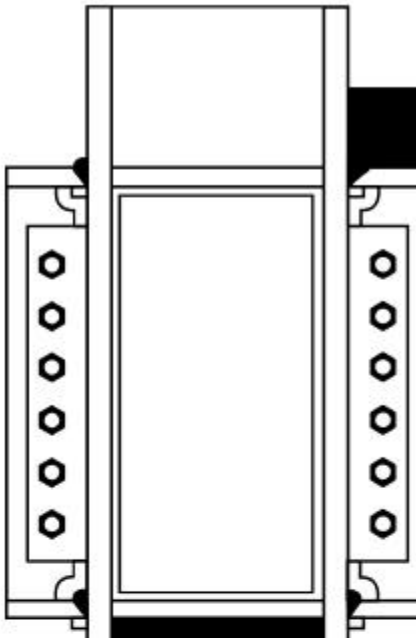

Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings

Program to Reduce the Earthquake Hazards of Steel Moment-Frame Structures

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DISCLAIMER

This document provides recommended criteria for the design of steel moment-frame buildings to resist the effects of earthquakes. These recommendations were developed by practicing engineers, based on professional judgment and experience, and by a program of laboratory, field and analytical research. While every effort has been made to solicit comments from a broad selection of the affected parties, this is not a consensus document. It is primarily intended as a resource document for organizations with appropriate consensus processes for the development of future design standards and building code provisions. **No warranty is offered, with regard to the recommendations contained herein, either by the Federal Emergency Management Agency, the SAC Joint Venture, the individual Joint Venture partners, or their directors, members or employees. These organizations and their employees do not assume any legal liability or responsibility for the accuracy, completeness, or usefulness of any of the information, products or processes included in this publication. The reader is cautioned to review carefully the material presented herein and exercise independent judgment as to its suitability for application to specific engineering projects.** These recommended criteria have been prepared by the SAC Joint Venture with funding provided by the Federal Emergency Management Agency, under contract number EMW-95-C-4770.

Cover Art. The beam-column connection assembly shown on the cover depicts the standard detailing used in welded steel moment-frame construction prior to the 1994 Northridge earthquake. This connection detail was routinely specified by designers in the period 1970-1994 and was prescribed by the *Uniform Building Code* for seismic applications during the period 1985-1994. It is no longer considered to be an acceptable design for seismic applications. Following the Northridge earthquake, it was discovered that many of these beam-column connections had experienced brittle fractures at the joints between the beam flanges and column flanges.

Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings

SAC Joint Venture

A partnership of
Structural Engineers Association of California (SEAOC)
Applied Technology Council (ATC)
California Universities for Research in Earthquake Engineering (CUREe)

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THE SAC JOINT VENTURE

SAC is a joint venture of the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC), and California Universities for Research in Earthquake Engineering (CUREe), formed specifically to address both immediate and long-term needs related to solving performance problems with welded, steel moment-frame connections discovered following the 1994 Northridge earthquake. SEAOC is a professional organization composed of more than 3,000 practicing structural engineers in California. The volunteer efforts of SEAOC's members on various technical committees have been instrumental in the development of the earthquake design provisions contained in the *Uniform Building Code* and the 1997 *National Earthquake Hazards Reduction Program (NEHRP) Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*. ATC is a nonprofit corporation founded to develop structural engineering resources and applications to mitigate the effects of natural and other hazards on the built environment. Since its inception in the early 1970s, ATC has developed the technical basis for the current model national seismic design codes for buildings; the *de-facto* national standard for postearthquake safety evaluation of buildings; nationally applicable guidelines and procedures for the identification, evaluation, and rehabilitation of seismically hazardous buildings; and other widely used procedures and data to improve structural engineering practice. CUREe is a nonprofit organization formed to promote and conduct research and educational activities related to earthquake hazard mitigation. CUREe's eight institutional members are the California Institute of Technology, Stanford University, the University of California at Berkeley, the University of California at Davis, the University of California at Irvine, the University of California at Los Angeles, the University of California at San Diego, and the University of Southern California. These university earthquake research laboratory, library, computer and faculty resources are among the most extensive in the United States. The SAC Joint Venture allows these three organizations to combine their extensive and unique resources, augmented by consultants and subcontractor universities and organizations from across the nation, into an integrated team of practitioners and researchers, uniquely qualified to solve problems related to the seismic performance of steel moment-frame structures.

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1. INTRODUCTION

1.1 Purpose

This report, *FEMA-350 – Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings* has been developed by the SAC Joint Venture under contract to the Federal Emergency Management Agency (FEMA) to provide organizations engaged in the development of consensus design standards and building code provisions with recommended criteria for the design and construction of new buildings incorporating moment-resisting steel frame construction to resist the effects of earthquakes. It is one of a series of companion publications addressing the issue of the seismic performance of steel moment-frame buildings. The set of companion publications includes:

- *FEMA-350 – Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings.* This publication provides recommended criteria, supplemental to *FEMA-302 – 1997 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures,* for the design and construction of steel moment-frame buildings and provides alternative performance-based design criteria.
- *FEMA-351 – Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings.* This publication provides recommended methods to evaluate the probable performance of existing steel moment-frame buildings in future earthquakes and to retrofit these buildings for improved performance.
- *FEMA-352 – Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings.* This publication provides recommendations for performing postearthquake inspections to detect damage in steel moment-frame buildings following an earthquake, evaluating the damaged buildings to determine their safety in the postearthquake environment, and repairing damaged buildings.
- *FEMA-353 – Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications.* This publication provides recommended specifications for the fabrication and erection of steel moment frames for seismic applications. The recommended design criteria contained in the other companion documents are based on the material and workmanship standards contained in this document, which also includes discussion of the basis for the quality control and quality assurance criteria contained in the recommended specifications.

The information contained in these recommended design criteria, hereinafter referred to as *Recommended Criteria,* is presented in the form of specific design and performance evaluation procedures together with supporting commentary explaining part of the basis for these recommendations. Detailed derivations and explanations of the basis for these design and evaluation recommendations may be found in a series of State of the Art Reports prepared in parallel with these *Recommended Criteria*. These reports include:

- *FEMA-355A – State of the Art Report on Base Metals and Fracture.* This report summarizes current knowledge of the properties of structural steels commonly employed in building construction, and the production and service factors that affect these properties.
- *FEMA-355B – State of the Art Report on Welding and Inspection*. This report summarizes current knowledge of the properties of structural welding commonly employed in building construction, the effect of various welding parameters on these properties, and the effectiveness of various inspection methodologies in characterizing the quality of welded construction.
- *FEMA-355C – State of the Art Report on Systems Performance of Steel Moment Frames Subject to Earthquake Ground Shaking.* This report summarizes an extensive series of analytical investigations into the demands induced in steel moment-frame buildings designed to various criteria, when subjected to a range of different ground motions. The behavior of frames constructed with fully restrained, partially restrained and fracture-vulnerable connections is explored for a series of ground motions, including motion anticipated at near-fault and soft-soil sites.
- *FEMA-355D – State of the Art Report on Connection Performance.* This report summarizes the current state of knowledge of the performance of different types of moment-resisting connections under large inelastic deformation demands. It includes information on fully restrained, partially restrained, and partial strength connections, both welded and bolted, based on laboratory and analytical investigations.
- *FEMA-355E – State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes.* This report summarizes investigations of the performance of steel moment-frame buildings in past earthquakes, including the 1995 Kobe, 1994 Northridge, 1992 Landers, 1992 Big Bear, 1989 Loma Prieta and 1971 San Fernando events.
- *FEMA-355F – State of the Art Report on Performance Prediction and Evaluation of Steel Moment-Frame Buildings*. This report describes the results of investigations into the ability of various analytical techniques, commonly used in design, to predict the performance of steel moment-frame buildings subjected to earthquake ground motion. Also presented is the basis for performance-based evaluation procedures contained in the design criteria documents, *FEMA-350, FEMA-351*, and *FEMA-352*.

In addition to the recommended design criteria and the State of the Art Reports, a companion document has been prepared for building owners, local community officials and other non-technical audiences who need to understand this issue. *A Policy Guide to Steel Moment Frame Construction (FEMA-354)* addresses the social, economic, and political issues related to the earthquake performance of steel moment-frame buildings. *FEMA-354* also includes discussion of the relative costs and benefits of implementing the recommended criteria.

1.2 Intent

These *Recommended Criteria* are primarily intended as a resource document for organizations engaged in the development of building codes and consensus standards for regulation of the design and construction of steel moment-frame structures that may be subject to the effects of earthquake

ground shaking. These criteria have been developed by professional engineers and researchers, based on the findings of a large multi-year program of investigation and research into the performance of steel moment-frame structures. Development of these recommended criteria was not subjected to a formal consensus review and approval process, nor was formal review or approval obtained from SEAOC's technical committees. However, it did include broad external review by practicing engineers, researchers, fabricators, and the producers of steel and welding consumables. In addition, two workshops were convened to obtain direct comment from these stakeholders on the proposed recommendations.

1.3 Background

For many years, the basic intent of the building code seismic provisions has been to provide buildings with an ability to withstand intense ground shaking without collapse, but potentially with some significant structural damage. In order to accomplish this, one of the basic principles inherent in modern code provisions is to encourage the use of building configurations, structural systems, materials and details that are capable of ductile behavior. A structure is said to behave in a ductile manner if it is capable of withstanding large inelastic deformations without significant degradation in strength, and without the development of instability and collapse. The design forces specified by building codes for particular structural systems are related to the amount of ductility the system is deemed to possess. Generally, structural systems with more ductility are designed for lower forces than less ductile systems, as ductile systems are deemed capable of resisting demands that are significantly greater than their elastic strength limit. Starting in the 1960s, engineers began to regard welded steel moment-frame buildings as being among the most ductile systems contained in the building code. Many engineers believed that steel moment-frame buildings were essentially invulnerable to earthquake-induced structural damage and thought that should such damage occur, it would be limited to ductile yielding of members and connections. Earthquake-induced collapse was not believed possible. Partly as a result of this belief, many large industrial, commercial and institutional structures employing steel moment-frame systems were constructed, particularly in the western United States.

The Northridge earthquake of January 17, 1994 challenged this paradigm. Following that earthquake, a number of steel moment-frame buildings were found to have experienced brittle fractures of beam-to-column connections. The damaged buildings had heights ranging from one story to 26 stories, and a range of ages spanning from buildings as old as 30 years to structures being erected at the time of the earthquake. The damaged buildings were spread over a large geographical area, including sites that experienced only moderate levels of ground shaking. Although relatively few buildings were located on sites that experienced the strongest ground shaking, damage to buildings on these sites was extensive. Discovery of these unanticipated brittle fractures of framing connections, often with little associated architectural damage, was alarming to engineers and the building industry. The discovery also caused some concern that similar, but undiscovered, damage may have occurred in other buildings affected by past earthquakes. Later investigations confirmed such damage in a limited number of buildings affected by the 1992 Landers, 1992 Big Bear and 1989 Loma Prieta earthquakes.

In general, steel moment-frame buildings damaged by the Northridge earthquake met the basic intent of the building codes. That is, they experienced limited structural damage, but did

not collapse. However, the structures did not behave as anticipated and significant economic losses occurred as a result of the connection damage, in some cases, in buildings that had experienced ground shaking less severe than the design level. These losses included direct costs associated with the investigation and repair of this damage as well as indirect losses relating to the temporary, and in a few cases, long-term, loss of use of space within damaged buildings.

Steel moment-frame buildings are designed to resist earthquake ground shaking based on the assumption that they are capable of extensive yielding and plastic deformation, without loss of strength. The intended plastic deformation consists of plastic rotations developing within the beams, at their connections to the columns, and is theoretically capable of resulting in benign dissipation of the earthquake energy delivered to the building. Damage is expected to consist of moderate yielding and localized buckling of the steel elements, not brittle fractures. Based on this presumed behavior, building codes permit steel moment-frame buildings to be designed with a fraction of the strength that would be required to respond to design level earthquake ground shaking in an elastic manner.

Steel moment-frame buildings are anticipated to develop their ductility through the development of yielding in beam-column assemblies at the beam-column connections. This yielding may take the form of plastic hinging in the beams (or, less desirably, in the columns), plastic shear deformation in the column panel zones, or through a combination of these mechanisms. It was believed that the typical connection employed in steel moment-frame construction, shown in Figure 1-1, was capable of developing large plastic rotations, on the order of 0.02 radians or larger, without significant strength degradation.

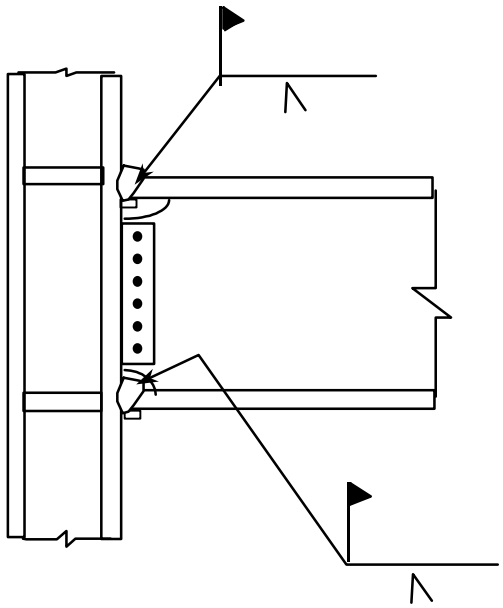


Figure 1-1 Typical Welded Moment-Resisting Connection Prior to 1994

Observation of damage sustained by buildings in the 1994 Northridge earthquake indicated that, contrary to the intended behavior, in many cases, brittle fractures initiated within the connections at very low levels of plastic demand, and in some cases, while the structures

remained essentially elastic. Typically, but not always, fractures initiated at the complete joint penetration (CJP) weld between the beam bottom flange and column flange (Figure 1-2). Once initiated, these fractures progressed along a number of different paths, depending on the individual joint conditions.

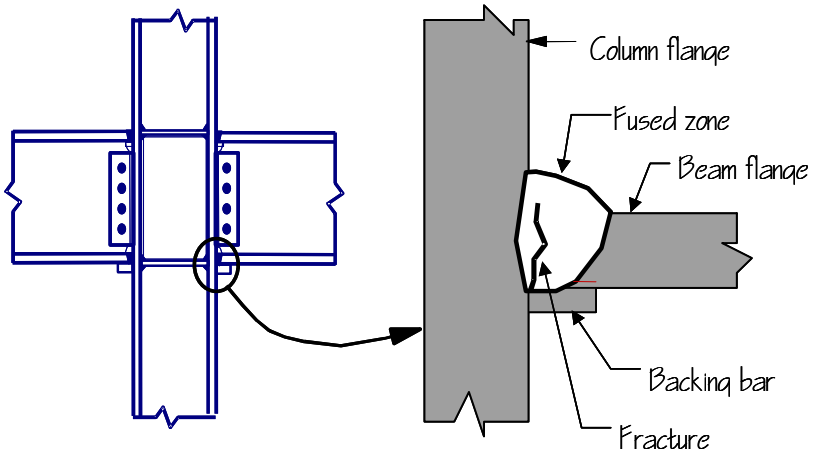
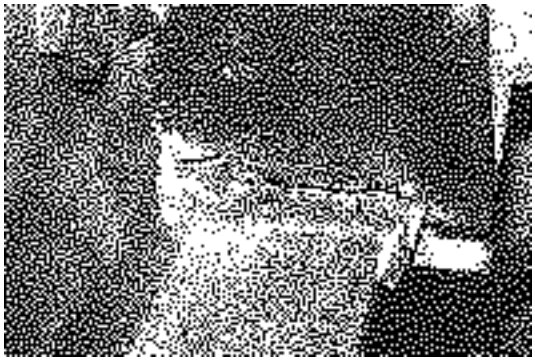


Figure 1-2 Common Zone of Fracture Initiation in Beam -Column Connection

In some cases, the fractures progressed completely through the thickness of the weld, and when fire protective finishes were removed, the fractures were evident as a crack through exposed faces of the weld, or the metal just behind the weld (Figure 1-3a). Other fracture patterns also developed. In some cases, the fracture developed into a crack of the column flange material behind the CJP weld (Figure 1-3b). In these cases, a portion of the column flange remained bonded to the beam flange, but pulled free from the remainder of the column. This fracture pattern has sometimes been termed a “divot” or “nugget” failure.

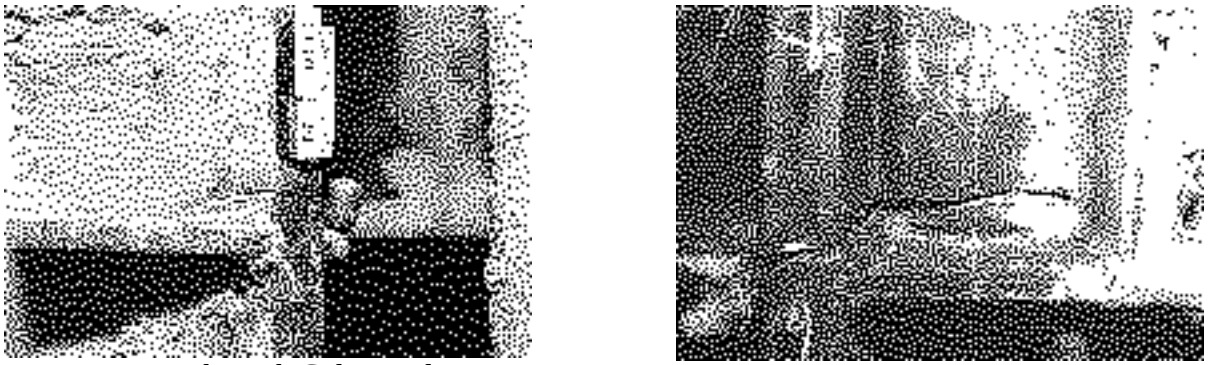
A number of fractures progressed completely through the column flange, along a near-horizontal plane that aligns approximately with the beam lower flange (Figure 1-4a). In some cases, these fractures extended into the column web and progressed across the panel zone (Figure 1-4b). Investigators have reported some instances where columns fractured entirely across the section.



a. Fracture at Fused Zone

b. Column Flange "Divot" Fracture

Figure 1-3 Fractures of Beam-to-Column Joints



a. Fractures through Column Flange

b. Fracture Progresses into Column Web

Figure 1-4 Column Fractures

Once such fractures have occurred, the beam-column connection has experienced a significant loss of flexural rigidity and strength to resist those loads that tend to open the crack. Residual flexural strength and rigidity must be developed through a couple consisting of forces transmitted through the remaining top flange connection and the web bolts. However, in providing this residual strength and stiffness, the bolted web connections can themselves be subject to failures. These include fracturing of the welds of the shear plate to the column, fracturing of supplemental welds to the beam web or fracturing through the weak section of shear plate aligning with the bolt holes (Figure 1-5).

Despite the obvious local strength impairment resulting from these fractures, many damaged buildings did not display overt signs of structural damage, such as permanent drifts or damage to architectural elements, making reliable postearthquake damage evaluations difficult. In order to determine if a building has sustained connection damage it is necessary to remove architectural finishes and fireproofing, and perform detailed inspections of the connections. Even if no damage is found, this is a costly process. Repair of damaged connections is even more costly. At least one steel moment-frame building sustained so much damage that it was deemed more practical to demolish the building than to repair it.

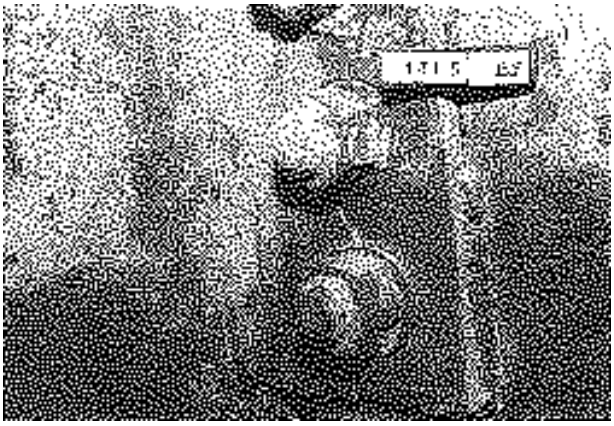


Figure 1-5 Vertical Fracture through Beam Shear Plate Connection

Initially, the steel construction industry took the lead in investigating the causes of this unanticipated damage and in developing design recommendations. The American Institute of Steel Construction (AISC) convened a special task committee in March, 1994 to collect and disseminate available information on the extent of the problem (AISC, 1994a). In addition, together with a private party engaged in the construction of a major steel building at the time of the earthquake, AISC participated in sponsoring a limited series of tests of alternative connection details at the University of Texas at Austin (AISC, 1994b). The American Welding Society (AWS) also convened a special task group to investigate the extent to which the damage was related to welding practice, and to determine if changes to the welding code were appropriate (AWS, 1995).

In September, 1994, the SAC Joint Venture, AISC, the American Iron and Steel Institute and National Institute of Standards and Technology jointly convened an international workshop (SAC, 1994) in Los Angeles to coordinate the efforts of the various participants and to lay the foundation for systematic investigation and resolution of the problem. Following this workshop, FEMA entered into a cooperative agreement with the SAC Joint Venture to perform problem-focused studies of the seismic performance of steel moment-frame buildings and to develop recommendations for professional practice (Phase I of SAC Steel Project). Specifically, these recommendations were intended to address the following: the inspection of earthquake-affected buildings to determine if they had sustained significant damage; the repair of damaged buildings; the upgrade of existing buildings to improve their probable future performance; and the design of new structures to provide reliable seismic performance.

During the first half of 1995, an intensive program of research was conducted to explore more definitively the pertinent issues. This research included literature surveys, data collection on affected structures, statistical evaluation of the collected data, analytical studies of damaged and undamaged buildings, and laboratory testing of a series of full-scale beam-column assemblies representing typical pre-Northridge design and construction practice as well as various repair, upgrade and alternative design details. The findings of these tasks formed the basis for the development of *FEMA-267 – Interim Guidelines: Evaluation, Repair, Modification, and Design of Welded Steel Moment Frame Structures*, which was published in August, 1995. *FEMA-267* provided the first definitive, albeit interim, recommendations for practice, following the discovery of connection damage in the 1994 Northridge earthquake.

In September 1995 the SAC Joint Venture entered into a contractual agreement with FEMA to conduct Phase II of the SAC Steel Project. Under Phase II, SAC continued its extensive problem-focused study of the performance of moment resisting steel frames and connections of various configurations, with the ultimate goal of develop seismic design criteria for steel construction. This work has included: extensive analyses of buildings; detailed finite element and fracture mechanics investigations of various connections to identify the effects of connection configuration, material strength, and toughness and weld joint quality on connection behavior; as well as more than 120 full-scale tests of connection assemblies. As a result of these studies, and independent research conducted by others, it is now known that the typical moment-resisting connection detail employed in steel moment-frame construction prior to the 1994 Northridge earthquake, and depicted in Figure 1-1, had a number of features that rendered it inherently susceptible to brittle fracture. These included the following:

- The most severe stresses in the connection assembly occur where the beam joins to the column. Unfortunately, this is also the weakest location in the assembly. At this location, bending moments and shear forces in the beam must be transferred to the column through the combined action of the welded joints between the beam flanges and column flanges and the shear tab. The combined section properties of these elements, for example the cross sectional area and section modulus, are typically less than those of the connected beam. As a result, stresses are locally intensified at this location.
- The joint between the bottom beam flange and the column flange is typically made as a downhand field weld, often by a welder sitting on top of the beam top flange, in a so-called "wildcat" position. To make the weld from this position each pass must be interrupted at the beam web, with either a start or stop of the weld at this location. This welding technique often results in poor quality welding at this critical location, with slag inclusions, lack of fusion and other defects. These defects can serve as crack initiators, when the connection is subjected to severe stress and strain demands.
- The basic configuration of the connection makes it difficult to detect hidden defects at the root of the welded beam-flange-to-column-flange joints. The backing bar, which was typically left in place following weld completion, restricts visual observation of the weld root. Therefore, the primary method of detecting defects in these joints is through the use of ultrasonic testing (UT). However, the geometry of the connection also makes it very difficult for UT to detect flaws reliably at the bottom beam flange weld root, particularly at the center of the joint, at the beam web. As a result, many of these welded joints have undetected significant defects that can serve as crack initiators.
- Although typical design models for this connection assume that nearly all beam flexural stresses are transmitted by the flanges and all beam shear forces by the web, in reality, due to boundary conditions imposed by column deformations, the beam flanges at the connection carry a significant amount of the beam shear. This results in significant flexural stresses on the beam flange at the face of the column, and also induces large secondary stresses in the welded joint. Some of the earliest investigations of these stress concentration effects in the welded joint were conducted by Richard, et al. (1995). The stress concentrations resulting from this effect resulted in severe strength demands at the root of the complete joint penetration welds between the beam flanges and column flanges, a region that often includes significant discontinuities and slag inclusions, which are ready crack initiators.
- In order that the welding of the beam flanges to the column flanges be continuous across the thickness of the beam web, this detail incorporates weld access holes in the beam web, at the beam flanges. Depending on their geometry, severe strain concentrations can occur in the beam flange at the toe of these weld access holes. These strain concentrations can result in low-cycle fatigue and the initiation of ductile tearing of the beam flanges after only a few cycles of moderate plastic deformation. Under large plastic flexural demands, these ductile tears can quickly become unstable and propagate across the beam flange.
- Steel material at the center of the beam-flange-to-column-flange joint is restrained from movement, particularly in connections of heavy sections with thick column flanges. This condition of restraint inhibits the development of yielding at this location, resulting in locally

high stresses on the welded joint, which exacerbates the tendency to initiate fractures at defects in the welded joints.

- Design practice in the period 1985-1994 encouraged design of these connections with relatively weak panel zones. In connections with excessively weak panel zones, inelastic behavior of the assembly is dominated by shear deformation of the panel zone. This panel zone shear deformation results in a local kinking of the column flanges adjacent to the beam-flange-to-column-flange joint, and further increases the stress and strain demands in this sensitive region.

In addition to the above, additional conditions contributed significantly to the vulnerability of connections constructed prior to 1994.

- In the mid-1960s, the construction industry moved to the use of the semi-automatic, self-shielded, flux-cored arc welding process (FCAW-S) for making the joints of these connections. The welding consumables that building erectors most commonly used inherently produced welds with very low toughness. The toughness of this material could be further compromised by excessive deposition rates, which unfortunately were commonly employed by welders. As a result, brittle fractures could initiate in welds with large defects, at stresses approximating the yield strength of the beam steel, precluding the development of ductile behavior.
- Early steel moment frames tended to be highly redundant and nearly every beam-column joint was constructed to behave as part of the lateral-force-resisting system. As a result, member sizes in these early frames were small and much of the early acceptance testing of this typical detail was conducted with specimens constructed of small framing members. As the cost of construction labor increased, the industry found that it was more economical to construct steel moment-frame buildings by moment-connecting a relatively small percentage of the beams and columns and by using larger members for these few moment-connected elements. The amount of strain demand placed on the connection elements of a steel moment frame is related to the span-to-depth ratio of the member. Therefore, as member sizes increased, strain demands on the welded connections also increased, making the connections more susceptible to brittle behavior.
- In the 1960s and 1970s, when much of the initial research on steel moment-frame construction was performed, beams were commonly fabricated using A36 material. In the 1980s, many steel mills adopted more modern production processes, including the use of scrap-based production. Steels produced by these more modern processes tended to include micro-alloying elements that increased the strength of the materials so that despite the common specification of A36 material for beams, many beams actually had yield strengths that approximated or exceeded that required for grade 50 material. As a result of this increase in base metal yield strength, the weld metal in the beam-flange-to-column-flange joints became under-matched, potentially contributing to its vulnerability.

At this time, it is clear that in order to obtain reliable ductile behavior of steel moment-frame construction a number of changes to past practices in design, materials, fabrication, erection and quality assurance are necessary. The recommended criteria contained in this document, and the companion publications, are based on an extensive program of research into materials, welding

technology, inspection methods, frame system behavior, and laboratory and analytical investigations of different connection details. The recommended criteria presented herein are believed to be capable of addressing the vulnerabilities identified above and providing for frames capable of more reliable performance in response to earthquake ground shaking.

1.4 Application

This publication supersedes the design recommendations for new construction contained in *FEMA-267, Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures*, and the *Interim Guidelines Advisories, FEMA-267A* and *FEMA-267B*. It is intended to be used as a basis for updating and revision of evaluation and rehabilitation guidelines and standards currently employed in steel moment-frame construction, in order to permit more reliable seismic performance in moment-resisting frame construction. This document has been prepared based on the provisions contained in *FEMA-302 1997 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (BSSC, 1997a), the 1997 *AISC Seismic Specification* (AISC, 1997), including supplements (AISC, 1999) and the 1998 *AWS D1.1 Structural Welding Code - Steel*, as it is anticipated that these documents form the basis for the current model building code, the 2000 edition of the *International Building Code*. Some users may wish to apply the recommendations contained herein to specific engineering projects, prior to the adoption of these recommendations by future codes and standards. Such users are cautioned to consider carefully any differences between the aforementioned documents and those actually enforced by the building department having jurisdiction for a specific project, and to adjust the recommendations contained in these guidelines accordingly. These users are also warned that these recommendations have not undergone a consensus adoption process. Users should thoroughly acquaint themselves with the technical data upon which these recommendations are based and exercise their own independent engineering judgment prior to implementing these recommendations.

1.5 Overview

The following is an overview of the general contents of chapters contained in these *Recommended Criteria*, and their intended use:

- **Chapter 2: General Requirements**. This chapter, together with Chapter 3, is intended to indicate recommended supplements to the building code requirements for design of steel moment-frame buildings. These chapters include discussion of referenced codes and standards; design performance objectives; selection of structural systems; configuration of structural systems; and analysis of structural frames to obtain response parameters (forces and deflections) used in the code design procedures. Also included is discussion of an alternative, performance-based design approach that can be used at the engineer's option, to design for superior or more reliable performance than is attained using the code based-approach. Procedures for implementation of the performance-based approach are contained in Chapter 4.
- **Chapter 3: Connection Qualification**. Steel moment frames can incorporate a number of different types of beam-column connections. Based on research conducted as part of this project, a number of connection details have been determined to be capable of providing

acceptable performance for use with different structural systems. These connections are termed prequalified. This chapter provides information on the limits of this prequalification for various types of connections and specific design and detailing recommendations for these prequalified connections. In some cases it may be appropriate to use connection details and designs which are different than the prequalified connections contained in this chapter, or to use one of the prequalified connection details outside the range of its prequalification. This chapter provides recommended criteria for project-specific qualification of a connection detail in such cases, as well as recommended procedures for new prequalifications for connections for general application. Reference to several proprietary connection types that may be utilized under license agreement with individual patent holders is also provided. When proprietary connections are used in a design, qualification data for such connections should be obtained directly from the licensor.

- **Chapter 4: Performance Evaluation.** This chapter presents a simplified analytical performance evaluation methodology that may be used, at an engineer's option, to determining the probable structural performance of regular, welded steel moment-frame structures, given the site seismicity. These procedures allow the calculation of a level of confidence that a structure will have less than a desired probability of exceeding either of two performance levels, an Immediate Occupancy level or a Collapse Prevention level. If the calculated level of confidence is lower than desired, a design can be modified and re-evaluated for more acceptable performance, using these same procedures.
- **Appendix A: Detailed Procedures for Performance Evaluation.** This appendix provides criteria for implementation of the detailed analytical performance evaluation procedures upon which the simplified procedures of Chapter 4 are based. Implementation of these procedures can permit more certain evaluation of the performance of a building to be determined than is possible using the simplified methods of Chapter 4. Engineers may find the application of these more detailed procedures beneficial in demonstrating that building performance is better than indicated by Chapter 4. Use of these procedures is required when a performance evaluation is to be performed for a building employing connections that have not been prequalified, or for a building that is irregular, as defined in *FEMA-273*.
- **References, Bibliography, and Acronyms.**

2. GENERAL REQUIREMENTS

2.1 Scope

These *Recommended Criteria* apply to the seismic design of Special Moment Frames and Ordinary Moment Frames designed using the *R*, C_d, and W_0 values given in Table 5.2.2, pages 45-50, of *FEMA-302*. They do not apply to structures designed in accordance with the applicable Provisions of *FEMA-302* for "Structural Steel Systems Not Specifically Detailed for Seismic Resistance". These *Recommended Criteria* replace and supercede all design guidelines contained in *FEMA-267*, *FEMA-267A*, and *FEMA-267B*.

This chapter presents overall criteria for the seismic design of steel moment frames for new buildings and structures. Included herein are general criteria on applicable references including codes, provisions and standards, recommended performance objectives, system selection, system analysis, frame design, connection design, specifications, quality control and quality assurance.

2.2 Applicable Codes, Standards, and References

Steel moment-frame systems should, as a minimum, be designed in accordance with the applicable provisions of the prevailing building code as supplemented by these *Recommended Criteria*. These *Recommended Criteria* are specifically written to be compatible with the requirements of *FEMA-302 – NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*. Where these *Recommended Criteria* are different from those of the prevailing code, it is intended that these *Recommended Criteria* should take precedence. The following are the major codes, standards and references referred to herein:

FEMA-302	*NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, 1997 Edition, Part 1 – Provisions (BSSC, 1997a)*
FEMA-303	*NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, 1997 Edition, Part 2 – Commentary (BSSC, 1997b)*
AWS D1.1	*Structural Welding Code, 1998 Edition (AWS, 1998)*
AISC Seismic	*Seismic Provisions for Structural Steel Buildings, April 15, 1997, (AISC, 1997) including Supplement No. 1, February 15, 1999 (AISC, 1999)*
AISC-LRFD	*Load and Resistance Factor Design Specifications for Structural Steel Buildings (AISC, 1993)*
AISC-Manual	*LRFD Manual of Steel Construction, Second Edition, 1998 (AISC, 1998b)*
FEMA-353	*Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications* (SAC, 2000d)
FEMA-273	*NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (ATC, 1997a)

Commentary: The 1997 AISC Seismic Provisions (AISC, 1997) provide design requirements for steel moment-frame structures. FEMA-302 adopts the AISC Seismic Provisions by reference as the design provisions for seismic-force-

resisting systems of structural steel. The International Building Code is based generally on the FEMA-302 Provisions, and incorporates design requirements for steel structures primarily based on the AISC Provisions. These Recommended Criteria are written to be compatible with the 1997 AISC Seismic Provisions and FEMA-302 Provisions and reference is made to sections of those documents where appropriate herein.

2.3 Basic Design Approach

The recommended design approach consists of the following basic steps:

Step 1: Select a structural system type and frame configuration in accordance with Section 2.5 of these *Recommended Criteria*.

Step 2: Select preliminary frame member sizes and perform a structural analysis for earthquake loading and frame adequacy using the applicable R, C_d and W_0 values, strength criteria, drift limits, and redundancy requirements of *FEMA-302*, as supplemented by Section 2.9 of these *Recommended Criteria*.

