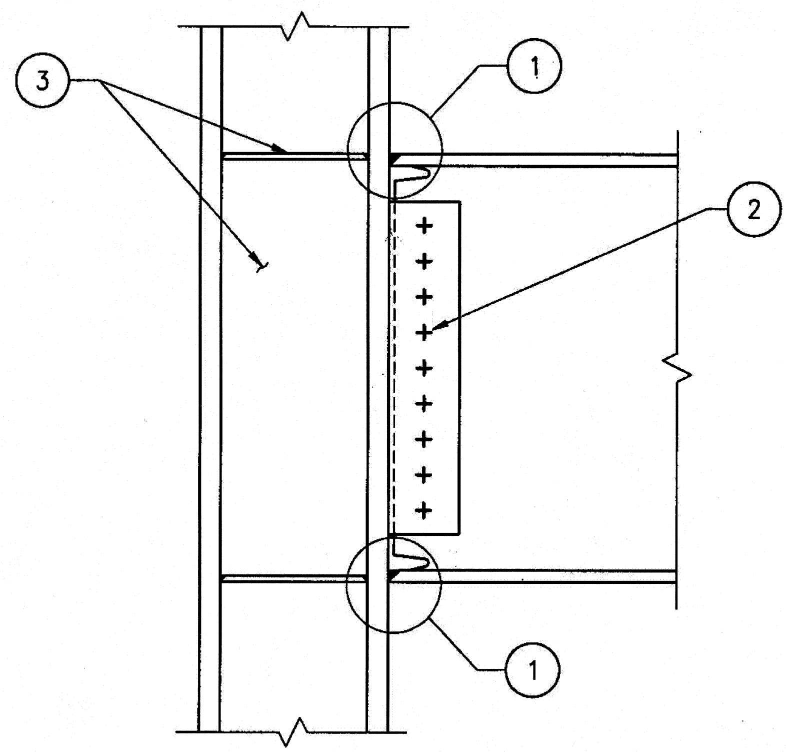
Connection Type	Criteria Section	Frame Type
Welded Unreinforced Flanges – Bolted Web (WUF-B)	3.5.1	OMF
Welded Unreinforced Flanges – Welded Web (WUF-W)	3.5.2	OMF, SMF
Free Flange (FF)	3.5.3	OMF, SMF
Reduced Beam Section (RBS)	3.5.4	OMF, SMF
Welded Flange Plate (WFP)	3.5.5	OMF, SMF

Commentary: FEMA-355D, State of the Art Report on Connection Performance, provides extensive information on the testing and performance of these connections, as well as others, that is not repeated in this document. The data presented in FEMA-355D have been prepared in support of the development of prequalification performance data, design procedures and limitations on design parameters for these connections. The design recommendations contained in FEMA-355D will not in all cases be identical to those contained herein. In some *cases, the format, notation, and context of the design formulae contained in FEMA-355D have been modified to provide for consistent application within the design procedures of these Recommended Criteria.*

3.5.1 Welded Unreinforced Flange – Bolted Web Connections

This section provides recommended criteria for design of fully restrained, Welded Unreinforced Flange – Bolted Web (WUF-B) connections. This type of connection is prequalified only for Ordinary Moment Frame applications, and within the parameters given in Table 3-2.

WUF-B connections utilize complete joint penetration (CJP) groove welds, meeting the requirements of *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment Frame Construction for Seismic Applications,* to join beam or girder flanges directly to column flanges. In this type of connection, no element other than weld metal, is used to join the flanges. Weld access holes are configured as indicated in Section 3.3.2.7. Web joints for these connections are made with slip-critical, high-strength bolts connecting the beam web to a shear tab that is welded to the column flange. Figure 3-7 provides a typical detail for this connection type. These connections should be designed in accordance with the criteria of this section.



Notes

1. See Figure 3-8 and Note 1 of Figure 3-8 for top and bottom flange weld requirements. QC/QA category AH/T. Refer to Figure 3-5 for weld access hole detail.
2. Bolted shear tab. Use pretensioned A325 or A490 bolts. Weld to column flange with fillet weld both

sides, or with CJP weld, to develop full shear strength of plate. Weld QC/QA Category BL/T.
3. See Figure 3-6 for continuity plate and web doubler plate requirements.

Figure 3-7 Welded Unreinforced Flange – Bolted Web (WUF-B) Connection

Commentary: This connection closely resembles the "prescriptive connection" commonly in use prior to the 1994 Northridge earthquake. After significant study, it has been concluded that with several improvements and appropriate levels of quality assurance with regard to workmanship and materials, this connection can perform reliably in frames designed as Ordinary Moment Frames (OMF) within the limitations indicated in Table 3-2.

The improvements incorporated in this connection over typical connections detailed prior to the 1994 Northridge earthquake include the following:

1. *Weld metal with appropriate toughness;*
2. *Removal of weld backing from bottom-beam-flange-to-column-flange welds, back-gouging and addition of a reinforcing fillet weld;*
3. *Use of improved weld access hole shape and finish;*
4. *Improvements to weld quality control, and quality assurance requirements and methods.*

Table 3-2 Prequalification Data WUF-B Connections

General:	
Applicable systems	Ordinary Moment Frame
Hinge location distance s_h	$d_c/2 + d_b/2$
Critical Beam Parameters:	
Maximum depth	W36 and shallower
Minimum span-to-depth ratio	7
Flange thickness	1" maximum
Permissible material specifications	A572 Grade 50, A992, A913 Grade 50/S75
Critical Column Parameters:	
Depth	W8, W10, W12, W14
Permissible material specifications	A572 Grade 50; A913 Grade 50 and 65; A992
Beam/Column Relations:	
Panel Zone strength	Section 3.3.3.2
Column/beam bending strength	No Requirement (OMF)
Connection Details	
Web connection	Shear tab welded to column, bolted to beam.
Continuity plate thickness	Section 3.3.3.1
Flange welds	See Fig. 3-8 and Section 3.3.2.5

Welding parameters	Section 3.3.2.4, 3.3.2.5, 3.3.2.6
Weld access holes	Section. 3.3.2.7

For best performance of this connection some limited panel zone yielding is beneficial. For this reason, it is recommended that panel zones not be over-reinforced.

3.5.1.1 Design Procedure

Step 1: Calculate M_{pr}, at hinge location, s_h, according to methods of Section 3.2.4.

Step 2: Calculate V_p, at hinge location, s_h, according to methods of Section 3.2.5.

Step 3: Calculate M_c, M_f, and C_y as described in Section 3.2.6 and 3.2.7.

Step 4: Calculate the required panel zone thickness using the procedures of Section 3.3.3.2.

Step 5: Calculate the connection shear as :

$$V_f = \frac{2M_f}{L - d_c} + V_g \qquad (3\text{-}8)$$

where:

V_f = maximum shear at the column face, kips
V_g = shear at the column face due to factored gravity loads, kips.

Step 6: Design the shear tab and bolts for V_f. Bolts should be designed for bearing, using a resistance factor f of unity

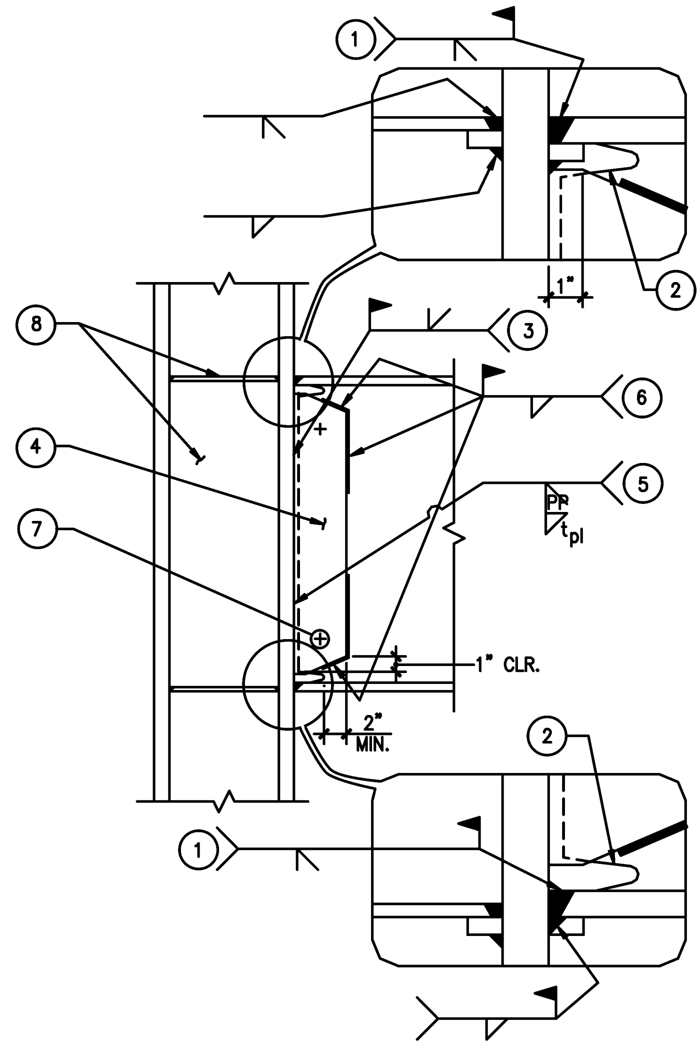
Step 7: Check requirements for continuity plates according to Section 3.3.3.1.

Step 8: Detail the connection as shown in Figure 3-7 and Note 1 of Figure 3-8.

3.5.2 Welded Unreinforced Flange – Welded Web Connections

This section provides guidelines for design of fully restrained, Welded Unreinforced Flange – Welded Web (WUF-W) connections. This type of connection is prequalified for use in Ordinary Moment Frame and Special Moment Frame systems within the parameters given in Table 3-3.

These connections utilize complete joint penetration (CJP) groove welds, meeting the requirements of *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications*, to join beam flanges or girder flanges directly to column flanges. In this type of connection, no reinforcement is provided except for the addition of a fillet weld applied to the groove weld. Web joints for these connections are made with complete joint penetration groove welds of the beam web to the column flange. Weld access holes for this type of connection should be in accordance with Section 3.3.2.7. Figure 3-8 provides a typical detail for this connection type. These connections should be designed in accordance with the procedures of this section.



Notes

1. CJP groove weld at top and bottom flanges. At top flange, either (1) remove weld backing, backgouge, and add 5/16" minimum fillet weld, or (2) leave backing in place and add 5/16" fillet under backing. At bottom flange, remove weld backing, backgouge, and add 5/16" minimum fillet weld. Weld: QC/QA Category AH/T.
2. Weld access hole, see Figure 3-5.
3. CJP groove weld full length of web between weld access holes. Provide non-fusible weld tabs. Remove weld tabs after welding and grind end of weld smooth at weld access hole. Weld: QC/QA Category BH/T.
4. Shear tab of thickness equal to that of beam web. Shear tab length shall be so as to allow 1/8" overlap with the weld access hole at top and bottom, and the width shall extend 2" minimum back along the beam, beyond the end of the weld access hole.
5. Full-depth partial penetration from far side. Weld: QC/QA Category BM/T.
6. Fillet weld shear tab to beam web. Weld size shall be equal to the thickness of the shear tab minus 1/16". Weld shall extend over the top and bottom one-third of the shear tab height and across the top and bottom. Weld: QC/QA Category BL/L.
7. Erection bolts: number, type, and size selected for erection loads.
8. For continuity plates and web doubler plates see Figure 3-6.

Figure 3-8 Welded Unreinforced Flange-Welded Web (WUF-W) Connection

Table 3-3 Prequalification Data WUF-W Connections

General:	
Applicable systems	OMF, SMF
Hinge location distance s_h	$d_c/2 + d_b/2$
Critical Beam Parameters:	
Maximum depth	W36 and shallower
Minimum span-to-depth ratio	OMF: 5 SMF: 7
Flange thickness	OMF: 1-1/2”or less SMF: 1” or less
Permissible material specifications	A572 Grade 50, A992, A913 Grade 50/S75
Critical Column Parameters:	
Depth	OMF: Not Limited SMF: W12, W14
Permissible material specifications	A572 Grade 50; A913 Grade 50 and 65; A992
Beam/Column Relations:	
Panel Zone strength	SMF: Section 3.3.3.2
Column/beam bending strength	Section 2.9.1
Connection Details	
Web connection	Special Connection – See Fig. 3-8
Continuity plate thickness	Section 3.3.3.1
Flange welds	Section 3.3.2.5
Welding parameters	Section 3.3.2.4, 3.3.2.5, 3.3.2.6
Weld access holes	Section. 3.3.2.7

Commentary: Development of connections with unreinforced flanges, suitable for use in Special Moment Frames, has required significant research, resulting in major modifications to the connection commonly in use prior to the 1994 Northridge earthquake. A summary list of revisions to the original prescriptive connection incorporated in this detail is as follows:

1. *limitations on permitted beam sizes,*
2. *filler metal with appropriate toughness,*
3. *removal of weld backing, back-gouging and addition of a reinforcing fillet weld,*
4. *use of improved weld-access hole shape and finish,*

5. *improvements to weld quality control and quality assurance requirements and methods, and*
6. *use of a full-strength welded web joint.*

Research indicates that this type of connection can be constructed to perform reliably if all of the procedures are complied with. Although this connection may appear to be economical, compared with other prequalified details, the designer should note carefully the importance of the features of this detail that improve its performance, and consider the effects of these features on the connection cost, before selecting it as a standard. Of particular importance is the rigorous level of quality assurance during field erection and welding, required for successful performance of this connection. Additionally, the beam size limitations may make it impractical in some buildings.

3.5.2.1 Design Procedure

Step 1: Calculate M_{pr}, at hinge location, S_h, according to methods of Section 3.2.4.

Step 2: Calculate V_p, at hinge location, S_h, according to methods of Section 3.2.5.

Step 3: Calculate M_c and C_y as described in Sections 3.2.6 and 3.2.7, respectively.

Step 4: Calculate the required panel zone thickness using the procedures of Section 3.3.3.2.

Step 5: Check requirements for continuity plates according to Section 3.3.3.1.

Step 6: Detail the connection as shown in Figure 3-8.

3.5.3 Free Flange Connections

This section provides guidelines for design of fully restrained Free Flange (FF) connections. This type of connection is prequalified for use in Special Moment Frame systems for beam sizes within the limits given in Table 3-4. For larger beams, the connection is prequalified for use in Ordinary Moment Frame systems.