Step 3: Select an appropriate connection type, in accordance with Section 2.5.3 of these *Recommended Criteria*. Connections may be prequalified, project qualified, or proprietary, as indicated in Chapter 3 of these *Recommended Criteria*.

Step 4: Perform an analysis in accordance with Sections 2.7 and 2.8 of these *Recommended Criteria*, considering the effects (if any) of the selected connection type on frame stiffness and behavior, to confirm the adequacy of member sizing to meet the applicable strength, drift, and stability limitations.

Step 5: Confirm or revise the member sizing based on the connection type selected and following Sections 2.9 and 3.2 of these *Recommended Criteria*. Return to Step 4, if necessary.

Step 6: Complete the design of the connections, in accordance with Chapter 3 of these *Recommended Criteria*.

As an option, when it is desired to design for specific performance, rather than simply achieving code compliance, a Performance Evaluation following the guidelines of Chapter 4 may be performed.

Commentary: This section outlines the basic steps recommended for design intended to meet the minimum criteria of the building code. Since the 1994 Northridge earthquake, the 1997 AISC Seismic Provisions have required that laboratory test data be submitted to demonstrate that connection detailing will be capable of adequate service. With the publication of these Recommended Criteria, and the establishment of a series of prequalified connection details, it is intended that substantiation of connection detailing by reference to laboratory test data will not be required for most design applications. However, design

procedures for some types of prequalified connections entail significant calculation.

The optional Performance Evaluation procedures contained in Chapter 4 and Appendix A of these Recommended Criteria need not be applied to designs intended only to meet the requirements of the building code. Regular, well-configured Special Moment Frame and Ordinary Moment Frame structures designed and constructed in accordance with FEMA-302, and building code requirements as supplemented by these Recommended Criteria, are expected to provide a high level of confidence of being able to resist collapse under Maximum Considered Earthquake demands. Section 2.4 of these Recommended Criteria and FEMA-303 provide additional information on this performance goal. Structures with significant irregularity, low levels of redundancy, or poor configuration may not be capable of such performance. The Performance Evaluation procedures of Chapter 4 and Appendix A may be used to confirm the capability of such structures to meet the performance intended by the building code, or may be used to implement performance-based designs intended to meet higher performance objectives.

2.4 Design Performance Objectives

Under *FEMA-302*, each building and structure must be assigned to one of three Seismic Use Groups (SUGs). Buildings are assigned to the SUGs based on their intended occupancy and use. Most commercial, residential and industrial structures are assigned to SUG I. Buildings occupied by large numbers of persons or by persons with limited mobility, or that house large quantities of potentially hazardous materials are assigned to SUG II. Buildings that are essential to postearthquake disaster response and recovery operations are assigned to SUG III. Buildings in each of SUG II and III are intended to provide better performance, as a group, than buildings in SUG I. As indicated in *FEMA-303*, buildings designed in accordance with the provisions for each SUG are intended, as a minimum, to be capable of providing the performance indicated in Figure 2-1.

The *FEMA-302* provision attempts to obtain these various performance characteristics through regulation of system selection, detailing requirements, design force levels, and permissible drift. This regulation is based on the SUG, the seismicity of the region containing the building site, and the effect of site-specific geologic conditions. All structures should, as a minimum, be assigned to an appropriate SUG, in accordance with the building code, and be designed in accordance with the applicable requirements for that SUG.

Although the *FEMA-303 Commentary* to *FEMA-302* implies that buildings designed in accordance with the requirements for the various SUGs should be capable of providing the performance capabilities indicated in Figure 2-1, *FEMA-302* does not contain direct methods to evaluate and verify the actual performance capability of structures, nor does it provide a direct means to design for performance characteristics other than those implied in Figure 2-1, should it be desired to do so. It is believed, based on observation of the performance of modern, code-

conforming construction in recent earthquakes, that *FEMA-302* provides reasonable reliability with regard to attaining Life Safe performance for SUG-I structures subjected to design events, as indicated in Figure 2-1. However, the reliability of *FEMA-302* with regard to the attainment of other performance objectives for SUG-I structures, or for reliably attaining any of the performance objectives for the other SUGs seems less certain and has never been quantified or verified.

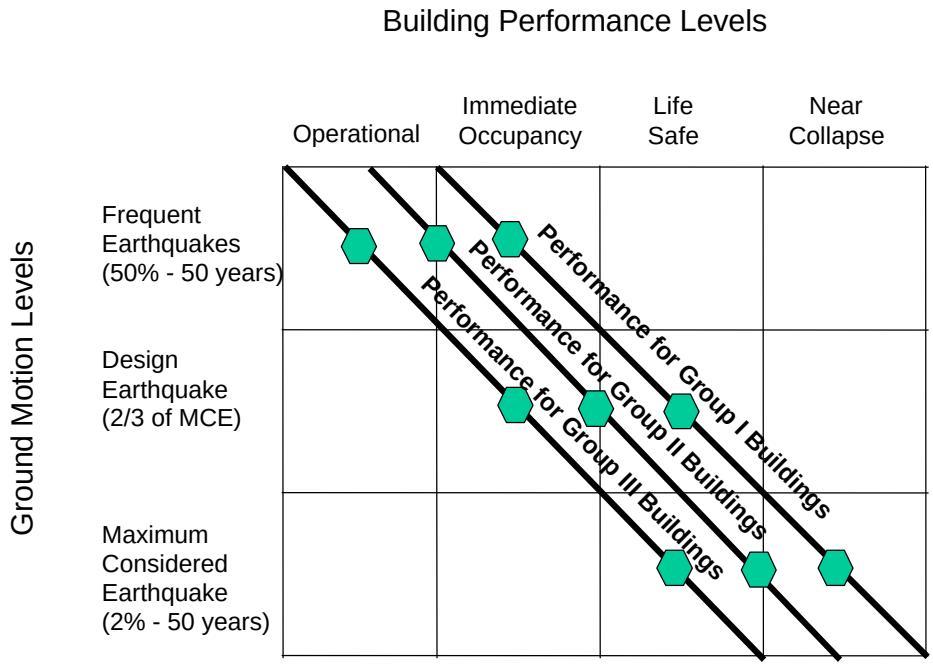


Figure 2-1 NEHRP Seismic Use Groups (SUG) and Performance

Chapters 2 and 3 of these *Recommended Criteria* present code-based design recommendations for steel moment-frame buildings. All buildings should, as a minimum, be designed in accordance with these recommendations. For buildings in which it is desired to attain other performance than implied by the code, or for which it is desired to have greater confidence that the building will actually be capable of attaining the desired performance, the procedures in Chapter 4 and Appendix A may be applied.

Commentary: FEMA-302 includes three types of steel moment frames , two of which are incorporated in these Recommended Criteria. The three types are: Special Moment Frames (SMF), Intermediate Moment Frames (IMF), and Ordinary Moment Frames (OMF). Building code provisions for SMF systems strictly regulate building configuration, proportioning of members and connection detailing in order to produce structures with superior inelastic response capability. Provisions for OMF systems have less control on these design features and therefore, as a class, OMF structures are expected to have poorer inelastic response capability than SMF systems. Following the 1994 Northridge earthquake, the building code was amended to include substantial

additional requirements for SMF system design and construction, resulting in an increase in the development cost for such structures. In 1997, the IMF system was added to FEMA-302 and the AISC Seismic Provisions to provide an economical alternative to SMF construction for regions of moderate seismicity. Studies conducted under this project have indicated that the inelastic response demands on IMF systems are similar to those for SMF systems and that, therefore, the reduction in design criteria associated with the IMF system was not justified. Consequently, only Special Moment Frame and Ordinary Moment Frame systems are included herein. These systems are described in more detail in Section 2.5. In FEMA-302, a unique R value and C_d factor are assigned to each of these systems, as are height limitations and other restrictions on use. Regardless of the system selected, FEMA-302 implies that structures designed to meet the requirements therein will be capable of meeting the Collapse Prevention performance level for a Maximum Considered Earthquake (MCE) ground motion level and will provide Life Safe performance for the Design Basis Earthquake (DBE) ground motion that has a severity 2/3 that of MCE ground motion. This 2/3 factor is based on an assumption that the Life Safety performance on which earlier editions of the NEHRP Recommended Provisions were based inherently provided a minimum margin of 1.5 against collapse. Except for sites located within a few kilometers of known active faults, the MCE ground motion is represented by ground shaking response spectra that have a 2% probability of exceedance in 50 years (approximately 2500-year mean return period). For sites that are close to known active faults, the MCE ground motion is taken either as this 2%/50-year spectrum, or as a spectrum that is 150% of that determined from a median estimate of the ground motion resulting from a characteristic event on the nearby active fault, whichever is less.

The FEMA-302 Provisions define classes of structures for which performance superior to that described above is mandated. Additionally, individual building owners may desire a higher level of performance. The FEMA-302 Provisions attempt to improve performance for SUG-II and SUG-III structures, (1) through use of an occupancy importance factor that increases design force levels, and therefore reduces the amount of ductility a structure must exhibit to withstand strong ground shaking, and (2) through specification of more restrictive drift limits than those applied to SUG-I structures. This combination of increased design forces and more restrictive drift limitations leads to substantially greater strength in systems such as SMFs, the design of which is governed by drift.

The FEMA-302 R factors, drift limits, and height limitations, as well as the inelastic rotation capability requirements corresponding to the R value for each moment-frame type (SMF, IMF, or OMF), are based more on historical precedent and judgment than they are on analytical demonstration of adequacy. In the research program on which these Criteria are based, an extensive series of nonlinear analytical investigations has been conducted to determine the drift

demands on structures designed in accordance with the current code when subjected to different ground motions, and for a variety of assumed hysteretic behaviors for connections. The results of these investigations have led to the conclusion that some of the FEMA-302 Provisions and 1997 AISC Seismic Provisions were not capable of reliably providing the intended performance. These Recommended Criteria directly modify those Provisions so as to increase the expected reliability of performance to an acceptable level. On the basis of these analytical studies, it is believed that regular, well-configured structures designed in accordance with these Recommended Criteria and constructed in accordance with FEMA-353, provide in excess of 90% confidence of being able to withstand Maximum Considered Earthquake demands without global collapse and provide mean confidence of being able to withstand such ground motion without local structural failure.

It should be recognized that application of the modifications suggested in these Recommended Criteria, while considered necessary to provide this level of confidence with regard to achieving the indicated performance for moment-resisting frames, may result in such systems having superior performance capabilities relative to some other systems, the design provisions for which do not have a comparable analytical basis. In other words, the design provisions contained in FEMA-302 for some other structural systems, both of steel and of other construction materials, may inherently provide a lower level of assurance that the resulting structures will be able to provide the intended performance.

The three classes of steel moment-frame systems contained in FEMA-302 are themselves not capable of providing uniform performance. OMFs will typically be stronger than either IMFs or SMFs, but can have much poorer inelastic response characteristics. The result of this is that OMFs should be able to resist the onset of damage at somewhat stronger levels of ground shaking than is the case for either IMFs or SMFs. However, as ground motion intensity increases beyond the damage threshold for each of these structural types, it would be anticipated that OMFs would present a much greater risk of collapse than would IMFs, which in turn, would present a more significant risk of collapse then SMFs. For these reasons, FEMA-302 places limitations on the applicability of these various structural systems depending on a structure's height and the seismic hazard at the site.

Refer to Chapter 4 for more detailed discussion of recommended performance objectives and their implications.

2.5 System Selection

2.5.1 Configuration and Load Path

Every structure should be provided with a complete lateral and vertical seismic-force-resisting system, capable of transmitting inertial forces from the locations of mass throughout the structure to the foundations. For steel moment-frame structures, the load path includes the floor and roof diaphragms, the moment-resisting frames, the foundations, and the various collector elements that interconnect these system components.

To the extent possible, the structural system should have a regular configuration without significant discontinuities in stiffness or strength and with the rigidity of the structural system distributed uniformly around the center of mass.

Commentary: The importance of maintaining regularity in structural systems can not be overemphasized. The analytical investigations of structural performance conducted as part of this project were limited to regular structural systems. Irregularities in structural systems can result in concentration of deformation demands on localized portions of a structure, and early development of P-D instabilities. FEMA-302 includes significant limitations on structural irregularity, particularly for structures in Seismic Design Categories D, E and F. However, it was not possible, within the scope of this project, to determine if these limitations are sufficient to ensure that the intended performance capability is achieved and this should be the subject of future investigations.

Structures categorized as regular under FEMA-302 may not actually behave in a regular manner. FEMA-302 categorizes a multistory buildings as being regular if the vertical distribution of lateral stiffness and strength is uniform. Thus, a structure with equal lateral stiffness and strength in every story would be categorized as regular. However, such structures would not actually behave as regular structures when responding to strong ground motion. Instead such structures would develop large concentrations of inelastic behavior and deformation at the lower stories of the structure. To provide true strength and stiffness regularity in multistory structures, it is necessary to maintain uniform ratios of (1), lateral strength to tributary mass, and (2), lateral stiffness to tributary mass, for each story of the structure, where tributary mass may be considered as that portion of the structure's mass supported at and above the story.

2.5.2 Structural System Selection

The moment frame may be designed either as an SMF or OMF. The selection of moment-frame type should be governed by the prevailing code and by the project conditions. Consideration should be given to using Special Moment Frames whenever conditions permit.

Commentary: FEMA-302 defines three types of steel moment frames: Special Moment Frames (SMF), Intermediate Moment Frames (IMF), and Ordinary Moment Frames (OMF). Detailing and configuration requirements are specified for each of these three systems to provide for different levels of ductility and global inelastic response capability, varying from highest in SMFs to lowest in OMFs. IMF systems have intentionally been omitted from these Recommended Criteria because nonlinear analyses of buildings designed to the criteria for IMF systems contained in FEMA-302 have indicated that the inelastic demands for these structures are nearly as large as those for SMF structures. Therefore, it is not possible to justify on technical grounds the use of the relaxed detailing criteria provided for IMFs in FEMA-302 unless more restrictive design force levels and drift criteria are also specified in order to limit the amount of inelastic demand these structures may experience. Rather than developing such criteria, it was decided to omit this system, which had only recently been introduced into the building codes, from further consideration.

Ordinary Moment Frames are relatively strong (compared to SMFs) but have much less ductility. As a result, Ordinary Moment Frame structures, as a class, would be anticipated to have less damage than SMFs for moderate levels of ground shaking and significantly more damage than SMFs for severe levels of ground shaking. In recognition of this, FEMA-302 places limitations on the height, occupancy and ground motion severity for which Ordinary Moment Frame systems can be used. In recognition of the superior performance characteristics of SMF systems when subjected to high-intensity ground shaking, it is recommended that designers consider their use, even when IMF or OMF systems are permitted under the building code.

2.5.3 Connection Type

Moment-resisting connections in SMFs and OMFs, except connections in OMFs designed to remain elastic under design level earthquake ground shaking, should be demonstrated by test and by analysis to be capable of providing the minimum levels of interstory drift angle capacity specified in Section 3.9 of these *Recommended Criteria*. Interstory drift angle is that portion of the interstory drift ratio in a frame resulting from flexural deformation of the frame elements, as opposed to axial deformation of the columns, as indicated in Figure 2-2. Sections 3.5, 3.6 and 3.7 present details and design procedures for a series of connections that are recommended as prequalified to meet the criteria of Section 3.9 without further analysis or testing, when used within the indicated limits applicable to each connection type.

Commentary: FEMA-302 and the 1997 AISC Seismic Provisions set minimum strength criteria for connections. In addition, except for connections in OMFs that are designed to remain elastic, the 1997 AISC Seismic Provisions require that connections be demonstrated capable of providing minimum levels of rotational capacity. The 1997 AISC Seismic Provisions uses plastic rotation angle as the performance parameter by which connections are qualified. In these

Recommended Criteria, interstory drift angle is used instead. This is because this parameter, (1) seems to be stable with regard to prediction of frame performance, (2) is closely related to plastic rotation angle, (3) is less ambiguous with regard to definition, and (4) is a quantity that is easily determined from the results of standard frame analyses using either linear or nonlinear methods.

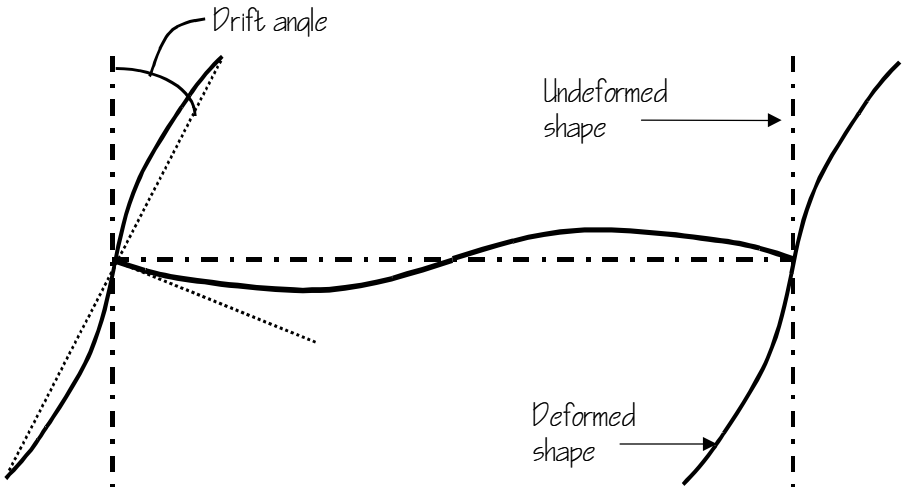


Figure 2-2 Interstory Drift Angle

Figure 2-2 illustrates the interstory drift angle, for a frame with fully restrained (FR) connections and rigid panel zones. Prior to lateral deformation, the beam and column are joined at right angles to each other. Under elastic deformation, the column and beam will remain joined at right angles and the beam will rotate in double curvature between the two columns. The interstory drift angle is measured as the angle between the undeformed vertical axis of the column and the deformed axis of the column at the center of the beam-column joint. For the idealized FR frame with rigid panel zones, shown in the figure, this same angle will exist between the undeformed horizontal axis of the beam and the deformed axis of the beam, at the beam-column connection. In FEMA-273, this angle is termed the chord angle and is used as the parameter for determining beam-column connection performance. However, for frames with panel zones that are not rigid, frames with partially restrained connections, or frames that exhibit plasticity at the connection, the chord angle of the beam will not be identical to the interstory drift angle. For such frames, the interstory drift angle, reduced for the effects of axial column elongation, is a better measure of the total imposed rotation on all elements of the connection, including panel zones and connection elements, and is used as the basis of these Recommended Criteria.

2.5.4 Redundancy

Structures assigned to Seismic Design Categories D, E, and F of *FEMA-302* shall be provided with sufficient bays of moment-resisting framing to satisfy the redundancy

requirements of those *Provisions*. In addition, the strength of members of the seismic-force-resisting system shall be evaluated for adequacy to resist horizontal earthquake forces that are factored by the redundancy factor r in accordance with the load combinations of *FEMA-302*.

Commentary: There are several reasons why structures with some redundancy in their structural systems should perform better than structures without such redundancy. The basic philosophy underlying the design provisions of FEMA-302 is to permit substantial inelastic behavior in frames under ground shaking of the severity of the design earthquake or more severe events. Under such conditions, occasional failures of elements may occur. Structures that have nonredundant seismic-force-resisting systems could potentially develop instability in the event of failure of one or more elements of the system. Redundant structures, on the other hand, would still retain some significant amount of lateral resistance following failure of a few elements.

Another significant advantage of providing redundant framing systems is that the use of a larger number of frames to resist lateral forces often permits the size of the framing elements to be reduced. Laboratory research has shown that connection ductile capacity generally increases as the size of the framing elements decreases.

FEMA-302 includes a redundancy factor r with values between 1.0 and 1.5, which is applied as a load factor on calculated earthquake forces for structures categorized as Seismic Design Category D, E, or F. Less redundant systems (frames with fewer participating beams and columns) are assigned higher values of the redundancy factor and therefore must be designed to resist higher design forces to compensate for their lack of redundancy. Minimum permissible levels of redundancy are set, through lower-bound values specified for the redundancy factor, for structures located in regions of high seismic risk.

The maximum permitted r values given in FEMA-302 were based only on the judgment of the writers of that document. They should not be construed as ideal or optimum values. Designers are encouraged to incorporate as much redundancy as is practical into steel moment-frame buildings.

2.5.5 Frame Beam Spans

The connection prequalification data provided for each prequalified connection in Chapter 3 includes specification of the minimum beam-span-to-depth ratio for which the connection is prequalified. Span-to-depth ratios for beams in moment frames should equal or exceed the minimum span-to-depth ratio applicable to the connection type being used, unless project-specific qualification testing is performed as described in Section 3.9, or other rational analysis is employed to demonstrate that hinge rotations or bending strains will not exceed those for which the connection is prequalified.

Where the effective span for a frame beam (distance between points of plastic hinging of the beam) is such that shear yielding of the beam will occur before flexural yielding, the web of the beam shall be detailed and braced as required by the 1997 *AISC Seismic Provisions* for long links in eccentric braced frames.

Commentary: In determining the layout of moment frames, it should be recognized that excessively short spans can affect both frame and connection behavior. Possible effects include the following:

1. *For connection types that move the hinge significantly away from the column face, the plastic rotation demand at the hinge will be significantly larger than the frame interstory drift angle, due to geometric effects.*

2. *The steeper moment gradient resulting from the shorter spans will decrease the length of the beam hinge, requiring that the beam develop greater bending strains to accommodate the same interstory drift angle.*

3. *If the effective span length becomes too short, shear yielding of the beam, rather than flexural yielding, will control inelastic behavior.*

Most testing of prequalified connections performed under this project used configurations with beam spans of about 25 feet. Most tested beams were either W30 or W36, so that span-to-depth ratios were typically in the range of 8 to 10. Refer to FEMA-355D, State of the Art Report on Connection Performance for more information on the effects of short spans.

2.6 Structural Materials

2.6.1 Material Specifications

Structural steel should conform to the specifications and grades permitted by the 1997 *AISC Seismic Provisions,* as modified by *FEMA-353,* and as indicated in the specific connection prequalifications, unless a project-specific qualification testing program is performed to demonstrate acceptable performance of alternative materials.

Commentary: Under the 1997 AISC Seismic Provisions, rolled shapes used in OMF or SMF applications may conform to the ASTM A36, A572 or A913 specifications. In the 1980s, it was common practice in some regions to design moment frames with columns conforming to the ASTM A572 Grade 50 specification and with beams conforming to the ASTM A36 specification, in order to obtain frames economically with strong columns and weak beams. During the 1990s, however, the steel production industry in the United States has undergone a significant evolution, with many of the older mills being replaced by newer mills that use scrap-based production processes. These newer mills routinely produce higher strength steel than did the older mills. Since the A36 and A572 specifications do not place an upper bound on material strength, much of the steel

shipped by these mills, particularly for material ordered as conforming to the A36 specification, is much stronger than the minimum strength controlled by the specification, and use of the combination of A36 and A572 materials to provide for strong-column-weak-beam conditions will not reliably achieve this goal. In 1997, ASTM introduced a new A992 specification to address this problem. The A992 specification is similar to the ASTM A572, Grade 50 specification, except that maximum as well as minimum yield strengths are specified to provide for more controlled design conditions. In addition, the A992 specification includes increased control on trace elements and can be more weldable than some A572 steels. It is recommended that either A992 or A913 steel be used in SMF applications.

2.6.2 Material Strength Properties

The strength of materials shall be taken as indicated in the *AISC Seismic Provisions* and as modified by these *Recommended Criteria*. Where these *Recommended Criteria* require the use of "expected strength," this shall be the quantity $R_y F_y$ as indicated in the *AISC Seismic Provisions*. The value of R_y for material conforming to *ASTM A992* shall be the same as for material conforming to *ASTM A572 Grade 50*. Where these *Recommended Criteria* require the use of lower-bound strength, or specified strength, the minimum specified value of the yield strength F_y as indicated in the applicable ASTM specification shall be used.

Commentary: The AISC Seismic Provisions specify values of R_y for various materials as indicated in Table 2-1. The quantity $R_y F_y$ is intended to approximate the mean value of the yield strength of material produced to a given specification and grade. The AISC Seismic Provisions permit other values of R_y to be used, if the value of the expected mean yield strength F_{ye} is determined by appropriate testing.

Table 2-1 Values of R_y for Various Material Grades

Material Specification	R_y
ASTM A36	1.5
ASTM A572 Gr. 42	1.3
Other Specifications	1.1

As part of the program of investigations conducted in support of the development of these Recommended Criteria, studies of the statistical variation in strength properties of rolled sections of Grade 50 steel were conducted. These studies indicate that the 1.1 value for R_y is a good representation of the mean value of yield strength when applied to the webs of cross sections. The flexural properties of structural steel, however, are more closely related to the yield strength of the flanges of rolled shapes, which tend to have somewhat lower strength than do the webs. When applied to calculations of the flexural strength

of beams, the use of an R_y value of 1.1 actually approximates a mean-plus-one-standard-deviation value. Since values of expected strength are used to estimate the amount of force that can be delivered to adjacent connected elements, the use of this conservative value is appropriate. More information on the statistical variation of steel strength may be found in FEMA-355A, State of the Art Report on Base Metals and Fracture.

2.7 Structural Analysis

An analysis should be performed for each structure to determine the distribution of forces and deformations under code-specified ground motion and loading criteria. The analysis should conform, as a minimum, to the code-specified criteria for the equivalent lateral force method or the modal response spectrum method, as applicable.

Chapter 4 provides guidance on analysis methods that can be used as part of the Performance Evaluation approach for steel moment-frame structures.

Commentary: Seismic design forces for low-rise and mid-rise buildings without major irregularities have traditionally been determined by using the simple "equivalent lateral force" method prescribed by the codes. Such methods are incorporated in FEMA-302 and are permitted to be used for structures designated as regular, and up to 240 feet in height. Buildings that are over 5 stories or 65 feet in height and have certain vertical irregularities, and all buildings over 240 feet in height, require use of dynamic (modal or response history) analysis. The use of inelastic response history or nonlinear static analysis is also permitted by some codes though few guidelines are provided in codes on how to perform or apply such an analysis. Projects incorporating nonlinear response-history analysis should be conducted in accordance with the Performance Evaluation provisions of Chapter 4. For such applications, structures should be demonstrated as capable, with 90% confidence, of providing Collapse Prevention performance for MCE hazards based on considerations of global behavior and column adequacy. A 50% confidence level should be demonstrated for connection behavior.

2.8 Mathematical Modeling

2.8.1 Basic Assumptions

In general, a steel moment-frame building should be modeled, analyzed and designed as a three-dimensional assembly of elements and components. Although two-dimensional models may provide adequate design information for regular, symmetric structures and structures with flexible diaphragms, three-dimensional mathematical models should be used for analysis and design of buildings with plan irregularity as defined by *FEMA-302*. The two-dimensional modeling, analysis, and design of buildings with stiff or rigid diaphragms is acceptable, if torsional effects are either sufficiently small to be ignored, or are captured indirectly.

Vertical lines of framing in buildings with flexible diaphragms may be individually modeled, analyzed and designed as two-dimensional assemblies of components and elements, or a three-dimensional model may be used, with the diaphragms modeled as flexible elements.

Explicit modeling of connections is required only for nonlinear procedures and only if (1) the connection is weaker than the connected components, or (2) the flexibility of the connection results in a significant increase in the relative deformation between connected components. Additional guidance in using these methods is found in Chapter 4.

Commentary: A finite-element model will provide information on forces and deformations only at places in the structure where a modeling element is inserted. When nonlinear deformations are expected in a structure, the designer must anticipate the location of the plastic hinges and insert nonlinear finite elements at these locations if the inelastic behavior is to be captured by the model. Additional information is found in Chapter 4.

2.8.2 Model Configuration

The analytical model should accurately account for the stiffness of frame elements and connections and other structural and nonstructural elements that may affect this stiffness. This section presents basic recommendations for analyses intended to meet the requirements of *FEMA-302*. More detailed modeling guidelines for the purposes of performance evaluation are presented in Chapter 4. Chapter 3 presents specific modeling guidelines for various prequalified connections, referred to by the guidelines of Section 2.8, and Chapter 4.

2.8.2.1 Regularity

Classification of a building as irregular, and analysis limitations based on regularity are discussed in *FEMA-302*. Such classification should be based on the plan and vertical configuration of the framing system, using a mathematical model that considers relevant structural members.

2.8.2.2 Elements Modeled

For the purpose of determining the adequacy of the structure to meet the strength and drift requirements of *FEMA-302*, only participating elements of the seismic-force-resisting system shall be included in the analytical model. When nonstructural or nonparticipating elements of the seismic-force-resisting system have significant influence on the stiffness or distribution of seismic forces within the elements of the seismic-force-resisting system, separate analyses should be performed to evaluate the effect of these elements on (1) the distribution of deformations and member forces, and (2) overall building performance.

Commentary: In order to comply with the requirements of FEMA-302, it is necessary that the seismic-force-resisting system be capable of resisting the design seismic forces without participation of other elements. However, steel moment-frame structures are inherently flexible. Rigid supported elements

including architectural wall systems, ramped floors, and large mechanical equipment items can affect both the stiffness of the structure and the distribution of forces within the structure. The best practice in the design and detailing of steel moment-frame structures is to detail elements that are not part of the seismic-force-resisting system such that they are isolated from participating in the resistance of earthquake-induced frame drifts. For those cases when such isolation is not possible, the effect of these elements on the behavior of the frame should be considered in the design.

FEMA-302 does not permit consideration of elements that are not part of the primary lateral-force-resisting system as effective in meeting the strength and stiffness requirements of the provisions. However, in many steel moment-frame structures, framing provided only to resist gravity loads can provide substantial additional stiffness and strength. It is recommended that the effect of these nonparticipating structural elements be considered when performing analyses in support of performance evaluations, conducted in accordance with Chapter 4 of these Recommended Criteria.

2.8.2.3 Connection Stiffness

For frames with fully restrained connections, it shall be permissible to model the frame using centerline-to-centerline dimensions for the purpose of calculating stiffnesses of beams and columns. Alternatively, when justified by appropriate analytical or test data, more realistic assumptions that account for the stiffness of panel zones and connections may be used. In either case, calculation of beam moments and shears should be performed at the face of the column.

For linear analysis of structures with partially restrained connections, beams should be modeled with an equivalent *EI*, using the method shown in Chapter 5 of *FEMA-273*. Chapter 3 of these *Recommended Criteria* provides guidelines for estimating connection stiffness parameters for use in this procedure for the various prequalified partially restrained connections. For nonlinear analysis of frames with partially restrained connections, the nonlinear force-deformation characteristics of the connections should be directly modeled.

Commentary: In analytical studies of moment-resisting frame behavior (FEMA-355C) conducted in support of the development of these Recommended Criteria, it has been demonstrated that panel-zone deformations have little effect on analytical estimates of drift and need not be explicitly modeled, provided the panel zones are not excessively weak. Inelastic analyses of frames designed in accordance with these Recommended Criteria indicate that explicit modeling of panel zone shear strength and flexibility results in similar, albeit slightly smaller estimates of interstory drift than is obtained from models in which panel zones are not modeled and center-line-to-center-line framing dimensions are used. Therefore, this document recommends use of the simpler approach, in which panel zones are neglected in the model and center-line-to-center-line framing dimensions are used. It is permissible to use realistic assumptions for the

stiffness of panel zones, to modify the effective flexural span length of beams and columns, provided that such assumptions are based on appropriate data. Some connections, such as large haunches or slide plates, may significantly increase frame stiffness, meriting the inclusion of their effects in the analytical model. Additional discussion on modeling considerations, including methods to model connections and panel zones explicitly may be found in FEMA-355C, State of the Art Report on Systems Performance.

2.8.3 Horizontal Torsion

The effects of horizontal torsion must be considered, as in *FEMA-302*. The total torsional moment at a given floor level includes the following two torsional moments:

a. Actual torsion: the moment resulting from the eccentricity between (1) the centers of mass at all floors above and including the given floor, and (2) the center of rigidity of the vertical seismic elements in the story below the given floor, and

b. Accidental torsion: an accidental torsional moment produced by an artificial horizontal offset in the centers of mass, at all floors above and including the given floor, equal to a minimum of 5% of the horizontal dimension at the given floor level measured perpendicular to the direction of the applied load.

When the effects of torsion are investigated, the increased forces and displacements from horizontal torsion should be evaluated and considered for design. The effects of torsion cannot be used to reduce force and deformation demands on components and elements.

Commentary: Actual torsion that is not apparent in an evaluation of the center of rigidity and center of mass in an elastic stiffness evaluation can develop during nonlinear response of the structure if yielding develops in an unsymmetrical manner. For example, if the frames on the east and west sides of a structure have similar elastic stiffness the structure may not have significant torsion during elastic response. However, if the frames on the east side of the structure yield significantly sooner than the framing on the west side, then inelastic torsion will develop. Although the development of such inelastic torsion can be a serious problem, FEMA-302 does not address these phenomena. Designers can reduce the potential for severe inelastic torsion by providing framing layouts that have both stiffness and strength as symmetrical as possible about the center of mass.

2.8.4 Foundation Modeling

Foundations should generally be modeled as unyielding. Soil-structure interaction may be modeled as permitted by the building code. Assumptions for the extent of fixity against rotation provided at the base of columns should realistically account for the relative rigidities of the frame and foundation system, including soil compliance effects, and the detailing of the column base connections.

Commentary: Most steel moment frames can be adequately modeled by assuming that the foundation provides rigid support for vertical loads. However, the flexibility of foundation systems (and the attachment of columns to those systems) can significantly alter the flexural stiffness at the base of the frame. Where relevant, these factors should be considered in developing the analytical model.

2.8.5 Diaphragms

Floor and roof diaphragms transfer earthquake-induced inertial forces to vertical elements of the seismic framing system. Connections between diaphragms and vertical seismic framing elements must have sufficient strength to transfer the maximum calculated diaphragm shear forces to the vertical framing elements. Requirements for design and detailing of diaphragm components are given in *FEMA-302*.

Diaphragms should be classified as flexible, stiff, or rigid in accordance with *FEMA-302*. For buildings with steel moment-frame systems, most floor slabs with concrete fill over metal deck may be considered to be rigid diaphragms. Floors or roofs with plywood diaphragms should be considered flexible. The flexibility of unfilled metal deck, and concrete slab diaphragms with large openings should be considered in the analytical model.

Mathematical models of buildings with diaphragms that are not rigid should be developed considering the effects of diaphragm flexibility. For buildings with flexible diaphragms at each floor level, the vertical lines of seismic framing may be designed independently, with seismic masses assigned on the basis of tributary area.

2.8.6 *P-D* Effects

The structure shall be investigated to ensure that lateral drifts induced by earthquake response do not result in a condition of instability under gravity loads. At each story, the quantity Y_i should be calculated for each direction of response, as follows:

$$Y_i = \frac{P_i R d_i}{V_{yi} h_i} \qquad (2\text{–}1)$$

where:

P_i = portion of the total weight of the structure including dead, permanent live, and 25% of transient live loads acting on all of the columns within story level *i*, kips,

R = response modification coefficient obtained applicable to the structural system and used to determine the design seismic forces

δ_i = calculated lateral drift at the center of rigidity of story *i*, when the design seismic forces are applied in the direction under consideration, inches,

V_{yi} = total plastic lateral shear force in the direction under consideration at story *i*,

h_i = height of story *i*, which may be taken as (1) the distance between the centerline of floor framing at each of the levels above and below, (2) the distance between the top of floor slabs at each of the levels above and below, or (3) the distance between similar common points of reference.

Commentary: The quantity Y_i is the ratio of the effective story shear produced by first order P-D effects at the calculated story drift to the maximum restoring force in the structure. When this ratio has a value greater than 1.0, the structure does not have enough strength to resist the P-D induced shear forces and unless restrained, will collapse in a sidesway mechanism. If the ratio is less than 1, the restoring force in the structure exceeds the story shear due to P-D effects and unless additional displacement is induced or lateral forces applied, the structure should not collapse.

The plastic story shear quantity, V_{yi}, should be determined by methods of plastic analysis. In a story in which all beam-column connections meet the strong-column-weak-beam criterion, the same number of moment-resisting bays is present at the top and bottom of the frame and the strength of moment-connnected girders at the top and bottom of the frame is similar, V_{yi} may be approximately calculated from the equation:

$$V_{yi} = \frac{2\sum_{j=1}^{n} M_{pG_j}}{h_i} \tag{2-2}$$

where:

M_{pGj} = *the plastic moment capacity of each girder "j" participating in the moment-resisting framing at the floor level on top of the story, and*

n = *the number of moment-resisting girders in the framing at the floor level on top of the story.*

In any story in which all columns do not meet the strong-column-weak-beam criterion, the plastic story shear quantity, V_{yi} may be calculated from the equation:

$$V_{yi} = \frac{2\sum_{k=1}^{m} M_{pC_k}}{h_i} \tag{2-3}$$

where:

m = *the number of columns in moment-resisting framing in the story under consideration, and*

M_{pCk} = *the plastic moment capacity of each column "k", participating in the moment-resisting framing, considering the axial load present on the column.*

For other conditions, the quantity V_{yi} must be calculated by plastic mechanism analysis, considering the vertical distribution of lateral forces on the structure.

In any story in which Y_i is less than or equal to 0.1, the structure need not be investigated further for stability concerns. When the quantity Y_i in a story exceeds 0.1, the analysis of the structure should explicitly consider the geometric nonlinearity introduced by *P-D* effects. Most linear dynamic analysis software packages have the ability to consider *P-D* effects automatically. For nonlinear analysis procedures, second-order effects should be considered directly in the analysis; the geometric stiffness of all elements and components subjected to axial forces should be included in the mathematical model. When Y_i in a story exceeds 0.3, the structure shall be considered unstable, unless a detailed global stability capacity evaluation for the structure, considering *P-D* effects, is conducted in accordance with the guidelines of Appendix A.

Commentary: P-D effects can have very significant impact on the ability of structures to resist collapse when subjected to strong ground shaking. When the non-dimensional quantity, Y, calculated in accordance with Equation 2-3 significantly exceeds a value of about 0.1, the instantaneous stiffness of the structure can be significantly decreased, and can effectively become negative. If earthquake induced displacements are sufficiently large to create negative instantaneous stiffness, collapse is likely to occur.

Analyses reported in FEMA-355F, State of the Art Report on Performance Prediction and Evaluation, included direct consideration of P-D effects in determining the ability of regular, well configured frames designed to modern code provisions to resist P-D-induced instability and P-D-induced collapse. For regular, well configured structures, it is believed that if the value of Y is maintained within the limits indicated in this section, P-D-induced instability is unlikely to occur. Values of Y greater than this limit suggest that instability due to P-D effects is possible. In such cases, the frame should be reconfigured to provide greater resistance to P-D-induced instability unless explicit evaluation of these effects using the detailed Performance Evaluation methods outlined in Appendix A are performed.

The evaluation approach for P-D effects presented in this section appears similar to but differs substantially from that contained in FEMA-302, and in use in the building codes for many years. The approach contained in FEMA-302 and the building codes was an interim formulation. The research conducted in support of these Recommended Criteria indicates that this interim approach was not

meaningful. Some of the research performed in support of these Recommended Criteria included the explicit evaluation of P-D effects for buildings of varying heights, subjected to many different types of ground motion, and designed using different building code provisions . Using these and other parameters, several tens of thousands of nonlinear analyses were run to investigate P-D effects. A complete discussion of the analyses supporting these recommendations may be found in FEMA-355F. Extensive additional discussion on the issue of P-D effects and their importance in the response of structures at large interstory drifts is contained in FEMA-355C, State of the Art Report on Systems Performance.

2.8.7 Multidirectional Excitation Effects

Buildings should be designed for seismic forces incident from any horizontal direction. For regular buildings, seismic displacements and forces may be assumed to act nonconcurrently in the direction of each principal axis of the building. For buildings with plan irregularity and buildings in which one or more components form part of two or more intersecting frames, multidirectional excitation effects should be considered. Multidirectional effects on components should include both torsional and translational effects.

The requirement that multidirectional (orthogonal) excitation effects be considered may be satisfied by designing frames for the forces and deformations associated with 100% of the seismic displacements in one horizontal direction plus the forces associated with 30% of the seismic displacements in the perpendicular horizontal direction. Alternatively, it is acceptable to use the square root of the sum of the squares (SRSS) to combine multidirectional effects where appropriate.

2.8.8 Vertical Excitation

The effects of vertical excitation on horizontal cantilevers and prestressed elements should be considered by static or dynamic response methods. Vertical earthquake shaking may be characterized by a spectrum with ordinates equal to 67% of those of the horizontal spectrum unless alternative vertical response spectra are developed using site-specific analysis. Vertical earthquake effects on other beams and column elements should be evaluated for adequacy to resist vertical earthquake forces, as specified in *FEMA-302*.

Commentary: There is no evidence that response to vertical components of ground shaking has had any significant effect on the performance of steel moment-frame structures. Consequently, the effect of this response is not recommended for consideration in the performance evaluation of these buildings, except as required by the building code.

Traditionally, vertical response spectra, when considered, have been taken as 2/3 of the horizontal spectra developed for the site. While this is a reasonable approximation for most sites, vertical response spectra at near-field sites, located within a few kilometers of the zone of fault rupture can have substantially

stronger vertical response spectra than indicated by this rule. Development of site-specific response spectra is recommended when vertical response must be considered for buildings on such sites.

2.9 Frame Design

The following provisions supplement the parallel provisions contained in the 1997 *AISC Seismic Provisions.*

2.9.1 Strength of Beams and Columns

Multi-story frames should be designed with a strong-column-weak-beam configuration, to avoid the formation of single-story mechanisms. As a minimum, Equation 9-3 of the 1997 *AISC Seismic Provisions* should be satisfied. In the application of Equation 9-3, the quantity M_c as defined in Section 3.2.6 of these *Recommended Criteria* should be substituted for the quantity M^*_{pb}.

Commentary: When subjected to strong ground shaking, multi-story structures with columns that are weaker in flexure than the attached beams can form single-story mechanisms, in which plastic hinges form at the base and top of all columns in a story. Once such a mechanism forms in a structure, nearly all of the earthquake-induced lateral displacement will occur within the yielded story, which can lead to very large local drifts and the onset of P-D instability and collapse.

Although weak-column-strong-beam designs are not desirable, the 1997 AISC Seismic Provisions does permit their use under certain conditions, even for Special Moment Frames. Before utilizing weak-column-strong-beam configurations, designers should be aware that the prequalified connections for Special Moment Frames contained in these Recommended Criteria are based on tests using strong columns. When considering moment frames which include columns deployed in the weak direction, designers should be aware that only one connection type (RBS, Section 3.5.5) has been tested for use with weak-direction columns for application in Special Moment Frames and, although those tests were successful, insufficient data exists to prequalify such connections.

Nonlinear analyses of representative frames have clearly shown that the use of the provisions described above will not completely prevent plastic hinging of columns. This is because the point of inflection in the column may move away from the assumed location at the column mid-height once inelastic beam hinging occurs, and because of global bending induced by the deflected shape of the building, of which the column is a part.

Except for the case when a column hinge mechanism forms, column hinging is not a significant problem, provided that the columns are designed as compact sections, are properly braced and axial loads are not high. It is well understood

that a column hinge will form at the base of columns which are continuous into a basement, or which are rigidly attached to a stiff and strong foundation.

2.9.2 Lateral Bracing of Column Flanges

Lateral bracing of column flanges, at beam-column connections should be provided whenever the following equation is not satisfied:

$$\frac{\sum M_{pc}^{*}}{\sum M_{c}} \geq 2.0 \qquad (2\text{-}4)$$

where:

M_{pc}^{*} is the quantity defined in Section 9.6 of the 1997 *AISC Seismic Provisions*

M_c is calculated as indicated in Section 3.2.6 of these *Recommended Criteria*.

Commentary: The relationship indicated in Equation 2-4 has been included in proposals for the 2000 NEHRP Recommended Provisions for New Buildings (now under consideration by the Building Seismic Safety Council) as a trigger for requirements for lateral bracing of column flanges. Large axial loads reduce the ductility of column hinges. Consideration should be given to applying larger factors for columns with axial loads exceeding 50% of the critical column load.

Bracing of the column at the location of the beam top flanges is normally supplied by the interconnection of the concrete slab, where such a slab occurs. At the location of the beam bottom flanges, sufficient lateral bracing can sometimes be shown to be provided by perpendicular beams and connected stiffeners for shallow column sections with wide flanges. Deeper beam-type sections, when used as columns, are typically less stable and normally will require direct lateral bracing of the flanges. See Section 2.9.6 for further guidelines on use of deep sections as columns.

2.9.3 Panel Zone Strength

Panel zones should conform to the strength requirements of Section 3.3.3.2 of these *Recommended Criteria* and the requirements of the individual prequalified connection design procedures.

Commentary: Connection performance can be affected either positively or negatively by panel zone strength. Some shear yielding of the panel zone can relieve the amount of plastic deformation that must be accommodated in other regions of the frame and many connections have been found to provide the largest inelastic deformation capacity when yielding is balanced between the panel zone and other connection elements. However, excessive panel zone deformation can induce large secondary stresses into the connection that can degrade connection

performance and increase fracture toughness demand on welded joints, and can also cause deformations which are undesirable for column performance. For this reason, the individual connection prequalifications include limitations on panel zone strength relative to beam strength.

2.9.4 Section Compactness Requirements

Beams should conform to the section compactness requirements of *AISC Seismic Provisions*. Columns should also be compact, unless it can be shown by nonlinear analysis that the columns will not yield in response to the design earthquake.

Commentary: The 1997 AISC Seismic Provisions provide section compactness requirements for beams used in moment frames, and for columns which may be subjected to hinging. The effect of beam flange b/t as it relates to connection performance is discussed in Section 3.3.1.1 Beam Flange Stability. The effect of beam section compactness on overall frame performance is directly related to how local buckling affects strength degradation of individual beams and columns in the frame. Flange local buckling and lateral torsional buckling are sources of strength degradation.

It should be noted that for Reduced Beam Section (RBS) connections, the b/t in the area of beam hinging is reduced by the flange reduction, thereby reducing the propensity for flange local buckling. This may justify use of sections which are otherwise non-compact in frames employing these connections. See Section 3.3.1.1 for recommendations.

2.9.5 Beam Lateral Bracing

The 1997 *AISC Seismic Provisions* require bracing of flanges of beams for Special Moment Frame systems. The unbraced length between supports is not permitted to exceed the quantity $2500\ r_y/F_y$. In addition, lateral supports are required where analysis indicates that a plastic hinge will form during inelastic deformations of the Special Moment Frame. General bracing of Special Moment Frame beams should conform to the AISC requirements. For bracing of beams at plastic hinges, refer to Section 3.3.1.5.

2.9.6 Deep Columns

The prequalified connections included in Chapter 3 of these *Recommended Criteria* are not prequalified for use with deep (beam-type) sections used as columns. The prequalified connections should only be used with W12 and W14 column sections.

Commentary: Nearly all of the beam-column connection assemblies tested as part of this project, as well as by other researchers, utilized W14 column sections. In recognition of the fact that some designers prefer to use W24 or other deep section columns in order to increase frame stiffness economically, two tests of reduced beam section assemblies with W24 columns were conducted. These

assemblies performed poorly and one column failed through development of a fracture between the column web and flange. This fracture resulted from the combined effects of local torsional instability of the column and the presence of low-toughness material at the flange-to-web region, sometimes referred to as the k-area. The problem of low toughness material at the k-area of rolled structural shapes is a well documented phenomena related to the straightening practice used by some mills for certain ranges of shape. Additional information on this phenomena may be found in FEMA-355A, State of the Art Report on Base Materials and Fracture. However, there is relatively little data available on the instability of deep section columns in moment-resisting connections and this project was not able to develop adequate data on this effect to allow prequalification of connections with deep columns.

2.9.7 Built-Up Sections

The prequalified connections included in Chapter 3 of these *Recommended Criteria* have not been tested with built-up sections used as beams or columns. The prequalified connections should only be used with such sections when it can be shown that the built-up section conforms with all of the requirements for rolled sections as specified for the connection to be used. Of particular concern should be the strength of the web-to flange connection of the built-up section.

2.10 Connection Design

Chapter 3 of these *Recommended Criteria* provides criteria for the design and detailing of several types of prequalified Fully Restrained (FR) and Partially Restrained (PR) connections. These prequalified connections are recommended as acceptable for use in steel moment-frame systems, within the limitations expressed in Chapter 3, without further qualification analyses or tests. Table 2-2 lists the prequalified connection details, and the systems for which they are prequalified. All of these prequalifications apply only to frames composed of wide flange beams connected to the major axis of wide flange columns.

In addition to the connections indicated in Table 2-2, Chapter 3 also provides information on several types of proprietary connections. Proprietary connections have not been prequalified by this project for service in specific systems. Engineers interested in the applicability of proprietary connections should obtain qualification information from the licensor.

For each connection in Table 2-2, a complete set of design criteria is presented in Chapter 3. Depending on the selected system type, the designer should select a suitable connection, then follow the design criteria to complete the design. Connections contained in Chapter 3 may be used in applications outside the indicated range of prequalification provided that a project-specific qualification program is followed, as indicated in Section 3.9.

Connection types not prequalified under the guidelines of Chapter 3 may also be used, subject to the project-specific qualification procedures.

Table 2-2 Prequalified Connection Details

Category	Connection Description	Acronym	Permissible Systems
Welded, fully restrained	Welded Unreinforced Flanges, Bolted Web	WUF-B	OMF
	Welded Unreinforced Flanges, Welded Web	WUF-W	OMF, SMF
	Free Flange	FF	OMF, SMF
	Welded Flange Plate	WFP	OMF, SMF
	Reduced Beam Section	RBS	OMF, SMF
Bolted, fully restrained	Bolted, Unstiffened End Plate	BUEP	OMF, SMF
	Bolted, Stiffened End Plate	BSEP	OMF, SMF
	Bolted Flange Plates	BFP	OMF, SMF
Bolted, partially restrained	Double Split Tee	DST	OMF, SMF

Commentary: For each of the prequalified connection types indicated in Table 2-2, sufficient laboratory testing, together with related analytical work, has been performed to provide an ability to predict with confidence the limiting modes of behavior for the connection when properly constructed and the probability that the connection will be able to sustain certain levels of inelastic deformation. This confidence only applies to application within certain limits, including material specifications, and member sizes. If a design falls outside the range of prequalification for a connection detail, it is necessary to extend the existing qualification for use in the specific application, by performing additional laboratory prototype testing. Chapter 3 indicates the extent of the additional testing recommended to extend connection qualification, on a project-specific basis, as well as more general recommendations for prequalifying connection details for broader application.

2.11 Specifications

FEMA-353 – Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications presents supplemental recommendations for fabrication and erection of steel moment-frame structures. These supplemental recommendations address welding and base materials, methods of fabrication and quality assurance. It is recommended that project specifications include the specific paragraphs of *FEMA-353* that are applicable to the design being used.

Commentary: FEMA-353 is written in the form of supplemental provisions to the existing provisions of the building codes, FEMA-302, and standard AISC, AWS and ASTM specifications. It is expected that eventually, these standard specifications and provisions will be amended to adopt the supplemental provisions recommended by FEMA-353. In the interim, the applicable sections and paragraphs can be reproduced in individual project specifications. When this

is done, it is recommended that the specific language taken from the reference be used without modification and attributed to the source, so that fabricators and erectors can readily recognize and become accustomed to the use of the FEMA-353 requirements.

2.12 Quality Control and Quality Assurance

FEMA-353 – Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications provides complete guidelines and commentary for Quality Control and Quality Assurance. The designer should utilize those guidelines to ensure the proper selection and handling of materials and shop and field fabrication of moment-frame connections.

Commentary: FEMA-353 has a complete discussion of quality control recommendations and the reasons for them. Quality control and quality assurance are important for the achievement of the intended performance.

2.13 Other Structural Connections

2.13.1 Column Splices

Column splices in moment frames should be designed to develop the full bending and shear strength of the column, unless an inelastic analysis is performed to determine the largest axial loads, moments and shears likely to occur at the location of the splice and the splice detail can be shown to be adequate to resist these axial loads, moments and shears, considering stress concentrations inherent in the types of joints being used.

Welded flange splices may be made either with full penetration groove welds, or with splice plates fillet welded to the column flanges. Weld metal with a minimum rated toughness as described in Section 3.3.2.4 should be used and weld tabs should be removed. Bolted column flange splices should be designed to preclude net section fracture, block shear failure, and bolt pull-through failure of the column flange or of the splice plates.

Column web splices may be either bolted or welded, or welded to one column piece and bolted to the other. Bolted splices using plates or channels on both sides of the column web are preferred because of the inherent extra safety afforded by "capturing" the web. Partial Joint penetration welded web splices are not recommended. Column web splices should be designed to resist the maximum shear force that the column is capable of producing.

Splices of columns that are not a part of the seismic-force-resisting system should be made in the center one-third of the column height, and should have sufficient shear capacity in both orthogonal directions to maintain the alignment of the column at the maximum shear force that the column is capable of producing.

Commentary: Section 8.3 of the 1997 AISC Seismic Provisions specifies requirements for design of column splices for columns that are part of the

seismic-force-resisting system. The requirements prohibit splices made with fillet welds or partial penetration groove welds located within four feet or within one-half the column clear height of the beam-to-column connections. This prohibition is because fillet welds in tensile applications and partial penetration butt welds are both details with relatively low tensile capacity and poor inelastic capability. For typical cases, the prohibition against such splices within four feet of a beam-column joint will control. The one-half column height requirement is intended to apply to those rare cases when the clear column height is less than eight feet. The 1997 AISC Seismic Provisions permit such splices in the mid-height zone of columns based on the belief that large flexural demands, and in particular inelastic demands are unlikely to occur in this region. Inelastic analyses of frames, however, clearly demonstrate that this presumption is incorrect for frames subjected to seismic loadings that exceed their elastic capacity. For this reason, as well as the severe potential consequences of column splice failure, the 1997 AISC Seismic Provisions are not considered to be sufficiently conservative in this area.

Because bending and axial stresses at column splice welds may be high, it is recommended that weld filler metals with rated notch toughness be used for these splices and that runoff tabs be removed. Where CJP welds are used, removal of backing is not judged to be necessary because the configuration of backing for column-to-column flange welds is not conducive to crack formation, as it is for the right-angle condition of beam-to-column flange joints. Properly designed bolted flange splices may be shown to be adequate for some column splice applications.

Bolted web connections are preferred by many engineers and contractors because they have advantages for erection, and, when plates are placed on both sides of the web, they are expected to maintain alignment of the column in the event of a flange splice fracture. Partial joint penetration welded webs are not recommended, because fracture of a flange splice would likely lead to fracture of the web splice, considering the stress concentrations inherent in such welded joints.

Inelastic analyses have shown the importance of the columns that are not part of the seismic-force-resisting system in helping to distribute the seismic shears between the floors. Even columns that have beam connections that act as pinned connections may develop large bending moments and shears due to non-uniform drifts of adjacent levels. For this reason, it is considered to be important that splices of such columns be adequate to develop the shear forces corresponding to development of plastic hinges at the ends of the columns in both orthogonal directions.

2.13.2 Column Bases

Column bases can be of several different types, as follows:

1. The column may continue into a basement, crawl space, or grade beam, in such a way that the column's fixity is assured without the need for a rigid base plate connection.
2. Large columns may be provided at the bottom level to limit the drift, and a "pinned base" may be utilized.
3. A connection which provides partial fixity may be provided, so that the column base is fixed up to some column moment, but the base itself yields before the column hinges.
4. A heavy base plate assembly may be provided which is strong enough to force yielding in the column.

In all of these cases, the designer should consider the base connection as similar to a beam-to-column connection and apply similar principles of design and detailing.

Notes:

1. For the first case above, the designer should recognize that hinging will occur in the column, just above the first floor. The horizontal shear to be resisted at the ends of the column in the basement level should be calculated considering the probable overstrength of the framing.
2. For the "pinned base", the designer should ensure that the required shear capacity of the base can be maintained up to the maximum rotation that may occur.
3. In designing a base with partial fixity , the designer should consider the principles used in the design of partially-restrained connections. This type of base may rely on bending of the base plate (similar to an end plate connection), bending of angles or tees, or yielding of anchor bolts. In the latter case, it is necessary to provide bolts or rods with adequate elongation capacity to permit the required rotation and sufficient unrestrained length for the yielding to occur. Shear capacity of the base plate to foundation connection must be assured at the maximum rotation.
4. For the fully fixed base, the designer should employ the same guidelines as given for the rigid fully-restrained connections. Such connections may employ thick base plates, haunches, cover plates, or other strengthening as required to develop the column hinge. Where haunched type connections are used, it must be recognized that the hinging will occur above the haunch, and appropriate consideration should be given to the stability of the column section at the hinge.

Commentary: It is well recognized that achievement of a mechanism in a moment frame requires a hinge at, or near to, the base of the column. The column base detail must accommodate the required hinging rotations while maintaining the strength required to provide the mechanism envisioned by the designer. These conditions are similar to the requirements for beam-to-column connections, as described.

2.13.3 Welded Collectors and Chords

Connections of highly loaded collectors and chords are often made with welded or bolted flange details comparable to those employed in moment frames. Design of such connections should incorporate the principles applied to moment-frame connections, unless it can be shown that the connection will remain elastic under the combination of the axial load, calculated at the limit strength of the system, and the corresponding rotation due to building drift.

Commentary: The rotational demand on rigid connections made for other purposes are often comparable to those of moment-frame beams. When coupled with high axial loads, demands on welded or bolted joints can be high. The principles of design for moment-frame beam-to-column connections are applicable to such conditions.

2.13.4 Simple Beam-to-Column Gravity Connections

Simple welded shear tab connections of beams to columns in buildings employing moment frames and other relatively flexible lateral-force-resisting systems should utilize details that have been demonstrated to have sufficient rotational capacity to accommodate the rotations that occur at the anticipated drifts, while maintaining capacity for the required gravity forces. In the absence of a more detailed analysis, adequate rotation capacity can be considered to be that associated with the design story drift calculated using the methods of *FEMA-302* multiplied by 1.5. As described in the commentary below, calculations to justify the adequacy of this condition should not be necessary under normal conditions.

When deep beams with deep bolt groups are connected to small columns, the columns should be compact, or sufficient rotational capacity should be provided in the connections to preclude hinging of the column when subjected to the drift calculated as described above.

Commentary: Research conducted under this project has shown that the plastic rotational capacity of simple bolted shear tab type connections, designed using the methods of the AISC LRFD Specification, and with adequate clearance of beam flanges from the column flanges to prevent bearing, is dependent on the depth of the bolt group, d_{bg}, and can reasonably be calculated as:

$$q_p = 0.15 - 0.0036 d_{bg} \tag{2-5}$$

where d_{bg} is the vertical dimension of the bolt group in inches. The additional elastic rotational capacity of these connections is estimated as about 0.02 radians. This gives a total estimated drift capacity for such connections of:

$$q_p = 0.17 - 0.0036 d_{bg} \tag{2-6}$$

The use of Equation 2-6 above will result in a calculated rotational capacity of more than 0.09 radian for an 8-bolt group with bolts spaced at 3", which will

be more than adequate for most conditions. Where the calculated rotational angle is not sufficient, slotted holes in the shear tab, or other means of accommodating larger rotations should be used. It should be noted that rotation capacities for connections made with clip angles bolted to the beam have not been found to be significantly higher than those for welded shear tabs. Refer to FEMA-355D for additional information.

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.

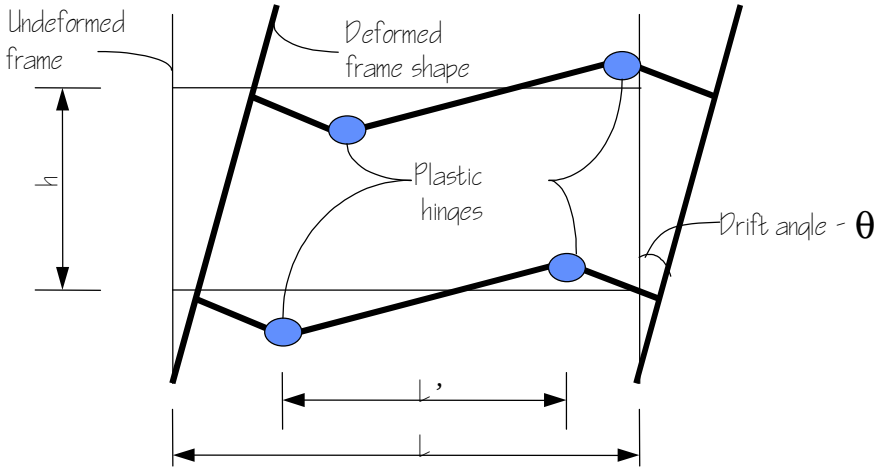


Figure 3-1 Inelastic Behavior of Frames with Hinges in Beam Span

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.

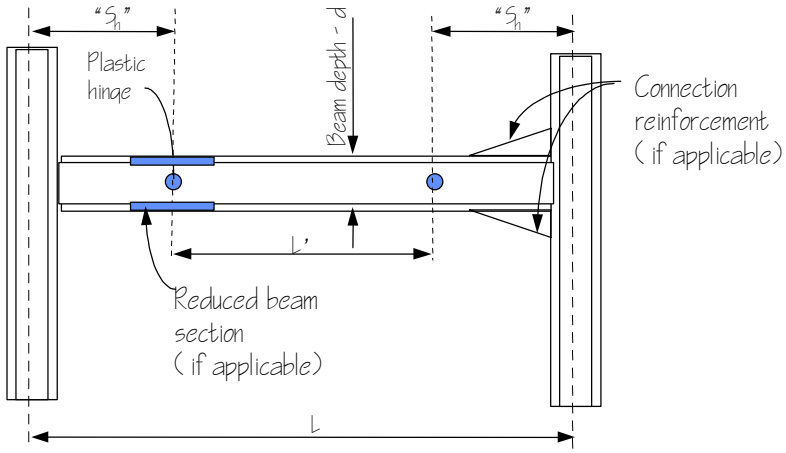


Figure 3-2 Location of Plastic Hinge Formation

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.

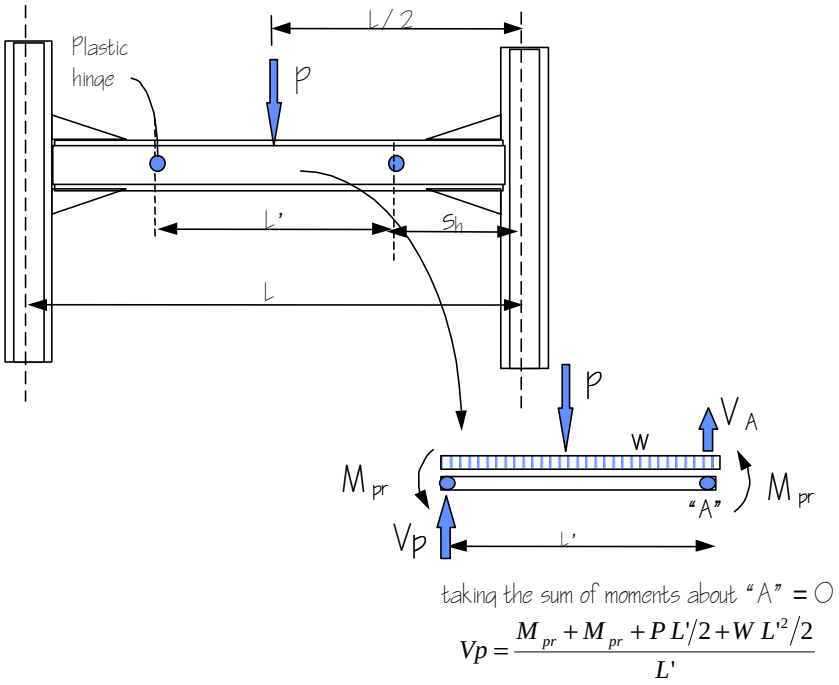


Figure 3-3 Sample Calculation of Shear at Plastic Hinge

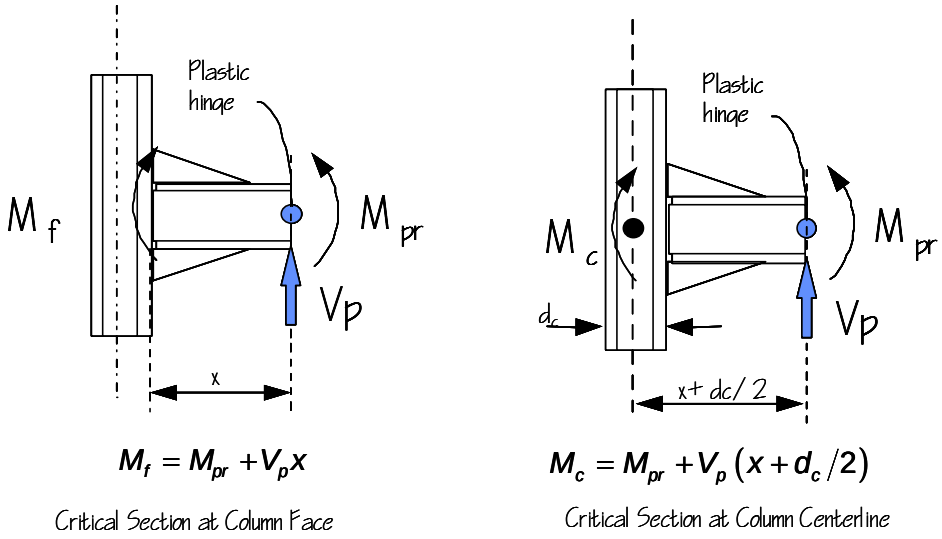


Figure 3-4 Calculation of Demands at Critical Sections

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:

$$M_{yf} = C_y M_f \tag{3-3}$$

where:

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

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 $52/\sqrt{F_y}$, 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 $52/\sqrt{F_y}$. 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 $54/\sqrt{F_y}$ 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 $520/\sqrt{F_y}$, 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 $418/\sqrt{F_y}$ 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^{o} 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^{o}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.*

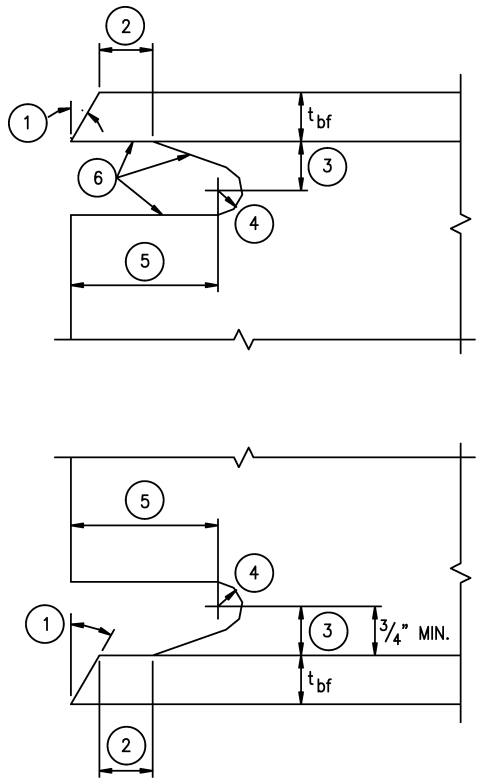
Commentary: The size, shape and workmanship of weld access holes can affect the connection in several different ways. If the hole is not large enough, this restricts welder access to the joint and increases the probability of low quality joints. Depending on the size and shape of the weld access hole, 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

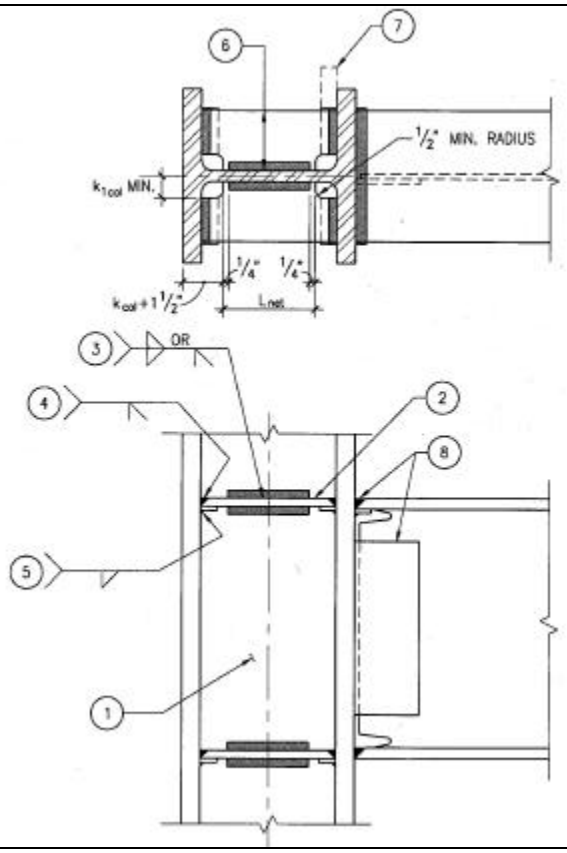
F_{yb} (F_{yc}) = Minimum specified yield stress of the beam (column) flange, ksi

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:

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

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

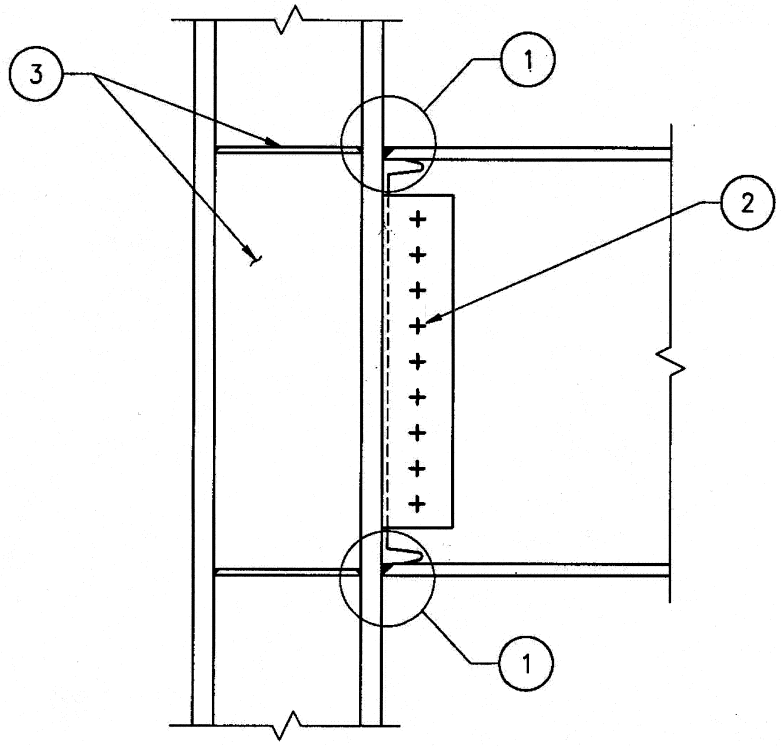
Commentary: FEMA-355D, State of the Art Report on Connection Performance, provides extensive information on the testing and performance of these connections, as well as others, that is not repeated in this document. The data presented in FEMA-355D have been prepared in support of the development of prequalification 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 :

$$V_f = \frac{2M_f}{L - d_c} + V_g \tag{3-8}$$

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

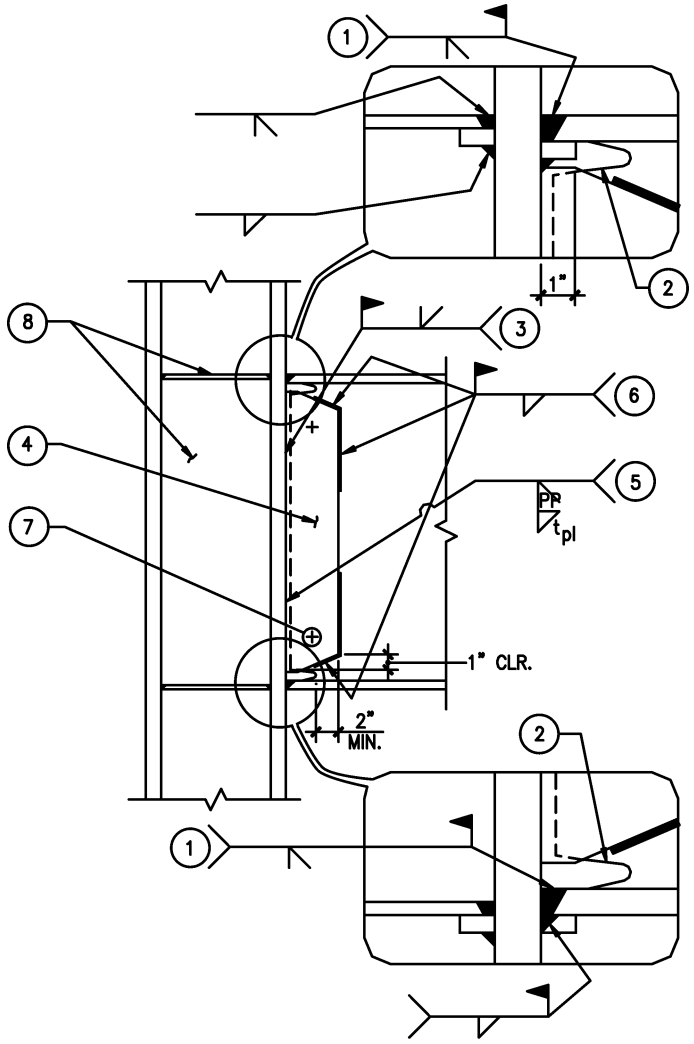
Step 7: Check requirements for continuity plates according to Section 3.3.3.1.