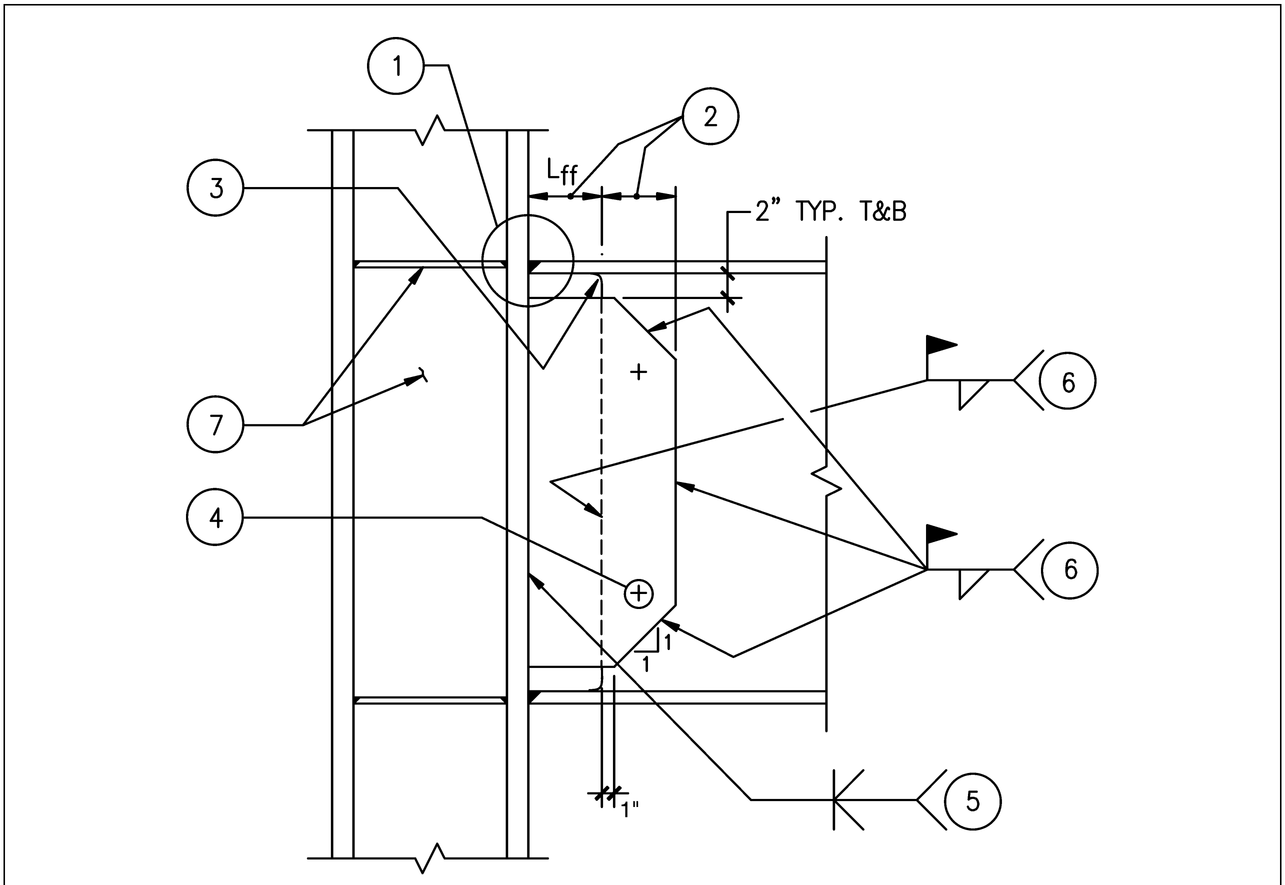
These connections utilize complete joint penetration groove welds, meeting the requirements of *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment Frame Construction for Seismic Applications*, to join beam flanges or girder flanges directly to column flanges. The web of the beam is removed in a single cut in the area adjacent to the column flange, and is replaced with a heavy trapezoidal-shaped shear tab. The shear tab is CJP groove welded to the column flange and welded on all sides with a fillet weld to the beam web. Figure 3-9 provides a typical detail for this connection type. These connections should be designed in accordance with the guidelines of this section.

Commentary: This connection type was developed at the University of Michigan and has been extensively tested both at that university and at the University of Texas at Austin. This connection type has demonstrated good performance, *similar to that exhibited by the WUF-W connection described in Section 3.5.2, and, in fact, has many similarities to that connection, as follows:*

1. *The flange weld is the same as the WUF-W;*
2. *The web cut-out provides an improvement similar to that provided by the improved weld-access hole;*
3. *The web connection is very substantial.*

Table 3-4 Prequalification Data for Free Flange Connections

General:	
Applicable systems	OMF, SMF
Hinge location distance s_h	$(d_c+d_b)/2$
Critical Beam Parameters:	
Maximum depth	OMF: W36 and shallower SMF: W30 and shallower
Minimum span-to-depth ratio	OMF: 5 SMF: 7
$b_f/2t_f$ of flange	$52/\sqrt{F_y}$
Flange thickness	OMF: 1-1/4” and less SMF: ¾” and less
Permissible material specifications	A572 Grade 50, A992, A913 Grade 50/S75
Critical Column Parameters:	
Depth	OMF: Not limited SMF: W12, W14
Permissible material specifications	A572 Grade 50; A913 Grade 50 and 65, A992
Beam/Column Relations:	
Panel zone strength	SMF: Section 3.3.3.2 ; C_{pr}=1.2
Column/beam bending strength	SMF: Section 2.9.1; C_{pr}=1.2
Connection Details	
Web connection	Heavy welded shear tab: See Figure 3-9
Continuity plate thickness	Section 3.3.3.1
Flange welds	Fig. 3-9
Welding parameters	Section 3.3.2.4, 3.3.2.5, 3.3.2.6
Weld access holes	Not applicable



Notes

1. CJP groove weld. Note 1 of Figure 3-8 applies. Weld: QC/QA Category AH/T.
2. See design procedure in Section 3.5.3.1, Steps 5 through 8, for web plate size and thickness.
3. ½" minimum radius.
4. Erection bolts: number, type and size selected for erection loads.
5. CJP double-bevel groove weld. Weld: QC/QA Category BH/T.
6. Fillet welds size, length, calculated in Section 3.5.3.1, Step 8. Weld: QC/QA Category BH/L.
7. For continuity plates and web doubler plates see Figure 3-6.

Figure 3-9 Welded Free Flange (FF) Connection

3.5.3.1 Design Procedure

Step 1: Calculate M_{pr} at hinge location, S_h, according to the methods of Section 3.2.4.

Step 2: Calculate V_p at hinge location, S_h, according to the methods of Section 3.2.5.

Step 3: Calculate M_f, M_c, and C_y as described in Sections 3.2.5, 3.2.6, and 3.2.7.

Step 4: Calculate the length of the free flange:

$$L_{ff} = a t_{fb} \tag{3-9}$$

where a may be selected in the range of 5 to 6.

Step 5: Calculate the shear in the shear tab from the equation:

$$V_{st} = \frac{2M_f}{L - d_c} + V_g \tag{3-10}$$

where:

V_{st} = shear in the shear tab, kips

L = span length measured from center to center of columns, ft

V_g = shear at the beam end due to factored gravity loads, kips

Step 6: Calculate the tension force on the shear tab, T_{st} from the equation:

$$T_{st} = \frac{M_f}{d_b - t_{fb}} - T_f = \frac{M_f}{d_b - t_{fb}} - R_y F_{yb} b_{fb} t_{fb} \tag{3-11}$$

Step 7: Calculate the required height of the shear tab from the equation:

$$h_{st} = d_b - 2t_{fb} - 2b \tag{3-12}$$

where b = 2 inches

Step 8: Calculate the required thickness of the shear tab and the weld sizes for the forces shown in Figure 3-10, based on principles of mechanics. Note that it is assumed that only the regions at the ends of the plate, and having a dimension $d_b/4$ are effective in resisting these forces.

Step 9: Determine the required panel zone thickness according to the methods of Section 3.3.3.2.

Step 10: Check requirements for Continuity Plates according to Section 3.3.3.1.

Step 11: Detail the connection as shown in Figure 3-9.

3.5.4 Welded Flange Plate Connections

This section provides guidelines for design of fully restrained Welded Flange Plate (WFP) connections. These connections utilize plates to connect the beam flanges to the column flange, without any direct connection of the beam flange to the column flange. The flange-plate-to-column-flange joint is a complete joint penetration groove weld. The flange plates are fillet welded to the top and bottom of the beam top and bottom flanges, respectively. Figure 3-11 provides a typical detail for this type of connection. These connections should be designed in accordance with the procedures of this section.

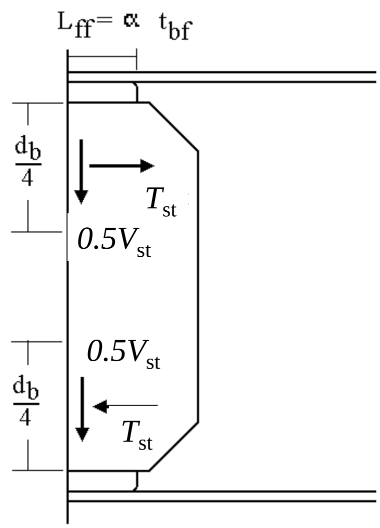
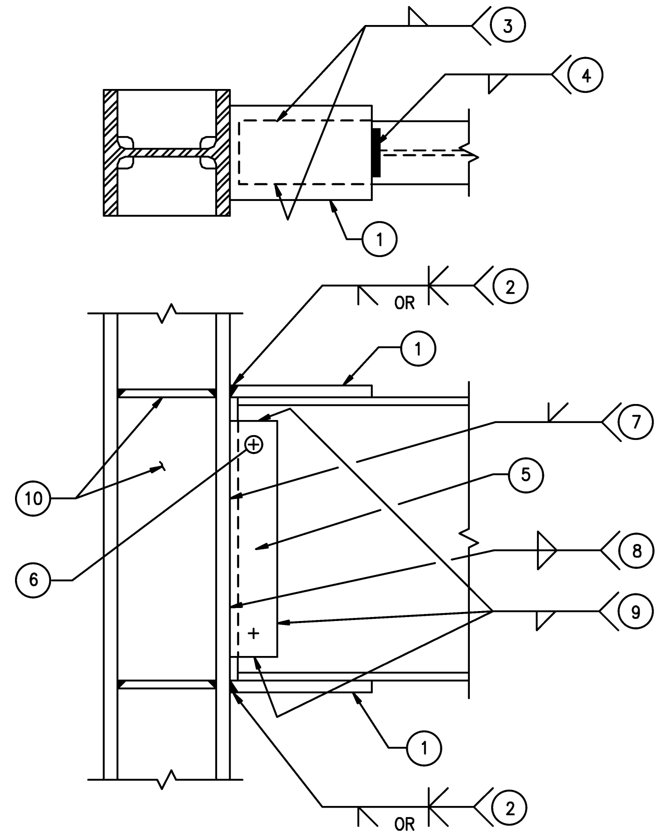


Figure 3-10 Schematic of the Forces for Design of the Free Flange Shear Tab

Table 3-5 Prequalification Data for WFP Connections

General	
Applicable systems	OMF, SMF
Hinge location distance s_h	$d_c/2 + l_p$
Critical Beam Parameters	
Maximum depth	W36 and shallower
Minimum span-to-depth ratio	OMF: 5 SMF: 7
Flange thickness	OMF: 1-1/2"or less SMF: 1"and less
Permissible material specifications	A572 Grade 50, A992, A913 Grade 50/S75
Critical Column Parameters	
Depth	OMF: Not limited SMF: W12, W14
Permissible material specifications	A572 Grade 50; A913 Grade 50 or 65, A992
Beam/Column/Flange Plate (FP) Relations	
Panel Zone strength	Section 3.3.3.2
Column/beam bending strength ratio,	Section 2.9.1
Connection Details	
Flange plate size	Section 3.5.4.1
Flange plate material	Grade 50
Flange welding	Fig. 3-11
Flange plate filler metals	Section 3.3.2.4
Web connection	Section 3.5.4.3 and Figure 3-11
Web welding parameters	Section 3.3.2.4
Continuity plate thickness	Sec 3.3.3.1, Consider dimensions of beam flange to be equal to dimension of flange plate.



Notes

1. Flange plate. See Section 3.5.4.1, Steps 1-4, for sizing requirements. Plates shall be fabricated with rolling direction parallel to the beam.
2. CJP groove weld: single or double bevel. Weld in shop or field. When using single-bevel groove weld, remove backing after welding, back-gouge, and reinforce with 5/16”-minimum fillet weld. When using double bevel weld, back-gouge first weld before welding other side. Weld QC/QA Category AH/T. If plates are shop welded to column, care must be exercised in locating and leveling plates, as shimming is not allowed between the plates and the beam flanges. If plates are field-welded to column after connecting to beam, weld access holes of sufficient size for weld backing and welding access shall be provided.
3. Fillet welds at edges of beam flanges to plate. Size welds according to the procedure in Section 3.5.4.1, Step 5. Welds may be shop or field. Provide weld tabs at end to provide full weld throat thickness to the end of the plate. Remove weld tabs and grind the end of the weld smooth. Use care to avoid grinding marks on the beam flange. Weld: QC/QA Category BH/L.
4. Fillet weld at end of flange plate to beam flange. Welds may be shop or field. Maintain full weld throat thickness to within 1” of the edge of the flange. Weld: QC/QA Category BH/T.
5. Shear tab of length equal to d_b-2k–2”. Shear tab thickness should match that of beam web.
6. Erection bolts: number, type, and size selected for erection loads.
7. Full depth-partial penetration from far side. Weld: QC/QA Category BM/T.
8. Fillet weld both sides. Fillet on side away from beam web shall be same size as thickness of shear tab. Fillet on the side of the beam web shall be ¼”. Weld: QC/QA Category BH/T.
9. Fillet weld shear tab to beam web. Weld size shall be equal to the thickness of the shear tab minus 1/16”. Weld: QC/QA Category BH/L.
10. For continuity plates and web doubler plates see Figure 3-6. For calculation of continuity plate requirements, use flange plate properties instead of beam flange properties.

Figure 3-11 Welded Flange Plate (WFP) Connection

Commentary: The WFP connection was tested at the University of California at Berkeley. Several similar connections had been tested by private parties prior to testing under this project. The connection has similarities to both the cover plated connection, which has been extensively used, and to the WUF-W connection. Its performance is comparable to that of the WUF-W. This connection, rather than the cover-plated connection commonly used from 1994 until publication of FEMA-267A, has been recommended for use in new buildings, because the welding of a single thickness of plate is considered to be more reliable than the welding of the combination of the beam flange and a cover-plate.

A CJP groove welded web connection is required for use in this prequalified connection, since such a web connection was used in the tested connections. Tests using bolted webs have not been reported.

The reader is referred to FEMA-355D, State Of the Art Report on Connection Performance, for more information on the testing and performance of this type of connection.

3.5.4.1 Design Procedure

Step 1: Select preliminary length of flange plate.

Step 2: Choose the width of the flange plate, b_p, based on beam flange width.

Step 3: Calculate M_{pr}, M_c, and M_{yf} according to Section 3.2.6.

Step 4: Calculate t_p based from the equation:

$$t_p = \frac{M_{yf}}{F_{yp} b_p \left(d_b + \frac{t_{pl_b} + t_{pl_t}}{2} \right)} \tag{3-13}$$

where:

b_p = Width of flange plate at column face. Tapered plates should be checked for the critical section

t_{pl_t} and t_{pl_b} are the thicknesses of the top and bottom flange plates, respectively.

Step 5: Calculate the length and thickness of the weld of the flange plate to the beam flange using the equation:

$$l_w t_w = \frac{M_f}{0.707 F_w} \tag{3-14}$$

where:

l_w = total length of weld including end weld (see Fig. 3-11).

F_w = nominal design strength of weld from *AISC-LRFD* = $0.60F_{EXX}$

$$t_w(\max) = t_p - \tfrac{1}{16}\text{inch}$$

If plate dimensions do not permit sufficient weld, return to Step 1 and select a longer plate length.

Step 6: Determine the required panel zone thickness according to the methods of Section 3.3.3.2. For purposes of this calculation, substitute $d_b+(t_{plt}+t_{plb})$ for d_b and the quantity $d_b - \frac{t_{plt}+t_{plb}}{2}$ for d_b - t_{fb}

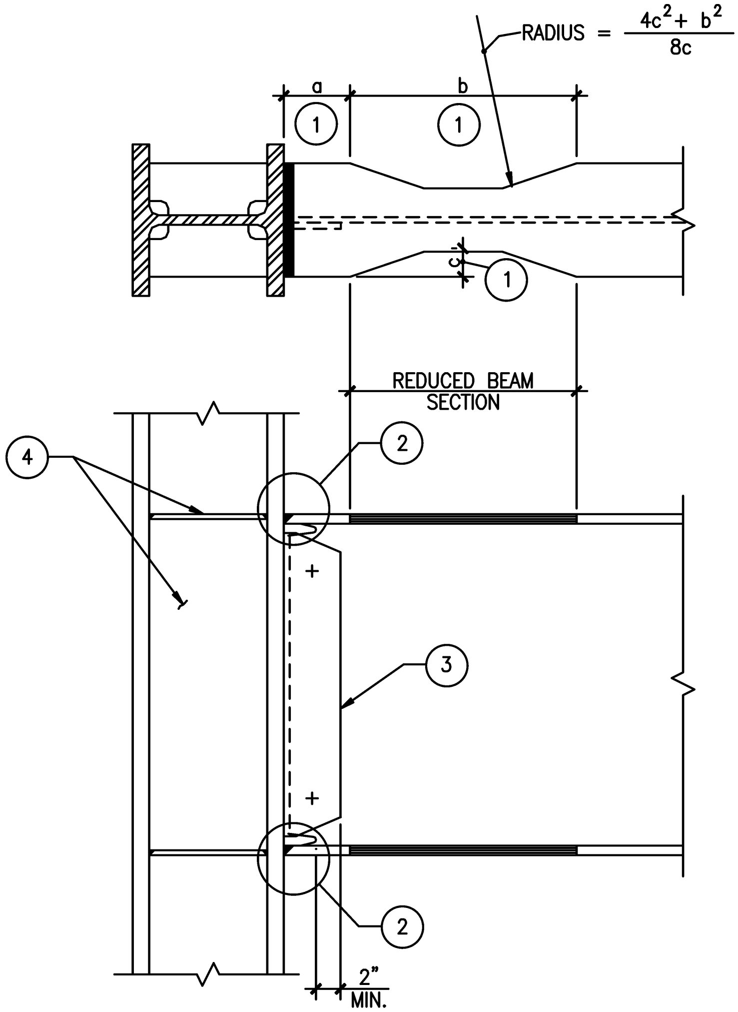
Step 7: Determine continuity plate requirements according to Section 3.3.3.1, For this purpose, use the plate width as the quantity b_f.