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.

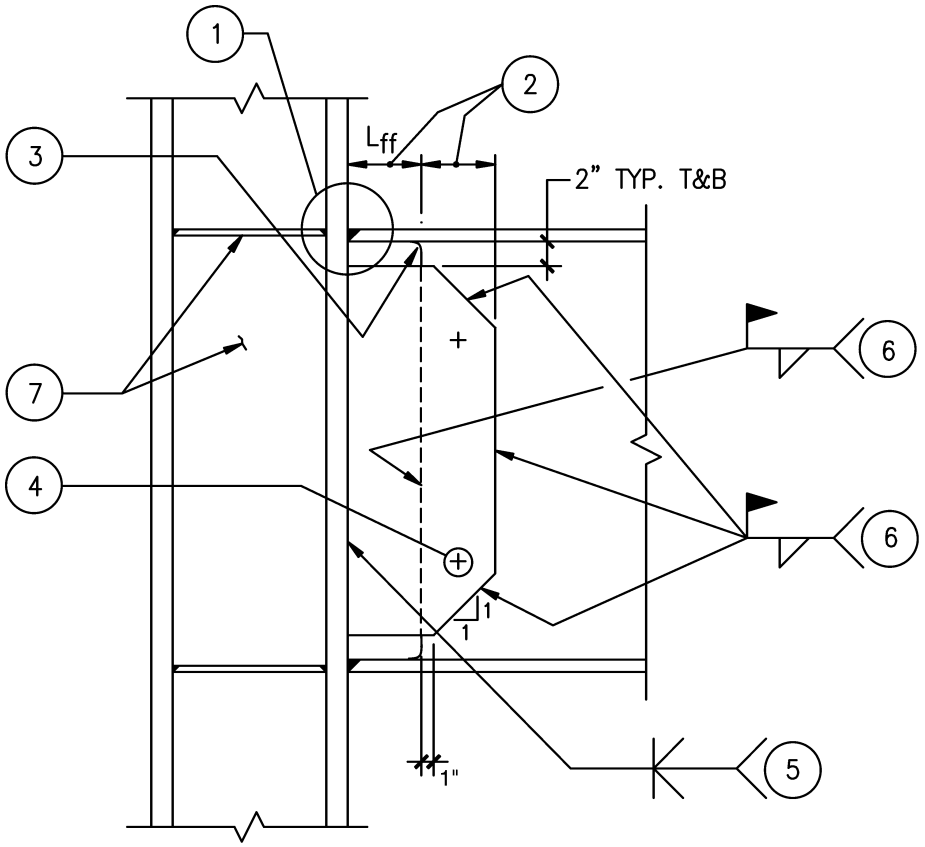
Commentary: This connection type was developed at the University of Michigan and has been extensively tested both at that university and at the University of Texas at Austin. This connection type has demonstrated good performance,

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.

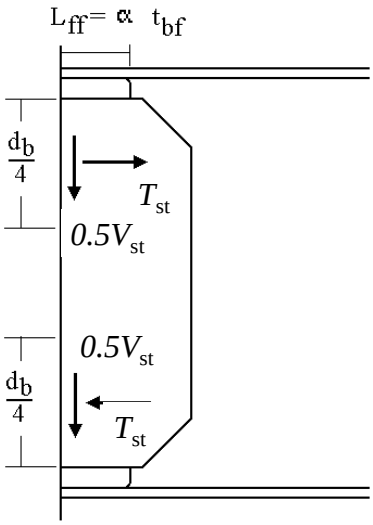
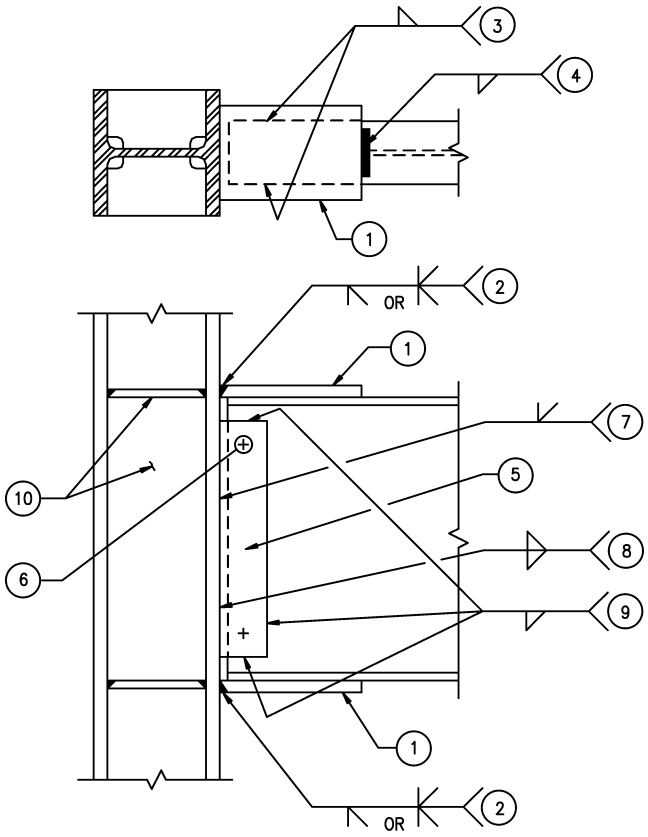


Figure 3-10 Schematic of the Forces for Design of the Free Flange Shear Tab

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* = $0.60F_{EXX}$

$t_w(\text{max}) = t_p - {}^{1}\!/_{16}\text{inch}$

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}

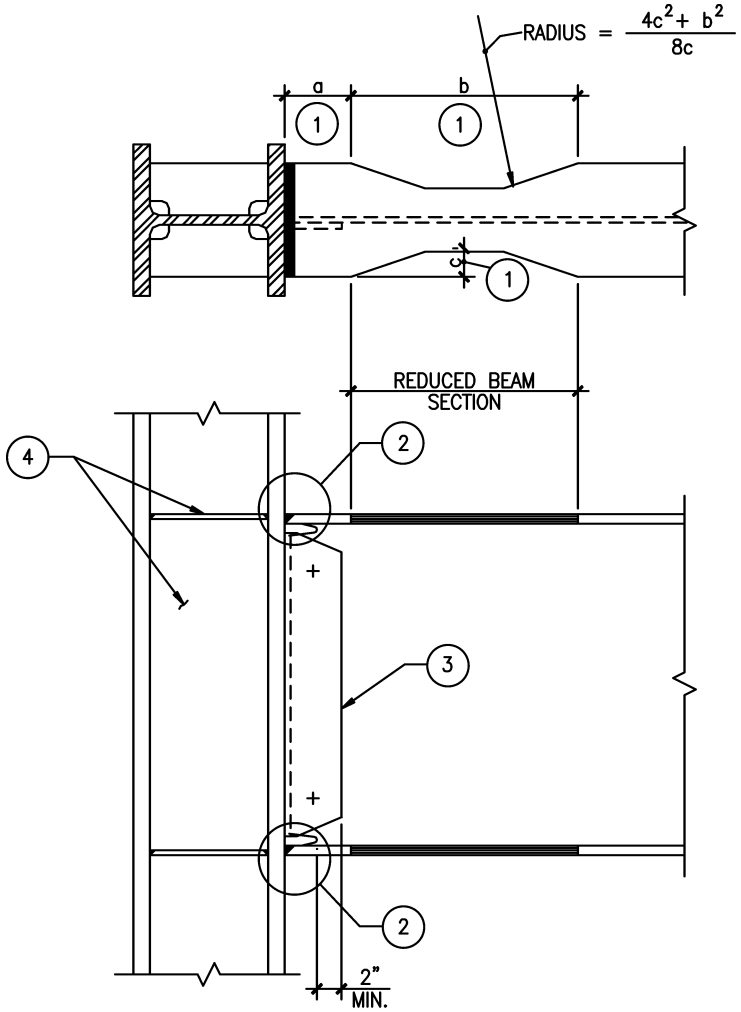
Step 7: Determine continuity plate requirements according to Section 3.3.3.1, For this purpose, use the plate width as the quantity b_f.

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 $52/\sqrt{F_y}$, 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 $c = 0.20 b_f$.

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 $M_f < C_{pr} R_y Z_b F_y$ 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

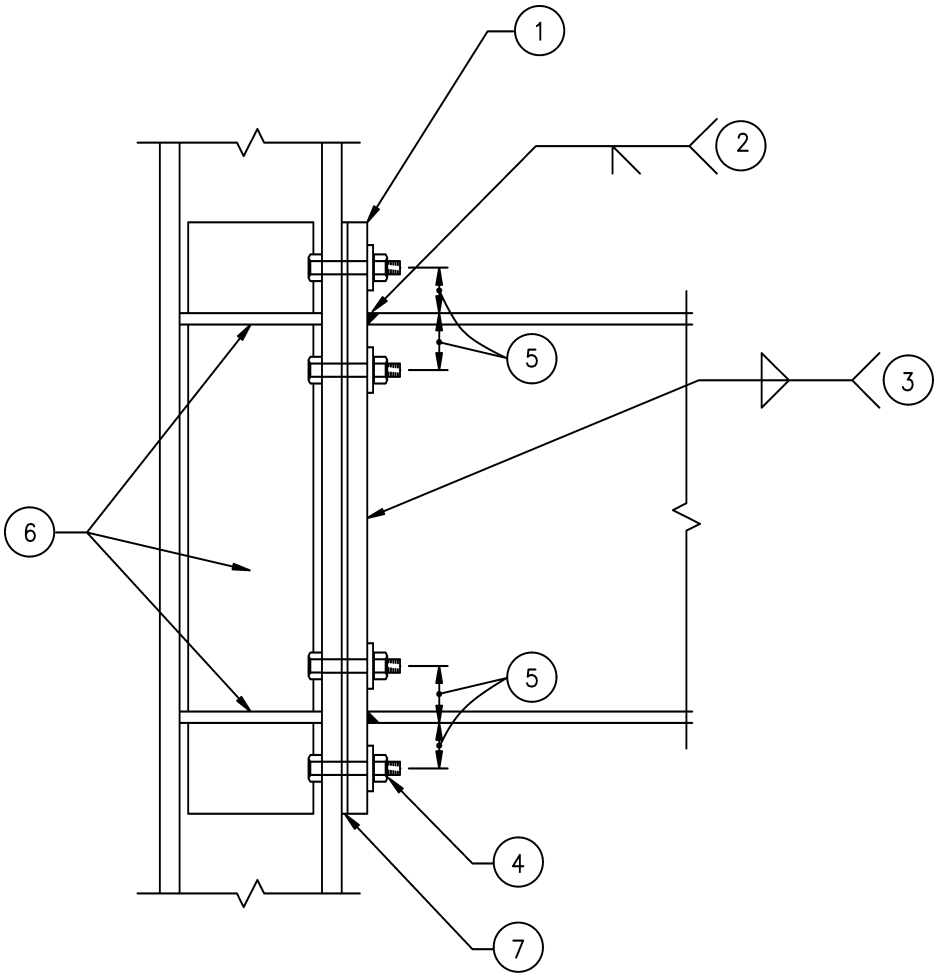
Connection Type	Criteria Section	Frame Type
Bolted Unstiffened End Plate (BUEP)	3.6.1	OMF, SMF
Bolted Stiffened End Plate (BSEP)	3.6.2	OMF, SMF
Bolted Flange Plate (BFP)	3.6.3	OMF, SMF
Double Split Tee (DST)*	3.7.1	OMF

*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:

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

where:

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

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

$$C_2 = \frac{b_{fc} - g}{2} \tag{3-28}$$

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

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.

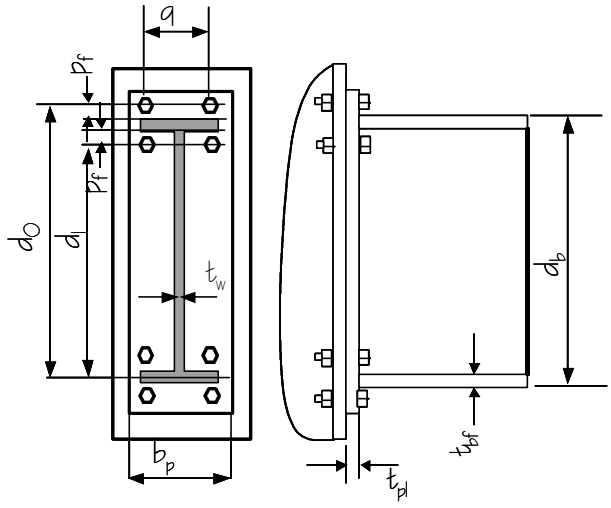
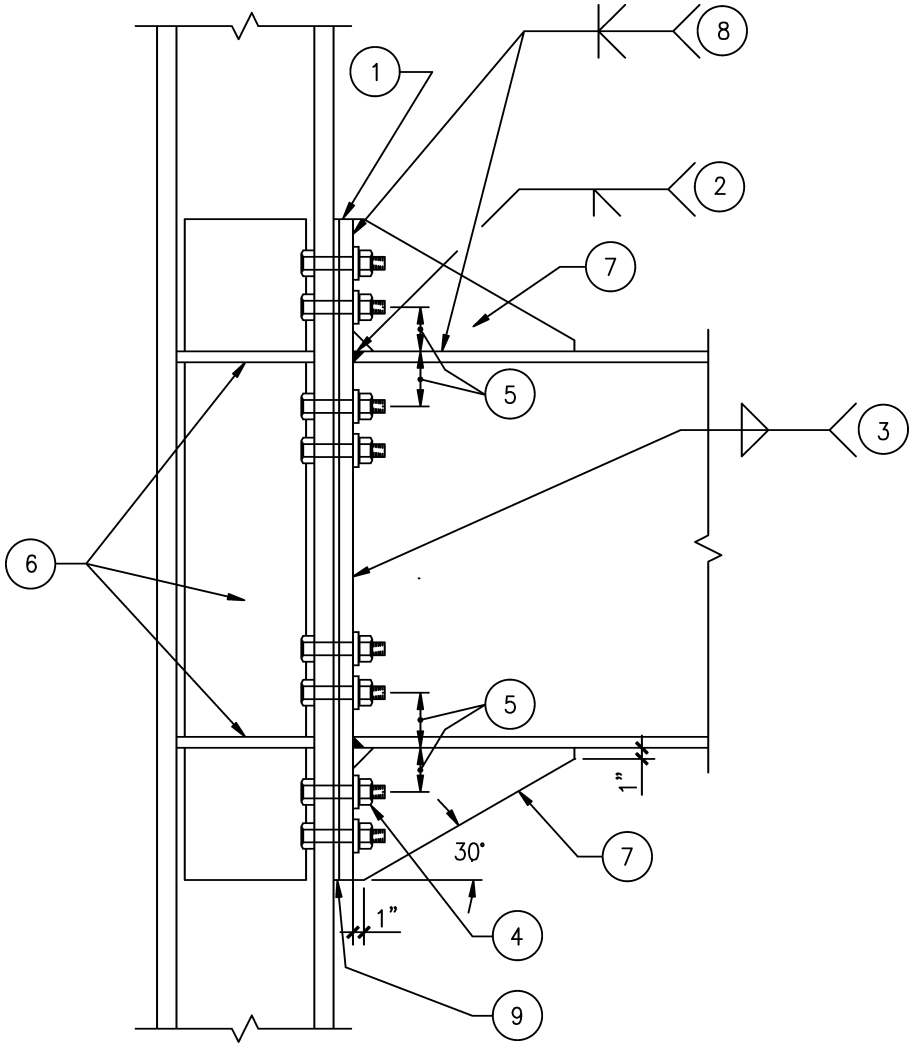


Figure 3-14 Geometry of Unstiffened End Plate Connection

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:

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

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.

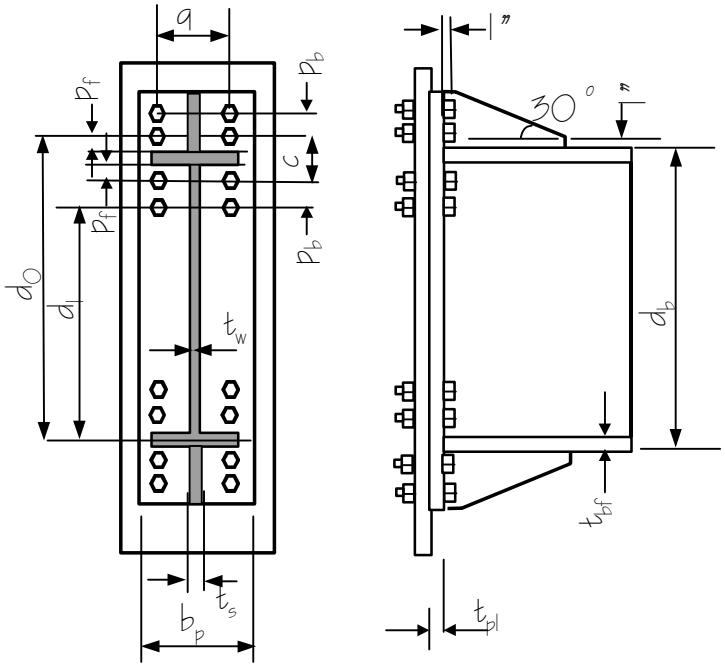


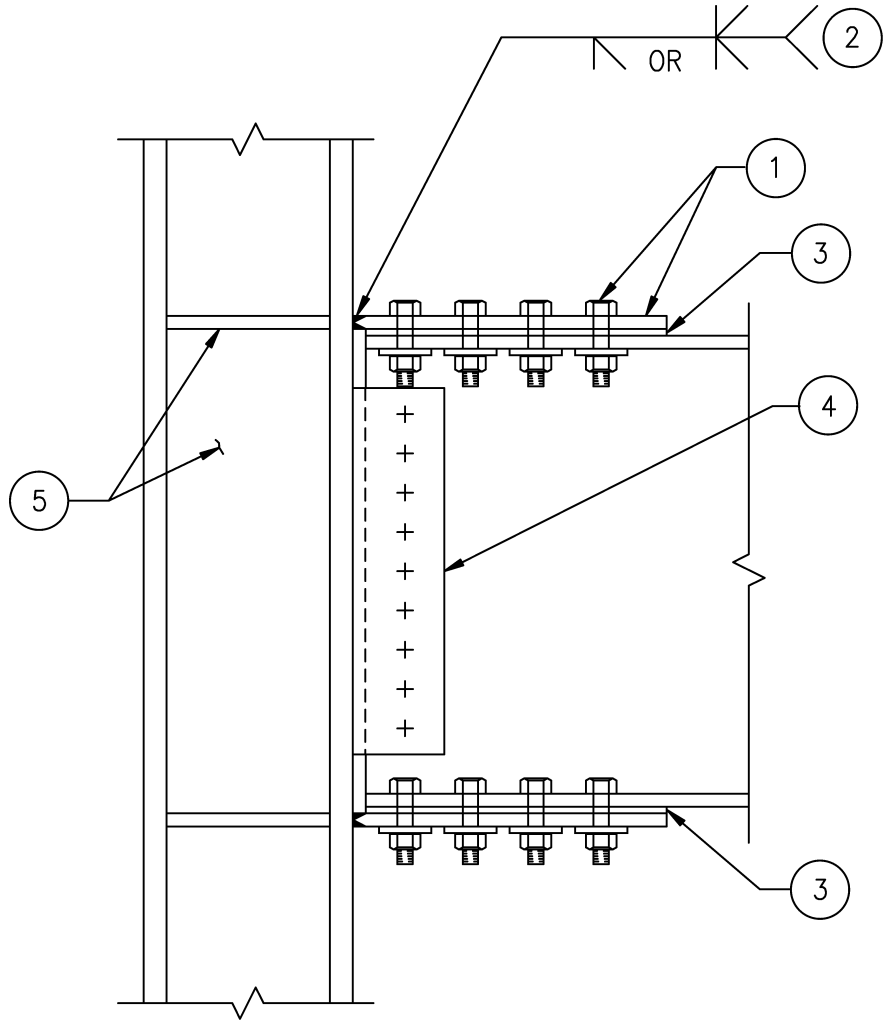
Figure 3-16 Geometry of Stiffened End Plate Connection

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.

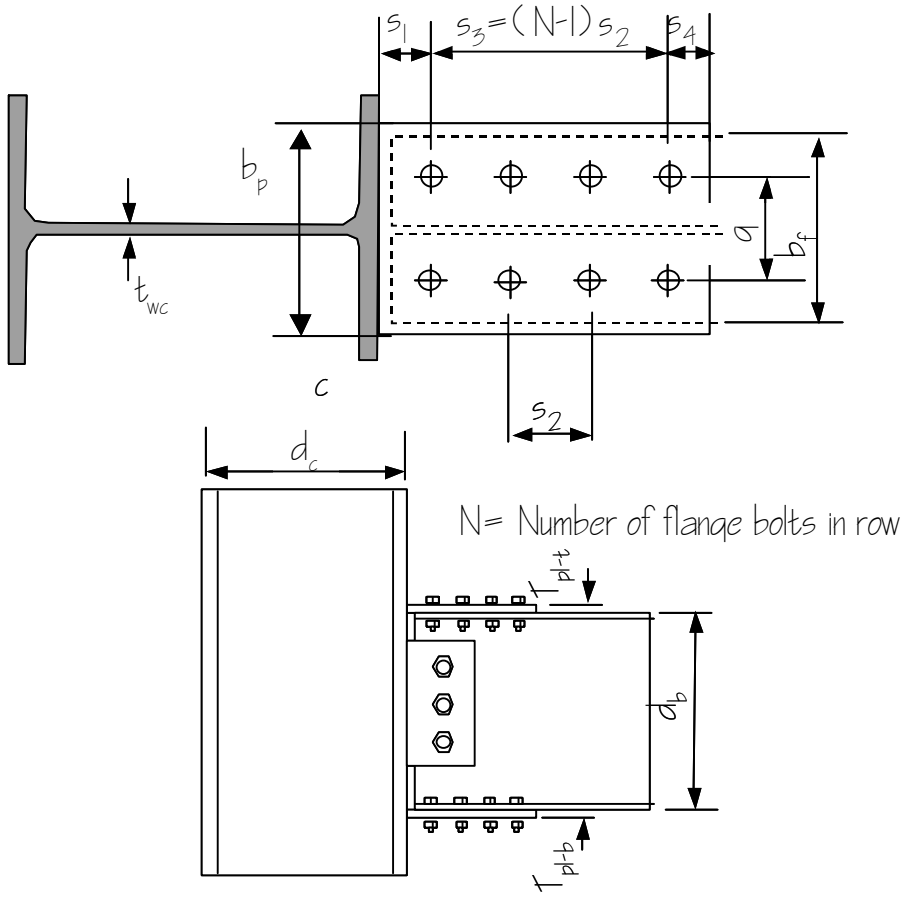


Figure 3-18 Geometry of the Bolted Flange Plate Connection

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:

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

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:

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

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:

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

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:

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

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.

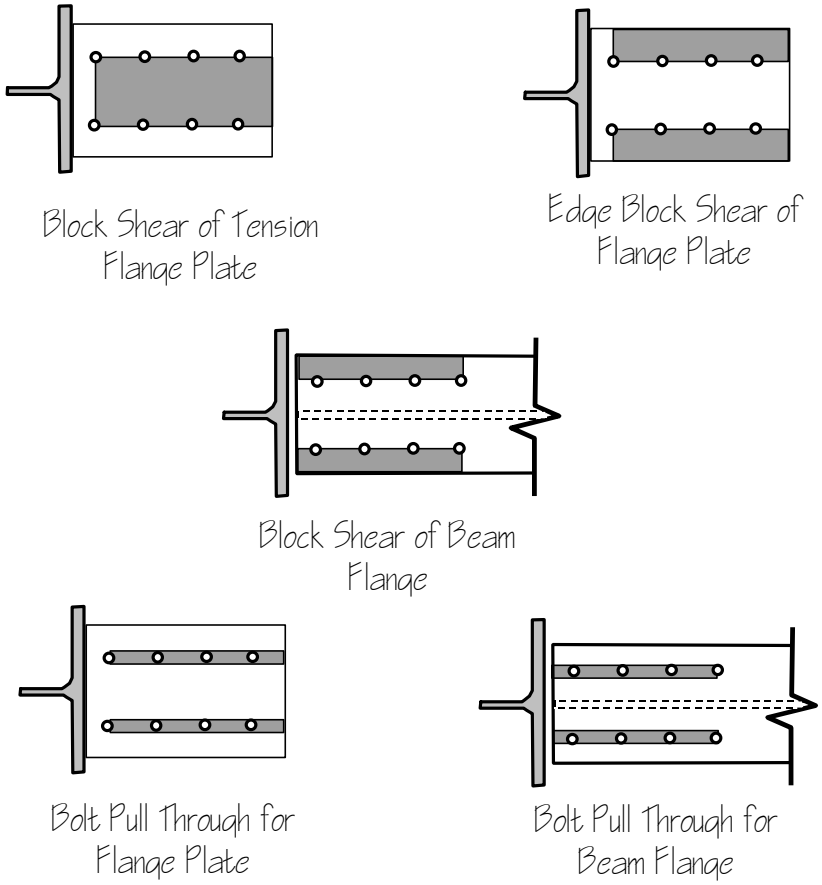


Figure 3-19 Block Shear and Pull-Through Failures

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.

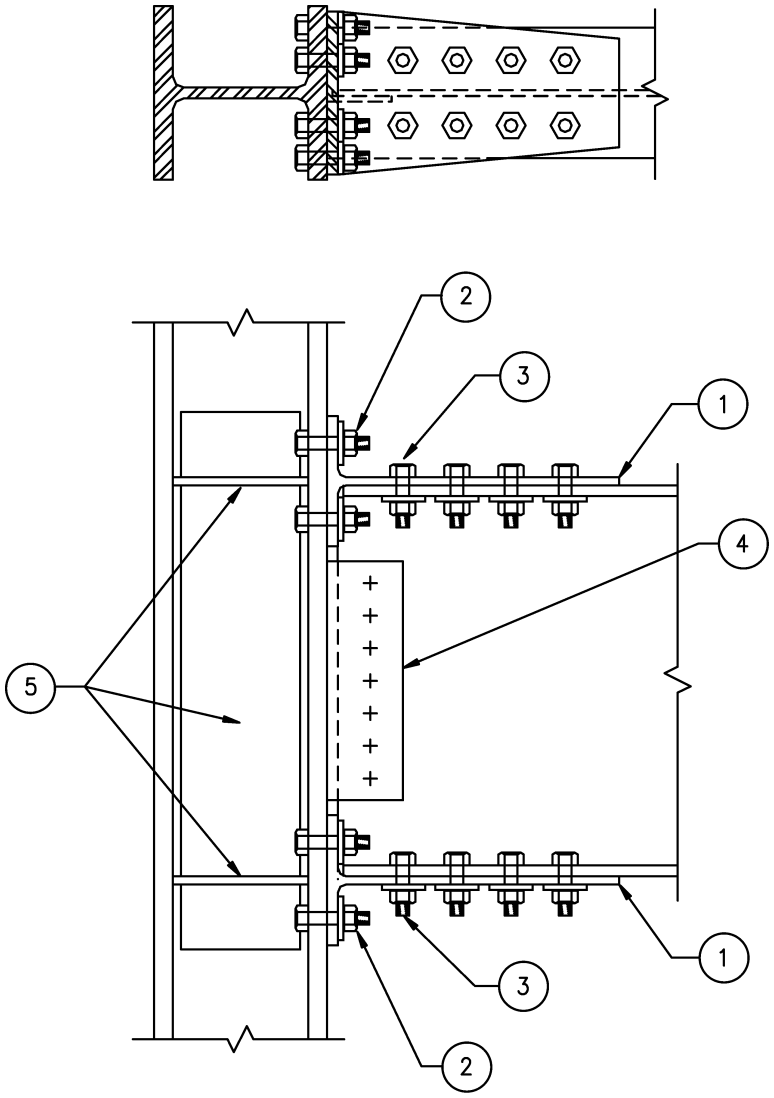
Moment connections that develop only partial strength, as well as partial stiffness, are sometimes used. Such connections are not prequalified in this document. This is not intended to preclude their use, if the system is properly justified by both analysis and testing. A significant amount of testing 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*.

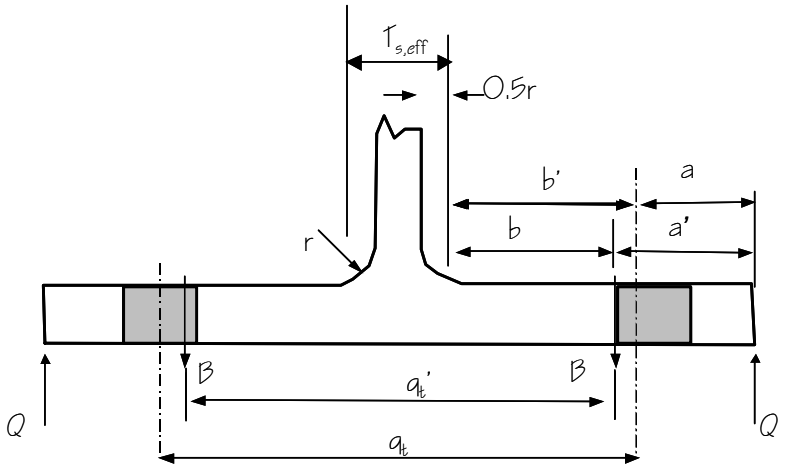


Figure 3-21 Geometry for Prying Forces and Bending of T-Section Flanges

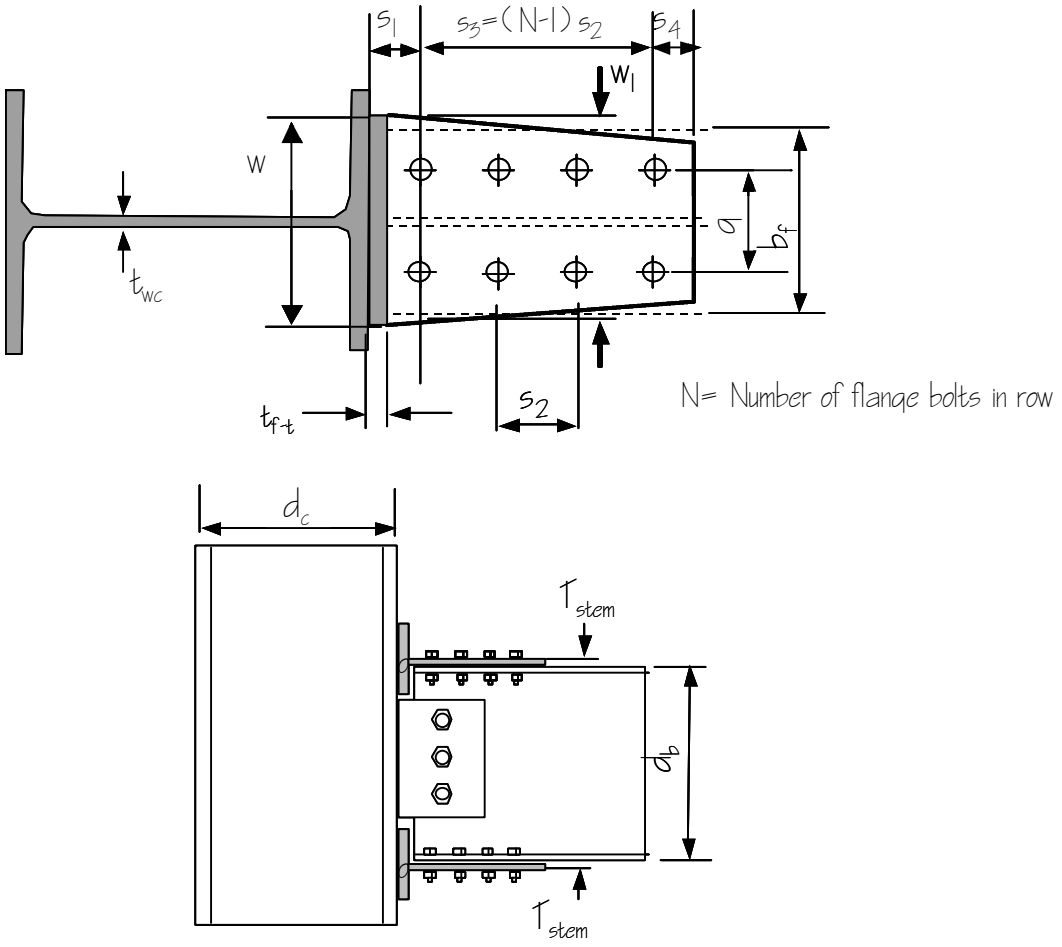


Figure 3-22 Geometry for Other T-Stub Failure Modes

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:

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

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:

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

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 f 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.

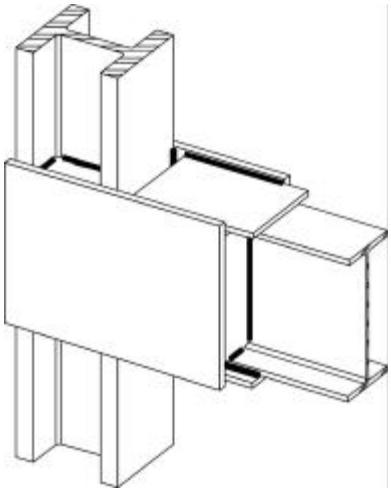


Figure 3-23 Proprietary Side Plate Connection

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.

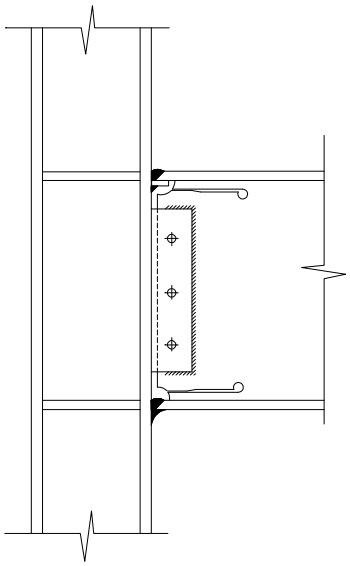


Figure 3-24 Proprietary Slotted Web Connection

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.

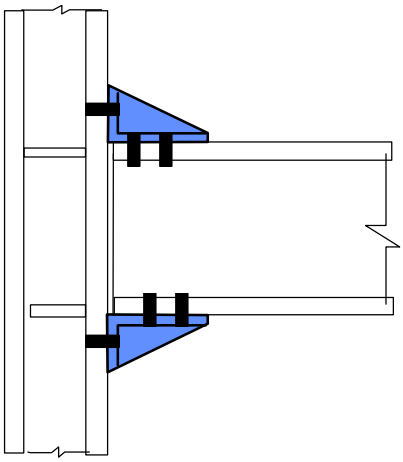


Figure 3-25 Bolted Bracket Connection

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.

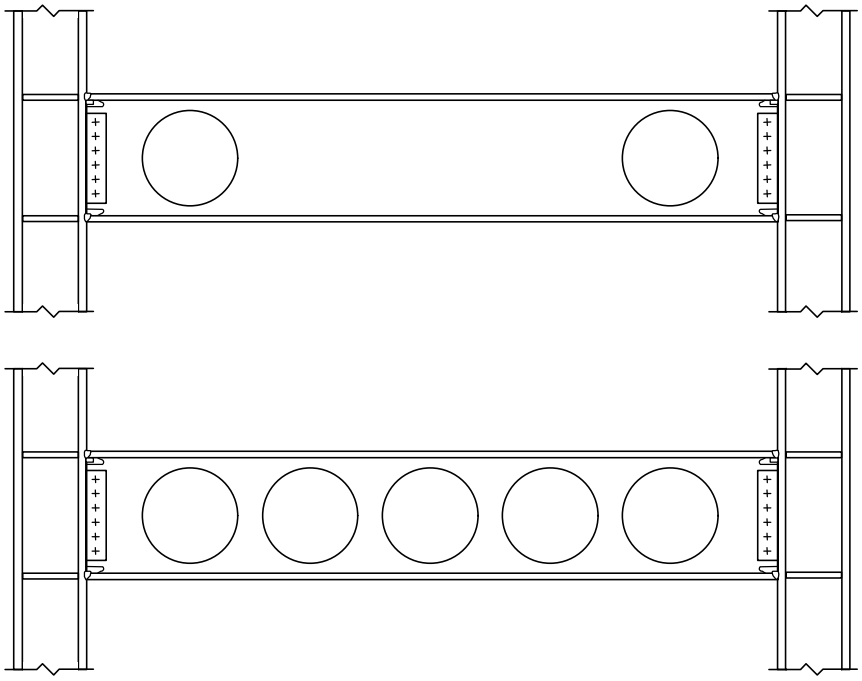


Figure 3-26 Reduced Web Connection

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.

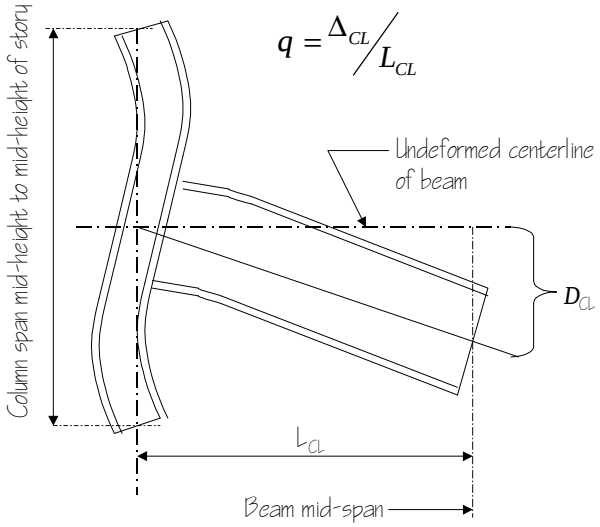


Figure 3-27 Angular Rotation of Test Assembly

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.

4. Performance Evaluation

4.1 Scope

This chapter provides simplified criteria for evaluating the probable seismic performance of welded steel moment-frame buildings. It may, as an option, be used in parallel with the design procedures of Chapters 2 and 3 of these *Recommended Criteria* to design steel moment-frame buildings for alternative performance capabilities and also to quantify the ability of a specific design to achieve desired performance objectives. It includes definitions of performance objectives, discussions of expected performance of buildings conforming to *FEMA-302, NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (BSSC, 1997a),* and procedures for estimating a level of confidence that a building will be able to provide a desired level of performance for specified earthquake hazards. It is applicable only to well-configured, regular structures as defined in *FEMA-302*. A more detailed procedure, applicable to irregular structures and performance objectives based on deterministic earthquake scenarios is presented in Appendix A of these *Recommended Criteria*.

Commentary: These criteria only address methods of evaluating structural performance of welded steel moment-frame buildings. Although the performance of nonstructural components of buildings is critically important to the way in which buildings are used following an earthquake, treatment of this topic is beyond the scope of this document. FEMA-273, NEHRP Guidelines for the Seismic Rehabilitation of Buildings, provides a more complete set of recommendations with regard to evaluating the performance of nonstructural components.

FEMA-355F, State of the Art Report on Performance Prediction and Evaluation, presents in detail the basis for the procedures contained herein and the derivation of the various parameters used in the procedures.

4.2 Performance Definition

The performance evaluation procedures contained in these *Recommended Criteria* permit estimation of a level of confidence that a structure will be able to achieve a desired performance objective. Each performance objective consists of the specification of a structural performance level and a corresponding hazard level, for which that performance level is to be achieved. For example, a design may be determined to provide a 95% level of confidence that the structure will provide Collapse Prevention or better performance for earthquake hazards with a 2% probability of exceedance in 50 years, or a 50% level of confidence that the structure will provide Immediate Occupancy or better performance, for earthquake hazards with a 50% probability of exceedance in 50 years.

Commentary: The performance evaluation procedures contained in these Recommended Criteria are based on an approach first developed in FEMA-273.

However, substantial modifications have been made to the procedures presented in that document.

In FEMA-273, performance objectives are expressed in a deterministic manner. Each performance objective consists of the specification of a limiting damage state, termed a performance level, together with a specification of the ground motion intensity for which that (or better) performance is to be provided. This implies a warranty that if the specified ground motion is actually experienced by a building designed using the FEMA-273 procedures, damage will be no worse than that indicated in the performance objective.

In reality, it is very difficult to predict with certainty how much damage a building will experience for a given level of ground motion. This is because there are many factors that affect the behavior and response of a building (such as the stiffness of nonstructural elements, the strength of individual building components, and the quality of construction) that cannot be precisely defined, and also because the analysis procedures used to predict building response are not completely accurate. In addition, the exact character of the ground motion that will actually affect a building is itself uncertain. Given these uncertainties, it is inappropriate to imply that performance can be predicted in an absolute sense, and correspondingly, that it is absolutely possible to produce designs that will achieve desired performance objectives.

In recognition of this, these Recommended Criteria adopt a reliability-based probabilistic approach to performance evaluation that explicitly acknowledges these inherent uncertainties. These uncertainties are expressed in terms of a confidence level. If an evaluation indicates a high level of confidence, for example 90 or 95% that a performance objective can be achieved, then this means it is very likely (but not guaranteed) that the building will be capable of meeting the desired performance. If lower confidence is calculated, for example 50%, this is an indication that the building may not be capable of meeting the desired performance objective. If still lower confidence is calculated, for example 30%, then this indicates the building will likely not be able to meet the desired performance objective. Increased confidence in a building's ability to provide specific performance can be obtained in three basic ways.

- *Providing the building with greater earthquake resistance, for example, by designing the structure to be stiffer and stronger.*
- *Reducing some of the uncertainty inherent in the performance evaluation process through the use of more accurate structural models and analyses and better data on the building's configuration, strength and stiffness.*
- *More accurately characterizing the uncertainties inherent in the performance evaluation process, for example, by using the more exact procedures of Appendix A of these Recommended Criteria.*

Refer also to the commentary in Section 4.2.1.2 for additional discussion of the probabilistic approach adopted by this document.

4.2.1 Hazard Specification

4.2.1.1 General

Earthquake hazards include direct ground fault rupture, ground shaking, liquefaction, lateral spreading, and land sliding. Of these various potential hazards, the one that effects the largest number of structures and causes the most widespread damage is ground shaking. Ground shaking is the only earthquake hazard that the structural design provisions of the building codes directly address. However, for structures located on sites where any of the other hazards can result in significant ground deformation, these hazards should also be considered in a structural performance evaluation.

4.2.1.2 Ground Shaking

Ground shaking hazards are typically characterized by a hazard curve, which indicates the probability that a given value of a ground motion parameter, for example peak ground acceleration, will be exceeded over a certain period of time, and by acceleration response spectra or ground motion accelerograms that are compatible with the values of the ground motion parameters obtained from the hazard curve and the local site geology. The ground shaking hazard maps contained in *FEMA-302* and provided with *FEMA-273* have been prepared based on hazard curves that have been developed by the United States Geologic Survey for a grid-work of sites encompassing the United States and its territories. *FEMA-302* defines two specific levels of hazard for consideration in design and specifies methods for developing response spectra for each of these levels. The two levels are:

1. Maximum Considered Earthquake (MCE) ground shaking. This is the most severe level of ground shaking that is deemed appropriate for consideration in the design process for building structures, though not necessarily the most severe level of ground shaking that could ever be experienced at a site. In most regions, this ground shaking has a 2% probability of exceedance in 50 years, or roughly a 2,500 year mean recurrence interval. In regions of very high seismicity, near major active faults, the MCE ground shaking level is limited by a conservative, deterministic estimate of the ground shaking resulting from a maximum magnitude earthquake on the known active faults in the region. The probability that such deterministic ground shaking will be experienced at a site can vary considerably, depending on the activity rate of the individual fault. Refer to *FEMA-303, Commentary to the NEHRP Recommended Provisions for Seismic Regulations of New Buildings and Other Structures,* for more detailed information on this issue.
2. Design Earthquake (DE) ground shaking. This is the ground shaking level upon which design lateral forces, used as the basis for analysis and design in *FEMA-302,* are based. It is defined as a spectrum that is 2/3 of the shaking intensity calculated for the MCE spectrum, at each period. The probability that DE ground shaking will be experienced varies, depending on the regional and in some cases, site seismicity.

Commentary: The mean recurrence interval for Design Earthquake (DE) ground shaking will vary depending on regional and site seismicity. In areas of low seismicity the DE return period will generally range between 750-1,250 years and will remain relatively constant across neighboring communities. In areas of high seismicity the recurrence interval may range between 300-600 years and can vary significantly within a distance of a few miles.

Performance evaluation conducted in accordance with these guidelines may be performed for any level of ground shaking. Ground shaking will typically be determined probabilistically, i.e., based on the probability that shaking of the specified intensity will be experienced at a site. Ground shaking must be characterized by an acceleration response spectrum or a suite of ground motion accelerograms compatible with that spectrum. In addition, a coefficient *k* that relates the rate of change in ground motion intensity with change in probability, is required. *FEMA-273* provides guidelines for development of ground motion response spectra at different probabilities of exceedance. The procedures of this chapter use a default value for the coefficient, *k*, as described in the commentary. Performance evaluation for deterministic ground motion based on specific earthquake scenarios, for example an earthquake of given magnitude on a specific fault, can also be performed. Appendix A provides procedures that may be used for deterministically defined hazards.

Commentary: Detailed guidelines on ground motion estimation and characterization are beyond the scope of this publication. Those interested in such information are referred to FEMA-303 and FEMA-274, Commentary to the NEHRP Guidelines for the Rehabilitation of Buildings, and references noted therein.

Although Section 4.2 of these Recommended Criteria indicates that performance objectives are an expression of the desired performance for a building, given that ground motion of certain intensity is experienced, this is a significant simplification. In reality, the performance objectives are statements of the total probability that damage experienced by a building in a period of years will be more severe than the desired amount (performance level), given our knowledge of the site seismicity. Although it is transparent to the user, this is obtained by integrating the conditional probability that building response exceeds the limiting response for a performance level, given a ground motion intensity, over the probability of experiencing different intensities of ground motion, as represented by the site hazard curve, and specifically, the coefficient k, which is the logarithmic slope of the hazard curve, at the desired hazard level. Thus, a performance objective that is stated as "meeting collapse prevention performance for ground motion with a 2% probability of exceedance in 50 years" should more correctly be stated as being "less than a 2% chance in 50 years that damage more severe than the collapse prevention level will occur, given the mean definition of seismicity."

It is important to note that the procedures contained in this chapter neglect uncertainties associated with the definition of the seismicity, that is, the intensity

of ground shaking at various probabilities. Such uncertainties can be as large, and perhaps larger, than the uncertainties associated with structural performance estimation. Thus, the confidence calculated in accordance with the procedures of this chapter is really a confidence associated with structural performance, given the presumed seismicity.

The simplified procedures presented in this chapter have been developed using hazard parameters typical of coastal California. They can be conservatively applied in regions of lower seismicity without the need to determine site-specific hazard parameters. However, accurate definition of the hazard is a critical part of the performance evaluation procedures contained herein and in regions of lower seismicity, may result in calculation of higher confidence. Appendix A of these Recommended Criteria presents more detailed procedures that may be used to consider directly the site-specific characteristics of hazard in the evaluation of performance.

4.2.1.3 Other Hazards

In order to predict reliably the probable performance of a structural design, it is necessary to determine if earthquake hazards other than ground shaking, including direct ground fault rupture, liquefaction, lateral spreading, and land sliding are likely to occur at a site and to estimate the severity of these effects. The severity of ground fault rupture, lateral spreading and land sliding is characterized by an estimate of permanent ground deformation. The severity of liquefaction is characterized by an estimate of the potential loss in bearing strength of subsoil layers and permanent ground settlement. In order to determine the performance of a structure that is subject to these hazards, the effects of the projected ground displacements should be evaluated using a mathematical model of the structure. The severity of these hazards (i.e. probability of exceedance) used in performance evaluation should be compatible with that used in the specification of ground shaking hazards.

Commentary: Most sites are not at significant risk from earthquake hazards other than ground shaking. However, these hazards can be very destructive to structures located on sites where they occur. Accurate determination of the propensity of a site to experience these hazards requires site-specific study by a competent earth scientist or geotechnical engineer. Guidelines on such assessments are beyond the scope of these Recommended Criteria.

4.2.2 Performance Levels

Building performance is a combination of the performance of both structural and nonstructural components. Table 4-1 describes the overall levels of structural and nonstructural damage that may be expected of buildings meeting two performance levels, termed Collapse Prevention and Immediate Occupancy. These performance descriptions are not precise and variation among buildings must be expected, within the same Performance Level. The structural performance levels are presented in Section 4.2.2.2.

Table 4-1 Building Performance Levels

	Building Performance Levels	
	Collapse Prevention Level	**Immediate Occupancy Level**
Overall Damage	**Severe**	**Light**
General	Little residual stiffness and strength, but gravity loads are supported. Large permanent drifts. Some exits may be blocked. Exterior cladding may be extensively damaged and some local failures may occur. Building is near collapse.	Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, ceilings, and structural elements. Elevators can be restarted. Fire protection operable.
Nonstructural components	Extensive damage.	Equipment and contents are generally secure, but may not operate due to mechanical failure or lack of utilities.
Comparison with performance intended by *FEMA-302* for SUG[1]-I buildings when subjected to the Design Earthquake	Significantly more damage and greater risk.	Much less damage and lower risk.
Comparison with performance intended by *FEMA-302* for SUG[1]-I buildings when subjected to the Maximum Considered Earthquake	Same level of performance	Much less damage and lower risk.

Note: 1. SUG = Seismic Use Group

Commentary: Building performance is expressed in terms of building performance levels. These building performance levels are discrete damage states selected from among the infinite spectrum of possible damage states that steel moment-frame buildings could experience as a result of earthquake response. The particular damage states identified as building performance levels have been selected because these performance levels have readily identifiable consequences associated with the postearthquake disposition of the building that are meaningful to the building user community and also because they are quantifiable in technical terms. These include the ability to resume normal functions within the building, the advisability of postearthquake occupancy, and the risk to life safety.

Although a building's performance is a function of the performance of both structural systems and nonstructural components and contents, only the structural

performance levels are defined in these Recommended Criteria. The reference to nonstructural components above is to remind the reader of the probable performance of these elements at the various performance levels.

4.2.2.1 Nonstructural Performance Levels

These *Recommended Criteria* only address methods of evaluating structural performance of steel moment-frame buildings. Although the performance of nonstructural components of buildings are critically important to the way in which buildings are used following an earthquake, treatment of this topic is beyond the scope of this document. *FEMA-273* provides a more complete set of recommendations for evaluating the performance of nonstructural components.

4.2.2.2 Structural Performance Levels

Two discrete structural performance levels, Collapse Prevention and Immediate Occupancy, are defined in these *Recommended Criteria*. Table 4-2 relates these structural performance levels to the limiting damage states for common framing elements of steel moment-frame buildings. Acceptance criteria, which relate to the permissible interstory drifts and earthquake-induced forces for the various elements of steel moment-frame buildings, are tied directly to these structural performance levels and are presented in later sections of these *Recommended Criteria*.

Commentary: FEMA-273 defines three structural performance levels, Immediate Occupancy, Life Safety, and Collapse Prevention, and also defines two performance ranges. These performance ranges, rather than representing discrete damage states, span the entire spectrum of potential damage states between no damage and total damage. No acceptance criteria are provided for these performance ranges in FEMA-273. Rather, these must be determined on a project-specific basis, by interpolation or extrapolation from the criteria provided for the three performance levels. Performance ranges, as such, are not defined in these *Recommended Criteria. However, compatible with the FEMA-273 approach, users have the ability to create their own, custom performance levels, and to develop acceptance criteria for these levels, based on interpolation between the two performance levels, to suit the needs of a specific project. When such interpolation is performed, it is not possible to associate a confidence level with achievement of these intermediate performance definitions.*

The Life Safety performance level contained in FEMA-273 and FEMA-302 is not included in these Recommended Criteria. As defined in FEMA-273 and FEMA-302, the Life Safety level is a damage state in which significant damage has been sustained, although some margin remains against either partial or total collapse. In FEMA-273 this margin is taken as 1/3. That is, it is anticipated that a ground motion level that is 1/3 larger than that which results in the Life Safety performance level for a building would be required to bring the building to the Collapse Prevention level. In FEMA-302, this margin is taken as ½, i.e., it is believed that buildings designed for Life Safety performance can experience

approximately 50% greater motion before they reach the Collapse Prevention level. Due to the somewhat arbitrary definition of this performance level, and the fact that different guidelines and codes have selected alternative definitions for it (as described above), the Life Safety level has not been included in these Recommended Criteria. However, as with the performance ranges, users desiring to evaluate buildings for the Life Safety performance level may do so by interpolating between the acceptance criteria provided for the Collapse Prevention and Immediate Occupancy levels.

Table 4-2 Structural Performance Levels

	Structural Performance Levels	
Elements	**Collapse Prevention**	**Immediate Occupancy**
Girder	Extensive distortion; local yielding and buckling. A few girders may experience partial fractures	Minor local yielding and buckling at a few places.
Column	Moderate distortion; some columns experience yielding. Some local buckling of flanges	No observable damage or distortion
Beam-Column Connections	Many fractures with some connections experiencing near total loss of capacity	Less than 10% of connections fractured on any one floor; minor yielding at other connections
Panel Zone	Extensive distortion	Minor distortion
Column Splice	No fractures	No yielding
Base Plate	Extensive yielding of anchor bolts and base plate	No observable damage or distortion
Interstory Drift	Large permanent	Less than 1% permanent

4.2.2.2.1 Collapse Prevention Performance Level

The Collapse Prevention structural performance level is defined as the postearthquake damage state in which the structure is on the verge of experiencing partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral-force-resisting system, large permanent lateral deformation of the structure, and, to a more limited extent, degradation in the vertical-load-carrying capacity. However, all significant components of the gravity-load-resisting system must continue to carry their gravity-load demands. The structure may not be technically or economically practical to repair and is not safe for re-occupancy; aftershock activity could credibly induce collapse.

4.2.2.2.2 Immediate Occupancy Performance Level

The Immediate Occupancy structural performance level is defined as the postearthquake damage state in which only limited structural damage has occurred. Damage is anticipated to be

so slight that it would not be necessary to inspect the building for damage following the earthquake, and such little damage as may be present would not require repair. The basic vertical- and lateral-force-resisting systems of the building retain nearly all of their pre-earthquake strength and stiffness. The risk of life-threatening injury as a result of structural damage is very low. Buildings meeting this performance level should be safe for immediate postearthquake occupancy, presuming that damage to nonstructural components is suitably light and that needed utility services are available.

Commentary: When a building is subjected to earthquake ground motion, a pattern of lateral deformations that varies with time is induced in the structure. At any given point in time, a particular state of lateral deformation will exist in the structure, and at some time within the period in which the structure is responding to the ground motion, a maximum pattern of deformation will occur. At relatively low levels of ground motion, the deformations induced within the building will be limited, and the resulting stresses that develop within the structural components will be within their elastic range of behavior. Within this elastic range, the structure will experience no damage. All structural components will retain their original strength, stiffness and appearance, and when the ground motion stops, the structure will return to its pre-earthquake condition.

At more severe levels of ground motion, the lateral deformations induced in the structure will be larger. As these deformations increase, so will demands on the individual structural components. At different levels of deformation, corresponding to different levels of ground motion severity, individual components of the structure will be strained beyond their elastic range. As this occurs, the structure starts to experience damage in the form of buckling, yielding and fracturing of the various components. As components become damaged, they degrade in stiffness, and some elements will begin to lose their strength. In general, when a structure has responded to ground motion within this range of behavior, it will not return to its pre-earthquake condition when the ground motion stops. Some permanent deformation may remain within the structure and damage may be evident throughout. Depending on how far the structure has been deformed, and in what pattern, the structure may have lost a significant amount of its original stiffness and, possibly, strength.

Brittle elements are not able to sustain inelastic deformations and will fail suddenly; the consequences may range from local and repairable damage to collapse of the structural system. At higher levels of ground motion, the lateral deformations induced in a structure will strain a number of elements to a point at which the elements degrade in stiffness and strength, or as a result of P-D effects, the structure loses stability. Eventually, partial or total collapse of the structure can occur. The structural performance levels relate the extent of a building's response to earthquake hazards to these various possible damage states.

At the Immediate Occupancy Level, degradation of strength and stiffness in beam-column connections is limited to approximately 10% of the connections on any given floor and throughout the structure as a whole. The structure retains a significant portion of its original stiffness and most, if not all, of its strength, although some slight permanent drift may result. At the Collapse Prevention level, the building has experienced extreme damage. If laterally deformed beyond this point, the structure can experience instability and can collapse.

4.3 Evaluation Approach

The basic process of performance evaluation, as contained in these *Recommended Criteria* is to develop a mathematical model of the structure and to evaluate its response to the earthquake hazards by one or more methods of structural analysis. The structural analysis is used to predict the value of various structural response parameters. These include:

- interstory drift
- axial forces on individual columns

These structural response parameters are related to the amount of damage experienced by individual structural components as well as the structure as a whole. For each performance level, these *Recommended Criteria* specify acceptance criteria (median estimates of capacity) for each of the design parameters indicated above. Acceptability of structural performance is evaluated considering both local performance (element level) and global performance. Acceptance criteria have been developed on a reliability basis, incorporating demand and resistance factors related to the uncertainty inherent in the evaluation process and variation inherent in structural response, such that a confidence level can be established with regard to the ability of a structure to actually provide specific performance at selected probabilities of exceedance.

Once an analysis is performed, predicted demands are adjusted by two factors, an analytical uncertainty factor g_a, which corrects the analytically predicted demands for bias and uncertainty inherent in the analytical technique, and demand variability factor g, which accounts for other sources of variability in structural response. These predicted demands are compared against acceptance criteria, which have been modified by resistance factors f to account for uncertainties and variation inherent in structural capacity. Procedures are given to calculate the level of confidence provided by a design to achieve specific performance objectives, based on the ratio of factored demand to factored capacity. If the predicted level of confidence is inadequate, then either more detailed analyses should be performed to improve the level of confidence attained with regard to performance, through the attainment of better understanding of the structure's probable behavior and modification of the demand and capacity factors, or the design can be revised such that a sufficient level of confidence can be attained given the level of understanding. If it is deemed appropriate to upgrade a design to improve its probable performance, an iterative approach consisting of trial design, followed by verification analysis, evaluation of design parameters against acceptance criteria, and calculation of confidence level is repeated until an acceptable design solution is found. Procedures for estimating confidence are found in Section 4.6.

Commentary: These Recommended Criteria adopt a demand and resistance factor design (DRFD) model for performance evaluation. This approach is similar to the Load and Resistance Factor design (LRFD) approach adopted by AISC LRFD except that the LRFD provisions are conducted on an element basis, rather than a structural system basis, and demands in this document can be drifts as well as forces and stresses. The purpose of this DRFD approach is to allow characterization of the confidence level inherent in a design to a specific performance objective.

The factored interstory drift demand $g_a gD$ calculated from the analysis, represents a median estimate of the probable maximum interstory drift demand, at the desired hazard level. Tables in these Recommended Criteria provide interstory drift capacities for the two performance levels for regular, well-configured structures, dependent on structural system and connection type, as well as resistance factors f that adjust the estimated capacity of the structure to median values. Appendix A provides procedures for determination of f factors for connections for which project-specific qualification testing is performed and a procedure that may be used to determine interstory drift capacities for irregular structures.

Once the factored demands and capacities are determined, an index parameter l is calculated from the ratio of the factored demands and capacities, as indicated in Section 4.6. The value of l is then used to determine an associated confidence level based on tabulated values related to the uncertainty inherent in the estimation of the building's demands and capacities.

4.4 Analysis

In order to evaluate the performance of a steel moment-frame building it is necessary to construct a mathematical model of the structure that represents its strength and deformation characteristics, and to conduct an analysis to predict the values of various design parameters when it is subjected to design ground motion. This section provides guidelines for selecting an appropriate analysis procedure and for modeling. General requirements for the mathematical model are presented in Section 4.5.

4.4.1 Alternative Procedures

Four alternative analytical procedures are available for use in performance evaluation of steel moment-frame buildings. The basic analytical procedures are described in detail in *FEMA-273*. This section provides supplementary guidelines on the applicability of the *FEMA-273* procedures and also provides supplemental modeling recommendations. The four procedures are:

- linear static procedure – an equivalent lateral force technique, similar, but not identical, to that contained in many model building code provisions,
- linear dynamic procedure – an elastic, modal, response-spectrum analysis,

- nonlinear static procedure – a simplified nonlinear analysis procedure in which the forces and deformations induced by a monotonically increasing lateral loading is evaluated using a series of incremental elastic analyses of structural models that are sequentially degraded to represent the effects of structural nonlinearity,
- nonlinear dynamic procedure – a nonlinear dynamic analysis procedure in which the response of a structure to a suite of ground motion histories is determined through numerical integration of the equations of motion for the structure. Structural stiffness is altered during the analysis to conform to nonlinear hysteretic models of the structural components.

Commentary: The purpose of the structural analyses performed as part of the performance evaluation process is to predict the values of key response parameters that are indicative of the structure's performance when it is subjected to ground motion. Once the values of these response parameters are predicted, the structure is evaluated for adequacy (appropriate level of confidence of achieving desired performance) using the basic approach outlined in Section 4.6.

Analyses performed in support of design, as required by FEMA-302, evaluate the strength and deformation of the structure when it is subjected to a somewhat arbitrary level of loading. The loading is based on, but substantially reduced from, that predicted by an elastic analysis of the structure's dynamic response to the expected ground motions. Specifically, the loading is reduced by a factor R to account approximately for the beneficial effects of inelastic response.

Analyses conducted in support of performance evaluation under these Recommended Criteria take a markedly different approach. Rather than evaluating the forces and deformations induced in the structure under arbitrarily reduced loading levels, these analysis procedures attempt to predict, within probabilistically defined bounds, the actual values of the important response parameters in response to design ground motion.

The ability of the performance evaluation to estimate reliably the probable performance of the structure is dependent on the ability of the analysis procedure to predict the values of these response parameters within acceptable levels of confidence. The linear dynamic procedure is able to provide relatively reliable estimates of the response parameters for structures that exhibit elastic, or near elastic, behavior. The linear static procedure inherently has more uncertainty associated with its estimates of the response parameters because it accounts less accurately for the dynamic characteristics of the structure. The nonlinear static procedure is more reliable than the linear procedures in predicting response parameters for structures that exhibit significant nonlinear behavior, particularly if they are irregular. However, it does not accurately account for the effects of higher mode response. If appropriate modeling is performed, the nonlinear dynamic approach is most capable of capturing the probable behavior of the real structure in response to ground motion. However, there are considerable

uncertainties associated even with the values of the response parameters predicted by this technique.

4.4.2 Procedure Selection

Table 4-3 indicates the recommended analysis procedures for various performance levels and conditions of structural regularity.

4.4.3 Linear Static Procedure

4.4.3.1 Basis of the Procedure

Linear static procedure (LSP) analysis of steel moment-frame structures should be conducted in accordance with the recommendations of *FEMA-273*, except as noted herein. In this procedure, lateral forces are applied to the masses of the structure, and deflections and component forces under this applied loading are determined. Calculated internal forces typically will exceed those that the building can develop, because anticipated inelastic response of components and elements is not directly recognized by the procedure. The predicted interstory drifts and column axial forces are evaluated using the procedures of Section 4.6.

Commentary: The linear static procedure is a method of estimating the response of the structure to earthquake ground shaking by representing the effects of this response through the application of a series of static lateral forces applied to an elastic mathematical model of the structure and its stiffness. The forces are applied to the structure in a pattern that represents the typical distribution of inertial forces in a regular structure responding in a linear manner to the ground shaking excitation, factored to account, in an approximate manner, for the probable inelastic behavior of the structure. It is assumed that the structure's response is dominated by the fundamental mode and that the lateral drifts induced in the elastic structural model by these forces represent a reasonable estimate of the actual deformation of the building when responding inelastically.

In the LSP, the building is modeled with linearly-elastic stiffness and equivalent viscous damping that approximate values expected for loading to near the yield point. The static lateral forces, whose sum is equal to the pseudo lateral load, represent earthquake demands for the LSP. The magnitude of the pseudo lateral load has been selected with the intention that when it is applied to the linearly elastic model of the building it will result in design displacement amplitudes approximating maximum displacements that are expected during the design earthquake. If the building responds essentially elastically to the design earthquake, the calculated internal forces will be reasonable approximations of those expected during the design earthquake. If the building responds inelastically to the design earthquake, as will commonly be the case, the internal forces that would develop in the yielding building will be less than the internal forces calculated on an elastic basis, but the predicted interstory drifts will approximate those that would actually occur in the structure.

Table 4-3 Analysis Procedure Selection Criteria

Structural Characteristics				Analytical Procedure			
Performance Level	Fundamental Period, T	Regularity	Ratio of Column to Beam Strength	Linear Static	Linear Dynamic	Nonlinear Static	Nonlinear Dynamic
Immediate Occupancy	$T \leq 3.5T_s$[1]	Regular or Irregular	Any Condition	Permitted	Permitted	Permitted	Permitted
	$T > 3.5\ T_s$[1]	Regular or Irregular	Any Conditions	Not Permitted	Permitted	Not Permitted	Permitted
Collapse Prevention	$T \leq 3.5T_s$[1]	Regular[2]	Strong Column[3]	Permitted	Permitted	Permitted	Permitted
			Weak Column[3]	Not Permitted	Not Permitted	Permitted	Permitted
		Irregular[2]	Any Conditions	Not Permitted	Not Permitted	Permitted	Permitted
	$T > 3.5T_s$	Regular	Strong Column[3]	Not Permitted	Permitted	Not Permitted	Permitted
			Weak Column[3]	Not Permitted	Not Permitted	Not Permitted	Permitted
		Irregular[2]	Any Conditions	Not Permitted	Not Permitted	Not Permitted	Permitted

Notes:

1. T_s is the period at which the response spectrum transitions from a domain of constant response acceleration (the plateau of the response spectrum curve) to one of constant spectral velocity. Refer to *FEMA-273* or *FEMA-302* for more information.