Step 8: Detail the connection as shown in Figure 3-11.

3.5.5 Reduced Beam Section Connections

This section provides procedures for design of fully restrained, Reduced Beam Section (RBS) connections. These connections utilize circular radius cuts in both top and bottom flanges of the beam to reduce the flange area over a length of the beam near the ends of the beam span. Welds of beam flanges to column are complete joint penetration groove welds, meeting the requirements of *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment Frame Construction for Seismic Applications*. In this type of connection, no reinforcement, other than weld metal, is used to join the flanges of the beam to the column. Web joints for these connections may be either complete penetration groove welds, or bolted or welded shear tabs. Table 3-6 provides limitations and details of the prequalification. Figure 3-12 provides typical details for this connection type. These connections are prequalified for use in Special Moment Frame or Ordinary Moment Frame systems within the limitations indicated in Table 3-6. When this type of connection is used, the elastic drift calculations should consider the effect of the flange reduction. In lieu of specific calculations, a drift increase of 9% may be applied for flange reductions ranging to 50% of the beam flange width, with linear interpolation for lesser values of beam flange reduction.

Commentary: This type of connection has performed adequately in tests with both welded and bolted web connections. While a welded web connection is more costly than the more conventional bolted web connection, it is believed that the welded web improves the reliability of the connection somewhat. The welded web provides for more effective force transfer through the web connection, thereby reducing stress levels at the beam flanges and beam flange groove welds.



Notes

1. See Section 3.5.5.1 for calculation of RBS dimensions. See *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment Frame Construction for Seismic Applications,* for fabrication details including cutting methods and smoothness requirements.
2. See Figure 3-8, and Note 1 to Figure 3-8, except that weld access hole may be as shown there, or as in AISC LRFD Vol. 1, Fig. C-J1.2, for rolled shapes or groove welded shapes.
3. Web Connection: Erection bolts: number, type, and size selected for erection loads.
   a. Alternative 1: CJP welded web. Weld QC/QA Category BM/L. Shear tab length is equal to the distance between the weld access holes plus ¼". Shear tab thickness is as required for erection and the tab serves as backing for CJP weld (3/8" min. thickness). Shear tab may be cut square, or tapered as shown. Weld of shear tab to column flange is minimum 3/16" fillet on the side of the beam web, and a fillet sized for erection loads (5/16" minimum) on the side away from the beam web. No weld tabs are required at the ends of the CJP weld and no welding of the shear tab to the beam web is required.Weld: QC/QA Category BM/L. Erection bolts are sized for erection loads.
   b. Alternative 2: Bolted shear tab. Shear tab and bolts are sized for shear, calculated as in Section 3.2 and using the methods of AISC. The shear tab should be welded to the column flange with a CJP groove weld or fillet of ¾ t_{pl} on both sides. Weld: QC/QA Category BL/T. Bolts shall be ASTM A325 or A490, and shall be fully-tightened.
4. For continuity plates and web doubler plates see Figure 3-6.

Figure 3-12 Reduced Beam Section (RBS) Connection

Table 3-6 Prequalification Data for RBS Connections

General	
Applicable systems	OMF, SMF
Hinge location distance s_h	$d_c/2 + a + b/2$
Critical Beam Parameters	
Depth range	W36 and shallower (maximum weight 300 lbs/ft)
Minimum span-to-depth ratio	OMF: 5. SMF: 7
$b_f/2t_f$	Up to *52/Ö* F_y, with b_f determined as described in Section 3.3.1.1
Flange thickness range	1-3/4” maximum
Permissible material specifications	A572 Grade 50, A992, A913 Grade 50/S75
Flange reduction parameters	Sec 3.5.5.1
Critical Column Parameters	
Depth range	OMF: Not Limited SMF: W12, W14
Permissible material specifications	A572 Grade 50; A913 Grade 50 and 65, A992
Beam / Column Relations	
Panel Zone strength	SMF: Section 3.3.3.2
Column/beam bending strength ratio	Section 2.9.1
Connection Details	
Web connection	Section 3.5.5.1 and Figure 3-12
Continuity plate thickness	Sec. 3.3.3.1
Flange welds	Fig. 3-12
Welding parameters	Sections 3.3.2.4, 3.3.2.5, 3.3.2.6
Weld access holes	See Fig. C-J1.2 AISC LRFD Vol. 1, or Section 3.3.2.7

As an alternative to a CJP groove weld, the beam web connection can also be made using a welded shear tab. The shear tab may be welded to the column using either fillet welds or groove welds. The shear tab, in turn, is then welded to the beam web with fillet welds. It is important to extend the tab as described in Figure 3-12, so as not to cause stress concentration near the end of the weld access hole. The web connection can also be made with a shear tab that is welded to the column flange and bolted to the beam web.

The effect of flange reduction on the elastic drift of frames can be readily calculated using prismatic beams with reduced moments of inertia or multi-segment beams that accurately represent the reduced section properties. Studies have been performed at the University of Texas (Grubbs, 1997) that have shown *ranges of drift increase from 4% to 7%, depending on the amount of flange reduction and other factors. The default factor to increase drift is expected to be slightly conservative for most cases.*

3.5.5.1 Design Procedure

Step 1: Determine the length and location of the beam flange reduction, based on the following:

$$a \cong (0.5 \text{ to } 0.75) b_f \tag{3-15}$$

$$b \cong (0.65 \text{ to } 0.85) d_b \tag{3-16}$$

where a and b are as shown in Figure 3-12, and b_f and d_b are the beam flange width and depth respectively.

Step 2: Determine the depth of the flange reduction, c, according to the following:

a) Assume $c = 0.20b_f$.

b) Calculate Z_{RBS}.

c) Calculate M_f according to the method of Section 3.2.6 and Figure 3-4 using $C_{pr} = 1.15$.

d) If $M_f < C_{pr}R_yZ_bF_y$ the design is acceptable. If M_f is greater than the limit, increase c. The value of c should not exceed 0.25 b_f.

Step 3: Calculate M_f and M_c based on the final RBS dimensions according to the methods of Section 3.2.7.

Step 4: Calculate the shear at the column face according to the equation:

$$V_f = 2\frac{M_f}{L - d_c} + V_g \tag{3-17}$$

Where: V_g= shear due to factored gravity load.

Step 5: Design the shear connection of the beam to the column. If a CJP welded web is used, no further calculations are required. If a bolted shear tab is to be used, the tab and bolts should be designed for the shear calculated in Step 4. Bolts should be designed for bearing, using a resistance factor f of unity.

Step 6: Design the panel zone according to the methods of Section 3.3.3.2.

Step 7: Check continuity plate requirements according to the methods of Section 3.3.3.1.

Step 8: Detail the connection as shown in Figure 3-12.

3.5.5.2 Fabrication Requirements

The RBS cut is normally made by thermal cutting. The finished cut should have a maximum surface roughness of 500 micro-inches, avoiding nicks, gouges, and other discontinuities. All corners should be rounded to minimize notch effects and cut edges should be ground in the direction of the flange length to have a surface roughness value as described in *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment Frame Construction for Seismic Applications.*

> *Commentary: Grinding parallel to the flange avoids grind marks perpendicular to the direction of stress, which can act as stress risers. It is not required to remove all vertical striations caused by flame cutting.*

3.5.5.3 Composite Construction

When composite metal deck and concrete are used, welded studs should not be placed in the area of the beam flange between the column face and 6 inches beyond the extreme end of the RBS. See Section 3.3.1.6.

3.6 Prequalified Bolted Fully Restrained Connections

This section provides recommended criteria for alternative types of prequalified bolted, fully restrained, steel moment-frame connections suitable for use in new construction within the limits indicated in the prequalification for each detail. Table 3-7 indicates the various types of prequalified fully restrained connections, and the structural systems for which they are prequalified. Additional prequalification data on these various connection types is provided in the sections that follow.

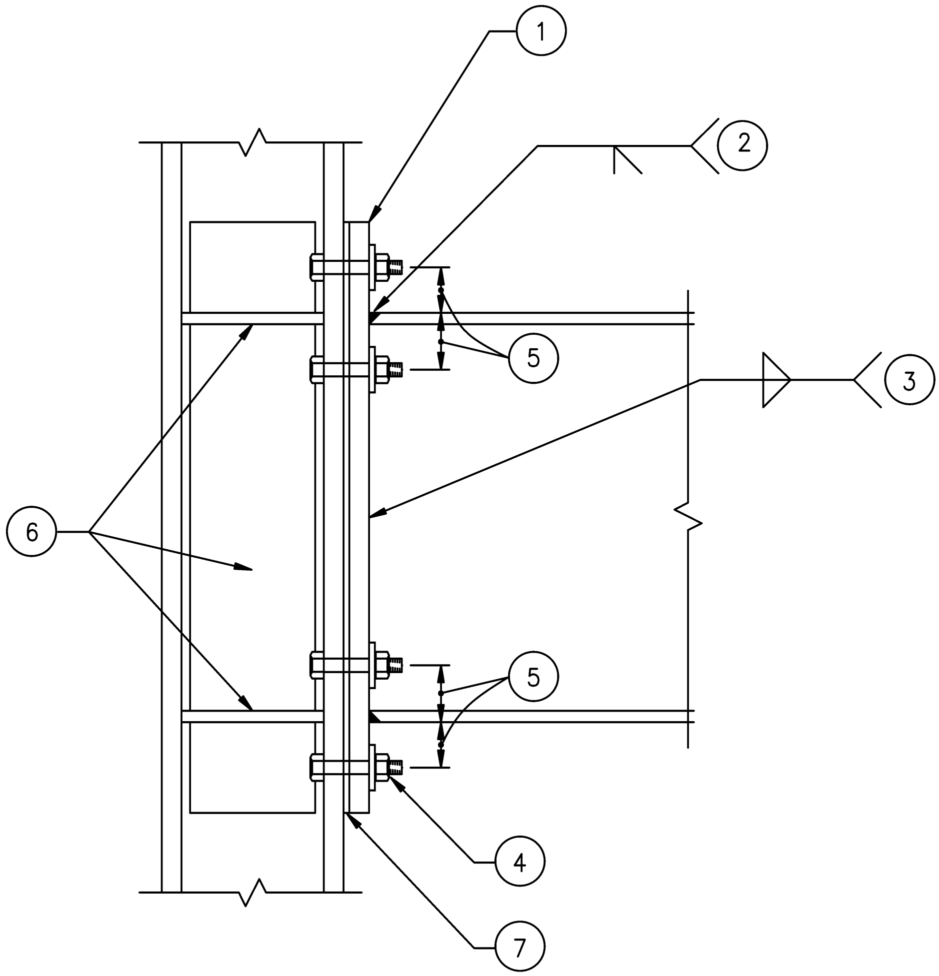
Table 3-7 Prequalified Bolted Fully Restrained Connections

Connection Type	Criteria Section	Frame Type
Bolted Unstiffened End Plate (BUEP)	3.6.1	OMF, SMF
Bolted Stiffened End Plate (BSEP)	3.6.2	OMF, SMF
Bolted Flange Plate (BFP)	3.6.3	OMF, SMF
Double Split Tee (DST)*	3.7.1	OMF

*This type of connection may be partially or fully restrained depending on design.

3.6.1 Bolted Unstiffened End Plate Connections

The bolted unstiffened end plate (BUEP) connection is made by shop welding the beam to an end plate using (1) a CJP welded joint of the beam flanges to the plate and (2) fillet welds for the beam web to the plate. The end plate is then field-bolted to the column. The CJP groove weld of the beam flange is made without using a weld access hole, and is therefore not a prequalified weld in the area of the beam web, where backing cannot be installed. However, qualification of this joint detail to meet AWS requirements is not necessary. This type of connection can be used in either Ordinary Moment Frame or Special Moment Frame systems within the member size limitations given in Table 3-8. Figure 3-13 presents a detail for the connection.



Notes

1. ASTM A36 end plate. For sizing see Section 3.6.1.1.
2. CJP groove weld. This weld has special requirements. See *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment Frame Construction for Seismic Applications,* for fabrication details. Weld: QC/QA Category AH/T.
3. Fillet weld both sides, or CJP weld; see Section 3.6.1.3 for sizing requirements. See *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment Frame Construction for Seismic Applications*, for fabrication details. Weld: QC/QA Category BM/L.
4. Pretensioned ASTM A325 or A490 bolts. Diameter not to exceed 1-1/2 inch. See Section 3.6.1.1 for sizing requirements.
5. Bolt location is part of the end plate design. See Section 3.6.1.1.
6. For continuity plates and web doubler plates, see Figure 3-6. For calculation of panel zone strength, see Section 3.6.1.1.
7. Shim as required. Finger shims shall not be placed with fingers pointing up.

Figure 3-13 Bolted Unstiffened End Plate (BUEP) Connection

Table 3-8 Prequalification Data for BUEP Connections

General	
Applicable systems	OMF, SMF
Hinge location distance, s_h	$d_c/2 + t_{pl} + d_b/3$
Critical Beam Parameters	
Maximum depth	W30 and smaller for OMF W24 and smaller for SMF
Minimum span-to-depth ratio	OMF: 5 SMF: 7
Flange thickness	Up to ¾”
Permissible material specifications	A572 Grade 50, A992, A913 Grade 50/S75
Critical Column Parameters	
Depth range	OMF: Not limited SMF: W8, W10, W12, W14
Flange thickness	Section 3.6.1.1, Step 7
Permissible material specifications	A572, Grade 50; A913 Grade 50, or 65, A992
Beam /Column Relations	
Panel zone strength	Sec. 3.3.3.2, Section 3.6.1.1, Step 9.
Column/beam bending strength ratio	Sec. 2.9.1
Connection Details	
Bolts:	
Bolt diameter	Section 3.6.1.1, Step 2
Bolt grades	A325 & A490.
Installation requirements	Pretensioned
Washers	Single F436 when required.
Hole type	Standard
End Plate:	
End plate thickness	Section 3.6.1.1, Steps 3 and 4
End plate material	A36
Flange Welds:	
Weld type	CJP groove weld similar to AWS TC-U4b, 3/8” fillet used as backing, root backgouged prior to start of groove weld. See Fig. 3-13.
Filler metal	Section 3.3.2.4
Weld access holes	Not permitted
Web connection:	Figure 3-13
Continuity plate thickness	Section 3.6.1.1, Steps 6 and 8

Commentary: The behavior of this type of connection can be controlled by a number of different modes including flexural yielding of the beam section, flexural yielding of the end plates, yielding of the column panel zone, tension failure of the end plate bolts, shear failure of the end plate bolts, and failure of the various welded joints. Some of these modes are brittle, and therefore are undesirable, while others have significant ductility. Flexural yielding of the beam and shear yielding of the column panel zone are behavioral modes capable of exhibiting acceptable levels of inelastic behavior. Other modes are not. In order to design a connection of this type, it is necessary to select which modes of behavior are to be permitted to control the connection's inelastic deformation. Once desired modes of behavior for the connection are selected, the various elements of the connection are designed with sufficient strength so that other modes are unlikely to occur. FEMA-355D, State of the Art Report on Connection Performance, provides further discussion of the performance of these connections, and summaries of test data and references.

3.6.1.1 Design Procedure

The connection shall be designed so that yielding occurs either as a combination of beam flexure and panel zone yielding or as beam flexure alone. The end plate, bolts and welds must be designed so that yielding does not occur in these elements.The design should be performed using the steps below. The various parameters used in the equations are defined in Figure 3-14 and in *AISC-LRFD*.

Step 1: Calculate M_f and M_c according to the methods of Section 3.2.7.

Step 2: Select end plate bolt size by solving Equation 3-20 for T_{ub} and selecting bolt type and A_{bolt} as required:

$$M_f < 2T_{ub}\left(d_o + d_i\right) \tag{3-18}$$

where:

T_{ub} = $90A_{bolt}$ for A325 bolts
= $113A_{bolt}$ for A490 bolts
and d_o and d_i are as defined in Figure 3-14

Step 3: Check the adequacy of the selected bolt size to preclude shear failure by ensuring that the area A_b of the bolts satisfies the formula:

$$A_b \geq \frac{\dfrac{2M_f}{L-d_c} + V_g}{3F_v} \tag{3-19}$$

Step 4: Determine the minimum end plate thickness t_p required to preclude end plate flexural yielding from the equation:

$$t_p = \sqrt{\frac{M_f}{0.8F_{yp}\left\{(d_b - p_t)\left[\frac{b_p}{2}\left(\frac{1}{p_f}+\frac{1}{s}\right)+\left(p_f+s\right)\frac{2}{g}\right]+\frac{b_p}{2}\left(\frac{d_b}{p_f}+\frac{1}{2}\right)\right\}}} \tag{3-20}$$

where:

$$s = \sqrt{b_p g} \tag{3-21}$$

g = is the bolt gage as defined in Figure 3-14
Note that the end plate is required to be ASTM A36 steel.

Step 5: Determine the minimum end plate thickness required to preclude end plate shear yielding from the equation:

$$t_p = \frac{M_f}{1.1F_{yp}b_p\left(d_b - t_{bf}\right)} \tag{3-22}$$

Step 6: Determine the minimum column flange thickness required to resist beam flange tension from the equation:

$$t_{fc} = \sqrt{\frac{\frac{M_f}{d_b - t_{fb}}\; C_1}{2\;F_{yc}c}} \tag{3-23}$$

where:

$$C_1 = \frac{g}{2} - k_1 \tag{3-24}$$

k_1 = Distance from centerline of column web to flange toe of fillet as defined in *AISC Manual*.

If the column flange thickness is less than the calculated requirement, continuity plates are required. Continuity plates, if required, shall be sized as required in Section 3.3.3.1.

Step 7: If continuity plates are required, the column flange thickness must be additionally checked for adequacy to meet the following:

$$t_{fc} > \sqrt{\frac{\frac{M_f}{2\,(d_b - t_{fb})}}{0.8\,F_{yc}Y_c}} \tag{3-25}$$

where:

$$Y_c = \left(\frac{c}{2} + s\right)\left(\frac{1}{C_2} + \frac{2}{C_1}\right) + \left(C_2 + C_1\right)\left(\frac{4}{c} + \frac{2}{s}\right) \tag{3-26}$$

$$C_1 = \frac{g}{2} - k_1 \tag{3-27}$$

$$\mathrm{C}_2 = \frac{b_{fc} - g}{2} \tag{3-28}$$

$$s = \sqrt{\frac{C_1 C_2}{C_2 + 2C_1}\left(2b_{fc}\,4k_1\right)} \tag{3-29}$$

If t_c is less than the calculated value, a column with a thicker flange must be selected.

Step 8: Check column flange thickness for adequacy for beam flange compression according to the following:

$$t_{fc} > \frac{M_f}{\left(d_b - t_{fb}\right)\left(6k + 2t_{pl} + t_{bf}\right)F_{yc}} \tag{3-30}$$

where k is the k-distance of the column from the *AISC Manual*.

If t_{fc} is less than given by Equation 3-30, than beam flange continuity plates are required in accordance with Section 3.3.3.1.

Step 9: Check the panel zone shear capacity in accordance with Section 3.3.3.2. For purposes of this calculation, d_b may be taken as the distance from one edge of the end plate to the center of the beam flange at the opposite flange.

Step 10: Detail the connection as shown in Figure 3-13.

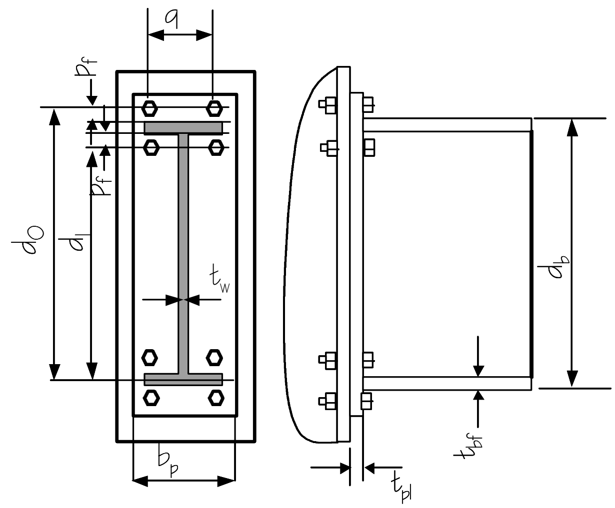
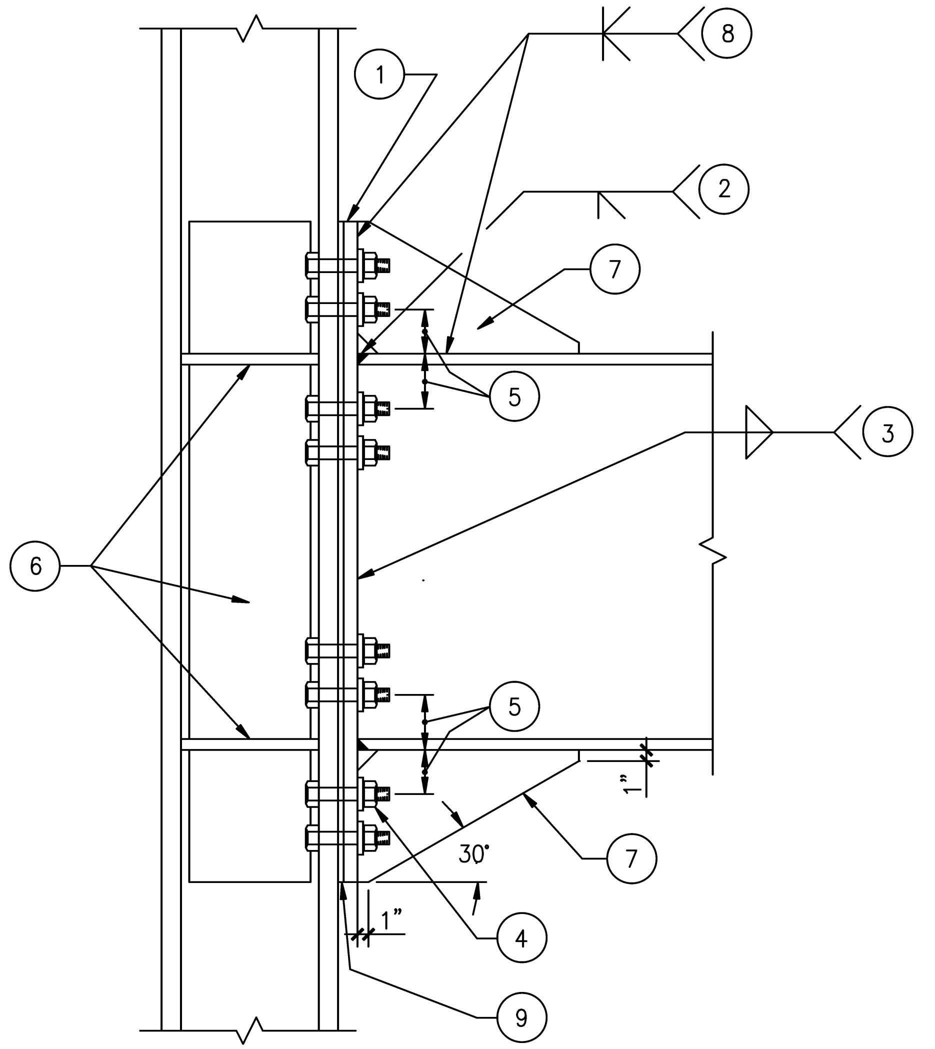


Figure 3-14 Geometry of Unstiffened End Plate Connection

3.6.2 Bolted Stiffened End Plate Connection

This bolted stiffened end plate (BSEP) connection is made by shop-welding the beam to the end plate using (1) a CJP welded joint for the beam flanges to the end plate and (2) fillet welds for the beam web to end plate. The endplate is then field-bolted to the column. The CJP groove weld of the beam flange is made without using a weld access hole, and is therefore not a prequalified weld in the area of the beam flange, where backing cannot be installed. However, qualification of this joint detail to meet AWS requirements is not necessary. The outstanding flanges of the end plate at the top and bottom of the beam are stiffened by a vertical fin plate that extends outward from the beam flanges. These stiffener plates are CJP double-bevel groove welded to the beam flanges and end plates. This type of connection can be used in either Ordinary Moment Frame or Special Moment Frame systems within the limitations given in Table 3-9. A detail of this connection type is shown in Fig. 3-15.

Commentary: The behavior of this type of connection can be controlled by a number of different behavioral modes including flexural yielding of the beam section, flexural yielding of the end plates, yielding of the column panel zone, tension failure of the end plate bolts, shear failure of the end-plate bolts, and failure of the various welded joints. Some of these modes are brittle, and therefore are undesirable while others have significant ductility. Flexural yielding of the beam and shear yielding of the column panel zone are behavioral modes capable of exhibiting acceptable levels of inelastic behavior. Other modes are not. The design procedure contained in this section is based on inelastic action occurring in preferred modes. The various elements of the connection are then designed with sufficient strength so that other modes are unlikely to occur. FEMA-355D, State Of Art Report on Connection Performance, provides further discussion of the performance of these connections and summaries of test data and references.



Notes

1. ASTM A36 end plate. For sizing, see Section 3.6.2.1.
2. CJP groove weld. This weld has special requirements. See *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment Frame Construction for Seismic Applications,* for fabrication details. Weld: QC/QA Category AH/T.
3. Fillet weld both sides, or CJP weld; see Section 3.6.2.4 for sizing requirements. See *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment Frame Construction for Seismic Applications*, for fabrication details. Weld: QC/QA Category BM/L.
4. Pretensioned ASTM A325 or A490 bolts. See Section 3.6.2.1 for sizing requirements.
5. Bolt location is part of the end plate design. See Section 3.6.2.1.
6. For continuity plates and web doubler plates, see Figure 3-6. For calculation of panel zone strength, see Section 3.6.2.1.
7. Stiffener is shaped as shown. Stiffener thickness shall be the same as that of the beam web.
8. Stiffener welds are CJP double-bevel groove welds to both beam flange and end plate. Weld: QC/QA Category AH/T for weld to endplate. BM/L for weld to beam..
9. Shim as required. Finger shims shall not be placed with fingers pointing up.

Figure 3-15 Stiffened End Plate Connection

Table 3-9 Prequalification Data for Bolted Stiffened End Plate Connections

General	
Applicable systems	OMF, SMF
Hinge location distance s_h	$d_c/2 + t_{pl} + L_{st}$
Critical Beam Parameters	
Maximum depth	W36
Minimum span-to-depth ratio	OMF: 5 SMF: 7
Flange thickness	1”
Permissible material specifications	A572 Grade 50, A992, A913 Gr50/S75
Critical Column Parameters	
Depth range	OMF: Not Limited SMF: W12, W14
Flange thickness	Section 3.6.2.1, Step 6
Permissible material specifications	A572, Grade 50; A913 Grade 50 and 65, A992
Beam /Column Relations	
Panel zone strength	Sec. 3.6.2.1, Step 7
Column/beam bending strength ratio	Sec. 2.9.1
Connection Details	
Bolts:	
Bolt diameter	Section 3.6.2.1, Step 1
Bolt grades	A325 and A490.
Installation requirements	Pretensioned
Washers	Single F436 when required
Hole type	Standard
End Plate:	
End plate thickness and rib size	Section 3.6.2.1, Step 2
End plate and rib material specification	A36
Flange welds:	
Weld type	CJP groove weld similar to AWS TC-U4b, 3/8” fillet used as backing, root backgouged prior to start of groove weld. See Fig. 3-15.
Weld metal	Section 3.3.2.4
Weld access holes	Not permitted
Web connection:	Figure 3-15
Continuity plate thickness	Section 3.6.2.1, Steps 4 and 5

3.6.2.1 Design Procedure

The connection shall be designed so that yielding occurs either as a combination of beam flexure and panel zone yielding or as beam flexure alone. The design should be performed using the steps below. The various parameters used in the equations are defined in Figure 3-16 and in *AISC-LRFD*.

Step 1: Calculate M_f and M_c according to the methods of Section 3.2.7.

Step 2: Select end plate bolt size by solving Equation 3-32 for T_{ub} and selecting bolt type and A_{bolt} as required:

$$M_f < 3.4T_{ub}\left(d_o + d_i\right) \tag{3-31}$$

where:

T_{ub} = $90A_{bolt}$ for A325 bolts
= $113A_{bolt}$ for A490 bolts
and d_o and d_i are as defined in Fig. 3-16

Confirm that T_{ub} satisfies the Equation:

$$T_{ub} \geq \frac{0.00002305\, p_f^{\;0.591}\left(F_{fu}\right)^{2.583}}{t_p^{\;0.895} d_{bt}^{\;1.909} t_s^{\;0.327} b_p^{\;0.965}} + T_b \tag{3-32}$$

Where T_b is the minimum bolt pretension per Table J3.1 of *AISC-LRFD*.

Adjust bolt size as required.

Step 3: Check the adequacy of the selected bolt size to preclude shear failure by ensuring that the area A_b of the bolts, satisfies the formula:

$$A_b \geq \frac{\dfrac{2M_f}{L - d_c} + V_g}{6F_v} \tag{3-33}$$

Step 4: Determine the minimum end plate thickness t_p required to preclude end plate flexural yielding as the larger of the values given by equations 3-34 or 3-35:

$$t_p \geq \frac{0.00609\; p_f^{\;0.9}\; g^{0.6}\; F_{fu}^{\;0.9}}{d_{bt}^{\;0.9}\; t_s^{\;0.1}\; b_p^{\;0.7}} \tag{3-34}$$

$$t_p \geq \frac{0.00413\; p_f^{\;0.25}\; g^{0.15}\; F_{fu}}{d_{bt}^{\;0.7}\; t_s^{\;0.15}\; b_p^{\;0.3}} \tag{3-35}$$

where:

$$F_{fu} = \frac{M_f}{d_b - t_{bf}} \tag{3-36}$$

and d_{bt} is the diameter of the bolt

Note that the end plate is required to be ASTM A36 steel and the stiffener plate must be at least as thick the beam web.

Step 5: Determine the minimum column flange thickness required to resist beam flange tension from the equation:

$$t_{cf} > \sqrt{\frac{a_m F_{fu} (C_3)}{0.9 F_{yc} (3.5 p_b + c)}} \tag{3-37}$$

where:

$$a_m = C_a \left(\frac{A_f}{A_w} \right)^{1/3} \frac{C_3}{(d_{bt})^{1/4}} \tag{3-38}$$

$$C_3 = \frac{g}{2} - \frac{d_{bt}}{4} - k_1 \tag{3-39}$$

and C_a = 1.45 for A325 bolts and 1.48 for A490 bolts when A36 end plates are used

If the column flange is thinner than required, continuity plates are required and should be provided in accordance with Section 3.3.3.1.

Step 6: Check column flange thickness for adequacy for beam flange compression according to the following:

$$t_{wc} = \frac{M_f}{(d_b - t_{fb})(6k + 2t_p + t_{fb}) F_{yc}} \tag{3-40}$$

where *k* is the *k*-distance of the column from the *AISC Manual*.

If the above relationship is not satisfied, continuity plates are required and should be provided in accordance with Section 3.3.3.1.

Step 7: If continuity plates are required, the column flanges must be at least as thick as the required end plate thickness, calculated in Step 4.

Step 8: Check the shear in the panel zone in accordance with Section 3.3.3.2. For purposes of this calculation, d_b may be taken as the distance from one end of the end plate to the center of the opposite flange.

Step 9: Detail the connection as shown is Figure 3-15.