2. Conditions of regularity are as defined in *FEMA-273*. These conditions are significantly different from those defined in *FEMA-302*.

3. A structure qualifies as having a strong column condition if at every floor level, the quantity SM_{prc}/SM_{prb} is greater than 1.0, where SM_{prc} and SM_{prb} are the sum of the expected plastic moment strengths of the columns and beams, respectively, that participate in the moment-resisting framing in a given direction of structural response.

The performance of steel moment-frame structures is most closely related to the total inelastic deformation demands on the various elements that comprise the structure, such as plastic rotation demands on beam-column assemblies and tensile demands on column splices. Linear analysis methods do not permit direct evaluation of such demands. However, through a series of analytical evaluations of typical buildings for a number of earthquake records, it has been possible to develop statistical correlation between the interstory drift demands predicted by a linear analysis and the actual inelastic deformation demands determined by more accurate nonlinear methods. These correlation relationships are reasonably valid for regular buildings, using the definitions of regularity in FEMA-273.

Although performance of steel moment-frame structures is closely related to interstory drift demand, there are some failure mechanisms, notably, the failure of column splices, that are more closely related to strength demand. However, since inelastic structural behavior affects the strength demand on such elements, linear analysis is not capable of directly predicting these demands, except when the structural response is essentially elastic. Therefore, when linear static analysis is performed for structures that respond in an inelastic manner, column axial demands should be estimated using a supplementary plastic analysis approach.

The linear static procedure is based on the assumption that the distribution of deformations predicted by an elastic analysis is similar to that which will occur in actual nonlinear response. This assumption is inaccurate and can become more so for structures that are highly irregular, that have response dominated by higher modes, or that experience large inelastic demands. It is for these reasons that alternative methods of analysis are recommended for irregular structures and structures with relatively long fundamental periods of vibration.

4.4.3.2 Period Determination

A fundamental period shall be calculated for each of two orthogonal directions of building response, by one of the following three methods.

Method 1. Eigenvalue (dynamic) analysis of the mathematical model of the building. The model for buildings with flexible diaphragms shall consider representation of diaphragm flexibility unless it can be shown that the effects of omission will not be significant.

Method 2. Evaluation of the following equation:

$$T = C_t h_n^{0.8} \tag{4-1}$$

where

T = fundamental period (in seconds) in the direction under consideration,
C_t = 0.028 for steel moment frames,

h_n = height (in feet) to the roof level, above the base.

Method 3. The fundamental period of a one-story building with a single-span, flexible diaphragm may be calculated as:

$$T = (0.1\Delta_w + 0.078\Delta_d)^{0.5} \quad (4\text{-}2)$$

where D_w and D_d are in-plane frame and diaphragm displacements, respectively, in inches, due to a lateral load, in the direction under consideration, equal to the weight tributary to the diaphragm. For multiple-span diaphragms, a lateral load equal to the gravity weight tributary to the diaphragm span under consideration should be applied to each diaphragm span to calculate a separate period for each diaphragm span. The loads from each diaphragm should then be distributed to the frames using tributary load assumptions.

Commentary: The approximate period formula indicated in Method 2 is different from that contained in either FEMA-273 or FEMA-302. This formula has been adapted from a recent study of the statistical distribution of measured periods in buildings obtained from accelerometer recordings of excitation occurring in past earthquakes (Goel and Chopra, 1997). This formula is intended to provide approximately an 84% confidence level (mean + one s) that the actual period will exceed the calculated value. The formula has intentionally been selected to under-estimate the actual period of the building as this will result in a conservatively large estimate of the calculated pseudo lateral force applied to the structure as a loading (see Section 4.4.3.3.1). The large pseudo lateral force will result in conservatively large estimates of interstory drift.

Use of the more accurate Method 1 procedure will typically result in lower estimates of interstory drift, and therefore increased confidence in the ability of a building to meet performance goals.

4.4.3.3 Determination of Actions and Deformations

4.4.3.3.1 Pseudo Lateral Load

A pseudo lateral load V, given by Equation 4-3, shall be independently calculated for each of the two orthogonal directions of building response, and applied to a mathematical model of the structure.

$$V = C_1 C_2 C_3 S_a W \quad (4\text{-}3)$$

where:

C_1 = a modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response. C_1 may be calculated using the procedure indicated in Section 3.3.3.3 in *FEMA-273* with the elastic base

shear capacity substituted for V_y. Alternatively, C_1 may be taken as having a value of 1.0 when the fundamental period T of the building response is greater than T_s and shall be taken as having a value of 1.5 when the fundamental period of the structure is equal to or less than T_0. Linear interpolation shall be used to calculate C_1 for intermediate values of T.

T_0 = period at which the acceleration response spectrum for the site reaches its peak value, as indicated in *FEMA-302*. It may be taken as $0.2T_s$.

T_S = the characteristic period of the response spectrum, defined as the period associated with the transition from the constant spectral response acceleration segment of the spectrum to the constant spectral response velocity segment of the spectrum, as defined in *FEMA-302*.

C_2 = a modification factor to represent the effect of hysteretic pinching on the maximum displacement response. For steel moment frames the value of C_2 shall be taken as 1.0.

C_3 = a modification factor to represent increased dynamic displacements due to *P-D* effects and stiffness degradation. C_3 may be taken from Table 4-4 or shall be calculated from the equation:

$$C_3 = 1 + \frac{5(q_i - 0.1)}{T} \geq 1.0 \qquad (4\text{-}4)$$

where:

q_i = the coefficient calculated in accordance with Section 2.1.1.2 of *FEMA-273*. The maximum value q_i for all stories in the building shall be used.

S_a = Response spectrum acceleration, at the fundamental period and damping ratio of the building in the direction under consideration, for the hazard level corresponding to the performance objective being evaluated (i.e., probability of exceedance). The value of S_a may be calculated using the procedure outlined in Section 2.6.1.5 of *FEMA-273*.

W = Total dead load and anticipated live load as indicated below:

- In storage and warehouse occupancies, a minimum of 25% of the floor live load
- The actual partition weight or minimum weight of 10 psf of floor area, whichever is greater
- The applicable snow load – see *FEMA-302*
- The total weight of permanent equipment and furnishings

Table 4-4 Modification Factor C_3 for the Linear Static Procedure

Performance Level	C_3
Immediate Occupancy	1.0
Collapse Prevention	
Connections meeting the criteria for Special Moment Frame structures in accordance with Chapter 3	1.2
Connections meeting the criteria for Ordinary Moment Frame structures in accordance with Chapter 3	1.4

Commentary: The pseudo lateral force, when distributed over the height of the linear-elastic model of the structure, is intended to produce calculated lateral displacements approximately equal to those that are expected in the real structure during the design event. If it is expected that the actual structure will yield during the design event, the force given by Equation 4-3 may be significantly larger than the actual strength of the structure to resist this force. The acceptance evaluation procedures in Section 4.6 are developed to take this into account.

The values of C_3 in Table 4-4 are conservative for most structures, and will generally result in calculation of an unduly low level of confidence. Use of Equation 4-4 to calculate C_3 is one way to improve calculated confidence without extensive additional effort, and is recommended.

4.4.3.3.2 Vertical Distribution of Seismic Forces

The lateral load F_x applied at any floor level x shall be determined as given in Section 3.3.1.3B of *FEMA-273*.

4.4.3.3.3 Horizontal Distribution of Seismic Forces

The seismic forces at each floor level of the building shall be distributed according to the distribution of mass at that floor level.

4.4.3.3.4 Diaphragms

Floor and roof diaphragms shall be evaluated using the procedure outlined in Section 3.3.1.3D in *FEMA-273*. The lateral seismic load on each flexible diaphragm shall be distributed along the span of that diaphragm, considering its displaced shape.

4.4.3.3.5 Determination of Interstory Drift

Interstory drifts shall be calculated using lateral loads in accordance with Section 4.4.3.3.1 and stiffness obtained from Section 4.5. Factored interstory drift demands $g_a g d_i$ at each story i shall be determined by applying the appropriate analysis uncertainty factor g_a and demand variability factor g obtained from Section 4.6.

4.4.3.3.6 Determination of Column Demands

Factored demands on columns and column splices shall be obtained by multiplying the calculated column forces by the applicable analysis uncertainty factor g_a and demand variability factor g obtained in Section 3.6.3. Column forces shall be calculated either as:

1. the axial demands from the unreduced linear analysis, or
2. the axial demands computed from the equation:

$$P'_c = \pm\left[2\left(\sum_{i=x}^{n}\frac{M_{pe}}{L}\right)_L - 2\left(\sum_{i=x}^{n}\frac{M_{pe}}{L}\right)_R\right] \quad (4\text{-}5)$$

where:

$\left(\sum_{i=x}^{n}\frac{M_{pe}}{L}\right)_L$ = the summation of the expected plastic moment strength (ZF_{ye}) divided by the span length L, of all beams framing into the left hand side of the column, above the level under consideration, and

$\left(\sum_{i=x}^{n}\frac{M_{pe}}{L}\right)_R$ = the summation of the expected plastic moment strength (ZF_{ye}) divided by the span length L, of all beams framing into the right hand side of the column, above the level under consideration.

When a column is part of framing that resists lateral forces under multiple directions of loading, the Seismic Demand shall be taken as the most severe condition resulting from application of 100% of the Seismic Demand computed for any one direction of response with 30% of the Seismic Demand computed for the orthogonal direction of response.

4.4.4 Linear Dynamic Procedure

4.4.4.1 Basis of the Procedure

Linear dynamic procedure (LDP) analysis of steel moment frames shall be conducted in accordance with the recommendations in Section 3.3.2 of *FEMA-273* except as specifically noted herein. Coefficients C_1, C_2, and C_3 should be taken as indicated in Section 4.4.3.3 of this document.

Estimates of interstory drift and column axial demands shall be evaluated using the applicable procedures of Section 4.6. Calculated displacements and column axial demands are factored by the applicable analysis uncertainty factor g_a and demand variability factor g obtained from Section 4.6 and compared with factored capacity values for the appropriate performance level. Calculated internal forces typically will exceed those that the building can sustain because of anticipated inelastic response of components and elements, but are generally not used to evaluate performance.

Commentary: The linear dynamic procedure (LDP) is similar in approach to the linear static procedure (LSP), described in Section 4.4.3. However, because it directly accounts for the stiffness and mass distribution of the structure in calculating the dynamic response characteristics, its use introduces somewhat less uncertainty than does the LSP. Coefficients C_1, C_2, and C_3, which account in an approximate manner for the differences between elastic predictions of response and inelastic behavior, are the same as for the linear static method. In the LDP, inertial seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements are determined using a linear-elastic, response spectrum analysis.

The basis, modeling approaches and acceptance criteria of the LDP are similar to those for the LSP. The main exception is that the response calculations are carried out using modal response spectrum analysis (RSA). Modal spectral analysis is carried out using unreduced, linear-elastic response spectra scaled to the hazard level (probability of exceedance) inherent in the desired performance objective. As with the LSP, it is expected that the LDP will produce estimates of displacements and interstory drifts that are approximately correct, but will produce estimates of internal forces that exceed those that would be obtained in a yielding building.

4.4.4.2 Analysis

4.4.4.2.1 General

The LDP shall conform to the criteria in Section 3.3.2.2 of *FEMA-273*. The analysis shall be based on appropriate characterization of the ground motion. The requirement that all significant modes be included in the response analysis may be satisfied by including sufficient modes to capture at least 90% of the participating mass of the building in each of the building's principal horizontal directions. Modal damping ratios should reflect the damping inherent in the building at deformation levels less than the yield deformation. Except for buildings incorporating passive or active energy dissipation devices, or base isolation technology, effective damping shall be taken as 5% of critical.

The interstory drift, and other response parameters calculated for each mode, and required for evaluation in accordance with Section 4.4.4.3, should be combined by recognized methods to estimate total response. Modal combination by either the SRSS (square root of the sum of squares) rule or the CQC (complete quadratic combination) rule is acceptable.

Multidirectional excitation effects may be accounted for by combining 100% of the response due to loading in direction A with 30% of the response due to loading in the direction B, and by combining 30% of the response in direction A with 100% of the response in direction B, where A and B are orthogonal directions of response for the building.

4.4.4.2.2 Ground Motion Characterization

The horizontal ground motion should be characterized by one of the following methods:

1. An elastic response spectrum, developed in accordance with the recommendations of Section 2.6.1.5 in *FEMA-273* for the hazard level contained in the desired performance objective.
2. A site-specific response spectrum developed in accordance with the recommendations of Section 2.6.2.1 of *FEMA-273* for the appropriate hazard level contained in the desired performance objective.

4.4.4.3 Determination of Actions and Deformations

4.4.4.3.1 Factored Interstory Drift Demand

Factored interstory drift demand shall be obtained by multiplying the interstory drift results of the response spectrum analysis by the product of the modification factors, C_1, C_2, and C_3, defined in Section 4.4.3 and by the applicable analysis uncertainty factor g_a and demand variability factor g obtained from Section 4.6.

4.4.4.3.2 Determination of Column Demands

Factored demands on columns and column splices shall be obtained by multiplying the calculated column forces, as given in Section 4.4.3.3.6, by the applicable analysis uncertainty factor g_a and demand variability factor g obtained from Section 4.6.3.

4.4.5 Nonlinear Static Procedure

4.4.5.1 Basis of the Procedure

Under the Nonlinear Static Procedure (NSP), a model directly incorporating the inelastic material and nonlinear geometric response is displaced to a target displacement, and resulting internal deformations and forces are determined. The nonlinear load-deformation characteristics of individual components and elements of the building are modeled directly. The mathematical model of the building is subjected to a pattern of monotonically increasing lateral forces or displacements until either a target displacement is exceeded or a mathematical instability occurs. The target displacement is intended to approximate the total maximum displacement likely to be experienced by the actual structure, at the hazard level corresponding to the selected performance objective. The target displacement should be calculated in accordance with the procedure presented in Section 3.3.3.3 of *FEMA-273* with the modifications indicated below. Because the mathematical model accounts directly for effects of material and geometric nonlinear response, the calculated internal forces will be reasonable approximations of those expected during the design earthquake, presuming that an appropriate pattern of loading has been applied.

Interstory drifts and column axial demands obtained from the NSP are evaluated using the applicable procedures of Section 4.6. Calculated interstory drifts, column forces, and column splice forces are factored, and compared directly with factored acceptance values for the applicable performance level.

Commentary: The nonlinear static analysis approach inherently assumes behavior is dominated by the first mode response of the structure. For this reason, these Recommended Criteria state that this approach be used only for structures with relatively short periods. What constitutes a building with a "short period" is dependent on the spectral characteristics of ground shaking anticipated at the site. The small magnitude events that dominate the hazard at many central and eastern U.S. sites tend to have most of their energy at short periods, particularly on firm soil and rock sites. For sites subject to such shaking, nonlinear static analyses may be valid only for short-period, rigid structures. The limitations on use of the Nonlinear Static Procedure (NSP), based on period, contained in Table 4-3, are based on recent work that indicates that higher mode response does not become significant in structures responding to ground shaking having typical response spectra unless the fundamental period of the structure is more than about 3.5 times the period at which the spectrum transitions from a range of constant response acceleration to constant response velocity.

A second potential limitation of this procedure is that in practice, two-dimensional models are often used to simulate three-dimensional response. Estimates of load distribution between the lateral-load-resisting elements in the building are required, and the accuracy of the analysis depends upon the accuracy of distribution. Three-dimensional linearly elastic models may be used to estimate this distribution; however, these models are unable to account for load redistribution occurring because of inelastic behavior. When many plastic hinges form nearly simultaneously, creating local frame mechanisms, the effects of torsional contributions may not be accurately represented. If a structure has significant torsional irregularity, three-dimensional models should be used.

The NSP is also limited with regard to evaluation of simultaneous response to ground shaking in different directions. Little research has been performed on appropriate methods of accounting for multi-directional response using this technique. Therefore, these Recommended Criteria have adapted standard approaches used in linear analysis for this purpose.

4.4.5.2 Analysis Considerations

4.4.5.2.1 General

In the context of these *Recommended Criteria*, the NSP involves the application of incrementally adjusted, monotonically increasing, lateral forces, or displacements, to a mathematical nonlinear model of a building, until the displacement of a control node in the mathematical model exceeds a target displacement. For buildings that are not symmetric about a plane perpendicular to the applied lateral loads, the lateral loads must be applied in both the positive and negative directions, and the maximum forces and deformations obtained from both directions used for design.

The relation between base shear force and lateral displacement of the control node should be established for control node displacements ranging between zero and 150% of the target displacement d_t given by Equation 3-11 of *FEMA-273*. Performance evaluation should be based on those column forces and interstory drifts corresponding to minimum horizontal displacement of the control node equal to the target displacement d_t corresponding to the hazard level appropriate to the performance objective being evaluated.

Gravity loads shall be applied to appropriate elements and components of the mathematical model during the Nonlinear Static Procedure (NSP). The loads and load combinations shall be as follows:

1. 100% of computed dead loads and permanent live loads shall be applied to the model.
2. 25% of transient floor live loads shall be applied to the model, except in warehouse and storage occupancies, where the percentage of live load used in the analysis shall be based on a realistic assessment of the average long-term loading.

The analysis model should be discretized in sufficient detail to represent adequately the load-deformation response of each component along its length. Particular attention should be paid to identifying locations of inelastic action along the length of a component, as well as at its ends.

Commentary: As with any nonlinear model, the ability of the analyst to detect the presence of inelastic behavior requires the use of a nonlinear finite element at the assumed location of yielding. The model will fail to detect inelastic behavior when appropriately distributed finite elements are not used. However, as an alternative to the use of nonlinear elements, it is possible to use linear elements and reconfigure the model, for example, by adjusting member restraints, as nonlinearity is predicted to occur. For example, when a member is predicted to develop a plastic hinge, a linear model can be revised to place a hinge at this location. When this approach is used, the internal forces and stresses that caused the hinging must be reapplied as a nonvarying static load.

The recommendation to continue the pushover analysis to displacements that are 150% of the target displacement is to allow an understanding of the probable behavior of the building under somewhat larger loading than anticipated. If the pushover analysis should become unstable prior to reaching 150% of the target displacement, this does not indicate that a design is unacceptable, but does provide an indication of how much reserve remains in the structure at the design ground motion.

4.4.5.2.2 Control Node

The NSP requires definition of a control node in the building. These *Recommended Criteria* consider the control node to be the center of mass at the roof of the building; the top of a penthouse should not be considered as the roof unless it is of such substantial area and construction as to materially affect the response. The displacement of the control node is

compared with the target displacement – a displacement that characterizes the effects of earthquake shaking at the desired hazard level.

4.4.5.2.3 Lateral Load Patterns

Lateral loads should be applied to the building in profiles given in Section 3.3.3.2C of *FEMA-273*.

4.4.5.2.4 Period Determination

The effective fundamental period T_e in the direction under consideration shall be calculated using the force-displacement relationship of the Nonlinear Static Procedure (NSP) as described in Section 3.3.3.2D of *FEMA-273*.

4.4.5.2.5 Analysis of Three-Dimensional Models

Static lateral forces shall be imposed on the three-dimensional mathematical model corresponding to the mass distribution at each floor level.

Independent analysis along each principal axis of the three-dimensional mathematical model is permitted unless multidirectional evaluation is required by Section 3.2.7 in *FEMA-273*. Refer also to Section 4.4.5.3.4 of these *Recommended Criteria*.

4.4.5.2.6 Analysis of Two-Dimensional Models

Mathematical models describing the framing along each axis of the building should be developed for two-dimensional analysis. The effects of horizontal torsion should be considered as required by Section 3.2.2.2 of *FEMA-273*.

4.4.5.3 Determination of Actions and Deformations

4.4.5.3.1 Target Displacement

The target displacement d_t for buildings with rigid diaphragms at each floor level shall be estimated using the procedures of Section 3.3.3.3A of *FEMA-273*. Actions and deformations corresponding to the control node displacement equal to the target displacement shall be used for evaluation of performance evaluation in accordance with Section 4.6.

4.4.5.3.2 Diaphragms

The lateral seismic load on each flexible diaphragm shall be distributed along the span of that diaphragm, considering its displaced shape.

4.4.5.3.3 Factored Interstory Drift Demand

Factored interstory drift demand shall be obtained by multiplying the maximum interstory drift calculated at the target displacement by the applicable analysis uncertainty factor g_a and demand variability factor g obtained from Section 4.6.

4.4.5.3.4 Multidirectional Effects

Multidirectional excitation effects may be accounted for by combining 100% of the response due to loading in direction A with 30% of the response due to loading in the direction B; and by combining 30% of the response in direction A with 100% of the response in direction B, where A and B are orthogonal directions of response for the building.

An acceptable alternative to this approach is to perform the Nonlinear Static Procedure (NSP) analysis simultaneously in two orthogonal directions by application of 100% of the loading in direction A simultaneously with 30% of the loading in direction B. Loading shall be applied until 100% of the target displacement in direction A is achieved. This procedure shall be repeated with 100% of the loading applied in direction A and 30% in direction B.

4.4.5.3.5 Factored Column and Column Splice Demands

Factored demands on columns and column splices shall be obtained by multiplying the calculated column forces at the target displacement by the applicable analysis uncertainty factor g_a and demand variability factor g from Section 4.6.

4.4.6 Nonlinear Dynamic Procedure

4.4.6.1 Basis of the Procedure

Under the Nonlinear Dynamic Procedure (NDP), inertial seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements are determined using an inelastic response-history dynamic analysis.

The basis, the modeling approaches, and the acceptance criteria for the NDP are similar to those for the NSP. The main exception is that the response calculations are carried out using response-history analysis. With the NDP, the design displacements are not established using a target displacement, but instead are determined directly through dynamic analysis using suites of ground motion records. Calculated response can be highly sensitive to characteristics of individual ground motions; therefore, it is necessary to carry out the analysis with more than one ground motion record. Because the numerical model accounts directly for effects of material and geometric inelastic response, the calculated internal forces will be reasonable approximations of those expected during the design earthquake.

Results of the NDP are to be checked using the applicable acceptance criteria of Section 4.6. Calculated displacements and internal forces are factored, and compared directly with factored acceptance values for the applicable performance level.

4.4.6.2 Analysis Assumptions

4.4.6.2.1 General

The NDP shall conform to the criteria given in Section 3.3.4.2A of *FEMA-273*.

4.4.6.2.2 Ground Motion Characterization

The earthquake shaking should be characterized by suites of ground motion acceleration histories, prepared in accordance with the recommendations of Section 2.6.2 of *FEMA-273* and corresponding to the hazard level appropriate to the desired performance objective. A minimum of three pairs of ground motion records shall be used. Each pair shall consist of two orthogonal components of the ground motion.

Consideration of multidirectional excitation effects required by Section 3.2.7 of *FEMA-273* may be satisfied by analysis of a three-dimensional mathematical model using simultaneously imposed pairs of earthquake ground motion records along the horizontal axes of the building.

The effects of torsion should be considered according to Section 3.2.2.2 of *FEMA-273*.

4.4.6.3 Determination of Actions and Deformations

4.4.6.3.1 Response Quantities

Response quantities shall be computed as follows:

1. If less than seven pairs of ground motion records are used to perform the analyses, each response quantity (for example, interstory drift demand, or column axial demand) shall be taken as the maximum value obtained from any of the analyses.
2. If seven or more pairs of ground motion records are used to perform the analyses, the median value of each of the response quantities computed from the suite of analyses may be used as the demand. The median value shall be that value exceeded by 50% of the analyses in the suite.

4.4.6.3.2 Factored Interstory Drift Demand

Factored interstory drift demand shall be obtained by multiplying the maximum of the interstory drifts calculated in accordance with Section 4.4.6.3.1 by the applicable analysis uncertainty factor g_a and demand variability factor g obtained from Section 4.6.

4.4.6.3.3 Factored Column and Column Splice Demands

Factored demands on columns and column splices shall be obtained by multiplying the column forces calculated in accordance with Section 4.4.6.3.1 by the applicable analysis uncertainty factor g_a and demand variability factor g from Section 4.6.

4.5 Mathematical Modeling

4.5.1 Basic Assumptions

In general, a steel moment-frame building should be modeled and analyzed as a three-dimensional assembly of elements and components. Although two-dimensional models may provide adequate response information for regular, symmetric structures and structures with

flexible diaphragms, three-dimensional mathematical models should be used for analysis and design of buildings with plan irregularity as defined in *FEMA-302*. Two-dimensional modeling, analysis, and design of buildings with stiff or rigid diaphragms is acceptable, if torsional effects are either sufficiently small to be ignored, or are captured indirectly.

Vertical lines of framing in buildings with flexible diaphragms may be individually modeled, analyzed and designed as two-dimensional assemblies of components and elements, or a three-dimensional model may be used with the diaphragms modeled as flexible elements.

Explicit modeling of connection force-deformation behaviors for fully restrained connections is not required for linear analysis procedures. The stiffness of partially restrained connections should be modeled in linear procedures in accordance with the recommendations of Section 4.5.2. In nonlinear procedures explicit modeling of connection stiffness is recommended for those cases when the connection is weaker than the connected components, or when it is appropriate to model strength degradation in the connection as a function of imposed deformation demand. Refer to Section 4.5.2.

Commentary: A finite element model will only collect information at places in the structure where a modeling element is inserted. When nonlinear deformations are expected in a structure, the analyst must anticipate the location of these deformations (such as plastic hinges) and insert nonlinear finite elements at these locations if the inelastic behavior is to be captured by the model.

4.5.2 Frame Configuration

The analytical model should accurately account for the stiffness of frame elements and connections. Element and component stiffness properties, strength estimates and locations of plastic hinge formation for both linear and nonlinear procedures can be determined from information given in Chapter 3 for prequalified connections.

4.5.2.1 Modeling

Only the beams and columns forming the lateral-force-resisting system need be modeled. However, it shall be permissible to model nonparticipating elements of the structure if realistic assumptions are made with regard to their stiffness, strength and deformation capacity.

Commentary: Analyses of buildings for the purposes of demonstrating compliance with the strength and drift criteria of FEMA-302 must neglect the participation of gravity-load-carrying beams and columns that are not intended to be part of the lateral-force-resisting system. Studies conducted in support of the development of these Recommended Criteria indicate that these connections are capable of contributing non-negligible stiffness through large interstory drift demands. Analyses made with models that neglect the participation of these elements will tend to over-estimate demands on the lateral-force-resisting elements and interstory drift demand on the structure. The demand factors provided in Section 4.6 have been calibrated to account for this over-estimation.

While it is permissible to conduct performance evaluations using models that neglect nonparticipating framing, models that include the stiffness of these elements can be used to provide improved levels of confidence with regard to the building's ability to meet desired performance objectives. This is an example of the process by which confidence can be improved – by performing more intense study to reduce the inherent uncertainty.

4.5.2.2 Connection Modeling

4.5.2.2.1 Fully Restrained Moment-Resisting Connections

Elastic analysis models of structures with fully restrained connections should be based on the assumption that the connection provides a fully rigid interconnection between the beam and column, located at the centerline of the column. Alternatively, realistic assumptions with regard to panel zone flexibility may be made, as indicated in Section 4.5.2.3.

Nonlinear analysis models of structures with fully restrained connections should be based on the assumption that the connection provides a fully rigid interconnection between the beam and column, located at the centerline of the column, until either the connection panel zone, beam or column yields, or a total interstory drift angle q_{SD} (obtained from Table 4-12) occurs. The expected yield strength of the material, as indicated in Section 2.6.2 should be used to calculate the yield capacity of beams, columns, and panel zones. If yielding occurs at total interstory drift angles less than q_{SD}, the yielding element should be assumed to exhibit plastic behavior. At interstory drifts greater than q_{SD} the connection should be assumed to be capable of transmitting 20% of the expected plastic moment capacity of the girder until a total interstory drift angle q_U, (also obtained from Table 4-12) occurs. At interstory drift angles greater than q_U, the connection should be presumed to have negligible strength.

4.5.2.2.2 Partially Restrained Moment-Resisting Connections

Models of frames incorporating partially restrained connections should explicitly account for the stiffness of the connection. For linear models, connection stiffness may be modeled by incorporating a rotational spring element between the beam and column. Alternatively, a modified beam with partially restrained connections may be modeled as rigidly attached to columns, and using an effective modulus of rigidity, EI_{eq}, for the beam that accounts for the reduced stiffness introduced by the connection. For beams with similar partially restrained connections on each end, the effective modulus of rigidity may be calculated as:

$$EI_{eq} = \frac{1}{\frac{6h}{l_b^2 K_q} + \frac{1}{EI_b}} \tag{4-6}$$

where:

E = the modulus of elasticity, kip/ square inch

h = the average story height of the columns above and below the beam, inches
I_b = the moment of inertia of the beam, $(\text{inches})^4$
l_b = the beam span center to center of columns, inches
K_q = the stiffness of the connection, kip-in/radian

Refer to Section 3.7.1 for recommended connection stiffness for Double Split Tee partially restrained connections. Stiffness for other partially restrained connections should be based on laboratory data or rational analysis.

For nonlinear analysis, the connection should be explicitly modeled as an elastic-perfectly-plastic nonlinear spring with an elastic stiffness calculated as indicated above, and a plastic strength equal to the expected strength of the yield mode for the connection. Section 3.7.1 provides recommendations for determining the expected strength of the yield mode for Double Split Tee partially restrained connections. Expected strength of other types of partially restrained connections should be based on laboratory data or rational analysis. Partially restrained connections should be assumed to have negligible strength at interstory drift angle demands that exceed q_u, as indicated in Section 4.6.

4.5.2.2.3 Simple Shear Tab Connections

When included in linear analytical models the stiffness of simple shear tab connections should be explicitly modeled as a rotational spring that connects the beam to the column. The spring stiffness, K_q should be taken as:

$$K_q = 28000\left(d_{bg} - 5.6\right) \tag{4-7}$$

where d_{bg} is the bolt group depth in inches and K_q is in units of *k*-inches per radian. In lieu of explicit modeling of the connection, beams that frame into columns with simple shear tab connections may be modeled with an equivalent rigidity, EI_{eq} calculated in accordance with Equation 4-6, of Section 4.5.2.2.2.

When simple shear tab connections are included in nonlinear analysis models, they should be explicitly modeled as an elastic-perfectly-plastic rotational spring. The elastic stiffness of the spring should be taken as given by Equation 4-7. The plastic strength of the spring should be determined as the expected plastic moment capacity of the bolt group, calculated as the sum of the expected yield strength of the bolts and their distance from the neutral axis of the bolt group. The expected yield strength shall be taken as125% of the capacity of the bolt group determined in accordance with *AISC LRFD* using a resistance factor f of unity. Simple shear tab connections should be assumed to have negligible strength at interstory drift angle demands that exceed q_u, as indicated in Section 4.6.

4.5.2.3 Panel Zone Stiffness

It shall be permissible for the model to assume centerline-to-centerline dimensions for the purpose of calculating stiffness of beams and columns. Alternatively, more realistic assumptions

that account for the rigidity of panel zones may be used. Regardless, calculation of moments and shears should be performed at the face of the column.

Commentary: Models that use centerline-to-centerline dimensions for calculation of beam and column stiffness will tend to overestimate the interstory drift demand on the structure. The demand factors provided in Section 4.6 have been calibrated to account for this overestimation. While it is permissible to conduct performance evaluations using models that neglect the stiffening effect of the panel zone on beam and column stiffness, models that include more realistic estimation of this stiffness can be used to provide improved levels of confidence with regard to the building's ability to meet desired performance objectives.

A number of models are available to represent panel zones of moment-resisting connections. These range from simple models that treat the panel zone as a series of rigid links extending outward from the center of the beam-column connection and along the axes of the beams and columns to scissors-type models that explicitly account for the shear stiffness of the panel zone, to complex multi-element models that account both for shear stiffness of the panel zone and the effects of geometric distortion of the zone. Analyses of buildings using these various models, reported in FEMA-355C indicate that the particular model used has relatively little impact on the predicted interstory drift demand. However, for nonlinear analysis models, the element selected to represent the panel zone can have significant impact on where plasticity in the structure is predicted to occur, e.g., in the panel zone itself, within the beam, or a combination of these regions.

4.5.3 Horizontal Torsion

The effects of actual horizontal torsion must be considered. In *FEMA-302*, the total torsional moment at a given floor level includes the following two torsional moments:

a. the actual torsion, that is, the moment resulting from the eccentricity between the centers of mass at all floors above and including the given floor, and the center of rigidity of the vertical seismic elements in the story below the given floor, and

b. the accidental torsion, that is, an accidental torsional moment produced by horizontal offset in the centers of mass, at all floors above and including the given floor, equal to a minimum of 5% of the horizontal dimension at the given floor level measured perpendicular to the direction of the applied load.

For the purposes of performance evaluation, under these *Recommended Criteria*, accidental torsion should not be considered. In buildings with diaphragms that are not flexible, the effect of actual torsion should be considered if the maximum lateral displacement d_{max} from this effect at any point on any floor diaphragm exceeds the average displacement d_{avg} by more than 10%.

Commentary: Accidental torsion is an artificial device used by the building codes to account for actual torsion that can occur, but is not apparent in an evaluation

of the center of rigidity and center of mass in an elastic stiffness evaluation. Such torsion can develop during nonlinear response of the structure if yielding develops in an unsymmetrical manner in the structure. For example if the frames on the east and west sides of a structure have similar elastic stiffness the structure may not have significant torsion during elastic response. However, if the frame on the east side of the structure yields significantly sooner than the framing on the west side, then inelastic torsion will develop. Rather than requiring that an accidental torsion be applied in the analysis, as do the building codes, these Recommended Criteria directly account for the uncertainty related to these torsional effects in the calculation of demand and resistance factors. Accidental torsion should be applied in analyses applied to the design of frames, as required by FEMA-302.

4.5.4 Foundation Modeling

In general, foundations should be modeled as unyielding. Assumptions with regard to the extent of fixity against rotation provided at the base of columns should realistically account for the relative rigidities of the frame and foundation system, including soil compliance effects, and the detailing of the column base connections. For purposes of determining building period and dynamic properties, soil-structure interaction may be modeled, as permitted by the building code.

Commentary: Most steel moment frames can be adequately modeled by assuming that the foundation provides rigid support for vertical loads. However, the flexibility of foundation systems (and the attachment of columns to those systems) can significantly alter the flexural stiffness at the base of the frame. Where relevant, these factors should be considered in developing the analytical model.