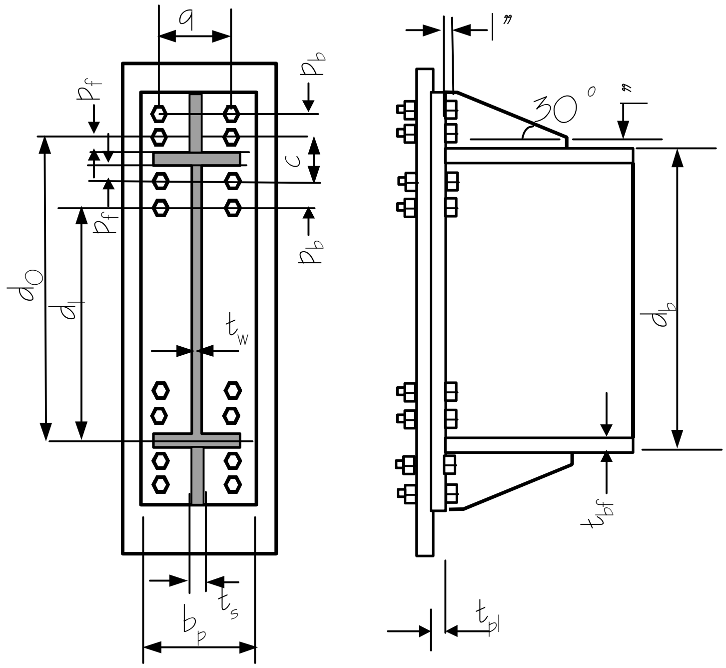
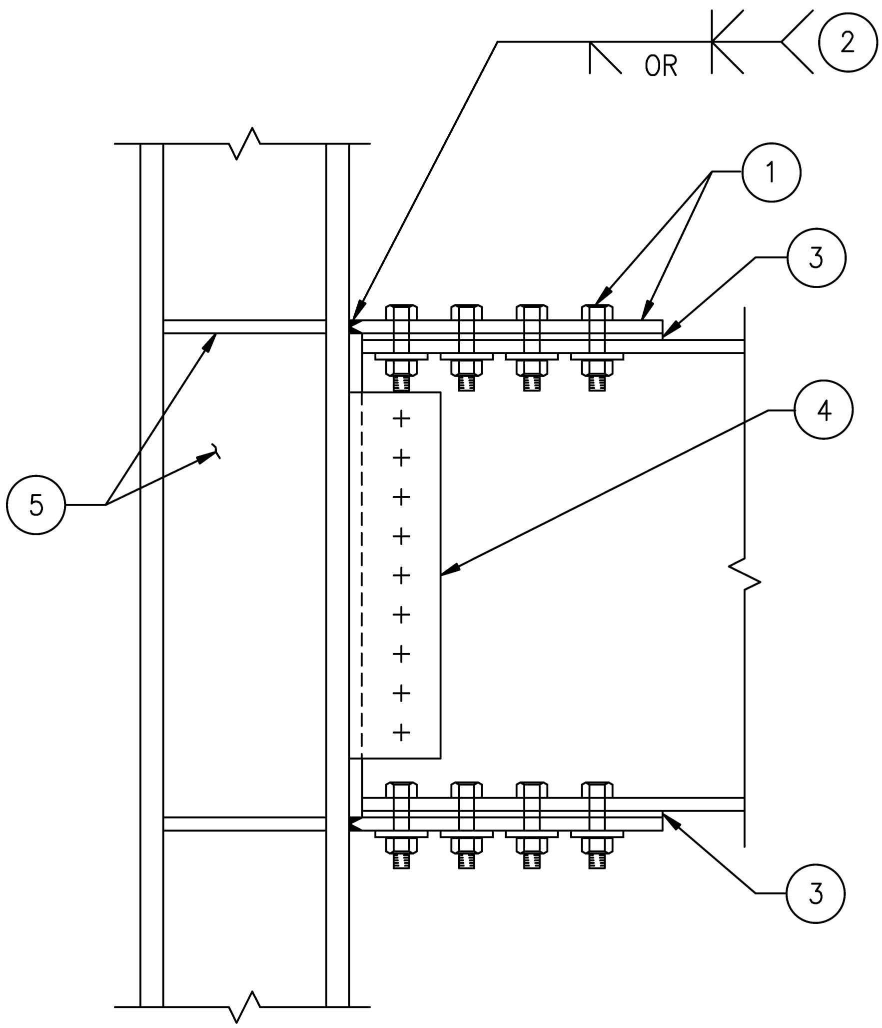


Figure 3-16 Geometry of Stiffened End Plate Connection

3.6.3 Bolted Flange Plate Connections

This section provides procedures for design of bolted flange plate (BFP) connections utilizing plates welded to the column flanges and bolted to the beam flanges. The flange plates are welded to the column flange using CJP welds following the recommendations given in sections 3.3.2.1 through 3.3.2.5. The flange plates are bolted to beam flanges following the recommendations of Sections 3.3.4.1 and this Section. The beam web is connected to the column flange with a bolted shear tab. A detail for this connection type is shown in Figure 3-17. Table 3-10 presents the limitations for this connection prequalification. Figure 3-18 shows dimensions and nomenclature to be used with the design procedure of Section 3.6.3.1.

Commentary: The behavior of this type of connection can be controlled by a number of different modes including: flexural yielding of the beam section, flexural yielding of the cover plates, yielding of the column panel zone, net-section tensile failure of the beam flange or cover plates, shear failure of the bolted connections, or failure of the welded joints. Some of these modes are brittle, while others have significant ductility. Connections of this type must be controlled by a preferred ductile behavior where the various elements of the connection are designed with sufficient strength that the other modes are unlikely to occur. Tests of connection assemblies incorporating this detail, as described in FEMA-355D, indicate that the best inelastic behavior is achieved with balanced yielding in all of the three preferred mechanisms: beam flexure, cover plate extension and compression, and panel zone yielding. When this balanced behavior occurs, the required rotations may be met without any of the mechanisms fully developing their maximum strain-hardened strength. For *example, $C_{pr}R_yF_yZ$ of the beam may not be reached at the beam yield section. For this reason, and unlike the case with some other prequalified connections, the design equations are developed at the onset of yielding, rather than at full yield.*



Notes

1. Size the flange plate and bolts in accordance with Section 3.6.3.1. Bolts are fully pretensioned ASTM A325 or A490, designed for bearing. Bolt holes in flange plate are oversize holes. Use standard holes in beam flange. Washers as required by RCSC, Section 7.
2. CJP groove weld, single or double bevel. Weld in shop or field. When using single-bevel groove weld, remove backing after welding, backgouge, and reinforce with 5/16" minimum fillet weld. When using double bevel weld, backgouge first weld before welding other side. Weld: QC/QA Category AH/T.
3. Shims are permitted between flange plates and flanges**.**
4. Size shear tab and bolts by design procedure in Section 3.6.3.2. Bolt holes in shear tab are short-slotted-horizontal; holes in web are standard. Weld QC/QA Category BM/L.
5. For continuity plates and web doubler plates see Figure 3-6. For calculation of continuity plate requirements, use flange plate properties as flange properties.

Figure 3-17 Bolted Flange Plate (BFP) Connection

Table 3-10 Prequalification Data for Bolted Flange Plate Connections

General	
Applicable systems	OMF, SMF
Hinge location distance s_h	$d_c/2 + L_p$
Critical Beam Parameters	
Maximum depth	OMF: up to W36 SMF: up to W30
Minimum span-to-depth ratio	OMF: 5 SMF: 8
Flange thickness	Up to 1-1/4” (OMF) Up to ¾” (SMF)
Permissible material specifications	A572 Grade 50, A992, A913 Gr50/S75
Critical Column Parameters	
Depth range	OMF: Not Limited SMF: W12, W14
Permissible material specifications	A572 Grade 50, A913 Grade 50 or 65, A992
Critical Beam Column Relations	
Panel zone strength	Section 3.6.3.1, Step 3.
Column/beam bending strength ratio	Section 2.9.1
Critical Connection Details	
Connection Plates:	
Permissible material specifications	A36, A572 Grade 42 or 50
Design method	Section 3.6.3.1, Step 4 and Step 5
Weld to flange	Fig. 3-17. Welding QC/QA Category AH.
Flange welding parameters	Section 3.3.2.4, 3.3.2.5, 3.3.2.6
Bolt Characteristics:	
Bolt diameter	Section 3.6.3.1, Steps 6 and 7; 1-1/8” maximum
Bolt grade	A325-X or A490-X
Bolt spacing	3x bolt diameter min.
Installation requirements	Pretensioned
Washers	F436 as required
Web Connection Parameters:	
Web Connection	Section 3.6.3.1, Step 12; Shear tab welded to column flange and bolted to beam. Bolt holes short-slotted horizontal. See Fig. 3-17.

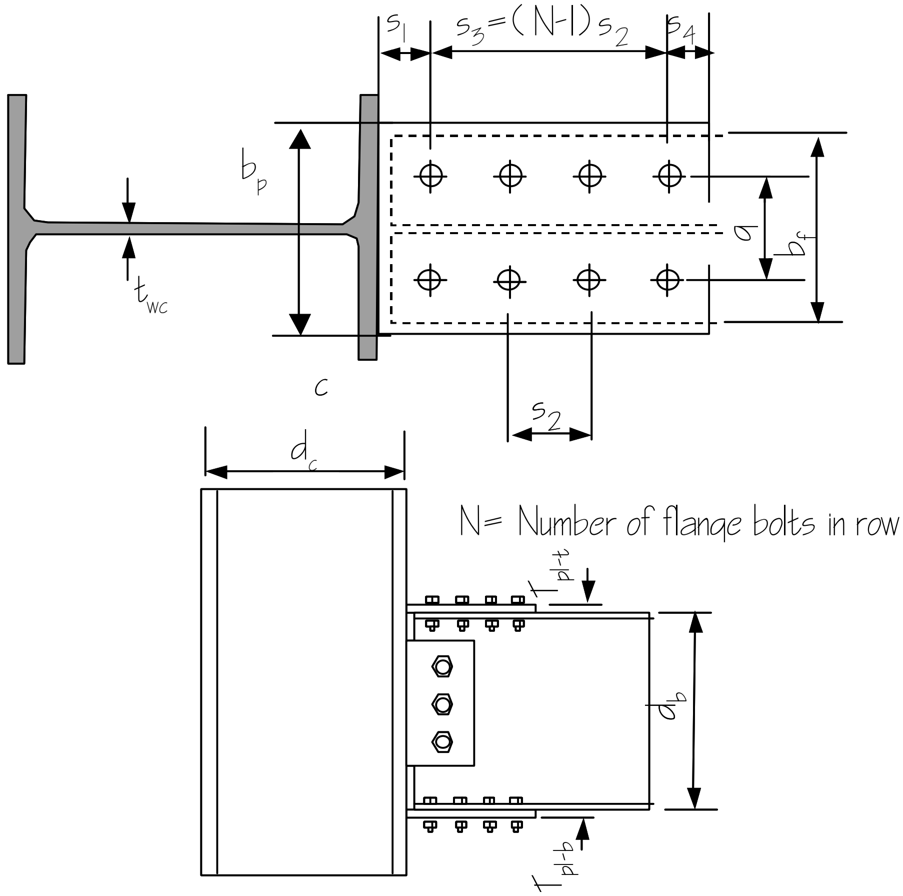


Figure 3-18 Geometry of the Bolted Flange Plate Connection

3.6.3.1 Design Procedure

The design of the connection should be performed using the steps below. The various parameters used in the equations are defined in Figure 3-18, and in *AISC/LRFD*.

Step 1: Calculate M_f and M_c according to the procedures in Section 3.2.7.

Step 2: Calculate the moment at the face of the column at onset of beam flange yielding, M_{yf}, according to Section 3.2.8.

Step 3: Calculate panel zone thickness requirements according to Section 3.3.3.2. It is recommended not to overstrengthen the panel zone for these connections. If the thickness of the panel zone is more than 1.5 times that required, it is recommended to use a different combination of beam and column sizes. Use the distance between the outer faces of the flange plates as d_b, and the center-to-center distance between plates in place of the quantity "d_b - t_f" in the application of the procedure of Section 3.3.3.2.

Step 4: Establish the width of the flange plate, b_p, based on the geometry of the beam and column.

Step 5: Calculate the minimum required thickness of the flange plates, t_{pl} from the equation:

$$t_{pl} = \frac{d_b - \sqrt{d_b^2 - \dfrac{4.4M_{yf}}{F_y b_p}}}{2} \tag{3-41}$$

Commentary: It is desirable not to oversize the flange plates, as best performance is achieved with a combination of yielding of the beam flange, the panel zone, and the flange plate.

Step 6: Select the number, size and grade of bolts in the beam-flange-to-flange-plate connection and evaluate the adequacy of the plate and beam, to preclude net section failures and bolt hole elongation failures, in accordance with Steps 7, 8, 9, 10 and 11 respectively,where in each case, Equation 3-42 must be satisfied:

$$1.2M_{yf} < M_{fail} \tag{3-42}$$

where:

M_{yf} = Moment at the face of the column at initiation of beam flange yielding, calculated in Step 2 above, and

M_{fail} = Moment at the face of the column at initiation of failure in the specific behavior mode being addressed in Steps 7 through 11.

Step 7: Determine M_{fail} the moment at the face of the column for shear failure of the bolts in accordance with Equation 3-43 and check for adequacy to meet the criteria of Equation 3-42, Step 6:

$$M_{fail_{bolts}} = 2NA_b\ (\ F_{v_{bolt}}\ )\ d_b\ L_{TF1} \tag{3-43}$$

where:

A_b = Area of bolt

$F_{v_{bolt}}$ = Nominal shear strength of bolt in bearing-type connections, from *AISC LRFD*.

L_{TF1} = Length ratio to transfer moment from center of bolt group to face of column given by Equation 3-44:

$$L_{TF1} = \frac{L - d_c}{L - d_c - (2S_1 + S_3)} \tag{3-44}$$

N = Number of bolts in connection of beam flange to flange plate

Step 8: Determine M_{fail} the moment at the face of the column for net section fracture of the flange plate in accordance with Equation 3-45 and check for adequacy to meet the criteria of Equation 3-42, in Step 6:

$$M_{fail_{FP}} = 0.85\ F_{u-pl}\ \left(b_p - 2\left(d_{bt_{hole}} + 0.062\right)\right) t_{pl}\ \left(d_b + t_{pl}\right) L_{TF2} \tag{3-45}$$

where:

d_{bthole} = diameter of flange plate bolt hole, inches.

L_{TF2} = ratio to transfer moment at bolt hole closest to column to column face given by Equation 3-46:

$$L_{TF2} = \frac{L - d_c}{L - d_c - 2S_1} \tag{3-46}$$

Step 9: Determine M_{fail} the moment at the face of the column for net section fracture of the beam flange in accordance with Equation 3-47 and check for adequacy to meet the criteria of Equation 3-42, Step 6:

$$M_{fail} = F_{u-b}\left(Z_b - 2\left(d_{bt\,hole} + 0.062\right)t_{fb}\left(d_b - t_{fb}\right)\right)\ L_{TF3} \tag{3-47}$$

where:

d_{bt} = diameter of bolt, inches

L_{TF3} = ratio to transfer moment from the bolt hole furthest from the column face to the column face, given by Equation 3-48:

$$L_{TF3} = \frac{L - d_c}{L - d_c - 2(S_1 + S_3)} \tag{3-48}$$

Step 10: Determine M_{fail} the moment at the face of the column for elongation of bolt holes in accordance with Equation 3-49 and check for adequacy to meet the criteria of Equation 3-42, Step 6:

$$M_{fail} = T_n\ \left(d_b + \frac{t_{PL-t}\ +\ t'_{PL-b}}{2}\right)\ L_{TF1} \tag{3-49}$$

where:

T_n is the lesser of the values given by Equations 3-50 or 3-51:

$$T_n = 2.4F_{u-b}\left(S_3 + S_1 - c\right)t_{fb} \tag{3-50}$$

$$T_n = 2.4F_{u-pl}\left(S_3 + S_4\right)t_{pl} \tag{3-51}$$

Step 11: Check block shear according to the requirements of *AISC LRFD* to ensure that the moment at the column face due to any of these modes meets the requirements of the relationship in Step 6. The block shear failure modes are shown in Figure 3-19. For the purpose of this calculation, the resistance factor f shall be taken as unity.