4.5.5 Diaphragms

Floor and roof diaphragms transfer earthquake-induced inertial forces to vertical elements of the seismic framing system. Connections between floor and roof diaphragms and vertical seismic framing elements must have sufficient strength to transfer the maximum calculated diaphragm shear forces to the vertical framing elements. Requirements for evaluation of diaphragm components are given in Section 3.3 of *FEMA-273*.

Development of the mathematical model should reflect the stiffness of the diaphragm. As a general rule, most floor slabs with concrete fill over metal deck may be considered to be rigid diaphragms and floors or roofs with plywood diaphragms should be considered flexible. The flexibility of unfilled metal deck, and concrete slab diaphragms with large openings should be considered in the analytical model.

Mathematical models of buildings with diaphragms that are not rigid should be developed considering the effects of diaphragm flexibility. For buildings with flexible diaphragms at each floor level, the vertical lines of seismic framing may be designed independently, with seismic masses assigned on the basis of tributary area.

4.5.6 *P-D* Effects

P-D effects, caused by gravity loads acting on the displaced configuration of the structure, may be critical in the seismic performance of steel moment-frame buildings, which are usually flexible and may be subjected to large lateral displacements.

The structure should be evaluated for *P-D* effects in accordance with the requirements of Section 2.8.6 of these *Recommended Criteria*. Where the quantity Y_i in any story calculated in accordance with Section 2.8.6 exceeds 0.1, the increased deflections resulting from *P-D* effects must be determined. Where the quantity Y_i in any story exceeds 0.3, the interstory drift capacity of the structure must be determined in accordance with Appendix A of these *Recommended Criteria*.

> *Commentary: The values of interstory drift capacity for the Collapse Prevention performance level, provided in Section 4.6, and the corresponding resistance factors, were computed considering P-D effects (FEMA-355F). For a given structure, it is believed that if the value of Y is less than 0.3 the effects of P-D have been adequately considered by these general studies. For values of Y greater than this limit, the statistics on frame interstory drift capacities in Section 4.6 are inappropriate. For such frames explicit determination of interstory drift capacities by considering P-D effects, and by using the detailed performance evaluation procedures outlined in Appendix A is required.*

4.5.7 Multidirectional Excitation Effects

Buildings should be evaluated for response due to seismic forces in any horizontal direction. For regular buildings, seismic displacements and forces may be assumed to act nonconcurrently in the direction of each principal axis of a building. For buildings with plan irregularity and buildings in which one or more components form part of two or more intersecting elements, multidirectional excitation effects should be considered, as indicated in Section 4.4 for the various analytical procedures.

4.5.8 Vertical Ground Motion

The effects of vertical excitation on horizontal cantilevers should be considered by static or dynamic response methods. Vertical earthquake shaking may be characterized by a spectrum with ordinates equal to 2/3 of those of the horizontal spectrum unless alternative vertical response spectra are developed using site-specific analysis. Vertical earthquake effects on other beam elements and column elements need not be considered.

> *Commentary: There is no evidence that response to the vertical component of ground shaking has had any significant effect on the performance of steel moment-frame buildings. Consequently, the effect of this response is not recommended for consideration in performance evaluation, except as required by the building code for the case of horizontal cantilever elements.*

Traditionally, vertical response spectra, when considered, have been taken as 2/3 of the horizontal spectra developed for the site. While this is a reasonable approximation for most sites, vertical response spectra at near-field sites, located within a few kilometers of the zone of fault rupture, can have substantially stronger vertical response spectra than indicated by this approximation. Development of site-specific response spectra is recommended when vertical response must be considered for buildings on such sites.

4.6 Acceptance Criteria

Acceptability of building performance should be evaluated by determining a level of confidence in the building's ability to meet the desired performance objective(s). The parameters in Table 4-5 must be independently evaluated, using the procedures of Section 4.6.1 and the parameters and acceptance criteria of Sections 4.6.2, 4.6.3 and 4.6.4, for each performance objective evaluated. The controlling parameter is that which results in the calculation of the lowest confidence for building performance.

Table 4-5 Performance Parameters Requiring Evaluation of Confidence

Parameter	Discussion
Interstory Drift	The maximum interstory drift computed for any story of the structure shall be evaluated for global and local behaviors (for Collapse Prevention and Immediate Occupancy). Refer to Section 4.6.2.
Column Axial Load	The adequacy of each column to withstand its calculated maximum compressive demand shall be evaluated both for Collapse Prevention and Immediate Occupancy. Refer to Section 4.6.3.
Column Splice Tension	The adequacy of column splices to withstand their calculated maximum tensile demands shall be evaluated both for Collapse Prevention and Immediate Occupancy. Refer to Section 4.6.4.

4.6.1 Factored-Demand-to-Capacity Ratio

Confidence level is determined through evaluation of the factored-demand-to-capacity ratio given by the equation:

$$l = \frac{g\, g_a D}{f C} \tag{4-8}$$

where:

C = capacity of the structure, as indicated in sections 4.6.2, 4.6.3, and 4.6.4, for interstory drift demand, column compressive demand and column splice tensile demand, respectively,

D = calculated demand for the structure, obtained from structural analysis,

g = a demand variability factor that accounts for the variability inherent in the prediction of demand related to assumptions made in structural modeling and prediction of the character of ground shaking as indicated in Sections 4.6.2, 4.6.3, and 4.6.4, for

interstory drift demand, column compressive demand and column tensile demand, respectively,

g_a = an analysis uncertainty factor that accounts for bias and uncertainty, inherent in the specific analytical procedure used to estimate demand as a function of ground shaking intensity as indicated in Section 4.6.2, 4.6.3 and 4.6.4, for interstory drift demand, column compressive demand and column tensile demand, respectively,

f = a resistance factor that accounts for the uncertainty and variability, inherent in the prediction of structural capacity as a function of ground shaking intensity, as indicated in Section 4.6.2, 4.6.3 and 4.6.4 for interstory drift demand, column compressive demand, and column tensile demand, respectively, and

l = a confidence index parameter from which a level of confidence can be obtained. See Table 4-7.

Factored demand to capacity ratio l shall be calculated using Equation 4-8 for each of the performance parameters indicated in Table 4-5, which also references the appropriate Section of this document where the various parameters, g_a, g, f required to perform this evaluation may be found. These referenced sections also define an uncertainty parameter b_{UT} associated with the evaluation of global and local interstory drift capacity, column compressive capacity, and column splice tensile capacity, respectively. These uncertainties are related to the building's configuration, the structural framing system (Ordinary Moment Frame or Special Moment Frame), the type of analytical procedure employed and the performance level being evaluated. Table 4-6 indicates the level of confidence associated with various values of the factored demand to capacity ratio l calculated using Equation 4-8, for various values of the uncertainty parameter b_{UT}. Linear interpolation between the values given in Table 4-6 may be used for values of factored demand to capacity ratio l and uncertainty b_{UT} intermediate to those tabulated.

Table 4-7 provides minimum recommended levels of confidence for each of the potential controlling behavior modes, that is, global stability, local connection capacity, column buckling or column splice tensile failure, for the Immediate Occupancy and Collapse Prevention performance levels, respectively.

Commentary: In order to predict structural performance, these procedures rely on the application of structural analysis and laboratory test data to predict the behavior of real structures. However, there are a number of sources of uncertainty inherent in the application of analysis and test data to performance prediction. For example, the actual strength of structural materials, the quality of individual welded joints, and the amount of viscous damping present is never precisely known, but can have impact on both the actual amount of demand produced on the structure and its elements and their capacity to resist these demands. If the actual values of these and other parameters that affect structural performance were known, it would be possible to accurately predict both demand and capacity. However, this is never the case. In these procedures, confidence is used as a measure of the extent that predicted behavior, based on assumed conditions, is likely to represent reality.

Table 4-6 Confidence Levels for Various Values of l, Given b_{UT}

Confidence Level	10	20	30	40	50	60	70	80	90	95	99
b_{UT} = 0.2											
l	1.37	1.26	1.18	1.12	1.06	1.01	0.96	0.90	0.82	0.76	0.67
b_{UT} = 0.3											
l	1.68	1.48	1.34	1.23	1.14	1.06	0.98	0.89	0.78	0.70	0.57
b_{UT} = 0.4											
l	2.12	1.79	1.57	1.40	1.27	1.15	1.03	0.90	0.76	0.66	0.51
b_{UT} = 0.5											
l	2.76	2.23	1.90	1.65	1.45	1.28	1.12	0.95	0.77	0.64	0.46
b_{UT} = 0.6											
l	3.70	2.86	2.36	1.99	1.72	1.48	1.25	1.03	0.80	0.64	0.43

Table 4-7 Recommended Minimum Confidence Levels

Behavior	Performance Level	
	Immediate Occupancy	Collapse Prevention
Global Behavior Limited by Interstory Drift	50%	90%
Local Connection Behavior Limited by Interstory Drift	50%	50%
Column Compression Behavior	50%	90%
Column Splice Tension Behavior	50%	50%

The extent of confidence inherent in a performance prediction is related to the possible variation in the several factors that affect structural demand and capacity, such as stiffness, damping, connection quality, and the analytical procedures employed. In this project, evaluations were made of the potential distribution of each of these factors and the effect of variation in these factors on the calculated value of structural demand and capacity. Each of these sources of uncertainty in structural demand and capacity prediction were characterized as part of the supporting research for this project, by a coefficient of variation, b_U. The coefficient, b_{UT} is the total coefficient of variation, considering all sources of uncertainty. It is used, together with other factors to calculate the demand and resistance factors. This assumes that demand and capacity are lognormally distributed relative to these uncertain parameters. This allows confidence to be calculated as a function of the number of standard deviations that factored demand-to-capacity-ratio, l, lies above or below a mean value. Table 4-6 provides a solution for this calculation, using a conservative estimate of the

hazard parameter, k=3.0, that is representative of the typical seismicity of coastal California. Further information on this method may be found in Appendix A. Appendix A also provides values of l that are more appropriate to other conditions of seismicity, and that may be used to provide increased estimates of confidence in other regions.

4.6.2 Performance Limited by Interstory Drift Angle

4.6.2.1 Factored Interstory Drift Angle Demand

Factored interstory drift demand should be computed as the quantity, $g g_a D$ where the demand D is the largest interstory drift computed from structural analysis, g_a is the coefficient obtained from Table 4-8 and g is the coefficient obtained from Table 4-9.

Commentary: Several structural response parameters are used to evaluate structural performance. The primary parameter is interstory drift. Interstory drift is an excellent parameter for judging the ability of a structure to resist P-D instability and collapse. It is also closely related to plastic rotation demand, or drift angle demand, on individual beam-column connection assemblies, and is therefore a good predictor of the performance of beams, columns and connections. For tall slender structures, a significant portion of interstory drift is a result of axial elongation and shortening of the columns. Although modeling of the structure should account for this frame flexibility, that portion of interstory drift resulting from axial column deformation in stories below the story under consideration may be neglected in determining local connection performance. This portion of the drift must be determined manually as most computer programs do not calculate this quantity separately.

Table 4-8 Interstory Drift Angle Analysis Uncertainty Factors g_a

Analysis Procedure	LSP		LDP		NSP		NDP	
System Characteristic	I.O.[1]	C.P.[2]	I.O.[1]	C.P.[2]	I.O.[1]	C.P.[2]	I.O.[1]	C.P.[2]
Special Moment Frames								
Low Rise (<4 stories)	0.94	0.70	1.03	0.83	1.13	0.89	1.02	1.03
Mid Rise (4-12 stories)	1.15	0.97	1.14	1.25	1.45	0.99	1.02	1.06
High Rise (> 12 stories)	1.12	1.21	1.21	1.14	1.36	0.95	1.04	1.10
Ordinary Moment Frames								
Low Rise (<4 stories)	0.79	0.98	1.04	1.32	0.95	1.31	1.02	1.03
Mid Rise (4-12 stories)	0.85	1.14	1.10	1.53	1.11	1.42	1.02	1.06
High Rise (> 12 stories)	0.80	0.85	1.39	1.38	1.36	1.53	1.04	1.10

Notes: 1. I.O. = Immediate Occupancy Performance Level
2. C.P. = Collapse Prevention Performance Level

Table 4-9 Interstory Drift Angle Demand Variability Factors g

Building Height	g	
	Immediate Occupancy (I.O.)	**Collapse Prevention (C.P.)**
Special Moment Frame		
Low Rise (1 - 3 stories)	1.5	1.3
Mid Rise (4 - 12 stories)	1.4	1.2
High Rise (> 12 stories)	1.4	1.5
Ordinary Moment Frame		
Low Rise (1 - 3 stories)	1.4	1.4
Mid Rise (4 - 12 stories)	1.3	1.5
High Rise (> 12 stories)	1.6	1.8

4.6.2.2 Factored Interstory Drift Angle Capacity

Interstory drift capacity may be limited either by the global response of the building, or by the local behavior of beam-column connections. Section 4.6.2.2.1 provides values for global interstory drift capacity for regular, well-configured structures as well as associated uncertainties, b_{UT}. Global interstory drift capacities for irregular structures must be determined using the detailed procedures of Appendix A. Section 4.6.2.2.2 provides values for local interstory drift capacity of prequalified Special Moment Frame (SMF) and Ordinary Moment Frame (OMF) connections. Local interstory drift capacities for connections that are not prequalified in Chapter 3 of these *Recommended Criteria* must be determined in accordance with the detailed procedures of Appendix A.

4.6.2.2.1 Global Interstory Drift Angle

Factored interstory drift angle capacity fC as limited by global response of the building, shall be based on the product of the resistance factor f and capacity C, obtained from Table 4-10. Table 4-11 provides values of the uncertainty coefficient b_{UT} to be used with global interstory drift evaluation.

4.6.2.2.2 Local Interstory Drift Angle

Factored interstory drift angle capacity fC, limited by local connection response, shall be based on the capacity C of the connection and the resistance factor f taken from Table 4-12 for prequalified connections. In Table 4-12 the capacity C for Collapse Prevention is the interstory

drift angle q_U while capacity C for Immediate Occupancy is the interstory drift angle q_{IO}. For connection types not contained in the Table 4-12, the more detailed procedures of Appendix A should be used to determine interstory drift angle capacity. Table 4-13 provides values of the uncertainty coefficient b_{UT} to be used with local interstory drift evaluation.

Table 4-10 Global Interstory Drift Angle Capacity C and Resistance Factors f for Regular SMF and OMF Buildings

Building Height	Performance Level			
	Immediate Occupancy		Collapse Prevention	
	Interstory Drift Angle Capacity C	Resistance Factor f	Interstory Drift Angle Capacity C	Resistance Factor f
Special Moment Frames (SMF)				
Low Rise (3 stories or less)	0.02	1.0	0.10	0.90
Mid Rise (4 – 12 stories)	0.02	1.0	0.10	0.85
High Rise (> 12 stories)	0.02	1.0	0.085	0.75
Ordinary Moment Frames (OMF)				
Low Rise (3 stories or less)	0.01	1.0	0.10	0.85
Mid Rise (4 – 12 stories)	0.01	0.9	0.08	0.70
High Rise (> 12 stories)	0.01	0.85	0.06	0.60

Table 4-11 Uncertainty Coefficient b_{UT} for Global Interstory Drift Evaluation

Building Height	Performance Level	
	Immediate Occupancy	Collapse Prevention
Special Moment Frames (SMF)		
Low Rise (3 stories or less)	0.20	0.3
Mid Rise (4 – 12 stories)	0.20	0.4
High Rise (> 12 stories)	0.20	0.5
Ordinary Moment Frames (OMF)		
Low Rise (3 stories or less)	0.20	0.35
Mid Rise (4 – 12 stories)	0.20	0.45
High Rise (> 12 stories)	0.20	0.55

Notes: 1- Value of b_{UT} should be increased by 0.05 for linear static analysis
2- Value of b_{UT} may be reduced by 0.05 for nonlinear dynamic analysis

Commentary: Table 4-12 presents median drift angle capacities C indicated by q_{IO}, q_{SD}, and q_U and resistance factors f for connections that are prequalified under Chapter 3 of these Recommended Criteria. These values were determined from cyclic tests of full-size connection assemblies using the testing protocols indicated in Chapter 3. The cyclic tests are used to determine the load-

deformation hysteresis behavior of the system and to determine the statistics (medians and standard deviations) for the connection drift angle at which the following behaviors occur:

1. *onset of local flange buckling of beams,*
2. *degradation of moment-resisting capacity of the assembly to a value below the nominal moment-resisting capacity,*
3. *initiation of fracture of bolts, welds, or base metal that results in significant strength degradation of the assembly,*
4. *complete failure of the connection, characterized by an inability of the connection to maintain integrity of the beam-to-column connection under gravity loading.*

Based on these data, limiting drift angle capacities have been obtained for the Immediate Occupancy and Collapse Prevention damage states, as indicated in Table 4-14.

Table 4-12 Drift Angle Capacity *C* (q_{IO}, q_U) for Prequalified Connections as Limited By Local Connection Response

Connection Type	Strength Degradation Limit Drift Angle (radians) q_{SD}	Immediate Occupancy		Collapse Prevention	
		Limit Drift Angle (radians) q_{IO}	Capacity Reduction Factor f	Limit Drift Angle (radians) q_U	Capacity Reduction Factor f
WUF-B	0.031-0.0003d_b	0.015	0.9	0.060-0.006d_b	0.9
WUF-W	0.051	0.020	0.9	0.064	0.9
FF	0.061-0.00064d_b	0.020	0.9	0.080-0.00064d_b	0.9
RBS	0.060-0.0003 d_b	0.020	0.9	0.080-0.0003 d_b	0.9
WFP	0.04	0.020	0.9	0.07	0.9
BUEP	0.071-0.0013 d_b	.015	0.9	0.081-0.0013 d_b	0.9
BSEP	0.071-0.0013 d_b	.015	0.9	0.081-0.0013 d_b	0.9
BFP	0.12-0.002 d_b	.015	0.9	0.10-0.001 d_b	0.9
DST	0.12-0.0032 d_b	.015	0.9	0.14-0.0032 d_b	0.9

Note: d_b is the beam depth, inches

The capacity values indicated in Table 4-14 corresponding to collapse prevention behavior, have been conservatively established, based generally on engineering judgement, as few of the laboratory tests conducted actually loaded the connections to the point where failure to provide gravity-load resistance occurred. Accordingly, these

values should be used with caution. For more information refer to FEMA-355D State of the Art Report on Connection Performance.

Tables 4-12 and 4-14 also present data on a drift angle capacity parameter indicated as q_{SD}. This is the drift angle at which strength degradation initiates in the connection assembly, which may be controlled by connection degradation (for example, fracture of a weld or bolt) or framing behavior (such as buckling of the beam). The value of q_{SD} is not directly used as a capacity index for evaluating performance. However, it is a critical parameter for determining whether a connection qualifies as a Special or Ordinary Moment Frame. Refer to Chapter 3 and Appendix A for additional information on this topic.

Table 4-13 Uncertainty Coefficient b_{UT} for Local Interstory Drift Evaluation

Building Height	Performance Level	
	Immediate Occupancy	Collapse Prevention
Special Moment Frames (SMF)		
Low Rise (3 stories or less)	0.30	0.30
Mid Rise (4 – 12 stories)	0.30	0.35
High Rise (> 12 stories)	0.30	0.40
Ordinary Moment Frames (OMF)		
Low Rise (3 stories or less)	0.30	0.35
Mid Rise (4 – 12 stories)	0.30	0.40
High Rise (> 12 stories)	0.30	0.40

Notes: 1. Value of b_{UT} should be increased by 0.05 for linear static analysis
2. Value of b_{UT} may be reduced by 0.05 for nonlinear dynamic analysis

Table 4-14 Behavior States for Performance Evaluation of Connection Assemblies

Drift Angle	Performance Level	Description
q_{IO}	Immediate Occupancy	The lowest drift angle at which any of behaviors 1, 2, or 3, above, occur.
q_{CP}	Collapse Prevention	The drift angle at which behavior 4 occurs
q_{SD}	-	The lowest drift angle at which any of behaviors 2, 3, or 4 occur

Note: The description involves the behavior list in the Commentary to Section 4.6.2.2.2.

4.6.3 Performance Limited by Column Compressive Capacity

4.6.3.1 Column Compressive Demand

The factored column compressive demand shall be determined for each column as the quantity $g g_a D$, where:

D = the compressive axial load on the column determined as the sum of the Dead Load, 25% of the unreduced Live Load, and the Seismic Demand. Seismic demand shall be determined by either of the following four analysis methods:

Linear: The axial demands may be those predicted by a linear static or linear dynamic analysis, conducted in accordance with Section 4 of these *Recommended Criteria*.

Plastic: The axial seismic demands may be based on plastic analysis, as indicated in Equation 4-5 of Section 4.4.3.3.6 of these *Recommended Criteria*.

Nonlinear Static: The axial demands may be based on the computed forces from a nonlinear static analysis, at the target displacement, in accordance with Section 4 of these *Recommended Criteria*.

Nonlinear Dynamic: The axial demands may be based on the computed design forces from a nonlinear dynamic analysis, in accordance with Section 4 of these *Recommended Criteria*.

g_a = Analysis uncertainty factor, taken from Table 4-15.

g = Demand variability factor, taken as having the constant value 1.05.

The uncertainty coefficient b_{UT} shall be taken as indicated in Table 4-15 based on the procedure used to calculate column compressive demand D.

Table 4-15 Analysis Uncertainty Factor g_a and Total Uncertainty Coefficient b_{UT} for Evaluation of Column Compressive Demands

Analytical Procedure	Analysis Uncertainty Factor g_a	Total Uncertainty Coefficient b_{UT}
Linear Static or Dynamic Analysis	1.15	0.35
Plastic Analysis (Section 4.4.3.3.6)	1.0	0.15
Nonlinear Static Analysis	1.05	0.20
Nonlinear Dynamic Analysis	$e^{1.4\,b^2}$	$\sqrt{0.0225 + b^2}$

Note: b may be taken as the coefficient of variation (COV) of the axial load values determined from the suite of nonlinear analyses.

Commentary: The value of g has been computed assuming a coefficient of variation for axial load values resulting from material strength variation and uncertainty in dead and live loads of 15%. The values of g_a have been calculated assuming coefficients of variation of 30%, 0% and 15% related to uncertainty in the analysis procedures for linear, plastic and nonlinear static analyses, respectively. In reality, for structures that are stressed into the inelastic range,

elastic analysis will typically overestimate axial column demands, in which case, a value of 1.0 could be used for g_a (0% coefficient of variation). However, for structures that are not loaded into the inelastic range, the indicated value is appropriate. Plastic analysis will also typically result in an upper-bound estimate of column demand, and application of additional demand factors is not appropriate. For nonlinear dynamic analysis, using a suite of ground motions, direct calculation of the analysis demand factor is possible, using the equation shown. All of these demand factors are based on the hazard parameter k having a value of 3.0, typical of conditions in coastal California.

4.6.3.2 Column Compressive Capacity

Factored compressive capacity of each column to resist compressive axial loads, shall be determined as the product of the resistance factor f and the nominal axial strength *C* of the column, which shall be determined in accordance with the *AISC Load and Resistance Factor Design Specification*. For the purposes of this evaluation, the effective length coefficient *k* shall be taken as having a value of 1.0 and the resistance factor f shall be assigned a value of 0.95.

4.6.4 Column Splice Capacity

The capacity of column tensile splices, other than splices consisting of complete joint penetration (CJP) groove welds of all elements of the column (flanges and webs) shall be evaluated in accordance with this section. Column splices consisting of CJP welds of all elements of the column, and in which the weld filler metal conforms to the recommendations of Section 3.3.2.4 of these *Recommended Criteria* need not be evaluated.

Commentary: Welded splices in which the flanges and welds of the butting sections are joined with CJP groove welds and in which weld access holes are provided in accordance with AISC requirements qualify as having all elements joined by CJP groove welds.

4.6.4.1 Column Splice Tensile Demand

Factored column splice tensile demand shall be determined for each column as the quantity $g g_a D$ where *D* is the column splice tensile demand. Column splice tensile demand shall be determined as the computed Seismic Demand in the column less 90% of the computed Dead Load demand. Seismic Demand shall be as determined for column compressive demand, in accordance with Section 3.6.3.1. The demand variability factor *g* shall be taken as having a value of 1.05 and the analysis uncertainty factor g_a shall be taken as indicated in Table 4-15. The total uncertainty coefficient b_{UT} shall also be taken as indicated in Table 4-15.

4.6.4.2 Column Splice Tensile Capacity

The capacity of individual column splices to resist tensile axial loads shall be determined as the product of the resistance factor f and the nominal tensile strength *C* of the splice, as determined in accordance with the *AISC Load and Resistance Factor Design Specification*.

Specifically, Chapter J therein shall be used to calculate the nominal tensile strength of the splice connection. For the purposes of this evaluation, f shall be assigned a value of 0.9.

A. DETAILED PROCEDURES FOR PERFORMANCE EVALUATION

A.1 Scope

This appendix provides detailed procedures for evaluating the performance capability of steel moment-frame buildings. These detailed procedures are provided as a supplement to the simplified performance evaluation procedures in Chapter 4. They may be used to demonstrate enhanced levels of confidence with regard to the ability of a particular building to meet desired performance objectives, relative to the confidence levels that may be derived using the more simplified procedures, and they must be used instead of the procedures of Chapter 4, for irregular structures and for structures with connections that have not been prequalified. This appendix also provides criteria for performance evaluation for deterministically defined hazards.

Commentary: Chapter 4 provides procedures for a simplified method of performance evaluation, using factored-demand-to-capacity ratios to determine a level of confidence with regard to a building's ability to provide a desired performance objective. The tabular values of demand and resistance factors and confidence indices contained in Chapter 4 were derived using the procedures presented in this appendix, applied to the performance evaluation of a suite of regularly configured model buildings. Since this suite of model buildings is not completely representative of any individual structure, the use of the tabular values inherently entails some uncertainty, and thus reduced levels of confidence, with regard to performance prediction. The detailed procedures in this appendix permit reduction in these uncertainties, and therefore enhanced confidence, with regard to prediction of building performance. These more detailed procedures must be used for those irregular building configurations not well represented by the model buildings used as the basis for the values contained in Chapter 4.

A.2 Performance Evaluation Approach

A.2.1 Performance Objectives and Confidence

As defined in Section 4.2 of these *Recommended Criteria*, performance is defined in terms of probabilistic performance objectives. A performance objective consists of the specification of a performance level and an acceptable low probability that poorer performance could occur within a specific period of time, typically taken as 50 years. Alternatively, deterministic performance objectives can also be evaluated. Deterministic performance objectives consist of the specification of a performance level and a specific earthquake, that is, fault location and magnitude, for which this performance is to be attained.

Two performance levels are defined: the Immediate Occupancy performance level and the Collapse Prevention performance level. Detailed descriptions of these performance levels may be found in Chapter 4. The evaluation procedures contained in this appendix permit estimation of a level of confidence associated with achievement of a performance objective. For example, a design may be determined to provide a 95% level of confidence that there is less than a 2% probability in 50 years of more severe damage than represented by the Collapse Prevention level.

For another example, a design may be determined to provide a 50% level of confidence that the structure will provide Immediate Occupancy performance, or a better performance, for a Richter magnitude 6 earthquake along a defined fault.

Commentary: The probability that a building may experience damage more severe than that defined for a given performance level is a function of two principal factors. The first of these is the structure's vulnerability, that is, the probability that it will experience certain levels of damage given that it experiences ground motion of certain intensity. The second of these factors is the site hazard, that is, the probability that ground shaking of varying intensities may occur in a given time period. The probability that damage exceeding a given performance level may occur in a period of time is calculated as the integral over time of the probability that damage will exceed that permitted within a performance level. Mathematically, this may be expressed as:

$$P(D > PL) = \int P_{D>PL}(x)h(x)dx \qquad \textit{(A-1)}$$

where:

$P(D>PL)$ = *Probability of damage exceeding a performance level in a period of t years*

$P_{D>PL}(x)$ = *Probability of damage exceeding a performance level given that the ground motion intensity is level x, as a function of x,*

$h(x)dx$ = *probability of experiencing a ground motion intensity of level (x) to (x + dx) in a period of t years*

Vulnerability may be thought of as the capacity of the structure to resist greater damage than that defining a performance level. Structural response parameters that may be used to measure capacity include the structure's ability to undergo global building drift, maximum tolerable member forces, and maximum tolerable inelastic deformations. Ground accelerations associated with the seismic hazard, and the resulting enforced global building drift, member forces and inelastic deformations produced by the hazard may be thought of as demands. If both the demand that a structure will experience over a period of time and the structure's capacity to resist this demand could be perfectly defined, then performance objectives, the probability that damage may exceed a performance level within a period of time, could be ascertained with 100% confidence. However, the process of predicting the capacity of a structure to resist ground shaking demands as well as the process of predicting the severity of demands that will actually be experienced entail significant uncertainties. Confidence level is a measure of the extent of uncertainty inherent in this process. A level of 100% confidence may be described as perfect confidence. In reality, it is never possible to attain such

confidence. Confidence levels on the order of 90 or 95% are considered high, while confidence levels less than 50% are considered low.

Generally, uncertainty can be reduced, and confidence increased, by obtaining better knowledge or using better procedures. For example, enhanced understanding and reduced uncertainty with regard to the prediction of the effects of ground shaking on a structure can be obtained by using a more accurate analytical procedure to predict the structure's response. Enhanced understanding of the capacity of a structure to resist ground shaking demands can be obtained by obtaining specific laboratory data on the physical properties of the materials of construction and on the damageability of individual beam-column connection assemblies.

The simplified performance evaluation procedures of Chapter 4 are based on the typical characteristics of standard buildings. Consequently, they incorporate significant uncertainty in the performance prediction process. As a result of this significant uncertainty, it is anticipated that the actual ability of a structure to achieve a given performance objective may be significantly better than would be indicated by those simple procedures. The more detailed procedures of this appendix may be used to improve the definition of the actual uncertainties incorporated in the prediction of performance for a specific structure and thereby to obtain better confidence with regard to the prediction of performance for an individual structure.

As an example, using the simplified procedures of Chapter 4, it may be found that for a specific structure, there is only a 50% level of confidence that there is less than a 10% chance in 50 years of poorer performance than the Collapse Prevention level. This rather low level of confidence may be more a function of the uncertainty inherent in the simplified procedures than the actual inadequate capacity of the building to provide Collapse Prevention performance. In such a case, it may be possible to use the procedures contained in this appendix to reduce the uncertainty inherent in the performance estimation and find that instead, there may be as much as a 95% level of confidence in obtaining such performance.

In both the procedures of this appendix and Chapter 4, the uncertainties associated with estimation of the intensity of ground motion have been neglected. These uncertainties can be quite high, on the order of those associated with structural performance or even higher. Thus, the confidence estimated using these procedures is really a confidence with regard to structural performance, given the seismicity as portrayed by the USGS hazard maps that accompany FEMA-273 and FEMA-302.

A.2.2 Basic Procedure

As indicated in Chapter 4, a demand and resistance factor design (DRFD) format is used to associate a level of confidence with the probability that a building will have less than a specified probability of exceedance of a desired performance level. The basic approach is to determine a confidence parameter, l, which may then be used, with reference to Table A-1, to determine the confidence level that exists with regard to performance estimation. The confidence parameter, l, is determined from the factored-demand-to-capacity equation:

$$l = \frac{g\, g_a D}{f C} \tag{A-2}$$

where:

C = median estimate of the capacity of the structure. This estimate may be obtained either by reference to default values contained in Chapter 4, or by more rigorous direct calculation of capacity using the procedures of this appendix,

D = calculated demand on the structure, obtained from a structural analysis,

g = a demand variability factor that accounts for the variability inherent in the prediction of demand related to assumptions made in structural modeling and prediction of the character of ground shaking,

g_a = an analysis uncertainty factor that accounts for the bias and uncertainty associated with the specific analytical procedure used to estimate structural demand as a function of ground shaking intensity,

f = a resistance factor that accounts for the uncertainty and variability inherent in the prediction of structural capacity as a function of ground shaking intensity,

l = a confidence index parameter from which a level of confidence can be obtained by reference to Table A-1.

Several structural response parameters are used to evaluate structural performance. The primary parameter used for this purpose is interstory drift. Interstory drift is an excellent parameter for judging the ability of a structure to resist *P-D* instability and collapse. It is also closely related to plastic rotation demand, or drift angle demand, on individual beam-column connection assemblies, and therefore a good predictor of the performance of beams, columns and connections. Other parameters used in these guidelines include column axial compression and column axial tension. In order to determine a level of confidence with regard to the probability that a building has less than a specified probability of exceeding a performance level over a period of time, the following steps are followed:

1. **The performance objective to be evaluated is selected**. This requires selection of a performance level of interest, for example, Collapse Prevention or Immediate Occupancy,

and a desired probability that damage in a period of time will be worse than this performance level. Representative performance objectives may include:

- 2% probability of poorer performance than Collapse Prevention level in 50 years
- 50% probability of poorer performance than Immediate Occupancy level in 50 years.

It is also possible to express performance objectives in a deterministic manner, where attainment of the performance is conditioned on the occurrence of a specific magnitude earthquake on an identified fault.

2. **Characteristic motion for the performance objective is determined.** For probabilistic performance objectives, an average estimate of the ground shaking intensity at the probability of exceedance identified in the performance objective definition (step 1) is determined. For example, if the performance objective is a 2% probability of poorer performance than Collapse Prevention level in 50 years, then an average estimate of ground shaking demands with a 2% probability of exceedance in 50 years would be determined. Ground shaking intensity is characterized by the parameter S_{aT1}, the 5% damped spectral response acceleration at the site for the fundamental period of response of the structure. *FEMA-273* provides procedures for determining this parameter for any probability of exceedance in a 50-year period.

   For deterministic performance objectives, an average estimate of the ground motion at the building site for the specific earthquake magnitude and fault location must be made. As with probabilistic estimates, the motion is characterized by S_{aT1}.

3. **Structural demands for the characteristic earthquake ground motion are determined.** A mathematical structural model is developed to represent the building structure. This model is then subjected to a structural analysis, using any of the methods contained in Chapter 4. This analysis provides estimates of maximum interstory drift demand, maximum column compressive demand, and maximum column-splice tensile demand, for the ground motion determined in step 2.

4. **Median estimates of structural capacity are determined**. Median estimates of the interstory drift capacity of the moment-resisting connections and the building frame as a whole are determined, as are median estimates of column compressive capacity and column-splice tensile capacity. Interstory drift capacity for the building frame, as a whole, may be estimated using the default values of Chapter 4 for regular structures, or alternatively, the detailed procedures of Section A.6 may be used. These detailed procedures are mandatory for irregular structures. Interstory drift capacity for moment-resisting connections that are prequalified in Chapter 3 of these *Recommended Criteria* may be estimated using the default values of Chapter 4, or alternatively, direct laboratory data on beam-column connection assembly performance capability and the procedures of Section A.5 of this appendix may be used. Median estimates of column compressive capacity and column-splice tensile capacity are made using the procedures of Chapter 4.