Step 12: Design a single-plate, bolted shear-tab connection sufficient to resist the shear given by Equation 3-52:

$$V_{web} = \frac{2M_f}{L - d_c} + V_g \tag{3-52}$$

where V_g is the shear at the column face due to factored gravity loads, kips.

Step 13: Calculate continuity plate requirements in accordance with the methods of Section 3.3.3.1 using the width and thickness of the flange plates for the quantities b_f, and t_f, respectively, in that section.

Step 14: Confirm the adequacy of the column size to meet the criteria of Section 2.8.1, considering the hinge location given in Table 3-10.

Step 15: Detail the connection as shown in Fig. 3-17. Bolts should be designed for bearing using a resistance factor f of unity.

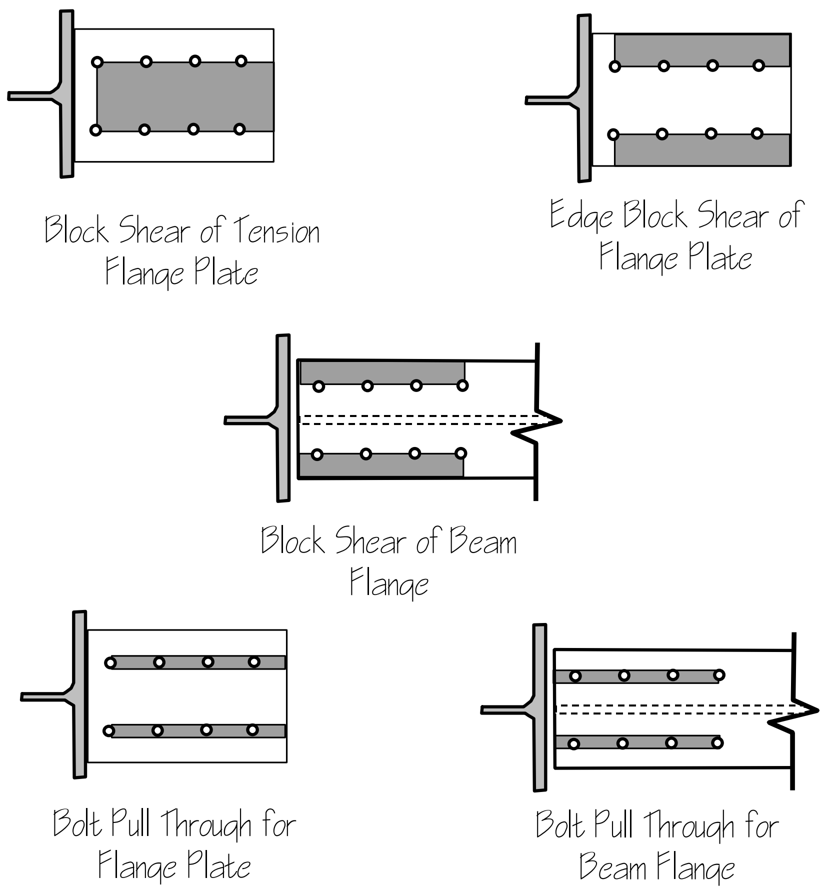


Figure 3-19 Block Shear and Pull-Through Failures

3.7 Prequalified Partially Restrained Connections

This section provides recommended criteria for one type of prequalified full strength / partial stiffness (Partially Restrained (PR)) steel moment-frame connection, suitable for use in new construction. Table 3-11 indicates the connection type, and the structural systems for which it is prequalified. A procedure is also provided for design of this connection type.

Table 3-11 Prequalified Bolted Partially Restrained Connections

Connection Type	Criteria Section	Frame Type
DST	3.7.1	OMF, SMF

Commentary: Several different types of partially restrained connections have been studied under the program of investigations performed in support of the development of these Recommended Criteria as well as under other programs. In preparing these Recommended Criteria, it was judged that sufficient supporting data had been developed for only one connection type to permit prequalification. This is not to suggest that the other types of connections studied are not suitable for use, but rather, that in the judgment of the project team, there were not sufficient data available for these other connection types to meet all of the requirements for prequalification in these Recommended Criteria, as described in Section 3.4. For additional information on other types of PR connections refer to FEMA-355D, State of the Art Report on Connection Performance.

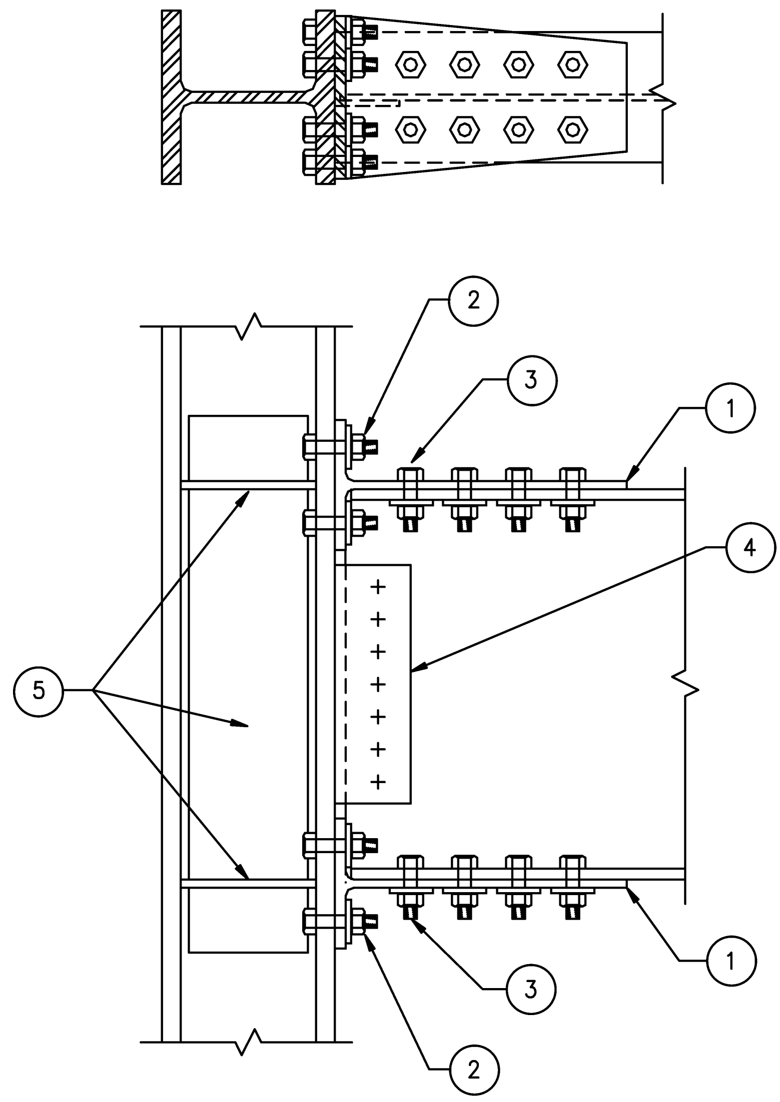
For the purposes of this document, connections are classified as partial stiffness, or Partially Restrained (PR) if the deformation of the connection itself will increase the calculated drift of the frame by more than 10%. The connection is considered to be a full strength connection when it can develop the full expected plastic moment of the beam itself.

Moment connections that develop only partial strength, as well as partial stiffness, are sometimes used. Such connections are not prequalified in this document. This is not intended to preclude their use, if the system is properly justified by both analysis and testing. A significant amount of testing does exist on such connection types, and is described in FEMA-355D.

3.7.1 Double Split Tee Connections

This section provides procedures for design of full-strength, partially restrained, double split tee (DST) connections employing bolted split tee connectors between the beam and column flanges. This type of connection is prequalified for use within the limitations indicated in Table 3-12. Figure 3-20 provides a typical detail for this connection type.

Commentary: The behavior of this type of connection can be controlled by a number of different modes including flexural yielding of the beam section, flexural yielding of the tee stems or flanges, shear yielding of the column panel zone, net-section tensile failure of the beam flange or tee stem, and shear or tension failure of the bolts, depending on the relative proportions of these various components. Some of these modes are brittle, while others have significant ductility. The design procedure contained in this Section is based on inelastic action occurring in preferred modes. The various elements of the connection are then designed with sufficient strength so that other modes are unlikely to occur. FEMA-355D, *State of the Art Report on Connection Performance, provides further discussion of the performance of these connections and summaries of test data and references.*



Notes

1. Split Tee: length, width, and thickness determined by design according to Section 3.7.1.2.
2. Fully pretensioned ASTM A325 or A490 bolts in standard holes sized for bearing. For sizing, see Section 3.7.1.2, Step 7.
3. Fully pretensioned ASTM A325 or A490 bolts in standard holes sized for bearing. For sizing, see Section 3.7.1.2, Step 4.
4. Shear tab welded to column flange with either CJP weld or two-sided fillet weld. For calculation of design strength of shear tab, welds, and bolts, see Section 3.7.1.2, Step 14. Weld: QC/QA Category BM/L.
5. For continuity plates and web doubler plates see Figure 3-6.

Figure 3-20 Double Split Tee (DST) Connection

Table 3-12 Prequalification Data for Full Strength DST Connections (FSDST)

General	
Applicable systems	OMF, SMF
Connection classification	Full-Strength – Partial-Stiffness (PR)
Hinge location distance s_h	End of T-stubs
Critical Beam Parameters	
Maximum depth	OMF: W36 SMF: W24
Minimum span-to-depth ratio	OMF: 5 SMF: 8
Permissible material specifications	A572 Grade 50, A992, A913, Grade 50/S75
Critical Column Parameters	
Depth range	OMF: Not Limited. SMF: W12, W14 Flange width governed by required length of T-stub flange
Permissible material specifications	A572 Grade 50, A913 Grade 50 or 65, A992
Flange thickness	Section 3.7.1.2, Steps 11 and 12.
Critical Beam Column Relations	
Panel zone strength	Section. 3.7.1.2, Step 3
Column/beam bending strength ratio	Sec. 2.9.1
Critical Connection Details	
T-stub Parameters:	
Hole type	Standard
Permissible material specifications	A572 Grade 50, A992
Design method	3.7.1.2
Web connection parameters:	
Shear tab:	
Permissible material specifications	A36, A572 Grade 50
Plate thickness	5/16” to ½”
Hole type	SSLT
Weld type	CJP groove or double fillet. See Fig. 3-20.
Weld metal	Section 3.3.2.4
Double web angle:	
Permissible material specifications	A36, A572 Grade 50
Angle thickness	5/16” to ½”
Hole type	STD, SSLT
Bolt Characteristics:	
Bolt diameter	7/8” or 1”
Bolt grade	A325-X or A490-X
Bolt spacing	3x bolt diameter min.
Installation requirements	Pretensioned
Washers	F436 as required

3.7.1.1 Connection Stiffness

The analysis of frames incorporating partially restrained connections must include explicit consideration of the stiffness of the connection. Stiffness of double split tee connections may be calculated from Equation 3-53:

$$k_s = \frac{d_b M_{fail}}{0.375} \tag{3-53}$$

where:

k_s = the rotational stiffness of the connection, kip-inches/radian

M_{fail} = the lesser of the moments that control the resistance of the connection, taken from steps 4 thru 8.

d_b = the beam depth, inches

or more accurate representations of stiffness may be used, when substantiated by rational analysis and test data.

Commentary: The stiffness of partially restrained connections, by definition, has significant effect on the total drift experienced by the building frame in response to lateral loading. In order to account properly for this effect it is necessary to include consideration of the connection stiffness in the analytical model used to determine building drift and the distribution of the forces on the members of the steel moment frame. This can be done either by explicitly including an element in the structural model with this calculated stiffness, or alternatively by modifying the stiffness of the beams in the model to include the effect of connection stiffness. Guidelines for both approaches are contained in Section 4.5.2.2.

3.7.1.2 Design Procedure

The connection shall be designed so that inelastic behavior is controlled either by flexural yielding of the beam in combination with shear yielding of the column panel zone, or by flexural yielding of the beam alone. The various elements of the connection are proportioned such that the moment at the face of the column, as limited by the controlling yielding behavior considering potential material over-strength and strain hardening, is less than the moment at the face of the column corresponding with failure of any of the other behavioral modes.

Based on the above, the design should proceed following the steps below. The various parameters used in the equations in these steps are defined in Figure 3-21 and 3-22, and in *AISC/LRFD*.

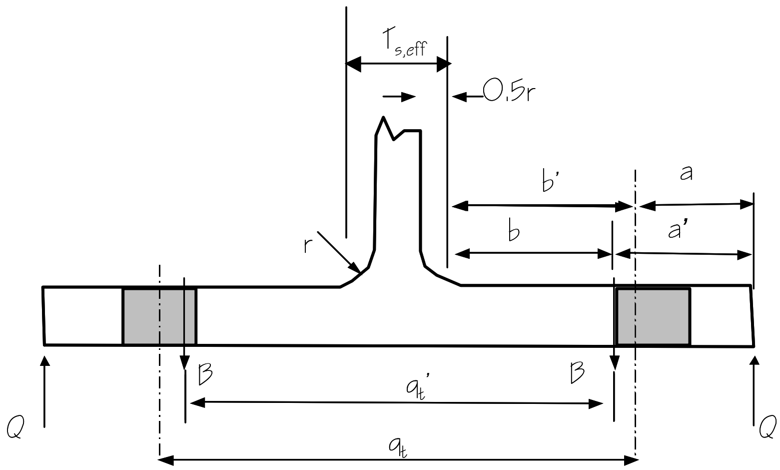


Figure 3-21 Geometry for Prying Forces and Bending of T-Section Flanges

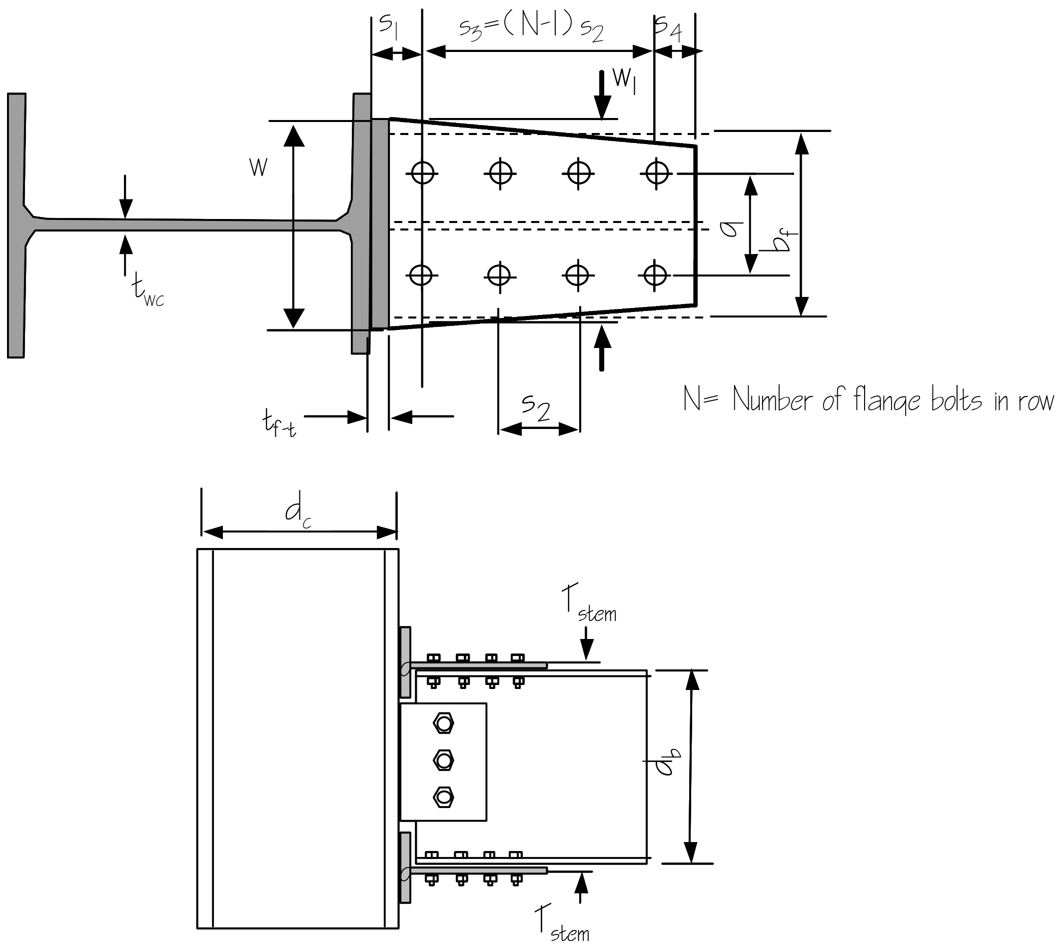


Figure 3-22 Geometry for Other T-Stub Failure Modes

Step 1: Calculate M_f and M_c according to the procedures in Section 3.2.7.

Step 2: Calculate the moment at the face of the column at beam first yield, M_{yf}, in accordance with the procedures of Section 3.2.8.

Step 3: Check the adequacy of the column for panel zone shear in accordance with the procedures of Section 3.3.3.2. For purposes of this calculation, d_b may be assumed to be equal to the distance from the outer end of the tee flange at one beam flange to the bottom of the opposite T stem, and the quantity "d_b-t_{fb}" can be taken as the quantity d_b, described above, minus $t_{stem}/2$. If the thickness of the panel zone is more than 1.5 times that required, it is recommended to use a different combination of beam and column sizes.

Step 4: Select the number, size, and grade of bolts in the beam flange and WT or ST flange. Select the size of the WT or ST, and evaluate the adequacy of the bolts, plate and beam to preclude brittle failure modes in accordance with Steps 5 through 10, where in each case, Equation 3-54 must be satisfied:

$$1.2M_{yf} < M_{fail} \tag{3-54}$$

where:

M_{yf} = Moment at the face of the column at initiation of beam flange yielding, calculated in Step 2 above, and

M_{fail} = Moment at the face of the column at initiation of failure in the specific behavior mode being addressed in Steps 5 through 12.

Step 5: Determine M_{fail} the moment at the face of the column at shear failure of the beam flange bolts in accordance with Equation 3-55 and check for adequacy to meet the criteria of Equation 3-54, in Step 4:

$$M_{fail_{bolts}} = 2NA_b \ (F_{v_{bolt}}) \ d_b L_{TF1} \tag{3-55}$$

where:

A_b = Area of bolt

$F_{v_{bolt}}$ = Nominal shear strength of bolt in bearing-type connections, from *AISC LRFD*.

L_{TF1} = Length ratio to transfer moment from center of bolt group to face of column given by the Equation:

$$L_{TF1} = \frac{L - d_c}{L - d_c - (2S_1 + S_3)} \tag{3-56}$$

N = Number of bolts in the connection of the beam flange to the flange plate

Step 6: Determine the moment M_{fail} at the face of the column at net section fracture of the T stem in accordance with Equation 3-57 and check for adequacy to meet the criteria of Equation 3-54 in Step 4:

$$M_{fail} = F_{uT} \left(w - 2 \left(d_{bt} + .125\right)\right) t_{stem} \left[d_b + t_{stem}\right] L_{TF2} \tag{3-57}$$

where:

w is taken as the lesser of the flange length of the T, the width of the T at the first line of bolts, as defined in Figure 22, or the quantity given by the equation:

$$w \le g + S_3 \tan q_{eff} \tag{3-58}$$

$$15^o \le q_{eff} = 60 t_{stem} \le 30^o \tag{3-59}$$

L_{TF2} is a ratio to transfer moment from the center line of the bolts closest to the column flange to the face of the column, and is given by the equation:

$$L_{TF2} = \frac{L - d_c}{L - d_c - 2S_1} \tag{3-60}$$

Step 7: Determine the moment M_{fail} at the face of the column at initiation of plastic bending of the tee flanges in accordance with Equation 3-61 and check for adequacy to meet the criteria of equation 3-54 in Step 4:

$$M_{fail} = \frac{\left(2a' - \frac{d_{bt}}{4}\right) w \; F_{yT} \; t_f^2 \; \left(d_b - t_{stem}\right)}{4a'b' - d_{bt} \; \left(b' + a'\right)} \tag{3-61}$$

where:

$$a' = a + \frac{d_{bt}}{2} \tag{3-62}$$

$$b' = b - \frac{d_{bt}}{2} \tag{3-63}$$

Step 8: Determine the moment M_{fail} at the face of the column at the initiation of tensile failure of the bolts at the tee flange, considering prying action, in accordance with Equation 3-64 and check for adequacy to meet the criteria of equation 3-54 in Step 4:

$$M_{fail} = N_{tb} \; \left(d_b + t_{stem}\right) \left[T_{ub} + \frac{wF_{yt} \; t_f^2}{16a'}\right] \frac{a'}{a' + b'} \tag{3-64}$$

where T_{ub} is the nominal tensile strength of bolts from the T flanges to the column flange which should be taken as the quantity $90A_{bolt}$ for A325 bolts and $113A_{bolt}$ for A490 bolts.

Step 9: Determine the moment M_{fail} at the face of the column at net section fracture of the beam flange, in accordance with Equation 3-65 and check for adequacy to meet the criteria of equation 3-54 in Step 4:

$$M_{fail} = \{F_{ubm} \left(Z_b - 2\left(d_{bthole} + 0.062\right)t_{fb}\left(d_b - t_{fb}\right)\right)\} \; L_{TF3} \tag{3-65}$$

where:

L_{TF3} is a length ratio to transfer moment from the bolt hole farthest from the column face, to the column face, given by Equation 3-66:

$$L_{TF3} = \frac{L - d_c}{L - d_c - 2(S_1 + S_3)} \tag{3-66}$$

Step 10: Determine the moment M_{fail} at the face of the column at initiation of block shear failure and pull-through patterns of the stem of the tee (See Figure 3-19), according to the methods in *AISC-LRFD.*

Step 11: Calculate the adequacy of column flange thickness for beam flange tension, in accordance with the equation:

$$t_{cf} \geq 1.5t_{f-t} \tag{3-67}$$

If the column flange thickness is less than that calculated in accordance with Equation 3-67, continuity plates are required. Continuity plates should be designed as described in Section 3.3.3.1.

Step 12: Calculate the adequacy of column web thickness for the beam flange compression forces, in accordance with the equation:

$$t_{wc} \geq \frac{M_f}{(d_b - t_{stem})(6k + c)\,F_{yc}} \tag{3-68}$$

where k is the dimension of the column-flange-to-web fillet, as indicated in *AISC Manual*.

If the column web thickness does not meet the criteria of Equation 3-68, then provide continuity plates in accordance with the criteria of Section 3.3.3.1.

Step 13: If continuity plates are required, the column flange thickness must be equal to or larger than the flange thickness, t_{ft}, of the T. If the column flange thickness is less than this amount, a column with a thicker flange must be selected.

Step 14: Design the shear connection between the beam web and column as a standard shear tab welded to the column and bolted to the beam. Bolts shall be sized for bearing using a resistance factor ϕ of unity. Design load for the shear tab shall be taken as given by the equation:

$$V_{st} = 2\frac{M_f}{L - d_c} + V_g \tag{3-69}$$

where:
V_{st} = Design shear force for the shear tab
V_g = Factored gravity load

Step 15: Detail the connection as shown in Figure 3-20.

3.8 Proprietary Connections

This section presents information on several types of fully restrained connections that have been developed on a proprietary basis. These connections are not categorized in these *Recommended Criteria* as prequalified, as the SAC Joint Venture has not examined the available supporting data in sufficient detail to confirm that they meet appropriate prequalification criteria. However, these proprietary connections have been evaluated by some enforcement agencies and found to be acceptable for specific projects and in some cases for general application within the jurisdiction's authority. Use of these technologies without the express permission of the licensor may be a violation of intellectual property rights, under the laws of the United States.

Discussion of several types of proprietary connections are included herein. Other proprietary connections may also exist. Inclusion or exclusion of proprietary connections in these *Recommended Criteria* should not be interpreted as either an approval or disapproval of these systems. The descriptions of these connections contained herein have in each case been prepared by the developer or licensor of the technology. This information has been printed with their permission. Neither the Federal Emergency Management Agency nor the SAC Joint Venture endorses any of the information provided or any of the claims made with regard to the attributes of these technologies or their suitability for application to specific projects. Designers wishing to consider specific proprietary connections for use in their structures should consult both the licensor of the connection and the applicable enforcement agency to determine the applicability and acceptability of the individual connection for the specific design application.

3.8.1 Side Plate

The proprietary side plate (SP) connection system is a patented technology shown schematically in Figure 3-23 for new construction. Physical separation between the face of column flange and end of beam eliminates peaked triaxial stress concentrations. Physical separation is achieved by means of parallel full-depth side plates that eliminate reliance on through-thickness properties and act as discrete continuity elements to sandwich and connect the beam and the column. The increased stiffness of the side plates inherently stiffens the global frame structure and eliminates reliance on panel zone deformation by providing three panel zones [i.e., the two side plates plus the column's own web]. Top and bottom beam flange cover plates are used, when dimensionally necessary, to bridge the difference between flange widths of the beam(s) and the column.

This connection system uses all shop fillet-welded fabrication. All fillet welds are made in either the flat or horizontal position using column tree construction. Shop fabricated column trees and link beams are erected and joined in the field using one of four link beam splice options to complete the moment-resisting frame. Link beam splice options include a fully welded CJP butt joint, bolted matching end plates, fillet-welded flange plates, and bolted flange plates.

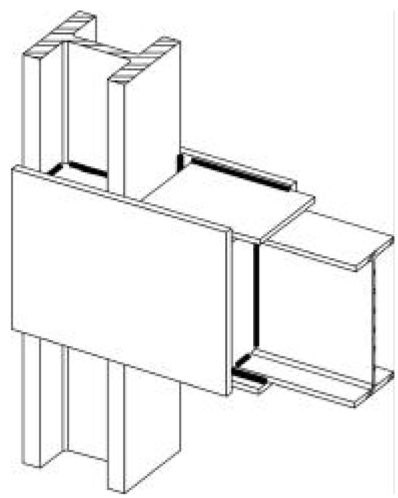


Figure 3-23 Proprietary Side Plate Connection

All connection fillet welds are loaded principally in shear along their length. Moment transfer from the beam to the side plates, and from the side plates to the column, is accomplished with plates and fillet welds using equivalent force couples. Beam shear transfer from the beam's web to the side plates is achieved with vertical shear plates and fillet welds. The side plates are designed with adequate strength and stiffness to force all significant plastic behavior of the connection system into the beam, in the form of flange and web local buckling centered at a distance of approximately 1/3 the depth of the beam away from the edge of the side plates.

All full-scale cyclic testing of this connection system was conducted at the Charles Lee Powell Structural Research Laboratories, University of California, San Diego, under the direction of Professor Chia-Ming Uang. Testing includes both prototype uniaxial and biaxial dual strong axis tests. Independent corroborative nonlinear analyses were conducted by the University of Utah and by Myers, Houghton & Partners, Structural Engineers.

Independent prequalification of this connection system was determined by ICBO Evaluation Service, Inc., in accordance with *ICBO ES Acceptance Criteria for Qualification of Steel Moment Frame Connection Systems (AC 129-R1-0797),* and was corroborated by the City of Los Angeles Engineering Research Section, Department of Building and Safety, which collectively invoke the qualification procedures contained in: *FEMA 267/267A/267B*; *AISC Seismic Provisions for Structural Steel Buildings, dated April 15, 1997*; and *County of Los Angeles*

Current Position on Design and Construction of Welded Moment Resisting Frame Systems CP-2, dated August 14, 1996. Refer to *ICBO Evaluation Service, Inc., Evaluation Report No. 5366, issued January 1, 1999*, and to *City of Los Angeles Research Report: COLA RR 25393* for allowable values and/or conditions of use. Additional independent jurisdictional scrutiny of this connection system, by Karl H. Frank, Ph.D., Egor P. Popov, Ph.D., C. Mark Saunders, S.E., and Robert L. Schwein, P.E. is contained in the *Los Angeles County Technical Advisory Panel (LACO-TAP) SMRF Bulletin No. 3 on Steel Moment-Resisting Frame Connection Systems, County of Los Angeles, Department of Public Works, dated March 4, 1997*. Additional design information for this connection type may be obtained from the licensor.

3.8.2 Slotted Web

The proprietary Slotted Web (SW) connection (Seismic Structural Design Associates, Inc. US Patent No. 5,680,738 issued 28 October 1997) is shown schematically in Figure 3-24. It is similar to the popular field-welded–field-bolted beam-to-column moment-frame connection, shown in the current AISC LRFD and ASD steel design manuals, that has become known as the “pre-Northridge” connection. Based upon surveys of seismic connection damage, modes of fracture, reviews of historic tests, and recent ATC-24 protocol tests, it was concluded by SEAOC (1996 Blue Book Commentary) that the pre-Northridge connection is fundamentally flawed and should not be used in the new construction of seismic moment frames. Subsequent finite element analyses and strain gage data from ATC-24 tests of this pre-Northridge connection have shown large stress and strain gradients horizontally across and vertically through the beam flanges and welds at the face of the column. These stress gradients produce a prying moment in the beam flanges at the weld access holes and in the flange welds at the column face that lead to beam flange and weld fractures and column flange divot modes of connection fracture. Moreover, these same studies have also shown that a large component, typically 50%, of the vertical beam shear and all of the beam moment, is carried by the beam flanges/welds in the pre-Northridge connection.

However, by (1) separating the beam flanges from the beam web in the region of the connection and (2) welding the beam web to the column flange, the force, stress and strain distributions in this field-welded-field-bolted connection are changed dramatically in the following ways:

1. The vertical beam shear in the beam flanges/welds is reduced from typically 50% to typically 3% so that essentially all vertical shear is transferred to the column through the beam web and shear plate.

2. Since most W-sections have a flange-to-beam modulus ratio of $0.65 < Z_{flg} / Z < 0.75$, both the beam web and flange separation and the beam web-to-column-flange weldment force the beam web to resist its portion of the total beam moment.

3. The beam web separation from the beam flange reduces the large stress and strain gradients across and through the beam flanges by permitting the flanges to flex out of plane. Typically, the elastic stress and strain concentration factors (SCFs) are reduced from 4.0 to 5.0 down to

1.2 to 1.4, which dramatically reduces the beam flange prying moment and the accumulated plastic strain and ductility demand under cyclic loading. These attributes enhance and extend the fatigue life of this moment frame connection.

4. The lateral-torsional mode of beam buckling that is characteristic of non-slotted beams is circumvented. The separation of the beam flanges and beam web allow the flanges and web to buckle independently and concurrently, which eliminates the twisting mode of buckling and its associated torsional beam flange/weld stresses. Elimination of this buckling mode is particularly important when the exterior cladding of the building is supported by seismic moment frames that are located on the perimeter of the building.

5. Residual weldment stresses are significantly reduced. The separation of the beam web and flanges in the region of the connection provides a long structural separation between the vertical web and horizontal flange weldments.

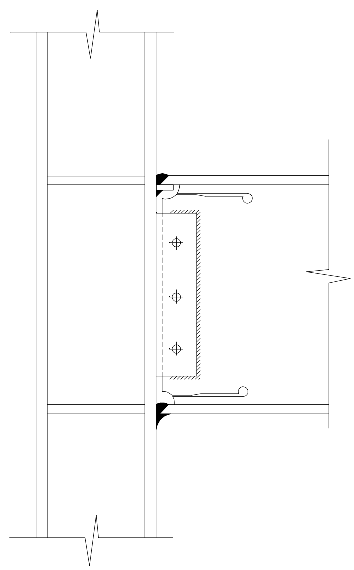


Figure 3-24 Proprietary Slotted Web Connection

The SW connection design rationale that sizes the beam/web separation length, shear plate and connection weldments, is based upon ATC-24 protocol test results and inelastic finite element analyses of the stress and strain distributions and buckling modes. Incorporated in this rationale are the UBC and AISC Load and Resistance Factor Design (LRFD) Specifications and the *AISC Seismic Provisions for Structural Steel Buildings*.

SSDA has successfully completed ATC-24 protocol tests on beams ranging from W27x94 to W36x280 using columns ranging from W14x176 to W14x550. None of these assemblies experienced the lateral-torsional mode of buckling that is typical of non-slotted beam and column assemblies.