5. **A factored-demand-to-capacity ratio,** l **is determined**. For each of the performance parameters, i.e., interstory drift as related to global building frame performance, interstory drift as related to connection performance, column compression, and column splice tension,

Equation A-2 is independently applied to determine the value of the confidence parameter l. In each case, the calculated estimates of demand *D* and capacity *C* are determined using steps 3 and 4, respectively. If the procedures of Chapter 4 are used to determine either demand or median capacity estimates, then the corresponding values of the demand factors g and resistance factors f should also be determined in accordance with the procedures of Chapter 4. If the procedures of this appendix are used to determine median demand, or capacity, then the corresponding demand and resistance factors should be determined in accordance with the applicable procedures of this appendix.

6. **Evaluate confidence**. The confidence obtained with regard to the ability of the structure to meet the performance objective is determined using the lowest of the l values determined in accordance with step 5 above, back-calculated from the equation:

$$l = e^{-b b_{UT}(K_X - k b_{UT}/2)} \tag{A-3}$$

where:

b = a coefficient relating the incremental change in demand (drift, force, or deformation) to an incremental change in ground shaking intensity, at the hazard level of interest, typically taken as having a value of 1.0,

b_{UT} = an uncertainty measure equal to the vector sum of the logarithmic standard deviation of the variations in demand and capacity resulting from uncertainty,

k = the slope of the hazard curve, in ln-ln coordinates, at the hazard level of interest, i.e., the ratio of incremental change in S_{aT1} to incremental change in annual probability of exceedance (refer to Section A.3.2),

K_X = standard Gaussian variate associated with probability x of not being exceeded as a function of number of standard deviations above or below the mean found in standard probability tables.

Table A-1 provides a solution for this equation, for various values of the parameters, k, l, and b_{UT}.

The values of the parameter b_{UT} used in Equation A-3 and Table A-1 are used to account for the uncertainties inherent in the estimation of demands and capacities. Uncertainty enters the process through a variety of assumptions that are made in the performance evaluation process, including, for example, assumed values of damping, structural period, properties used in structural modeling, and strengths of materials. Assuming that the amount of uncertainty introduced by each of the assumptions can be characterized, the parameter b_{UT} can be calculated using the equation:

$$b_{UT} = \sqrt{\sum_i b_{ui}^2} \tag{A-4}$$

where: b_{ui} are the standard deviations of the natural logarithms of the variation in demand or capacity resulting from each of these various sources of uncertainty. Sections A.4, A.5 and A.6 indicate how to determine b_{ui} values associated with demand estimation, beam-column connection assembly behavior, and building global stability capacity prediction, respectively.

A.3 Determination of Hazard Parameters

Two basic hazard parameters are required by these performance evaluation procedures. The first of these, S_{aT1}, is the median, 5%-damped, linear spectral response acceleration, at the fundamental period of the building, at the desired hazard level (probability of exceedance in a 50-year period or specific earthquake magnitude and fault). Section A.3.1 provides guidelines for obtaining this parameter. The second parameter is the slope *k* of the hazard curve in logarithmic space, also evaluated at the desired hazard level. Section A.3.2 provides guidelines for obtaining this parameter.

A.3.1 Spectral Response Acceleration

Probabilistic, 5%-damped, linear spectral response acceleration, S_{aT1} at the fundamental period of the building, at the desired hazard level (probability of exceedance in a 50-year period), may be determined in several different ways. These include:

a. Site-specific seismological and geotechnical investigation. *FEMA-273* provides guidelines for this method.

b. Use of national hazard maps developed by the United States Geologic Survey. *FEMA-273* also provides guidelines for the use of these maps for this purpose.

Deterministic 5%-damped, linear spectral response acceleration S_{aT1} at the fundamental period of the building shall be determined based on site-specific seismological and geologic study.

The spectral response acceleration S_{aT1} is used as a reference point, through which a response spectrum is plotted. This response spectrum may be used directly in the structural analysis, or alternatively, may be used as a basis for the development of ground motion accelerograms used in the structural analysis. Refer to Chapter 4 for guidelines on analysis.

A.3.2 Logarithmic Hazard Curve Slope

In these procedures, the logarithmic slope *k* of the hazard curve at the desired hazard level is used to determine the resistance factors, demand factors and also the confidence levels. The hazard curve is a plot of probability of exceedance of a spectral amplitude versus that spectral amplitude, for a given period, and is usually plotted on a log-log scale. In functional form it can be represented by the equation:

$$H_{Si}(S_i) = k_0 S_i^{-k} \tag{A-5}$$

where:

$H_{Si}(S_i)$ = the probability of ground shaking having a spectral response acceleration greater than S_i,
k_0 = a constant, dependent on the seismicity of the individual site,
k = the logarithmic slope of the hazard curve.

Table A-1 Confidence Parameter λ, as a Function of Confidence Level, Hazard Parameter k, and Uncertainty β_{UT}

Confidence Level	2%	5%	10%	20%	30%	40%	50%	60%	70%	80%	90%	95%	99%
$\beta_{UT} = 0.1$													
k=1	1.24	1.19	1.14	1.09	1.06	1.03	1.0	0.98	0.95	0.92	0.88	0.85	0.80
k=2	1.24	1.19	1.15	1.10	1.07	1.04	1.01	0.99	0.96	0.93	0.90	0.85	0.80
k=3	1.25	1.20	1.15	1.11	1.07	1.04	1.02	0.99	0.96	0.93	0.90	0.86	0.80
k=4	1.26	1.20	1.16	1.11	1.08	1.05	1.02	1.0	0.97	0.94	0.90	0.87	0.81
$\beta_{UT} = 0.2$													
k=1	1.55	1.41	1.32	1.21	1.13	1.07	1.02	0.97	0.92	0.86	0.79	0.73	0.64
k=2	1.58	1.45	1.34	1.23	1.16	1.09	1.04	0.99	0.94	0.88	0.81	0.75	0.66
k=3	1.61	1.48	1.37	1.26	1.18	1.12	1.06	1.01	0.96	0.90	0.82	0.76	0.67
k=4	1.64	1.51	1.40	1.28	1.20	1.14	1.08	1.03	0.97	0.91	0.84	0.78	0.68
$\beta_{UT} = 0.3$													
k=1	1.95	1.71	1.54	1.35	1.23	1.13	1.05	0.97	0.89	0.81	0.71	0.64	0.52
k=2	2.04	1.79	1.61	1.41	1.28	1.18	1.09	1.01	0.93	0.85	0.75	0.66	0.55
k=3	2.14	1.88	1.68	1.48	1.34	1.23	1.13	1.06	0.98	0.89	0.78	0.70	0.57
k=4	2.23	1.96	1.76	1.54	1.40	1.29	1.20	1.11	1.02	0.93	0.82	0.73	0.60
$\beta_{UT} = 0.4$													
k=1	2.49	2.10	1.81	1.52	1.33	1.20	1.08	0.98	0.88	0.77	0.65	0.56	0.43
k=2	2.70	2.27	1.96	1.65	1.45	1.30	1.17	1.06	0.95	0.84	0.70	0.61	0.47
k=3	2.92	2.46	2.12	1.78	1.57	1.40	1.27	1.15	1.03	0.90	0.76	0.66	0.50
k=4	3.16	2.66	2.30	1.93	1.70	1.52	1.38	1.25	1.11	0.98	0.83	0.71	0.55
$\beta_{UT} = 0.5$													
k=1	3.21	2.59	2.15	1.73	1.48	1.28	1.13	1.0	0.87	0.74	0.60	0.50	0.36
k=2	3.63	2.93	2.44	1.96	1.67	1.45	1.28	1.13	0.99	0.84	0.68	0.56	0.40
k=3	4.11	3.32	2.76	2.22	1.90	1.65	1.45	1.28	1.12	0.95	0.77	0.64	0.46
k=4	4.66	3.76	3.13	2.52	2.14	1.87	1.65	1.45	1.26	1.08	0.87	0.72	0.52
$\beta_{UT} = 0.6$													
k=1	4.17	3.22	2.58	1.99	1.65	1.39	1.20	1.03	0.87	0.72	0.56	0.44	0.30
k=2	5.00	3.86	3.09	2.39	1.97	1.67	1.43	1.23	1.04	0.86	0.66	0.53	0.36
k=3	5.98	4.62	3.70	2.86	2.35	2.00	1.72	1.48	1.25	1.03	0.80	0.64	0.43
k=4	7.15	5.52	4.42	3.42	2.82	2.39	2.05	1.76	1.49	1.23	0.95	0.76	0.52

The slope of the hazard curve is a function of the hazard level, location and response period. USGS maps provide values of 5%-damped, spectral response accelerations at periods of 0.2 seconds, termed $S_{s,}$ and 1 second, termed $S_{1,}$ for ground motions having 2% and 10% probabilities of exceedance in 50 years, for all locations in the United States. This information is also available on their web site and on a CD-ROM. Since most steel moment-frames have relatively long fundamental periods, the slope of the hazard curve may be determined for most such structures using the S_1 values published by the USGS for probabilities of exceedance of 2% and 10% in 50 years, and substitution of these values into the following equation:

$$k = \frac{\ln\left(\frac{H_{S1(10/50)}}{H_{S1(2/50)}}\right)}{\ln\left(\frac{S_{1(2/50)}}{S_{1(10/50)}}\right)} = \frac{1.65}{\ln\left(\frac{S_{1(2/50)}}{S_{1(10/50)}}\right)} \qquad \text{(A-6)}$$

where:

$S_{1(10/50)}$ = spectral amplitude for 10/50 hazard level
$S_{1(2/50)}$ = spectral amplitude for 2/50 hazard level
$H_{S1(10/50)}$ = probability of exceedance for 10% in 50 years = 1/475 = 0.0021
$H_{S1(2/50)}$ = probability of exceedance for 2% in 50 years = 1/2475 = 0.00040

The accompanying sidebar provides an example of how k may be determined using this procedure, for a representative site. As an alternative to using this detailed procedure, an approximate value of k may be obtained from Table A-2. When deterministic ground shaking demands (specific magnitude earthquake on a fault) are used as the basis for a performance objective, the value of k shall be taken as 4.0, regardless of the site seismicity.

Table A-2 Default Values of the Logarithmic Hazard Curve Slope *k* for Probabilisitc Ground Shaking Hazards

Region	*k*
Alaska, California and the Pacific Northwest	3
Intermountain Region, Basin & Range Tectonic Province	2
Other U.S. locations	1

Note: For deterministic ground shaking demands, use a value of $k = 4.0$

Example determination of the parameter, *k*, the logarithmic slope of the hazard curve using hazard data from the USGS.

Example site location: Los Angeles City Hall
Referencing USGS maps, web site, find $S_{1(10/50)}$ = 0.45g, $S_{1(2/50)}$ = 0.77g
Substituting into equation A-5, find:

$$k = \frac{1.65}{\ln\left(\frac{0.77g}{0.45g}\right)} = \frac{1.65}{0.537} = 3.07$$

A.4 Determination of Demand Factors

The demand variability factor g and analysis uncertainty factor g_a are used to adjust the calculated interstory drift, column axial load and column-splice tension demands to their mean values, considering the variability and uncertainty inherent in drift demand prediction.

Variability in drift demand prediction is primarily a result of the fact that due to relatively subtle differences in acceleration records, a structure will respond somewhat differently to different ground motion records, even if they are well characterized by the same response spectrum. Since it is not possible to predict the exact acceleration record that a structure may experience, it is necessary to account for the probable variation in demand produced by all possible different records. This is accomplished by developing a nonlinear mathematical model of the structure, and running nonlinear response history analyses of the structure for a suite of ground motion records, all of which are scaled to match the 5% damped linear spectral response acceleration, S_{aT1}, described in Section A.3.1. From these analyses, statistics are developed for the median value and standard deviation of the natural logarithm of the various demand parameters including maximum interstory drift, column axial load, and column splice tension. These standard deviations of the natural logarithms of these response parameters are denoted b_{D_R}.

Once the value of b_{D_R} has been determined, the demand variability factor, γ, is calculated from the equation:

$$g = e^{\frac{k}{2b} b_{D_R}^2} \tag{A-7}$$

where:

k is the logarithmic slope of the hazard curve, taken in accordance with Section A.3.1

b is a coefficient that represents the amount that demand increases as a function of hazard, and may normally be taken as having a value of 1.0

Uncertainty in the prediction of demands is due to an inability to define accurately the value of such parameters as the yield strength of the material, the viscous damping of the structure, the effect of nonstructural components, the effect of foundation flexibility on overall structural response, and similar modeling issues. Although it is not feasibly practical to do so, it is theoretically possible to measure each of these quantities for a building and to model their effects exactly. Since it is not practical to do this, instead likely values are used for each of these effects in the model, to account for the possible inaccuracies introduced by using these likely values, rather than real values. These inaccuracies are accounted for by developing a series of models to represent the structure, accounting for the likely distribution of these various parameters. Each of these models is used to run analyses with a single ground motion record, and statistics are developed for the effect of variation in these parameters on predicted demands. As with the variability due to ground motion, the standard deviation of the natural logarithms of the response parameters are calculated, and denoted by β_{D_U}. This parameter is used to calculate the analytical uncertainty factor, γ_a.

In addition to uncertainty in demand prediction, the analytical uncertainty factor γ_a also accounts for inherent bias, that is, systematic under- or over-prediction of demand, inherent in an analytical methodology. Bias is determined by using the analytical methodology, for example, elastic modal analysis, to predict demand for a suite of ground motions and then evaluating the ratio of the demand predicted by nonlinear time history analysis of the structure to that predicted by the methodology for the same ground motion. This may be represented mathematically as:

$$C_B = \frac{\textit{demand predicted by nonlinear time history analysis}}{\textit{demand predicted by analysis method}} \tag{A-8}$$

where C_B is the bias factor. The bias factor that is applicable to a specific structure is taken as the median value of C_B calculated from a suite of ground motions. The variation in the bias factors obtained from this suite of ground motions is used as one of the components in the calculation of β_{D_U}.

Once the median bias factor, C_B and logarithmic standard deviation in demand prediction β_{D_U} have been determined, the analysis uncertainty factor, γ_a is calculated from the equation:

$$\gamma_a = C_B e^{\frac{k}{2b}\beta_{D_U}^2} \tag{A-9}$$

The analysis uncertainty factors presented in Chapter 3 were calculated using this approach as applied to a suite of typical buildings. In addition to the uncertainties calculated using this procedure, it was assumed that even the most sophisticated methods of nonlinear time history analysis entail some uncertainty relative to the actual behavior of a real structure. Additional uncertainty was associated with other analysis methods to account for effects of structural irregularity, which were not adequately represented in the suite of model buildings used in the study. The value of the total logarithmic uncertainty β_{D_U} used as a basis for the analysis uncertainty factors presented in Chapter 4 are summarized in Table A-3. The bias factors C_B used in Chapter 4 are summarized in Table A-4. It is recommended that these default values for C_B and β_{D_U} be used for all buildings. If it is desired to calculate building-specific β_{D_U} values, it

is recommended that these values not be taken as less than those indicated in Table A-3 for nonlinear dynamic analysis, for the applicable building characteristics.

Table A-3 Default Logarithmic Uncertainty b_{DU} for Various Analysis Methods

	Analysis Procedure							
	Linear Static		Linear Dynamic		Nonlinear Static		Nonlinear Dynamic	
Performance Level	IO	CP	IO	CP	IO	CP	IO	CP
Type 1 Connections								
Low Rise (<4 stories)	0.17	0.22	0.15	0.16	0.14	0.17	0.10	0.15
Mid Rise (4 – 12 stories)	0.18	0.29	0.15	0.23	0.15	0.23	0.13	0.20
High Rise (> 12 stories)	0.31	0.25	0.19	0.29	0.17	0.27	0.17	0.25
Type 2 Connections								
Low Rise (<4 stories)	0.19	0.23	0.16	0.25	0.18	0.18	0.10	0.15
Mid Rise (4 – 12 stories)	0.20	0.30	0.17	0.33	0.14	0.21	0.13	0.20
High Rise (> 12 stories)	0.21	0.36	0.21	0.31	0.18	0.33	0.17	0.25

Table A-4 Default Bias Factors C_B

	Analysis Procedure							
	Linear Static		Linear Dynamic		Nonlinear Static		Nonlinear Dynamic	
Performance Level	IO	CP	IO	CP	IO	CP	IO	CP
Type 1 Connections								
Low Rise (<4 stories)	0.90	0.65	1.00	0.80	1.10	0.85	1.00	1.00
Mid Rise (4 – 12 stories)	1.10	0.85	1.10	1.15	1.40	0.95	1.00	1.00
High Rise (> 12 stories)	1.05	1.0	1.15	1.0	1.30	0.85	1.00	1.00
Type 2 Connections								
Low Rise (<4 stories)	0.75	0.90	1.00	1.20	0.90	1.25	1.00	1.00
Mid Rise (4 – 12 stories)	0.80	1.00	1.05	1.30	1.08	1.35	1.00	1.00
High Rise (> 12 stories)	0.75	0.70	1.30	1.20	1.30	1.30	1.00	1.00

Commentary: Although it may be possible, for certain structures, to increase the confidence associated with a prediction of probable earthquake demands on the structure, through calculation of structure-specific analysis uncertainty factors, in general this is a very laborious process. It is recommended that the default values of b_{DU} and C_B, contained in Tables A-3 and A-4, be used for most

structures. However, the procedures of this section can be used to adjust the analysis uncertainty and demand variability factors for the site seismicity k.

A.5 Determination of Beam-Column Connection Assembly Capacities

The probable behavior of beam-column connection assemblies at various demand levels can best be determined by full-scale laboratory testing. Such testing can provide indications of the probable physical behavior of such assemblies in buildings. Depending on the characteristics of the assembly being tested, meaningful behaviors may include the following: onset of local buckling of flanges; initiation of fractures in welds, base metal or bolts; a drop in the moment developed by the connection beyond predetermined levels; or complete failure, at which point the connection is no longer able to maintain attachment between the beam and column under the influence of gravity loads. If sufficient laboratory data are available, it should be possible to obtain statistics, including a median value and standard deviation, on the demand levels at which these various behaviors occur.

In the past, most laboratories used plastic rotation as the demand parameter by which beam-column connection assembly behavior was judged. However, since plastic deformation may occur at a number of locations within a connection assembly, including within the beam itself, within the connection elements, and within the column panel zone or column, many laboratories have measured and reported plastic rotation angles from testing in an inconsistent manner. Therefore, in these *Recommended Criteria*, total interstory drift angle, as indicated in Section 4.6, is the preferred demand parameter for reporting laboratory data. This parameter is less subject to erroneous interpretation by testing laboratories and also has the advantage that it is a quantity directly predicted by linear structural analyses.

Median drift angle capacities, C, and resistance factors, f, for various prequalified connection types are presented in Chapter 4. These values were determined from cyclic tests of full-size connection assemblies using the testing protocols indicated in Section 3.9. The cyclic tests are used to determine the load-deformation hysteresis behavior of the system and the connection drift angle at which the following behaviors occur:

1. onset of local flange buckling of beams,
2. degradation of moment-resisting capacity of the assembly to a value below the nominal moment-resisting capacity,
3. initiation of fracture of bolts, welds, or base metal that results in significant strength degradation of the assembly, and
4. complete failure of the connection, characterized by an inability of the connection to maintain its integrity under gravity loading.

Based on this data, drift angle statistics, including a median value and logarithmic standard deviation are obtained for the Immediate Occupancy and Collapse Prevention damage states, as indicated in Table A-5. The quantity q_U, the ultimate capacity of the connection, is used to evaluate the acceptability of connection behavior for the Collapse Prevention performance level as limited by local behavior.

Table A-5 Behavior States for Performance Evaluation of Connection Assemblies

Symbol	Performance Level	Description
q_{IO}	Immediate Occupancy	The lowest drift angle at which any of behaviors 1, 2, or 3, occur (see Section A.5, above)
q_U	Ultimate	The drift angle at which behavior 4 occurs
q_{SD}	Strength Degradation	The lowest drift angle at which any of behaviors 2, 3, or 4 occur

A.5.1 Connection Test Protocols

Two connection test protocols have been developed under this project. The standard protocol is intended to represent the energy input and cyclic deformation characteristics experienced by connection assemblies in steel moment frames which are subjected to strong ground shaking from large magnitude earthquakes, but which are not located within a few kilometers of the fault rupture. This protocol presented in Section 6.9 is similar to that contained in *ATC-24* (ATC, 1992) and consists of ramped cyclic loading, starting with initial cycles of low energy input within the elastic range of behavior of the assembly, and progressing to increasing deformation of the beam tip until assembly failure occurs. However, unlike *ATC-24*, the protocol incorporates fewer cycles of large-displacement testing to balance more closely the energy input to the assembly, with that likely experienced by framing in a real building. The second protocol is intended to represent the demands experienced by connection assemblies in typical steel moment-frame buildings responding to near-fault ground motion, dominated by large velocity pulses. This protocol (Krawinkler et al., 2000) consists of an initial single large displacement, representing the initial response of a structure to a velocity pulse, followed by repeated cycles of lesser displacement.

Performance characteristics of connection assemblies, for use in performance evaluation of buildings, should be selected based on the characteristics of earthquakes dominating the hazard for the building site, at the specific hazard level. Most buildings are not located on sites that are likely to be subjected to ground shaking with near-field pulse characteristics. Connection performance data for such buildings should be based on the standard protocols. Buildings on sites that are close to a major active fault are most likely to experience ground shaking with these strong pulse-like characteristics and connection performance for such buildings should be based on the near-fault protocol. However, qualification of connections for classification as either Special Moment Frame or Ordinary Moment Frame connections should be based on the standard protocol.

A.5.2 Determination of Beam-Column Assembly Capacities and Resistance Factors

Median drift angle capacities for the quantities q_{IO} and q_U should be taken directly from available laboratory data. The median value should be taken as that value from all of the available tests that is not exceeded by 50% of the tests. The value of the quantity f, for each of the Immediate Occupancy and ultimate (Collapse Prevention) states should be determined by the following procedure.

1. Obtain the logarithmic standard deviation of the q_{IO} or q_U values available from the laboratory data. That is, take the standard deviation of the natural logarithms of the q_{IO} or q_U values respectively, obtained from each laboratory test. Logarithmic standard deviation may be determined from the formula:

$$b = \sqrt{\frac{\sum_{i=1}^{n} \left(\ln x_i - \overline{\ln x_i}\right)^2}{n-1}} \qquad \text{(A-10)}$$

where:
b = the standard deviation of the natural logarithms of the test data
x_i = individual test data value
n = the number of tests from which data is available
$\overline{\ln x_i}$ = the mean of the logarithms of the x_i values.

2. Calculate the connection resistance factor f_R due to randomness, the observed variation in connection behavior, from laboratory testing, using the equation:

$$f_R = \mathrm{e}^{-\frac{k}{2b}b^2} \qquad \text{(A-11)}$$

where:
k = the slope of the hazard curve, determined in accordance with Section A.3.2
b = a coefficient that relates the change in hazard to the change in demand, and which may be taken as having a value of 1.0
b = the logarithmic standard deviation calculated in accordance with Equation A-10.

3. Determine the connection resistance factor accounting for random and uncertain behaviors from the equation:

$$f = f_R f_U = f_R \mathrm{e}^{-\frac{k}{2b}(0.2)^2} \qquad \text{(A-12)}$$

where:
f_R = the resistance factor accounting for random behavior
f_U = the resistance factor accounting for uncertainty in the relationship between laboratory findings and behavior in real buildings, and assumed in these *Recommended Criteria* to have a logarithmic standard deviation b_u of 0.2

A.6 Global Stability Capacity

For the Collapse Prevention performance level, in addition to consideration of local behavior, that is, the damage sustained by individual beams and beam-column connection assemblies, it is also important to consider the global stability of the frame. The procedures indicated in this section are recommended for determining an interstory drift capacity C and resistance factor f associated with global stability of the structure.

The global stability limit is determined using the Incremental Dynamic Analysis (IDA) technique. This requires the following steps:

1. Choose a suite of ten to twenty accelerograms representative of the site and hazard level for which the Collapse Prevention level is desired to be achieved.
2. Select one of these accelerograms and perform an elastic time-history analysis of the building. Determine a scaling factor for this accelerogram such that the elastic time history analysis would result in response that would produce incipient yielding in the structure. Determine the 5%-damped, spectral response acceleration S_{aT1} for this scaled accelerogram at the fundamental period of the structure. On a graph with an abscissa consisting of peak interstory drift and an ordinate axis of S_{aT1}, plot the point consisting of the maximum calculated interstory drift from the scaled analysis and the scaled value of S_{aT1}. Draw a straight line from the origin of the axes to this point. The slope of this line is referred to as the elastic slope, S_e
3. Increase the scaling of the accelerogram, such that it will produce mild nonlinear behavior of the building. Perform a nonlinear time-history analysis of the building for this scaled accelerogram. Determine the S_{aT1} for this scaled accelerogram and the maximum predicted interstory drift from the analysis. Plot this point on the graph. Call this point D_1.
4. Increase the scaling amplitude of the accelerogram slightly and repeat Step 3. Plot this point as D_2. Draw a straight line between points D_1 and D_2.
5. Repeat Step 4 until the straight line slope between consecutive points D_i and D_{i+1}, is less than 0.2 S_e. When this condition is reached, D_{i+1} is the global drift capacity for this accelerogram. If $D_{i+1} \geq 0.10$ then the drift capacity is taken as 0.10. Figure A-1 presents a typical series of plots obtained from such analyses.
6. Repeat Steps 2 through 5 for each of the accelerograms in the suite selected as representative of the site and hazard and determine an interstory drift capacity for the structure for each accelerogram.
7. Determine a median interstory drift capacity C for global collapse as the median value of the calculated set of interstory drift capacities, determined for each of the accelerograms. The median value is that value exceeded by 50% of the accelerograms.
8. Determine a logarithmic standard deviation b for random differences in ground motion accelerograms, using Equation A-10 of Section A.5.2. In this equation, x_i is the interstory drift capacity predicted for the i^{th} accelerogram, and n is the number of accelerograms contained in the analyzed suite.
9. Calculate the global resistance factor f_R due to randomness in the predicted global collapse capacity for various ground motions from the equation:

$$f_R = e^{-\frac{k}{2b}b^2} \tag{A-13}$$

where k and b are the parameters described in Section A.5.2 and b is the logarithmic standard deviation calculated in the previous step.

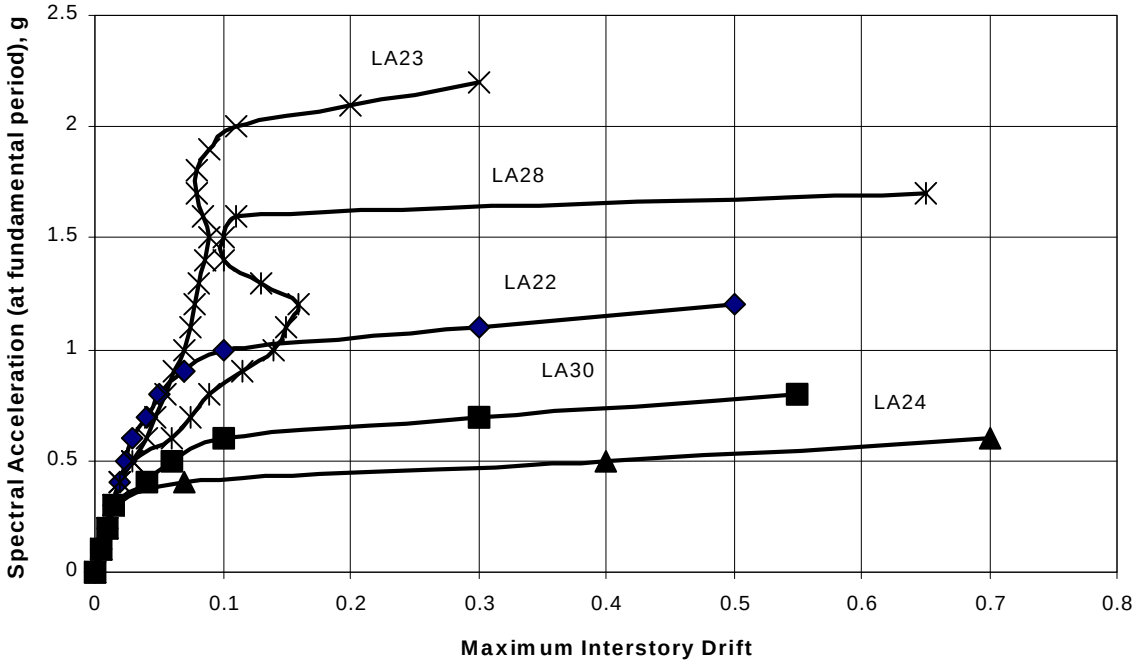


Figure A-1 Representative Incremental Dynamic Analysis Plots

10. Determine a resistance factor for global collapse from the equation:

$$f = f_U f_R = e^{-\frac{k}{2b} b_U^2} f_R \qquad \text{(A-14)}$$

where:

f_R is the global resistance factor due to randomness determined in Step 9.

b_U is the logarithmic standard deviation related to uncertainty in analytical prediction of global collapse prevention taken as having a value of 0.15 for low-rise structures, 3 stories or less in height; a value of 0.2 for mid-rise structures, 4 stories to 12 stories in height; and taken as having a value of 0.25 for high-rise structures, greater than 12 stories in height.

It is important that the analytical model used for determining the global drift demand be as accurate as possible. The model should include the elements of the moment-resisting frame as well as framing that is not intended to participate in lateral load resistance. A nominal viscous damping of 3% of critical is recommended for most buildings. The element models for beam-column assemblies should realistically account for the effects of panel zone flexibility and yielding, element strain hardening, and stiffness and strength degradation, so that the hysteretic behavior of the element models closely matches that obtained from laboratory testing of comparable assemblies.

Commentary: As noted above, accurate representation of the hysteretic behavior of the beam-column assemblies is important. Earthquake-induced global collapse initiates when displacements produced by the response to ground shaking are

large enough to allow P-D instabilities to develop. Prediction of the onset of P-D instability due to ground shaking is complex. It is possible that ground shaking will displace a structure to a point where static P-D instability would initiate, but will bring the structure back again before collapse can occur, due to a reversal in ground shaking direction.

The basic effect of P-D instability is that a negative stiffness is induced in the structure. That is, P-D effects produce a condition in which increased displacement can occur at a reduced lateral force. A similar and equally dangerous effect can be produced by local hysteretic strength degradation of beam-column assemblies (FEMA-355C). Hysteretic strength degradation typically occurs after the onset of significant local buckling in the beam-column assemblies. It is important when performing Incremental Dynamic Analyses that these local strength degradation effects, which show up as a concave curvature in the hysteretic loops in laboratory data, are replicated by the analytical model. Nonlinear analysis software that is currently commercially available is not, in general, able to model this behavior. It is possible to account for these effects approximately by increasing the amount of dead load on the structure, so as to produce artificially the appropriate negative stiffness.

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Acronyms.

A, acceleration response
ACAG, air carbon arc gouging
ACIL, American Council of Independent Laboratories
AISC, American Institute for Steel Construction
ANSI, American National Standards Institute
API, American Petroleum Institute
ASNT, American Society for Nondestructive Testing
ASTM, American Society for Testing and Materials
ATC, Applied Technology Council
A2LA, American Association for Laboratory Accreditation
AWS, American Welding Society
BB, Bolted Bracket (connection)
BFP, Bolted Flange Plates (connection)
BOCA, Building Officials and Code Administrators
BSEP, Bolted Stiffened End Plate (connection)
BUEP, Bolted Unstiffened End Plate (connection)
CAC-A, air carbon arc cutting
CAWI, Certified Associate Welding Inspector
CJP, complete joint penetration (weld)
CP, Collapse Prevention (performance level)
CUREe, California Universities for Research in Earthquake Engineering
CVN, Charpy V-notch
CWI, Certified Welding Inspector
D, displacement response
DST, Double Split Tee (connection)
DTI, Direct Tension Indicator
EGW, electrogas welding
ELF, equivalent lateral force
ESW, electroslag welding
FCAW-S, flux-cored arc welding – self-shielded
FCAW-G, flux-cored arc welding – gas-shielded
FEMA, Federal Emergency Management Agency
FF, Free Flange (connection)
FR, fully restrained (connection)
GMAW, gas metal arc welding
GTAW, gas tungsten arc welding
HAZ, heat-affected zone
IBC, *International Building Code*
ICBO, International Conference of Building Officials
ICC, International Code Council
IMF, Intermediate Moment Frame
IO, Immediate Occupancy (performance level)

ISO, International Standardization Organization
IWURF, Improved Welded Unreinforced Flange (connection)
L, longitudinal
LDP, Linear Dynamic Procedure
LRFD, load and resistance-factor design
LS, Life Safety (performance level)
LSP, Linear Static Procedure
MCE, Maximum Considered Earthquake
MMI, Modified Mercalli Intensity
MRS, modal response spectrum
MRSF, steel moment frame
MT, magnetic particle testing
NBC, *National Building Code*
NDE, nondestructive examination
NDP, Nonlinear Dynamic Procedure
NDT, nondestructive testing
NEHRP, National Earthquake Hazard Reduction Program
NES, National Evaluation Services
NSP, Nonlinear Static Procedure
NVLAP, National Volunteer Laboratory Accreditation Program
OMF, Ordinary Moment Frame
PGA, peak ground acceleration
PJP, partial joint penetration (weld)
PIDR, pseudo interstory drift ratio
PQR, Performance Qualification Record
PR, partially restrained (connection)
PT, liquid dye penetrant testing
PWHT, postweld heat treatment
PZ, panel zone
QA, quality assurance
QC, quality control
QCP, Quality Control Plan, Quality Certification Program
RBS, Reduced Beam Section (connection)
RCSC, Research Council for Structural Connections
RT, radiographic testing
SAC, the SAC Joint Venture; a partnership of the Structural Engineers Association of California, the Applied Technology Council, and California Universities for Research in Earthquake Engineering
SAW, submerged arc welding
SBC, *Standard Building Code*
SBCCI, Southern Building Code Congress International
SCWI, Senior Certified Welding Inspector
SEAOC, Structural Engineers Association of California
SFRS, seismic-force-resisting system
SMAW, shielded metal arc welding
SMF, Special Moment Frame
SP, Side Plate (connection)
SUG, Seismic Use Group
SW, Slotted Web (connection)
T, transverse
TIGW, tungsten inert gas welding
UBC, *Uniform Building Code*
UT, ultrasonic testing
VI, visual inspection
WBH, Welded Bottom Haunch (connection)
WCPF, Welded Cover Plate Flange (connection)
WFP, Welded Flange Plate (connection)
WPQR, Welding Performance Qualification Record
WPS, Welding Procedure Specification
WSMF, welded steel moment frame
WT, Welded Top Haunch (connection)
WTBH, Welded Top and Bottom Haunch (connection)
WUF-B, Welded Unreinforced Flanges – Bolted Web (connection)
WUF-W, Welded Unreinforced Flanges – Welded Web (connection)

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