Both analytical studies and ATC-24 protocol tests have demonstrated that the Seismic Structural Design Associates Slotted Web connection designs develop the full plastic moment capacity of the beam and do not reduce the elastic stiffness of the beam. All of the above attributes of this proprietary connection enhance its strength and ductility, which makes it applicable for use in new construction in seismic moment frames. Specific qualification and design information for the Slotted Web connection may be obtained from the licensor.

3.8.3 Bolted Bracket

The Bolted Bracket (BB) connection type is shown schematically in Figure 3-25. Beam shear and flexural stresses are transferred to the column through a pair of heavy, bolted brackets, located at the top and bottom beam flanges. The concept of using bolted brackets to connect beams to columns rigidly is within the public domain. However, generic prequalification data have not been developed for this connection type. One licensor has developed patented steel castings of the bolted brackets, for which specific design qualification data has been prepared. Specific qualification and design information for this connection type may be obtained from the licensor.

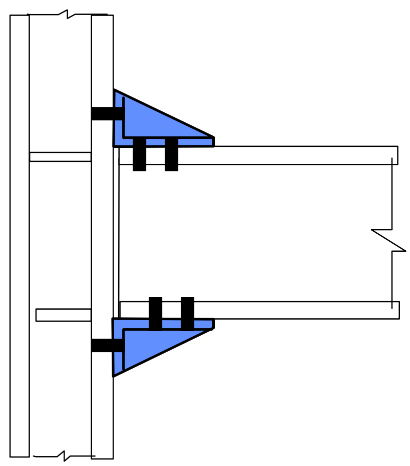


Figure 3-25 Bolted Bracket Connection

3.8.4 Reduced Web

The reduced web (RW) section utilizes capacity design principles to protect the beam column connection from high stresses by introducing large openings in the web. The openings are large enough to cause yielding of the web along the beam span, allowing the connection region to remain nominally elastic. The configuration of openings can be adjusted to control the yielding mechanism and yield strength. Two configurations are illustrated in Figure 3-26.

An understanding of the utility of this system for resisting seismic actions is developing. At this writing, five W21x68 Grade 50 beams have been tested under reversed cyclic loading using the modified SAC loading protocol. Stable hysteretic loops were maintained to interstory drifts as high as 6%, and the predicted deformation mechanisms developed. Modified pre-Northridge details, consisting of a field-bolted web connection and full penetration flange welds were shown to be successful. These followed the detailing recommended for the WUF-B connection except the web was not welded. Use of the beams may reduce construction cost if mechanical equipment is passed through the openings, thereby allowing story heights to be reduced. The technology is protected by US Patent 6,012,256; inquiries are welcome.

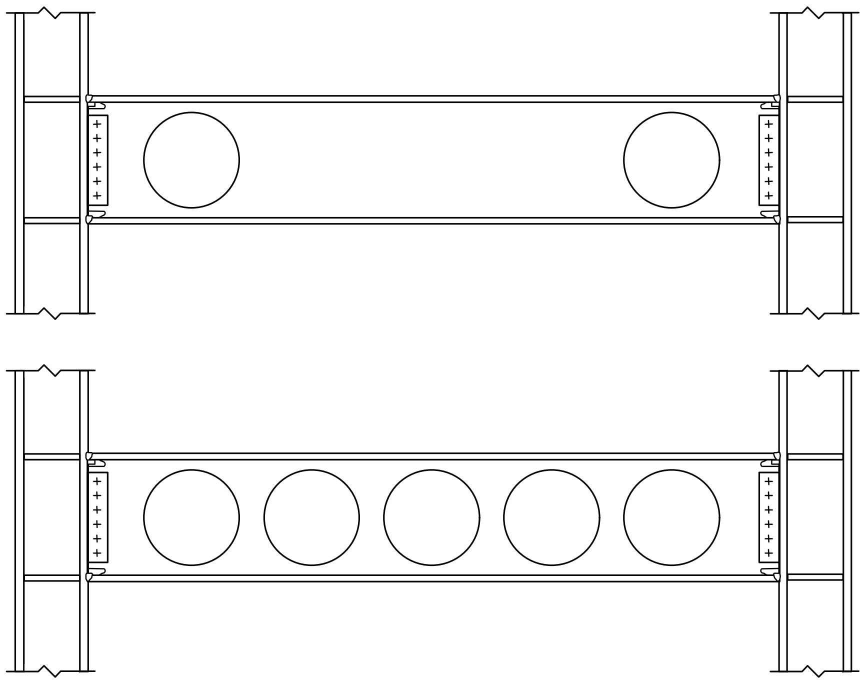


Figure 3-26 Reduced Web Connection

3.9 Project-Specific Connection Qualification

This section provides criteria for design and project-specific qualification of connections for which there is no current prequalification or for prequalified connections that are to be utilized outside the parametric limitations for the applicable prequalification. Project-specific qualification includes a program of connection assembly prototype testing supplemented by a suitable analytical procedure that permits prediction of behaviors identified in the testing program.

Commentary: While it is not the intent of these Recommended Criteria to require testing for most design situations, there will arise circumstances where none of the prequalified connections will be appropriate, or where a prequalified connection must be used outside the parametric limits for which it is prequalified. In these situations, these criteria recommend a program of prototype testing in addition to analytically based connection design, reflecting the view that the behavior of connection assemblies under severe cyclic loading cannot be reliably predicted by analytical means alone. The program of laboratory testing is used to demonstrate that the behavioral modes of the connection are predictable and that the connection assembly is capable of adequate performance. The testing is accompanied by an analytical procedure that permits the connection design to be applied to framing sizes that are not identical to those used in the tests, while *retaining confidence that the connection will continue to behave as demonstrated by the testing.*

Testing is costly and time consuming, and these recommendations attempt to keep testing requirements as simple as possible. Test conditions should match the conditions in the structure as closely as possible, but it is recognized that test setups simultaneously account for the behavior and interdependence of many variables whose behavior is understood imprecisely. Where conditions in the structure differ significantly from the conditions implied in this section, additional testing to that recommended in these criteria may be required.

3.9.1 Testing Procedures

The testing program should follow the requirements of Appendix S of AISC Seismic with the exceptions and modifications discussed below. The program should include tests of at least two specimens for a given combination of beam and column size. The results of the tests should be capable of predicting the median value of the drift angle capacity for the performance states described in Table 3-13. The interstory drift angle q shall be defined as indicated in Figure 3-27. Acceptance criteria shall be as indicated in Section 3.9.2.

Table 3-13 Interstory Drift Angle Limits for Various Performance Levels

Performance Level	Symbol	Drift Angle Capacity
Strength degradation	q_{SD}	Taken as that value of q, from Figure 3-27 at which either failure of the connection occurs or the strength of the connection degrades to less than the nominal plastic capacity, whichever is less.
Ultimate	q_U	Taken as that value of q, from Figure 3-27 at which connection damage is so severe that continued ability to remain stable under gravity loading is uncertain.

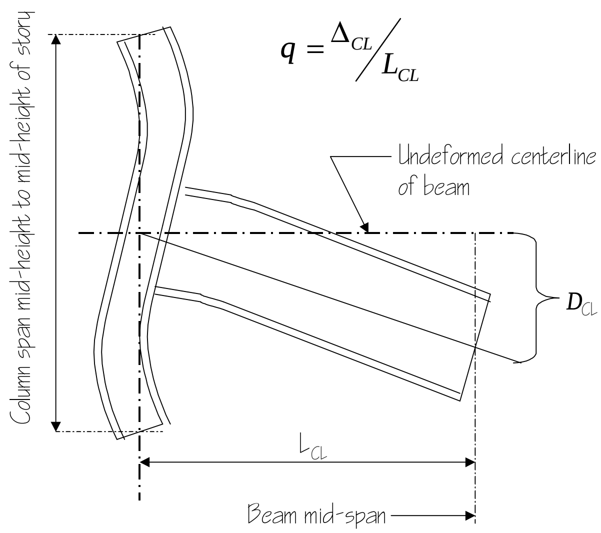


Figure 3-27 Angular Rotation of Test Assembly

The following modifications and clarifications apply to Appendix S of the 1997 *AISC Seismic Provisions* as modified by Supplement No. 1:

- In lieu of the requirements in Section S5.2, the size of the beam used in the test specimen shall be at least the largest depth and heaviest weight used in the structure. The column shall be selected to represent properly the anticipated inelastic action of the column in the real structure for the beam used in the test specimen. Extrapolation beyond the limits stated in this section is not recommended.
- As an alternative to the loading sequence specified in Section S6.3, the FEMA/SAC loading protocol (Krawinkler et al., 2000) is considered acceptable. In the basic loading history, the cycles shall be symmetric in peak deformations. The history is divided into steps and the peak deformation of each step *j* is given as q_j, a predetermined value of the drift angle. The loading history, shown in Table 3-14, is defined by the following parameters:

  q_i = the peak deformation in load step *j*

  n_j = the number of cycles to be performed in load step *j*

Table 3-14 Numerical Values of q_j and n_j

Load Step #	**Peak deformation** q	**Number of cycles, *n***
1	0.00375	6
2	0.005	6
3	0.0075	6
4	0.01	4
5	0.015	2
6	0.02	2
7	0.03	2

Continue incrementing θ in steps of 0.01 radians, and perform two cycles at each step until assembly failure occurs. Failure shall be deemed to occur when the peak loading in a cycle falls to 20% of that obtained at maximum load or, if the assembly has degraded, to a state at which stability under gravity load becomes uncertain.

Commentary: The AISC Seismic Provisions (AISC, 1997) have been adopted by reference into the 1997 NEHRP Recommended Provisions for New Buildings. The AISC Seismic Provisions include, and require the use of, Appendix S – Qualifying Cyclic Tests of Beam-to-Column and Link-to-Column Connections, for qualification of connections that are not prequalified. Appendix S includes a complete commentary on the requirements.

Under Appendix S, the Test Specimen must represent the largest beam anticipated in the project. The column must be selected to provide a flexural *strength consistent with the strong-column-weak-beam requirements and panel zone strength requirements. The permissive weight and size limits contained in Section S5.2 of Appendix S have been eliminated.*

The AISC loading history and acceptance criteria are described in terms of plastic rotation, while the FEMA/SAC loading protocol, acceptance criteria, and design recommendations, contained in these Recommended Criteria, are controlled by total drift angle, as previously defined. The engineer should ensure that appropriate adjustments are made when using the AISC loading history with these Recommended Criteria.

The calculation of q *illustrated in Figure 3-27 assumes that the top and the bottom of the column are restrained against lateral translation. The height of the test specimen column should be similar to that of the actual story height to prevent development of unrealistically large contributions to* q *from flexure of the column. In general, total drift angle is approximately equal to plastic rotation, measured as indicated in Figure 3-27, plus 0.01 radians. However, the engineer is cautioned that plastic rotation demand is often measured in different ways and may require transformation to be consistent with the measurement indicated in Figure 3-27.*

3.9.2 Acceptance Criteria

For frames of typical configuration conforming in all respects to the applicable requirements of *FEMA-302*, and Chapter 2 of these *Recommended Criteria*, the median value of the interstory drift angle capacity at strength degradation, q_{SD}, and at connection failure, θ_U, obtained from qualification testing shall not be less than indicated in Table 3-15. The coefficient of variation for these two parameters shall not exceed 10% unless the mean value, less one standard deviation, is also not less than the value indicated in Table 3-15.

Table 3-15 Minimum Qualifying Total Interstory Drift Angle Capacities, q_{SD}, and q_U for OMF and SMF Systems

Structural System	Qualifying Drift Angle Capacity – Strength Degradation, q_{SD} (radians)	Qualifying Drift Angle Capacity – Ultimate, q_U (radians)
OMF	0.02	0.03
SMF	0.04	0.06

Note:
Refer to Section 4.6.2.2.2 for definitions of q_{SD} and q_U

Where the clear-span-to-depth ratio of beams in the steel moment frame is less than 8, the qualifying total drift angle capacities indicated in Table 3-15 shall be increased to q'_{SD} and q'_U, given by equations 3-70 and 3-71, respectively:

$$q'_{SD} = \frac{8d}{L}\left(1+\frac{L-L'}{L}\right)q_{SD} \tag{3-70}$$

$$q'_U = \left(1+\frac{L-L'}{L}\right)q_U \tag{3-71}$$

where: q'_{SD} = Qualifying strength degradation drift angle capacity for spans with $L/d < 8$
q_{SD} = the basic qualifying strength degradation drift angle capacity, in accordance with Table 3-15
q'_U = the qualifying ultimate drift angle capacity, for spans with $L/d < 8$
q_U = the basic qualifying ultimate drift angle capacity, in accordance with Table 3-15
L = the center-to-center spacing of columns, from Figure 3-1, inches.
L' = the distance between points of plastic hinging in the beam, inches.
d = depth of beam in inches

Commentary: This section sets criteria for use in project-specific qualification of connections, in accordance with Section 3.9, and for development of new connection prequalifications in accordance with Section 3.10 of these Recommended Criteria. Two interstory drift angle capacities are addressed. The values indicated in Table 3-15 formed the basis for extensive probabilistic evaluations of the performance capability of various structural systems, reported in FEMA-355F, State of the Art Report on Performance Prediction and Evaluation. These probabilistic evaluations indicate a high confidence, on the order of 90%, that regular, well-configured frames meeting the requirements of FEMA-302 and constructed with connections having these capabilities, can meet the intended performance objectives with regard to protection against global collapse, and moderate confidence, on the order of 50%, that connections can resist maximum considered earthquake demands without local life-threatening damage.

Connection details with capacities lower than those indicated in this section should not be incorporated in structures unless a specific probabilistic analysis using the performance evaluation procedures contained in Chapter 4 and Appendix A of these Recommended Criteria indicates that an acceptable level of confidence of adequate performance can be obtained.

Connections in frames where beam span-to-depth ratios are less than those used for the prequalification testing, will experience larger flange strains, at the plastic hinges at a particular frame drift, than those tested. For this reason, connections used in such frames need to be qualified for larger drifts as indicated *by the formulae in this section, unless the frames are designed to experience proportionately lower drifts than permitted by FEMA-302.*

3.9.3 Analytical Prediction of Behavior

Connection qualification should include development of an analytical procedure to predict the limit states of the connection assembly, as demonstrated by the qualification tests. The analytical procedure should permit identification of the strength and deformation demands and limit states on various elements of the assembly at the various stages of behavior. The analytical procedure should be sufficiently detailed to permit design of connections employing members similar to those tested within the limits identified in Section S5.2 of AISC Seismic.

Commentary: It is important for the designer to have an understanding of the limiting behaviors of any connection detail so that the detail may be designed and specified on a rational basis for assemblies that vary, within specified limits, from those tested.

3.10 Prequalification Testing Criteria

This section provides guidelines for prequalification of connections for which there is no current prequalification or to extend the parametric limitations for prequalification listed in Sections 3.5 and 3.6. Prequalification includes a program of connection assembly prototype testing supplemented by a suitable analytical procedure that permits prediction of behaviors identified in the testing program.

Commentary: The purpose of this section is to provide recommended procedures for prequalification of a connection that is not currently prequalified in these Recommended Criteria or to extend the range of member sizes that may be used with currently prequalified connections for general application. These criteria are intended to require significantly more testing than are required for a project-specific qualification program, as once a connection is prequalified, it can see wide application. Prequalification of a connection should incorporate the testing described in this section as well as due consideration of the four criteria described in the Commentary for Section 3.4.

The potential for limit states leading to local collapse (i.e. loss of gravity-load capacity) is an important consideration in evaluating the performance of a prototype connection. Establishing this limit state required by Section 3.9.1 will necessitate imposing large deformations on the connection. This will require loading setups capable of delivering long strokes while withstanding correspondingly large out-of-plane or large torsional deformations. Many tests are terminated before the ultimate failure of the connection to protect the loading apparatus. These early terminations will limit the range over which a connection may be prequalified.

3.10.1 Prequalification Testing

Testing and acceptance criteria should follow the recommendations in Section 3.9 except that at least five non-identical test specimens shall be used. The resulting range of member sizes that will be prequalified should be limited to the range represented by the tested specimens.

3.10.2 Extending the Limits on Prequalified Connections

Testing and acceptance criteria should follow the recommendations in Section 3.9 except that at least two non-identical test specimens shall be tested. The resulting range of member size that will be prequalified should be limited to the those contained in the data base of tests for the connection type.